10/5/2009

Fairfield Inn & Suites, Marriot
Pittsburgh, PA

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EXECUTIVE SUMMARY

In this technical report, the existing structural conditions of the Fairfield Inn and Suites are discussed to gain a better understanding of the structural design of the building. An analysis of the structural system, gravity loads, and lateral loads can be found through detailed descriptions and diagrams, as well as, the materials and codes used in the actual design of the building. Building layout and detailed calculations for each analysis performed can be found in an Appendix at the end of the report.

The Fairfield Inn and Suites is a 10-story hotel located in the city of Pittsburgh. Tourists and visitors to Pittsburgh will occupy the 135 guest rooms the hotel has to offer, along with enjoying the pool and fitness center.

The hotel is approximately 78,803 square feet and reaches a height of 112'-8" above grade with a basement below grade. The typical floor to floor height is 9'-4" and the lobby extends to a height of 18'-0". The typical floor system is precast plank floors while the ground floor is a reinforced concrete slab on grade. The foundation consists of auger cast piles with grade beams transferring wall loads down to the foundation. The transfer beams on the second level, carry the building loads into columns that extend down into the pile caps. The lateral resisting system is composed of reinforced concrete masonry shear walls.

In order to get a better understanding of the lateral system of the structure, wind and seismic loads were analyzed using ASCE 7-05. To examine wind loads on the building, the Analytical Procedure was used to find the wind pressures in both the North/South direction and East/West direction. The wind in the North/South direction was found to control over the wind in the East/West direction. Due to the North/South façade of the building being longer, it is required to resist greater wind pressures and verifies the wind calculations make sense. Seismic loads were examined using the Equivalent Lateral Force Procedure. The seismic loads resulted in giving the building a larger base shear than the wind loads, showing that the seismic loads will control when determining lateral force on the structure.

Spot checks of gravity loads were performed on various structural members of the building. Spot checks were conducted on an interior column, transfer beam, a girder to validate the member sizes chosen. All members were found to be adequately designed, but overdesigned. This is due to the fact that only gravity loads were analyzed. Since the column, beam, and girder are also a part of the lateral system, the loads due to lateral forces were also considered in the actual design of the building. Taking into account the lateral forces will be further analyzed in later reports, which will account for the overdesigned members.

INTRODUCTION: Fairfield Inn & Suites

Fairfield Inn and Suites is a 10-story hotel. The hotel is located in the heart of Pittsburgh within walking distance to downtown Pittsburgh, Heinz Field (football stadium), the new Rivers casino, plus many other Pittsburgh attractions. The hotel's closest attraction, directly across the street, is the Pittsburgh Pirates baseball stadium, PNC Park. Being in such a prime location, this hotel with accommodate thousands of guests visiting the area throughout the year making it an essential addition to the community.

The hotel occupies 135 guest rooms in addition to an indoor pool and fitness center for its guests. There will be a variety of typical king/queen size rooms to king/queen suites to satisfy the needs of all guests. Guests to the hotel will enter into an 18' lobby off of Federal St. where the main entrance exists. The lobby consists of a large reception desk for check-in/out, a breakfast area, and a large seating area featuring a cherry finished wood fireplace. The hotel holds a basement below grade that consists of the electrical, mechanical, and maintenance rooms, along with the laundry room and break room for employees.

The façade of the building is similar for all views. Cast-stone decorates the exterior levels one thru four. Brick veneer than extends to the roof of the building. As one approaches the 18' lobby entrance a glass curtain wall system surrounds the entrance doors and extends above the entrance two stories adding verticality to the building. The entrance is then emphasized by a large steel supported, tempered glass awning shading the lobby. On street level, the lobby is lined by additional high glass windows also shaded with smaller glass awnings. From the highway that passes the buildings north façade, one will notice the hotel by its large illuminated sign placed inside a 56'x18' bond-face brick detailed rectangle accenting this view.

The structural system for the hotel is primarily precast plank floors with load bearing masonry shear walls that resist the lateral forces against building. Steel transfer beams at the second floor transfer the loads of the interior shear walls to columns supporting the 18' lobby. The ground floor is a concrete slab on grade that transfers the gravity loads of the building to a foundation system that is composed of auger cast piles and steel grade beams.

The purpose of Technical Report 1 is to take a closer look at the concepts and existing conditions of the structural system of the Fairfield Inn and Suites. An overview is given in regards to the framing, structural slabs, lateral force resisting systems, and foundations to better explain how each of the components work together and form a structural system that resists gravity and lateral loads.

STRUCTURAL SYSTEM

Foundation

A geotechnical soils report was conducted for the Fairfield Inn and Suites site on November 27, 2007 by Construction Engineering Consultants. In the study, it was found that the typical soil found on site is brown silt, clay, and sand. The reported water level was approximately 25'-0" on site. The depth of the basement is 12'-8" below grade, therefore there should not be a concern regarding the uplift pressures on the foundation due to the water level. Due to the moderate depth to bedrock and precaution taken in regards to water level, the deep foundation system consists of auger cast friction piles and grade beams. With the foundation not extending below 33 ft., the net allowable bearing pressure on site is 200 psf.

The ground floor rest on a 6" concrete slab which is 5 ksi normal weight concrete (NWC). The slab increases in thickness from 6" to 12" within the core shear walls where the elevator pit and area well are located. The slab reinforcement consists of W/ 6x6-W1.2xW1.2 welded wire fabric and #5 bars located 12" o.c. top and bottom and each way. The slab depth is approximately 12'-8" below grade, while the elevator pit extends to 17'-5" below grade.

The piles extend 12'-8" deep below grade and are spaced approximately between 26' to 31' apart (refer to Appendix A). The typical size of the pile caps is a 7'-6" square approximately 4' deep with four 16" diameter piles per cap. The core shear walls incasing the stairs and elevator have additional rectangular pile caps and piles for more support. Pile caps are reinforced with #8 bars at 6" o.c. The typical column piers extending from the pile caps are composite 24"x24" columns with horizontal ties and vertical bar reinforcement. (see Figure 1.1)

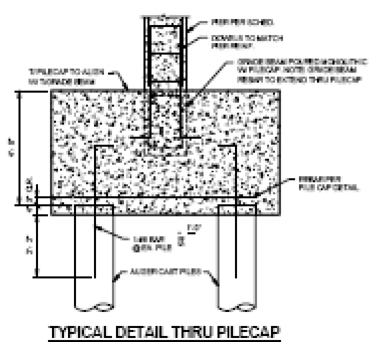


Figure 1.1

Grade beams run between pile caps transferring the loads from the façade and interior shear walls to the foundation (refer to Figure 1.2). Depth of beams ranges between 36" and 48" depending on location. Reinforcement and size varies per grade beam.

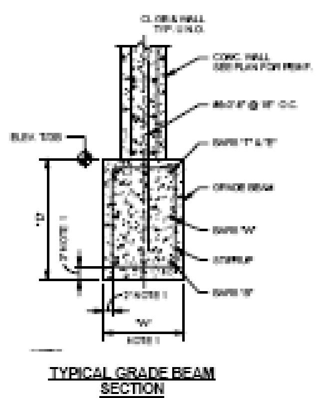


Figure 1.2

Floor System

Fairfield Inn and Suites typical floor system is a precast concrete plank floor with a thickness of 8” untopped. The hollow core concrete plank floor allows for the building to be supported without the use of columns on floors two thru ten and longer plank spans. Concrete compressive strength for floors is f’c=5000 psi. The typical span of the precast plank floors are 31’-0” and 26’-0”. The floor systems support concrete masonry bearing walls.

The floor system for the first floor is a combination between 4” slab on grade and the 8” precast concrete plank floor. There is no basement below the first floor running along the south wall and the entrance on the west wall of the building (see Figure 2.1). Due to a pool being located in this area, the hollow core of the typical plank floor would not be sufficient in supporting the weight of the pool and lobby live loads. Therefore, the floor system is a 4” slab on grade with W/6x6-W1.4xW1.4 weld wire fabric reinforcement.

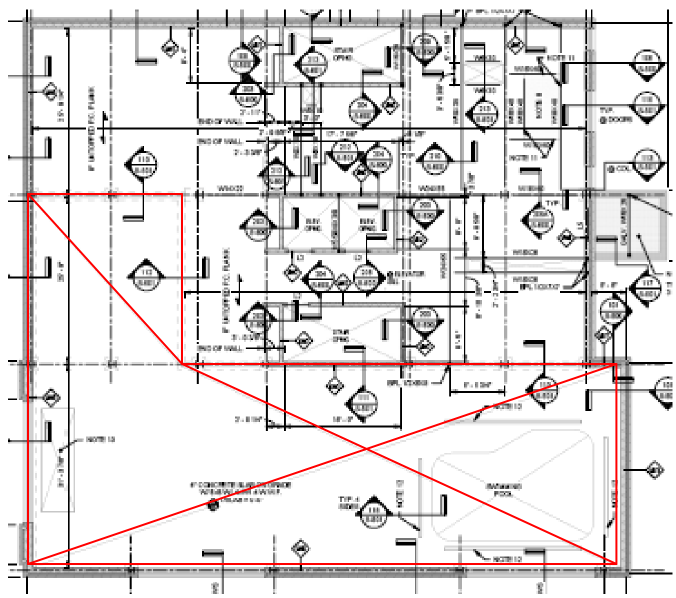


Figure 3.1: Partial First Floor Slab

Since the floor system is a precast plank floor, there are a limited number of steel beams girders throughout the structure. These transfer beams range in size from W 33x118 to W 40X149.With no columns to support floors two thru ten, the majority of the beams present are transfer beams on the second floor that transfer loads from the floors above to the columns extending from the pile caps and thus transferring all loads to the foundation system. The transfer beams run along the back of the elevator shafts from the west wall to the east wall, and along the back of south wall of stair B extending from the west wall to the east wall (see Figure 2.2). Transfer beams range in size from W 33x118 to W 40x149. Girders run along the first floor supporting mechanical equipment loads and tying into the beams and shear walls supporting the first floor. Girders and beams throughout the building are non-composite systems.

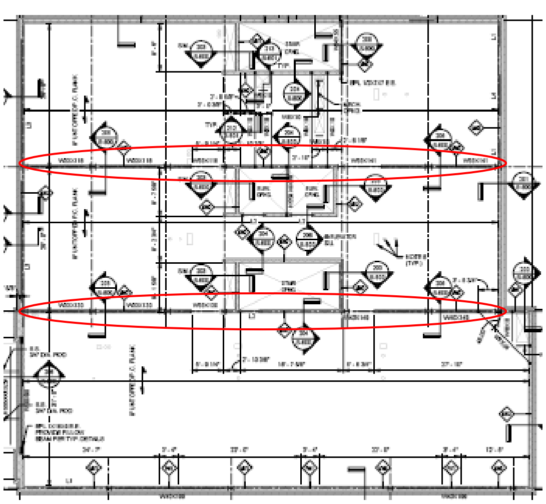


Figure 2.2: Second Floor Transfer Beams

The roof system and smaller high roof system are the same use the same 8” untopped precast concrete plank floor. W8x28 beams run along the shear walls inclosing the elevator and stair shaft while W8x18's extend outward from the corners of the shear walls inclosing the shaft. Hoist beams support the top of the elevator shaft in high roof system. There are a total of six drains located on the roof for the drainage system. (refer to Appendix A)

Columns

The only columns used in the Fairfield Inn and Suites are the ones extending from the pile caps to the second floor supporting the 18' first floor. The columns range in size from W10x100's to W 12x120's depending on location. All columns connect into the pile caps where the weight each column supports transfers the load down to the foundation (refer to Figure 3.1). The base plates are ½" thick and typically 14"x14". Each plate utilizes a standard 4 bolt connection using 1" A325 bolts.

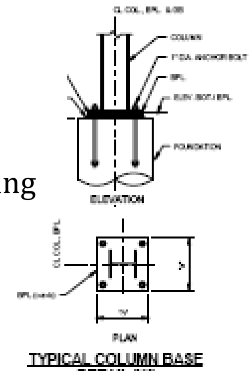


Figure 3.1

Lateral System

The lateral system for the Fairfield Inn and Suites is a combination of ordinary reinforced concrete masonry shear walls. The exterior shear walls are 10" concrete masonry and the core shear walls are 8" concrete masonry. The core shear walls surround the staircases and elevator shaft. On floors two thru ten, two additional shear walls extending from the west wall to the east wall run along the south wall of staircase B and the north wall of the elevator shafts (see Figure 4.1). Shear walls supporting the ground floor to the fourth floor support a compressive strength of f'c=8000 psi. All other shear walls support a compressive strength of f'c=5000 psi. The typical reinforcement in both the 10" and 8" shear walls is #5 bars at 16" o.c., 24" o.c., or 32" o.c. with bars centered in wall and solid grout wall.

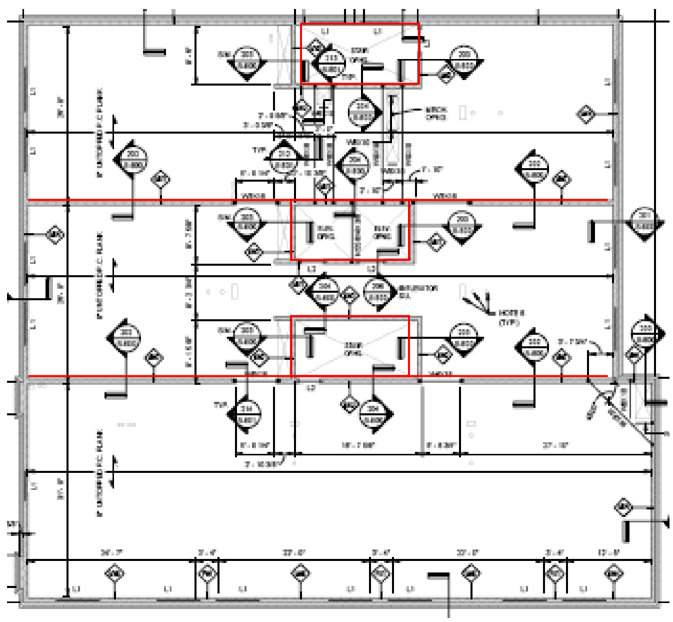


Figure 4.1: Lateral Shear Wall System

The wind and seismic loads, as well as gravity loads, reach the foundation by first traveling through the rigid building diaphragm (floor system) to the shear walls. From there the loads carry through the transfer beams and girders which connect to the columns at second floor. All loads travel in the columns to the basement level and into the auger cast piles and grade beam foundation. This load path is governed by the concept of relative stiffness.

CODES AND REQUIREMENTS

Various references were used by the engineer of record in order to carry out the structural design of the Fairfield Inn and Suites:

- The 2006 International Building Codes as amended by the city of Pittsburgh
- The Building Code Requirements for Structural Concrete (ACI 318-05), American Concrete Institute
- Specifications for Structural Concrete (ACI 301-05), American Concrete Institute
- The Building Code Requirements for Masonry Structures (ACI 530), American Concrete Institute
- Specifications for Masonry Structures (ACI 530.1), American Concrete Institute
- PCI Design Handbook – Precast/Prestressed Concrete Institute
- Specifications for Structural Steel Buildings – Allowable Stress Design and Plastic Design (AISC), American Institute of Steel Construction
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05), American Society of Civil Engineers

MATERIALS

Multiple materials are used in the construction of the Fairfield Inn and Suites. The details of these materials are listed as follows:

Concrete

Foundation	f'c = 3000 psi
Slab on Grade	f'c = 4000 psi
Exterior Slab on Grade	f'c = 4500 psi
Walls	f'c = 5000 psi
Grade Beams	f'c = 4000 psi
Columns	f'c = 8000 psi

Reinforcement Steel

Deformed Bars	ASTM A615, Grade 60
Welded Wire Fabric	ASTM A185

Structural Steel

Wide flange	ASTM A992
Channels	ASTM A 572, Grade 50
Angles and Plates	ASTM A36
Steel Tubes/Steel Pipes (HSS)	ASTM A500, Grade B
Connection Bolts	ASTM A325
Anchor Bolts	ASTM A36

GRAVITY LOADS

The gravity load conditions determined by ASCE 7-05 are provided for reference:

Dead Loads:

Concrete	150 pcf
Steel	490 pcf
Partitions	15 psf
MEP	10 psf
Finishes and Miscellaneous	5 psf
Roof	20 psf

Live Loads:

Description	Design Load Used By Engineer	ASCE 7-05
Public Areas	100 psf	100 psf
Lobbies	100 psf	100 psf
First Floor Corridors	100 psf	100 psf
Corridors above First Floor	80 psf	80 psf
Private Hotel Rooms	40 psf	40 psf
Stairs	100 psf	100 psf
Roof	75 psf	20 psf
Mechanical	150 psf	150 psf

LATERAL LOADS

Wind Analysis

Wind loads were calculated in accordance with ASCE 7-05, Chapter 6. To examine the wind loads in the North/South direction and the West/East direction, the Analytical Procedure – Method two described in Section 6.5, was used to find design pressures. The variables used in this analysis are located in Table 1a. Please refer to Appendix C for equations and base calculations used for the execution of this procedure. Figure 5.1 shows the wind direction made to the typical floor plan. A more detailed and accurate analysis of lateral loads will be studied in a future technical report.

Figure 5.1: Wind Direction

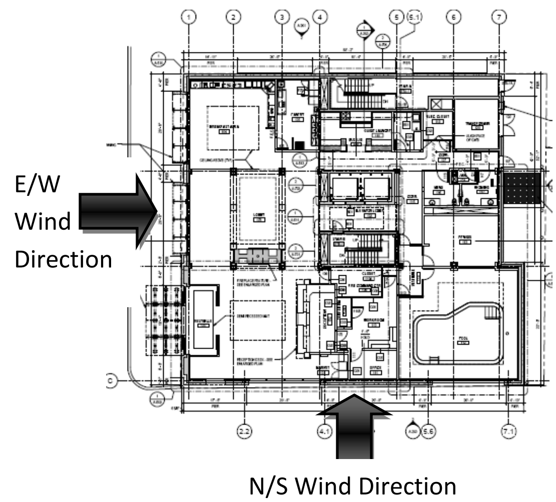


Table 1a

Wind Variables			ASCE References
Basic Wind Speed	V	90	Fig. 6-1
Directionality Factor	K_d	0.85	Table 6-4
Importance Factor	I	1.15	Table 6-1
Exposure Category		C	§ 6.5.6.3
Topographic Factor	K_{zt}	1.00	§ 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 Pressure at Mean Roof Height	q_h	20.47	Eq. 6-15
Equivalent Height of Structure	>	64.6'	Table 6-2
Intensity of Turbulence	$I_{\dot{z}}$	0.268	Eq. 6-5
Integral Length Scale of Turbulence	$L\dot{z}$	208.81	Eq. 6-7
Background Response Factor (East/West)	Q	0.792	Eq. 6-6
Background Response Factor (North/South)	Q	0.788	Eq. 6-6
Gust Effect Factor (East/West)	G	0.808	Eq. 6-4
Gust Effect Factor (North/South)	G	0.806	Eq. 6-4
External Pressure Coefficient (Windward)	C_p	0.8	Fig. 6-6
External Pressure Coefficient (E/W Leeward)	C_p	-0.03	Fig. 6-6
External Pressure Coefficient (N/S Leeward)	C_p	-0.05	Fig. 6-6

Table 1b was developed to determine the wind pressures in the North/South direction. This direction is adjacent to an existing building and a major highway, which neither structure is significant enough to block the building from receiving full wind loads. These wind loads are currently the most prevalent at this site. Wind loads can clearly be seen in the diagram of windward and leeward pressures at each level (Figure 5.2). A basic loading diagram also provided for reference as seen in Figure 5.5.

Table 1b

Wind Loads (North/South Direction)

B = 91'-0" L = 83'-0"

Level	Height above ground - z (ft.)	Story Height (ft.)	K_z	q_z	Wind Pressure (psf)		Total Pressure (psf)	Force of Windward Pressure Only (k)	Force of Total Pressure (k)	Windward Shear Story (k)	Total Story Shear (k)	Windward Moment (ft-k)	Total Moment (ft-k)
					Windward	Leeward							
PH Roof	112.66	10.00	1.02	20.67	17.02	-11.93	28.95	3.73	6.35	3.73	6.35	401.92	683.81
Roof	102.66	10.00	1.00	20.27	16.75	-11.93	28.69	15.25	26.11	18.98	32.46	1488.97	2549.55
10	92.66	9.33	0.97	19.66	16.36	-11.93	28.30	13.89	24.02	32.87	56.48	1222.43	2114.02
9	83.33	9.33	0.94	19.05	15.97	-11.93	27.90	13.56	23.69	46.43	80.17	1066.63	1863.69
8	74.00	9.34	0.90	18.24	15.45	-11.93	27.38	13.13	23.27	59.56	103.45	910.26	1613.48
7	64.66	9.33	0.87	17.63	15.06	-11.93	26.99	12.78	22.91	72.34	126.36	766.88	1374.77
6	55.33	9.33	0.83	16.82	14.53	-11.93	26.47	12.34	22.47	84.68	148.83	625.13	1138.49
5	46.00	9.34	0.79	16.01	14.01	-11.93	25.94	11.91	22.05	96.59	170.88	492.13	911.35
4	36.66	9.33	0.74	15.00	13.36	-11.93	25.29	11.34	21.47	107.93	192.35	362.82	687.00
3	27.33	9.33	0.68	13.78	12.57	-11.93	24.51	10.67	20.81	118.60	213.16	241.93	471.58
2	18.00	18.00	0.60	12.16	11.53	-11.93	23.46	18.88	38.43	137.48	251.59	169.92	345.85
1	0	0	0	0	0	0	0	0	0	137.48	251.59	0	0
									Σ Windward Story Shear =	137.48	kips		
									Σ Total Story Shear =	251.59	kips		
									Σ Windward Moment =	7749.01	ft-k		
									Σ Total Moment =	13753.59	ft-k		

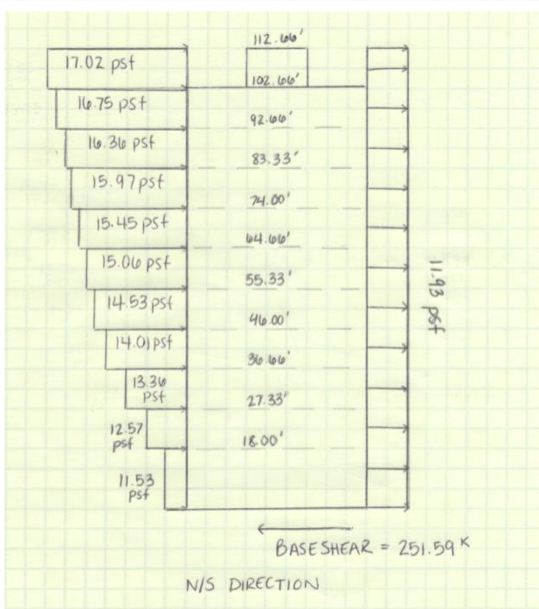


Figure 5.2: Wind Pressures in the North/South Direction

Table 1c was developed to determine the wind pressures in the East/West direction. There are currently adjacent buildings blocking the wind on the lower levels on the hotel, but wind in this direction must be examined in the case that these buildings will not be present in the future and the full wind load will be applied to the building. Wind loads in this direction can clearly be seen in the diagram of windward and leeward pressures at each level (Figure 5.3). A basic loading diagram also provided for reference as seen in Figure 5.4.

Table 1c

Wind Loads (East/West Direction)													
B = 83'-0" L = 91'-0"													
Level	Height above ground - z (ft.)	Story Height (ft.)	K_z	q_z	Wind Pressure (psf)		Total Pressure (psf)	Force of Windward Pressure Only (k)	Force of Total Pressure (k)	Windward Shear Story (k)	Total Story Shear (k)	Windward Moment (ft-k)	Total Moment (ft-k)
					Windward	Leeward							
PH Roof	112.66	10.00	1.02	20.67	17.05	-8.65	25.70	4.54	6.84	4.54	6.84	488.88	736.82
Roof	102.66	10.00	1.00	20.27	16.79	-8.65	25.43	13.93	21.11	18.47	27.95	1360.70	2061.57
10	92.66	9.33	0.97	19.66	16.39	-8.65	25.04	12.70	19.39	31.17	47.34	1117.11	1706.30
9	83.33	9.33	0.94	19.05	16.00	-8.65	24.65	12.39	19.09	43.56	66.43	974.72	1501.44
8	74.00	9.34	0.90	18.24	15.48	-8.65	24.12	12.00	18.70	55.56	85.13	831.80	1296.52
7	64.66	9.33	0.87	17.63	15.08	-8.65	23.73	11.68	18.38	67.24	103.51	700.77	1102.49
6	55.33	9.33	0.83	16.82	14.56	-8.65	23.21	11.27	17.97	78.51	121.48	571.23	910.47
5	46.00	9.34	0.79	16.01	14.04	-8.65	22.68	10.88	17.58	89.39	139.06	449.69	726.72
4	36.66	9.33	0.74	15.00	13.38	-8.65	22.03	10.36	17.06	99.75	156.12	331.52	545.75
3	27.33	9.33	0.68	13.78	12.59	-8.65	21.24	9.75	16.45	109.51	172.57	221.05	372.81
2	18.00	18.00	0.60	12.16	11.55	-8.65	20.19	17.25	30.17	126.76	202.74	155.25	271.51
1	0	0	0	0	0	0	0	0	0	126.76	202.74	0	0
									Σ Windward Story Shear =	126.76	kips		
									Σ Total Story Shear =	202.74	kips		
									Σ Windward Moment =	7202.70	ft-k		
									Σ Total Moment =	11232.39	ft-k		

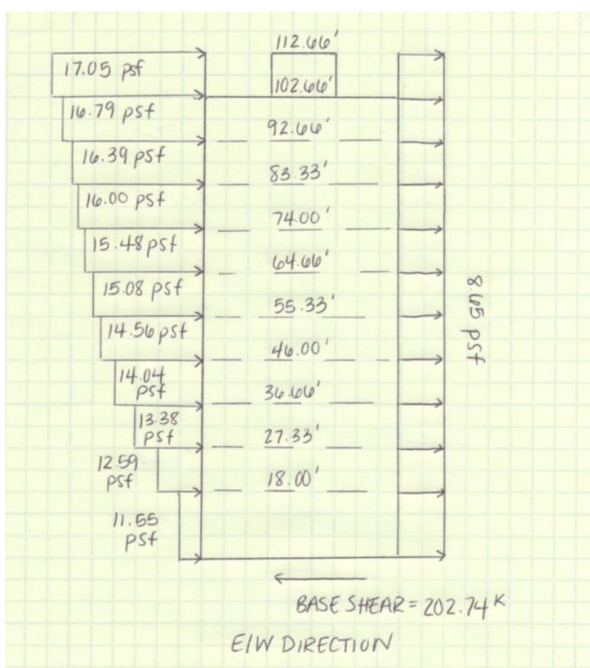


Figure 5.3: Wind Pressures in the East/West Direction

Wind Load Diagrams

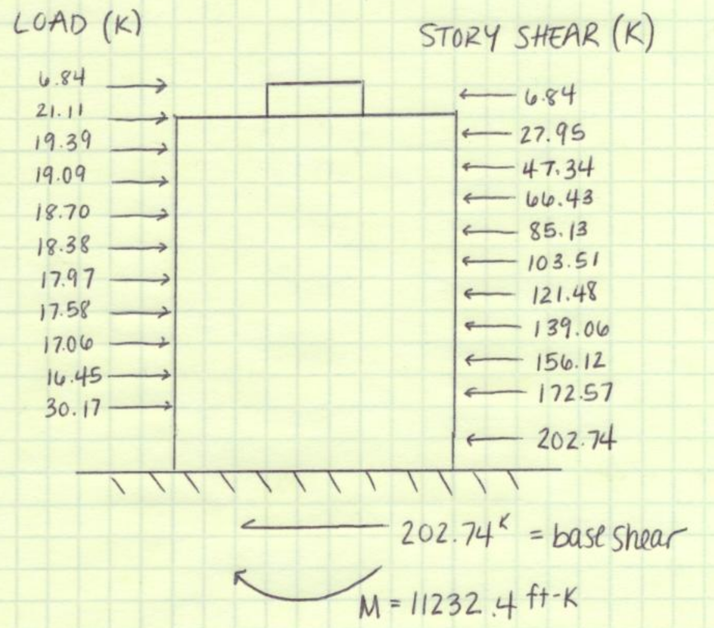


Figure 5.4: Wind Loading Diagram in East/West Direction

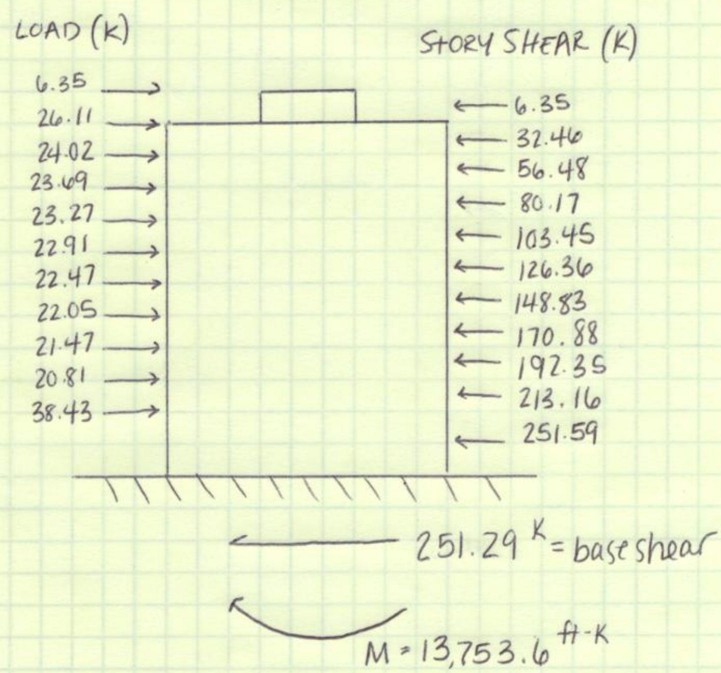


Figure 5.5: Wind Loading Diagram in North/South Direction

Seismic Analysis

Seismic loads were analyzed referencing Chapters 11 and 12 of ASCE 7-05. Upon investigation of the geotechnical report, the Fairfield Inn and Suites falls under the Site D classification. The variables needed to calculate base shear according to ASCE 7-05 are located in Table 2a.

Table 2a

Seismic Design Variables			ASCE References
Site Class		D	Table 20.3-1
Occupancy Category		II	Table 1-1
Importance Factor		1.00	Table 11.5-1
Structural System		Ordinary reinforced masonry shear walls	Table 12.2-1
Spectral Response Acceleration, short	S_s	0.125	USGS
Spectral Response Acceleration, 1 s	S_1	0.049	USGS
Site Coefficient	F_a	1.6	Table 11.4-1
Site Coefficient	F_v	2.4	Table 11.4-2
MCE Spectral Response Acceleration, short	S_{ms}	0.2	Eq. 11.4-1
MCE Spectral Response Acceleration, 1 s	S_{m1}	0.1176	Eq. 11.4-2
Design Spectral Acceleration, short	S_{ds}	0.133	Eq. 11.4-3
Design Spectral Acceleration,1 s	S_{d1}	0.0784	Eq. 11.4-4
Seismic Design Category	S_{dc}	B	Table 11.6-2
Response Modification Coefficient	R	2.0	Table 12.2-1
Approximate Period Parameter	C_t	0.02	Table 12.8-2
Building Height (above grade)	h_n	112.66	
Approximate Period Parameter	x	0.75	Table 12.8-2
Calculated Period Upper Limit Coefficient	C_u	1.70	Table 12.8-1
Approximate Fundamental Period	T_a	0.692	Eq. 12.8-7
Fundamental Period	T	1.17	§ 12.8.2
Long Period Transition Period	T_L	12	Fig. 22-15
Seismic Response Coefficient	C_s	0.034	Eq. 12.8-2
Structural Period Exponent	k	1.335	§ 12.8.3

The base shear calculated for seismic analysis includes the effective seismic building weight. An excel sheet was set up to determine the total weight that accumulated at each floor above grade. A summation of each floor resulted in the effective building weight which was used to determine the base shear and overturning moments due to seismic loads. Please refer to Appendix D for detailed calculations used to obtain building weight, as well as, base shear and overturning moments for each floor as seen in Table 2b. A seismic loading diagram is provided as reference to relate forces and shears that resulted as seen Figure 6.1.

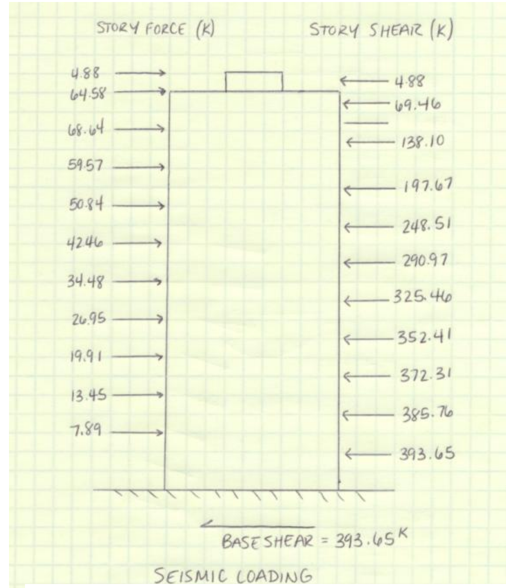


Figure 6.1: Seismic Loading Diagram

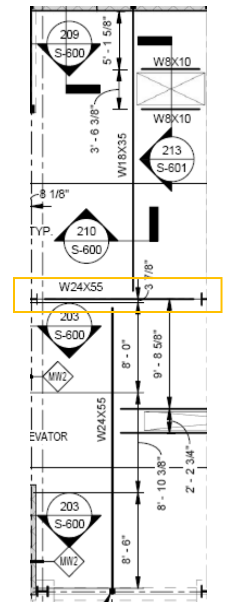
Table 2b

Base Shear and Overturning Moment Distribution							
Story	h_x (ft)	Story Weight (k)	$w_x h_x^k$	C_{vx}	Lateral Force F_x (k)	Story Shear V_x (k)	M_x (ft-k)
PH Roof	112.66	61.87	33932	0.012	4.88	4.88	525.16
Roof	102.66	927.40	449249	0.164	64.58	69.46	6307.25
10	92.66	1130.16	477463	0.174	68.64	138.10	6039.95
9	83.33	1130.16	414389	0.151	59.57	197.67	4686.25
8	74.0	1130.16	353641	0.129	50.84	248.51	3524.68
7	64.66	1130.16	295350	0.108	42.46	290.97	2547.34
6	55.33	1130.16	239878	0.088	34.48	325.46	1747.17
5	46.0	1130.16	187465	0.068	26.95	352.41	1113.84
4	36.66	1130.16	138463	0.051	19.91	372.31	636.87
3	27.33	1130.16	93552	0.034	13.45	385.76	304.82
2	18.0	1157.72	54877	0.020	7.89	393.65	71.00
1	0	390.00	0	0	0.00	393.65	0.00
			2738259				

Total Building Weight =	**11578.23**	**k**
Base Shear =	**393.65**	**k**
Total Moment =	**27504.33**	**ft-k**

SPOT CHECKS

Spot checks were performed to confirm the engineer of record's design methods. The spot check locations can be viewed in Figure __. Only gravity loads were applied when doing this calculations, therefore at least some variation can be due to the fact that lateral loads will be present and require separate analysis. Please refer to Appendix E for detailed calculations of each of the spot check descriptions.



Girder Spot Check Location

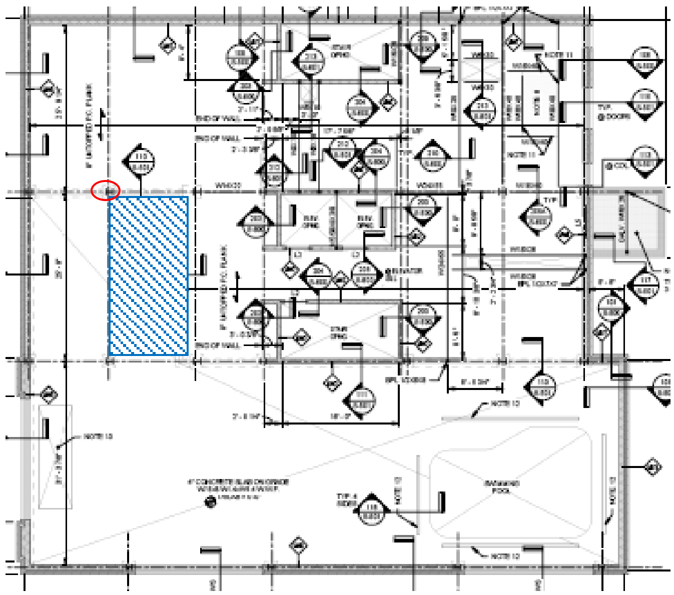


Figure 7.1: First Floor Plan Slab and Column Spot Check Locations

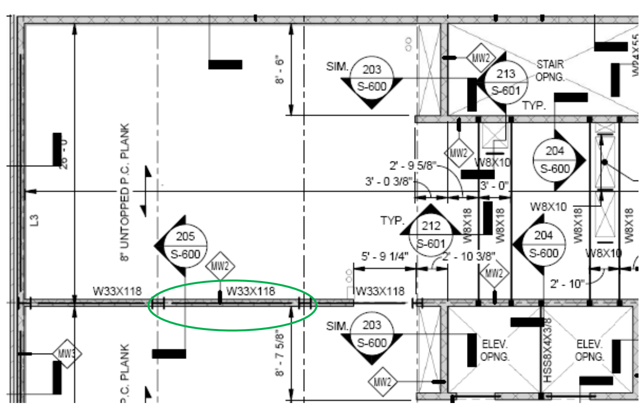


Figure 7.2: Second Floor Plan Transfer Beam Spot Check Location

A spot check of interior column A/2 was performed. Dead loads applied to the column were computed using the floor weights from the seismic calculations, taking into account the tributary area for the interior column (see Table 3a). A summary of accumulated load on the column at each floor is located in Appendix E. Live loads were applied in accordance with ASCE-07. A live load reduction was taken on the reducible live loads. It was assumed that the effective length, KL, of the column was equivalent to the floor to floor height of the column. It must be noted however that these loads are only due to gravity loads. Axial forces were applied to each column and no moments or additional forces due to lateral loads were taken into account. This accounts for the large gap between the column capacity and the present loads, due to the absence of other loads likely applied.

Table 3

Column Loads								
Level Supported	Tributary Area (sf)	Dead Load (psf)	Live Load (psf)	Reduction LL	Dead Load (kips)	Live Load (kips)	Total Load (1.2DL+1.6L) kips	Accumulated Load (kips)
Roof	339.13	123.57	75	49.28	41.9	16.7	77.02	77.02
10	339.13	150.58	55	36.14	51.1	12.3	80.89	157.91
9	339.13	150.58	55	36.14	51.1	12.3	80.89	238.80
8	339.13	150.58	55	36.14	51.1	12.3	80.89	319.68
7	339.13	150.58	55	36.14	51.1	12.3	80.89	400.57
6	339.13	150.58	55	36.14	51.1	12.3	80.89	481.46
5	339.13	150.58	55	36.14	51.1	12.3	80.89	562.34
4	339.13	150.58	55	36.14	51.1	12.3	80.89	643.23
3	339.13	150.58	55	36.14	51.1	12.3	80.89	724.12
2	339.13	154.26	55	36.14	52.3	12.3	82.38	806.50
1	339.13	51.96	100	100	17.6	33.9	149.33	955.83

A spot check was performed on an interior transfer beam. The calculations show that the typical W33x118 beam can carry the bending moment created by placing the concrete during construction. Once the concrete is placed and the two materials are working together as a composite system, the moment capacity is increased and the system can then carry the factored moment resulting from applied dead and live loads.

Next, a spot check was performed on a girder to examine that the member can transfer the loads from the beams to the columns. The calculations confirmed that a W24x55, the member chosen, can carry the induced moment created by the beams loading the girder on both sides.

The final spot check was done on the slab system on the first level. The slab was analyzed to validate the current design based on determined loads. Using the Direct Design Method, reference in Chapter 13 ACI 318-08, the panel was checked for minimum slab thickness. The column and middle strips were then checked for flexure based on the dead and live loads applied to the building and the design reinforcement was sufficient for the amount of reinforcement required. The was also a check of punching shear at column A/3 revealing that no special shear reinforcement was required at column.

CONCLUSION

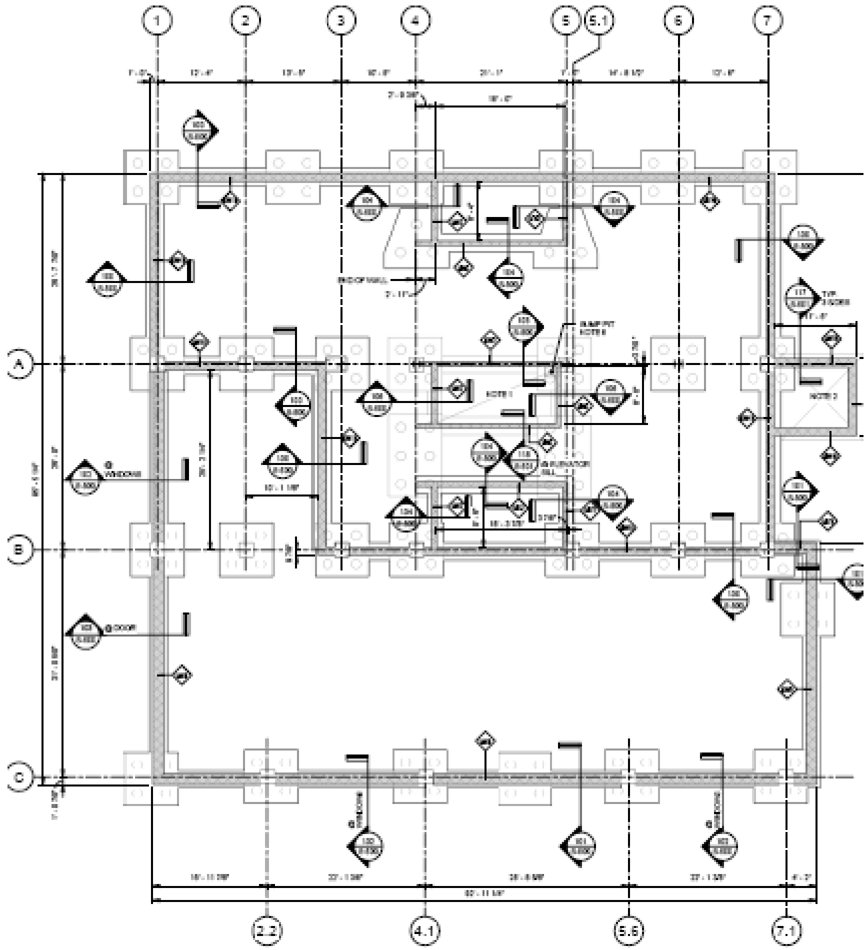
In analyzing the existing structural conditions of the Fairfield Inn and Suites, a better understanding of the design decisions and how the structure works as a whole was formed. After an examination and calculation of gravity and lateral loads, it is found that the Fairfield Inn and Suites is adequately designed to withstand the forces against the structure.

Calculations were done on various structural members to verify the structural design. A transfer beam and girder were analyzed for exposure to gravity loads. The spot checks concluded that the members were adequate to carry the design forces and it was found that the floor system meets serviceability criteria in accordance with the allowable live load deflection. The column spot check was performed using flexural buckling analysis and was adequately designed. These members were all satisfactorily designed, but seemed to be over designed. This can account for that fact that each of these members is also a part of the lateral system of the building. Being a part of the lateral system, these members undergo more than just the gravity loads alone. Therefore, these members were over designed to carry the moments created by the lateral forces on the building.

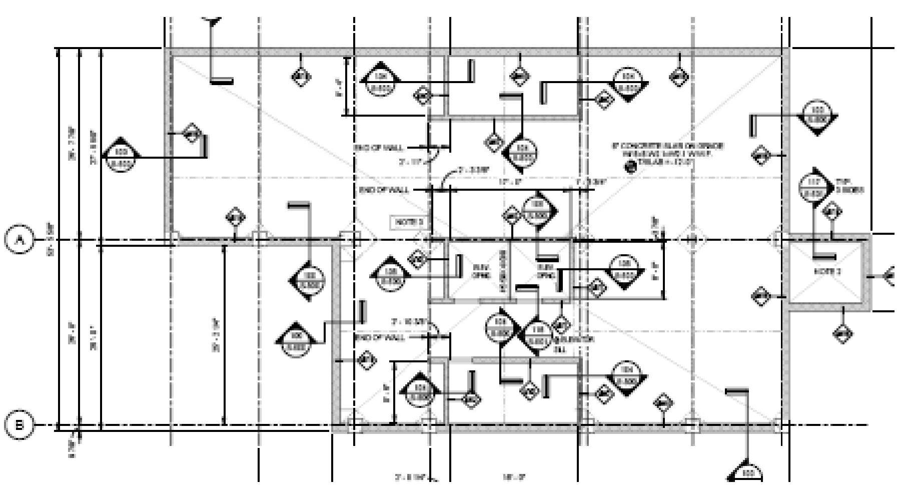
The lateral forces against the building were found through wind and seismic analysis. It was found that the North/South wind direction loads controlled over the East/West direction, which make sense since that building façade is longer and should resist larger loads. Through comparing the base shear forces on the building between wind loads and seismic loads, it was found that the seismic loads control over the wind loads with a slightly larger base shear force without considering torsion effects. The lateral system is comprised of reinforced shear walls with loads carried through transfer beams then transferred into columns down into the foundation. This justifies why the over designed column, girder, and transfer beam, were chosen to account for the lateral forces. These forces will be taken into account in further research of the Fairfield Inn and Suites building structure.

Please refer to the following appendices for detailed calculations, diagrams, and tables referring to each analysis done on the structure.

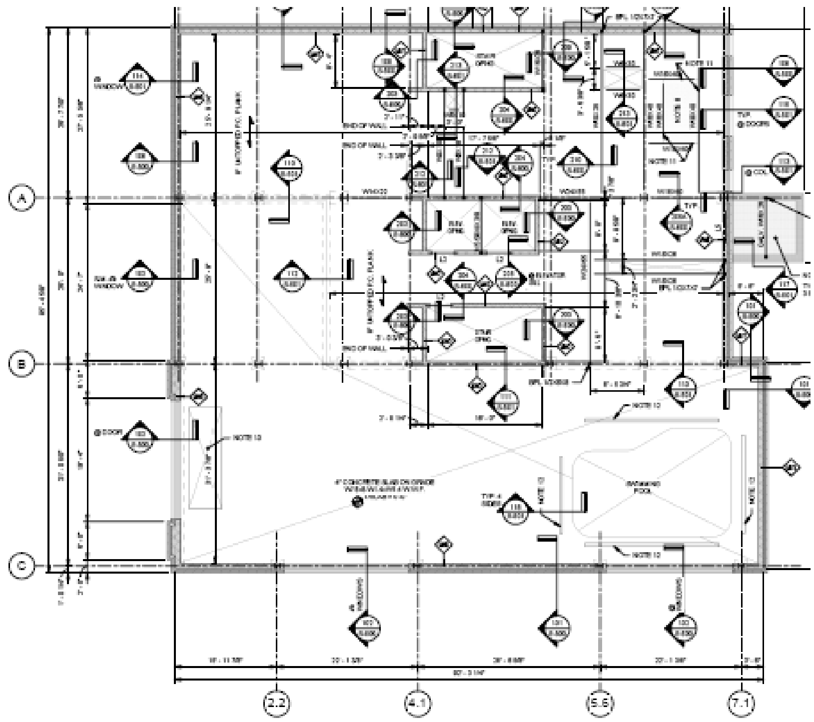
APPENDIX A: BUILDING LAYOUT



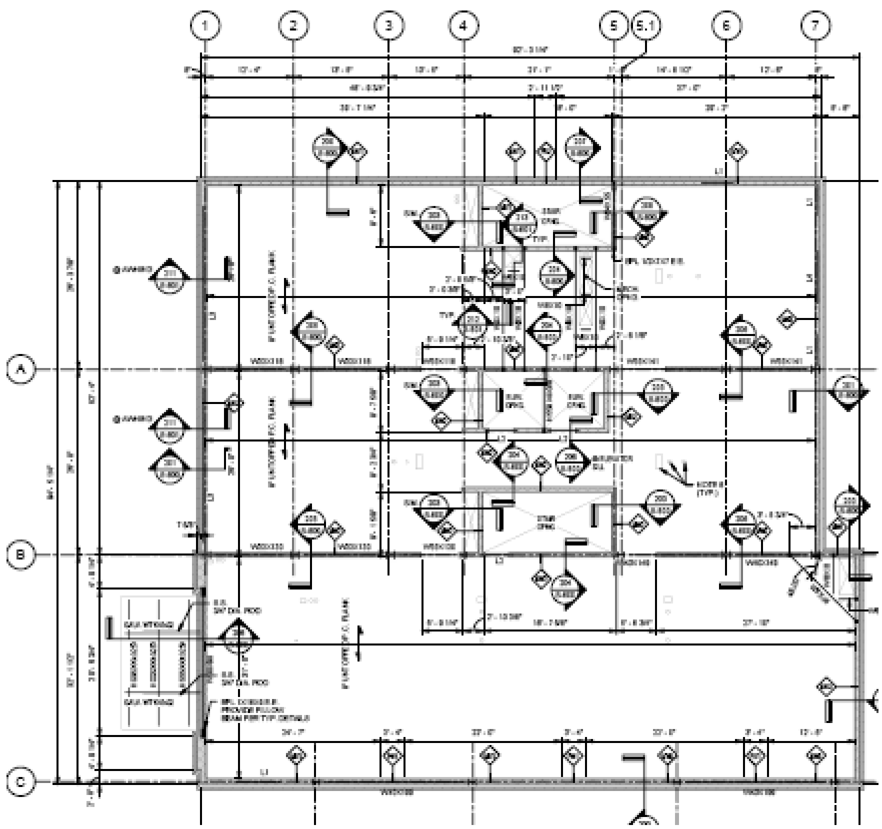
Foundation Plan



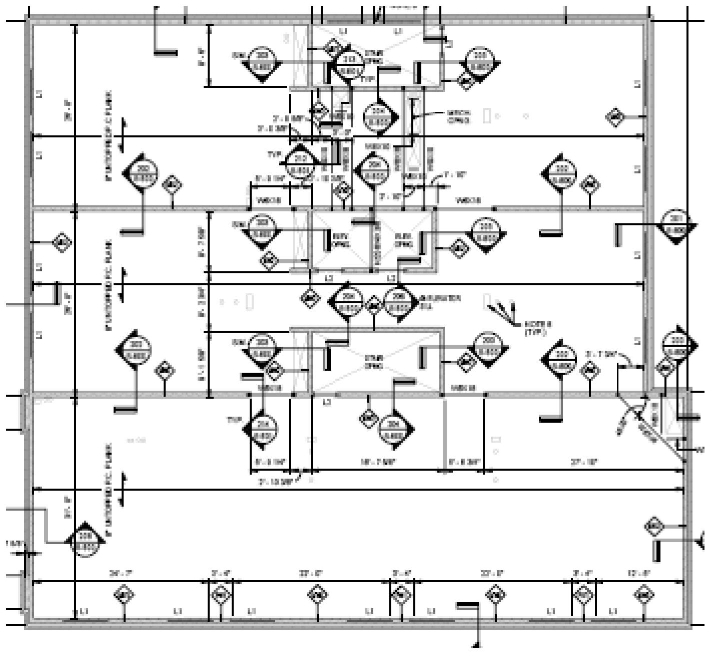
Basement Plan



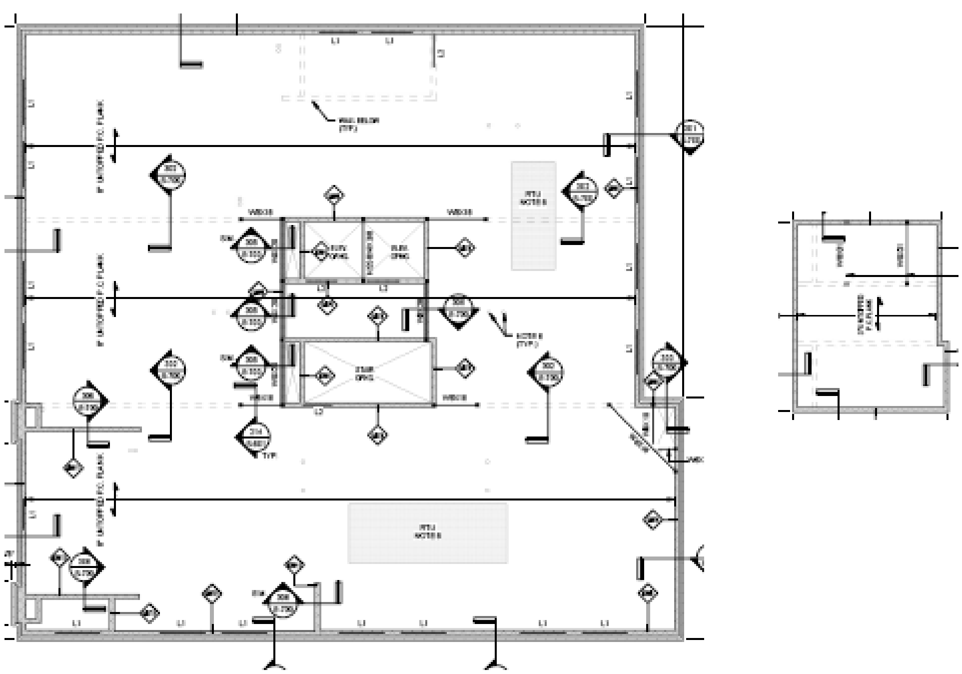
First Floor Framing Plan



Second Floor Framing Plan



Third thru Tenth Floor Framing Plan



Roof/Penthouse Roof Plan

APPENDIX B: SNOW ANALYSIS

1/2

TECH REPORT I - CALCULATIONS
A. SMITH

SNOW LOADS

- ground snow load $P_g = 25$ psf (fig 7-1)
  - AES used 30 psf

- flat roof snow load

$$P_f = 0.7\, C_e C_t I P_g > 20(I) \quad \text{(equ. 7-1)}$$

C_e = exposure factor = 1.0 (partially exp. - B) (table 7-2)
C_t = thermal factor = 1.0 (table 7-3)
I = importance factor = 1.1 (occupancy III) (table 7-4)

$$P_f = 0.7(1.0)(1.0)(1.1)(25) = 19.25 \text{ psf} < 20(1.1) = 22 \text{ psf}$$

will use $\boxed{P_g = 30 \text{ psf}}$ to be conservative

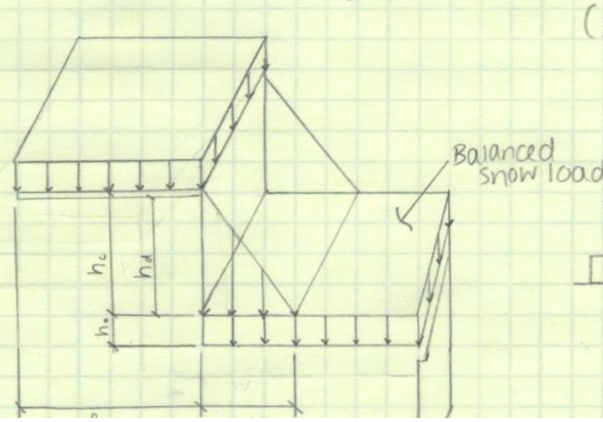
$$P_f = 0.7(1.0)(1.0)(1.1)(30) = 24 \text{ psf} > 20(1.1) = 22 \text{ psf}$$

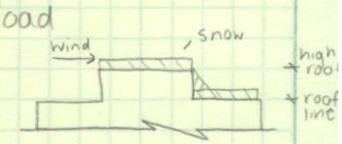
$$\boxed{P_f = 24 \text{ psf}}$$

- SNOW DRIFT

due to high roof rising above main roof line

(FIG 7-8) (sect. 7.7.1 in ASCE-7)





2/2

SNOW DRIFT (CONT)

l_u = length of roof upwind drift

h_c = clear height from top balanced snow load to closest upper roof

h_b = height of balanced snow load P_s/γ

h_d = height of snow drift

w = distance from eave to ridge

$p_d = h_d \gamma$

$\gamma = 0.13 p_g + 14 = 0.13(30) + 14 = 17.9$ pcf

$h_d = 0.43 \sqrt[3]{l_u} \sqrt[4]{p_g + 10} - 1.5$

$l_u = 21'-11\frac{1}{4}''$ (N/S WIND)
$= 26'-7\frac{5}{8}''$ (E/W WIND)

$h_d = 1.53'$ (N/S)
$= 1.730'$ (E/W)

$h_b = P_g/\gamma = 1.676'$

$W = 4h_d = 6'-1\frac{3}{8}''$ (N/S)
$= 6'-11''$ (E/W)

APPENDIX C: WIND ANALYSIS

1/3

TECH REPORT 1 - CALCULATIONS
A. SMITH

WIND LOADS

METHOD 2 - analytical procedure

- Determine wind variables

$V = 90$ mph
$K_d = 0.85$
$I = 1.15$
exposure = B
$K_{zt} = 1.00$

interpolate K_z:

(table 6-3)
case 2

level	height	K_z
1	0	0
2	18'-0"	0.60
3	27'-4"	0.68
4	36'-8"	0.74
5	46'-0"	0.79
6	55'-4"	0.83
7	64'-8"	0.87
8	74'-0"	0.90
9	83'-4"	0.94
10	92'8"	0.97
roof	102'8"	1.00
high roof	112'8"	1.02

q_z (velocity pressure) $= 0.00256 \boxed{K_z} K_{zt} K_d V^2 I$
varies by level

$q_z = 0.00256 \boxed{K_z} (1.00)(0.85)(90^2)(1.15)$

example @ level 2: $q_z = 12.16$ *COMPLETED IN TABLE FOR ALL LEVELS*

q_h @ mean roof height $\bar{z} = \dfrac{102.66 + 112.66}{2} = 107.66$ ft. $= K_z = 1.01$

$\bar{z} = 0.6h = 0.6(107.66) = \boxed{64.6'} > z_{min} = 30'$ ✓

$q_h = 0.00256(1.01)(0.85)(1.00)(90^2)(1.15) = \boxed{20.47}$

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{64.6}\right)^{1/6} = \boxed{0.268}$

$L_{\bar{z}} = \ell\left(\bar{z}/33\right)^{\bar{\epsilon}} = 320\left(64.6/33\right)^{1/3} = \boxed{208.81}$

2/3

WIND LOADS (cont)

$n = 107.66$

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

North/South $B = 91'-0"$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{91+107.66}{208.8}\right)^{0.63}}}$$

$\boxed{Q_{N/S} = 0.788}$

East/West $B = 83'$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{83+107.66}{208.8}\right)^{0.63}}}$$

$\boxed{Q_{E/W} = 0.792}$

$$G = 0.925\left(\frac{1+1.7g_Q I_{\bar{z}} Q}{1+1.7g_v I_{\bar{z}}}\right)$$

g_Q / g_v should be 3.4

$\boxed{G_{N/S} = 0.806}$

$\boxed{G_{E/W} = 0.808}$

C_p - pressure coefficient

North/South
windward = 0.8
leeward = -0.5
$L/B = 0.91$
$L = 83'$ $B = 91'$

East/West
windward = 0.8 (q_z)
leeward = -0.3 (q_h)
$L/B = 1.09$
$L = 91'$ $B = 83'$

- Wind Pressure

$P_z = q_z G C_p - q_h GC_{pi}$ (windward)

$P_h = q_h G C_p - q_h GC_{pi}$ (leeward)

$GC_{pi} = +0.18$
-0.18
FOR ENCLOSED BUILDINGS

North/south

example @ level 2: $P_z = 12.16\,(0.806)(0.8) - 20.47(-0.18)$

$P_z = 11.53$ psf

$P_h = 20.47(0.806)(-0.5) - 20.47(0.18)$

$P_h = -11.93$ psf

3/3

WIND Pressure (cont)

East/West

example @ level 2:

$$P_z = 12.16(0.808)(0.8) - 20.47(-0.18)$$

$$P_z = 11.54 \text{ psf}$$

$$P_h = 20.47(0.808)(-0.3) - 20.47(0.18)$$

$$P_h = -8.65 \text{ psf}$$

* wind pressures for each story calculated in table *

- Force of windward (ONLY)

$$f = B(\text{story height})\, p_z$$

N/S example @ level 5: $F = 91'(9.34')(14.01) = 11.91\text{ K}$

- Force Total pressure

N/S example @ level 5: $F = 91'(9.34')(25.94\text{psf}) = 22.05\text{ K}$

- Windward shear story

N/S example @ level 9: $F = f_{\text{windward}}$ @ (PH roof + roof + 10 + 9)

$$F = (3.73 + 15.25 + 13.89 + 13.56) = 46.43\text{ K}$$

- Total shear story

N/S example @ level 10: $F = f_{\text{total}}$ @ (PH roof + roof + 10)

$$F = (6.35 + 26.11 + 24.02) = 56.48\text{ K}$$

APPENDIX D: SEISMIC ANALYSIS

Seismic Force Resisting System: Floor Weights

Floor 1					
Approximate Area:	7505.12	sf			
Floor to Floor Ht.	18.0	ft			
Walls:			**Superimposed:**		
Perimeter:	665.46	ft.	Partitions:	15	psf
Height:	9.33	ft.	MEP:	10	psf
Unit Wt:	20	psf	Finished:	5	psf
Weight =	**124.17**	**k**	**Weight =**	**225.15**	**k**
Slab:					
Do Not Include Slab Weight					
Columns:					
Shape	Quantity	Weight (lb/ft)	Column Height (ft)	Total Weight (k)	
W12x120	1	120	18	2.16	
W12x96	2	96	18	3.46	
W10x100	2	100	18	3.60	
W12x120	5	120	18	10.80	
W12x106	2	106	18	3.82	
W10x112	4	112	12	5.38	
W10x68	2	68	18	2.45	
			Weight =	**31.66**	**k**
Beams:					
Shape	Quantity	Weight (lb/ft)	Beam length (ft)	Total Weight (k)	
W 8x10	1	10	3	0.03	
W 8x10	2	10	6.65	0.13	
W 8x18	2	18	17.5	0.63	
W 14x22	1	22	10.43	0.23	
W 16x26	2	26	22.11	1.15	
W 16x26	1	26	8.33	0.22	
W 18x35	1	35	25.58	0.90	
W 18x40	1	40	12.5	0.50	
W 18x40	2	40	25.58	2.05	
W 18x40	3	40	8.5	1.02	
W 24x55	1	55	24.58	1.35	
W 24x55	1	55	14.71	0.81	
			Weight =	**9.01**	**k**
Total Weight of Floor =			**390.00**	**k**	
or			51.96	psf	

Seismic Force Resisting System: Floor Weights

Floor 2					
Approximate Area:		7505.12	sf		
Floor to Floor Ht.		9.33	ft		
Walls:			Superimposed:		
Perimeter:	816.64	ft.	Partitions:	15	psf
Height:	9.33	ft.	MEP:	10	psf
Unit Wt:	20	psf	Finished:	5	psf
Weight =	**152.39**	**k**	**Weight =**	**225.15**	**k**
Slab:					
Thickness:		8	in		
Unit Weight:		150	pcf		
Weight =		**750.512**	**k**		
Beams:					
Shape	Quantity	Weight (lb/ft)	Beam length (ft)	Total Weight (k)	
W 8x10	5	10	3	0.15	
W 8x10	1	10	4.77	0.05	
W 8x18	4	18	17.5	1.26	
W 8x18	1	18	12.88	0.23	
W 24x55	1	55	8.5	0.47	
W 30x90	1	90	20.56	1.85	
W 33x118	1	118	10.42	1.23	
W 33x118	1	118	13.42	1.58	
W 33x118	1	118	12.33	1.45	
W 33x130	1	130	10.42	1.35	
W 33x130	1	130	13.42	1.74	
W 33x130	1	130	12.33	1.60	
W 33x141	1	141	14.71	2.07	
W 33x141	1	141	12.5	1.76	
W 40x149	1	149	12.5	1.86	
W 40x149	1	149	14.71	2.19	
W 40x199	2	199	22.11	8.80	
			Weight =	**29.67**	**k**
Total Weight of Floor =			**1157.72**	**k**	
or			154.26	psf	

Seismic Force Resisting System: Floor Weights

Floors 3 thru 10					
Approximate Area:		7505.12	sf		
Floor to Floor Ht.		9.33	ft		
Walls:			**Superimposed:**		
Perimeter:	816.64	ft.	Partitions:	15	psf
Height:	9.33	ft.	MEP:	10	psf
Unit Wt:	20	psf	Finished:	5	psf
Weight =	**152.39**	**k**	**Weight =**	**225.15**	**k**
Slab:					
Thickness:		8	in		
Unit Weight:		150	pcf		
Weight =		**750.512**	**k**		
Beams:					
Shape	Quantity	Weight (lb/ft)	Beam length (ft)	Total Weight (k)	
W 8x10	5	10	3	0.15	
W 8x10	1	10	4.77	0.05	
W8x18	4	18	17.5	1.26	
W8x18	1	18	12.88	0.23	
W8x18	4	18	5.77	0.42	
			Weight =	**2.10**	**k**
Total Weight of Floor =			**1130.16**	**k**	
or			150.58	psf	

Seismic Force Resisting System: Floor Weights

Roof					
Approximate Area:		7505.12	sf		
Walls:			**Superimposed:**		
Perimeter:	125.54	ft.			
Height:	10	ft.	MEP:	10	psf
Unit Wt:	20	psf	Roof Mat:	10	psf
Weight =	**25.11**	**k**	**Weight =**	**150.1**	**k**
Slab:					
	Thickness:	8	in		
	Unit Weight:	150	pcf		
	Weight =	**750.512**	**k**		
Beams:					
Shape	Quantity	Weight (lb/ft)	Beam length (ft)	Total Weight (k)	
W 8x10	1	10	3	0.03	
W 8x10	1	10	4.77	0.05	
W 8x18	1	18	12.88	0.23	
W 8x18	4	18	5.77	0.42	
W 8x28	4	28	8.5	0.95	
			Weight =	**1.68**	**k**
		Total Weight of Floor =	**927.40**	**k**	
		or	123.57	psf	

Seismic Force Resisting System: Floor Weights

PH Roof					
Approximate Area:		557.68	sf		
Superimposed:					
	Roof Mat:	10	psf		
	Weight =	**5.58**	**k**		
Slab:					
	Thickness:	8	in		
	Unit Weight:	150	pcf		
	Weight =	**55.768**	**k**		
Beams:					
Shape	Quantity	Weight (lb/ft)	Beam length (ft)	Total Weight (k)	
W 8x31	2	31	8.5	0.53	
			Weight =	**0.53**	**k**
		Total Weight of Floor =	**61.87**	**k**	
		or	110.94	psf	

1/2

TECH I CALCULATIONS
A SMITH

SEISMIC LOADS

- $S_{MS} = F_a S_s$
  $S_{MS} = 1.6(0.125)$ $\boxed{S_{MS} = 0.2}$
- $S_{DS} = 2/3 (S_{MS})$
  $\boxed{S_{DS} = 0.133}$
- $S_{M1} = F_v S_1$
  $S_{M1} = 2.4(0.049)$ $\boxed{S_{M1} = 0.1176}$
- $S_{d1} = 2/3 (S_{M1})$
  $\boxed{S_{d1} = 0.0784}$
- T_a (approximate fund. period) = $C_t h_n^x$
  $T_a = 0.02(112.66)^{0.75}$
  $\boxed{T_a = 0.692 \text{ s}}$
- $T = T_a(C_u) = 0.692(1.7) = \boxed{1.17 \text{ s}}$
- $C_s = \min \begin{cases} \dfrac{S_{D1}}{T(R/I)} = \dfrac{0.0784}{1.17(2/1)} = \boxed{0.0334} \geq 0.01 \\ \dfrac{S_{DS}}{R/I} = \dfrac{0.133}{2} = 0.0665 \\ \dfrac{S_{D1} T_L}{T^2(R/I)} = \dfrac{0.0784(12)}{1.17^2(2)} = 0.344 \end{cases}$
- $k = 0.75 + 0.5(T) = 0.75 + 0.5(1.17) = \boxed{1.335 = k}$

SEE EXCEL SHEET FOR FLOOR WEIGHTS

FLOOR 1:	7505.12 sf	51.96 psf
FLOOR 2:	7505.12 sf	154.26 psf
FLOOR 3-10:	7505.12 sf	150.58 psf
ROOF:	7505.12 sf	123.57 psf
PH ROOF:	557.68 sf	110.94 psf

TOTAL BUILDING WEIGHT

- $W_T = 7505.12(51.96) + 7505.12(154.26) + 8(7505.12)(150.58) + 7505.12(123.57) + 557.68(110.94)$

$W_T = 11578^k$

2/2

SEISMIC LOADS (CONT)

- base shear (V)

$V = C_s W_T = 0.034(11578)$

$\boxed{V = 393.65\ K}$

- $W_x h_x^K$ varies @ height

example for level 4: $W_x = 1130.16\ K$

$h_x = 36.66'$

$K = 1.335$

$= 1130.16(36.66)^{1.335}$

$\boxed{= 138464}$

$\Sigma w_i h_i^K$ = sum of $w_x h_x^K$ for each floor $= \boxed{2738259\ ft\text{-}K}$

- $C_{vx} = \dfrac{W_x h_x^K}{\Sigma W_x h_x^K}$ varies @ heigh

example for floor 8: $C_{vx} = \dfrac{353641}{2738259} \boxed{= 0.129}$

- $F_x = C_{vx} V$ (lateral force)

example @ floor 6: $C_{vx} = 0.088$

$V = 393.65$

$F_x = 0.088(393.65)$

$\boxed{F_x = 34.48\ K}$

- Story Shear (V_x)

V_x = lateral force (F_x) @ level + (F_x) @ all levels above

example floor 9: $V_x = F_x(PH) + F_x(Roof) + F_x(10) + F_x(9)$

$\boxed{V_x = 197.07\ K}$

- Moments (M_x)

M_x = (trib area ht.) x F_x

example @ 9: $M_x = (78.665)(59.57)$

$\boxed{M_x = 4686.07\ ft\,K}$

APPENDIX E: SPOT CHECKS

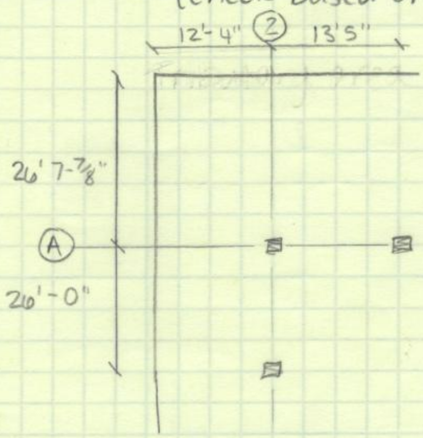
1/3

TECH I REPORT
A. SMITH

SPOT CHECKS

COLUMN A-2, LEVEL 1

(check based on axial-gravity loads only)



- Tributary Area $= (6.17' + 6.71') + (13' + 13.33')$

$A_T = 339.13 \text{ ft}^2$

- Live Loads (IBC 2006 table 1607.1)

$= 40$ psf (hotel rooms, 9 floors)

$= 75$ psf (roof)

$= 15$ psf (partitions)

$= 100$ psf (lobby)

Live Load element factor $K_{LL} = 4$

(interior column, IBC 2006 table 1607.9.1)

The live loads (hotel, roof, partition) can be reduced by ($K_{LL}A_T \geq 400 \text{ ft}^2$)

- reduced hotel Live Load

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) \quad \text{(eqn: 16-24)}$$

$$= L_o\left(0.25 + \frac{15}{\sqrt{4 \times 339.13}}\right)$$

$L = (0.657)L_o$

$L = 0.657(40)$

$L = 26.3$ psf

- reduced roof live load

$L = (0.657)L_o$

$L = 0.657(75)$

$L = 49.3$ psf

- reduced partition LL

$L = 0.657(L_o)$

$L = 0.657(15)$

$L = 9.855$ psf

2/3

SPOT CHECKS
COLUMN (CONT)

- Dead Loads (hotel rooms and roof)
  - include 8" slab/walls/superimposed loads
- See seismic spreadsheet

Roof = 123.57 psf
FLOORS 3-10 = 150 psf
FLOOR 2 = 154.3 psf
FLOOR 1 = 51.96

- Reduction live loads chart

ROOM	# floors	LIVE LOADS
guest room	9	35.3
lobby	1	100
roof	1	49.3

- See excel sheet for total accumulated loads
- include self weight of column 24" x 24" on 1st floor ONLY

$$\left(\left(\frac{24'' \times 24''}{12^2}\right) \times 102.6\right)\left(\frac{150 \text{ pcf}}{1000}\right) = 61.56^{K}$$

- critical combination load

$1.2D + 1.6L$

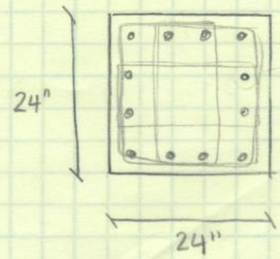
FOR COLUMN A-2 ON LEVEL 1 WILL SUPPORT LEVELS 2-ROOF

$P_u = 951.76 \text{ K}$

3/3

SPOT checks
column (cont)

actual design



(12) #10 bars

tie reinforcement
#3 @ 16" O.C.

$A_{st} = 15.24 \text{ in}^2$

$F_y = 60$ ksi

$f'_c = 8$ ksi

$$\phi P_{n,max} = 0.80\phi\left[0.85f'_c(A_g - A_{st}) + f_y A_{st}\right] \quad \text{(eqn. 10-2 ACI 318-08)}$$

$$\phi P_{n,max} = 0.80(0.65)\left[0.85(8)(24'' \times 24'' - 15.24) + 60(15.24)\right]$$

$$\phi P_{n\,max} = 2458.3 \text{ K}$$

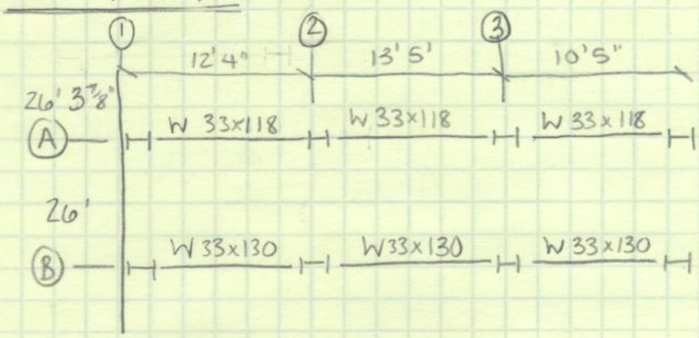
$$\boxed{P_u = 951.76^K < \phi P_n = 2458.3^K} \quad \checkmark \text{ OKAY}$$

* columns so much large than need be to resist moments due to lateral loads

1/3

SPOT CHECK

transfer beam



2nd Level transfer beam

- LOADS
  - Live: assume 100 psf ← (partitions/rooms/corridors)
  - Dead: = 1591.0 psf (see appendix D: seismic analysis) for loads supported by transfer beam
- Floor System
  - 8" PC plank floor (N.W.C)
- load combination $1.2D + 1.6L$

factored load: $1.2D + 1.6L$

$W_u = 1.2(1591.0\text{ psf}) + 1.6(100\text{ psf})$

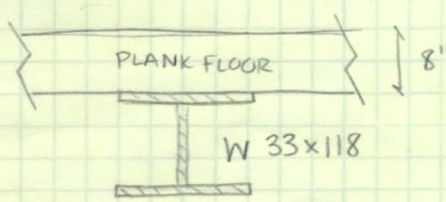
$W_u = 2069.2\text{ psf}$

trib. width = 26'

$W_u = 2069.2\text{psf}(26)'/1000 = 53.8\text{ Klf}$

$M_u = \frac{W_u l^2}{8} = \frac{53.8(13.42')^2}{8} = 1211.14\text{ K-ft.}$

$b_{eff} = \min \left| \begin{array}{l} \text{spacing} = 26 \times 12 = 312'' \\ \frac{\text{span}}{4} = \frac{13'(12)}{4} = 40'' \end{array} \right.$



2/3

Spot checks
beam

check for deflection under construction loads:

$$\Delta_{CONST} = \frac{5 w_{conc} l^4}{384 EI}$$

$W_{conc} = 145pcf(8/12) = 97 psf$

$W_{conc} = 97psf(26') = 2.513$ klf

$$\Delta_{allow} = l/360 = \frac{13.42(12)}{360} = 0.447''$$

$$I_{req.} = \frac{5 w_{con} l^4}{384 \Delta_{con} E} = \frac{5(2.513)(13.42)^4 \times 1728}{384(0.447)(29000)}$$

$I_{req.} = 141.47 \text{ in}^4$

$I_{y,\,W33\times118} = 187 \text{ in}^4$ $\boxed{187 \text{ in}^4 > I_{req}}$ ✓ OKAY

check bending for construct.

$W_{CONC} = 2.513$ Klf

$W_{LIVE} = 20 psf(26') = .520$ Klf

$W_u = 1.2(2.513) + 1.6(0.520)$

$W_u = 3.85$ Klf

$$M_u = \frac{W_u l^2}{8} = \frac{3.85(13.42)^2}{8} = 86.62'^K$$

$\phi M_{n\,(W33\times118)} = 192'^K > 86.62'^K$ ✓ OKAY

From table 3-19: Steel Manual

assume $\Sigma Q_n = 659^K$

$$a = \frac{\Sigma Q_n}{0.85(f'c)(b_{eff})} = \frac{659}{0.85(4)(40)} = 4.85''$$

$Y_2 = 8'' - a/2 = 8'' - 4.85/2 = 5.6''$ [ROUND ↓ to 5.5" TO BE CONSERVATIVE]

3/3

Spot checks

transfer beam

table 3-19:

W 33x118 $Y_2 = 5.5"$ $\Sigma Q_n = 659^k$ @ PNA #6

$\phi M_n = 2340\ ^{'k}$

$\phi M_n > M_u = 1211.14\ ^{'k}$ ✓ OKAY

check deflection:

table 3-20, Steel Manual

$Y_2 = 5.5" \rightarrow I_{LB} = 10500$

$$\Delta_{LL} = \frac{5 w_{LL} l^4}{384 E I_{LB}}$$

$w_{LL} = 100(26')/1000 = 2.60$ klf

$$\Delta_{allow} = \frac{l}{360} = \frac{13.42 \times 12}{360} = 0.447"$$

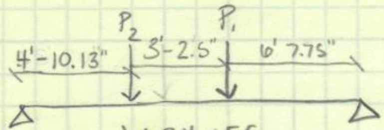
$$\Delta_{LL} = \frac{5(2.60)(13.42)^4}{384(29000)(10500)}$$

$\Delta_{LL} = 0.01" < 0.447"$ ✓OKAY

1/2

SPOT CHECK

Girder



W 24 x 55

P_1 =

$W_{u,DL} = 1.2(154.26)(4.93')/1000 = 0.913$ klf

$W_{u,LL} = 1.6(100psf)(4.93')/1000 = 0.79$ klf

P_2 =

$W_{u,DL} = 1.2(154.26)(4.03')/1000 = 0.746$ klf

$W_{u,LL} = 1.6(100)(4.03')/1000 = 0.645$ klf

$P_{1DL} = \frac{0.913(25.6)}{2} = 11.69^K$ $\quad P_1 = (11.69 + 10.112) = 21.8^K$ $\times 2 = 43.8^K$

$P_{1LL} = \frac{0.79(25.6)}{2} = 10.112^K$

$P_{2DL} = \frac{0.746(25.6)}{2} = 9.55^K$ $\quad P_2 = (9.55 + 8.26) = 17.81^K$ $\times 2 = 35.62^K$

$P_{2LL} = \frac{0.645(25.6)}{2} = 8.26^K$

$M_{max} = \frac{(P_1(l-a) + P_2 b)}{l} a = 527.81^K$

assume $Y_2 = 4"$ $\longrightarrow$ requiring PNA # 7 (table 3-19)

$\Sigma Q_n = 203^K$

$b_{eff} = \min \begin{cases} \text{spacing} = 25.6' = 307.12" \\ \text{span}/4 = 14.71'/4 = 44" \leftarrow \text{CONTROLS} \end{cases}$

$a = \frac{\Sigma Q_n}{0.85 f'c\, b_{eff}} = \frac{203}{0.85(4)(44)} = 1.36"$

$Y_2 = 6" - 1.36/2 = 5.32 \rightarrow 5"$ $\boxed{\text{round} \downarrow 5" \text{ CONSERVATIVE}}$

2/2

Spot check
Girder

$Y_2 = 5''$ $\Sigma Q_n = 203^k$ W 24x55 PNA # 7

$\phi M_n = 720'^k$

$\phi M_n > M_u = 527.8'^k$ ✓OKAY

check deflection:
(table 3-20)

$I_{LB} = 2260$

$$\Delta_{LL} = \frac{5 w_{LL} \ell^4}{384 E I_{LB}} = \frac{5(2.56)(14.71)^4}{384(29000)(2260)} \times 1728 = 0.041''$$

$W_{LL} = 100\text{ psf}(25.6)/1000 = 2.56$

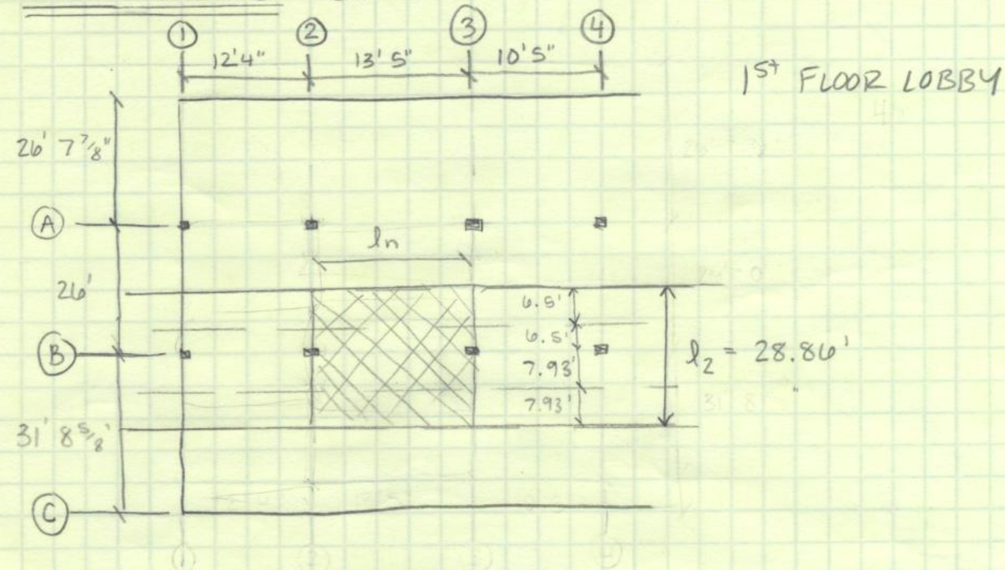
$$\Delta_{allow} = \frac{14.71 \times 12}{360} = 0.49''$$

$0.041'' < 0.49''$ ✓OKAY

1/8

TECH REPORT I
A SMITH

SPOTCHECKS – SLAB



• 4" CONC. SLAB ON GRADE
#5 bars @ 12" OC
each way / top & bottom

$l_1 = 13.42'$

$l_n = 13.42' - \frac{1}{2}(24") - \frac{1}{2}(24")$

$\boxed{l_n = 11.42'}$

$l_2 = \dfrac{26' + 31' 8\frac{5}{8}"}{2}$

$\boxed{l_2 = 28.86'}$

• Dead Loads

CONC = 150 pcf (4/12") = 50 psf

MEP/PARTITIONS = 15 psf

• LIVE LOADS

LOBBY = 100 psf

2/8

SPOT CHECKS
SLAB (CONT)

- $q_u = 1.2D + 1.6L$
  $= 1.2(65) + 1.6(100)$
  $\boxed{q_u = 238 \text{ psf}}$

- $M_u = \dfrac{q_u l_2 l_n^2}{8}$ (ACI 318-05 13.6.2.2)
  $= \dfrac{0.238(28.86')(11.42')^2}{8}$
  $\boxed{M_u = 111.97 \text{ ft-K}}$

- Negative & positive factored moments (ACI 318-05 13.6.3.2)
  $M^- = 0.65(M_u)$
  $M^+ = (0.35)M_u$

  $M^- = 0.65(111.97'^K)$
  $\boxed{= 72.78 \text{ ft K}}$
  $M^+ = 0.35(111.97'^K)$
  $\boxed{= 39.19 \text{ ft K}}$

- COLUMN STRIP MOMENTS (ACI 318-05 13.6.4.1)
  $l_1 = 13.42'$ $\quad l_1/l_2 = \dfrac{13.42}{28.86} = 0.465$
  $l_2 = 28.86'$

  $M^-_{COLM} = 0.75(M^-)$
  $= 0.75(72.78)$
  $\boxed{= 54.6 \text{ ft-K}}$

  $M^+_{COLM} = 0.60(M^+)$
  $= 0.60(39.19)$
  $\boxed{= 23.52 \text{ ft K}}$

3/8

SPOT CHECK
SLAB (CONT)

- MIDDLE Strip moments (ACI 318-05 13.6.6.1)

$M^-_{Mid} = 0.25(M^-)$
$= 0.25(72.78) = \boxed{18.20 \text{ ft K}}$

$M^+_{Mid} = 0.40(M^+)$
$= 0.40(39.19)$
$\boxed{= 15.676 \text{ ft K}}$

- Slab Strength

$f_y = 60$ Ksi
$f'_c = 5$ Ksi

Column strip: $b = 173.16''$
$h = 4''$
$d = 4 - 0.5 - 2(0.625) \rightarrow$ 2 #5 bars top & bottom
0.5 = CLR
$d = 2.25''$

reinforcement:
(15) #5 (6) #6

$A_s = 15(0.31) + 6(0.44) = 7.29 \text{ in}^2$

$a = \dfrac{A_s f_y}{0.85 f'_c b} = \dfrac{7.29(60)}{0.85(5)(176.16)} = 0.584$

$c = \dfrac{a}{\beta_1} = \dfrac{0.584}{0.8} = 0.730$

$\varepsilon_y = \dfrac{\varepsilon_c}{c}(d-c)$

$\varepsilon_y = \dfrac{0.003}{0.730}(2.25 - 0.730) = 0.0062 > 0.005$

TENSION CONTROLLED $\rightarrow \phi = 0.9$

$\phi M_n = \phi A_s f_y (d - a/2)$

$\phi M_n = 0.9(7.29)(60)(2.25 - \frac{0.584}{2}) = 802.51 \text{ in K}$

$\phi M_n = 66.9 \text{ ft K}$

$\boxed{\phi M_n > M^-_{column\ strip}}$ ✓ OKAY

4/8

spot checks
Slab (cont)

positive moment reinforcement

#5 @ 12" oc.

$\frac{173.16}{12} = 15$ bars

$A_s = 15(0.31) = 4.65\ in^2$

$a = \frac{4.65(60)}{0.85(5)(173.16)} = 0.373$

$c = \frac{0.373}{\beta_1} = \frac{0.373}{0.8} = 0.466$

$\varepsilon_y = \frac{0.003}{0.466}(2 - 0.466) = 0.011$

$\varepsilon_y = 0.011 > 0.005 \rightarrow$ TENSION CONTROLLED $\phi = 0.9$

$\phi M_n = A_s f_y (d - a/2)$

$= 4.65(60)(2.25 - \frac{0.373}{2}) = 575.72$ ink

$\phi M_n = 0.9(575.72) = 43.24$ K

$\boxed{\phi M_n > M^+_{\text{COLUMN STRIP}}}$ ✓ OKAY

MIDDLE STRIP:

b = 173.16"
h = 4"
d = 2.25"
$f'_c = 5$ ksi
$f_y = 60$ ksi

Negative moment reinf.

#5@12 o'c.

$A_s = (14)\#5 = (14)(0.31) = 4.34\ in^2$

$a = \frac{4.34(60)}{0.85(5)(176.16)} = 0.348$

5/8

Spot checks
Slab (cont)

$C = \frac{0.348}{0.8} = 0.435$

$\varepsilon_y = \frac{0.003}{0.435}(2.25 - 0.435) = 0.0125 > 0.005$

tension controlled $\rightarrow \phi = 0.9$

$\phi M_n = 0.9(4.34)(60)(2.25 - 0.348/2)$
$= 486.5$ in K

$\phi M_n = 40$ ft K

$\boxed{\phi M_n \geq M_{middle}}$ √ OKAY

Positive moment reinf.

#5 @ 12 oc – 14 bars

$A_s = 14(0.31) = 4.34 \text{ in}^2$

$a = \frac{4.34(60)}{0.85(5)(176.16)} = 0.348$

$c = 0.435$

$\varepsilon_y = \frac{0.003}{0.435}(2.25 - 0.435) = 0.0125 > 0.005$

tension controlled $\rightarrow \phi = 0.9$

$\phi M_n = 0.9(4.34)(60)(2.25 - 0.348/2) = 486.5$ in K

$\phi M_n = 40$ ft K

$\boxed{\phi M_n \geq M^+_{middle}}$ √ OKAY

* strength of column strip and middle strip selected exceeds required strength per ACI 318-05 Chapter 13.

$\boxed{\text{SLAB IS OKAY IN FLEXURE}}$

6/8

Spot checks
Slab (cont)

- Slab deflection:

longest span is between columns 2A & 3A

$l_n = 11.42'$

Min slab thickness per ACI 318-08 13.1.4

- table 9.5(c)

$t > l_n/36$

$t >= \frac{11.42(12)}{36} = \frac{137.04}{36}$

$\boxed{t > 3.81 \text{ in}}$

actual slab thickness = 4"

* slab thickness exceeds that required by 9.5.3

* No need to check deflections

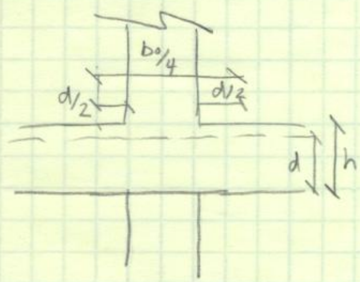
7/8

Spot checks
Slab (cont)

- Punching shear:

Shear per ACI 318-08 chapter 13
↳ provisions of chapter 11 shall apply

- column A3



24" x 24" w/ 4" slab
$f'_c = 5$ ksi
#5 @ 12" ON BOTH DIRECTIONS

$V_c = 4\sqrt{f'_c}\, b_o d$ (eqn. 13.11a)

$d = 4" - \frac{1}{2}" - \frac{1}{2}(0.625) = 3.188$

$\frac{b_o}{4} = 2\left(\frac{d}{2}\right) + 24 = 2\left(\frac{3.188}{2}\right) + 24 = 27.19"$

↳ $b_o = 108.75"$

$$\phi V_c = \min \left| \begin{array}{l} \left(2 + \frac{4}{\beta}\right)\sqrt{f'_c}\, b_o d \\ \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d \\ 4\sqrt{f'_c}\, b_o d \end{array} \right. \quad (11.12.2.1)$$

$\beta = \frac{24}{24} = 1$

$\alpha_s = 40$ for interior columns

$\phi\left(2 + \frac{4}{1}\right)\sqrt{5000}(108.75)(3.19) = 142.7^k(0.75)$
$= 110.4^k$

$\phi\left(\frac{(40)(3.19)}{108.75} + 2\right)\sqrt{5000}(108.75)(3.19)$
$= 77.84^k$ ⟵ GOVERNS $\boxed{\phi V_c = 77.84^k}$

$\phi\, 4\sqrt{5000}(108.75)(3.19)$
$= 98.12^k$

8/8

spot checks
slab (cont.)

Trib. area of column 3A

$$A_{TRIB} = \left(\frac{10'5''}{2} + \frac{13'5''}{2}\right)\left(\frac{26'}{2} + \frac{26'\text{-}7\tfrac{3}{8}''}{2}\right) - \frac{24 \times 24}{144}$$

$$A_{TRIB} = 309.97 \text{ ft}^2$$

$q_u = 238$ psf

$V_u = 309.97(238)$
$= 73773$ lbs
$= 73.77^k$

$\boxed{\phi V_c > V_u}$ ✓OKAY

Beam action shear

$b_w = 176.16'' \times 2 = 352.32$ in

$V_c = 2\sqrt{f'c}\, b_w d$
$= 2\sqrt{5000}\,(352.32)(3.188)$

$V_c = 158.84$

$\phi V_c = 119.13^k$

$\rightarrow \boxed{\phi V_c > V_u}$ ✓OKAY