Cambria Suites Hotel

Pittsburgh, PA

Technical Report 3

Lateral System Analysis and Confirmation Design

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November 29, 2010

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Executive Summary

The following technical report provides an analysis of the existing design of the lateral force resisting system of the Cambria Suites Hotel. All loads that were calculated in the existing structural conditions report were applied to the lateral force resisting system which was analyzed for this report. The lateral force resisting system is comprised of reinforced concrete masonry shear walls. A detailed description of the structural system of the building and how all loads are transferred to the foundation is given in the report.

Through the following report, an ETABS model was created to compare the analysis results to the hand calculations performed for the Cambria Suites Hotel. It is important to note that this model represents an analysis of the existing lateral members only; shear walls and rigid diaphragms. This was done to simplify the attempt of creating a model of the Cambria Suites Hotel. In accordance, all hand calculations only accounted for the shear walls as the lateral resisting system. Diaphragms were modeled as rigid area elements with applied area masses which were determined in the existing structural conditions report. The ETABS model was also used to determine the Fundamental Period of the building. Lateral loads were applied to the model to determine center of rigidity, center of mass, torsion, overturning moment, story drift, and story shear. These results were all compared to the hand calculations and the checked against allowable code limits.

After comparing the ETABS model with the hand calculations, a few differences were found in the location of the center of rigidity. This difference is most likely a result of the hand calculations only accounting for the shear walls, whereas the ETABS model includes the rigid diaphragms. Due to this difference, the center of rigidity values calculated by hand will be used in determining relative stiffness, torsion, shear, and overturning moment. The hand calculations verify that the shear walls are properly reinforced and are providing the majority of the lateral resistance. This suggests that it is only necessary to include the shear walls for this analysis.

The result of the overturning moment calculations show that the gravity system of the building will resist any uplift or torsion created on the building due to the lateral loads. This is because the lateral loads are a small fraction of the gravity loads. Other results such as displacements and story drifts were also found to be within the allowable code limits, and are verified by hand calculations, as well as the ETABS model.

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 7th 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 2nd-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 2nd Floor level and below, as well as vertical strips to separate window pairings and to accent building corners.

The following report will analyze the lateral system of the Cambria Suites Hotel and will determine if the building design is sufficient to resist the lateral loads that act on the building. An ETABS model of the building was designed to compare the results of the hand calculations with the lateral analysis of the building model.

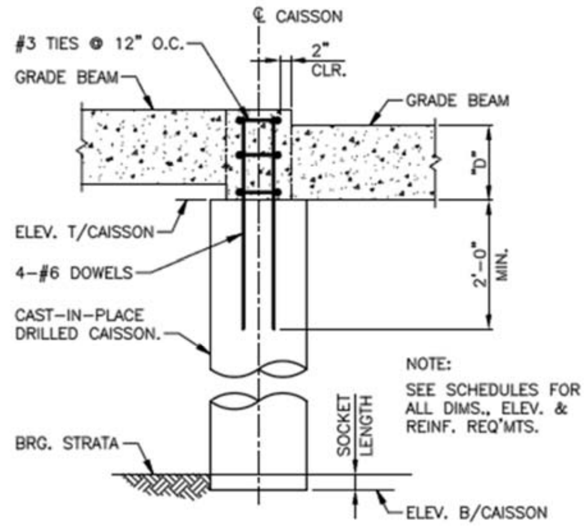
Structural System

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.

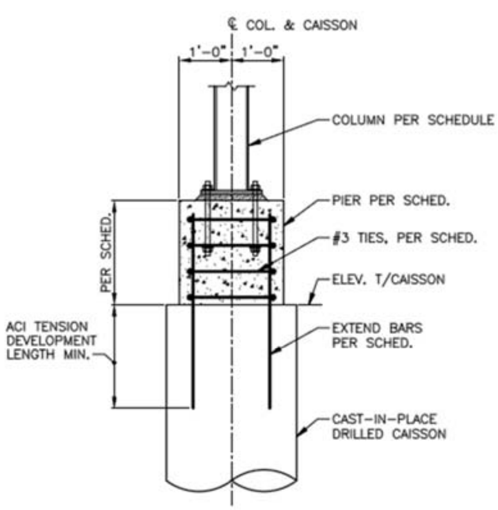
As a result of GeoMechanic's geotechnical study, the foundation of Cambria Suites Hotel incorporates a drilled cast-in-place concrete caissons and grade beams designed to support the load bearing walls and columns. The ground floor is comprised of a 4" concrete slab on grade, as well as, 10" precast 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)

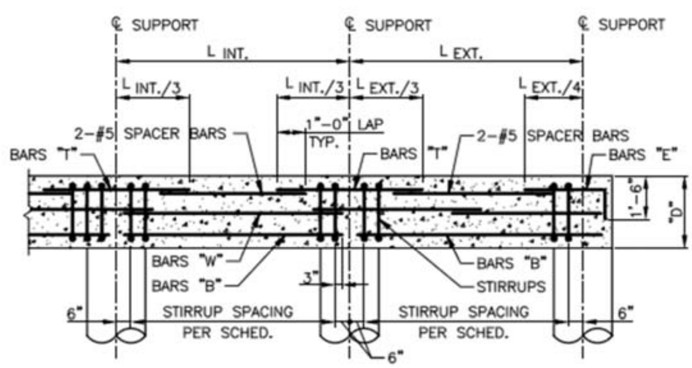


Typical Caisson & Grade Beam Detail
Figure 1.1

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

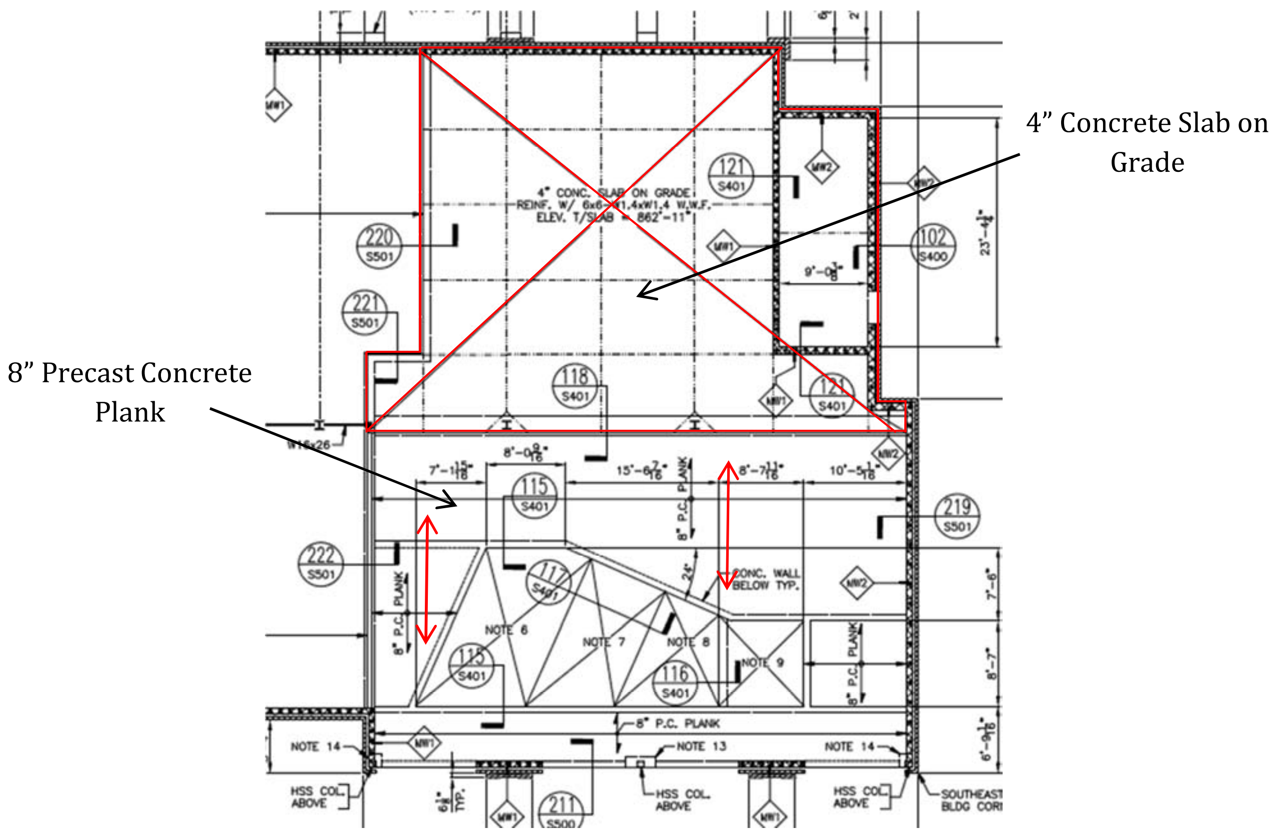


Typical Grade Beam Reinforcing Detail
Figure 1.3

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.

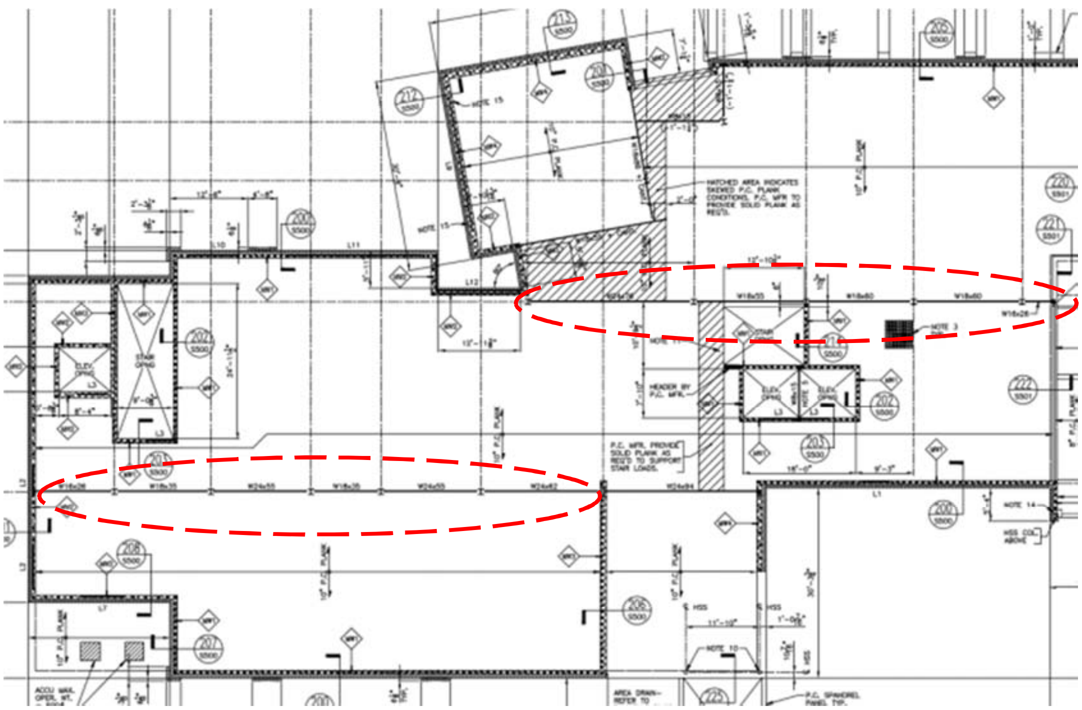
The Plaza level floor system is a combination of 10" precast concrete plank, 8" precast concrete plank and 4" slab on grade. Since there is no basement in the North-East section of the hotel due to the fitness center and pool, the site was excavated properly in order to place the 4" slab on grade and 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)



<u>Partial Hotel Level Floor Slab</u>

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

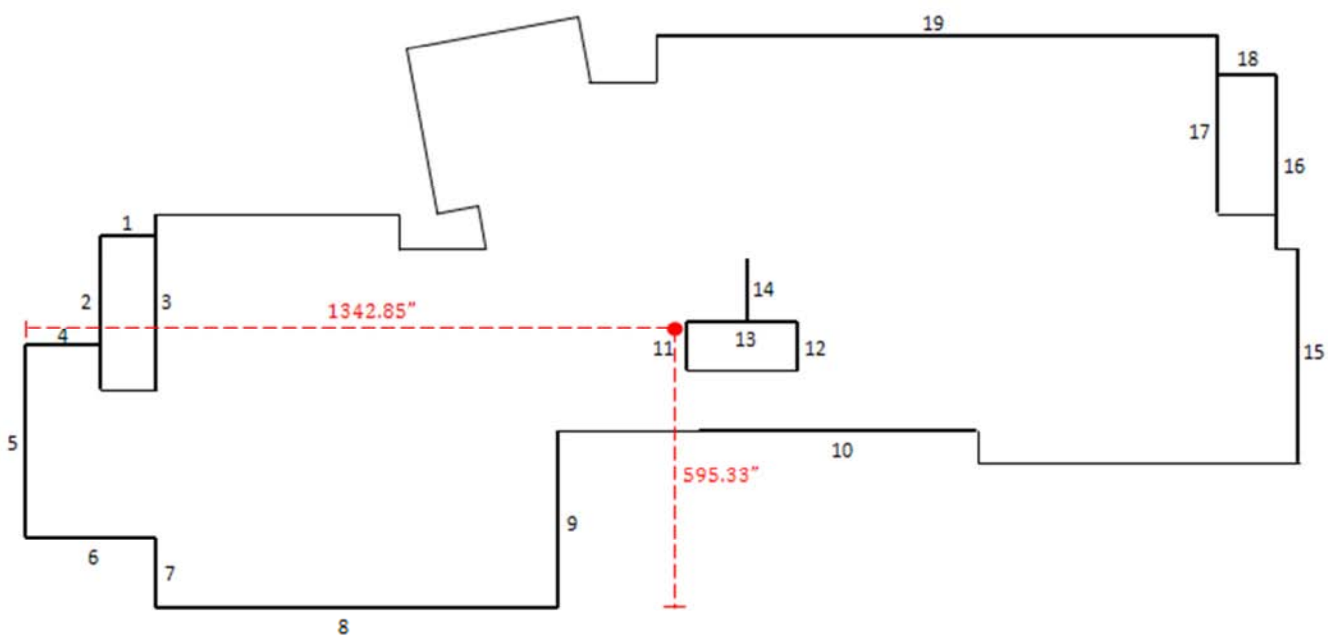
Figure 2.2

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.

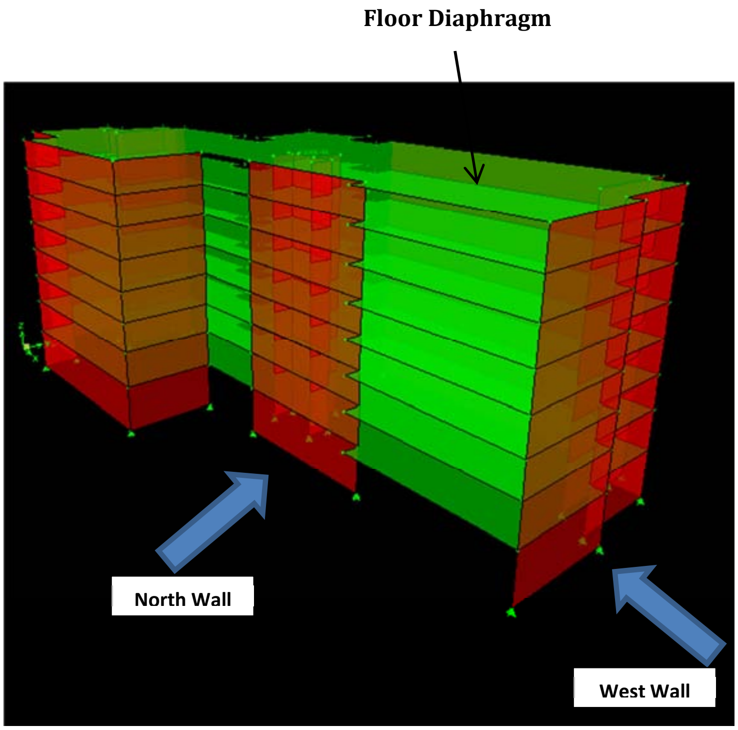
Wind and seismic loads, as well as gravity loads, are transferred to the foundation by first traveling thru the rigid diaphragm; the precast concrete plank floor system. Loads are then transferred to the concrete masonry shear walls. From there, loads are transferred down to the preceding floor system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



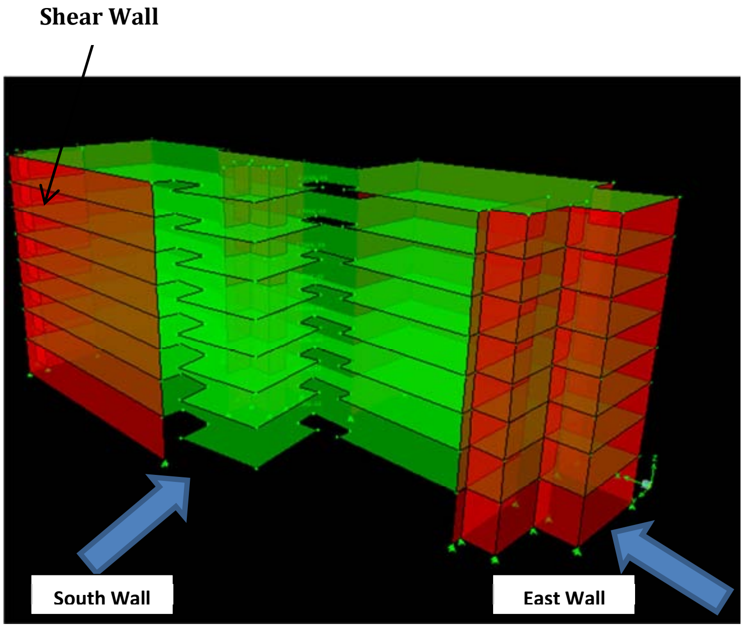
Lateral Shear Wall System
Figure 3.1

ETABS Model

ETABS is a recognized industry leader for building analysis and design software developed by Computers and Structures, Inc. One of the advantages of this program is the ability to look at each floor of the building strictly as a rigid diaphragm against lateral loading. Therefore, for the analysis of this technical assignment, the building's lateral system and diaphragms were the only building components modeled. As shown in Figures 4.1 and 4.2, the shear walls and diaphragms were the only components modeled. Material properties were inputted for the shear walls, and a rigid diaphragm was assigned for the floor. Gravity loads were then applied as additional area masses to the floor diaphragms. Wind and seismic loads were applied about the centers of rigidity of the building. In addition to comparing the results of hand calculations, an ETABS model effectively determines the following: center of mass, center of rigidity, the controlling ASCE 7-05 load combinations, story displacements, story drifts, story shears, and the effects of torsion.



ETABS Model: North & West Walls
Figure 4.1



ETABS Model: South & East Walls
Figure 4.2

Codes and Requirements

References used by the engineer of record in order to carry out the structural design of the Cambria Suites Hotel.

- 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
- ETABS Building Analysis and Design Software – Computers and Structures, Inc.

Drift Criteria

The following allowable drift criteria used to check deflection of the Cambria Suites Hotel will be in accordance with the International Building Code, 2006 edition.

Allowable Building Drift ⇒ $\Delta_{wind} = H/400$

Allowable Story Drift ⇒ $\Delta_{seismic} = 0.02H_{sx}$

Load Combinations

The following list shows the various load combinations according to ASCE 7-05 for factored loads using strength design and from the International Building Code, 2006 edition. These load combinations are used in the analysis of the lateral system for this report.

$1.4D$

$1.2D + 1.6L + 0.5L_r$

$1.2D + 1.6L_r + 1.0(L \text{ or } W)$

$1.2D + 1.6W + 1.0L + 0.5L_r$

$1.2D + 1.0E + 1.0L$

$0.9D + 1.6W$

$0.9D + 1.0E$

All load combinations were considered in the analysis of the ETABS model. After evaluating story displacements, shears, and drifts computed by ETABS for each of the above load combinations, it was concluded that the controlling load combination for the North/South direction was $1.2D+1.6W+1.0L+0.5L_r$ due to its large surface area. The controlling load combination for the East/West direction was $0.9D+1.0E$.

Gravity Loads

The gravity load conditions determined by ASCE 7-05 are provided for reference and are compared to the Design Loads used by AES:

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

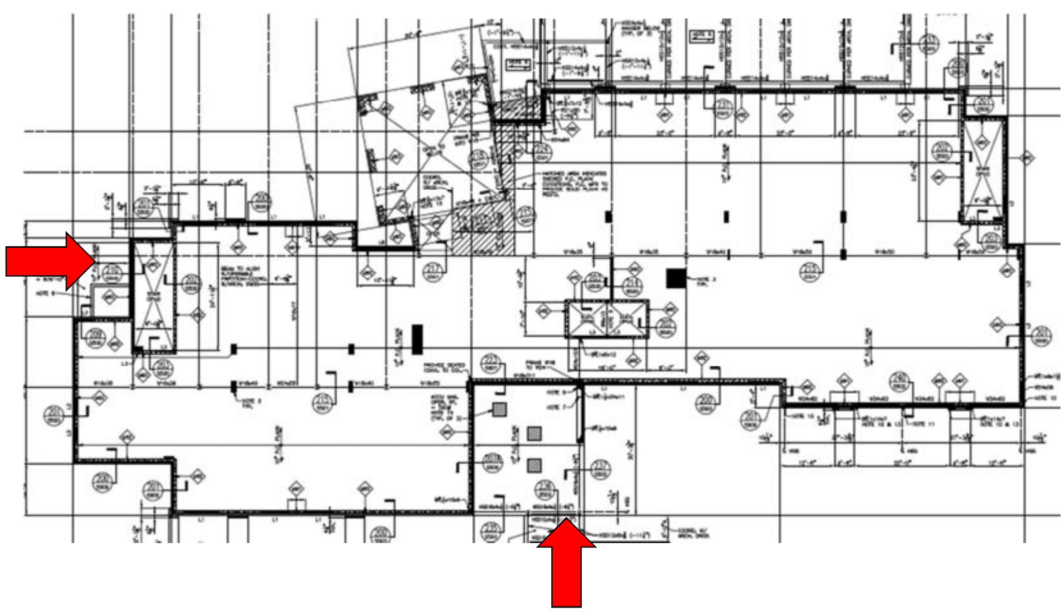
Wind Analysis

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 B. The North/South and East/West wind directions are labeled on the typical floor plan in Figure 5.1.

<u>Wind Directions</u>

Figure 5.1

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 ≥ 1 Hz ∴ 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 5.2 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 5.4 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	12.0	-7.27	19.3	5.52	8.88	5.52	8.88	521.64	839.16
Roof	86.833	10	0.95	16.7	11.36	-7.27	18.6	24.95	40.86	30.47	49.74	2042.10	3343.58
7	76.833	10	0.92	16.2	11.02	-7.27	18.3	24.21	40.20	54.68	89.94	1738.91	2887.66
6	66.833	10	0.88	15.5	10.54	-7.27	17.8	23.15	39.10	77.84	129.04	1431.63	2417.75
5	56.833	10	0.84	14.8	10.06	-7.27	17.3	22.10	38.00	99.93	167.04	1145.45	1969.80
4	46.833	10	0.79	13.9	9.45	-7.27	16.7	20.76	36.73	155.62	203.77	868.40	1536.48
3	36.833	10	0.74	13.0	8.84	-7.27	16.1	19.42	35.39	140.11	239.16	618.16	1126.53
2	26.833	10	0.63	11.1	7.55	-7.27	14.8	16.59	32.56	156.70	271.72	362.10	710.78
1	14.833	12	0.56	9.87	6.71	-7.27	14.0	17.69	36.85	174.38	308.57	156.24	325.51
B	0	14.833	0	0	0	0	0	0	0	174.38	308.57	0	0

Σ Windward Story Shear =	174.38	kips
Σ Total Story Shear =	308.57	kips
Σ Windward Moment =	8884.63	ft-k
Σ Total Moment =	15157.26	ft-k

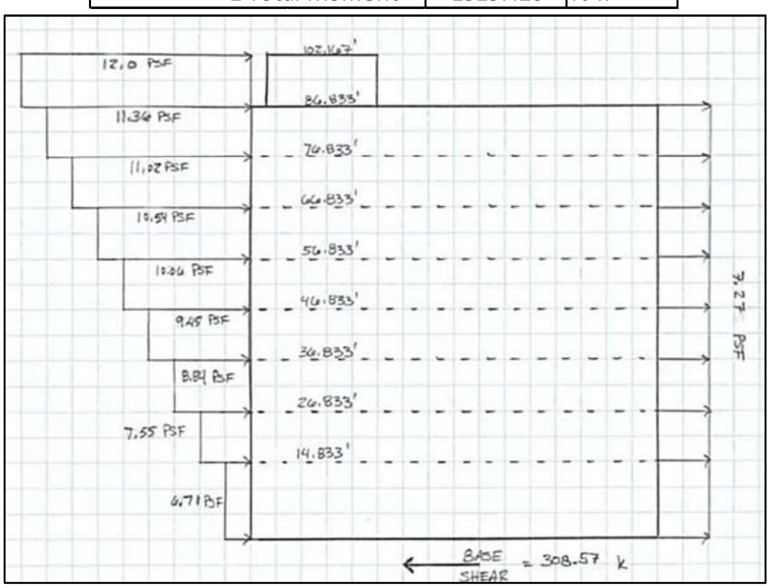


Figure 5.2

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 5.3 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 5.5 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	12.0	-2.91	14.9	5.52	6.850	5.52	6.850	521.64	647.33
Roof	86.833	10	0.95	16.7	11.36	-2.91	14.3	10.32	12.97	15.84	19.820	844.89	1061.39
7	76.833	10	0.92	16.2	11.02	-2.91	13.9	10.02	12.66	25.86	32.481	719.44	909.48
6	66.833	10	0.88	15.5	10.54	-2.91	13.5	9.58	12.23	35.44	44.710	592.52	756.11
5	56.833	10	0.84	14.8	10.06	-2.91	13.0	9.15	11.80	44.59	56.5	474.27	611.40
4	46.833	10	0.79	13.9	9.45	-2.91	12.4	8.59	11.24	53.19	67.744	359.49	470.17
3	36.833	10	0.74	13.0	8.84	-2.91	11.8	8.04	10.68	61.22	78.427	255.84	340.06
2	26.833	10	0.63	11.1	7.55	-2.91	10.5	6.86	9.51	68.09	87.935	149.83	207.59
1	14.833	12	0.56	9.87	6.71	-2.91	9.6	7.32	10.50	75.41	98.432	64.68	92.72
B	0	14.833	0	0	0	0	0	0	0	75.41	98.43	0	0

Σ Windward Story Shear =	75.41	kips
Σ Total Story Shear =	98.43	kips
Σ Windward Moment =	3982.60	ft-k
Σ Total Moment =	5096.26	ft-k

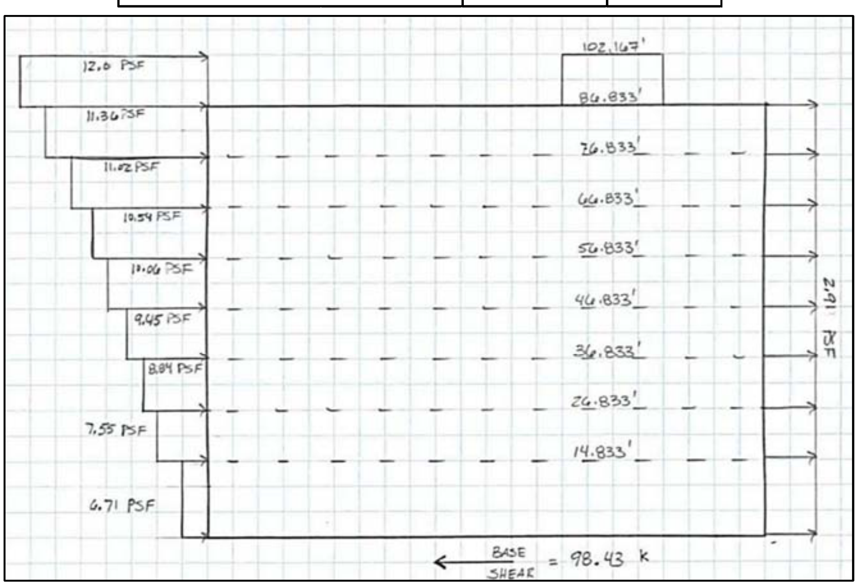


Figure 5.3
East/West Wind Pressures

Wind Load Diagrams

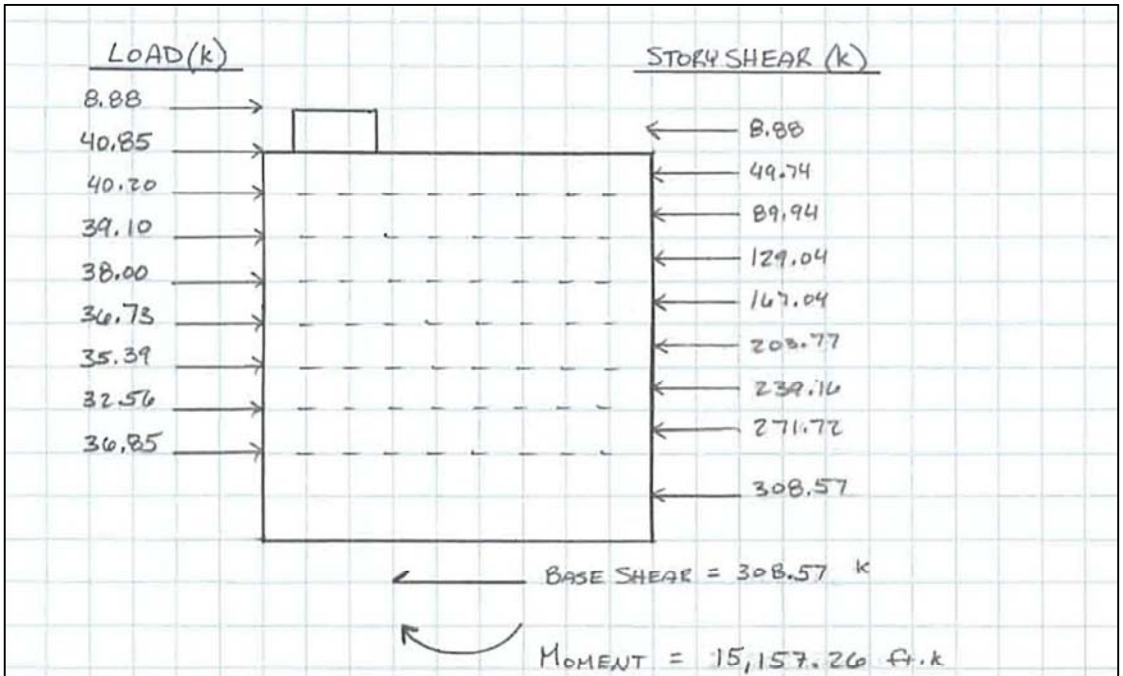


Figure 5.4
North/South Wind Loading Diagram

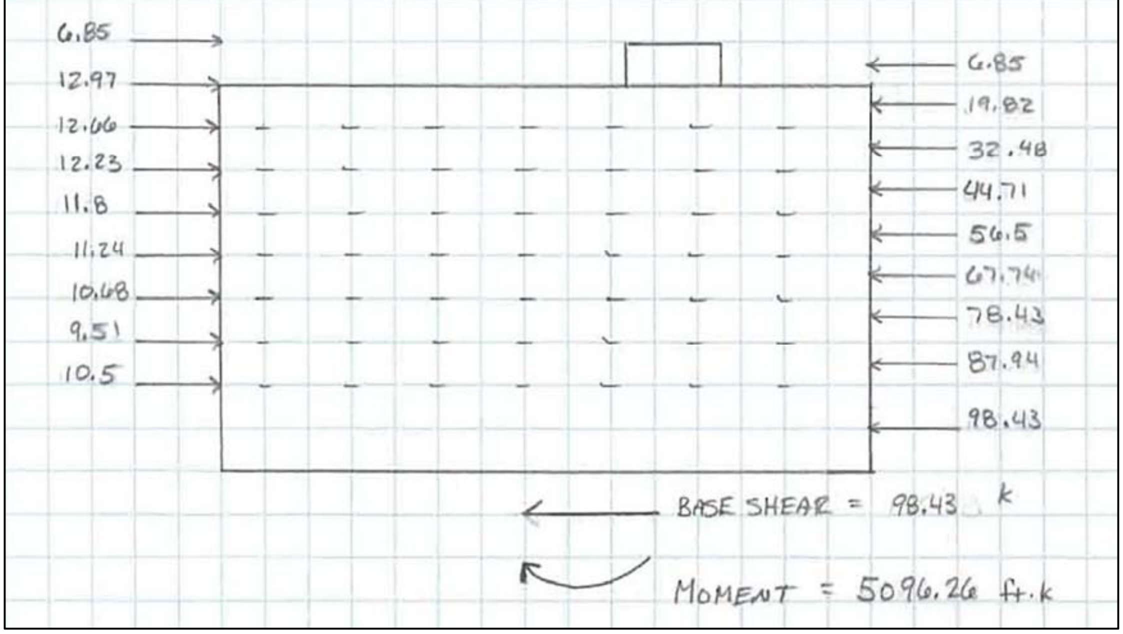


Figure 5.5
East/West Wind Loading Diagram

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 B 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 6.1)

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.

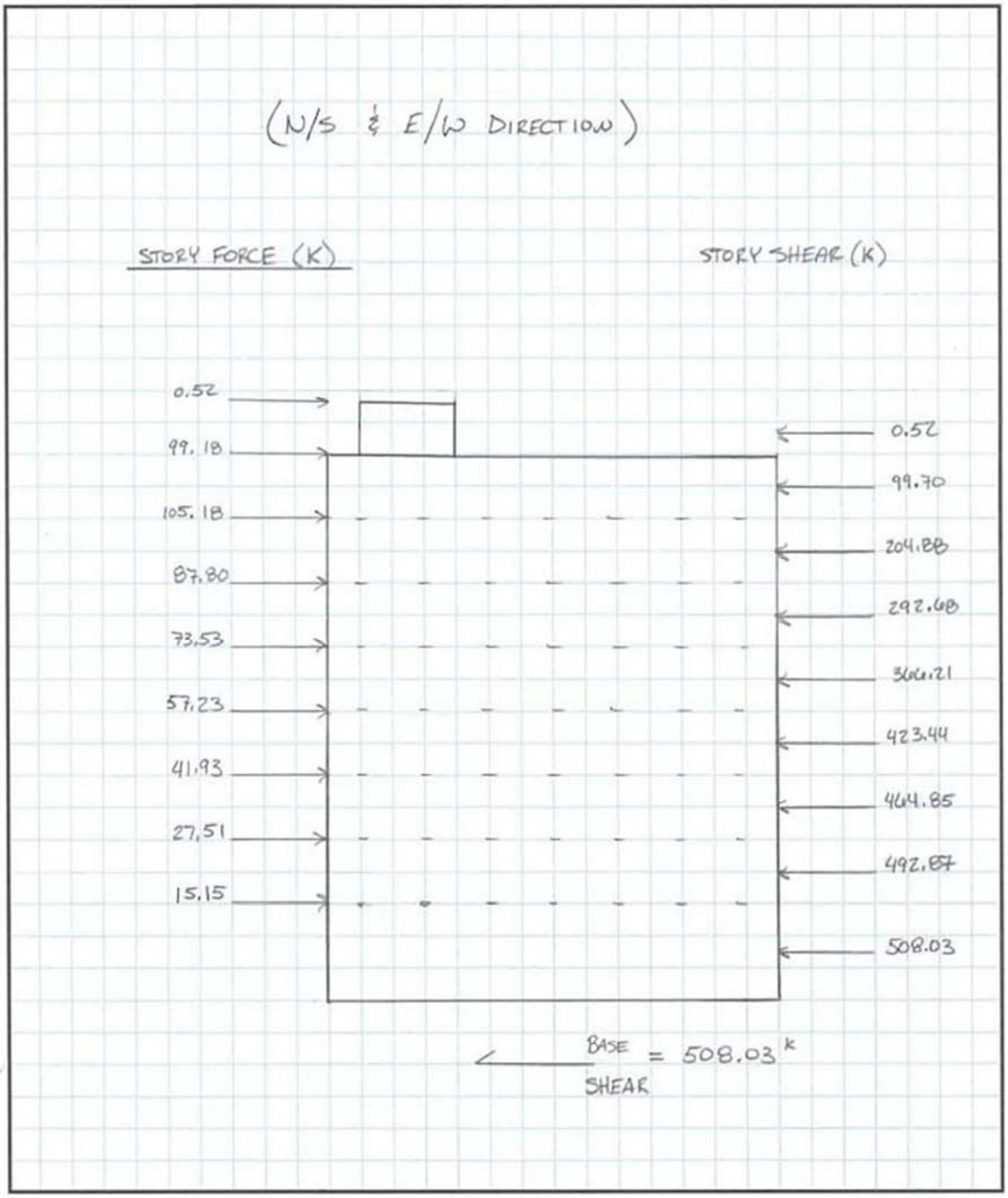
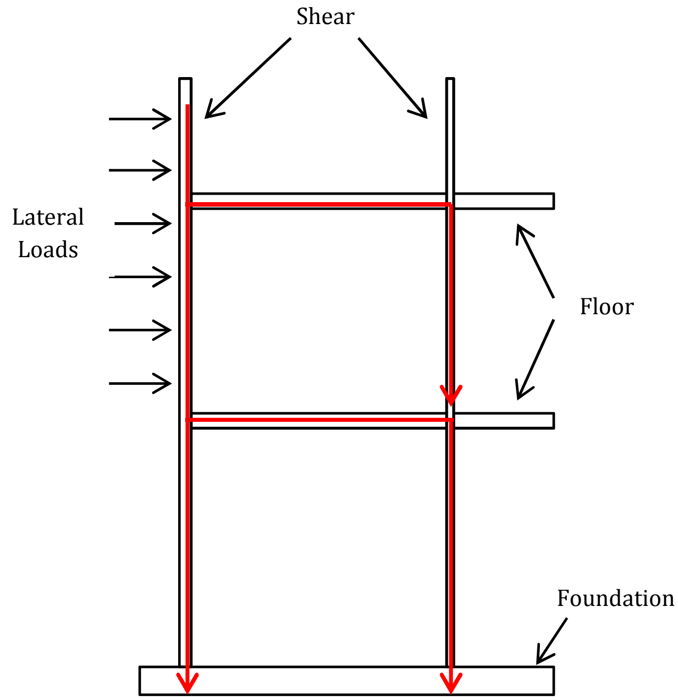


Figure 6.1
Seismic Loading Diagram

Load Distribution

Load Path

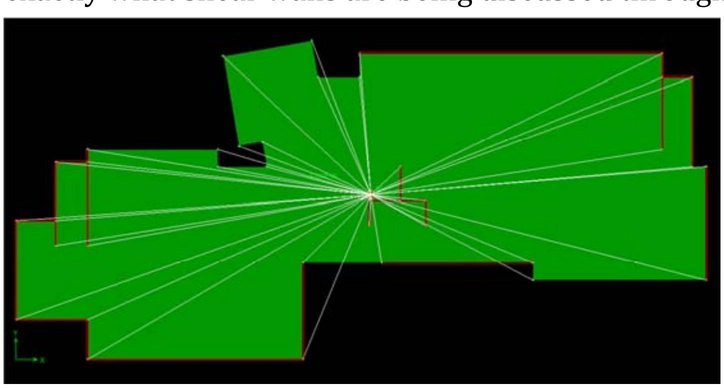
Lateral force resisting systems transfer lateral loads (wind and/or seismic) to the building's foundation where the loads dissipate. This load path is assumed to be governed by the concept of relative stiffness, which states that the most rigid members in a building draw the most forces to them. In the case of Cambria Suites Hotel, the lateral forces come in contact with the exterior of the building, are then transmitted through the rigid diaphragms, to the masonry shear walls, and lastly down into the foundation (grade beams and caissons). This load path is shown in Figure 7.1. The exterior shear walls with longer spans resist the majority of the lateral forces because they have minimal assistance from the slab. The steel frame which extends through the middle of the building only transfers gravity loads to the foundation.



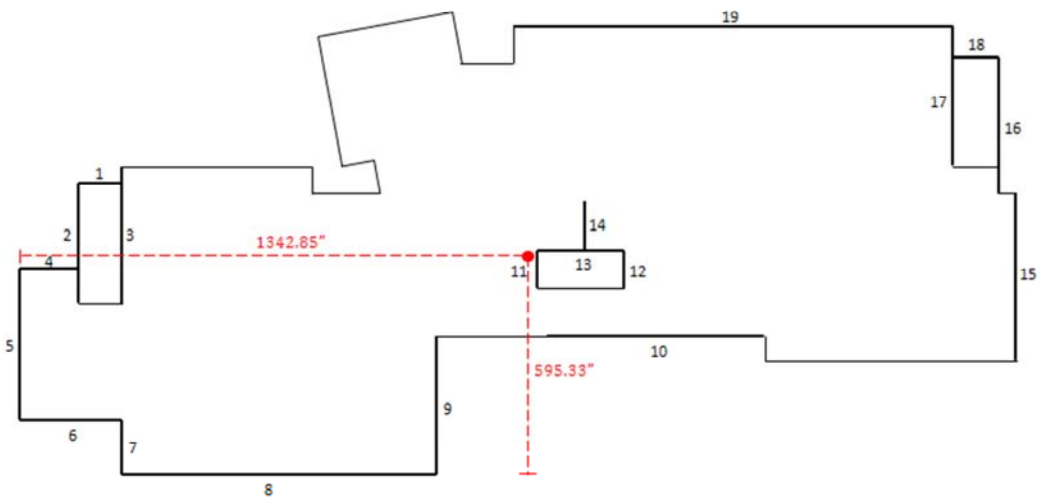
Load Path Diagram
Figure 7.1

Center of Rigidity & Mass

Every concrete masonry wall in the Cambria Suites Hotel is essentially a shear wall because they are all reinforced and grouted. For this assignment, the shear walls analyzed consisted of walls with minimal or no openings for windows. For organization purposes, Figure 7.3 was created which assigns a number to each shear wall to better reference exactly what shear walls are being discussed throughout the analysis. All shear walls are 8" thick but vary in length and are located at different distances from the center of rigidity. The rigidity of each wall is based on the thickness, height of wall from base, and length of wall. Figure 7.2 shows the center of mass of Cambria Suites Hotel.



ETABS Center of Mass
Figure 7.2



Lateral Shear Wall System

Figure 7.3

Individual wall rigidities are shown in Tables in Appendix C. Table 9a provides the rigidities for walls spanning in the North/South direction, whereas Table 9b provides the rigidities for the East/West walls. The rigidities of each wall were calculated using the following equation:

$$R = \frac{Et}{4\left(\frac{H}{L}\right)^3 + 3\left(\frac{H}{L}\right)}$$

The rigidities of each wall can then be used to determine the center of rigidity of each floor using the following equation:

$$Center\ of\ Rigidity = \frac{\Sigma[(R)(Distance\ between\ origin\ and\ element)]}{\Sigma R}$$

The centers of rigidity for each floor can be found in Table 3a. Since Cambria Suites Hotel has a non-rectangular floor plan, the center of mass was taken from the ETABS model and is consistent throughout every floor. Center of rigidity values differ from the hand calculations and the ETABS model because the hand calculations only account for the shear walls, whereas the ETABS model takes into account floor diaphragms. Hand calculations will be used whenever center of rigidity is needed for other calculations. Detailed calculations can be found in Appendix C.

Table 3a - ETABS vs. Hand Calculation Comparison						
	Center of Rigidity				Center of Mass	
	ETABS Calculation		Hand Calculation		ETABS Calculation	
	X	Y	X	Y	X	Y
Story 7	893.223	637.406	1413.98	732.85	1349.669	624.978
Story 6	925.841	637.857	1411.72	720.58	1349.669	624.978
Story 5	987.077	635.834	1408.58	705.92	1349.669	624.978
Story 4	1055.513	629.144	1404.10	688.56	1349.669	624.978
Story 3	1123.123	618.645	1397.48	668.42	1349.669	624.978
Story 2	1183.795	604.869	1387.43	645.84	1349.669	624.978
Story 1	1230.503	587.569	1371.96	621.87	1349.669	624.978
Plaza	1253.537	564.343	1342.86	595.33	1349.669	624.978

Relative Stiffness

Relative stiffness is the percentage of lateral force that is being distributed into each shear wall. Since the wall rigidities were determined, we can use them to find the relative stiffness of each wall at each floor using the following equation:

$$Relative\ Stiffness = \frac{R}{\Sigma R}$$

The values for the North/South walls at every floor can be found in Table 3b. The values for East/West walls at every floor can be found in Table 3c. Appendix C will show detailed calculations for the relative stiffness of walls. The relative stiffness of each wall will help determine how much of the load each wall will have to resist.

Table 3b - Relative Stiffness (%)											
North/South Force											
Floor Level	Wall 2	Wall 3	Wall 5	Wall 7	Wall 9	Wall 11	Wall 12	Wall 14	Wall 15	Wall 16	Wall 17
7	9.1	13.6	14.9	1.0	13.6	0.3	0.3	0.6	21.7	11.2	13.6
6	9.2	13.6	14.9	1.0	13.6	0.3	0.3	0.7	21.5	11.3	13.6
5	9.3	13.6	14.9	1.1	13.6	0.3	0.3	0.7	21.3	11.3	13.6
4	9.4	13.7	14.9	1.1	13.7	0.3	0.3	0.7	21.0	11.4	13.7
3	9.5	13.7	14.8	1.2	13.7	0.3	0.3	0.8	20.5	11.5	13.7
2	9.8	13.7	14.7	1.3	13.7	0.3	0.3	0.9	19.9	11.7	13.7
1	10.2	13.7	14.6	1.6	13.7	0.4	0.4	1.1	18.8	11.9	13.7
B	10.8	13.4	14.1	2.5	13.4	0.8	0.8	1.8	17.0	12.1	13.4

Table 3c - Relative Stiffness (%)								
East/West Force								
Floor Level	Wall 1	Wall 4	Wall 6	Wall 8	Wall 10	Wall 13	Wall 18	Wall 19
7	0.1	0.2	1.4	27.6	11.4	0.7	0.1	58.4
6	0.1	0.3	1.5	28.1	12.0	0.8	0.1	57.1
5	0.2	0.3	1.7	28.6	12.7	0.9	0.1	55.5
4	0.2	0.4	2.0	29.1	13.7	1.0	0.2	53.5
3	0.2	0.4	2.4	29.5	14.8	1.3	0.2	51.2
2	0.3	0.6	3.0	29.7	16.2	1.6	0.3	48.3
1	0.5	0.9	4.1	29.3	17.5	2.3	0.4	44.9
B	1.1	1.9	6.2	27.6	18.3	4.1	1.0	39.9

Torsion

Torsion is present when the center of rigidity and the center of mass do not occur at the same location. Eccentricity (the distance between the center of rigidity and center of mass) induces a moment, which creates an additional force on the building called torsional shear. When determining the torsional effects on the building, two different types of torsional moment need to be taken into account. According to ASCE 7-05, torsion for rigid diaphragms is the sum of the inherent torsional moment and the accidental torsional moment. The inherent torsional moment, M_t, is a result from the eccentricity between the locations of the center of rigidity and center of mass. This eccentricity times the lateral force at the specified floor level will give the inherent torsional moment. The accidental torsional moment, M_{ta}, is caused by an assumed displacement of the center of mass. This displacement is equal to 5% of the center of mass dimension each way from the actual location perpendicular to the direction of the applied force. Torsional moments produced can be seen in Tables 4a and 4b. Appendix D shows detailed calculations for building torsion.

Table 4a - Overall Building Torsion					
North/South Direction					
Story Level	Factored Lateral Force (k)	COR-COM (ft.)	M_t (ft-k)	M_{ta} (ft-k)	$M_{t,tot}$ (ft-k)
Story 7	65.4	5.4	353.2	738.7	1091.9
Story 6	64.3	5.2	334.4	726.8	1061.2
Story 5	62.6	4.9	306.7	707.0	1013.7
Story 4	60.8	4.5	273.6	687.1	960.7
Story 3	58.8	4.0	235.2	664.1	899.3
Story 2	56.6	3.1	175.5	639.8	815.3
Story 1	52.1	1.9	98.9	588.6	687.5
Plaza	59.0	-0.6	-35.4	666.3	630.9
				Total:	7160.5

Table 4b - Overall Building Torsion					
East/West Direction					
Floor Level	Factored Lateral Force (k)	COR-COM (ft.)	M_t (ft-k)	M_{ta} (ft-k)	$M_{t,tot}$ (ft-k)
Story 7	12.97	9.0	116.6	67.4	184.0
Story 6	12.66	8.0	100.9	65.8	166.7
Story 5	12.23	6.7	82.5	63.6	146.1
Story 4	11.80	5.3	62.5	61.3	123.8
Story 3	11.24	3.6	40.7	58.4	99.1
Story 2	10.68	1.7	18.6	55.6	74.1
Story 1	9.51	-0.3	-2.5	49.4	47.0
Plaza	10.50	-2.5	-25.9	54.6	28.6
				Total:	869.5

Shear

The overall shear force is the combination of direct and torsional shear. Direct shear forces relate to the relative stiffness of the shear walls, whereas the torsional shear forces relate to the torsional moments produced on each floor which results from the wind or seismic loads.

Direct Shear

Direct shear is the distribution of the lateral forces among the shear walls at each level of the building. The greater the stiffness of a shear wall, the greater the load the wall can resist. Tables 5a and 5b show the direct shears applied to each wall for each floor level. Detailed calculations for obtaining the direct shear for the North/South and East/West direction may be found in Appendix E.

Table 5a - North/South Direct Shear													
Load Combination $1.2D+1.6W+1.0L+0.5L_r$	Force (k)	Factored Force (k)	Distributed Force (k)										
			Wall 2	Wall 3	Wall 5	Wall 7	Wall 9	Wall 11	Wall 12	Wall 14	Wall 15	Wall 16	Wall 17
Roof	40.86	65.4	5.97	8.91	9.76	0.66	8.91	0.16	0.16	0.42	14.18	7.35	8.91
Floor 7	40.20	64.3	5.90	8.77	9.59	0.66	8.77	0.16	0.16	0.42	13.85	7.25	8.77
Floor 6	39.10	62.6	5.79	8.54	9.32	0.66	8.54	0.17	0.17	0.42	13.33	7.09	8.54
Floor 5	38.00	60.8	5.70	8.31	9.04	0.67	8.31	0.17	0.17	0.43	12.77	6.93	8.31
Floor 4	36.73	58.8	5.60	8.04	8.72	0.69	8.04	0.18	0.18	0.45	12.07	6.77	8.04
Floor 3	35.39	56.6	5.55	7.75	8.35	0.74	7.75	0.19	0.19	0.48	11.25	6.61	7.75
Floor 2	32.56	52.1	5.31	7.12	7.59	0.83	7.12	0.22	0.22	0.55	9.81	6.20	7.12
Floor 1	36.85	59.0	6.37	7.91	8.29	1.48	7.91	0.46	0.46	1.04	10.01	7.14	7.91

Table 5b - East/West Direct Shear										
Load Combination 0.9D+1.0E	Force (k)	Factored Force (k)	Distributed Force (k)							
			Wall 1	Wall 4	Wall 6	Wall 8	Wall 10	Wall 13	Wall 18	Wall 19
Roof	12.97	12.97	0.02	0.03	0.18	3.58	1.48	0.09	0.01	7.58
Floor 7	12.66	12.66	0.02	0.03	0.19	3.55	1.52	0.10	0.02	7.23
Floor 6	12.23	12.23	0.02	0.04	0.21	3.49	1.56	0.11	0.02	6.79
Floor 5	11.80	11.80	0.02	0.04	0.23	3.43	1.61	0.12	0.02	6.31
Floor 4	11.24	11.24	0.03	0.05	0.26	3.31	1.67	0.14	0.02	5.75
Floor 3	10.68	10.68	0.04	0.06	0.32	3.17	1.73	0.18	0.03	5.16
Floor 2	9.51	9.51	0.05	0.09	0.39	2.79	1.67	0.22	0.04	4.27
Floor 1	10.50	10.50	0.12	0.20	0.65	2.90	1.92	0.43	0.10	4.19

Torsional Shear

A torsional shear force is present on the building due to the torsional moments produced on each floor caused by the eccentricity. Thus, each shear wall will have to resist this additional force. The total torsional shear at each wall is dependent on the relative stiffness of each shear wall. The greater the relative stiffness, the greater the shear force on that wall. To determine the torsional shear, the following equation is used:

$$T = \frac{V_{tot} e d_i R_i}{J}$$

- V_{tot} = total story shear
- e = eccentricity
- d_i = distance from center of rigidity to shear wall
- R_i = relative stiffness of shear wall
- J = torsional moment of inertia

The torsional shear forces were determined for the shear walls supporting floor 4 and can be found in Table 6a. Detailed calculations for obtaining the torsional shear can be found in Appendix E.

Table 6a - Torsional Shear in Shear Walls Supporting Floor 4							
		Factored Story Shear V_{tot} (k)	Relative Stiffness R_i	Distance from COM to COR e (in)	Distance from Wall i to COR d_i (in)	$(R_i)(d_i^2)$	Torsional Shear (k)
Wall 1	E/W	108.39	0.002	63.6	61.4	7.5	0.001
Wall 2	N/S	326.03	0.095	54.4	1254.0	149389.0	1.371
Wall 3	N/S	326.03	0.137	54.4	1132.2	175617.1	1.785
Wall 4	E/W	108.39	0.004	63.6	160.6	103.2	0.003
Wall 5	N/S	326.03	0.148	54.4	1404.1	291781.5	2.391
Wall 6	E/W	108.39	0.024	63.6	538.6	6961.1	0.058
Wall 7	N/S	326.03	0.012	54.4	1132.2	15382.0	0.156
Wall 8	E/W	108.39	0.295	63.6	688.6	139863.9	0.908
Wall 9	N/S	326.03	0.137	54.4	312.1	13344.7	0.492
Wall 10	E/W	108.39	0.148	63.6	322.6	15398.7	0.214
Wall 11	N/S	326.03	0.003	54.4	60.1	10.8	0.002
Wall 12	N/S	326.03	0.003	54.4	155.9	72.9	0.005
Wall 13	E/W	108.39	0.013	63.6	85.6	95.2	0.005
Wall 14	N/S	326.03	0.008	54.4	84.9	57.6	0.008
Wall 15	N/S	326.03	0.205	54.4	1223.9	307075.9	2.887
Wall 16	N/S	326.03	0.115	54.4	1169.9	157396.6	1.548
Wall 17	N/S	326.03	0.137	54.4	1055.9	152744.7	1.665
Wall 18	E/W	108.39	0.002	63.6	385.4	297.1	0.003
Wall 19	E/W	108.39	0.512	63.6	475.4	115714.6	1.089
Torsional Moment of Inertia $J = \Sigma(R_i)(d_i^2) =$						1541314.2	

Shear Strength Check

In order to verify if there is sufficient reinforcement in the shear walls, a shear strength check must be performed. According to ACI 318-08, the shear strength of a reinforced concrete masonry shear wall can be obtained by the following equation:

$$V_n = A_{cv}[\alpha_c \lambda \sqrt{f'c} + \rho_t f_y]$$

The shear wall strength checks performed for walls supporting floor 4 can be found in table 7a. Each shear wall was within the capacity determined by the shear strength which verifies that the masonry reinforcement is adequately designed. Detailed calculations for shear strength can be found in Appendix E.

Table 7a - Shear Wall Strength Check												
Supporting Floor 4												
Floor **	Direct Shear (k)	Torsional Shear (k)	V_u (k)	Vertical Reinforcement	Spacing (in)	Length (in)	Thicknes s (in)	A_{cv} (in^2)	α_c	ρ_t	ΦV_n (k)	
Wall 1	0.08	0.001	0.08	(1) #5	24	122	8	976	2	0.001615	136.3846	OK
Wall 2	23.36	1.371	24.73	(1) #5	24	318	8	2544	2	0.001615	355.4942	OK
Wall 3	34.52	1.785	36.31	(1) #5	24	366	8	2928	2	0.001615	409.1537	OK
Wall 4	0.14	0.003	0.15	(1) #5	24	150	8	1200	2	0.001615	167.6859	OK
Wall 5	37.72	2.391	40.11	(1) #5	24	378	8	3024	2	0.001615	422.5686	OK
Wall 6	0.81	0.058	0.87	(1) #5	24	271	8	2168	2	0.001615	302.9526	OK
Wall 7	2.65	0.156	2.81	(1) #5	24	150	8	1200	2	0.001615	167.6859	OK
Wall 8	14.06	0.908	14.96	(1) #5	8	820	8	6560	2	0.004844	1869.933	OK
Wall 9	34.52	0.492	35.01	(1) #5	24	366	8	2928	2	0.001615	409.1537	OK
Wall 10	6.17	0.214	6.38	(1) #5	24	576	8	4608	2	0.001615	643.914	OK
Wall 11	0.66	0.002	0.67	(1) #5	24	94	8	752	2	0.001615	105.0832	OK
Wall 12	0.66	0.005	0.67	(1) #5	24	94	8	752	2	0.001615	105.0832	OK
Wall 13	0.42	0.005	0.43	(1) #5	24	216	8	1728	2	0.001615	241.4678	OK
Wall 14	1.70	0.008	1.71	(1) #5	24	129	8	1032	2	0.001615	144.2099	OK
Wall 15	54.12	2.887	57.01	(1) #5	24	432	8	3456	2	0.001615	482.9355	OK
Wall 16	28.62	1.548	30.17	(1) #5	24	342	8	2736	2	0.001615	382.324	OK
Wall 17	34.52	1.665	36.19	(1) #5	24	366	8	2928	2	0.001615	409.1537	OK
Wall 18	0.06	0.003	0.07	(1) #5	24	114	8	912	2	0.001615	127.4413	OK
Wall 19	27.91	1.089	29.00	(1) #5	8	1152	8	9216	2	0.004844	2627.028	OK

Drift and Displacement

The overall drift is a concern for nonstructural members and should be limited as much as possible. Building drift and deformation becomes a larger factor as the height of the building increases. According to IBC 2006, wind load drift is limited to an allowable drift of $\Delta = \ell/400$, whereas the seismic drift is limited to an allowable drift of $\Delta = 0.02h_{sx}$. Wind controls the drift in the North/South direction of the building and the seismic forces control the drift in the East/West direction. The allowable building drift limit for Cambria Suites Hotel will be:

$$\Delta limit = 1042"/400 = 2.605"$$

In order to determine the overall building drift, the displacement and story drift of each individual floor will be summed. The following equation was used to determine the overall building drift:

$$\Delta cantilever = \Delta flexural + \Delta shear$$

Actual hand calculations used to determine the drift and displacement can be found in Tables 10a, 10b, and 10c in Appendix F.

Overturning Moments

Since lateral forces and moments are exerted on the building, overturning affects must be considered. These overturning moments are a concern due to the impact that they could potentially have on the foundation system. Therefore, a calculation must be conducted to determine if the dead load of the building will be sufficient enough to resist the impact of the overturning moments. As shown in table 8a, total overturning moments are provided due to wind and seismic loads. Note that the wind loads controlled in the North/South direction, whereas the seismic loads controlled in the East/West direction. In order to verify that the dead load was adequate to resist these overturning moments due to wind and seismic loads, the stresses due to the lateral loads were compared to the stresses due to the self-weight of the building. It was concluded that the stresses due to the lateral loads were such a small fraction of the stresses due to the dead loads; thus the foundation will experience minimal overturning affects. However, a force will be present along the perimeter of the building due to the moment exerted on the structure. Detailed calculations for overturning moments can be found in Appendix G.

Table 8a - Overturning Moments						
Floor	Height Above Ground Z (ft)	Story Height (ft)	N/S Wind Forces		E/W Seismic Forces	
			Lateral Force F_x (k)	Total Moment M_x (ft-k)	Lateral Force F_x (k)	Total Moment M_x (ft-k)
PH Roof	102.167	15.333	8.88	839.16	0.52	48.78
Roof	86.833	10	40.86	3343.58	99.18	8116.49
7	76.833	10	40.20	2887.66	105.18	7555.37
6	66.833	10	39.10	2417.75	87.80	5429.14
5	56.833	10	38.00	1969.80	73.53	3811.07
4	46.833	10	36.73	1536.48	57.23	2393.96
3	36.833	10	35.39	1126.53	41.93	1334.71
2	26.833	10	32.56	710.78	27.51	573.04
1	14.833	12	36.85	325.51	15.15	112.39
Plaza	0	14.833	0	0	0	0
		Total =	308.57	15157.26	508.03	29374.97

Conclusion

After modeling the lateral force resisting system of Cambria Suites Hotel in ETABS and completing a thorough analysis of the system, the following conclusions were made:

- Upon evaluating the basic load combinations as defined in ASCE 7-05, it was determined through ETABS that the load case 1.2D+1.6W+1.0L+0.5L_r controls in the North/South direction, whereas 0.9D+1.0E controls East/West direction.

- Before evaluating the load combinations in ETABS, it was necessary to revise the wind load analysis performed in Technical Report 1. As a result of these changes, it was still found that wind loads controlled in the North/South direction due to the larger façade and the seismic loads controlled in the East/West direction.

- An ETABS model was used as a reference to verify that the model and hand calculations were providing similar and reasonable results. It was also concluded that the values computed by hand were to be used in all subsequent calculations.

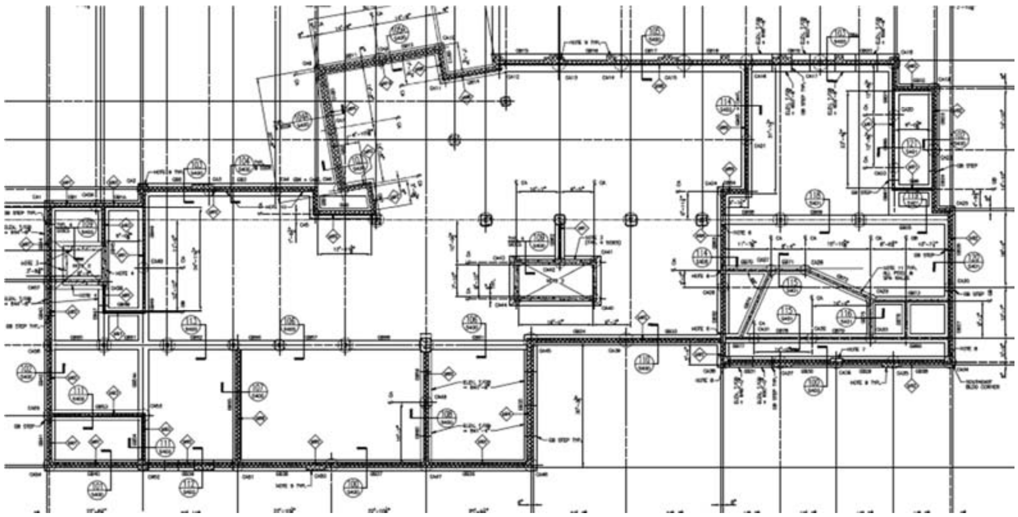
- It was found that the center of rigidity values differed from the ETABS model and hand calculations. This was due to the hand calculations only taking into account for the shear walls, whereas the computer model also included the floor diaphragm.

- Torsion was present in the building due to the eccentricity between the center of mass and rigidity. This created a torsional shear in addition to the direct shear which was already acting on the shear walls. A shear strength check was performed to determine if the reinforcement and thickness of the shear walls was designed adequately to resist the total shear.

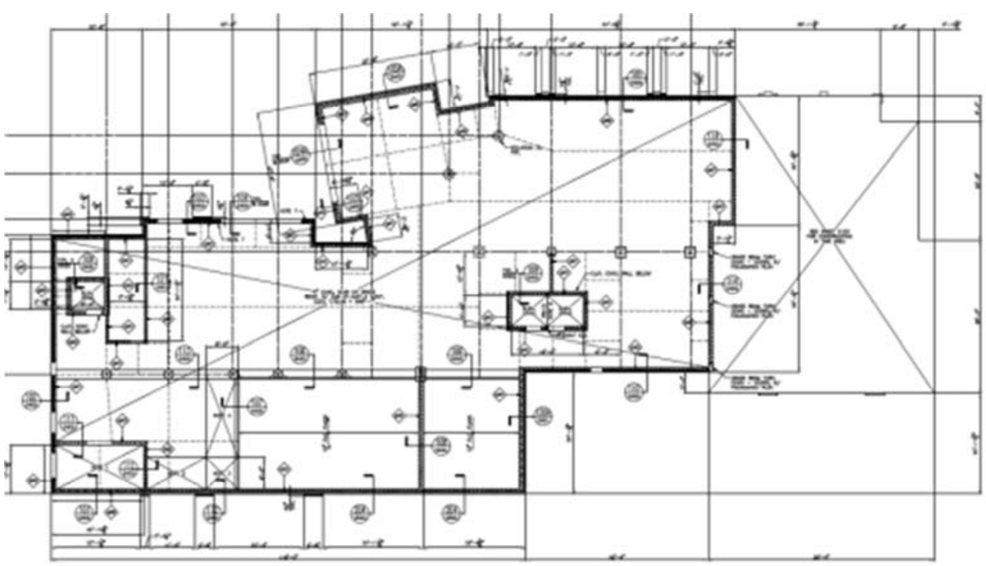
- The overall building drift was determined by ETABS and by hand calculations to be within the allowable code limitations. Although, since the calculations neglect that the interior core shear walls act as a whole, the drifts and displacements can only be an approximation.

- Overturning moments were found to be present due to the lateral loads on the building. However, a stress check was performed to determine that the self-weight of the building resists the overturning moments and the impact on the foundations due to overturning is minimal.

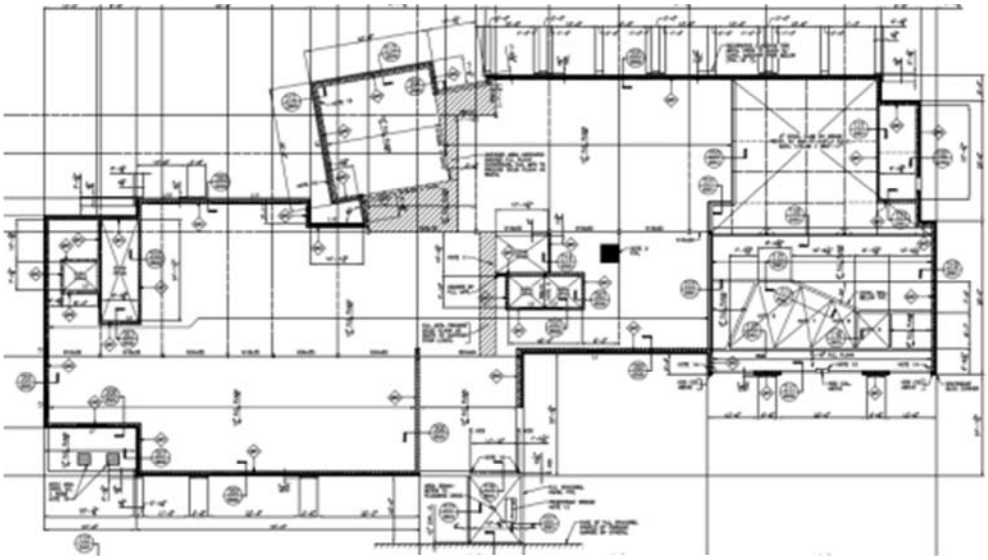
Appendix A: Building Layout



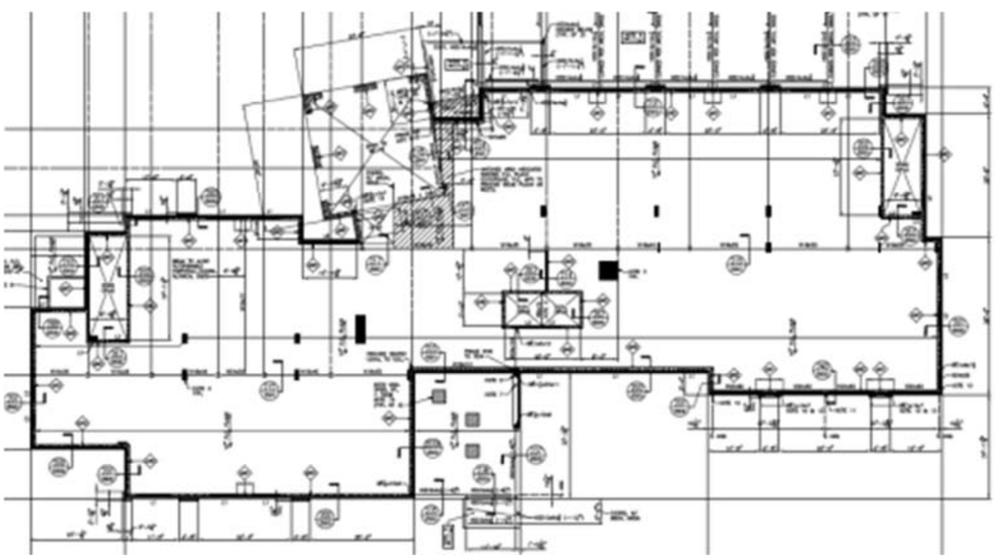
Foundation Plan



Plaza Level Framing Plan

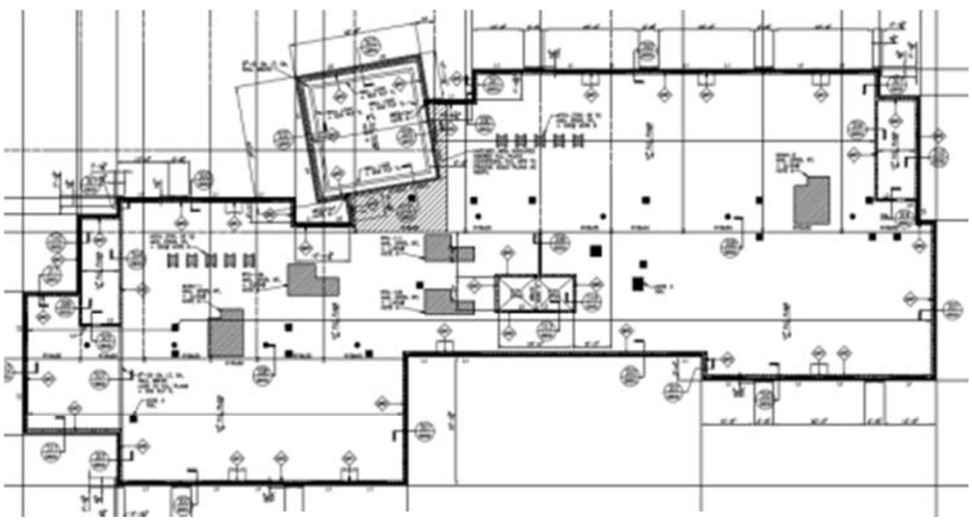


Hotel Level Framing Plan

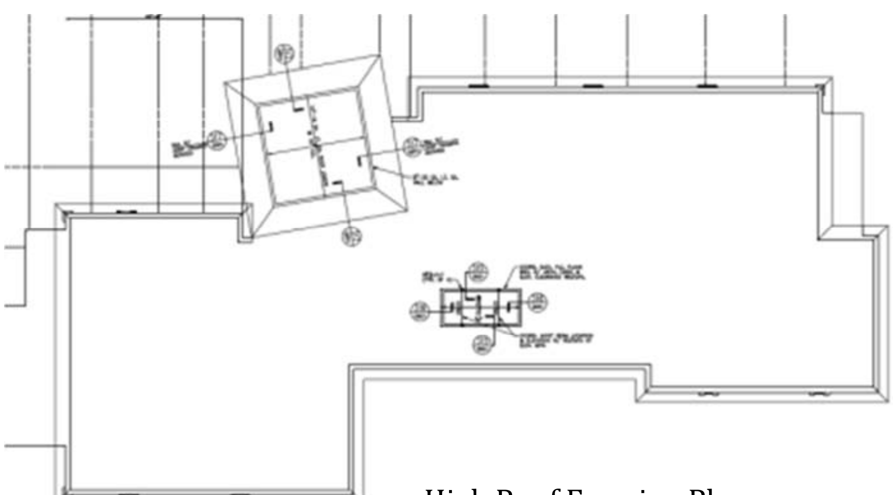


Second Level Framing Plan

Third thru Seventh Level Framing



Roof Framing Plan



High Roof Framing Plan

Appendix B: Loads

Wind Loads

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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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WIND LOADS (CONT.)

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

$\Downarrow$

$K_{zt} = 0.97$

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

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

• C_p - EXTERNAL PRESSURE COEFFICIETS

NORTH / SOUTH	EAST / WEST
WINDWARD = 0.8	WINDWARD = 0.8
LEEWARD = -0.5	LEEWARD = -0.2
L/B = 0.45	L/B = 2.22
L = 98.92' B = 219.67'	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

$p_z = 9.87(0.85)(0.8)$
$= \underline{6.71 \text{ PSF}}$

$p_h = 17.10(0.85)(-0.5)$
$= \underline{\underline{-7.27 \text{ PSF}}}$

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WIND LOADS (CONT.)

EAST/WEST EXAMPLE : @ LEVEL 1

$P_z = 9.87(0.85)(0.8)$
$= 6.71$ PSF

$P_h = 17.10(0.85)(-0.2)$
$= -2.91$ 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 $F_W = (219.67')(12')(6.71) = 17.69$ k

- FORCE OF TOTAL PRESSURE

N/S EXAMPLE : @ LEVEL 1 $F_T = (219.67')(12')(14.0\text{ PSF}) = 36.85$ k

- WINDWARD SHEAR STORY

N/S EXAMPLE : @ LEVEL 7 $F = F_W$ @ (HIGH ROOF + ROOF + 7)
$= 5.52 + 24.95 + 24.21 = 54.68$ k

- TOTAL STORY SHEAR

N/S EXAMPLE : @ LEVEL 7 $F = F_T$ @ (HIGH ROOF + ROOF + 7)
$= 8.88 + 40.86 + 40.20 = 89.94$ k

Seismic Loads

Example of Floor Weights Found

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	

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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) (FIG. 22-1 thru 22-14)
  &
  SPECTRAL RESPONSE ACCELERATION, 1s (S_1)

$$S_S = 0.125$$
$$S_1 = 0.049$$

- 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)
- $S_{DS} = \frac{2}{3}(S_{MS}) = \frac{2}{3}(0.15)$ $\boxed{S_{DS} = 0.100}$ (Eq. 11.4-3)
- $S_{M1} = F_v S_1 = 1.7(0.049)$ $\boxed{S_{M1} = 0.0833}$ (Eq. 11.4-2)
- $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.0833)$ $\boxed{S_{D1} = 0.055}$ (Eq. 11.4-4)

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SEISMIC LOADS (cont.)

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

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

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

- $C_s =$ MIN.
  $\frac{S_{D1}}{T(R/I)} = \frac{0.055}{1.09(2/1)} = \boxed{0.025} \geq 0.01$
  $\frac{S_{DS}}{(R/I)} = \frac{0.100}{(2/1)} = 0.05 \geq 0.01$ AES used this value in their calculations
  $\frac{S_{D1} T_L}{T^2(R/I)} = \frac{0.055(12)}{(1.09)^2(2/1)} = 0.278 \geq 0.01$

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

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

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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)

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

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

- 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 : $w_x = 2829.8^k$, $h_x = 14.833'$, $k = 1.295$

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

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SEISMIC LOADS (cont.)

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

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

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

• $F_x = C_{vx}(V)$ (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 C: Load Distribution

Rigidity/Relative Stiffness

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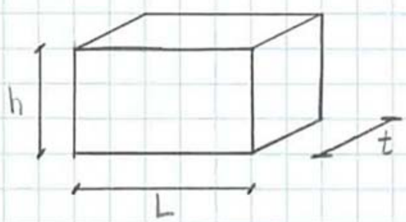
• Rigidity (Y-coord.) : Plaza Floor

$$R = \frac{E \cdot t}{4(h/L)^3 + 3(h/L)}$$

$$E = 33w^{1.5}\sqrt{f'_c} = 33(145\text{ PCF})^{1.5}\sqrt{2000\text{ PSI}} = 2.577 \times 10^6\text{ PSI}$$

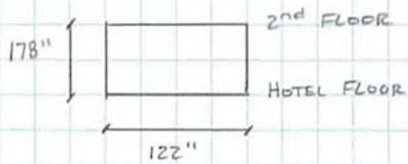
or

2577 KSI (All Floors)



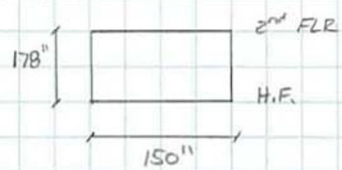
t = 8"
h = height from base to each level (varies)
L = length of wall element

Wall 1 :



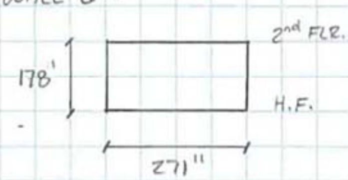
$$R_{1-1} = \frac{(2577\text{ KSI})(8")}{4\left(\frac{178}{122}\right)^3 + 3\left(\frac{178}{122}\right)} = 1227$$

Wall 4 :



$$R_{4-1} = \frac{(2577\text{ KSI})(8")}{4\left(\frac{178}{150}\right)^3 + 3\left(\frac{178}{150}\right)} = 2012$$

Wall 6 :



$$R_{6-1} = \frac{(2577\text{ KSI})(8")}{4\left(\frac{178}{271}\right)^3 + 3\left(\frac{178}{271}\right)} = 6642$$

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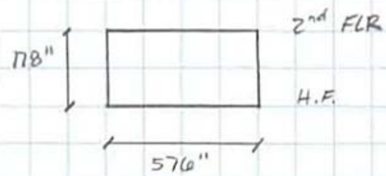
WALL 8:

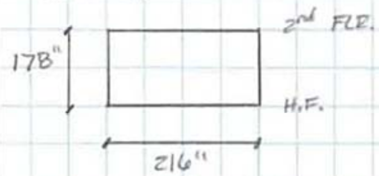
$$R_{8-1} = \frac{(2577\text{ KSI})(8")}{4\left(\frac{178}{820}\right)^3 + 3\left(\frac{178}{820}\right)} = 29{,}786$$

WALL 10:



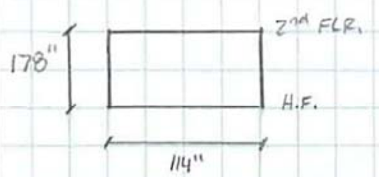
$$R_{10-1} = \frac{(2577\text{ KSI})(8")}{4\left(\frac{178}{576}\right)^3 + 3\left(\frac{178}{576}\right)} = 19{,}726$$

WALL 13:



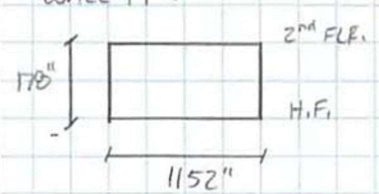
$$R_{13-1} = \frac{(2577\text{ KSI})(8")}{4\left(\frac{178}{216}\right)^3 + 3\left(\frac{178}{216}\right)} = 4376$$

WALL 18:



$$R_{18-1} = \frac{(2577\text{ KSI})(8")}{4\left(\frac{178}{114}\right)^3 + 3\left(\frac{178}{114}\right)} = 1035$$

WALL 19:



$$R_{19-1} = \frac{(2577\text{ KSI})(8")}{4\left(\frac{178}{1152}\right)^3 + 3\left(\frac{178}{1152}\right)} = 43{,}103$$

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$$\Sigma R = R_{1-1} + R_{4-1} + R_{6-1} + R_{8-1} + R_{10-1} + R_{14-1} + R_{18-1} + R_{19-1}$$
$$= 107{,}907$$

• RELATIVE STIFFNESS

$$\% = \frac{R}{\Sigma R} \times 100 \qquad \text{WALL } 1 = \frac{1227}{107{,}907} = 1.1\%$$

WALL 4 = 1.9%
WALL 6 = 6.2%
WALL 8 = 27.6%
WALL 10 = 18.3%
WALL 13 = 4.1%
WALL 18 = 1.0%
WALL 19 = 39.9%

• CENTER OF RIGIDITY

$$\frac{\Sigma R \cdot d}{\Sigma R} = \frac{1227(750) + 2012(528) + 6642(150) + 29786(4) + 19726(366) + 4376(603) + 1035(1014) + 43103(1164)}{107{,}907}$$

$$\boxed{Y\text{-COORD} = 595.33''}$$

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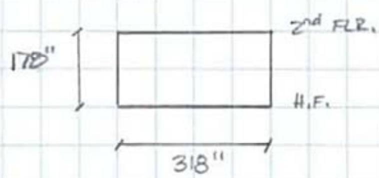
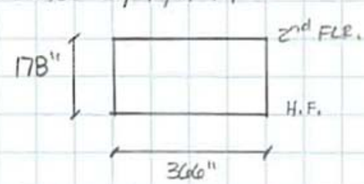
• RIGIDITY (X-COORD.): HOTEL FLOOR

WALL 2:



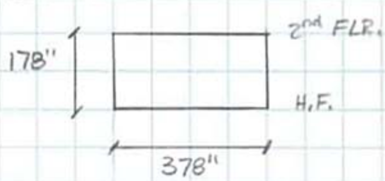
$$R_{2-1} = \frac{(2577\text{ KSI})(8'')}{4\left(\frac{178}{318}\right)^3 + 3\left(\frac{178}{318}\right)} = 8659$$

WALL 3, 9, & 17:



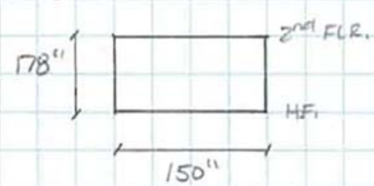
$$R_{3-1,\ 9-1,\ 17-1} = \frac{(2577\text{ KSI})(8'')}{4\left(\frac{178}{366}\right)^3 + 3\left(\frac{178}{366}\right)} = 10{,}742$$

WALL 5:



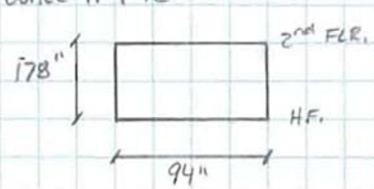
$$R_{5-1} = \frac{(2577\text{ KSI})(8'')}{4\left(\frac{178}{378}\right)^3 + 3\left(\frac{178}{378}\right)} = 11{,}263$$

WALL 7:



$$R_{7-1} = \frac{(2577\text{ KSI})(8'')}{4\left(\frac{178}{150}\right)^3 + 3\left(\frac{178}{150}\right)} = 2013$$

WALL 11 & 12



$$R_{11-1,\ 12-1} = \frac{(2577\text{ KSI})(8'')}{4\left(\frac{178}{94}\right)^3 + 3\left(\frac{178}{94}\right)} = 628$$

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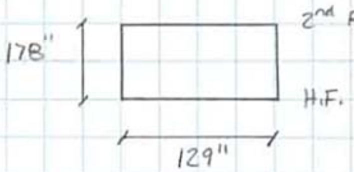
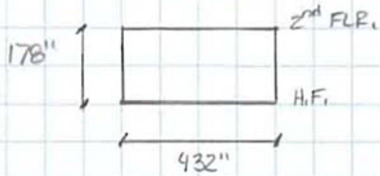
WALL 14:



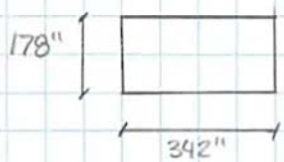
$$R_{14-1} = \frac{(2577\ \text{KSI})(8'')}{4\left(\frac{178}{129}\right)^3 + 3\left(\frac{178}{129}\right)} = 1407$$

WALL 15:



$$R_{15-1} = \frac{(2577\ \text{KSI})(8'')}{4\left(\frac{178}{432}\right)^3 + 3\left(\frac{178}{432}\right)} = 13{,}599$$

WALL 16:



$$R_{16-1} = \frac{(2577\ \text{KSI})(8'')}{4\left(\frac{178}{342}\right)^3 + 3\left(\frac{178}{342}\right)} = 9700$$

$$\Sigma R = R_{2-1} + R_{3-1} + R_{9-1} + R_{17-1} + R_{5-1} + R_{7-1} + R_{11-1} + R_{12-1} + R_{14-1} + R_{15-1} + R_{16-1}$$
$$= 80{,}123$$

• RELATIVE STIFFNESS

$$\% = \frac{R}{\Sigma R} \times 100$$

WALL 2 = 10.8 %
WALL 3,9,17 = 13.4 %
WALL 5 = 14.1 %
WALL 7 = 2.5 %
WALL 11,12 = 0.8 %
WALL 14 = 1.8 %
WALL 15 = 17.0 %
WALL 16 = 12.1 %

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• CENTER OF RIGIDITY

$$\frac{\Sigma R \cdot d}{\Sigma R} = \frac{8659(150) + 10742(272) + 10742(1092) + 10742(2460) + 11263(4) + 2013(272) + 628(1344) + 628(1560) + 1407(1489) + 13599(2628) + 9700(2574)}{80{,}123}$$

$$\boxed{\text{X- COORD} = 1342.85''}$$

• CENTER OF MASS FOR HOTEL FLOOR:

X- Coord. = 1349.669" or 112.47'
Y- Coord. = 624.978" or 52.08'

• CENTER OF RIGIDITY FOR HOTEL FLOOR:

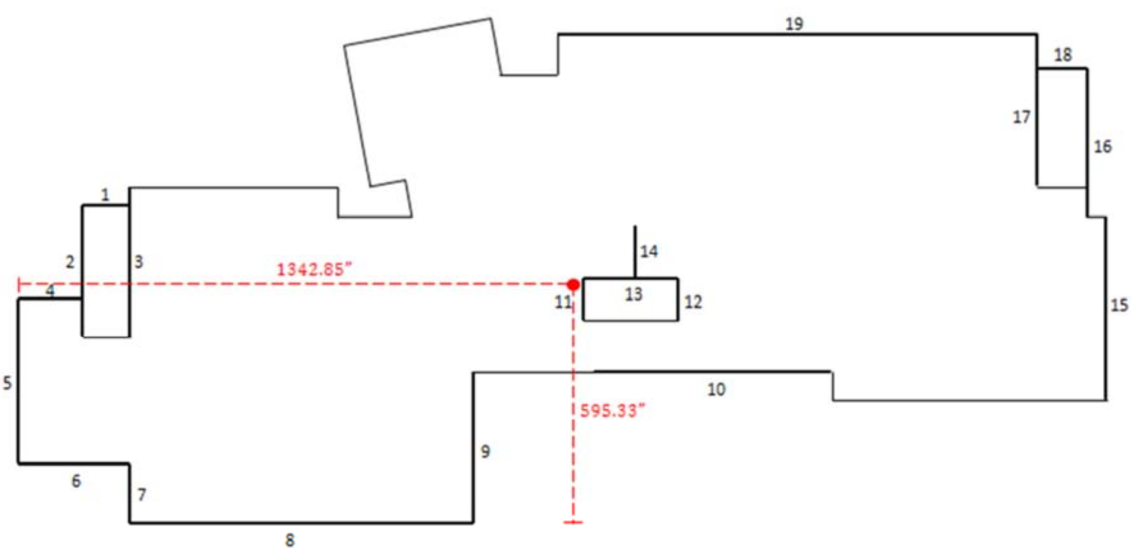

Table 9a - Wall Rigidity Calculation (N/S Span)														
Supported Floor	Height (in)	Wall 2 (L=318)	Wall 3 (L=366)	Wall 5 (L=378)	Wall 7 (L=150)	Wall 9 (L=366)	Wall 11 (L=94)	Wall 12 (L=94)	Wall 14 (L=129)	Wall 15 (L=432)	Wall 16 (L=342)	Wall 17 (L=366)	ΣRigidity	Center of Rigidity (X)
7	1042	137	204	224	15	204	4	4	10	325	169	204	1500	1413.98
6	922	194	288	315	22	288	5	5	14	455	238	288	2115	1411.72
5	802	287	424	463	33	424	8	8	21	662	352	424	3105	1408.58
4	682	449	655	713	53	655	13	13	34	1007	547	655	4795	1404.10
3	562	753	1080	1171	93	1080	24	24	60	1622	909	1080	7895	1397.48
2	442	1383	1933	2082	185	1933	48	48	120	2803	1648	1933	14115	1387.43
1	322	2867	3844	4100	448	3844	121	121	296	5296	3345	3844	28125	1371.96
B	178	8659	10742	11263	2012	10742	628	628	1407	13600	9700	10742	80125	1342.86

Table 9b - Wall Rigidity Calculation (E/W Span)											
Supported Floor	Height (in)	Wall 1 (L=122)	Wall 4 (L=150)	Wall 6 (L=271)	Wall 8 (L=820)	Wall 10 (L=576)	Wall 13 (L=216)	Wall 18 (L=114)	Wall 19 (L=1152)	ΣRigidity	Center of Rigidity (Y)
7	1042	8	15	86	1715	708	44	7	3634	6218	732.85
6	922	12	22	123	2276	972	64	10	4631	8109	720.58
5	802	18	33	183	3088	1377	95	15	5996	10805	705.92
4	682	29	53	289	4298	2023	152	24	7911	14779	688.56
3	562	51	93	492	6165	3104	263	42	10693	20904	668.42
2	442	103	185	927	9189	5017	510	85	14972	30988	645.84
1	322	253	448	2007	14516	8677	1163	213	22266	49543	621.87
B	178	1227	2012	6642	29786	19726	4376	1035	43103	107908	595.33

Appendix D: Torsion

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OVERALL BUILDING TORSION

$M_{t,tot} = M_t + M_{ta}$

Factored Lateral Force = 1.6 W
= 1.6 (Total wind pressure force @ story)

M_t = (factored lateral force)(eccentricity)

eccentricity = CoR - CoM

• Example @ Floor 4 in N/S Direction:

$e = 1404.10'' - 1349.669'' = 54.4''$ or $\underline{\underline{4.5'}}$

Factored Lateral Force = $1.6(38.00^k) = 60.8\ k$

$M_t = 60.8\ k\,(4.5') = 273.6$ ft.k

M_{ta} = (factored lateral force)(5% assumed displacement each way of CoM)

• Example @ Floor 4 in N/S Direction (ASCE 7-05, Sec. 12.8.4.2)

CoM = 1349.669''
5% displacement in each direction = 135'' or $\underline{\underline{11.3'}}$

Factored Lateral Force = 1.6 (38.0 k) = 60.8 k

$M_{ta} = 60.8\ k\,(11.3') = 687.04$ ft.k

$M_{t,tot} = M_t + M_{ta} = 273.6 + 687.04 = 960.64$ ft.k

Appendix E: Shear

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SHEAR

- Controlling Load Combinations

North | South : $1.2D + 1.6W + 1.0L + 0.5L_R$
East / West : $0.9D + 1.0E$

DIRECT SHEAR

$$\text{Direct Shear} = (\text{factored story force})\left(\frac{\text{relative stiffness \%}}{100}\right)$$

- Example for Floor 4 in N/S Direction @ Wall 2

Direct Shear = 58.8 k (0.095) = 5.59 k

TORSIONAL SHEAR

$$T = \frac{V_{TOT} \cdot e d_i R_i}{J}$$

V_{TOT} = story shear
e = distance from center of mass to center of rigidity
d_i = distance from element to center of rigidity
R_i = relative stiffness of element
J = torsional moment of inertia

- Example for Wall 7 Supporting Floor 4

- factored story shear = 1.6 (203.77 k) = 326.032 k
- CoR (X-coord) = 1404.10"
- CoM (X-coord) = 1349.669"

e = CoR - CoM = 54.431

- R_i = 0.012

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- Location of Wall 7 = 271.92" (X-coord.)

$d_i = wall_i - CoR_i = 271.92'' - 1404.10'' = -1132.18''$
$R_i \times d_i^2 = 0.012\,(-1132.18)^2 = 15{,}382$

$J = 1541314.2$

$$T = \frac{(326.032\text{ k})(54.431'')(1132.18'')(0.012)}{1541314.2}$$

$T = 0.156\text{ k}$

- calculated values of all shear walls supporting Floor 4 can be found in Table

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SHEAR STRENGTH

• ACI 318.08 (sec. 21.9.4) ⇒ structural walls shall not exceed V_n

$$\phi V_n = \phi A_{cv}(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y)$$

$\phi = 0.75$
A_{cv} = gross area of concrete
α_c = Coefficient
= 2 if $h_w / l_w > 2.0$
$\rho_t = A_v / s \cdot h$

s = shear reinforcement spacing
h = thickness of wall

• Example for wall 9 supporting Floor 4

– Direct Shear = distributed direct force on all floors above floor 4 of wall 9

From Table 5a $= 8.31 + 8.54 + 8.77 + 8.91 = 34.53^k$

– Torsional Shear (Table 6a)

$$V_u = 34.53^k + 0.492^k = 35.02^K$$

– Vertical Reinforcement : (1) #5 @ 24" o.c.

$$\rho_t = \frac{(1)(0.31)}{(24)(8)} = 0.00161$$

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$A_{cv} = (366^{"})(8^{"}) = 2928 \text{ in}^2$

$\phi V_n = 0.75\,(2928 \text{ in}^2)\left[2.0\left(\frac{\sqrt{2000}}{1000}\right) + 0.00161\,(60)\right]$

$\phi V_n = 408.6^K > 35.02^K = V_u \quad \therefore \text{OK}$

- Remaining Shear Strength Calculations are located in Table 7a.

Appendix F: Drift and Displacement

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STORY DISPLACEMENT

- Approximate the story shear by $\Sigma\Delta_{cont.}$ of each story
- Story Drift

$$\Delta = 0.020 h_{sx}$$

h_{sx} = story height below story x (ASCE 7-05, Table 12.12-1)

$$\Delta_{cont.} = \Delta_{flexural} + \Delta_{shear}$$

$$\Delta_{flexural} = \frac{Ph^3}{3E_cI}$$

$$\Delta_{shear} = \frac{1.2Ph}{E_r A}$$

$$\Delta_{cont.} = \frac{Ph^3}{3E_cI} + \frac{1.2Ph}{E_r A}$$

$$E_c = 33(145\ PCF)^{1.5}\sqrt{2000\ PSI} = 2577\ KSI \quad \text{(ALL STORIES)}$$

$$E_r = 0.4E_c = 0.4(2577) = 1031\ KSI \quad \text{(ALL STORIES)}$$

$$A = (\text{length}) \times (\text{thickness})$$

$$I = \frac{(\text{thickness}) \times (\text{length})^3}{12}$$

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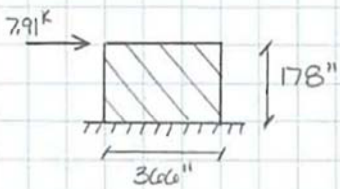
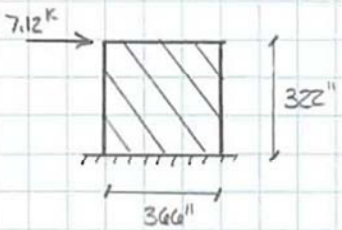
• Example for Wall 9 in N/S Direction

Floor 1 Supported:



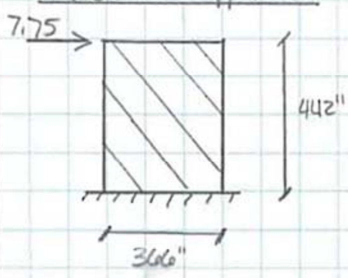
$$A = 8''(366'') = 2928 \text{ in}^2$$
$$I = 8''(366)^3/12 = 32685264 \text{ in}^4$$
$$\Delta_1 = \frac{(7.91^K)(178)^3}{3(2577)I} + \frac{1.2(7.91)(178)}{1031(2928)}$$
$$\Delta_1 = 1.77E{-4} + 5.59E{-4}$$
$$\underline{\Delta_1 = 7.36E{-4} \text{ in.}}$$

Floor 2 Supported:



$$A = 2928 \text{ in}^2$$
$$I = 32685264 \text{ in}^4$$
$$\Delta_2 = \frac{(7.12)(322)^3}{3(2577)I} + \frac{1.2(7.12)(322)}{1031(2928)}$$
$$\Delta_2 = 9.41E{-4} + 9.113E{-4}$$
$$\underline{\Delta_2 = 0.001852 \text{ in.}}$$

Floor 3 Supported:



$$A = 2928 \text{ in}^2$$
$$I = 32685264 \text{ in}^4$$
$$\Delta_3 = \frac{(7.75)(442)^3}{3(2577)I} + \frac{1.2(7.75)(442)}{1031(2928)}$$
$$\Delta_3 = 0.002648 + 0.001362$$
$$\underline{\Delta_3 = 0.004009 \text{ in.}}$$

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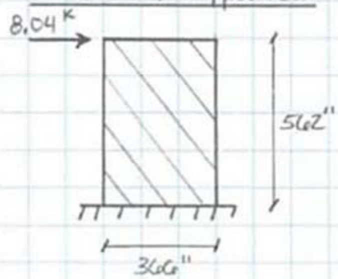
FLOOR 4 Supported



$A = 2928 \text{ in}^2$

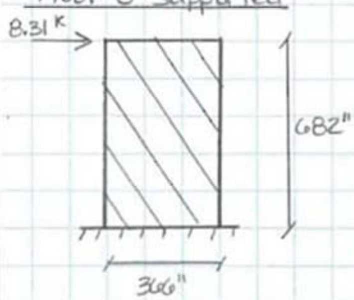
$I = 32685264 \text{ in}^4$

$$\Delta_4 = \frac{(8.04)(562)^3}{3(2577)I} + \frac{1.2(8.04)(562)}{1031(2928)}$$

$\Delta_4 = 0.005647 + 0.001796$

$\underline{\Delta_4 = 0.007443 \text{ in.}}$

Floor 5 supported



$A = 2928 \text{ in}^2$

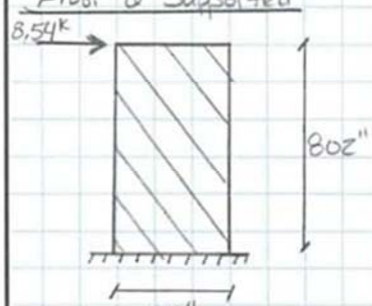
$I = 32685264 \text{ in}^4$

$$\Delta_5 = \frac{(8.31)(682)^3}{3(2577)I} + \frac{1.2(8.31)(682)}{1031(2928)}$$

$\Delta_5 = 0.0104319 + 0.002253$

$\underline{\Delta_5 = 0.012685 \text{ in}}$

Floor 6 Supported



$A = 2928 \text{ in}^2$

$I = 32685264 \text{ in}^4$

$$\Delta_6 = \frac{(8.54)(802)^3}{3(2577)I} + \frac{1.2(8.54)(802)}{1031(2928)}$$

$\Delta_6 = 0.01743385 + 0.0027226$

$\underline{\Delta_6 = 0.020156 \text{ in.}}$

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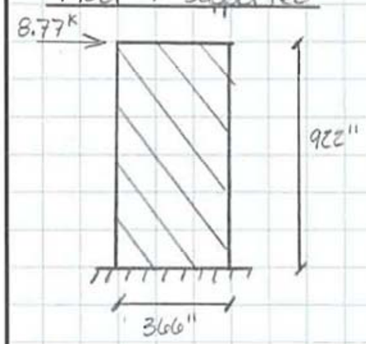
Floor 7 Supported



$A = 2928 \text{ in}^2$

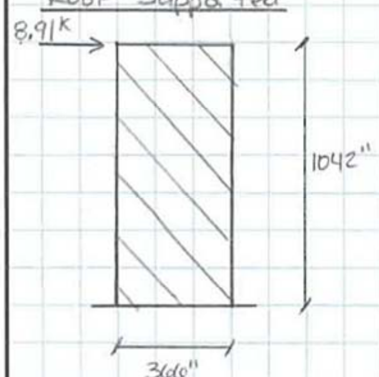
$I = 32685264 \text{ in}^4$

$$\Delta_7 = \frac{(8.77)(922)^3}{3(2577)I} + \frac{1.2(8.77)(922)}{1031(2928)}$$

$\Delta_7 = 0.0272022 + 0.003214$

$\Delta_7 = 0.030416$ in.

Roof Supported



$A = 2928 \text{ in}^2$

$I = 32685264 \text{ in}^4$

$$\Delta_R = \frac{(8.91)(1042)^3}{3(2577)I} + \frac{1.2(8.91)(1042)}{1031(2928)}$$

$\Delta_R = 0.039893 + 0.003691$

$\Delta_R = 0.043584$ in

- Overall Displacement of Wall 9

$$\Sigma\Delta = 7.36E\text{-}4 + 0.001852 + 0.004009 + 0.007443 + 0.012685 + 0.020156$$

$\Sigma\Delta = 0.046881$ in.

$\Delta_{limit} = H/400 = 1042''/400 = 2.605'' > 0.046881''$ ∴ OK

- Calculated displacements and story drifts for Walls 8, 10, & 15 can be found in Tables 10a-c .

Table 10a - Wall 8 Story Displacements

Floor Supported	Lateral Force (k)	E_c (ksi)	E_r (ksi)	I (in^4)	Thickness (in)	Length (in)	Height (in)	Δ_{flex}	Δ_{shear}	Story Displacement (in)	Story Drift: N/S Direction (in)	Story Drift: E/W Direction (in)	Allowable Story Drift (in)
Roof	3.58	2577	1031	32685264	8	820	1042	0.016019	0.000661	0.01668008	0.000071	0.000023	1.7367
7	3.55	2577	1031	32685264	8	820	922	0.011022	0.000581	0.011602871	0.000074	0.000022	1.53367
6	3.49	2577	1031	32685264	8	820	802	0.007134	0.000497	0.007631494	0.000077	0.000021	1.3367
5	3.43	2577	1031	32685264	8	820	682	0.004307	0.000415	0.004721695	0.000079	0.00002	1.137
4	3.31	2577	1031	32685264	8	820	562	0.002329	0.000331	0.002659129	0.000078	0.000018	0.9367
3	3.17	2577	1031	32685264	8	820	442	0.001083	0.000248	0.001330984	0.000073	0.000015	0.7367
2	2.79	2577	1031	32685264	8	820	322	0.000368	0.000159	0.000527229	0.000062	0.000011	0.5367
1	2.90	2577	1031	32685264	8	820	178	6.47E-05	9.15E-05	0.000156181	0.000039	0.000006	0.2967
									Total Wall Displacement (in) =	0.045309664			

Table 10b - Wall 10 Story Displacements

Floor Supported	Lateral Force (k)	E_c (ksi)	E_r (ksi)		Thickness (in)	Length (in)	Height (in)	Δ_{flex}	Δ_{shear}	Story Displacement (in)	Story Drift: N/S Direction (in)	Story Drift: E/W Direction (in)	Allowable Story Drift (in)
Roof	1.48	2577	1031	32685264	8	576	1042	0.006615	0.000389	0.007003637	0.000104	0.000016	1.7367
7	1.52	2577	1031	32685264	8	576	922	0.004708	0.000353	0.005061613	0.000105	0.000016	1.53367
6	1.56	2577	1031	32685264	8	576	802	0.003181	0.000316	0.003496494	0.000107	0.000015	1.3367
5	1.61	2577	1031	32685264	8	576	682	0.002027	0.000278	0.00230487	0.000106	0.000014	1.137
4	1.67	2577	1031	32685264	8	576	562	0.001172	0.000237	0.001409103	0.000102	0.000013	0.9367
3	1.73	2577	1031	32685264	8	576	442	0.000591	0.000193	0.000784084	0.000092	0.000011	0.7367
2	1.67	2577	1031	32685264	8	576	322	0.00022	0.000135	0.000355468	0.000076	0.000008	0.5367
1	1.92	2577	1031	32685264	8	576	178	4.28E-05	8.63E-05	0.000129103	0.000046	0.000005	0.2967
									Total Wall Displacement (in) =	0.020544373			

Table 10c - Wall 15 Story Displacements

Floor Supported	Lateral Force (k)	E_c (ksi)	E_r (ksi)		Thickness (in)	Length (in)	Height (in)	Δ_{flex}	Δ_{shear}	Story Displacement (in)	Story Drift: N/S Direction (in)	Story Drift: E/W Direction (in)	Allowable Story Drift (in)
Roof	14.18	2577	1031	32685264	8	432	1042	0.063467	0.004974	0.068441201	0.000117	0.000008	1.7367
7	13.85	2577	1031	32685264	8	432	922	0.042945	0.004299	0.047243944	0.000118	0.000009	1.53367
6	13.33	2577	1031	32685264	8	432	802	0.027215	0.003601	0.030815784	0.000119	0.000008	1.3367
5	12.77	2577	1031	32685264	8	432	682	0.016029	0.002933	0.018961909	0.000117	0.000008	1.137
4	12.07	2577	1031	32685264	8	432	562	0.008481	0.002285	0.010766327	0.000111	0.000008	0.9367
3	11.25	2577	1031	32685264	8	432	442	0.003843	0.001674	0.005517235	0.0001	0.000007	0.7367
2	9.81	2577	1031	32685264	8	432	322	0.001296	0.001064	0.002359644	0.000081	0.000005	0.5367
1	10.01	2577	1031	32685264	8	432	178	0.000223	0.0006	0.000823306	0.000048	0.000003	0.2967
									Total Wall Displacement (in) =	0.18492935			

Appendix G: Overturning Moments

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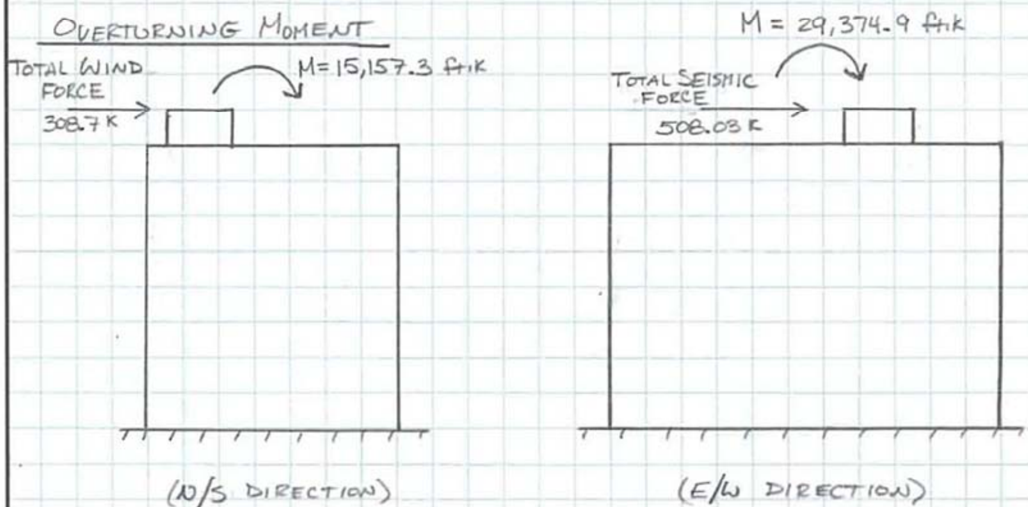
JOB TECH. REPORT 3 - CALCULATIONS
SHEET NO. 1 OF 2
CALCULATED BY A. KACZMAREK DATE
CHECKED BY DATE
SCALE

OVERTURNING MOMENT



- Lateral loads will create overturning moment, while the gravity loads will try to resist the overturning moment.
- To determine if the gravity loads will exceed the lateral loads, the stress due to seismic and lateral loads will be calculated.

STRESS DUE TO DEAD LOADS

$$\Rightarrow \frac{\text{Weight of building}}{\text{Square Footage of Foundation}} = \frac{20{,}321^{K}}{12{,}808 \text{ SF}} \times 1000 \text{ lb.} = 1586.6 \text{ PSF}$$

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STRESS DUE TO E/W SEISMIC LOADS

$$\Rightarrow \frac{508.03^{k}\ (1000\ lb)}{12{,}808\ SF} = 39.67\ PSF$$

$$\Rightarrow \frac{39.67}{1586.6} \times 100\% = 2.5\%\ \text{of Dead Load}$$

STRESS DUE TO N/S LATERAL LOADS

$$\Rightarrow \frac{308.7^{k}\ (1000\ lb)}{12{,}808\ SF} = 24.1\ PSF$$

$$\Rightarrow \frac{24.1}{1586.6} \times 100\% = 1.5\%\ \text{of Dead Load}$$

- Since the stresses of the lateral and seismic loads are a much smaller percentage of the gravity loads, overturning is not a concern for the design of Cambria Suites Hotel.