Cambria Suites Hotel

Pittsburgh, PA

Technical Report 1

Structural Concepts & Existing Conditions

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Executive Summary

The following technical report describes the structural concepts, as well as, the existing structural conditions of the Cambria Suites Hotel to better understand the structural design of the building. Along with this summary, an analysis of the structural system is provided for gravity and lateral loading through the use of detailed descriptions and diagrams. The analysis includes dead, live, snow, wind, and seismic loading. An identification of the structural materials and design codes used for the actual design will also be within the report. Building plans and detailed computations performed will be provided in an Appendix at the end of the report.

Cambria Suites is a 7-story, all-suite hotel located in Pittsburgh, PA. This luxury hotel is located directly adjacent to the new CONSOL Energy Center, home of the Pittsburgh Penguins, and will accommodate 142 guests and also offer a state-of-the-art fitness center and pool. The hotel is approximately 120,000 square feet and reaches a building height of 86'-10" above the Plaza level (102'-2" to the High Roof level). The typical floor system is 10" precast hollow concrete plank floors while the ground floor is a reinforced concrete slab on grade. The typical floor height is 10'-0" and the lobby entrance extends to 22'-0". The foundation is designed as a combination of drilled cast-in-place caissons which are combined with grade beams which transfer all loads to bedrock. The gravity system consists of an integrated reinforced concrete masonry wall and steel beam/column structure. The lateral resisting system is comprised of reinforced concrete masonry shear walls.

To fully understand the lateral system of the structure, an analysis of the wind and seismic loads was done according to ASCE 7-05. The ASCE 7-05 was also the code referenced by the structural engineers at Atlantic Engineering Services (AES), in the design of Cambria Suites. The wind in the North/South direction was found to control of the East/West wind direction due to the longer façade perpendicular to the North/South direction. Seismic loads were examined using the Equivalent Lateral Force Procedure and it was found that the base shear due to seismic loads was slightly larger than the wind load base shear. Thus, seismic loads will control when determining lateral force on the structure.

Spot checks were performed for various floor framing elements within gravity load areas of the structure. Spot checks were performed on an interior gravity column, beam, and P.C. plank floor to validate the member sizes chosen by AES professionals. All members were found to be adequately designed. Any overdesigns were a result of only considering gravity loads and not taking into account the lateral forces that are in contact with the building.

Introduction: Cambria Suites Hotel

Cambria Suites Hotel is the newest, upscale, contemporary all-suite hotel located at 1320 Center Avenue in Pittsburgh, Pennsylvania. This luxury hotel is built adjacent to the new CONSOL Energy Center, home to the Pittsburgh Penguins hockey team, and numerous concerts and special events. The 142-suite hotel contains a total of 7 levels above grade and was built on a quite challenging site. The hotel will have a variety of room suites, such as the double/queen suite, king suite, one bedroom suite, deluxe tower king suite, and hospitality suite.

The Plaza Floor level will mainly consist of a few bistro-style restaurants which open to an outdoor terrace which will overlook the city of Pittsburgh and the CONSOL Energy Center. At the Hotel Floor level, guests will be greeted by an airy two-story lobby where they can take part in a state-of-the-art fitness center or the relaxing indoor pool and spa. There are also two meeting rooms and a board room for guest use, as well as, a large kitchen/bar off of the lobby entrance. At the North end of the Second Floor level, a steel Porte Cochere will be cantilevered to cover part of the main entrance. In addition, the property will feature an 1800 square foot presidential suite with one of a kind skyline view of downtown Pittsburgh and a 7^{th} floor concierge lounge that will offer a wet bar and lounge space for guests to use and enjoy.

The hotel is fully landscaped and will also have an exclusive 143 space onsite parking garage with access to the CONSOL Energy Center for event patrons staying at the property. The Hotel Floor level will have a precast concrete pedestrian bridge leading to the top level of the parking garage. The bridge is supported by the hotel and the garage. The South end of the bridge will be supported by the garage on slide-bearings to allow for differential lateral movement between the two structures. The exterior of Cambria Suites Hotel is mainly brick and cast-stone veneer, architectural decisions made to resemble the bordering CONSOL Energy Center. A lighter color brick is used from the 2^{nd}-Roof Floor levels, with the addition of a cast-stone band at Floor levels 2 and 7. The darker color brick is used from the 2^{nd} Floor level and below, as well as vertical strips to separate window pairings and to accent building corners.

The following report will take a closer look into the structural concepts and existing conditions Cambria Suites' structural system. To get a better understanding of how the structural components work, a detailed description of the foundation, floor system, gravity system, and lateral system is provided.

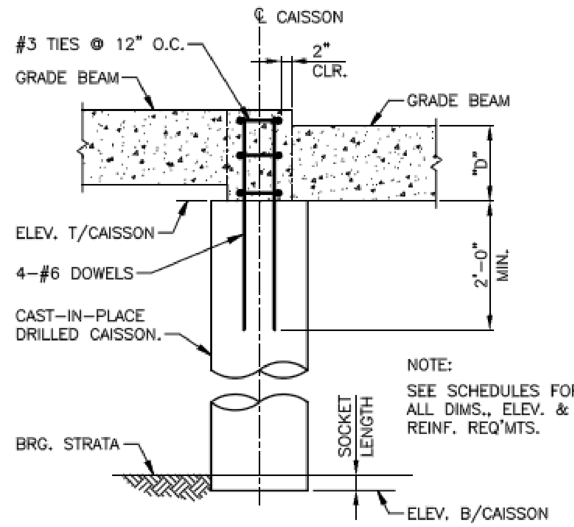
Structural System

Foundation

The geotechnical engineering study for the Cambria Suites Hotel was completed by GeoMechanics Inc. on December 29, 2008. In the study, the site of Cambria Suites Hotel is underlain by sedimentary rocks of the Conemaugh group of rocks of Pennsylvania age. The Conemaugh group is predominantly comprised of clay stones and sandstones interbedded with thin limestone units and thin coal beds. The soil zone conditions consisted of materials of three distinct geologic origins: man-made fill, alluvial deposits, and residual soils. The fill in the hotel test borings was placed in conjunction with the recent demolition and regarding of St. Francis Hospital in order to build Cambria Suites and CONSOL Energy Center. Ground water exists locally as a series of perched water tables located throughout the sol zone and new the upper bedrock surface. Excavations in soils and bedrock can be expected to encounter perched water. The volume of inflow into excavations should be relatively minor, should diminish with time and should be able to be removed by standard pump collection/pumping techniques. The report also states that the most economical deep foundation solution for Cambria Suites included a system of drilled-in, cast-in-place concrete caissons with grade beams spanning between adjacent caissons to support the anticipated column and wall loads of the structure. With varying types of bedrock on site, the allowable end bearing pressure ranges from 8, 16, and 30 KSF. As for the floor slab, GeoMechanics Inc. recommended to place a ground floor slab on a minimum six-inch thick granular base and to provide expansion joints between the ground floor slab and any foundation walls and/or columns. This is done to permit independent movement of the two support systems.

As a result of GeoMechanic's geotechnical study, the foundation of Cambria Suites Hotel incorporates a drilled cast-in-place concrete caissons and grade beams designed to support the load bearing walls and columns. The ground floor is comprised of a 4" concrete slab on grade, as well as, 10" precast concrete plank in the Southern portion of the building. The 4" concrete slab on grade is reinforced with 6x6-W1.4 welded wire fabric and has 4000 PSI normal weight concrete. The slab increases to 8" in thickness with #5 @ 16" O.C. in the South-West corner of the building, and increases to 24" with #5 @ 12" O.C. within the core shear walls where the elevator shaft is located. For the majority of the slab on grade, the slab depth is 14'-0" below finish grade.

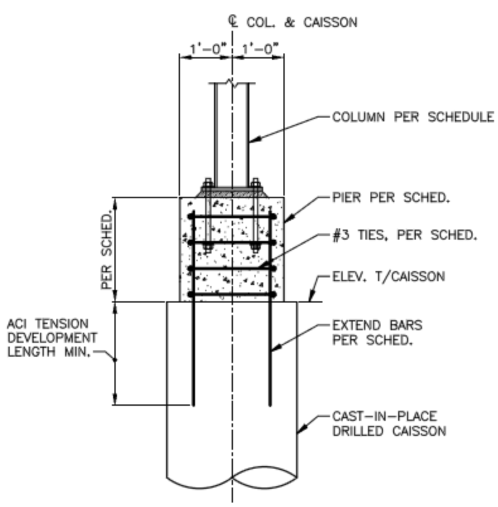
The drilled cast-in-place caissons extend anywhere from 20-30 feet deep below grade and are socketed at least 3' into sound bedrock. Caisson end bearing capacity is 30 KSF (15 ton/SF) on Birmingham Sandstone bedrock. The caissons are designed with a compressive strength of 4000 PSI, range from 30-42 inches in diameter, and are spaced approximately between 15' and 30' apart (refer to Appendix A). Typical caissons terminate at the Plaza level and are tied into a grade beam with #3 ties @ 12" O.C. (horizontal reinforcement) and 4-#6 dowels (vertical reinforcement) embedded at least two feet into the drilled caisson. Where steel columns are located, a pier is poured integrally with the grade beam and reinforced with 8-#8 vertical bars and #3 @ 8" O.C. horizontal ties. (As shown in Figures 1.1 & 1.2)



Typical Caisson & Grade Beam Detail

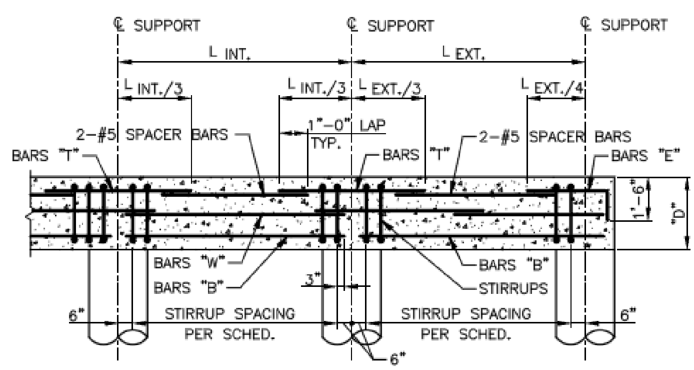
Figure 1.1

The grade beams have a compressive strength of 3000 PSI and range from 30-48 inches in width and 36-48 inches in depth. Each grade beam is reinforced with top and bottom bars which vary according to the size of the beam. Grade beams span between drilled cast-in-place caissons which transfer the loads from bearing walls, shear walls, and columns into the caissons. From the caissons, the loads are then transferred to bedrock. (As shown in Figures 1.1 & 1.3)



Typical Caisson Cap Detail

Figure 1.2



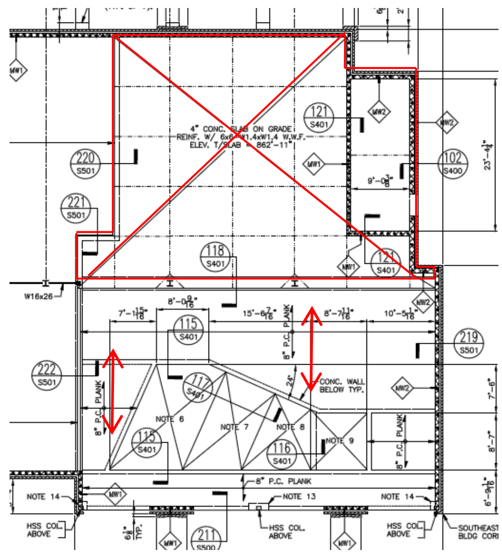
Typical Grade Beam Reinforcing Detail

Figure 1.3

Superstructure System

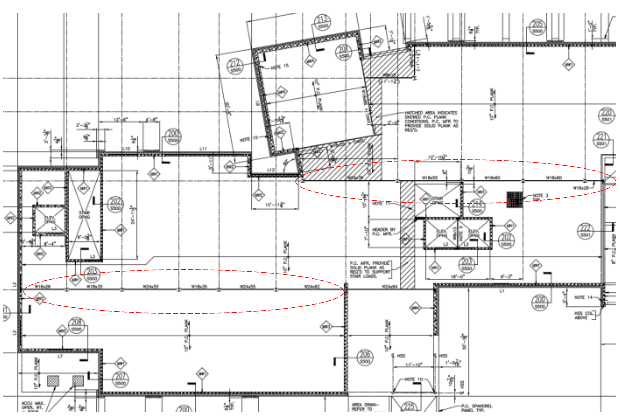
The typical floor system of Cambria Suites Hotel consists of 10" precast hollow-core concrete plank with 1" leveling topping. The precast plank allowed for quicker erection, longer spans, open interior spaces, and serves as an immediate work deck for other trades. Concrete compressive strength for precast plank floors is 5000 PSI and uses normal weight concrete. The typical spans of the plank floors range from 30'-0" to 40'-0". The floor system is supported by exterior load bearing concrete masonry walls, as well as, interior steel beams and columns.

The Plaza level floor system is a combination of 10" precast concrete plank, 8" precast concrete plank and 4" slab on grade. Since there is no basement in the North-East section of the hotel due to the fitness center and pool, the site was excavated properly in order to place the 4" slab on grade and 8" precast concrete plank. The 4" slab on grade will be for the fitness center where as the 8" concrete plank will surround the pool area. (As shown in Figure 2.1)



Partial Hotel Level Floor Slab
Figure 2.1

Since the masonry bearing walls are typically located on the perimeter of the hotel, steel beams and columns were needed thru the center of the building to support the precast concrete plank floors. Steel beam sizes range from W16x26 to W24x94, and steel column sizes range from W8x58 to W18x175. Each column connects into concrete piers within the grade beams via base plates which vary in size. Base plates use either a 4-bolt or 8-bolt connection, typically using 1" A325 anchor bolts which extend 12" or 18" respectively into the concrete pier. The steel beams vary in length from 13'-0" to 19'-0" and typically span in the East-West direction. Exterior bearing masonry walls and the steel beams will take a reaction load from the precast concrete plank flooring, as well as other loads from levels above, which will then transfer thru steel columns and exterior bearing walls and thus transferring all loads to the foundation system. (As shown in Figure 2.2)



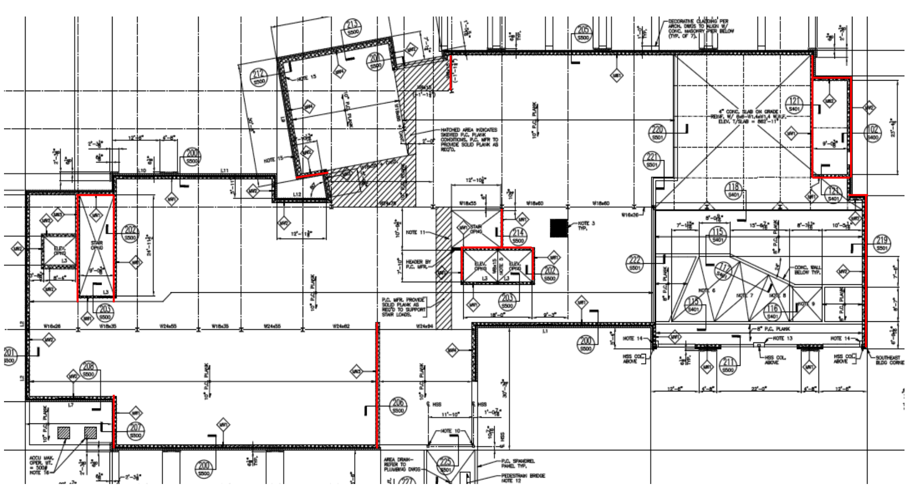
Typical Partial Floor Plan
Figure 2.2

The roof structural system at both the Second level and main Roof level uses untopped 10" precast concrete plank. Reinforced concrete masonry extends passed the Roof level to support a light gauge cornice which wraps the entire building. A high roof is constructed for hotel identification purposes and uses 10"-16 GA light gauge roof joists @ 16" O.C., supported by 8"-20 GA light gauge wall framing below. W8x21 hoist beams support the top of the elevator shaft which rest on ½"x7"x7" base plates. There are a total of eight drains located on the roof for the drainage system. (As shown in Appendix A)

Lateral System

The lateral system for the Cambria Suites hotel consists of reinforced concrete masonry shear walls. The exterior shear walls, as well as the core interior shear walls, are constructed of 8" concrete masonry, with the exception of a few 12" concrete masonry walls on the lower floor levels. All shear walls are solid concrete masonry walls which extend the entire height of the structure without openings for windows or doors. The core shear walls are located around the staircases and elevator shafts. The exterior shear walls are scattered around the perimeter of the building. (As shown in Figure 3.1) Shear walls supporting the Plaza level to the Third Floor level have a compressive strength of 2000 PSI. All other shear walls support a compressive strength of 1500 PSI. In addition, all concrete masonry shall be grouted with a minimum compressive strength of 3000 PSI. Typical reinforcement for all shear walls is comprised with #5 bars at either 8" O.C. or 24" O.C.

Wind and seismic loads, as well as gravity loads, are transferred to the foundation by first traveling thru the rigid diaphragm; the precast concrete plank floor system. Loads are then transferred to the concrete masonry shear walls. From there, loads are transferred down to the preceding floor system and/or transferred the entire way to the grade beam foundation, finally travelling thru the concrete caissons which are embedded in bedrock.



Lateral Shear Wall System

Figure 3.1

Codes and Requirements

- International Building Code (IBC), 2006
  (As amended by the City of Pittsburgh)

- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05), American Society of Civil Engineers

- Building Code Requirements for Structural Concrete (ACI 318-05), American Concrete Institute
- Specifications for Structural Concrete (ACI 301-05), American Concrete Institute

- PCI Design Handbook – Precast/Prestressed Concrete Institute

- The Building Code Requirements for Masonry Structures (ACI 530), American Concrete Institute

- Specifications for Masonry Structures (ACI 530.1), American Concrete Institute

- Specifications for Structural Steel Buildings – Allowable Stress Design and Plastic Design (AISC), American Institute of Steel Construction

Materials

Reinforced Concrete

Caissons & Piers	f'_c = 4000 PSI
Grade Beam Foundations	f'_c = 3000 PSI
Slabs on Grade	f'_c = 4000 PSI
Walls	f'_c = 4000 PSI
Exterior Bar or Wire Reinforcement Slabs	f'_c = 5000 PSI

Reinforcement Steel

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

Structural Steel

Structural W Shapes	ASTM A992
Channels	ASTM A572, Grade 50
Steel Tubes (HSS Shapes)	ASTM A500, Grade B
Steel Pipe (Round HSSS)	ASTM A500, Grade B
Angles & Plates	ASTM A36
Structural Shapes & Rods	ASTM A123
Bolts, Fasteners, & Hardware	ASTM A153

Masonry

8” & 12” CMU	f'_m = 2000 PSI
Grout	f'_c = 3000 PSI

Design Load Summary

Live Loads (LL)			
Area	**AES Design Load (PSF)**	**ASCE 7-05 Load (PSF)**	**Design Load (PSF)**
Public Areas	100	100	100
Lobbies	100	100	100
First Floor Corridors	100	100	100
Corridors above First Floor	40	40	40
Private Hotel Rooms	40	40	40
Partitions	15	≥15	15
Mechanical	150	150	150
Stairs	100	100	100
Roof	20	20	20
Dead Loads (DL)			
Material	**AES Design Load (PSF)**	**ASCE 7-05 Load (PSF)**	**Design Load (PSF)**
10" Concrete Plank	Unknown	Section 3.1	91
8" Masonry Wall (Fully Grouted)	Unknown		91
8" Masonry Wall (Partially Grouted w/ Reinf. @ 24" O.C.)	Unknown		69
8" Masonry Wall (Partially Grouted w/ Reinf. @ 48" O.C.)	Unknown		60
Steel	Unknown		varies
Partitions	Unknown		15
MEP	Unknown		10
Finishes & Miscellaneous	Unknown		5
Roof	Unknown		20
***Snow Load (SL)**			
Area	**AES Design Load (PSF)**	**ASCE 7-05 Load (PSF)**	**Design Load (PSF)**
Flat Roof	21	21	21
*Refer to Appendix B for Snow Analysis			

Lateral Loads

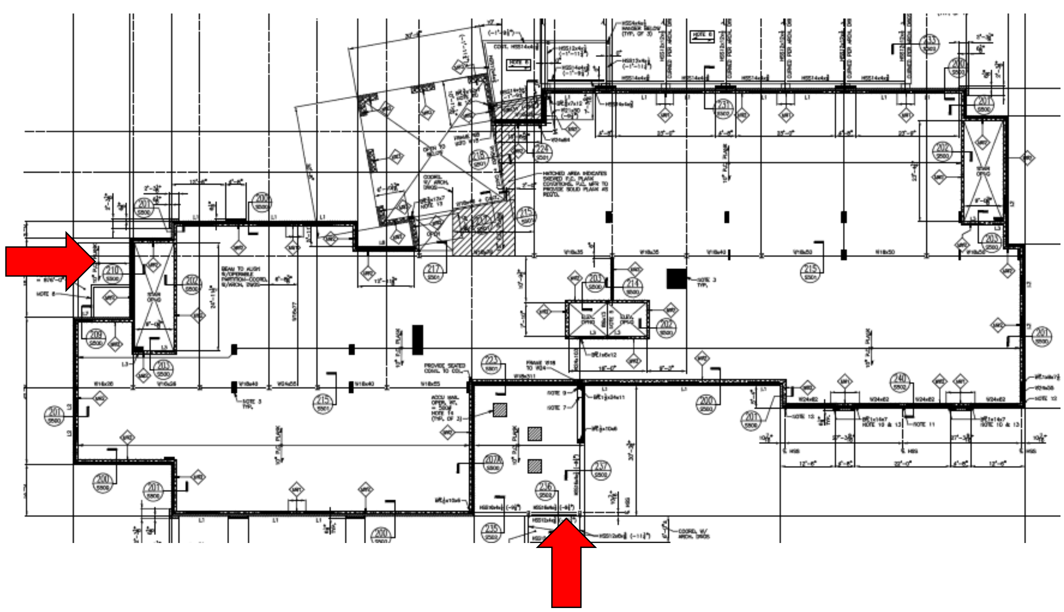
Wind Analysis

In the following wind analysis, wind loads were determined according to ASCE 7-05, Chapter 6. This is the same code that Atlantic Engineering Services referenced when calculation the wind loads. Since the overall building height of Cambria Suites hotel reaches 86’-10” (High Roof extends to 102’-2”), it is required to determine the wind loads through the use of Section 6.5: Method 2 – Analytical Procedure because it exceeds the 60’-0” maximum building height stated in Section 6.4: Method 1 – Simplified Procedure. The wind variables used during this analysis to calculate the design wind pressures are located in Table 1a. For detailed equations and base calculations used for this procedure, refer to Appendix C. The North/South and East/West wind directions are labeled on the typical floor plan in Figure 4.1.

<u>Wind Directions</u>

Figure 4.1

E/W Wind Direction



N/S Wind Direction

Table 1a: Wind Variables

Wind Variables				ASCE Reference
Basic Wind Speed	V	90 mph		Fig. 6-1
Directional Factor	K_d	0.85		Table 6-4
Importance Factor	I	1.0		Table 6-1
Occupancy Category		II		Table 1-1
Exposure Category		B		Sec. 6.5.6.3
Enclosure Classification		Enclosed		Sec. 6.5.9
Building Natural Frequency	n_1	1.47 (Rigid)		Eq. C6-19
Topographic Factor	K_{zt}	1.0		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 Pressure at Mean Roof Height	q_h	17.1		Eq. 6-15
Gust Effect Factor	G	0.85		Sec. 6.5.8.1
Product of Internal Pressure Coefficient and Gust Effect Factor	GC_{pi}	0.18		Fig. 6-5
		-0.18		
External Pressure Coefficient (Windward)	C_p	0.80	(All Values)	Fig. 6-6
External Pressure Coefficient (Leeward)	C_p	-0.5	(N/S Direction, L/B = 0.45)	
		-0.2	(E/W Direction, L/B = 2.22)	

***Equation C6 – 19:**

$f_{n1} = (150/H)$ where H = building height (ft.)

$f_{n1} = (150/102.167) = 1.47 \geq 1$ Hz $\therefore$ The building is considered rigid

The wind pressures in the North/South direction were determined and are located in the following table, (Table 1b). This wind direction is of more concern since the wind contacts a building length of 219’-8”, compared to 98’-11” in the East/West direction. The direction of wind is adjacent to a road that services the front of hotel, and a parking garage that does not extend passed the Hotel level of Cambria Suites. Neither obstruction from the front or back of the hotel will cause a significant wind load blockage to the structure. An elevation view of the hotel is provided in Figure 4.2 which shows the wind loads of the windward and leeward pressures at each level, as well as the base shear. A basic loading diagram is also provided in Figure 4.4 to show wind loads and story shears.

Table 1b: North/South Wind Loads

Wind Loads (North/South Direction)													
B = 219'-8" L = 98'-11"													
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							
High Roof	102.167	15.333	1.00	17.6	15.0	-10.35	25.4	6.90	11.68	6.90	11.68	652.05	1103.77
Roof	86.833	10	0.95	16.7	14.4	-10.35	24.8	31.63	54.48	38.53	66.16	2588.38	4458.26
7	76.833	10	0.92	16.2	14.1	-10.35	24.5	30.97	53.82	69.50	119.98	2224.67	3866.05
6	66.833	10	0.88	15.5	13.6	-10.35	24.0	29.88	52.72	99.38	172.7	1847.57	3259.84
5	56.833	10	0.84	14.8	13.1	-10.35	23.5	28.78	51.62	128.16	224.32	1491.75	2675.62
4	46.833	10	0.79	13.9	12.5	-10.35	22.9	27.46	50.30	155.62	274.62	1148.73	2104.20
3	36.833	10	0.74	13.0	11.9	-10.35	22.3	26.14	48.99	181.76	323.61	832.11	1559.50
2	26.833	10	0.63	11.1	10.6	-10.35	21.0	23.28	46.13	205.04	369.74	508.27	1007.16
1	14.833	12	0.56	9.87	9.79	-10.35	20.1	25.81	52.98	230.85	422.72	227.98	467.97
B	0	14.833	0	0	0	0	0	0	0	230.85	422.72	0	0

Σ Windward Story Shear =	230.85	kips
Σ Total Story Shear =	422.72	kips
Σ Windward Moment =	11521.52	ft-k
Σ Total Moment =	20502.36	ft-k

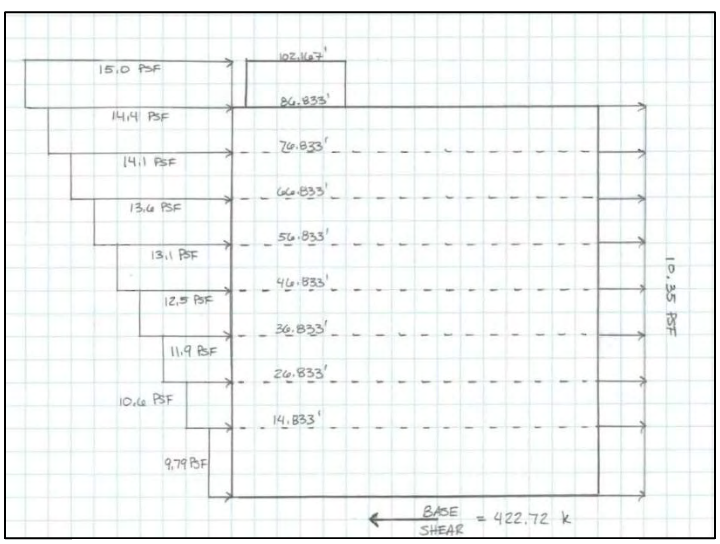


Figure 4.2

North/South Wind Pressures

The wind pressures in the East/West direction were determined and are located in the following table, (Table 1c). Since there are buildings adjacent to Cambria Suites on both the East and West side, wind blockage can have an effect on the full wind loading for the structure. However, wind loading in this direction must be examined as if these surrounding buildings were not present. An elevation view of the hotel is provided in Figure 4.3 which shows the wind loads of the windward and leeward pressures at each level, as well as the base shear. A basic loading diagram is also provided in Figure 4.5 to show wind loads and story shears.

Table 1c: East/West Wind Loads

Wind Loads (East/West Direction)													
B = 98'-11" L = 219'-8"													
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							
High Roof	102.167	15.333	1.00	17.6	15.0	-5.98	21.0	6.90	9.660	6.90	9.660	652.05	912.87
Roof	86.833	10	0.95	16.7	14.4	-5.98	20.4	14.24	20.18	21.14	29.84	1165.30	1651.39
7	76.833	10	0.92	16.2	14.1	-5.98	20.1	13.95	19.88	35.09	49.72	1002.07	1428.04
6	66.833	10	0.88	15.5	13.6	-5.98	19.6	13.45	19.39	48.54	69.11	831.65	1198.94
5	56.833	10	0.84	14.8	13.1	-5.98	19.1	12.96	18.89	61.50	88.0	671.76	979.13
4	46.833	10	0.79	13.9	12.5	-5.98	18.5	12.36	18.30	73.86	106.3	517.06	765.54
3	36.833	10	0.74	13.0	11.9	-5.98	17.9	11.77	17.71	85.57	124.01	374.67	563.76
2	26.833	10	0.63	11.1	10.6	-5.98	16.6	10.49	16.42	96.06	140.43	229.03	358.50
1	14.833	12	0.56	9.87	9.79	-5.98	15.8	11.62	18.75	107.68	159.18	102.64	165.62
B	0	14.833	0	0	0	0	0	0	0	107.68	159.18	0	0

Σ Windward Story Shear =	107.68	kips
Σ Total Story Shear =	159.18	kips
Σ Windward Moment =	5546.23	ft-k
Σ Total Moment =	8023.79	ft-k

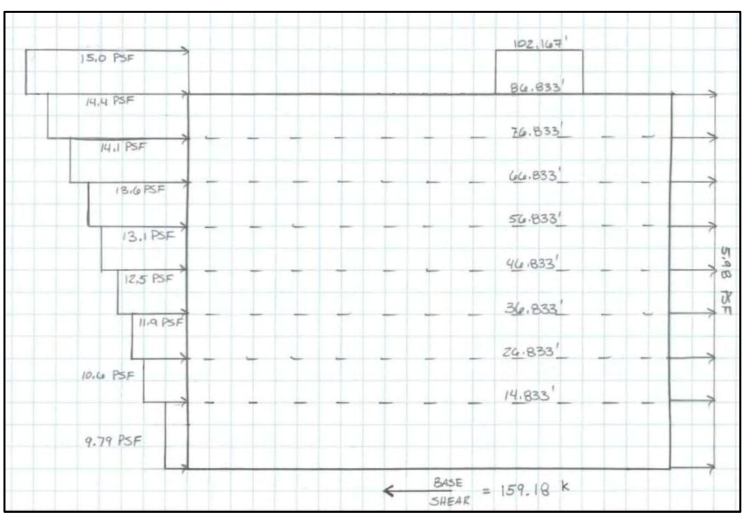


Figure 4.3

East/West Wind Pressures

Wind Load Diagrams

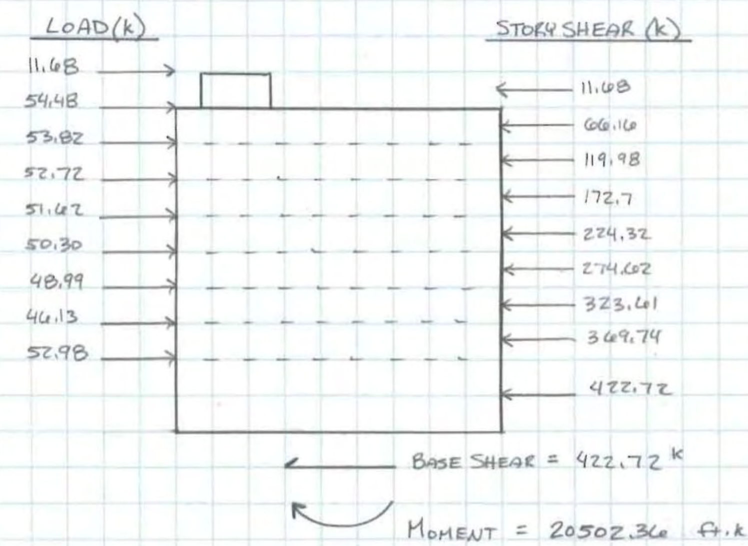


Figure 4.4
North/South Wind Loading Diagram

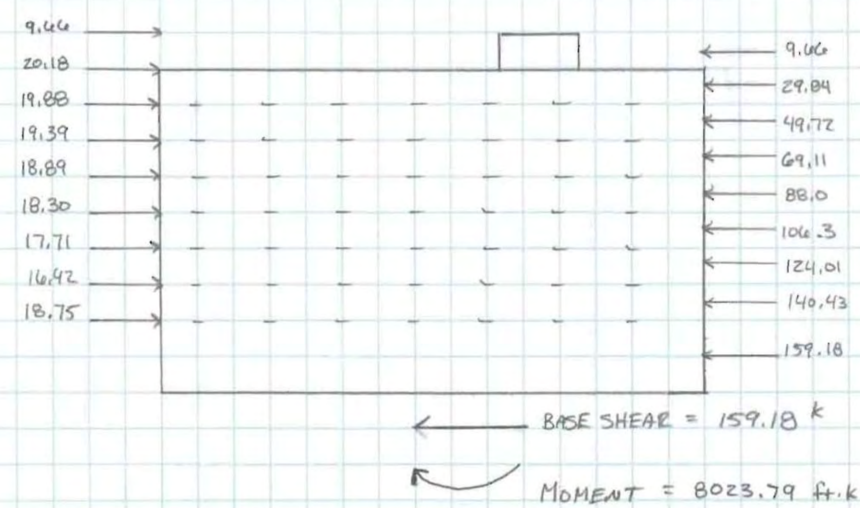


Figure 4.5
East/West Wind Loading Diagram

Note: Seismic Loads are the same in both North/South and East/West direction because the structural type is the same in both directions (Table 12.8-2)

Seismic Analysis

In the following seismic analysis, seismic loads were determined according to ASCE 7-05, Chapters 11 and 12. As identified in Section 1613 of the International Building Code (IBC), Cambria Suites Hotel is to be designed and constructed to resist the effects of earthquake motions. According to IBC 2006 criteria, Site Class for Seismic Design of "C" should be used for existing conditions. Other variables used in this analysis that are needed to calculate base shear and overturning moments, according to ASCE 7-05, are located in Table 2a.

Table 2a

Seismic Design Variables				ASCE References
Site Class		C		Table 20.3-1
Occupancy Category		II		Table 1-1
Importance Factor		1.0		Table 11.5-1
Structural System		Ordinary Reinforced Masonry Shear Walls		Table 12.2-1
Spectral Response Acceleration, short	S_s	0.125		Fig. 22-1 thru 22-14
Spectral Response Acceleration, 1 s	S_1	0.049		Fig. 22-1 thru 22-15
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}	0.15		Eq. 11.4-1
MCE Spectral Response Acceeration, 1 s	S_{m1}	0.0833		Eq. 11.4-2
Design Spectral Acceleration, short	S_{ds}	0.100		Eq. 11.4-3
Design Spectral Acceleration, 1 s	S_{d1}	0.055		Eq. 11.4-4
Seismic Design Category	S_{dc}	A		Table 11.6-2
Response Modification Coefficient	R	2.0		Table 12.2-1
Building Height (above grade)(ft)	h_n	102.167		
		North/South	East/West	
Approximate Period Parameter	C_t	0.02	0.02	Table 12.8-2
Approximate Period Parameter	x	0.75	0.75	Table 12.8-2
Calculated Period Upper Limit Coefficient	C_u	1.7	1.7	Table 12.8-1
Approximate Fundamental Period	T_a	0.643	0.643	Eq. 12.8-7
Fundamental Period	T	1.09	1.09	Sec. 12.8.2
Long Period Transition Period	T_L	12	12	Fig. 22-15
Seismic Respose Coefficient	C_s	0.025	0.025	Eq. 12.8-2
Structural Period Exponent	k	1.295	1.295	Sec. 12.8.3

Note: Seismic Loads are the same in both North/South and East/West direction because the structural type is the same in both directions (Table 12.8-2)

An Excel spread sheet was created to determine the story weight of each individual floor (above grade), as well as the total building weight. Using the story weight values, the base shear and overturning moments due to seismic loads were also determined. Please refer to Appendix D for detailed Excel spread sheet calculations used to determine the building weight, as well as, the base shear and overturning moments at each story level provided in Table 2b. In addition, a seismic loading diagram was generated to show the story forces and story shears at each level. (As shown in Figure 5.1)

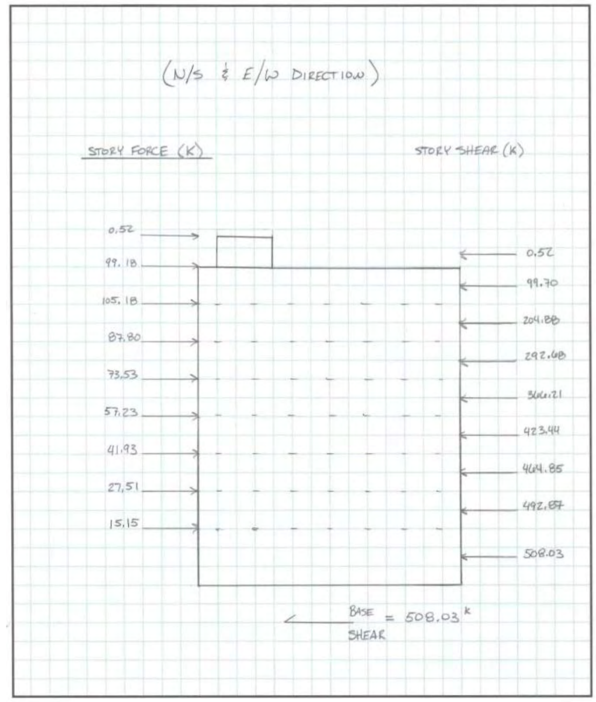


Figure 5.1
Seismic Loading Diagram

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)
High Roof	102.167	7.92	3168	0.001	0.52	0.52	48.78
Roof	86.833	1878.42	608681	0.195	99.18	99.70	8116.49
7	76.833	2333.98	645478	0.207	105.18	204.88	7555.37
6	66.833	2333.98	538841	0.173	87.80	292.68	5429.14
5	56.833	2410.92	451222	0.145	73.53	366.21	3811.07
4	46.833	2410.92	351194	0.113	57.23	423.44	2393.96
3	36.833	2410.92	257312	0.083	41.93	464.85	1334.71
2	26.833	2383.72	168804	0.054	27.51	492.87	573.04
1	14.833	2829.80	93003	0.030	15.15	508.03	112.39
B	0	1320.07	0	0	0	508.03	0
			3117703				

Total Building Weight =	20321	k
Base Shear =	508.03	k
Total Moment =	29374.97	ft-k

Note: Since the basement level is mainly above grade (≈75%), the building weight of this level is included in the seismic analysis.

Spot Checks

Spot checks were performed on a few different floor framing elements to confirm AES's structural design. The spot checks performed are show in the Figures 5.1. Make note that all spot checks are done considering applied gravity loads only. Therefore, slight variations in calculations may occur due to the fact that lateral loads are present and will affect the overall design of the structural members. For detailed calculations of the spot checks, please refer to Appendix E.

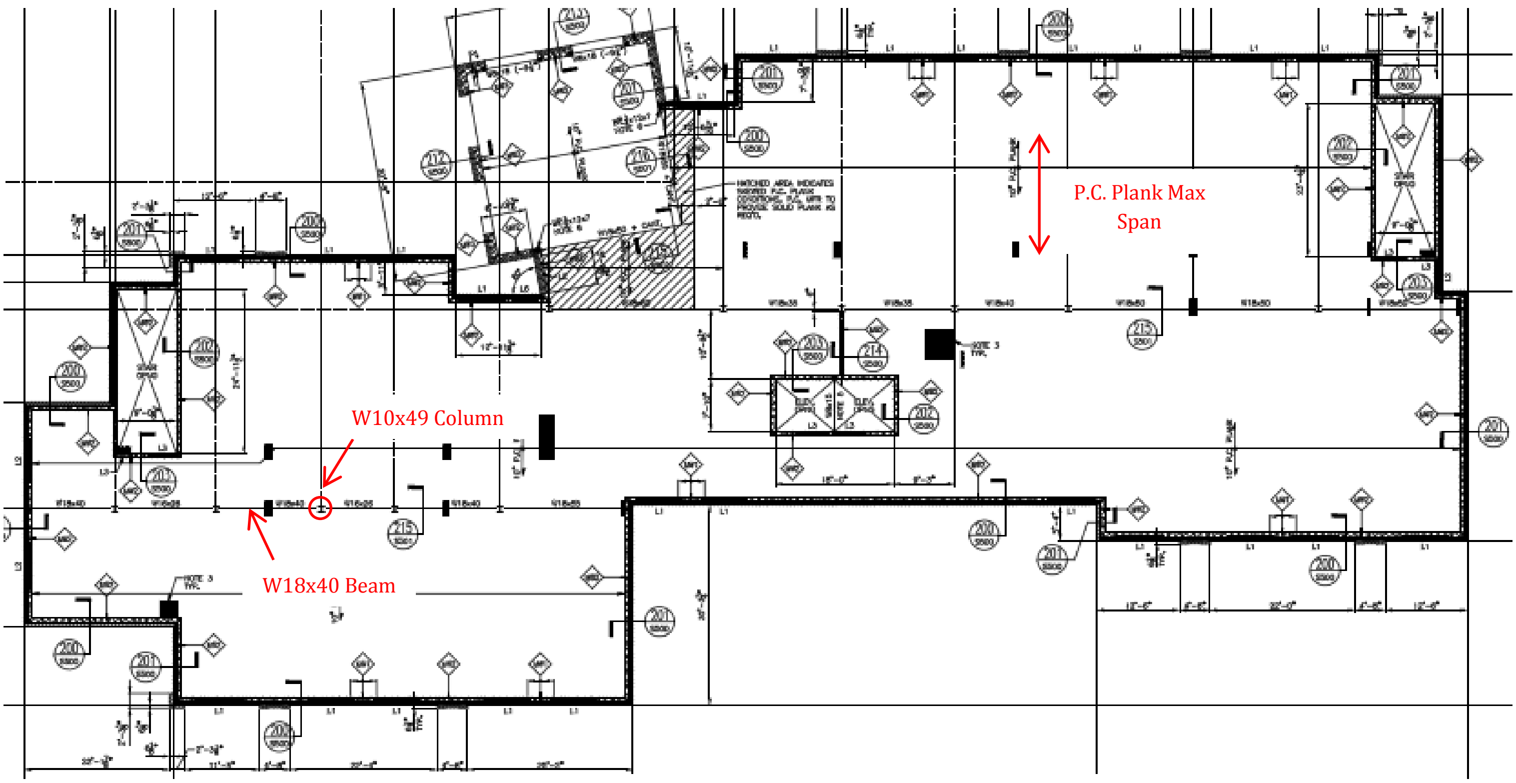


Figure 5.1
Spot Check Locations for Interior Column, Beam, and P.C. Hollow Core Plank

A spot check of an interior steel column (C-3) on the Fifth Level was performed using the floor weights calculated from the seismic analysis, as well as, the live loads present at each floor. Tributary area was also considered for the interior column and a live load reduction was taken into account on all reducible live loads (Refer to Table 3a). A detailed summary of the accumulated floor load on the column at each level is provided in Appendix E. Since the column being referenced has an un-braced length of 30' in the x-axis and a braced length of 10' in the y-axis, the effective length, $KL_{eff,}$ was necessary to calculate. Axial forces due to gravity loads only were applied to each column and no moments or additional forces due to lateral loads were taken into account for this study.

Table 3a

Column Loads								
Level Supported	Tributary Area (SF)	Dead Load (PSF)	Live Load (PSF)	LL Reduction (PSF)	Dead Load (k)	Live Load (k)	Total Load (1.2D+1.6L) kips	Accumulated Load (k)
Roof	455.05	124.29	20	12.04	56.6	9.1	82.4	82.4
7	455.05	154.44	55	33.11	70.3	25.0	124.4	206.8
6	455.05	154.44	55	33.11	70.3	25.0	124.4	331.2
5	455.05	159.53	55	33.11	72.6	25.0	127.2	458.3
4	455.05	159.53	55	33.11	72.6	25.0	127.2	585.5
3	455.05	159.53	55	33.11	72.6	25.0	127.2	712.7
2	455.05	157.73	55	33.11	71.8	25.0	126.2	838.8
1	455.05	174.29	100	100	79.3	45.5	168.0	1006.8
B	455.05	103.07	100	100	46.9	45.5	129.1	1135.9

A spot check of an interior steel beam on the Fifth Level was performed. The detailed calculations, which can be found in Appendix E, show that the W10x49 can carry the bending moment due to the weight of the P.C. plank and construction load. The beam is also checked for the factored moment due to dead and live loads, as well as, maximum allowable deflection and deflection under construction loads.

The final spot check was performed on the typical P.C. hollow core plank above the Hotel Level. This was done to examine that the maximum span of the plank could safely withstand the dead and live loads from each floor level. The calculation verifies that the plank is adequate by checking the technical specifications for the concrete plank from the manufacturer (Pittsburgh Flexicore).

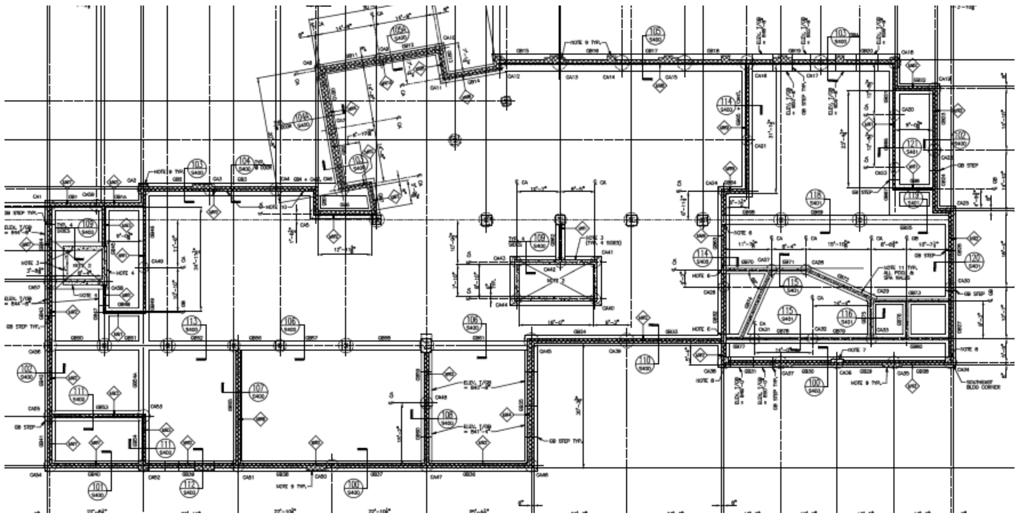
Conclusion

Through analyzing the existing structural conditions of the Cambria Suites Hotel, a better understanding of the design decisions and how the structure works was made. After an analysis of the gravity and lateral loads, it is found that Cambria Suites Hotel was designed according to code and can withstand all forces against the structure.

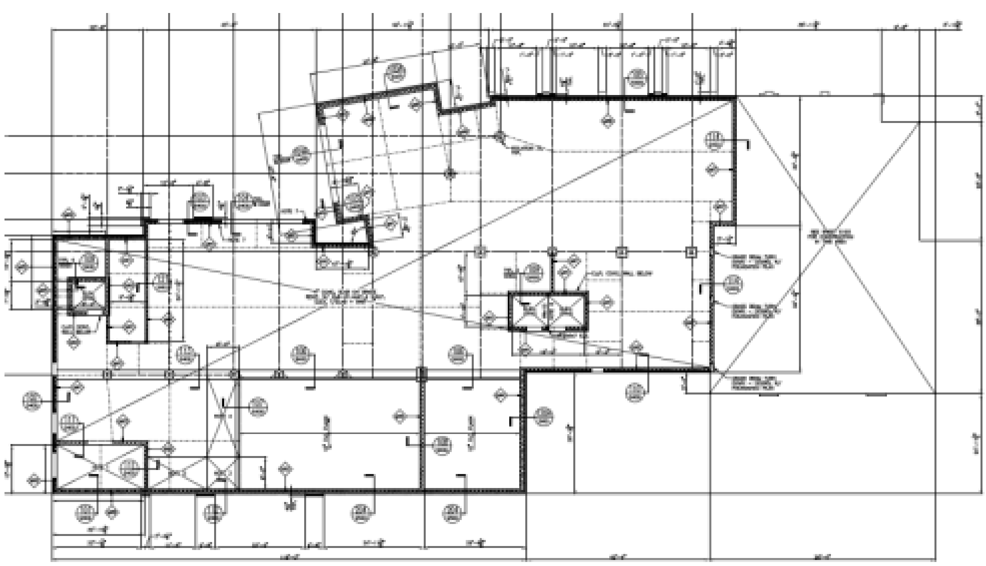
Various calculations were done on the gravity members to verify the structural design. A spot check of an interior column was analyzed to carry gravity loads of the preceding floors. The calculation concluded that the column referenced was adequate to carry the dead and live loads being transferred to it. The interior beam spot check was performed for exposure to gravity loads. This examination concluded that the member was sufficient to carry the design forces. The final spot check was done on the pre-cast concrete hollow core planks. It is verified by calculation, that the concrete plank can withstand the design forces of each typical floor above the Hotel Level.

Wind and seismic analysis were performed according to ASCE 7-05 to check the lateral forces against the structure. It was found that the North/South wind loads controlled over the East/West wind loads. This is highly reasonable due to the fact that the façade perpendicular to the North/South wind direction is much longer than that of the East/West wind direction. When comparing the base shears of wind and seismic loads, it was found that the seismic loads control over the wind loads without considering torsion effects. The structural system is comprised of reinforced masonry walls with a steel beam and column interior. The lateral system consists of reinforced masonry shear walls which extend through the entire height of the building without any openings for windows and/or doors. This justifies why the columns and beams were over designed because lateral forces were excluded from the spot checks. Further in depth research and calculations of Cambria Suites Hotel will include these additional forces.

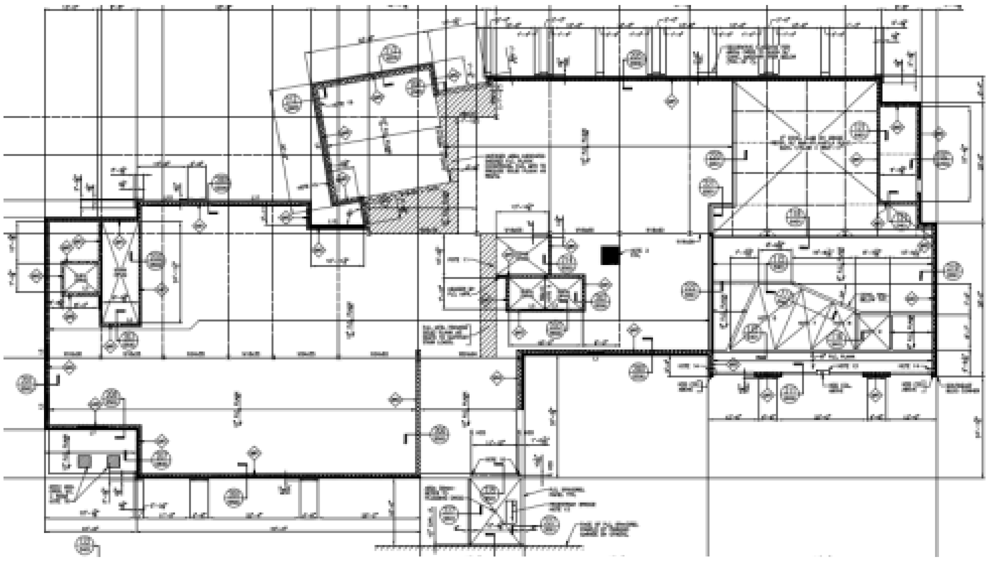
Appendix A: Building Layout



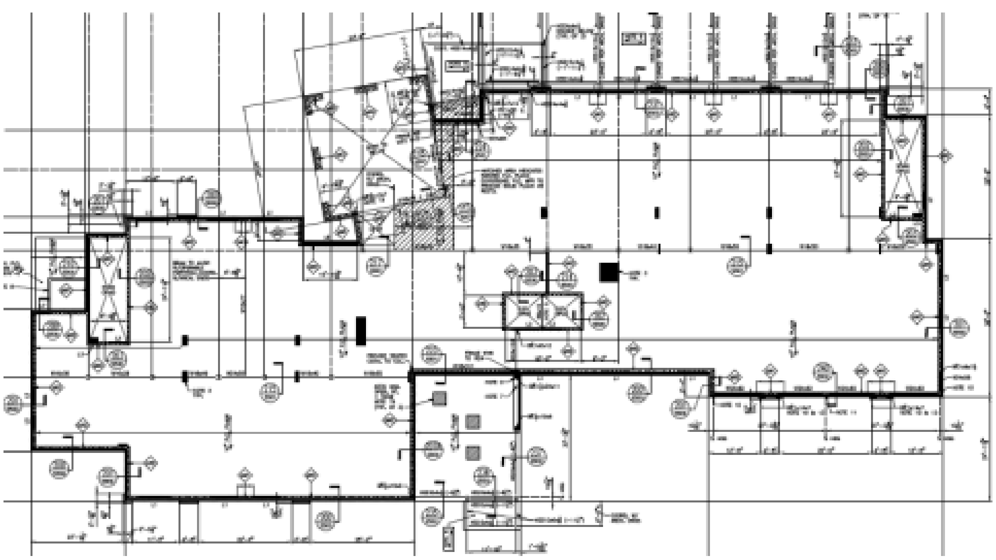
Foundation Plan



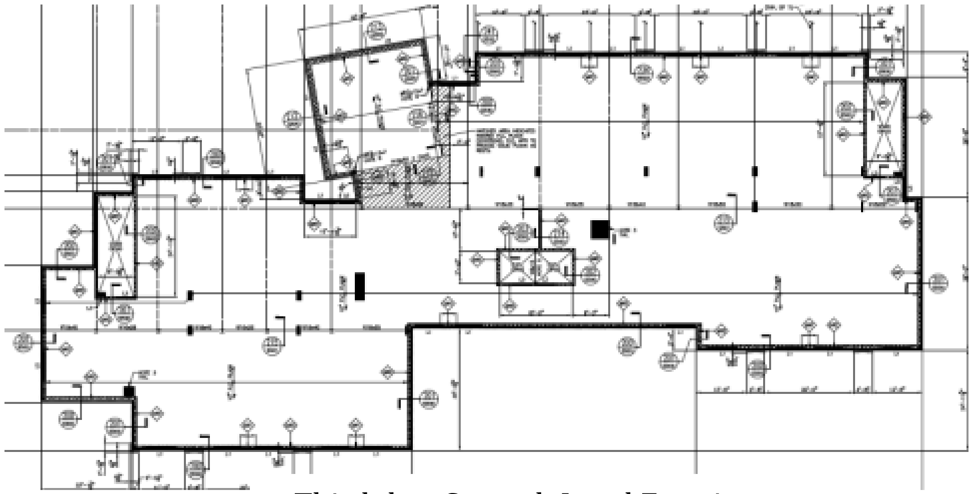
Plaza Level Framing Plan



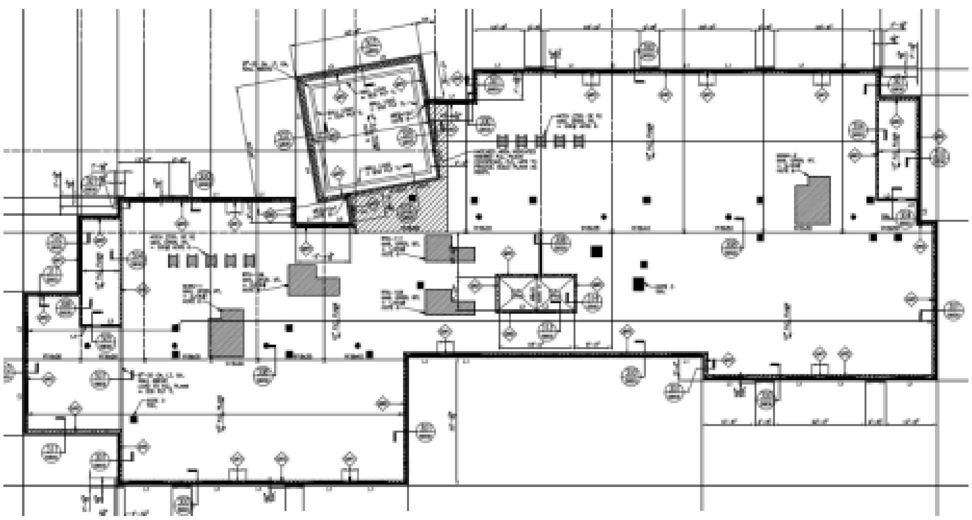
Hotel Level Framing Plan



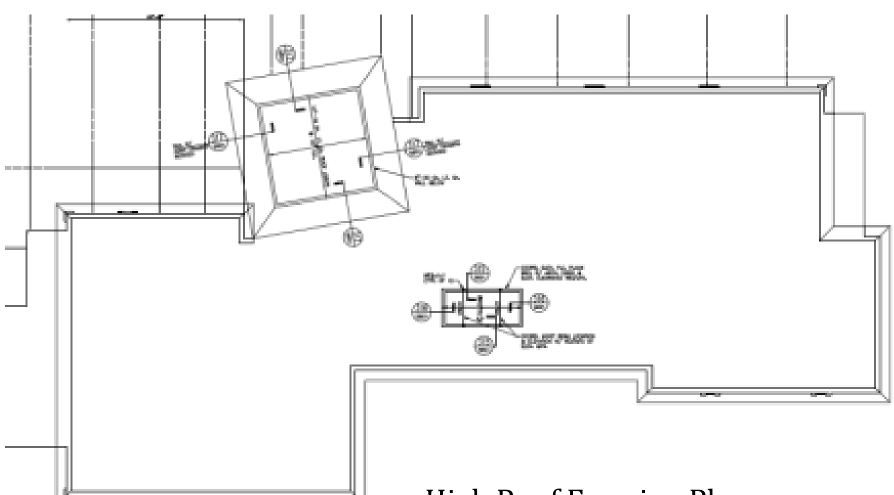
Second Level Framing Plan



Third thru Seventh Level Framing



Roof Framing Plan



High Roof Framing Plan

Appendix B: Snow Analysis

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JOB TECH. REPORT 1 - CALCULATIONS
SHEET NO. 1 OF 3
CALCULATED BY A. KACZMAREK DATE
CHECKED BY DATE
SCALE

SNOW LOADS

- GROUND SNOW LOAD (p_g) p_g = 25 PSF (FIG. 7-1)

NOTE: AES used p_g = 30 PSF

- FLAT ROOF SNOW LOAD (p_f)

$p_f = 0.7 C_e C_t I p_g > 20(I)$ (Eq. 7-1)

C_e = EXPOSURE FACTOR = 1.0 • Partially Exposed • Exposure B (TABLE 7-2)

C_t = THERMAL FACTOR = 1.0 (TABLE 7-3)

I = IMPORTANCE FACTOR = 1.0 • Occupancy II (TABLE 7-4)

$p_f = 0.7(1.0)(1.0)(1.0)(25\text{ PSF}) = 17.5 < 20(1.0) = 20$

∴ NOT OK

USE p_g = 30 PSF TO BE CONSERVATIVE

$p_f = 0.7(1.0)(1.0)(1.0)(30\text{ PSF}) = 21 > 20(1.1) = 20$ ✓

∴ OK

$\boxed{p_g = 30\text{ PSF}}$ $\boxed{p_f = 21\text{ PSF}}$

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JOB TECH. REPORT 1 - CALCULATIONS
SHEET NO. 2 OF 3
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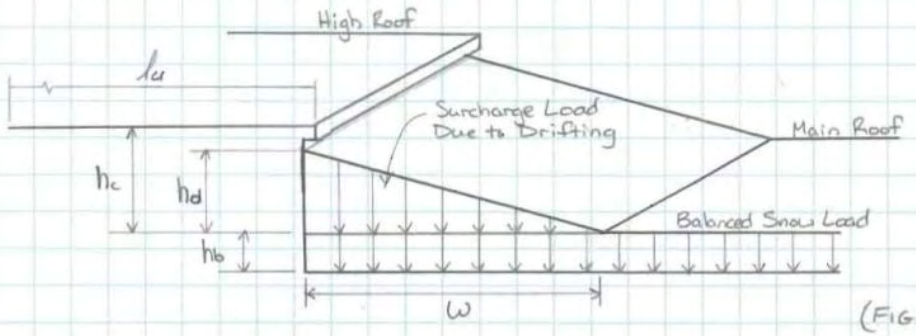
SNOW DRIFT (SECT. 7.7.1)

- Must be calculated due to high roof that rises above main roof level.



(FIG. 7-8)

l_u = length of roof upwind drift (ft)

h_c = clear height from top of balanced snow load to closest point on high roof. (ft)

h_d = height of snow drift (ft)

w = distance from eave to ridge (ft)

$p_d = h_d \gamma$

$\gamma = 0.13 p_g + 14 = 0.13(30 \text{ PSF}) + 14 = 17.9 \text{ PCF} < 30 \text{ PCF}$ ✓
∴ OK

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

$l_u = 30'-8''$ (N/S)
$l_u = 30'-8''$ (E/W)

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JOB: TECH. REPORT 1 - CALCULATIONS
SHEET NO. 3 OF 3
CALCULATED BY: A. KACZMAREK
DATE:
CHECKED BY:
DATE:
SCALE:

SNOW DRIFT (cont.)

$h_d = 1.88'$ (N/S)
$1.88'$ (E/W)

$h_b = p_g / \gamma = 30/17.9 = 1.676'$

$w = 4h_d = 4(1.88) = 7.52' \approx 7'-6\frac{1}{4}''$ (N/S) & (E/W)

Appendix C: Wind Analysis

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JOB TECH. REPORT 1 - CALCULATIONS
SHEET NO. 1 OF
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CHECKED BY DATE
SCALE

WIND LOADS

METHOD 2 : ANALYTICAL PROCEDURE

• WIND VARIABLES

$V = 90$ mph
$K_d = 0.85$
$I = 1.0$
EXPOSURE : B
$K_{zt} = 1.0$

(TABLE 6-3)
CASE 2

NOTE : INTERPOLATE K_z VALUES

LEVEL	HEIGHT	K_z
B	0'	0'
1	14'-10"	0.56
2	26'-10"	0.63
3	36'-10"	0.74
4	46'-10"	0.79
5	56'-10"	0.84
6	66'-10"	0.88
7	76'-10"	0.92
ROOF	86'-10"	0.95
HIGH ROOF	102'-2"	1.00

$$q_z = 0.00256 \underbrace{\boxed{K_z}}_{\text{VARIES BY LEVEL}} K_{zt} K_d V^2 I \qquad \text{(Eq. 6-15)}$$

$$q_z = 0.00256\, K_z (1.0)(0.85)(90^2)(1.0)$$

* THIS IS COMPLETED FOR ALL LEVELS AND PUT IN TABLE

EXAMPLE @ LEVEL 1 : $q_z = 0.00256(0.56)(1.0)(0.85)(90^2)(1.0)$
$= \underline{\underline{9.87}}$ PSF

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JOB TECH. REPORT 1 - CALCULATIONS
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CALCULATED BY A. KACZMAREK DATE
CHECKED BY DATE
SCALE

WIND LOADS (CONT.)

q_h @ MEAN ROOF HEIGHT : $\bar{z} = \dfrac{86.833' + 102.167'}{2} = 94.5'$

$\Downarrow$

$K_{zt} = 0.97$

$\bar{z} = 0.6h = 0.6(94.5') = 56.7' > \bar{z}_{min} = 30'$

$q_h = 0.00256(0.97)(1.0)(0.85)(90^2)(1.0) = \underline{17.10 \text{ PSF}}$

- C_p - EXTERNAL PRESSURE COEFFICIETS

NORTH / SOUTH

WINDWARD = 0.8
LEEWARD = -0.5

$L/B = 0.45$

$L = 98.92'$ $B = 219.67'$

EAST / WEST

WINDWARD = 0.8
LEEWARD = -0.2

$L/B = 2.22$

$L = 219.67'$ $B = 98.92'$

- WIND PRESSURE

$p_z = q_z G C_p - q_h G C_{pi}$ (WINDWARD)

$p_h = q_h G C_p - q_h G C_{pi}$ (LEEWARD)

$GC_{pi} = \pm 0.18$
FOR ENCLOSED BUILDINGS

NORTH / SOUTH EXAMPLE : @ LEVEL 1

$p_z = 9.87(0.85)(0.8) - 17.10(-0.18)$
$= \underline{\underline{9.79 \text{ PSF}}}$

$p_h = 17.10(0.85)(-0.5) - 17.10(0.18)$
$= \underline{\underline{-10.35 \text{ PSF}}}$

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JOB TECH. REPORT 1 – CALCULATIONS
SHEET NO. 3 OF
CALCULATED BY A. KACZMAREK DATE
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SCALE

WIND LOADS (CONT.)

EAST / WEST EXAMPLE : @ LEVEL 1

$p_z = 9.87(0.85)(0.8) - 17.10(-0.18)$
$= \underline{9.79 \text{ PSF}}$

$p_h = 17.10(0.85)(-0.2) - 17.10(0.18)$
$= \underline{-5.98 \text{ PSF}}$

* WIND PRESSURES CALCULATED FOR EACH STORY AND PUT IN TABLE

• FORCE OF WINDWARD ONLY

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

N/S EXAMPLE : @ LEVEL 1 $F_w = (219.67')(12')(9.79) = \underline{25.81 \text{ k}}$

• FORCE OF TOTAL PRESSURE

N/S EXAMPLE : @ LEVEL 1 $F_T = (219.67')(12')(20.1 \text{ PSF}) = \underline{52.98 \text{ k}}$

• WINDWARD SHEAR STORY

N/S EXAMPLE : @ LEVEL 7 $F = F_w$ @ (HIGH ROOF + ROOF + 7)
$= 6.90 + 31.63 + 30.97 = \underline{69.5 \text{ k}}$

• TOTAL STORY SHEAR

N/S EXAMPLE : @ LEVEL 7 $F = F_T$ @ (HIGH ROOF + ROOF + 7)
$= 11.68 + 54.48 + 53.82 = \underline{119.98 \text{ k}}$

Appendix D: Seismic Analysis

Seismic Force Resisting System: Floor Weights					
Basement					
Approximate Area:		12,808	SF		
Floor to Floor Height:		14.833	ft.		
Walls:			**Superimposed:**		
Perimeter:	681.5	ft.	Partitions:	15	PSF
Height:	14.833	ft.	MEP:	10	PSF
Unit Weight:	91	PSF	Finishes:	5	PSF
Weight =	**919.89**	**k**	**Weight =**	**384.24**	**k**
Slab:					
Slab Weight Not Included					
Columns:					
Shape	Quantity	Weight (PLF)	Column Height (ft)	Total Weight (k)	
W8x58	1	58	12	0.696	
W10x45	1	45	12	0.54	
W10x60	1	60	12	0.72	
W10x77	5	77	12	4.62	
W10x88	1	88	12	1.056	
W10x100	5	100	12	6	
W18x175	1	175	12	2.1	
			Weight =	**15.732**	**k**
Beams:					
Shape	Quantity	Weight (PLF)	Beam Length (ft)	Total Weight (k)	
W8x15	1	15	14	0.21	
			Weight =	**0.21**	**k**
Total Weight of Floor =			**1320.07**	**k**	
			103.07	PSF	

Seismic Force Resisting System: Floor Weights					
Floor 1					
Approximate Area:		16,236	SF		
Floor to Floor Height:		12	ft.		
Walls:			**Superimposed:**		
Perimeter:	763.91	ft.	Partitions:	15	PSF
Height:	12	ft.	MEP:	10	PSF
Unit Weight:	91	PSF	Finishes:	5	PSF
Weight =	**834.19**	**k**	**Weight =**	**487.08**	**k**
Slab:					
Thickness:	10	in.			
Unit Weight:	91	PSF			
Weight =	**1477.476**	**k**			
Columns:					
Shape	Quantity	Weight (PLF)	Column Height (ft)	Total Weight (k)	
W8x58	1	58	12	0.696	
W10x45	1	45	12	0.54	
W10x60	1	60	12	0.72	
W10x77	5	77	12	4.62	
W10x88	1	88	12	1.056	
W10x100	5	100	12	6	
W18x175	1	175	12	2.1	
			Weight =	**15.732**	**k**
Beams:					
Shape	Quantity	Weight (PLF)	Beam Length (ft)	Total Weight (k)	
W8x15	1	15	14	0.21	
W8x15	1	15	8	0.12	
W8x15	1	15	7.83	0.12	
W16x26	1	26	5	0.13	
W16x26	1	26	13	0.34	
W18x35	1	35	15.42	0.54	
W18x35	1	35	11.17	0.39	
W18x55	1	55	18	0.99	
W18x55	1	55	22	1.21	
W18x60	1	60	17.17	1.03	
W18x60	1	60	17.33	1.04	
W18x86	1	86	21.4	1.84	
W24x55	2	55	16	1.76	
W24x62	1	62	19.56	1.21	
W24x76	1	76	26.5	2.01	
W24x94	1	94	25.33	2.38	
			Weight =	**15.32**	**k**
Total Weight of Floor =			**2829.80**	**k**	
			174.29	PSF	

Seismic Force Resisting System: Floor Weights					
Floor 2					
Approximate Area:		15,113	SF		
Floor to Floor Height:		10	ft.		
Walls:			**Superimposed:**		
Perimeter:	753.9	ft.	Partitions:	15	PSF
Height:	10	ft.	MEP:	10	PSF
Unit Weight:	69	PSF	Finishes:	5	PSF
Weight =	**520.191**	**k**	**Weight =**	**453.39**	**k**
Slab:					
Thickness:	10	in.			
Unit Weight:	91	PSF			
Weight =	**1375.283**	**k**			
Columns:					
Shape	Quantity	Weight (PLF)	Column Height (ft)	Total Weight (k)	
W10x45	1	45	10	0.45	
W10x60	1	60	10	0.6	
W10x77	5	77	10	3.85	
W10x88	1	88	10	0.88	
W10x100	5	100	10	5	
			Weight =	**10.78**	**k**
Beams:					
Shape	Quantity	Weight (PLF)	Beam Length (ft)	Total Weight (k)	
W8x15	1	15	7.83	0.12	
W8x31	1	31	12	0.37	
W16x26	1	26	13	0.34	
W16x26	1	26	15.42	0.40	
W16x26	1	26	7.25	0.19	
W16x77	1	77	38	2.93	
W18x35	1	35	19	0.67	
W18x35	1	35	17.17	0.60	
W18x40	2	40	16	1.28	
W18x40	1	40	17.33	0.69	
W18x50	1	50	13.55	0.68	
W18x50	2	50	19	1.90	
W18x55	1	55	19.56	1.08	
W18x46	1	46	22	1.01	
W18x60	1	60	22	1.32	
W18x76	1	76	26.5	2.01	
W18x311	1	311	25.33	7.88	
W24x55	1	55	11.17	0.61	
			Weight =	**24.07**	**k**
Total Weight of Floor =			**2383.72**	**k**	
			157.73	PSF	

Seismic Force Resisting System: Floor Weights					
Floors 3 thru 5					
Approximate Area:		15,113	SF		
Floor to Floor Height:		10	ft.		
Walls:			**Superimposed:**		
Perimeter:	815.33	ft.	Partitions:	15	PSF
Height:	10	ft.	MEP:	10	PSF
Unit Weight:	69	PSF	Finishes:	5	PSF
Weight =	**562.5777**	**k**	**Weight =**	**453.39**	**k**
Slab:					
Thickness:	10	in.			
Unit Weight:	91	PCF			
Weight =	**1375.283**	**k**			
Columns:					
Shape	Quantity	Weight (PLF)	Column Height (ft)	Total Weight (k)	
W10x33	1	33	10	0.33	
W10x45	1	45	10	0.45	
W10x49	4	49	10	1.96	
W10x54	1	54	10	0.54	
W10x68	6	68	10	4.08	
			Weight =	**7.36**	**k**
Beams:					
Shape	Quantity	Weight (PLF)	Beam Length (ft)	Total Weight (k)	
W8x15	1	15	7.83	0.12	
W16x26	1	26	15.42	0.40	
W16x26	1	26	11.17	0.29	
W18x35	1	35	18	0.63	
W18x35	1	35	17.17	0.60	
W18x40	1	40	13	0.52	
W18x40	2	40	16	1.28	
W18x40	1	40	17.33	0.69	
W18x50	2	50	19	1.90	
W18x50	1	50	13.55	0.68	
W18x50	1	50	22	1.10	
W18x55	1	55	19.56	1.08	
W18x60	1	60	26.5	1.59	
W18x65	1	65	22	1.43	
			Weight =	**12.31**	**k**
		Total Weight of Floor =	**2410.92**	**k**	
			159.53	PSF	

Seismic Force Resisting System: Floor Weights					
Floors 6 thru 7					
Approximate Area:		15,113	SF		
Floor to Floor Height:		10	ft.		
Walls:			**Superimposed:**		
Perimeter:	815.33	ft.	Partitions:	15	PSF
Height:	10	ft.	MEP:	10	PSF
Unit Weight:	60	PSF	Finishes:	5	PSF
Weight =	**489.198**	**k**	**Weight =**	**453.39**	**k**
Slab:					
Thickness:	10	in.			
Unit Weight:	91	PCF			
Weight =	**1375.283**	**k**			
Columns:					
Shape	Quantity	Weight (PLF)	Column Height (ft)	Total Weight (k)	
W10x33	13	33	10	4.29	
			Weight =	**4.29**	**k**
Beams:					
Shape	Quantity	Weight (PLF)	Beam Length (ft)	Total Weight (k)	
W8x15	1	15	7.83	0.12	
W16x26	1	26	15.42	0.40	
W16x26	1	26	11.17	0.29	
W18x35	1	35	18	0.63	
W18x35	1	35	17.17	0.60	
W18x40	1	40	13	0.52	
W18x40	1	40	16	0.64	
W18x40	1	40	17.33	0.69	
W18x50	1	50	19	0.95	
W18x50	1	50	13.55	0.68	
W18x50	2	50	22	2.20	
W18x55	1	55	19.56	1.08	
W18x60	1	60	26.5	1.59	
W18x65	1	65	22	1.43	
			Weight =	**11.82**	**k**
Total Weight of Floor =			**2333.98**	**k**	
			154.44	PSF	

Seismic Force Resisting System: Floor Weights					
Roof					
Approximate Area:		15,113	SF		
Walls:			**Superimposed:**		
Perimeter:	789.58	ft.			
Height:	4	ft.	MEP:	10	PSF
Unit Weight:	60	PSF	Roof Mat:	10	PSF
Weight =	**189.4992**	**k**	**Weight =**	**302.26**	**k**
Slab:					
Thickness:	10	in.			
Unit Weight:	91	PCF			
Weight =	**1375.283**	**k**			
Beams:					
Shape	Quantity	Weight (PLF)	Beam Length (ft)	Total Weight (k)	
W8x21	1	15	7.83	0.12	
W16x26	1	26	13	0.34	
W16x26	1	26	15.42	0.40	
W18x35	2	35	16	1.12	
W18x35	1	35	18	0.63	
W18x35	1	35	17.17	0.60	
W18x40	1	40	19.56	0.78	
W18x40	1	40	17.33	0.69	
W18x50	1	50	22	1.10	
W18x50	1	50	13.55	0.68	
W18x50	2	50	19	1.90	
W18x60	1	60	26.5	1.59	
W18x65	1	65	22	1.43	
			Weight =	**11.38**	**k**
		Total Weight of Floor =	**1878.42**	**k**	
			124.29	PSF	

Seismic Force Resisting System: Floor Weights					
High Roof					
Approximate Area:		576	SF		
			Superimposed:		
			Roof Mat:	10	PSF
			Weight =	**5.76**	**k**
Roof Joists:					
Unit Weight:	5	PLF			
Weight =	**2.16**	**k**			
		Total Weight of Floor =	**7.92**	**k**	
			13.75	PSF	

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SEISMIC LOADS

- SITE CLASS C - Very Dense Soil & Soft Rock (TABLE 20.3-1)
- OCCUPANCY CATEGORY II (TABLE 1-1)
- IMPORTANCE FACTOR : 1.0 (TABLE 11.5-1)
- SPECTRAL RESPONSE ACCELERATION, SHORT (S_S) & SPECTRAL RESPONSE ACCELERATION, 1s (S_1) (FIG. 22-1 thru 22-14)

$S_S = 0.125$
$S_1 = 0.049$

- SITE COEFFICIENTS (F_a & F_v) (TABLE 11.4-1 & 11.4-2)

$F_a = 1.2$
$F_v = 1.7$

- $S_{MS} = F_a S_s = 1.2(0.125)$ $\boxed{S_{MS} = 0.15}$ (Eq. 11.4-1)
- $S_{DS} = 2/3(S_{MS}) = 2/3(0.15)$ $\boxed{S_{DS} = 0.100}$ (Eq. 11.4-3)
- $S_{M1} = F_v S_1 = 1.7(0.049)$ $\boxed{S_{M1} = 0.0833}$ (Eq. 11.4-2)
- $S_{D1} = 2/3\, S_{M1} = 2/3(0.0833)$ $\boxed{S_{D1} = 0.055}$ (Eq. 11.4-4)

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SEISMIC LOADS (cont.)

- $T_a = C_t h_n^x$
  $= 0.02(102.167)^{0.75}$ $\boxed{T_a = 0.643\ s}$ (Eq. 12.8-7)

- $C_u = 1.7$ (TABLE 12.8-1)

- $T = T_a C_u$
  $= 0.643(1.7)$ $\boxed{T = 1.09\ s}$ (SEC. 12.8.2)

- $C_s =$ MIN. $\left[\begin{array}{l} \dfrac{S_{D1}}{T(R/I)} = \dfrac{0.055}{1.09(2/1)} = \boxed{0.025} \geq 0.01 \\ \dfrac{S_{DS}}{(R/I)} = \dfrac{0.100}{(2/1)} = 0.05 \geq 0.01 \\ \dfrac{S_{D1}T_L}{T^2(R/I)} = \dfrac{0.055(12)}{(1.09)^2(2/1)} = 0.278 \geq 0.01 \end{array}\right.$

AES used this value in their calculations

WHERE: $R = 2$ (TABLE 12.2-1)
$I = 1.0$ (TABLE 11.5-1)
$T_L = 12$ (FIG. 22-15)

- $k = 0.75 + 0.5(T)$
  $= 0.75 + 0.5(1.09)$ $\boxed{k = 1.295}$ (SEC. 12.8.3)

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SEISMIC LOADS (cont.)

• SEE EXCEL SPREADSHEETS FOR FLOOR WEIGHTS

FLOOR	APPROX. FLOOR AREA	TOTAL WEIGHT
B	12808 SF	103.07 PSF
1	16236 SF	174.29 PSF
2	15113 SF	157.73 PSF
3-5	15113 SF	159.53 PSF
6-7	15113 SF	154.44 PSF
ROOF	15113 SF	124.29 PSF
HIGH ROOF	576 SF	13.75 PSF

• TOTAL BUILDING WEIGHT (W_T)

$$W_T = 12808(103.07) + 16236(174.29) + 15113(157.73) + 3(15113)(159.53) + 2(15113)(154.44) + 15113(124.29) + 576(13.75)$$

$$W_T = \underline{\underline{20321}}^{k}$$

• BASE SHEAR (V)

$$V = C_s W_T = 0.025(20321)$$

$$\boxed{V = 508.025^{k}}$$

• $w_x h_x^k$ (varies @ height)

EXAMPLE @ LEVEL 1 : $w_x = 2829.8^{k}$, $h_x = 14.833'$, $k = 1.295$

$$w_x h_x^k = 2829.8\,(14.833')^{1.295} = \boxed{93003.1 \text{ ft}\cdot\text{k}}$$

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SEISMIC LOADS (cont.)

- $\Sigma w_i h_i^k \Rightarrow$ sum of $w_x h_x^k$ of all floors $= \boxed{3117703 \text{ ft-k}}$

- $C_{vx} = \dfrac{w_x h_x^k}{\Sigma w_i h_i^k}$ (varies @ height) (Eq. 12.8-12)

  EXAMPLE @ LEVEL 1 : $C_{vx} = \dfrac{93003}{3117703} = \boxed{0.030}$

- $F_x = C_{vx}(V)$ (Eq. 12.8-11)

  EXAMPLE @ LEVEL 1 : $F_x = 0.0298(508.025^k) = \boxed{15.15^k}$

- STORY SHEAR (V_x)

  $V_x = F_x$(@ level) + F_x(@ all levels above)

  EXAMPLE @ LEVEL 7: $V_x = F_x(HR) + F_x(ROOF) + F_x(7)$
  $= 0.52 + 99.18 + 105.18$

  $V_x @ 7 = \boxed{204.88^k}$

- MOMENTS (M_x)

  M_x = (Tributary Floor Area Height)(F_x)

  EXAMPLE @ LEVEL 7 : $M_x = ((76.833 + 66.833)/2)(105.18)$

  $M_x @ 7 = \boxed{7555.39 \text{ ft.k}}$

Appendix E: Spot Checks

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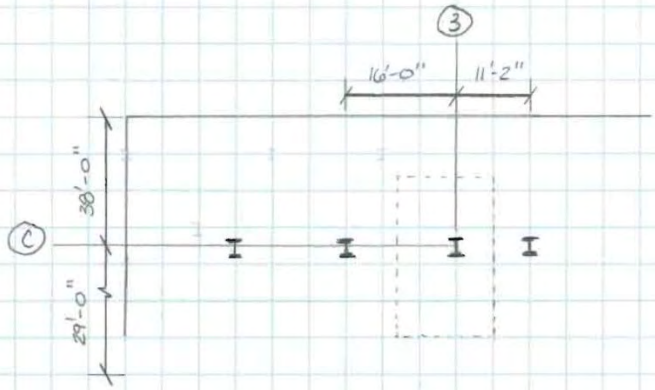
INTERIOR COLUMN SPOT CHECK



- Column C-3, FIFTH LEVEL
- check based on axial-gravity loads only

- TRIBUTARY AREA $= \left(\frac{16' + 11.167'}{2}\right)\left(\frac{29' + 38'}{2}\right) = 455.05\ ft^2$

- LIVE LOADS

Hotel Rooms (6 floors) = 40 PSF
Roof = 20 PSF
Partitions = 15 PSF
Lobby = 100 PSF

- LIVE LOAD ELEMENT FACTOR (K_{LL})

$K_{LL} = 4$ (for columns)

- LIVE LOAD REDUCTION
  - only for hotel rooms, roof, & partitions
  - $K_{LL} A_T \geq 400\ ft^2$

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COLUMN SPOT CHECK (cont.)

- REDUCED HOTEL LIVE LOAD

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) \quad \text{(Eq. 4-1 ASCE 7-05)}$$

$$= L_o\left(0.25 + \frac{15}{\sqrt{4(455.05)}}\right)$$

$$L = L_o(0.602)$$
$$= 40(0.602)$$
$$= 24.08 \text{ PSF}$$

- REDUCED PARTITION LIVE LOAD

$$L = L_o(0.602)$$
$$= 15(0.602)$$
$$= 9.03 \text{ PSF}$$

- REDUCED ROOF LIVE LOAD

$$L = L_o(0.602)$$
$$= 20(0.602)$$
$$= 12.04 \text{ PSF}$$

- DEAD LOADS
  - hotel rooms & roof
  - include 10" slab, walls, & superimposed dead loads

* Refer to Seismic Excel Spreadsheet

ROOF = 124.29 PSF
FLOORS 6-7 = 154.44 PSF
FLOORS 3-5 = 159.53 PSF
FLOOR 2 = 157.73 PSF
FLOOR 1 = 174.29 PSF
Basement = 103.07 PSF

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COLUMN SPOT CHECK (cont.)

- REDUCTION LIVE LOAD

ROOM	# FLOORS	LIVE LOADS
GUEST ROOM	6	33.11 PSF
LOBBY	1	100 PSF
ROOF	1	12.04 PSF

* Refer to Excel Spreadsheet for Accumulated Axial Loads

- LOAD COMBINATION

$LC = 1.2D + 1.6L$

- For Column C-3 on the FIFTH Level, which supports levels 6^{th} – ROOF $\Rightarrow$ $P_u = 331.2^K$

- AVAILABLE COLUMN STRENGTH

Column C-3 : W10 x 49
$L = 30'-0''$
$KL_{eff} = \frac{L}{(r_x/r_y)} = \frac{30'}{1.71} = 17.52 \text{ ft.}$

$r_x = 4.35$
$r_y = 2.54$
$r_x/r_y = 1.71$

Interpolate between $KL = 17'$ and $KL = 18'$

$\Rightarrow \phi P_n = 394^k > 331.2^k = P_u$

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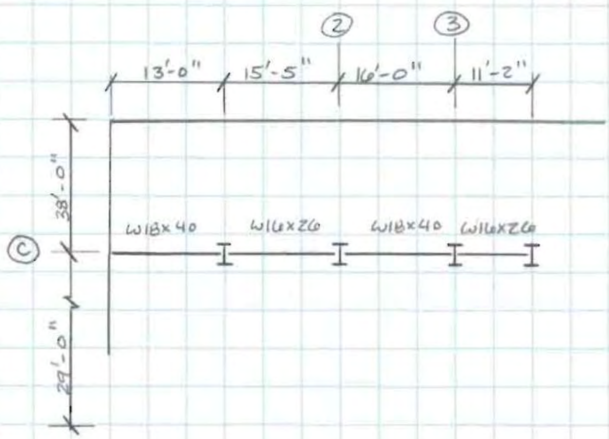
INTERIOR BEAM SPOT CHECK

- 5th Level Beam
- W18 × 40



- LOADS

LIVE LOAD: 40 PSF (Hotel Rooms)
+ 15 PSF (Partitions)
55 PSF TOTAL LIVE LOAD

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

where $K_{LL} = 2$ (columns)

$$= 55\left(0.25 + \frac{15}{\sqrt{2(536)}}\right)$$

$$= 38.95 \text{ PSF}$$

$$A_T = 16'\left(\frac{38' + 29'}{2}\right) = 536 \text{ ft}^2$$

DEAD LOAD: 91 PSF (Precast Plank)
10 PSF (Superimposed DL)
+ 5 PSF (Finishes)
106 PSF TOTAL DEAD LOAD

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- LOAD COMBINATION

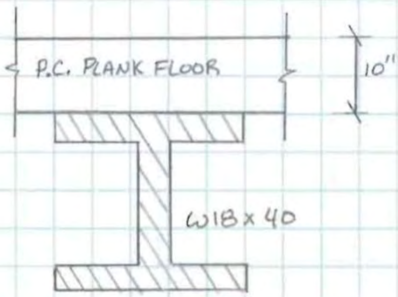
$$W_u = 1.2D + 1.6L = 1.2(106\text{ PSF}) + 1.6(38.95\text{ PSF})$$
$$= 189.5\text{ PSF}$$

$$\text{TRIBUTARY WIDTH} = \left(\frac{38' + 29'}{2}\right) = 33.5'$$

$$W_u = 189.5\text{ PSF}(33.5') = 6.35\text{ KLF}$$

$$M_u = \frac{w_u l^2}{8} = \frac{6.35(16')^2}{8} = \underline{\underline{203.2\text{ ft}\cdot\text{k}}}$$

$$b_{eff} = \text{MIN}\begin{cases} 33.5'(12'') = 402'' \\ \frac{\text{SPAN}}{4} = \frac{16'(12'')}{4} = \boxed{48''} \end{cases}$$



- CHECK DEFLECTION UNDER CONSTRUCTION LOADS

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

$$w_{conc} = 91\text{ PSF}(33.5') = 3.048\text{ KLF}$$

$$\Delta_{allow} = l/360 = \frac{16'(12'')}{360} = 0.533''$$

$$I_{Req.} = \frac{5 w_{conc} l^4}{384 \Delta_{conc} E} = \frac{5(3.048)(16)^4}{384(0.533)(29000)} \times 1728$$

$$I_{Req.} = 299.2\text{ in}^4$$

$$\underset{\text{W18x40}}{I_x} = 610\text{ in}^4 \gg 299.2\text{ in}^4 = I_{Req.} \quad \therefore \text{OK}$$

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• CHECK BENDING FOR CONSTRUCTION

$w_{conc.} = 3.048$ KLF
$w_{LIVE} = 20$ PSF $(33.5') = 0.670$ KLF
$w_u = 1.2(3.048) + 1.6(0.670)$
$= \underline{4.73 \text{ KLF}}$

$M_u = \frac{w_u l^2}{8} = \frac{4.73(16)^2}{8} = 151.36$ ft.k

$\phi M_n = 294$ ft.k > 151.36 ft.k $= M_u$ ∴ OK
W18x40

• AVAILABLE STRENGTH IN FLEXURE (TABLE 3-19) STEEL MANUAL

ASSUME $\Sigma Q_n = 588$

$a = \frac{\Sigma Q_n}{0.85(f'_c)(b_{eff})} = \frac{588}{0.85(4)(48)} = 3.6$

$Y_2 = 10'' - a/2 = 10'' - 3.6/2 = 8.2''$ (Round down to 7")

W18x40 $Y_2 = 7.0$ $\Sigma Q_n = 588^k$ @ PNA @ TFL

$\Rightarrow \phi M_n = 704$ ft.k $\gg 203.2$ ft.k $= M_u$
∴ OK

• DEFLECTION (TABLE 3-20) STEEL MANUAL

$Y_2 = 7.0$ $w_{LL} = 38.95(33.5') = 1.30$ KLF
$I_b = 2110$ $\Delta_{ALLOW} = l/360 = \frac{16'(12)}{360} = 0.533''$

$\Delta_{LL} = \frac{5 w_{LL} l^4}{384 E I_b} = \frac{5(1.30)(16)^4}{384(29000)(2110)} \times 1728$

$\Delta_{LL} = 0.032'' < 0.533'' = \Delta_{ALLOW}$ ∴ OK

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PRECAST HOLLOW CORE PLANK SPOT CHECK

- For typical floors above Hotel Level
- Max Span = 38'-0"
- 10" thick concrete plank, 4'-0" wide sections

LL = 40 PSF
DL = 15 PSF (Partitions)
10 PSF (MEP)
+ 5 PSF (Finishes)

30 PSF TOTAL DL

LOAD COMBINATION

LC: $1.2DL + 1.6LL = 1.2(30) + 1.6(40) = \boxed{100 \text{ PSF}}$

The Manufacturer of the hollow core precast plank is Pittsburgh Flexicore. According to their 10" precast plank technical specifications, a plank spanning 38'-0" can safely be loaded to 105 PSF (untopped) or 120 PSF (2" topping). Through the use of this load combination, the planks are adequate.

Appendix F: Construction Photos