12/1/2009

TECHNICAL
REPORT III

FAIRFIELD INN & SUITES, MARRIOT PITTSBURGH, PA

Amanda Smith | Advisor: Dr. Ali Memari   Structural Option

TABLE OF CONTENTS

TABLE OF CONTENTS

EXECUTIVE SUMMARY

The purpose of this report is to provide a detailed analysis of the existing design for the lateral force resisting system of the Fairfield Inn and Suites. The loads calculated in the existing structural conditions report determined were applied to the lateral force resisting system comprised of reinforced concrete masonry shear walls. A description of the structural system of the building and the path the loads travel to reach the foundation is given.

To verify the shear strength system of the building, an ETABS model was created to compare the analysis results to the hand calculations done for the Fairfield Inn and Suites. The ETABS model only modeled the shear walls and rigid diaphragms for the building. The gravity columns and transfer beams were not modeled at this stage to simplify the attempt in creating a model of the Fairfield Inn and Suites. The calculations done by hand only took into account the shear walls as the lateral resisting system. The lateral loads were applied to the model to determine center of rigidity, torsion, overturning, and story drifts all taken from the ETABS outputs and compared to the hand calculation and allowable limits set forth by the code and industry.

With the comparison of the ETABs model and the hand calculations, there were a few differences found in the location for the center of rigidity. Since the hand calculations only accounted for the shear walls and the ETABs model include the rigid diaphragms, the center of rigidity values see a difference. In general, the values still follow the same pattern of increasing or decreasing as the floor height increases. Center of rigidity values were taken in both the North/South direction and East/West direction. Therefore, with this difference, the center of rigidity from the hand calculations was used in determining relative stiffness, torsion, shear, and overturning. There were no concerns regarding the calculated torsion and shear results. It verifies that the shear walls are properly reinforced and are providing the majority of the lateral resisting system, with minimal assistance from the slabs. These results suggest that it was only necessary to look at the shear walls in this analysis.

The overturning results show that the dead loads gravity system of the building will resist any uplift or torsion created on the building due to the lateral loads, since the lateral loads are a small fraction of the gravity loads. The story drifts and displacements were found to be within the allowable limits of the code. The hand calculations and the ETABs model both conclude the story drift is sufficient and does not exceed the limits for the Fairfield Inn and Suites.

Each analysis done on the lateral system of the building can be seen through detailed descriptions and diagrams, as well as, the materials and codes used in the analysis and design. Building layout and detailed calculations for each analysis performed can be found in an Appendix at the end of the report.

INTRODUCTION: Fairfield Inn & Suites

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

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

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

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

Technical Report 3 runs an analysis of the lateral system of the Fairfield Inn and Suites. This analysis determines if the building design is sufficient to resist the lateral loads that were determined in Technical Report 3 against the building. An ETABS model of the building was used to compare the results of the hand calculations with lateral analysis of the building from the model.

STRUCTURAL SYSTEM

Foundation

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

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

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

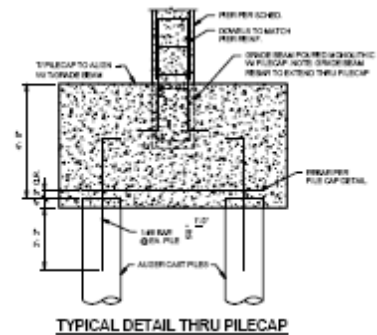
TYPICAL DETAIL THRU PILECAP

Figure 1.1

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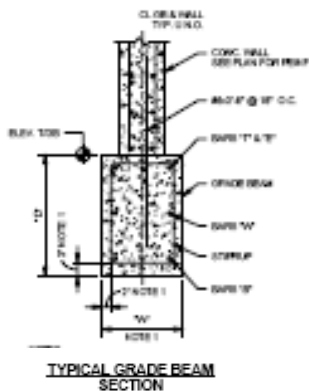

TYPICAL GRADE BEAM SECTION

Figure 1.2

Floor System

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

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

Figure 2.1: Partial First Floor Slab

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

The roof system and smaller high roof system are the same use the same 8" untopped precast

Figure 2.2: Second Floor Transfer Beams

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

Columns

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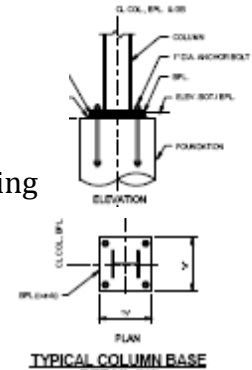

Figure 3.1

Lateral System

The lateral system for the Fairfield Inn and Suites is a combination of ordinary reinforced concrete masonry shear walls. The exterior shear walls are 10" concrete masonry and the core shear walls are 8" concrete masonry. The core shear walls surround the staircases and elevator shaft. On floors two thru ten, two additional load bearing masonry walls extend from the west wall to the east wall running along the south wall of staircase B and the north wall of the elevator shafts (see Figure 4.1). Elevations of each of these shear walls can be found in Appendix B.

Shear walls supporting the ground floor to the fourth floor support a compressive strength of f'c=8000 psi. All other shear walls support a compressive strength of f'c=5000 psi. The typical vertical reinforcement in both the 10" and 8" shear walls is #5 bars at 16" o.c., 24" o.c., or 32" o.c. with bars centered in wall and solid grout wall.

Figure 4.1: Lateral Shear Wall System
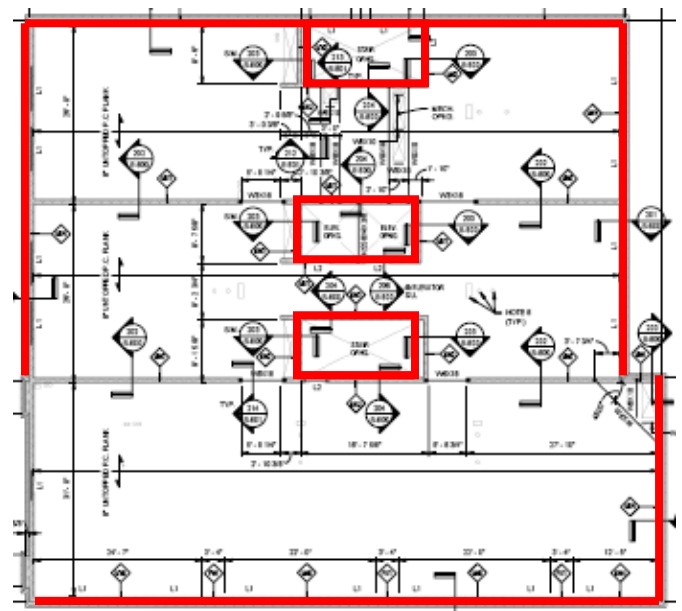
With the majority of the exterior walls being shear walls, the center of rigidity stays pretty central between the East and West walls. Due to the core shear walls not be centered in the building, the center of rigidity shifts slightly north. When the center of rigidity is not in line with the resultant lateral force, eccentricity and moments due to torsion become a factor.

ETABS MODEL

ETABS is a computer modeling and analysis program developed by Computer 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 in this technical assignment, the building's lateral system and diaphragms were the only components modeled. As seen in Figures 5.1 and 5.2, the shear walls and floor slabs were the only elements modeled. Material properties and geometric properties were inputted for the floor slabs and each shear wall. The simplification of only modeling lateral components allowed for the gravity loads to be applied as additional area masses to the diaphragms. Both wind and seismic loads were applied about the centers of rigidity of the structure for analysis. The results from this model were compared to values produced by hand calculations of the center of mass, centers of rigidity, and story displacements. The overall building drift and controlling loads in each direction were also pulled from the model analysis results.
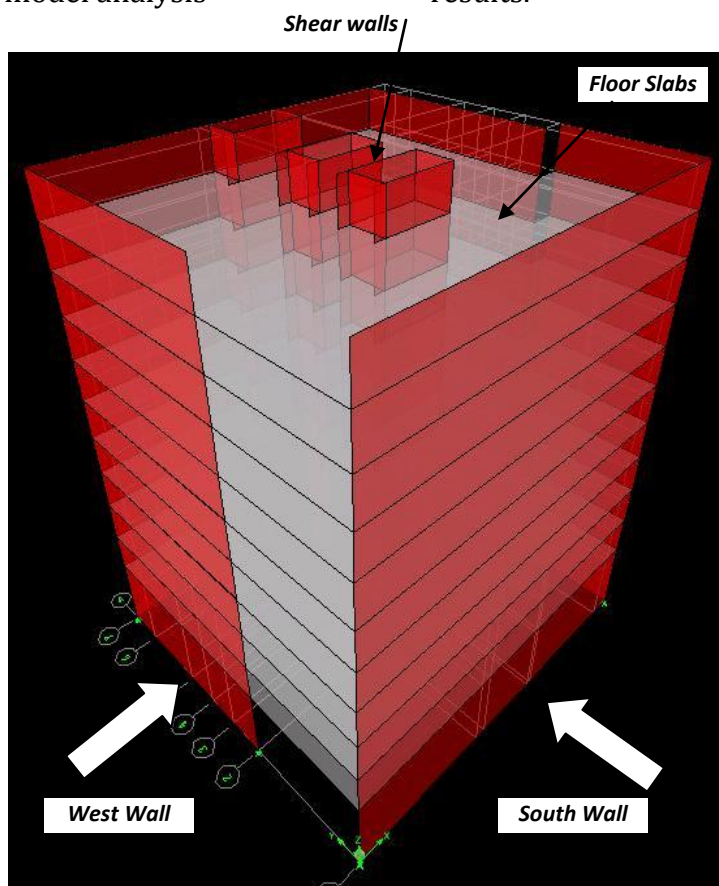
Figure 5.1 - ETABS Model - South & West Walls
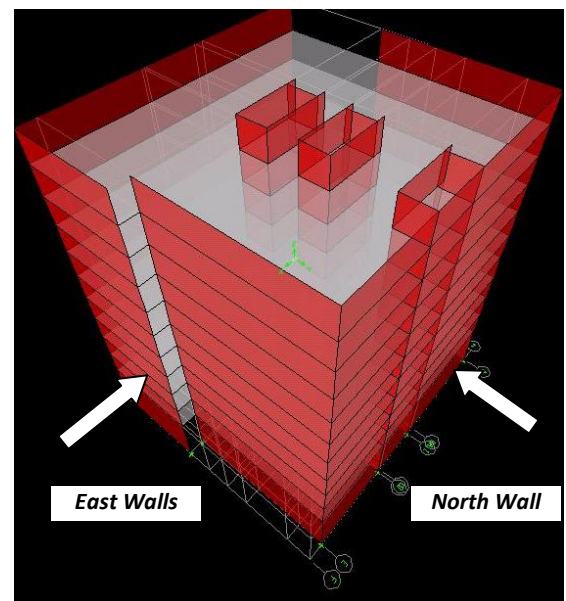
Figure 5.2 - ETABS Model View - North & East Walls

CODES AND REQUIREMENTS

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

- The 2006 International Building Codes as amended by the city of Pittsburgh

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

- Specifications for Structural Concrete (ACI 301-05), American Concrete Institute

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

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

- PCI Design Handbook – Precast/Prestressed Concrete Institute

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

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

- ETABS Modeling and Analysis - Computer & Structures, Inc

Drift Criteria

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

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

(Allowable Story Drift) $\Delta_{seismic} = 0.015H_{sx}$

Load Combinations

The list below shows the various load cases according to ASCE-07 section 2.3 for factored loads using strength design and from the International Building Code, 2006 edition. These were the load cases used in the analysis of the lateral system for this report.

1.4D

$1.2D + 1.6L + 0.5L_r$

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

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

$1.2D + 1.0E + 1.0L$

$0.9D + 1.6W$

$0.9D + 1.0E$

These combinations were all considered in the ETABS Model. After analyzing story displacements, shears, and drifts, it was concluded that the load combination 1.2D + 1.6W + 1.0L + 0.5Lr controls in the North/ South direction, with the wind controlling in the North/South direction due to its larger surface area creating higher forces. The load combination 0.9D + 1.0E controls in the East/West direction due to the slightly smaller surface areas allowing seismic forces to control in this direction.

GRAVITY LOADS

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

Dead Loads:

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

Live Loads:

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

LATERAL LOADS

Wind Analysis

Wind loads were calculated in accordance with ASCE 7-05, Chapter 6. To examine the wind loads in the North/South direction and the West/East direction, the Analytical Procedure – Method two described in Section 6.5, was used to find design pressures. The variables used in this analysis are located in Table 1a. Please refer to Appendix C for equations and base calculations used for the execution of this procedure. Figure 6.1 shows the wind direction made to the typical floor plan.

E/W Wind Direction
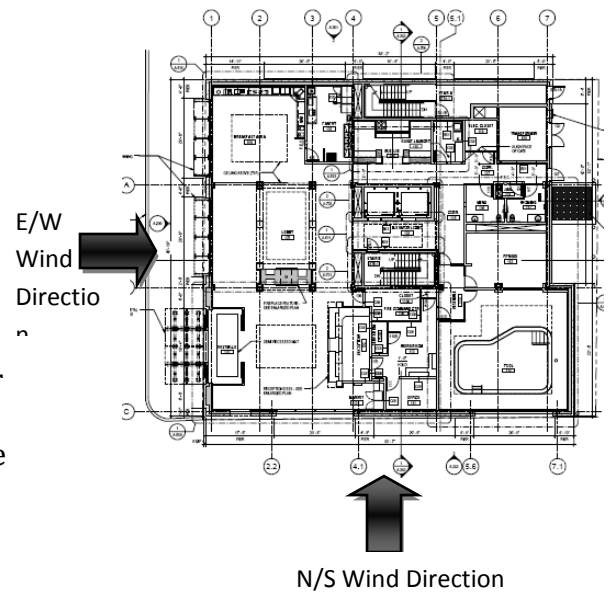
N/S Wind Direction

Figure 6.1: Wind Direction

Table 1a

Wind Variables			ASCE References
Basic Wind Speed	V	90	Fig. 6-1
Directionality Factor	K_d	0.85	Table 6-4
Importance Factor	I	1.15	Table 6-1
Exposure Category		C	§ 6.5.6.3
Topographic Factor	K_{zt}	1.00	§ 6.5.7.1
Velocity Pressure Exposure Coefficient evaluated at Height Z	K_z	Varies	Table 6-3
Velocity Pressure at Height z	q_z	Varies	Eq. 6-15
Velocity Pressure at Mean Roof Height	q_h	20.47	Eq. 6-15
Equivalent Height of Structure	>	64.6'	Table 6-2
Intensity of Turbulence	$I_{\dot{z}}$	0.268	Eq. 6-5
Integral Length Scale of Turbulence	$L_{\dot{z}}$	208.81	Eq. 6-7
Background Response Factor (East/West)	Q	0.792	Eq. 6-6
Background Response Factor (North/South)	Q	0.788	Eq. 6-6
Gust Effect Factor (East/West)	G	0.808	Eq. 6-4
Gust Effect Factor (North/South)	G	0.806	Eq. 6-4
External Pressure Coefficient (Windward)	C_p	0.8	Fig. 6-6
External Pressure Coefficient (E/W Leeward)	C_p	-0.03	Fig. 6-6
External Pressure Coefficient (N/S Leeward)	C_p	-0.05	Fig. 6-6

Tables and calculations of wind pressures in each direction can be found in Table 9a and Table 9b referenced in Appendix C. One table was developed to determine the wind pressures in the North/South direction. This direction is adjacent to an existing building and a major highway, which neither structure is significant enough to block the building from receiving full wind loads. These wind loads are currently the most prevalent at this site. The other table was developed to determine the wind pressures in the East/West direction. There are currently adjacent buildings blocking the wind on the lower levels on the hotel, but wind in this direction must be examined in the case that these buildings will not be present in the future and the full wind load will be applied to the building. Basic loading diagrams for wind forces in each direction are provided for reference in Figures 6.2 and 6.3.
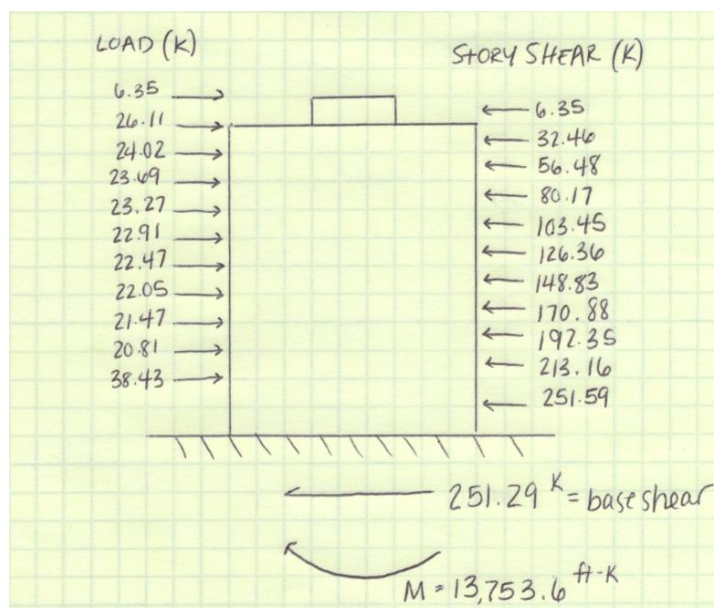

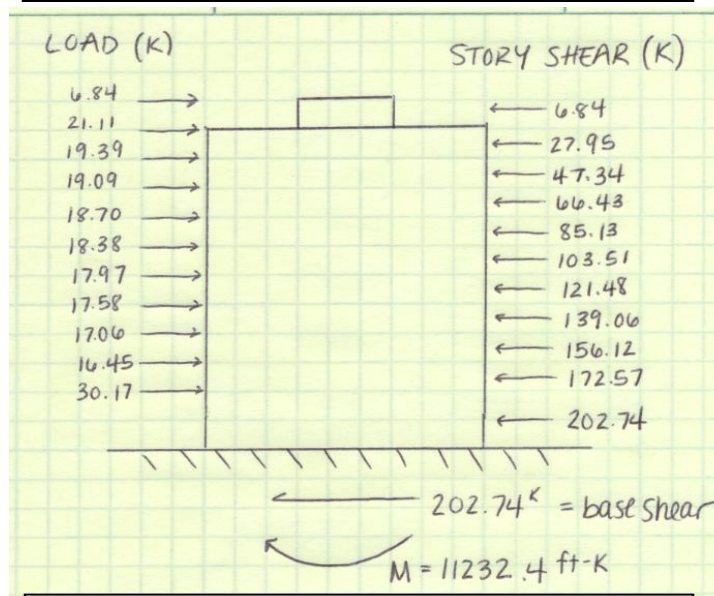*Figure 6.2: Wind Loading Diagram in North/South Direction*

Figure 6.3: Wind Loading Diagram in East/West Direction

Seismic Analysis

An assumption was made in this seismic analysis that the Fairfield Inn and Suites employs a rigid diaphragm and therefore allows the use of the Equivalent Lateral Force procedure found in Chapters 11 and 12 of ASCE 7-05. Upon investigation of the geotechnical report, the Fairfield Inn and Suites falls under the Site D classification. The variables needed to calculate base shear according to ASCE 7-05 are located in Table 2a.

Table 2a

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

The base shear calculated for seismic analysis includes the effective seismic building weight. An excel sheet was set up to determine the total weight that accumulated at each floor above grade. A summation of each floor resulted in the effective building weight which was used to determine the base shear and overturning moments due to seismic loads. Please refer to Appendix C for detailed calculations used to obtain building weight, as well as, base shear and overturning moments for each floor as seen in Table 2b. The story shear is also determined for each level and can be found in Table 2b. A seismic loading diagram is provided as reference to relate forces and shears that resulted as seen Figure 7.1.

Figure 7.1: Seismic Loading Diagram
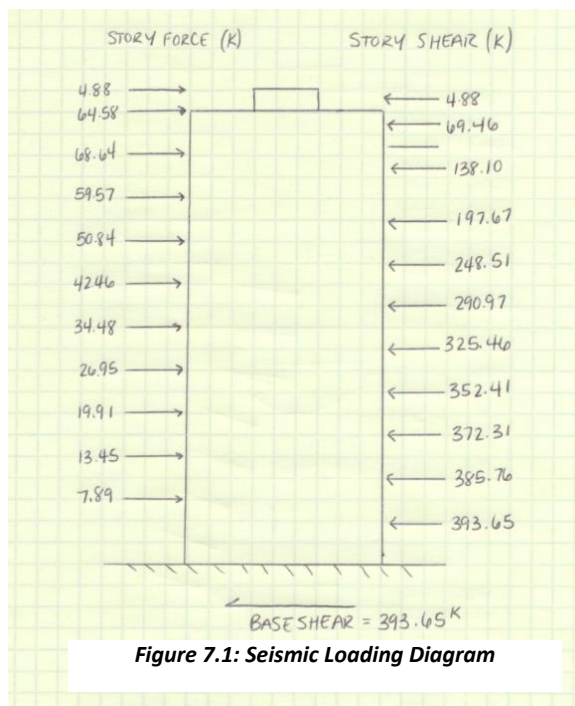
Table 2b

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

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

LOAD DISTRIBUTION

Load Path

The wind and seismic loads that act against the building need a way of traveling through the structure into the foundation, ultimately reaching the ground. This load path is assumed to be governed by the concept of relative stiffness. The members that are most rigid in a building draw the forces to them. As the lateral forces come in contact with the building, the loads are transmitted through the rigid diaphragms, to the shear walls, and then down into the mat foundation. (See Figure 8.1) The shear walls that have minimal assistance from the slabs resist the majority of the lateral forces. The columns on the first level only transmit the gravity loads from the transfer beams that hold the weight of the floors above.
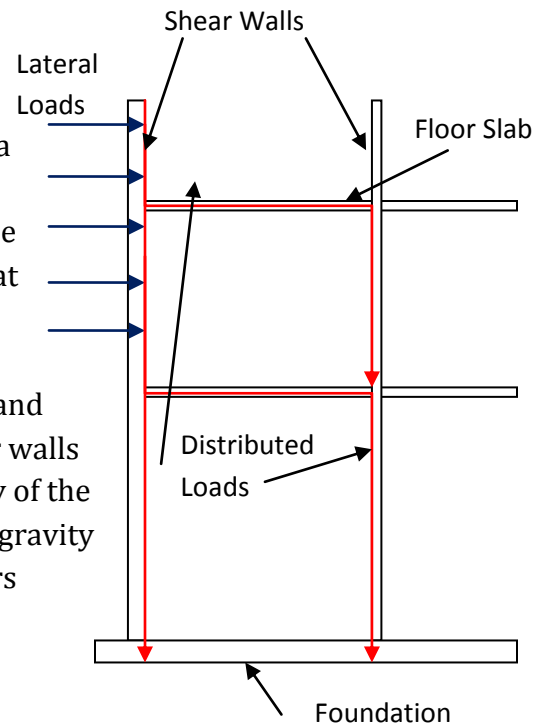
Figure 8.1 – Load path diagram

Center of Rigidity and Mass

The Fairfield Inn and Suites and a number of shear walls. Essentially the entire building frame is shear walls except for a few members. There is a shear wall located along the north, south, west, and east face of the building in addition to a shear wall core. There are four shear walls that surround the staircases and the elevator shaft. Figure 8.3 shows the numbered system assigned to each wall to better reference exactly which shear walls are being discussed throughout the analysis. The core shear walls are 8" thick throughout their heights, while the surrounding shear walls are a thickness of 10". These walls do vary in length and are located different distances from the center of rigidity of the building. The thickness, height, and distance from center of rigidity all affect the rigidity of the walls and altering the relative stiffness of each wall.
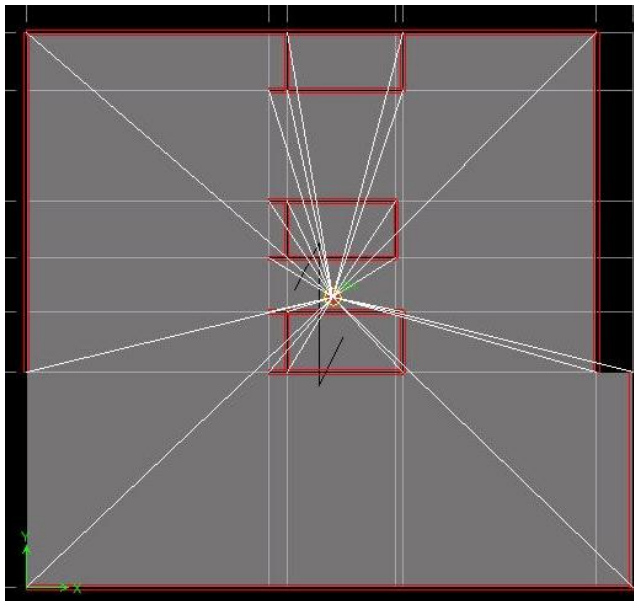
Figure 8.2 - ETABS Rigidity Diagram
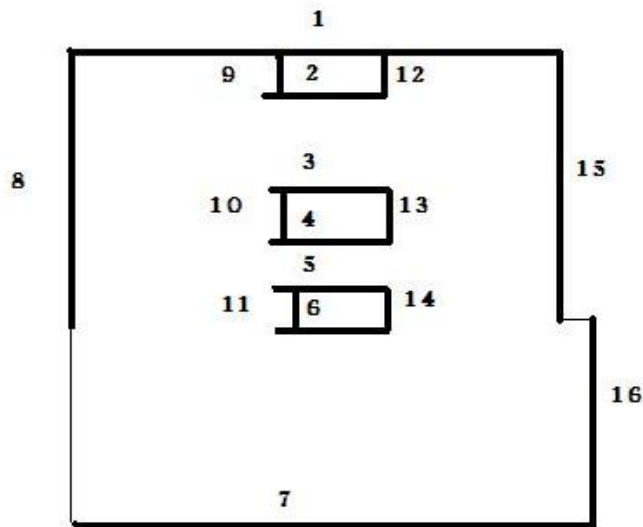
Figure 8.3 - Numbered Shear Walls

Tables in Appendix D define the rigidities of Walls 8-16 which are parallel to the north/south lateral forces, and of Walls 1-7 which are parallel to the east/west lateral forces. The rigidities of each wall were calculated using the following equation:

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

The equation has to take into account that walls supporting up to floor 4 have an f'c = 8000 psi and the walls above floor 4 have an f'c = 5000 psi. The rigidities of each wall can then be used to determine the center of rigidity of each floor through the following equation:

$$\text{Center of Rigidiy} = \frac{\Sigma[(R)(\text{distance between origin and element})]}{\Sigma R}$$

The values for the center of rigidity can be found in Table 3a for each floor. Since the building structure is ultimately rectangular, it makes it easy to determine the center of mass of the building. The center of mass does not vary from floor to floor and is consistent throughout the building. Along with the center of rigidity, the center of mass values can be found in Table 3a. The coordinates found by the hand calculations and the ETABS output results are compared in this Table as well. The values differ because of the assumptions made for each calculation. The rigidity calculated by hand assumes only the shear walls are to be considered, but the ETABS model takes into account the building diaphragms when determining the rigidity. The hand calculated values will be those used whenever the center of mass and center of rigidity are needed. Detailed calculations can be found in Appendix D.

	Table 3a - ETABS Vs. Hand Calculation Comparison							
	Center of Rigidity				Center of Mass			
	ETABS Calculation		Hand Calculation		ETABS Calculation		Hand Calculation	
	X	Y	X	Y	X	Y	X	Y
Story 10	717.90	649.60	614.11	535.68	554.394	524.396	554.986	524.4
Story 9	709.60	635.20	599.93	533.96	554.394	524.396	554.986	524.4
Story 8	700.47	619.80	580.41	531.92	554.394	524.396	554.986	524.4
Story 7	690.95	604.12	563.13	529.50	554.394	524.396	554.986	524.4
Story 6	681.28	588.20	549.52	526.63	554.394	524.396	554.986	524.4
Story 5	671.81	572.80	539.18	523.17	554.394	524.396	554.986	524.4
Story 4	662.91	558.40	531.36	518.96	554.394	524.396	554.986	524.4
Story 3	654.99	546.30	525.42	513.65	554.394	524.396	554.986	524.4
Story 2	648.46	537.10	520.85	506.63	554.394	524.396	554.986	524.4
Story 1	643.93	533.70	514.66	497.07	554.394	524.396	554.986	524.4

Relative Stiffness

With the rigidity of the walls determined, we can use them to find the relative stiffness of each wall at each floor. The relative stiffness dictates what percentage of the lateral force is distributed to each wall. The relative stiffness will not be consistent throughout the entire height of the building. This can be calculated using the following equation:

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

The values for walls 8-16 at every floor can be found in Table 3b. The values for walls 1-7 at every floor can be found in Table 3c. Detailed calculations can be found in Appendix D. Knowing the relative stiffness of each wall, the values can be directly applied to the loads at each floor to determine how much of the load each wall will have to resist.

Table 3b - Relative Stiffness (%)									
North - South Force									
	Wall 8	Wall 9	Wall 10	Wall 11	Wall 12	Wall 13	Wall 14	Wall 15	Wall 16
Floor 10	37.12	1.73	1.51	2.04	1.73	1.51	2.04	33.01	19.33
Floor 9	41.28	0.74	0.63	0.91	0.74	0.63	0.91	35.95	18.21
Floor 8	43.94	0.44	0.37	0.54	0.44	0.37	0.54	37.24	16.14
Floor 7	45.86	0.31	0.26	0.39	0.31	0.26	0.39	37.83	14.37
Floor 6	47.28	0.25	0.21	0.31	0.25	0.21	0.31	38.09	13.08
Floor 5	48.33	0.22	0.18	0.27	0.22	0.18	0.27	38.17	12.17
Floor 4	49.11	0.19	0.16	0.24	0.19	0.16	0.24	38.17	11.52
Floor 3	49.70	0.18	0.15	0.23	0.18	0.15	0.23	38.15	11.04
Floor 2	50.15	0.17	0.14	0.21	0.17	0.14	0.21	38.11	10.69
Floor 1	50.76	0.16	0.13	0.20	0.16	0.13	0.20	38.03	10.24

Table 3c - Relative Stiffness (%)							
East- West Force							
	Wall 1	Wall 2	Wall 3	Wall 4	Wall 5	Wall 6	Wall 7
Floor 10	44.7	0.8	0.7	0.7	0.8	0.8	51.4
Floor 9	44.7	0.9	0.8	0.8	0.9	0.9	51.1
Floor 8	44.7	0.9	0.8	0.8	0.9	0.9	50.8
Floor 7	44.7	1.0	0.9	0.9	1.0	1.0	50.4
Floor 6	44.6	1.2	1.0	1.0	1.2	1.2	49.8
Floor 5	44.3	1.4	1.2	1.2	1.4	1.4	49.1
Floor 4	43.8	1.7	1.5	1.5	1.7	1.7	48.1
Floor 3	43.0	2.2	1.9	1.9	2.2	2.2	46.7
Floor 2	41.3	2.9	2.6	2.6	2.9	2.9	44.6
Floor 1	38.6	4.2	3.8	3.8	4.2	4.2	41.3

TORSION

When the center of rigidity and the center of mass do not occur at the same location, torsion is present. The difference between the center of rigidity and center of mass is the eccentricity. Moments are produced by this eccentricity and torsional shear becomes an additional force on the building.

For rigid diaphragms, like Fairfield Inn and Suites, two separate moments need to be taken into account when determining torsion in a building. According to ASCE 7-05, torsion in rigid diaphragms is the sum of the inherent moment and the accidental moment. The inherent moment, M_t, is caused by the eccentricity between the center of rigidity and the center of mass. The lateral force exerted on the building at that level; times the eccentricity

of the floor gives the inherent moment. The accidental moment, M_{ta}, is due to the rigidity of the slab. The accidental moment takes into account an assumed displacement of the center of mass. The displacement is a distance 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 by forces in both directions can be seen in Tables 4a and 4b. Detailed calculations of this method can be found in Appendix E.

Table 4a - Overall Building Torsion					
North/South Direction					
	Factored Lateral Force (k)	COR-COM (ft)	M_t (ft-k)	M_{ta} (ft-k)	$M_{t,tot}$ (ft-k)
Story 10	41.78	4.93	205.82	193.01	398.83
Story 9	38.43	3.75	143.93	177.56	321.49
Story 8	37.90	2.12	80.30	175.12	255.42
Story 7	37.23	0.68	25.27	172.01	197.28
Story 6	36.66	-0.46	-16.69	169.35	152.66
Story 5	35.95	-1.32	-47.36	166.10	118.74
Story 4	35.28	-1.97	-69.45	162.99	93.54
Story 3	34.35	-2.46	-84.64	158.71	74.07
Story 2	33.30	-2.84	-94.72	153.83	59.11
Story 1	61.49	-3.36	-206.6	284.07	77.43
				Total:	1748.55

Table 4b - Overall Building Torsion					
East/West Direction					
	Factored Lateral Force (k)	COR-COM (ft)	M_t (ft-k)	M_{ta} (ft-k)	$M_{t,tot}$ (ft-k)
Story 10	21.11	0.94	19.85	92.25	112.10
Story 9	19.39	0.80	15.44	84.73	100.18
Story 8	19.09	0.63	11.96	83.42	95.38
Story 7	18.70	0.43	7.95	81.72	89.67
Story 6	18.38	0.19	3.41	80.32	83.73
Story 5	17.97	-0.10	-1.84	78.53	76.69
Story 4	17.58	-0.45	-7.97	76.82	68.85
Story 3	17.06	-0.90	-15.28	74.55	59.27
Story 2	16.45	-1.48	-24.36	71.89	47.53
Story 1	30.17	-2.28	-68.71	131.84	63.14
				Total:	796.55

SHEAR

In order to determine the shear forces on each level of the building, the direct and torsion forces need to be calculated. The combination of the two forces is the overall shear force occurring at each level. The direct shear forces relate to relative stiffness of the shear walls. The torsion forces relate to the torsion moments produced on each floor due to the wind or seismic loads.

Direct Shear

The lateral forces acting on a building must be distributed among the shear walls in the structure to be directed down through the load path. The distribution of these forces is the direct shear force that occurs at each level of a building. The story shear forces are distributed dependent on the relative stiffness of each shear wall. The greater the stiffness of the wall, the greater the load the wall can receive. The direct shears applied to each wall can be seen in Tables 5a and 5b. Detailed calculations of obtaining the direct shears in both directions can be found in Appendix F.

Table 5a - North/South Direct Shear											
Load Combination 1.2D+1.6W+L+0.5Lr	Force (k)	Factored Force (k)	Distributed Force (k)								
			Wall 8	Wall 9	Wall 10	Wall 11	Wall 12	Wall 13	Wall 14	Wall 15	Wall 16
Roof	26.11	41.776	15.51	0.72	0.63	0.85	0.72	0.63	0.85	13.79	8.07
Floor 10	24.02	38.43	15.87	0.28	0.24	0.35	0.28	0.24	0.35	13.82	7.00
Floor 9	23.69	37.90	16.65	0.17	0.14	0.21	0.17	0.14	0.21	14.11	6.12
Floor 8	23.27	37.23	17.08	0.12	0.10	0.14	0.12	0.10	0.14	14.09	5.35
Floor 7	22.91	36.66	17.33	0.09	0.08	0.11	0.09	0.08	0.11	13.96	4.80
Floor 6	22.47	35.95	17.38	0.08	0.06	0.10	0.08	0.06	0.10	13.72	4.37
Floor 5	22.05	35.28	17.33	0.07	0.06	0.09	0.07	0.06	0.09	13.47	4.06
Floor 4	21.47	34.35	17.07	0.06	0.05	0.08	0.06	0.05	0.08	13.10	3.79
Floor 3	20.81	33.30	16.70	0.06	0.05	0.07	0.06	0.05	0.07	12.69	3.56
Floor 2	38.43	61.49	31.21	0.10	0.08	0.12	0.10	0.08	0.12	23.39	6.29

Table 5b - East/West Direct Shear									
Load Combination 0.9D+1.0E	Force (k)	Factored Force (k)	Distributed Force (k)						
			Wall 1	Wall 2	Wall 3	Wall 4	Wall 5	Wall 6	Wall 7
Roof	21.11	21.11	9.44	0.17	0.15	0.15	0.17	0.17	10.85
Floor 10	19.39	19.39	8.68	0.17	0.15	0.15	0.17	0.17	9.92
Floor 9	19.09	19.09	8.54	0.18	0.16	0.16	0.18	0.18	9.70
Floor 8	18.7	18.70	8.36	0.19	0.17	0.17	0.19	0.19	9.42
Floor 7	18.38	18.38	8.19	0.22	0.19	0.19	0.22	0.22	9.16
Floor 6	17.97	17.97	7.96	0.25	0.22	0.22	0.25	0.25	8.83
Floor 5	17.58	17.58	7.71	0.30	0.26	0.26	0.30	0.30	8.46
Floor 4	17.06	17.06	7.33	0.37	0.33	0.33	0.37	0.37	7.97
Floor 3	16.45	16.45	6.80	0.48	0.43	0.43	0.48	0.48	7.33
Floor 2	30.17	30.17	11.64	1.26	1.15	1.15	1.26	1.26	12.45

Torsional Shear

Due to the torsion present in the structure, an additional force is present on the building. Each shear wall within in the building will have to resist a torsional shear force. The torsional shear is due to the torsion moments produced on each floor caused by the eccentricity. The total torsional shear present at each wall also relates to the relative stiffness of each shear wall. Once again, the greater the relative stiffness, the greater the shear force will be against that wall.   To determine the torsional shear values the following equation is used:

$$T = \frac{Vtot \; e \; di \; Ri}{J}$$

- Vtot = total story shear
- e = eccentricity (distance from center of rigidity to center of mass)
- di = distance from center of rigidity to shear wall
- Ri = relative stiffness of shear wall
- J = torsional moment of inertia

The torsional shear forces were determined for the shear walls supporting floor 6 and can be found in Table 6a. Further detailed calculations of how to determine the torsional shear can be found in Appendix F.

Table 6a - Torsional Shear in Shear Walls Supporting Floor 6							
		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	194.37	0.443	2.2	521.6	120525.5	0.194
Wall 2	E/W	194.37	0.014	2.2	420.6	2476.7	0.005
Wall 3	E/W	194.37	0.012	2.2	218.6	573.4	0.002
Wall 4	E/W	194.37	0.012	2.2	114.6	157.6	0.001
Wall 5	E/W	194.37	0.014	2.2	16.6	3.9	0.000
Wall 6	E/W	194.37	0.014	2.2	93.4	122.1	0.001
Wall 7	E/W	194.37	0.491	2.2	481.7	113929.1	0.199
Wall 8	N/S	238.13	0.483	5.5	544.5	143200.0	0.677
Wall 9	N/S	238.13	0.002	5.5	146.5	42.9	0.001
Wall 10	N/S	238.13	0.002	5.5	146.5	42.9	0.001
Wall 11	N/S	238.13	0.003	5.5	146.5	64.4	0.001
Wall 12	N/S	238.13	0.002	5.5	137.5	37.8	0.001
Wall 13	N/S	238.13	0.002	5.5	125.5	31.5	0.001
Wall 14	N/S	238.13	0.003	5.5	137.5	56.7	0.001
Wall 15	N/S	238.13	0.382	5.5	486.5	90412.6	0.478
Wall 16	N/S	238.13	0.122	5.5	552.5	37241.3	0.173
Torsional Moment of Inertia J = Σ $(R_i)(d_i^2)$ =						508918.4	

Shear Strength Check

With the direct shear forces and the torsional forces acting on each shear wall, a check needs to be done on each wall to determine if the reinforcement is sufficient to support the loads. According to ACI 318-08, the shear strength of a reinforced concrete shear wall can be defined by this equation:

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

A shear strength done on the shear walls supporting Floor 6 were conducted and detailed calculations can be found in Appendix F. Each shear wall was within the capacity determined by the shear strength. The reinforcement for each wall proved to be adequately designed. The shear wall checks and verifications can be found in Table 7a.

Table 7a

Shear Wall Strength Check											
(Supporting Floor 6)											
Floor 6	Direct Shear (K)	Torsional Shear (k)	V_u (k)	Vertical Reinf.	Spacing (in)	Length (in)	Thickness (in)	A_{cv} (in^2)	α_c	ρ_t	ϕV_n (k)	
Wall 1	51.17	0.194	51.36	(2) #5	8	1041	10	10410	2	0.0078	4735	Okay
Wall 2	1.18	0.005	1.18	(2) #5	32	258	8	2064	2	0.0024	444	Okay
Wall 3	1.03	0.002	1.03	(2) #5	32	246	8	1968	2	0.0024	423	Okay
Wall 4	1.03	0.001	1.03	(2) #5	32	246	8	1968	2	0.0024	423	Okay
Wall 5	1.18	0.000	1.18	(2) #5	32	258	8	2064	2	0.0024	444	Okay
Wall 6	1.18	0.001	1.18	(2) #5	32	258	8	2064	2	0.0024	444	Okay
Wall 7	57.88	0.199	58.08	(2) #5	16	1107	10	11070	2	0.0039	3104	Okay
Wall 8	99.81	0.677	100.49	(2) #5	24	696	8	5568	2	0.0032	1400	Okay
Wall 9	1.46	0.001	1.46	(2) #5	32	102	8	816	2	0.0024	175	Okay
Wall 10	1.25	0.001	1.25	(2) #5	32	96	8	768	2	0.0024	165	Okay
Wall 11	1.76	0.001	1.76	(2) #5	32	110	8	880	2	0.0024	189	Okay
Wall 12	1.46	0.001	1.46	(2) #5	32	102	8	816	2	0.0024	175	Okay
Wall 13	1.25	0.001	1.25	(2) #5	32	96	8	768	2	0.0024	165	Okay
Wall 14	1.76	0.001	1.76	(2) #5	32	110	8	880	2	0.0024	189	Okay
Wall 15	83.49	0.478	83.97	(2) #5	24	624	10	6240	2	0.0026	1387	Okay
Wall 16	35.71	0.173	35.89	(2) #5	24	389	10	3890	2	0.0026	865	Okay

DRIFT AND DISPLACEMENT

The overall drift of a building should be limited as much as possible. The drift is a serviceability consideration that relates to the rigidity of each of the shear walls. The higher a building, the more important the overall drift of a building becomes a factor. The wind drift is limited to an allowable drift of $\Delta = \ell/400$. The wind controls the drift in the North/South direction of the building. The seismic forces control the drift in the East/West direction. The seismic drift is limited to an allowable drift of $\Delta = 0.015h_{sx}$. For the Fairfield Inn and Suites the allowable building drift limit will be:

$$\Delta_{limit} = (1224")/400 = 3.06"$$

Each floor will be examined independently to determine an approximate story displacement and story drift, adding up to overall building drift. A hand calculation was done to determine the displacements on each floor, keeping in mind that the modulus of elasticity and rigidity change as the f'c of shear walls supporting up to level 4 changes from f'c = 8000 to f'c = 5000. The hand calculations done were determined using the following equation:

$$\Delta_{cantilever} = \Delta_{flexural} + \Delta_{shear}$$

The ETABS model also analyzed the story drift of the building. The building drifts were taken in the x-direction which related to the east/west forces, and in the y-direction which related to the forces in the north/south direction. The drift in the x-direction was 0.61", and 1.84" in the y-direction. Drifts in both directions are less than 3.06", therefore well within the limits enforced. The hand calculations done according to drift are an approximation. In order to computer the story drift and displacements of all the shear walls working together by hand would be very intricate and beyond the scope of this assignment. ETABS does analyze the drift and displacements with all the shear walls working together as a lateral resisting system, therefore, the values computed by hand can't be directly compared with the ETAB results.

The actual hand calculations used to determine the drift and displacement can be found in Appendix G and tables 11a, 11b, and 11c for walls 7, 8, and 16.

OVERTURNING

Moments caused against the building could result in overturning affects. The lateral forces against the building result in overturning moments. The foundation for the Fairfield Inn and Suites would experience the most impact from overturning moments. The dead load of the building would serve as the system to resist the overturning. Table 8a shows the moments due to wind and seismic loads. In the north/south direction, the wind loads controlled and the seismic loads in the east/west direction. These moments are transformed into axial loads and transmitted through the lateral elements to the foundation. A rough estimate was done to check if the overturning would be an issue to the Fairfield Inn and Suites. Stresses due to the lateral loads were compared with the stresses due to the self weight of the building resisting. The stresses from the lateral loads are a small fraction of the stresses from the dead loads; therefore the foundation will have minimal overturning affects. Since moments are present, there will however be a force along the perimeter of the building. Detailed calculations of the overturning check can be found in Appendix H.

Table 8a – Overturning Moments						
			N/S Wind Forces		E/W Seismic Forces	
Floor	Height Above Ground - Z (ft)	Story Height (ft)	Lateral Force F_x (k)	Moment Total (ft-k)	Lateral Force F_x (k)	Moments M_x (ft-k)
PH Roof	112.66	10.00	6.35	683.81	4.88	525.16
Roof	102.66	10.00	26.11	2549.55	64.58	6307.25
10	92.66	9.33	24.02	2114.02	68.64	6039.95
9	83.33	9.33	23.69	1863.69	59.57	4686.25
8	74.00	9.33	23.27	1613.48	50.84	3524.68
7	64.66	9.33	22.91	1374.77	42.46	2547.34
6	55.33	9.33	22.47	1138.49	34.48	1747.17
5	46.00	9.33	22.05	911.35	26.95	1113.84
4	36.66	9.33	21.47	687.00	19.91	636.87
3	27.33	9.33	20.81	471.58	13.45	304.82
2	18.00	18.00	38.43	345.85	7.89	71.00
1	0.00	0.00	0.00	0.00	0.00	0.00
		Total =	251.58	13753.59	393.65	27504.33

CONCLUSION

In analyzing the existing lateral system of the Fairfield Inn and Suites, the loads determined in the technical report 1 were applied to the lateral system of the building. The loads were factored using the ASCE 07-05 load combinations for strength design. It was determined through the ETABs model output that the controlling load combinations were $1.2D + 1.6W + 1.0L + 0.5 L_r$ in the North/South direction and the combination $0.9D + 1.0E$ in the East/West direction. The wind controlled in the north/south direction because that side of the façade is larger than the other side resulting in higher wind forces in that direction than seismic loads in that direction. The seismic controls in the east/west direction because of the result of the poor soil site class D.

The ETABS model was used as a reference and in comparisons to verify that the model and hand calculations were providing similar and reasonable results. In comparison, it was determined the values computed by hand were to be used in all subsequent calculations. This was determined because it was concluded after finding the center of rigidity of each floor that the model was taking the slab as the rigid diaphragm into account as a member providing lateral resistance rather than just the shear walls. Also, with this being the first attempt at using ETABS to model the building, there was some uncertainty as to whether everything was input under the proper assumptions that the hand calculations made. Therefore, to ensure consistency in the assumptions made and to verify only the shear walls were analyzed as acting to resist the lateral forces, the hand calculations were used in each analysis.

Through this analysis, it confirms that looking into the shear walls as the only lateral resisting system was reasonable. Torsion was present in the building due to the eccentricity between the center of mass and the center of rigidity for each floor of the building. This added torsional shear in addition to the direct shear acting on the shear walls. A shear strength check was done to determine the thickness and reinforcement of the shear walls was designed sufficiently to resist the total shear. The overall building drift was determined by ETABS to be within the allowable limits of the building determined by the code. The story drifts and displacements determined by hand were also found to be within the allowable limits. Since the calculations neglect the core shear walls working as a unit, the drifts and displacements can only be an approximation. The values are most likely smaller than actual story drifts for the building. Overturning was found present in the building due to the lateral loads on the building, but a stress check determined that the self weight of the building resists these loads making this issue irrelevant to the Fairfield Inn and Suites. At this stage, the overall analysis done determines that the shear walls designed were satisfactory to resist the various load combinations present on the building.
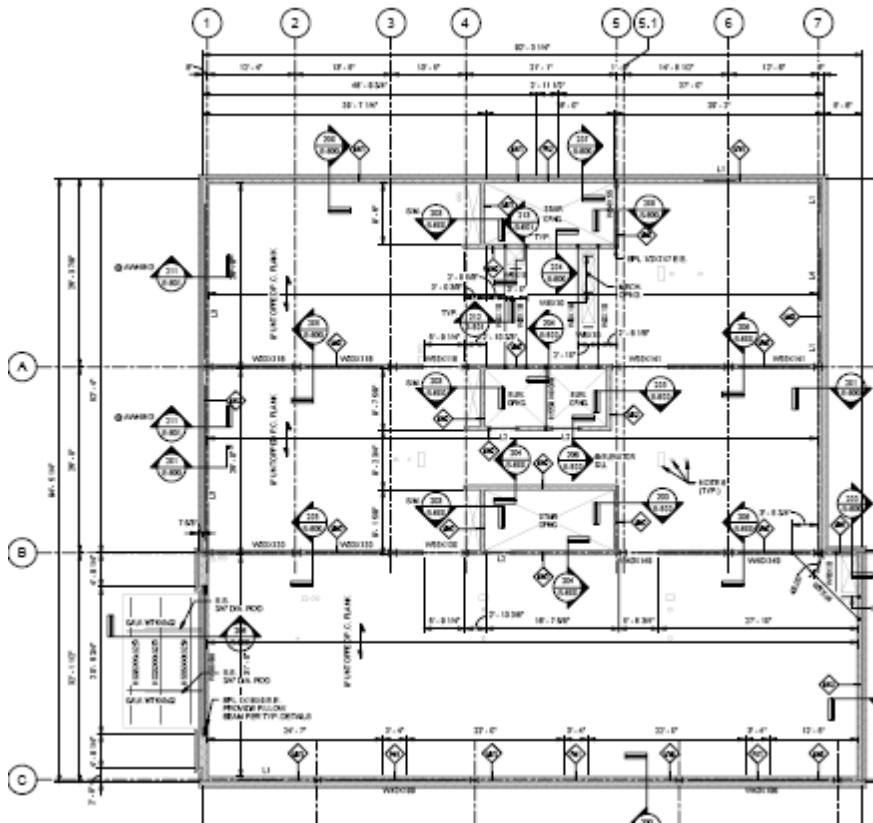
APPENDIX A

Building Layout

(This page is left blank intentionally)

Foundation Plan

Basement Plan
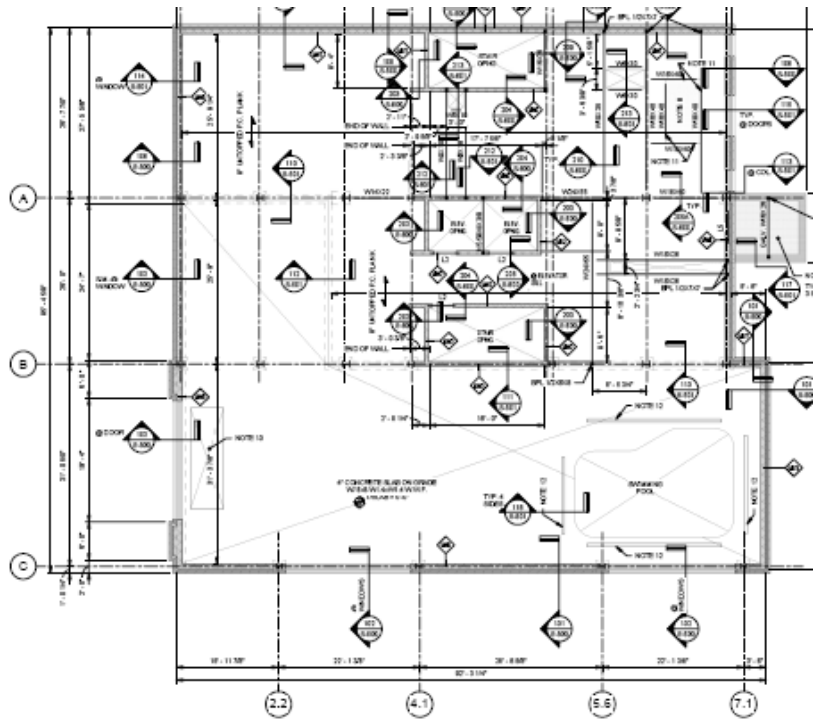
First Floor Framing Plan

Second Floor Framing Plan

Third thru Tenth Floor Framing Plan

Roof/Penthouse Roof Plan

APPENDIX B

Shear Wall Elevations

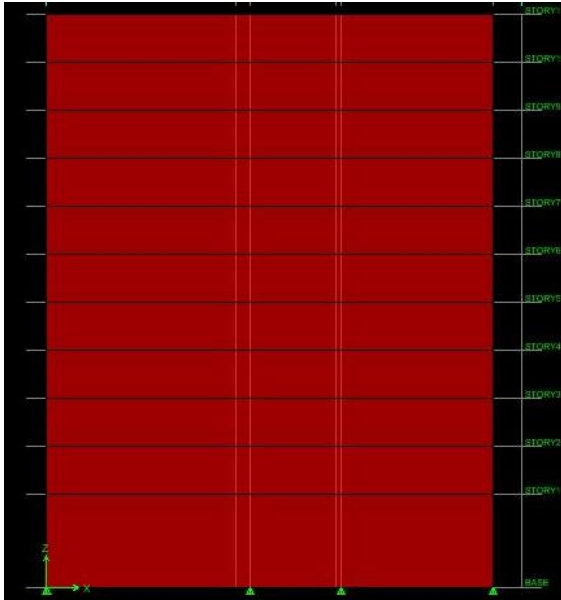(This page is intentionally left blank)
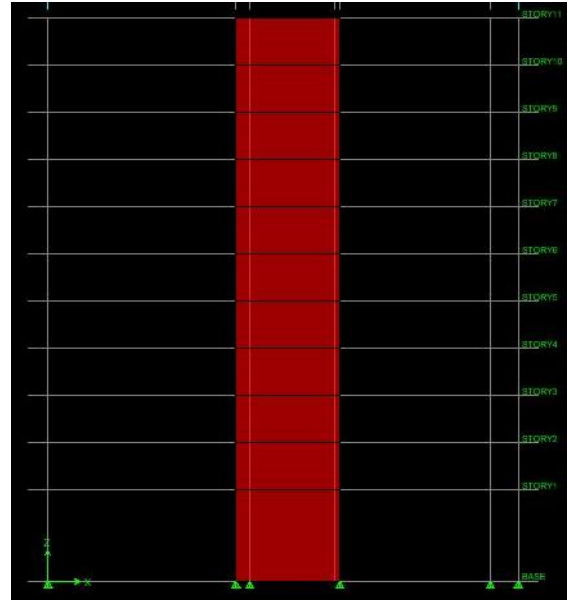
Figure 4 - Shear Wall 1 Elevation
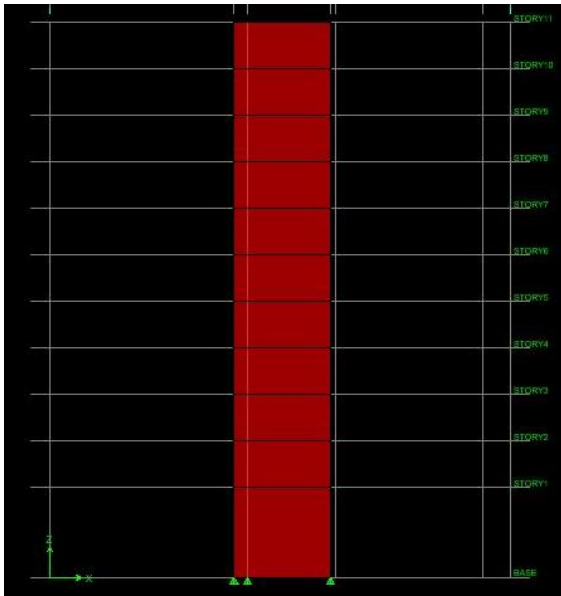

Figure 3 - Shear Walls 2, 5, 6 Elevations

Figure 5 – Shear Wall 3, 4 Elevations
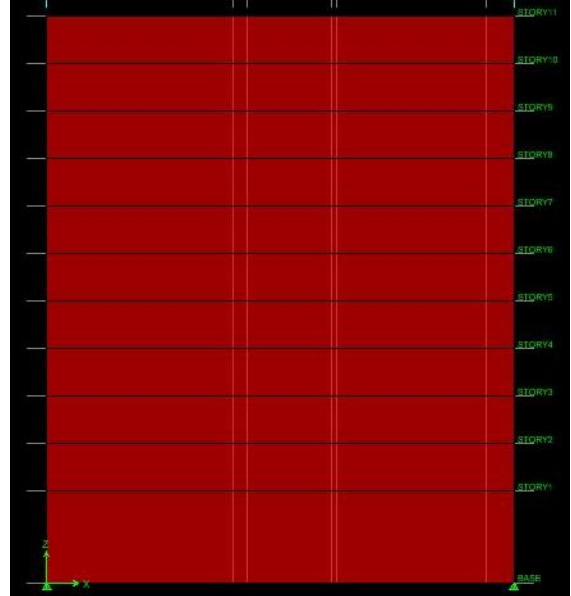
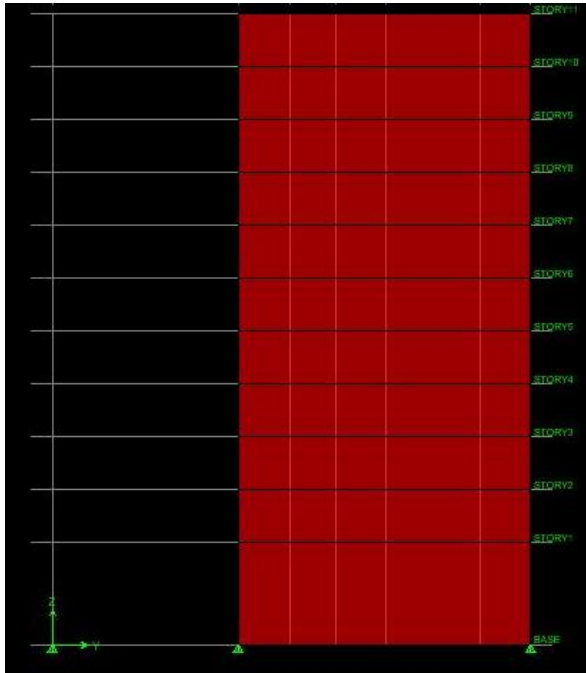*Figure 6 - Shear Wall 7 Elevation*
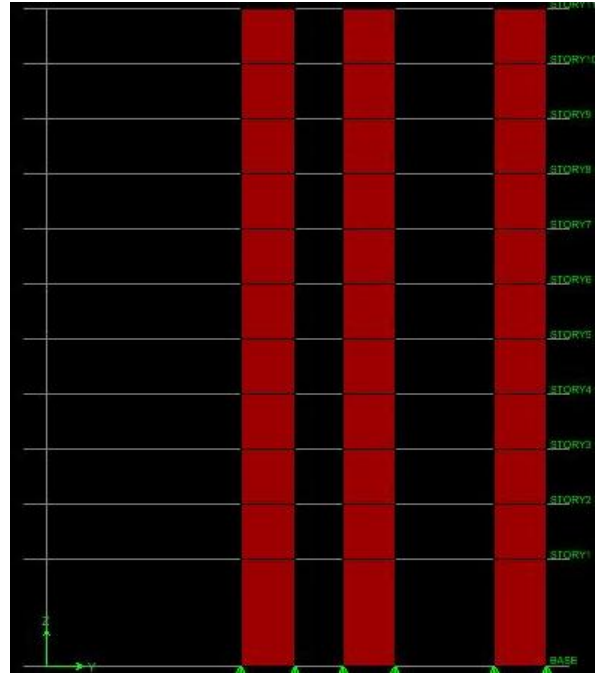
Figure 7 - Shear Wall 8 Elevation

Figure 8 - Shear Walls 9, 10, 11 Elevations
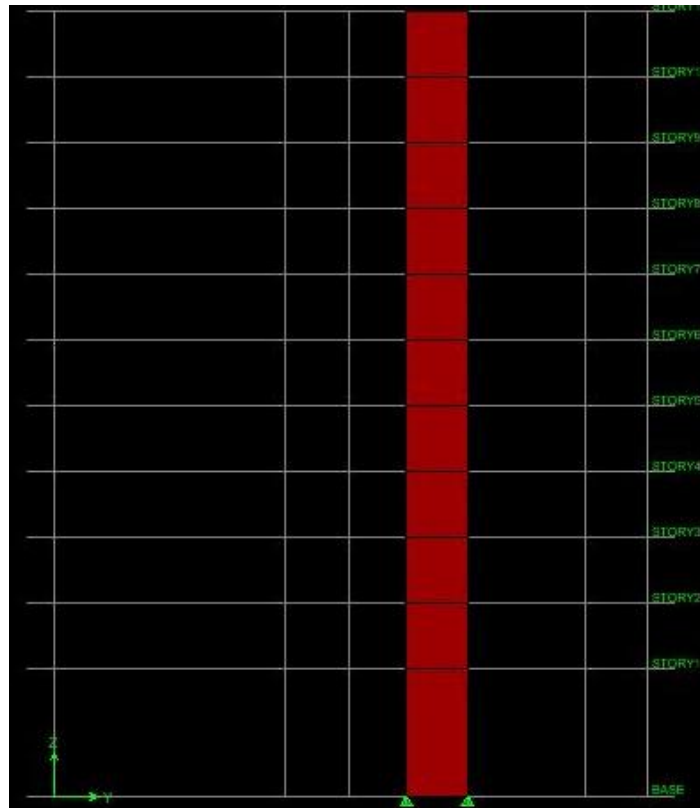

Figure 9 - Shear Walls 12 & 14 Elevations
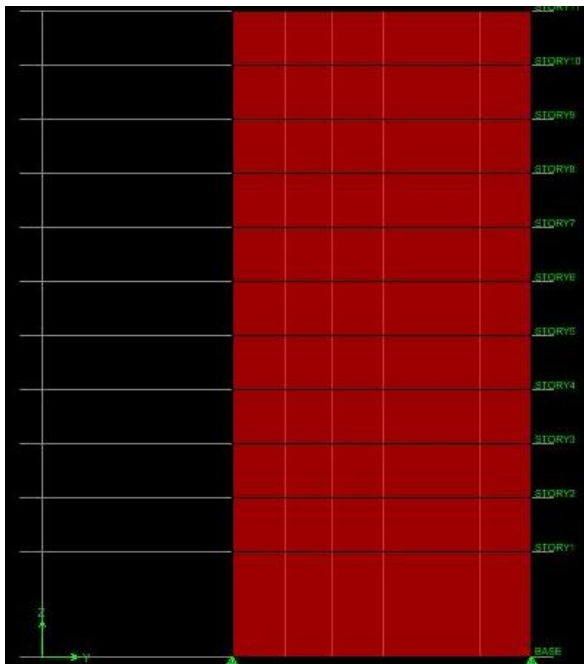
Figure 10 - Shear Wall 13 Elevation

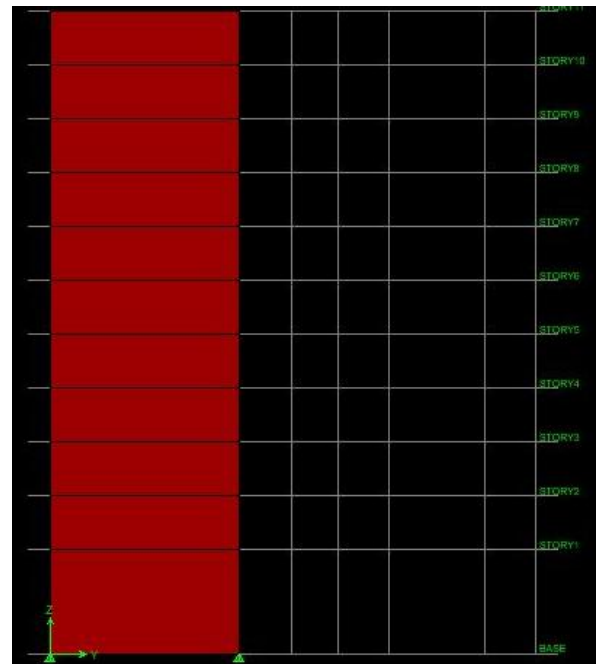*Figure 12 - Shear Wall 14 Elevation*

Figure 11 - Shear Wall 15 Elevation

APPENDIX C

LOADS

(This page is intentionally left blank)

Wind Loads

TECH REPORT 1 - CALCULATIONS
A. SMITH

WIND LOADS

METHOD 2 - analytical procedure
· Determine wind variables

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

interpolate K_z:
(table 6-3)
case 2

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

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

↳varies by level

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

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

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

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

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

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

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

2/3

WIND LOADS (cont)

$h = 107.66$

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

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

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

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

East/West $B = 83'$

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

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

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

g_a/g_v should be 3.4

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

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

C_p - pressure coefficient

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

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

(q_z)
(q_h)

• 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)

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

North/South
   example @ level 2: $P_z = 12.16 \, (0.806)(0.8) - 20.47(-0.18)$
                       $P_z = 11.53 \text{ psf}$

                   $P_h = 20.47(0.806)(-0.5) - 20.47(0.18)$
                   $P_h = -11.93 \text{ psf}$

3/3

WIND PRESSURE (cont)

East/West

example @ level 2:

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

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

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

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

* wind pressures for each story calculated in table   *

◦ force of windward (ONLY)

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

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

◦ Force Total pressure

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

◦ Windward shear story

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

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

◦ Total shear story

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

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

Table 9a

Wind Loads (North/South Direction) B = 91'-0" L = 83'-0"													
Level	Height above ground - z (ft.)	Story Height (ft.)	K_z	q_z	Wind Pressure (psf) Windward	Wind Pressure (psf) Leeward	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)
PH Roof	112.66	10.00	1.02	20.67	17.02	-11.93	28.95	3.73	6.35	3.73	6.35	401.92	683.81
Roof	102.66	10.00	1.00	20.27	16.75	-11.93	28.69	15.25	26.11	18.98	32.46	1488.97	2549.55
10	92.66	9.33	0.97	19.66	16.36	-11.93	28.30	13.89	24.02	32.87	56.48	1222.43	2114.02
9	83.33	9.33	0.94	19.05	15.97	-11.93	27.90	13.56	23.69	46.43	80.17	1066.63	1863.69
8	74.00	9.34	0.90	18.24	15.45	-11.93	27.38	13.13	23.27	59.56	103.45	910.26	1613.48
7	64.66	9.33	0.87	17.63	15.06	-11.93	26.99	12.78	22.91	72.34	126.36	766.88	1374.77
6	55.33	9.33	0.83	16.82	14.53	-11.93	26.47	12.34	22.47	84.68	148.83	625.13	1138.49
5	46.00	9.34	0.79	16.01	14.01	-11.93	25.94	11.91	22.05	96.59	170.88	492.13	911.35
4	36.66	9.33	0.74	15.00	13.36	-11.93	25.29	11.34	21.47	107.93	192.35	362.82	687.00
3	27.33	9.33	0.68	13.78	12.57	-11.93	24.51	10.67	20.81	118.60	213.16	241.93	471.58
2	18.00	18.00	0.60	12.16	11.53	-11.93	23.46	18.88	38.43	137.48	251.59	169.92	345.85
1	0	0	0	0	0	0	0	0	0	137.48	251.59	0	0
								Σ Windward Story Shear =		137.48	kips		
								Σ Total Story Shear =		251.59	kips		
								Σ Windward Moment =		7749.01	ft-k		
								Σ Total Moment =		13753.59	ft-k		

Table 9b

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

Seismic Loads

Seismic Force Resisting System: Example of Floor Weights Found

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

$^1/_2$

TECH 1 CALCULATIONS
A SMITH

SEISMIC LOADS

○ $S_{MS} = F_a S_s$

$S_{MS} = 1.6(0.125)$ $\boxed{S_{MS} = 0.2}$

○ $S_{DS} = {}^2/_3 (S_{MS})$ $\boxed{S_{DS} = 0.133}$

• $S_{M1} = F_v S_1$

$S_{M1} = 2.4(0.049)$ $\boxed{S_{M1} = 0.1176}$

○ $S_{d1} = {}^2/_3 (S_{M1})$ $\boxed{S_{d1} = 0.0784}$

○ T_a (approximate fund. period) = $C_r h_n{}^x$

$$T_a = 0.02 (112.66)^{0.75}$$

$$\boxed{T_a = 0.692 \ s}$$

○ $T = T_a (C_u) = 0.692(1.7) = \boxed{1.17 \ s}$

○ $C_s = \min \begin{cases} \dfrac{S_{D1}}{T(R/I)} = \dfrac{0.0784}{1.17(^2/_1)} = \boxed{0.0334} \geq 0.01 \\[2mm] \dfrac{S_{DS}}{R/I} = \dfrac{0.133}{2} = 0.0665 \\[2mm] \dfrac{S_{D1} T_L}{T^2(R/I)} = \dfrac{0.0784(12)}{1.17^2(2)} = 0.344 \end{cases}$

○ $K = 0.75 + 0.5(T) = 0.75 + 0.5(1.17) = \boxed{1.335 = K}$

SEE EXCEL SHEET FOR FLOOR WEIGHTS

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

TOTAL BUILDING WEIGHT

• $W_T = 7505.12(51.96) + 7505.12(154.26) + 8(7505.12)(150.58)$

     $+ 7505.12(123.57) + 557.68(110.94)$

$W_T = 11578 \ ^K$

$^2/_2$

SEISMIC LOADS (CONT)

- base shear (V)

$$V = C_s W_T = 0.034(11578)$$

$$\boxed{V = 393.65^K}$$

○ $W_x h_x^K$ varies @ height
example for level 4: $W_x = 1130.16$ K
$h_x = 36.66'$
$K = 1.335$

$$= 1130.16(36.66)^{1.335}$$

$$\boxed{= 138464}$$

$\sum W_i h_i^K$ = Sum of $W_x h_x^K$ for each floor $= \boxed{2738259 \ ft\text{-}K}$

○ $C_{v_x} = \dfrac{W_x h_x^K}{\sum W_x h_x^K}$ varies @ heigh

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

• $F_x = C_{v_x} V$ (lateral force)

example @ floor 6: $C_{v_x} = 0.088$
$V = 393.65$
$F_x = 0.088(393.65)$

$$\boxed{F_x = 34.48 \ K}$$

• Story Shear (V_x)

V_x = lateral force (F_x) @ level + (F_x) @ all levels above
example floor 9: $V_x = F_x(PH) + F_x(Roof) + F_x(10) + F_x(9)$

$$\boxed{V_x = 197.67 \ K}$$

○ Moments (M_x)

M_x = (trib area ht.) x F_x
example @ 9: $M_x = (78.665)(59.57)$

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

APPENDIX D

Load Distribution

(This page is intentionally left blank)

Rigidity/Relative Stiffness

° RIGIDITY   1/4

FIRST FLOOR
SHEAR WALL
LAYOUT

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

$E = 57000\sqrt{f'c}$

$E = 57000\sqrt{8000} = 5.098 \times 10^6$ PSI (FLOORS 1-3)

$E = 57000\sqrt{5000} = 4.030 \times 10^6$ PSI (FLOORS 4-10)

$t = 8"$ & $10"$

h = height from base to top each level (varies)

L = length of wall element

wall 1:

216"   1041"   FLOOR 2 / FLOOR 1

$$R_{1-1} = \frac{(5.098 \times 10^3 \, KSi)(10")}{4\left(\frac{216}{1041}\right)^3 + 3\left(\frac{216}{1041}\right)} = 77452$$

wall 2, 5, 6:

216"   258"   FLR 2 / FLR 1

$$R_{2-1} = \frac{(5.098 \times 10^3)(8")}{4\left(\frac{216}{258}\right)^3 + 3\left(\frac{216}{258}\right)} = 8394$$

wall 3, 4:

216"   246"   FLR 2 / FLR 1

$$R_{3-1} = \frac{(5.098 \times 10^3)(8")}{4\left(\frac{216}{246}\right)^3 + 3\left(\frac{216}{246}\right)} = 7635$$

wall 7:

216"   1107"   FLR 2 / FLR 1

$$R_{7-1} = \frac{(5.098 \times 10^3)(10")}{4\left(\frac{216}{1107}\right)^3 + 3\left(\frac{216}{1107}\right)} = 82883$$

$$\Sigma R = R_{1-1} + R_{2-1} + R_{3-1} + R_{4-1} + R_{5-1} + R_{6-1} + R_{7-1} = 158723$$

2/4

- RELATIVE STIFFNESS

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

wall 1: $= \dfrac{50980}{157736} = 38.6\%$

wall 2,5,6 $= \dfrac{8394}{157736} = 4.2\%$

wall 3,4 $= \dfrac{7635}{157736} = 3.8\%$
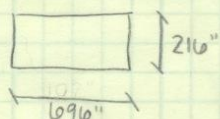
wall 7 $= \dfrac{66307}{157736} = 41.3\%$

○ CENTER RIGIDITY
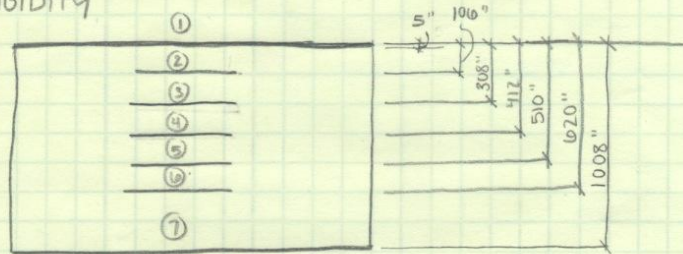
$$\frac{\Sigma R \cdot d}{\Sigma R} = \frac{(77452)(5) + (8394)(106) + (8394)(510) + (8394)(620) + 7635(412) + 7635(308) + 82883(1008)}{158723}$$

y-coordinate = 497.07

○ RIGIDITY (x-coord)

wall 8:

$$R_{8-1} = \frac{(5.098 \times 10^3 \, \text{KSi})(10)}{4\left(\frac{216}{696}\right)^3 + 3\left(\frac{216}{696}\right)} = 48525$$

wall 9 & 12:
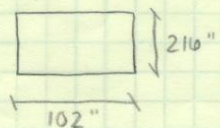
$$R_{9-1} = \frac{(5.098 \times 10^3 \, \text{KSi})(8)}{4\left(\frac{216}{102}\right)^3 + 3\left(\frac{216}{102}\right)} = 920$$

wall 10 & 13:
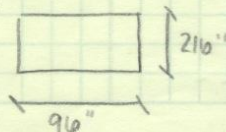
$$R_{10-1} = \frac{(5.098 \times 10^3)(8)}{4\left(\frac{216}{96}\right)^3 + 3\left(\frac{216}{96}\right)} = 780$$

$^{3}/_{4}$
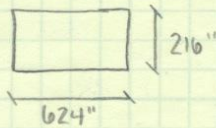
Wall 11 & 14:

216"

110"
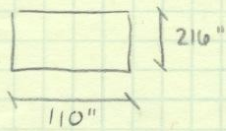
$$R_{11-1} = \frac{(5.098 \times 10^3)(8)}{4\left(\frac{216}{110}\right)^3 + 3\left(\frac{216}{110}\right)} = 1127$$
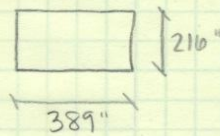
Wall 15:

216"

624"

$$R_{15-1} = \frac{(5.098 \times 10^3)(10)}{4\left(\frac{216}{624}\right)^3 + 3\left(\frac{216}{624}\right)} = 42329$$

Wall 16:

216"

389"

$$R_{16-1} = \frac{(5.098 \times 10^3)(10)}{4\left(\frac{216}{389}\right)^3 + 3\left(\frac{216}{389}\right)} = 21688$$
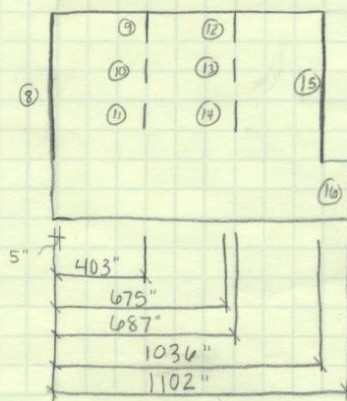
$$\Sigma R = 118195$$

○ RELATIVE STIFFNESS

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

Wall 8:   = 41.05%

Wall 9 & 12: 0.78%

Wall 10 & 13: 0.66%

Wall 11 & 14: 0.95%

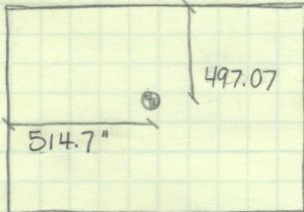Wall 15:   35.81%

Wall 16:   18.35%

○ CENTER RIGIDITY

⑨ ⑫
⑩ ⑬   ⑮
⑧
⑪ ⑭

⑯

5"
403"
675"
687"
1036"
1102"

4/4

$$\frac{\Sigma R \cdot d}{\Sigma R} = \frac{(5)(48525) + (403)(920) + (403)(780) + (403)(1127) + (675)(780) + (687)(920) + (687)(1127)}{+ (1036)(42329) + (1102)(21688)}$$

$$118195$$

x-coordinate = 514.7"

center rigidity of floor 1:

497.07

514.7"

○ Rigidity, Relative stiffness, Center of rigidity for each floor can be found in tables

• CENTER of MASS

　○ Since building footprint is rectangular center of mass coordinates:
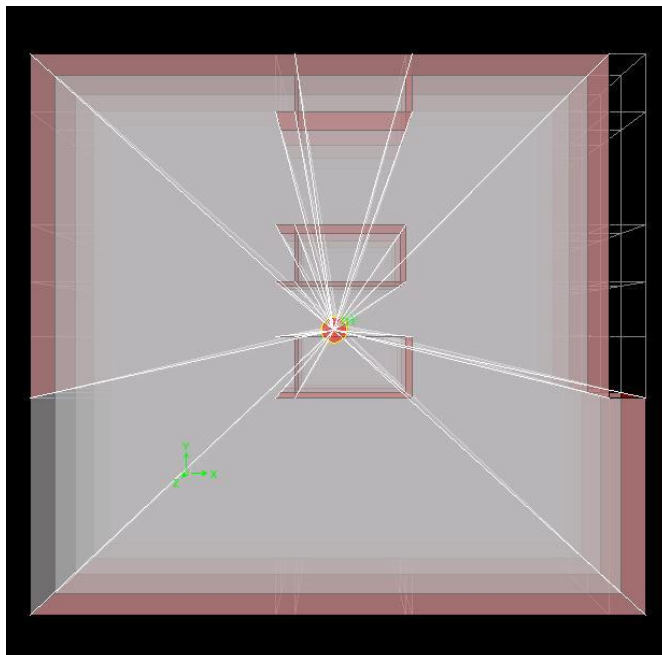
$$(554.97", 524.4")$$

Figure 9.1 – ETABS model of finding the center of rigidity of the floors

Table 10a

Supported Floor	Height (in.)	Wall 8 $\ell = 696$	Wall 9 $\ell = 102$	Wall 10 $\ell = 96$	Wall 11 $\ell = 110$	Wall 12 $\ell = 102$	Wall 13 $\ell = 96$	Wall 14 $\ell = 110$	Wall 15 $\ell = 624$	Wall 16 $\ell = 389$	Σ Rigidity	Center of Rigidity (x)
Floor 10	112	102077	4748	4140	5605	4748	4140	5605	90778	53147	274987	614.1
Floor 9	224	46393	833	705	1023	833	705	1023	40398	20464	112377	599.9
Floor 8	336	26855	267	224	331	267	224	331	22760	9863	61122	580.4
Floor 7	448	13443	92	77	114	92	77	114	11089	4213	29311	563.1
Floor 6	560	8961	48	40	59	48	40	59	7218	2480	18952	549.5
Floor 5	672	6203	28	23	35	28	23	35	4899	1562	12834	539.2
Floor 4	784	4430	18	15	22	18	15	22	3444	1039	9021	531.4
Floor 3	896	3251	12	10	15	12	10	15	2495	722	6541	525.4
Floor 2	1008	2443	8	7	10	8	7	10	1857	521	4872	520.8
Floor 1	1224	1491	5	4	6	5	4	6	1117	301	2937	514.7

Table 10b

Supported Floor	Height (in.)	Wall 1 $\ell = 1041$	Wall 2 $\ell = 258$	Wall 3 $\ell = 246$	Wall 4 $\ell = 246$	Wall 5 $\ell = 258$	Wall 6 $\ell = 258$	Wall 7 $\ell = 1107$	Σ Rigidity	Center of Rigidity (y)
Floor 10	1224	5083	92	80	80	92	92	5844	11364	535.7
Floor 9	1112	6309	122	106	106	122	122	7213	14102	534.0
Floor 8	1000	7931	167	145	145	167	167	9009	17731	531.9
Floor 7	888	7993	186	162	162	186	186	9013	17888	529.5
Floor 6	776	10351	274	239	239	274	274	11578	23227	526.6
Floor 5	664	13654	425	372	372	425	425	15135	30806	523.2
Floor 4	552	18426	707	621	621	707	707	20232	42021	519.0
Floor 3	440	25668	1292	1141	1141	1292	1292	27917	59742	513.6
Floor 2	328	37651	2679	2391	2391	2679	2679	40587	91058	506.6
Floor 1	216	61227	6635	6035	6035	6635	6635	65520	158723	497.1

APPENDIX E

Torsion

(This page is intentionally left blank)

TORSION $\frac{1}{1}$

· overall Building torsion

in order to find $M_{t,tot}$ we need the inherent moment M_t, which is due to eccentricity and the accidental moment, M_{ta} which is the assumed displacement of center of mass

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

Factored lateral force = 1.6W
$$= 1.6 \times (\text{force of total wind pressure})$$
@ story
  – can be found in wind tables for each direction

$$M_t = (\text{factored lateral force}) \times (\text{eccentricity})$$
$$\text{eccentricity} = \text{center of rigidity} - \text{center of mass}$$

· example @ floor 4 in N/S direction:

$$e = 525.4 - 554.97 = -29.52 = -2.46$$

factored lateral force = $1.6 \times 21.47 = 34.35^K$

$$M_t = 34.35 \times -2.46 = -84.64^{K-ft}$$

$$M_{ta} = (\text{factored lateral force}) \times (5" \text{ assummed displacement each way of center of mass})$$

· example @ floor 4 in N/S direction: (supporting floor 4)
center of mass = 554.97"

%5 displacement in each direction = 55.5" = 4.62'

factored lateral force = $1.6 \times 21.47 = 34.35^K$

$$M_{ta} = 34.35^K \times 4.62 = 158.71^{K-ft}$$

· $M_{t,tot} = M_t + M_{ta} = -84.64 + 158.71 = 74.07$

overall Building torsion for each floor in each direction in tables

APPENDIX F

Shear

(This page in intentionally left blank)

SHEAR   ½

- controlling loads

    North/South: $1.2D + 1.6W + L + 0.5Lr$

    East/West: $0.9D + 1.0E + 1.6H$

- Direct shear

$$= \frac{(\text{factored story force}) \times \text{relative stiffness \%}}{100}$$

    example: FLOOR 7 in N/S direction @ wall 8

$$= 36.66 \text{ K} \times 0.4586 = 17.33$$

- direct shear values for each floor can be found in Tables

- TORSIONAL SHEAR

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

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

    example: wall 7 supporting floor 6

    - factored story shear $= (1.6 \times 121.5^K) = 194.37^K$
    - center of rigidity (y-coord) $= 526.6''$
    - center of mass (y-coord) $= 524.4''$

        $e = 526.6 - 524.4 = 2.2''$

    - $R_i = 0.491$

2/2

- location of wall 7 = 1008" (y-coord)

$d_i = wall_i - COR_i = 1008" - 526.6" = 481.7"$

$R_i \times d_i^2 = 0.491 \times 481.7^2 = 113929.1$

- J = 508740.5   (from table)

$$T = \frac{(194.37)(2.2)(481.7)(0.491)}{508918}$$

$T = 0.199 \, K$

- calculated values for all shear walls supporting Floor 6 can be found in Table

SHEAR STRENGTH $\frac{1}{1}$

ACI 318.08 sect. 21.9.4

– structural walls shall not exceed V_n

$$\phi V_n = \phi A_c \left(\alpha_c \lambda \sqrt{f'_c} + p_t f_y \right)$$

$\phi = 0.75$

$A_{cv} =$ gross area of concrete

$\alpha_c =$ coefficient 2.0 if $h_w/\ell_w > 2.0$

$p_t = \dfrac{A_v}{s \cdot h}$ $S =$ shear reinforcement spacing
$h =$ thickness of wall ($8"$; $10"$)

example: Wall 8 supporting Floor 6

– direct shear = distributed direct force on all floors above
floor 6 of wall 8

FROM TABLE $= 15.87 + 16.65 + 17.08 + 17.33 + 17.38 = 99.81^k$

– torsional shear = from table

$V_u = 99.81 + 0.677 = 100.49$

Vertical Reinforcement: (2) #5 @ 8" OC

$$p_t = \frac{(2)(0.31)}{(8)(10)} = 0.0078$$

$$A_{cv} = (1041)(10) = 10410 \text{ in}^2$$

$$\phi V_n = 0.75(10410) \left[2 \frac{\sqrt{5000}}{1000} + 0.0078(60) \right]$$

$$\phi V_n = 4758^k$$

$$\phi V_n = 4758^k > 100.49^k \quad \text{OKAY!}$$

· the remaining calculated shear strengths for these
walls can be found in Table

APPENDIX G

Drift and Displacement

(This page is intentionally left blank)

STORY DISPLACEMENT

<div align="right">1/5</div>

- an approximate method used to determine story shear is to Δ_{cont} up the building

- story drift

$$\Delta = 0.015 h_{sx}$$

h_{sx}: story height below story x
(ASCE-7 table 12.12-1)

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

$$\Delta_{flex} = \frac{Ph^3}{3E_cI} \qquad \Delta_{shear} = \frac{1.2 Ph}{E_r A}$$

$$\Delta_{cont} = \frac{Ph^2}{3E_cI} + \frac{1.2 Ph}{E_r A}$$

$$E_c = 57000\sqrt{8000} = 5.098 \times 10^3 \text{ Ksi} \quad (\text{story 1-3})$$
$$E_c = 57000\sqrt{5000} = 4.030 \times 10^3 \text{ Ksi} \quad (\text{story 4-10})$$

E_r = Modulus of rigidity
$$= 0.4 E_c$$
$$E_r = 2.04 \times 10^3 \text{ Ksi} \quad (\text{story 1-3})$$
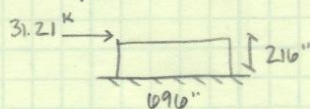$$E_r = 1.61 \times 10^3 \text{ Ksi} \quad (\text{story 4-10})$$

$$A = (\text{length}) \times (\text{thickness}) \qquad \text{thickness} = 10'' \text{ or } 8''$$
$$I = \frac{(\text{thickness}) \times (\text{length})^3}{12}$$

- example for Wall 8 in N/S direction
$$1.2D + 1.6W + 1.0L + 1.5L_r$$

FLOOR 2 supported:

31.21^k → ⌐ 216''

696''

$$\Delta_1 = \frac{31.21^k (216)^3}{3(5.098 \times 10^3)(I)} + \frac{1.2(31.21)(216)}{2.04 \times 10^3 (6960)}$$

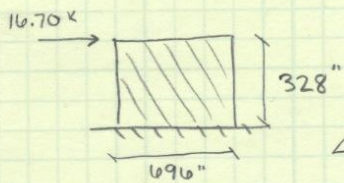$$A = 10'' \times 696'' = 6960 \text{ in}^2 \qquad \Delta_1 = 0.0000732 + 0.000570$$
$$I = \frac{10'' \times (696)^3}{12} = 280901280 \qquad \boxed{\Delta_1 = 0.000643 \text{ in}}$$
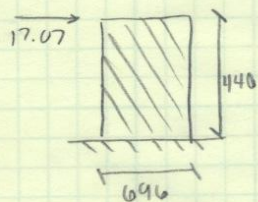
FLOOR 3 Supported:

16.70^k

328"

696"

$A = 6960 \ in^2$
$I = 280961280 \ in^3$

$$\Delta_2 = \frac{16.70(328)^3}{3(5.098 \times 10^3)(I)} + \frac{1.2(16.70)(328)}{2.04 \times 10^3 (6960)}$$

$$\Delta_2 = 0.000137 + 0.000463$$

$$\boxed{\Delta_2 = 0.000600"}$$

FLOOR 4 Supported:

17.07

440

696

$A = 6960 \ in^2$
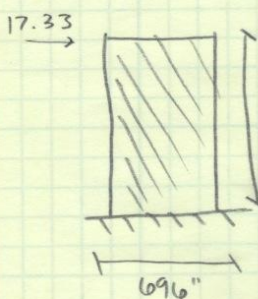$I = 280961280 \ in^5$

$$\Delta_3 = \frac{17.07(440)^3}{3(5.098 \times 10^3) I} + \frac{1.2(17.07)(440)}{2.04 \times 10^3 (6960)}$$

$$\Delta_3 = 0.000338 + 0.000635$$

$$\boxed{\Delta_3 = 0.000973"}$$

FLOOR 5 Supported:

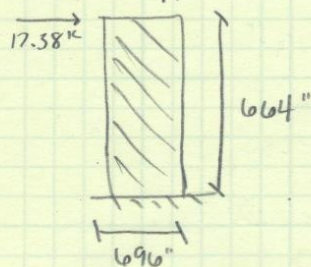17.33

552"

696"

$A = 6960 \ in^2$
$I = 280961280 \ in^3$

$$\Delta_4 = \frac{17.33(552)^3}{3(4.030 \times 10^3) I} + \frac{1.2(17.33)(552)}{1.610 \times 10^3 (6960)}$$

$$\Delta_4 = 0.000858 + 0.001025$$

$$\boxed{\Delta_4 = 0.001882"}$$

Floor 6 Supported:

17.38^K

664"

696"

$A = 6960 \text{ in}^2$
$I = 2809 61280 \text{ in}^3$

$$\Delta_5 = \frac{17.38 (664)^3}{3(4.030 \times 10^3)(I)} + \frac{17.38 (1.2)(664)}{1.610 \times 10^3 (6960)}$$

$$\Delta_5 = 0.001498 + 0.0012236$$

$$\boxed{\Delta_5 = 0.002733 \text{ "}}$$

Floor 7 Supported:

17.33^K
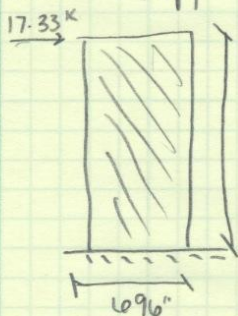
776"

696"

$A = 6960 \text{ in}^2$
$I = 2809 61280 \text{ in}^3$

$$\Delta_6 = \frac{17.33 (776)^3}{3(4.030 \times 10^3) I} + \frac{17.33 (1.2)(776)}{1.610 \times 10^3 (6960)}$$

$$\Delta_6 = 0.002384 + 0.001440$$

$$\boxed{\Delta_6 = 0.003825 \text{ "}}$$

Floor 8 Supported:

17.08^K

888"

696"
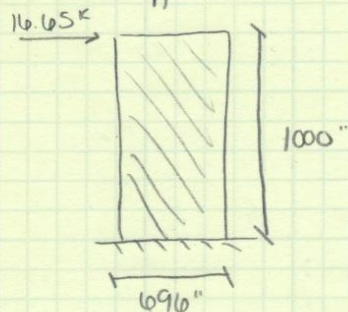
$A = 6960 \text{ in}^2$
$I = 2809 61280 \text{ in}^3$

$$\Delta_7 = \frac{17.08 (888)^3}{3(4.030 \times 10^3) I} + \frac{17.08 (1.2)(888)}{1.610 \times 10^3 (6960)}$$

$$\Delta_7 = 0.003520 + 0.001624$$

$$\boxed{\Delta_7 = 0.005144 \text{ "}}$$

Floor 9 supported:
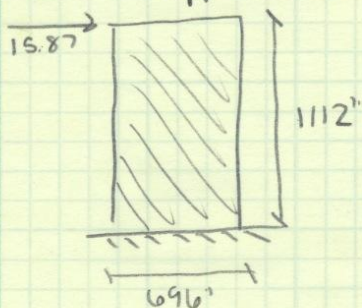

16.65^k

$1000"$

$696"$

$A = 6960 \text{ in}^2$

$I = 280961280 \text{ in}^3$

$$\Delta_8 = \frac{16.65(1000)^3}{3(4.030\times10^3)(I)} + \frac{16.65(1.2)(1000)}{1.610\times10^3(6960)}$$

$\Delta_8 = 0.004903 + 0.001783$

$\boxed{\Delta_8 = 0.006686"}$

Floor 10 supported:

15.87

$1112"$

$696"$

$A = 6960 \text{ in}^2$

$I = 280961280 \text{ in}^3$

$$\Delta_9 = \frac{15.87(1112)^3}{3(4.030\times10^3)I} + \frac{15.87(1.2)(1112)}{1.610\times10^3(6960)}$$

$\Delta_9 = 0.006423 + 0.001889$

$\boxed{\Delta_9 = 0.008312"}$

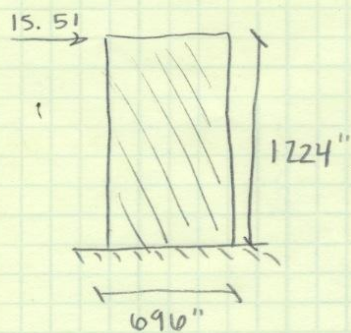Roof supported:

15.51

$1224"$

$696"$

$A = 6960 \text{ in}^2$

$I = 280961280 \text{ in}^3$

$$\Delta_{10} = \frac{15.51(1224)^3}{3(4.030\times10^3)I} + \frac{15.51(1.2)(1224)}{1.610\times10^3(6960)}$$

$\Delta_{10} = 0.008372 + 0.002033$

$\boxed{\Delta_{10} = 0.010404"}$

$\frac{4}{5}$

Overall wall displacement: 5/5

$\Sigma\Delta = 0.010404'' + 0.008312'' + 0.006686'' + 0.005144'' + 0.$

$+ 0.003825'' + 0.002733'' + 0.001882'' + 0.000973'' + 0.00060''$

$+ 0.0006643''$

$\Sigma\Delta = 0.04120''$

$\Delta_{LIMIT} = H/400 = 1224/400 = 3.06''$

$0.0412'' < 3.06''$ ✓OKAY

The calculated displacements and story drifts for walls 7, 8, 16 can be found in tables

Table 11a

							Wall 8 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 (in.)
Roof	15.51	4030	1610	10	696	1224	0.008372	0.002033	0.010404	0.000009
10	15.87	4030	1610	10	696	1112	0.006423	0.001889	0.008312	0.000007
9	16.65	4030	1610	10	696	1000	0.004903	0.001783	0.006686	0.000007
8	17.08	4030	1610	10	696	888	0.003520	0.001624	0.005144	0.000006
7	17.33	4030	1610	10	696	776	0.002384	0.001440	0.003825	0.000005
6	17.38	4030	1610	10	696	664	0.001498	0.001236	0.002733	0.000004
5	17.33	4030	1610	10	696	552	0.000858	0.001024	0.001882	0.000003
4	17.07	5098	2040	10	696	440	0.000338	0.000635	0.000973	0.000002
3	16.70	5098	2040	10	696	328	0.000137	0.000463	0.000600	0.000002
2	31.21	5098	2040	10	696	216	0.000073	0.000570	0.000643	0.000003
								Total Wall Displacement (in.) =	0.041202858	

Table 11b

							Wall 16 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 (in.)
Roof	8.07	4030	1610	10	389	1224	0.024966	0.001894	0.026859	0.000022
10	7.00	4030	1610	10	389	1112	0.016226	0.001491	0.017717	0.000016
9	6.12	4030	1610	10	389	1000	0.010313	0.001172	0.011485	0.000011
8	5.35	4030	1610	10	389	888	0.006319	0.000911	0.007230	0.000008
7	4.80	4030	1610	10	389	776	0.003779	0.000713	0.004492	0.000006
6	4.37	4030	1610	10	389	664	0.002160	0.000557	0.002716	0.000004
5	4.06	4030	1610	10	389	552	0.001152	0.000430	0.001582	0.000003
4	3.79	5098	2040	10	389	440	0.000431	0.000252	0.000683	0.000002
3	3.56	5098	2040	10	389	328	0.000167	0.000177	0.000344	0.000001
2	6.29	5098	2040	10	389	216	0.000085	0.000206	0.000290	0.000001
								Total Wall Displacement (in.) =	0.073398876	

Table 11c

							Wall 7 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 (in.)
Roof	10.85	4030	1610	10	1107	1224	0.001456	0.000895	0.002351	0.000002
10	9.92	4030	1610	10	1107	1112	0.000998	0.000743	0.001740	0.000002
9	9.70	4030	1610	10	1107	1000	0.000710	0.000653	0.001363	0.000001
8	9.42	4030	1610	10	1107	888	0.000483	0.000563	0.001046	0.000001
7	9.16	4030	1610	10	1107	776	0.000313	0.000479	0.000792	0.000001
6	8.83	4030	1610	10	1107	664	0.000189	0.000395	0.000584	0.000001
5	8.46	4030	1610	10	1107	552	0.000104	0.000315	0.000419	0.000001
4	7.97	5098	2040	10	1107	440	0.000039	0.000186	0.000226	0.000001
3	7.33	5098	2040	10	1107	328	0.000015	0.000128	0.000143	0.000000
2	12.45	5098	2040	10	1107	216	0.000007	0.000143	0.000150	0.000001
								Total Wall Displacement (in.) =	0.00881312	

APPENDIX H

Overturning

(This page is intentionally left blank)

OVERTURNING

½

M = 27504 ^{1}k

TOTAL SEISMIC
FORCE
393.65

(E-W DIRECTION)

Total Wind
Force
251.6 ^{K}

M = 13753.6 ^{1}k

(N-S DIRECTION)
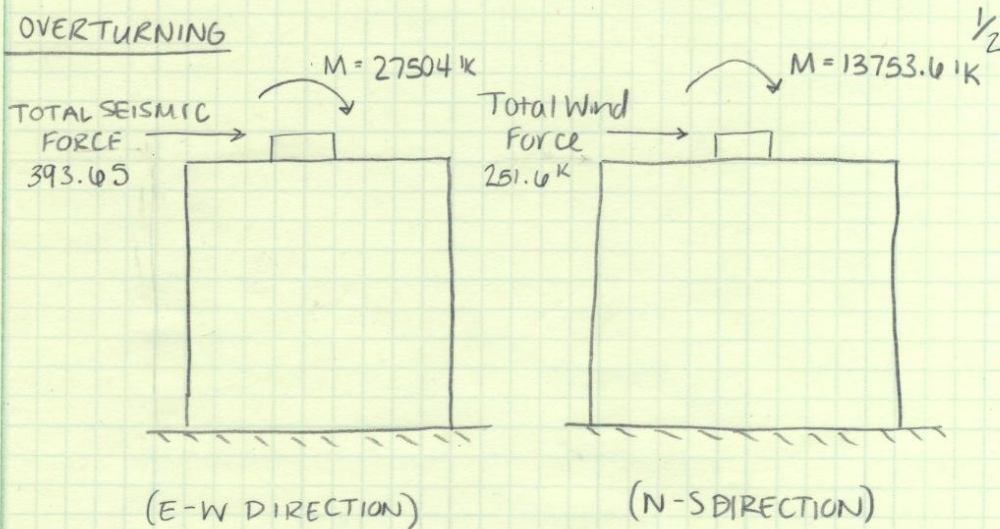
○ The lateral forces will create the overturning moment while the gravity loads (dead loads) will try and resist that moment.

○ Stresses to be determined to see if the gravity loads will exceed lateral loads

92.25'

84.4'
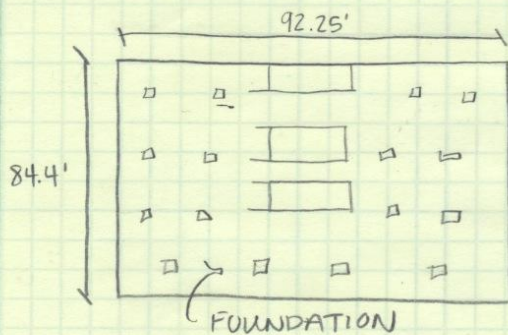
FOUNDATION

○ STRESS DUE TO DEAD LOADS:

$$= \frac{\text{weight of building}}{\text{square foot of foundation}}$$

$$= \frac{11578.23 \quad ^{K}}{(84.4)(92.25)} \times 1000 \text{ lb.}$$

$$= 1487.1 \text{ psf}$$

· Stress due to E/W Seismic:

$$= \frac{393.65^{K}(1000 \text{ lb})}{(84.4 \times 92.25)} = 50.56 \text{ psf}$$

$$= \frac{50.56}{1487.1} \times 100 = 3.4 \% \text{ of Dead Load}$$

2/2

- stresses due N/S wind:

$$= \frac{251.6^K \times 1000\ lb.}{(84.4)(92.25)} = 32.32\ psf$$

$$= \frac{32.32}{1487.1} \times 100 = 2.2\%\ of\ Dead\ Load$$

- Overturning will not be critical to the design of the building since the stresses of the lateral loads are such a smaller percentage of the gravity loads which are resisting the lateral forces.