Contents

Executive Summary

The purpose of Technical Report 3 is to investigate and analyze how lateral loads are distributed and resisted in the Hotel N.E.U.S. To achieve this, ASCE 7-05 was used to calculate the wind and seismic loads in detail and computer models were constructed.

The Hotel N.E.U.S. resists its lateral loads through a total of 23 masonry shear walls. The majority of these walls are in the North-South direction, also referred to as the Y direction in this report. Being 4 times shorter than the East-West direction this is necessary to prevent overturning.

Lateral loads are distributed to the floor diaphragm and resisted by the shear walls, which transfer the loads to strip foundations. The stiffness of each shear wall is calculated along with the relative stiffness per wall/floor. The center of mass and rigidity are also evaluated by hand and compared to computer model constructed in ETABS and RAM. The comparison proves to be successful and confirms that the models are fairly accurate.

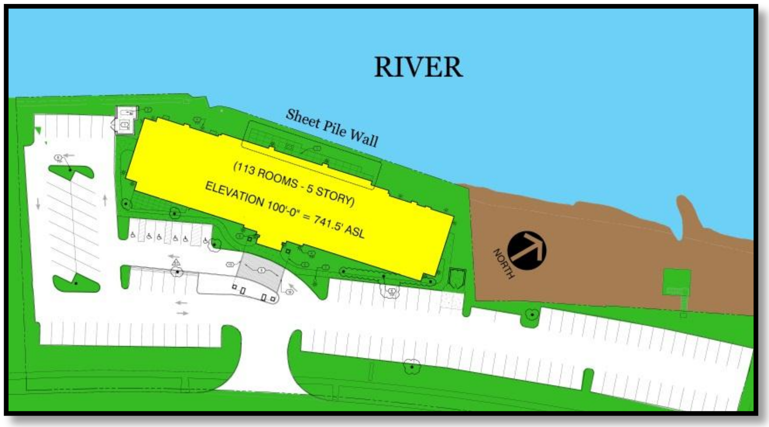
To further explore all the effects of lateral loads, 65 load combinations are developed to use in the ETABS model. This study allowed for a deep understanding of the way loads can be arranged and which ones are causes for concern in the Hotel N.E.U.S. In RAM the load cases and combinations are calculated via ASCE 7-05 with exposure, importance, and other input. This proved to be a good exercise in evaluating the output from different computer program.

It was determined that due to the weight of the building and low seismic response factor, the combination of dead load and earthquake was the controlling case. The base shear caused by seismic loads is capped based on the region. With the values being the same for each direction, the 3 shear walls in the East-West Direction (also referred to as X direction) were a cause for concern. Since there is only one line of shear walls, there is no ability for the building to resist torsion. Therefore a shear wall in this direction was checked with direct loads and found to be over capacity. The required length for the walls to resist the applied wind and seismic loads was calculated. Less than 5’ extra was needed for wind while nearly twice the building length was needed for seismic. Due to this analysis, it is likely that the design engineer did not consider seismic loads to be controlling. In reality, there will be some resistance due to other connected elements such as discontinuous walls that will take small amounts of shear and provide enough resistance for wind forces. Deflections and drifts were compared to code allowed values and deemed adequate. Overturning moments in the critical direction were found to be within an acceptable limit compared to the resisting moment.

Introduction

Located along a river in the Northeast United States (henceforth referred to as Hotel N.E.U.S.), this five story, 113 room hotel is constructed with masonry bearing walls and a precast concrete floor system. It stands in place of an old steel mill and was constructed as part of the area's development in the 1990's.

At its tallest, the building is 60'-8" tall with a long slender shape that allows for windows in every room. Its façade consists of arching exterior insulation finishing system (EIFS) and a brick veneer. The warm colors of beige and brown provide a sense of comfort and soothing that communicate the architecture's purpose, a place to rest.



All of the amenities of a hotel are included, such as a pool, fitness area, meeting room, ADA accessible rooms, and sunlight for all rooms. There is an overhang at the entrance allowing for drop off and pick up with protection from the elements. The Hotel N.E.U.S. provides 75,209 ft^2 of floor area to a location lacking such facilities. Construction started in October of 2011 and is slated to finish in November of 2012 and cost $9.2 million dollars.

Note: The overhang at the entrance is not considered in the analysis or evaluation of this building at any point.

All photos/plans/documents provided by Atlantic Engineering Services/Meyer Associates

Structural Overview

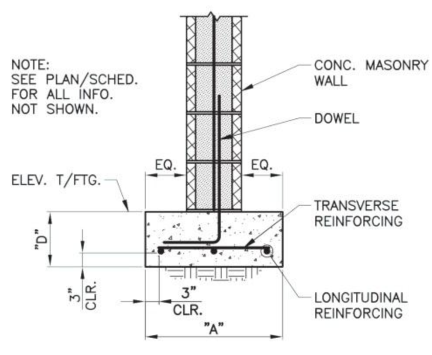
Foundations

Michael Baker Jr., Inc. provided the Geotechnical report in July of 2011. They included a history of the site that impacts the features below grade for this project. Pre-1986 the site of the Hotel N.E.U.S. was occupied by a steel mill. Cooling towers were located at the footprint of the current building while a gantry crane and tracks were to the Southwest. The sheet pile retaining wall was constructed in 1979. In 1990's a development of the area began and the mill was removed. Foundations and other below grade structures were usually removed to about to about one foot below grade. In 2001 a Damon's Restaurant and parking lot were constructed in the area that the hotel is to be located. Fill was added to the site during this time.

Geotechnical Consultants, Inc. drilled seven boring in April of 2001 to support Damon's Restaurant and those reports were included and mostly consisted of Slag and Concrete with little Silt. Terra Testing excavated four test pits and drilled thirteen test borings in April of 2011. They totaled 10 linear feet of rock and 282 linear feet of soil (see Figure 3 for location of all borings). The major finding in these tests was that there were buried concrete obstructions. They were determined to be the concrete pad that supported the cooling towers in the past.

The fill was considered to be suitable for a shallow spread foundation system. The bearing pressure was controlled by a limiting settlement of one inch and the capacity of the soil. The allowable bearing capacity of the soil increases with the size of the footing. Larger footings cause much higher stresses however, so the bearing pressure decreases with larger sizes (see Figure 1 for tables providing various sizes). A minimum of a 3' x 3' reinforced footing was suggested and no less than 16.7' center-to-center distance between wall footings. Footings bearing on the concrete pad were allowed a reduction of 1.5'.

Continuous wall footings range from 2'-0" wide to 9'-0" wide with typically #5 or #7 for longitudinal and transverse reinforcement. Column footings ranged from 6'x6'x1'-6" to 8'x8'x1'-8" (see Figure 1 for footing schedule). Typical piers are 24"x24" with 4-#6 vertical with #3 at 12" ties.



TYPICAL CONTINUOUS MASONRY WALL FOOTING DETAIL

CONTINUOUS WALL FOOTING SCHEDULE

MARK	WIDTH "A"	DEPTH "D"	LONGITUDINAL REINFORCING	TRANSVERSE REINFORCING	MARK
WF1	2'–0"	1'–0"	2–#5 CONT.	#5x1'–6" @ 24" O.C.	WF1
WF2	3'–0"	1'–0"	3–#5 CONT.	#5x2'–6" @ 24" O.C.	WF2
WF3	9'–0"	1'–6"	6–#7 CONT.	#7x8'–6" @ 12" O.C.	WF3
WF4	5'–0"	1'–0"	3–#7 CONT.	#7x4'–6" @ 18" O.C.	WF4
WF5	6'–0"	1'–3"	6–#5 CONT.	#5x5'–6" @ 18" O.C.	WF5

Figure 1: Continuous Masonry Wall Footing detail and schedule

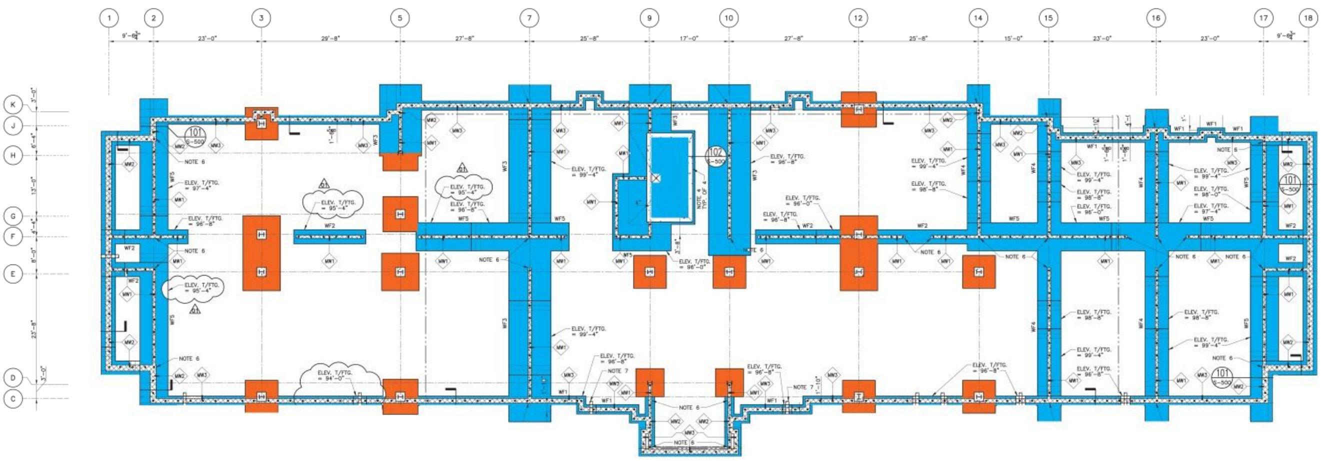


Figure 2: Foundation Plan.
Blue- wall footings
Orange- Column Footings

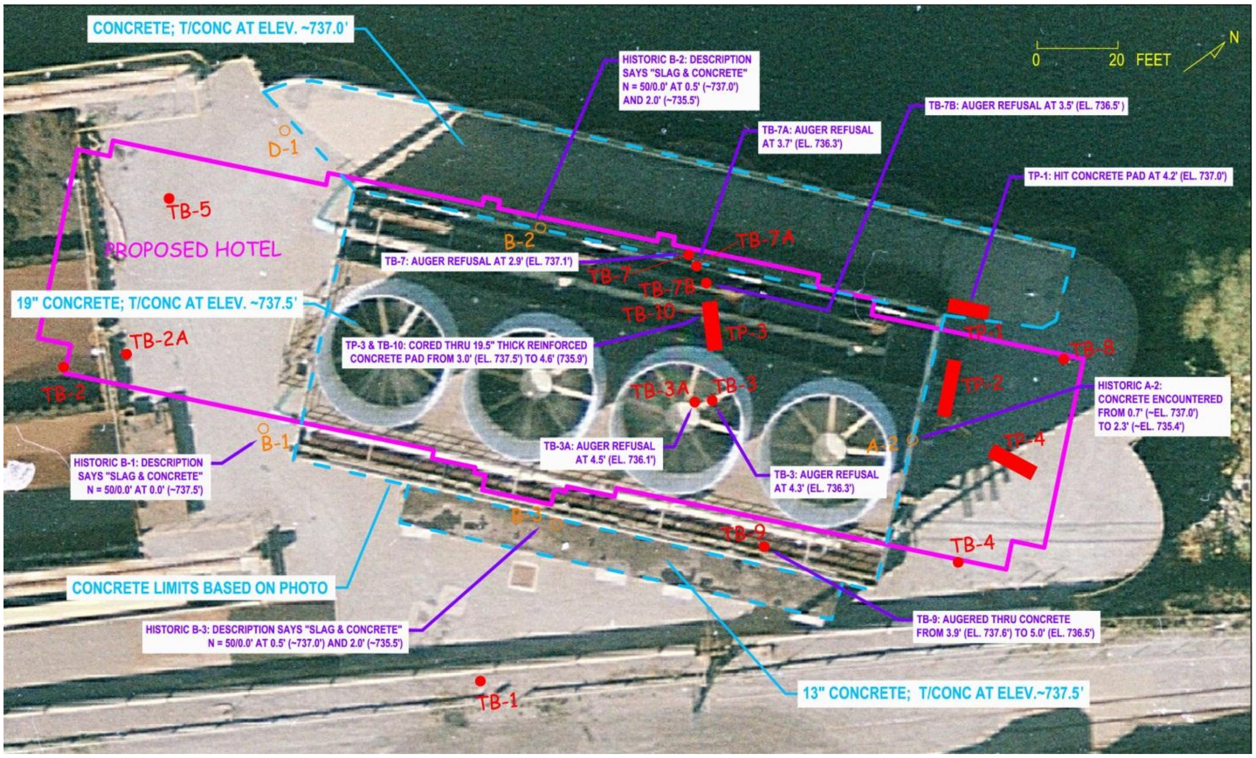


Figure 3: Site map showing test borings, existing mat foundation, hotel footprint, and location of former cooling towers.

Floor System

The floor system is composed of 8" Hollowcore precast concrete plank. There is a 3/4" topping to level off the floor since the planks have camber when they come out of production. The plank allows for long spans between the bearing walls. The smallest span is 15'-0" while the largest is 29'-8". Due to the large open spaces on the first floor, large transfer beams are used to carry the walls on the second floor up to the roof. These wide flange beams are approximately 30" in depth and weigh anywhere from 90 to 191 pounds per foot. Smaller beams span the corridor between walls and are much smaller, ranging from W6x25 to W24x68.

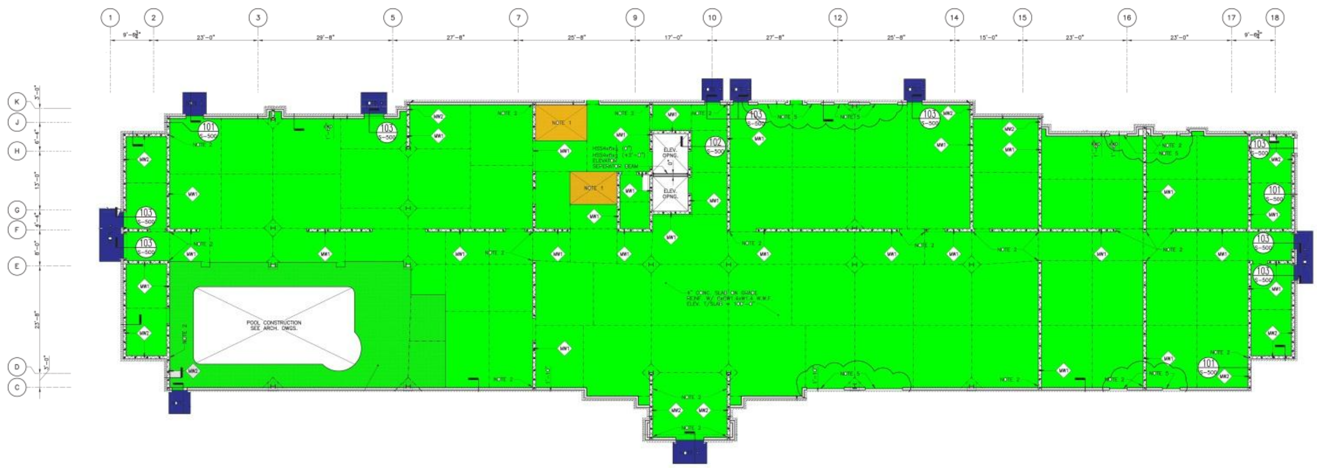


Figure 4: Slab on grade. Light green- 4" Conc. Slab on grade w/ 6x6W1.4xW1.4 W.W.F.

Orange- 3'-0" thick Conc. Slab w/ #5@12" O.C. Top and B.E.W. Isolated from adjacent slab.

Blue- Exterior 4" Conc. Slab on grade w/ 6x6W1.4xW1.4 W.W.F sloped away from building.

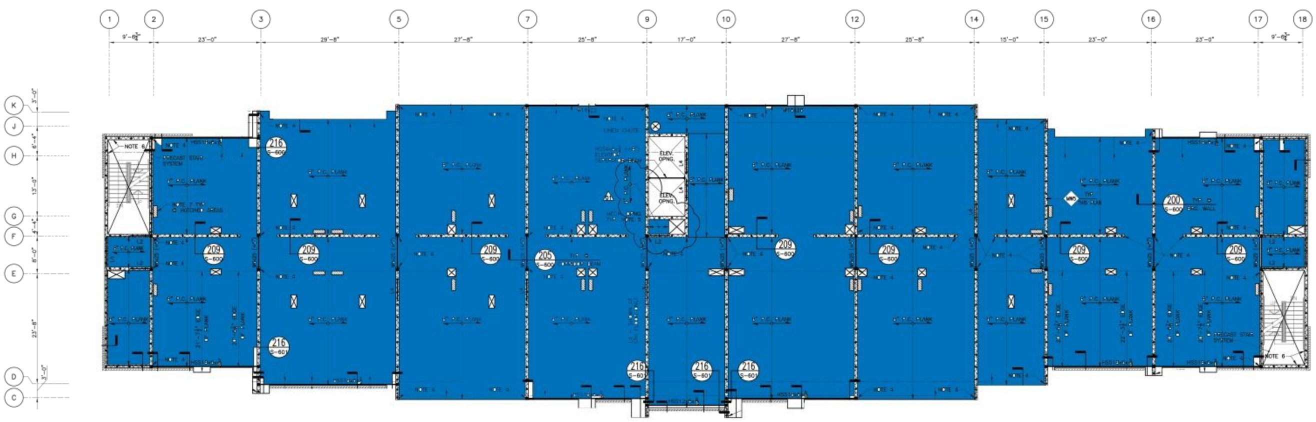


Figure 5: Typical Floor plank layout

Framing System

The framing system for the Hotel N.E.U.S consists of steel columns on the first floor mixed with masonry bearing walls. Due to the gathering areas and general openness of the first floor, steel columns are used. These columns only exist on this floor, save for column C12 and E12 that span the first two floors (see Figure 7) Everywhere else in the building, masonry walls are used to support the floor system. The exterior is supported by cold-formed steel (see Figure 7 for sections) Bays are typical except for on the second floor where an opening exists for an open ceiling breakfast region. The longest bearing wall is about 28' long, located on column line 9 near the center of the building where it is widest.

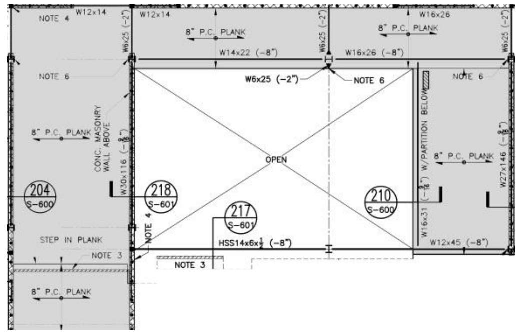
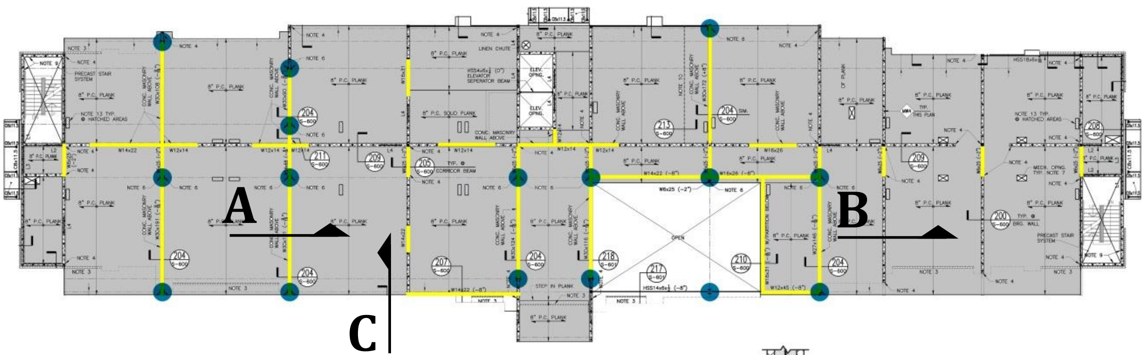
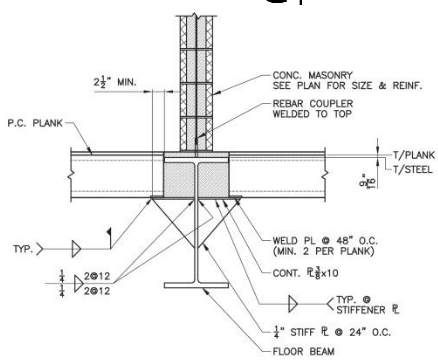
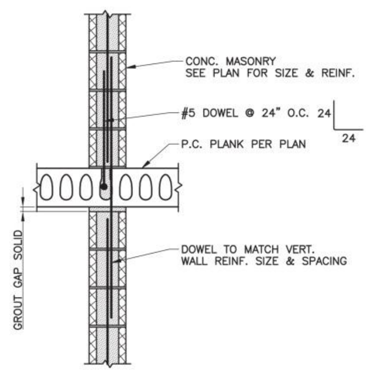


Figure 6: Open section on second floor

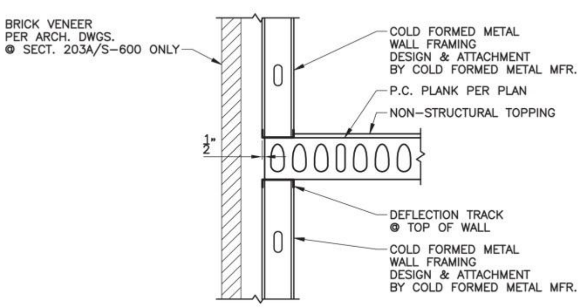




SECTION A- Beam carrying masonry wall



SECTION B- Plank on masonry wall



SECTION C- Plank resting on cold-formed steel at exterior

Figure 7: Second Story framing
Yellow indicates beams
Blue indicates columns

Lateral System

In the Hotel N.E.U.S, the lateral system consists is the same as the gravity system. Reinforced masonry shear walls provide the resistance to lateral loads applied to the building. The masonry is 8" wide with #5 bars at 24" on center. Cells with reinforcement are grouted solid. As with the gravity system, these walls are controlled by the fact that the first floor requires a space without obstructions. Therefore the shear walls are located in an irregular pattern shown in Figure 8. Due to the slenderness of the building, much more resistance is required perpendicular to the long side of the building.

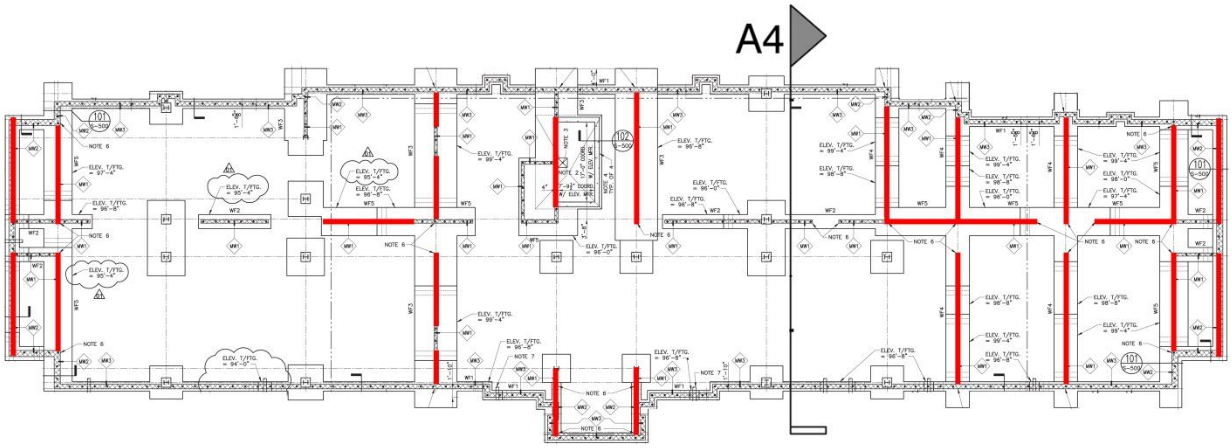


Figure 8: Location of shear walls on foundation plan

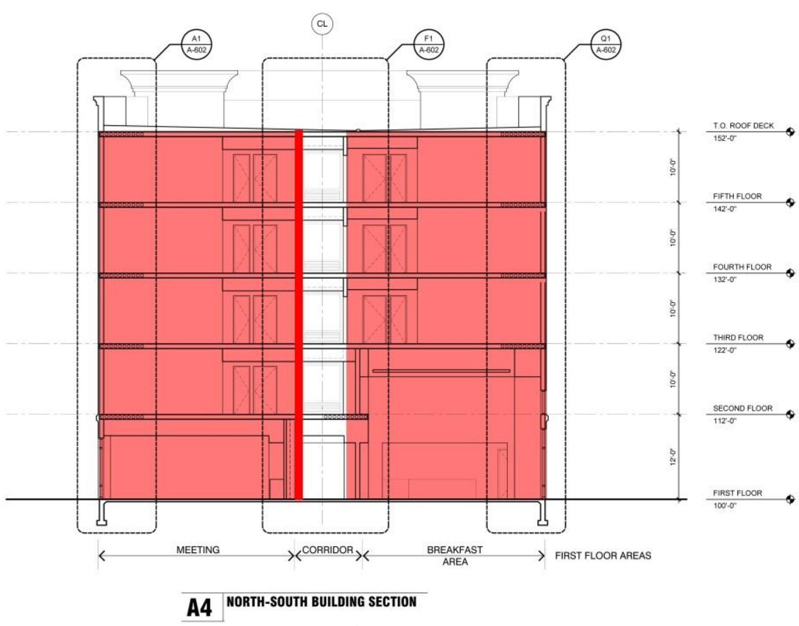
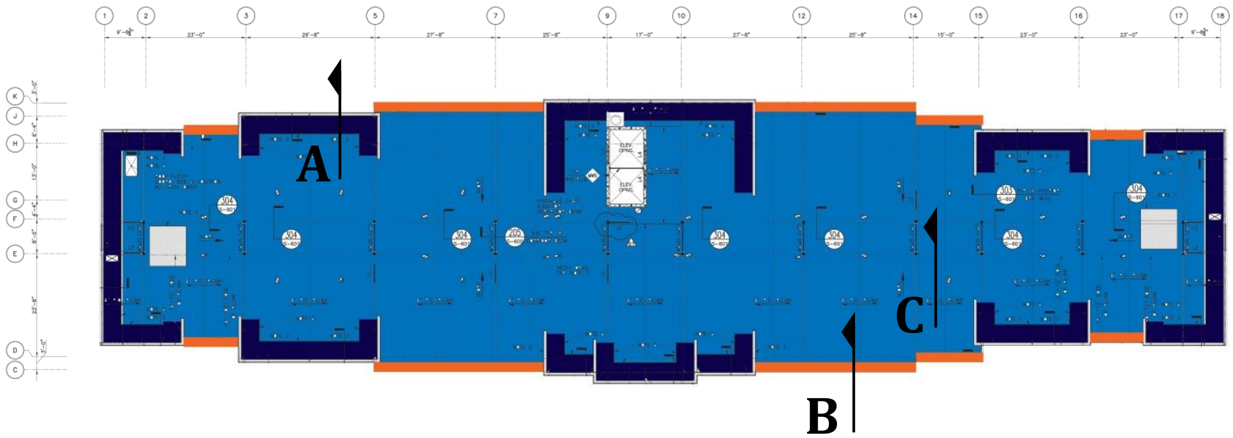
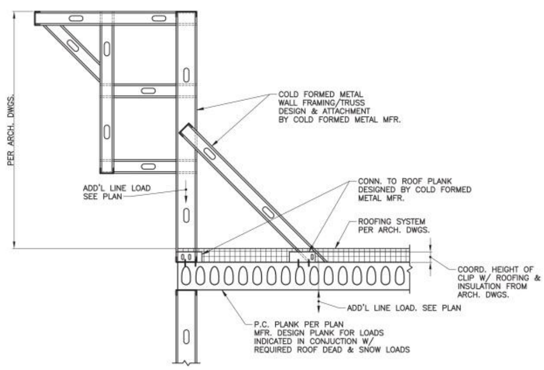


Figure 9: Section showing orientation of shear walls.

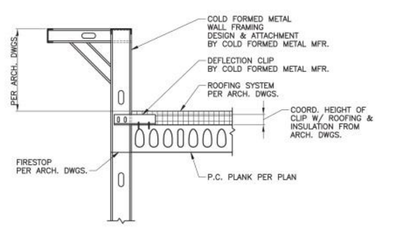
Roof System

As with the floor system, the roof is constructed of 8" Hollowcore Precast plank with insulation on top. A parapet constructed of cold-formed steel engrosses the entire perimeter and is to 8'-8" high. Mechanical units weighing 4,000 lbs each are located at either end of the roof.

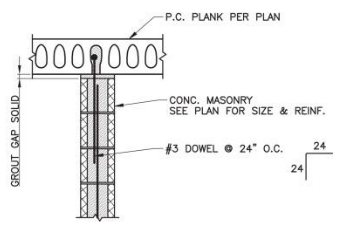




SECTION A- 8'-8" Cold-formed steel parapet wall



SECTION B- 5'-0" Cold-formed steel parapet wall



SECTION C- Roof plank on top of masonry wall

Figure 10: Roof layout.
Blue- 8" Hollowcore Precast Plank
Orange- 5'-0" Cold-formed steel parapet wall
Dark Blue- 8'-8" Cold-formed steel parapet wall

Materials

Listed in Figure 11 are the materials used in the construction of the Hotel N.E.U.S. They were gathered from the structural engineer's general notes and specifications.

Shallow Foundations Wall Footing Capacity	
Width	Allowable Bearing Pressure
2'-0"	4,100 PSF
3'-0"	4,600 PSF
4'-0"	4,500 PSF
5'-0"	3,800 PSF
6'-0"	3,250 PSF
7'-0"	2,800 PSF
8'-0"	2,500 PSF

Column Footing Capacity	
Width	Allowable Bearing Pressure
3'-0"	4,600 PSF
4'-0"	4,500 PSF
5'-0"	3,800 PSF
6'-0"	3,250 PSF
7'-0"	2,800 PSF
8'-0"	2,500 PSF
9'-0"	6,650 PSF
10'-0"	6,250 PSF
11'-0"	5,500 PSF

Reinforced Concrete	
Type	Design Compression Strength (f'c)
Foundations and Concrete Fill	3,000 PSI
Walls	4,000 PSI
Slabs and Grade	4,000 PSI
Reinforcement	
Deformed Bars	ASTM A625 GRADE 60
Deformed Bars (weldable)	ASTM A706, GRADE 60
Welded Wire Fabric	ASTM A185

Figure 11: Material Standards used in Hotel N.E.U.S.

Masonry	
Mortar	ASTM C270
	Type M for all F'm = 2,500 PSI, Type S for all structural masonry
Grout	F'c = F'm but no less than 2,000 PSI

Face Brick
ASTM C216, Grade SW, Type FBS absorption not more than 9% by dry weight per ASTM C67.

Structural Steel	
W shapes	ASTM 992
M, S, C, MC, and L shapes	ASTM A36
HP shapes	ASTM A572, GRADE 50
Steel Tubes (HSS shapes)	ASTM A500, GRADE B
Steel Pipe (Round HSS)	ASTM A500, GRADE B
Plates and Bars	ASTM A36
Bolts	ASTM A325, TYPE 1, 3/4" U.N.O.

Galvanized Structural Steel	
Structural Shapes and Rods	ASTM A123

Precast Concrete	
Type	Design Compression Strength (f'c)
Reinforcement (deformed)	ASTM A 615/A 615M, Grade 60
Welded Wire Reinforcement:	ASTM A 185
Pretensioning Strand	ASTM A 416/A 416M, Grade 250 or Grade 270, uncoated, 7-wire, low-relaxation strand wire or ASTM A 886/A 886M, Grade 270, indented, 7-wire, low-relaxation strand
Portland Cement	ASTM C 150

Figure 12: Material Standards used in Hotel N.E.U.S.

Design Codes

Because of the wide variety of materials used on this project there are also many different codes to abide by. These are listed in Figure 13. The codes used for analysis in this thesis are listed in Figure 14. For a list of other codes used see Appendix A.

Design Codes	
Reinforced Concrete	Building Code Requirements for Structural Concrete (ACI 318, latest)
	Specifications for Structural Concrete (ACI 301, latest)
Masonry	Building Code Requirements for Masonry Structures (ACI 530)
	Specifications for Masonry Structures (ACI 530.1)
Precast Concrete	Building Code Requirements for Structural Concrete (ACI 318, latest)
	Commentary (ACI 318R, latest)
	PCI Design Handbook - Precast and Prestressed Concrete (PCI MNL 120)
Structural Steel	Specification for Structural Steel Buildings (ANSI/AISC 360-05)
Metal Decking	Steel Roof Deck Specifications and Load Tables (Steel Deck Institute, latest edition)
Cold Formed Steel	Most current edition of the "North American Specification for the Design of Cold-Formed Steel Framing"
Wind and Seismic	ASCE 7-05
Loads	International Building Code 2009

Figure 13: Codes used by the engineer of record to design this structure

Thesis Analysis Codes	
Reinforced Concrete	Building Code Requirements for Structural Concrete (ACI 318-11)
Precast Concrete	PCI Design Handbook - Precast and Prestressed Concrete (PCI MNL 120)
Structural Steel	AISC Steel Manual 14th Edition, A
	AISC 360 2010 Specification for Structural Steel Buildings
Wind and Seismic	ASCE 7-05
Loads	International Building Code 2009
Masonry	Building Code Requirements forMasonry (ACI 530-05)

Figure 14: Codes used for thesis

Gravity Loads

The dead loads for this structure were either provided by the engineer of record or assumed by referencing structural handbooks. The plank weight was obtained using PCI Manual 120 and Masonry walls were determined using NCMA TEK 14-13B. The density was assumed as 105 lb/ft^3 as it was described as "medium" in the specifications. The topping is to level the surface since the camber of the plank will cause it to be uneven. These loads prove to be very similar to the overall load used by the engineer of record as the spot checks performed give good results.

Dead Loads	
Location	Load (psf)
8" Precast Plank	56
3/4" Topping	6
MEP/Misc.	5
Ceiling	3
Roof Insulation	12
C.F. Studs	5
Roof	20
Masonry Walls	43-53

Figure 15: Dead Loads for Hotel N.E.U.S.

Live loads were listed in the general notes on sheet S001. All of them were in accordance with the International Building Code 2009. Due to the typical layout of floors in a hotel, 40 psf was used on the entire floor except for stairwells on floors two through five. The engineer of record used live load reduction when determining loads for the beams, columns, and column footings. However, there was no reduction for the wall footing.

Live Loads			
Location	Design Live Load (psf)	IBC 2009 Live Load (psf)	Reference Note
Public Areas	100	100	Residential - hotels and multifamily dwellings - public rooms and corridors serving them
Guest Rooms and Corridors	40	40	Residential - hotels and multifamily dwellings - private rooms and corridors serving them
Paritions	20	20	
Stairs	100	100	Stairs and exits - all other
Roof	20	20	Roofs - ordinary flat, pitched, and curved roofs

Figure 16: Live Load comparison and references

Lateral Load Distribution

The Hotel N.E.U.S. has a gravity and lateral system constructed of masonry. Masonry shear walls act as cantilevers with strip footings in the ground. This means that in the Hotel N.E.U.S., the shear walls were taken as the ones that continue from roof to foundation so loads can be disappated. The steel framing on the first floor interupts a large portion of the bearing walls and although they will resist some shear, they were not taken into account in this report as a conservative assumption. No details are made to indicate that moment can be transferred through these steel sections. In Figure 17 the shear walls are shown in red while bearing walls are shown in blue.

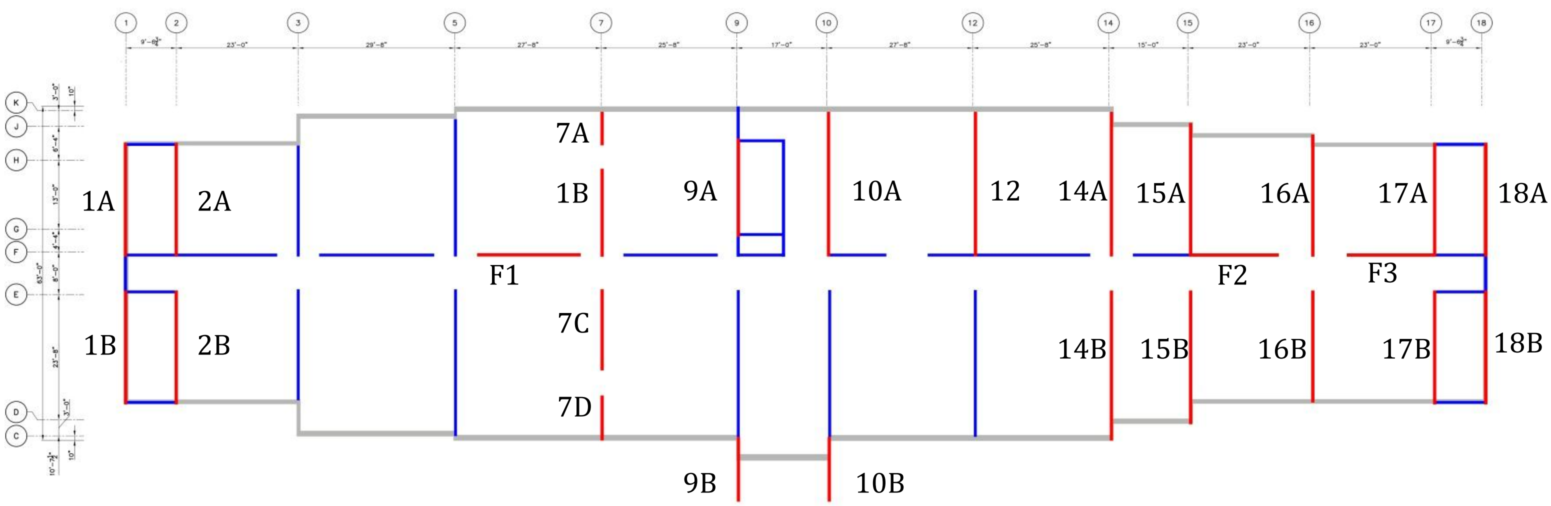


Figure 17: Blue-Gravity Walls
Red-Lateral Walls

Shear Walls	
1A	F1
1B	F2
7A	F3
7B	
7C	
7D	
9A	
9B	
10A	
10B	
12	
14	
15A	
15B	
16A	
16B	
17A	
17B	
18A	
18B	

There are a total of 23 shear walls in the Hotel N.E.U.S. They are designated by the column line they run along and a letter. The letter is used to distinguish between those along the same column lines. Walls labeled with "A" are at the top of plan view and work their way towards the bottom (or left to right).

The walls are all 52' feet high and 8" thick with #5 bars at 24" O.C. Cells with reinforcement are grouted solid. Therefore the difference in capacity for all walls is based on the length.

Wind Analysis

Using ASCE 7-05, the wind loads for the Hotel N.E.U.S. were evaluated. It was determined that the overturning moment in the North-South direction was four times greater than the East-West direction. This is a result of the large difference in surface area from side to side. Appendix B shows all the factors and coefficients used in the calculations. The velocity pressures along with the pressures and forces calculated for design are listed as well.

The wind loads for the Main Wind Force Resisting System were calculated by the analytical procedure outlined in chapter 6 of ASCE 7-05. The building was simplified into a rectangle that was 258' x 61'. The tallest parapet height of 60'-8" was assumed to encompass the entire perimeter. Although the footprint of the building sits at an angle, the North-South direction is associated with the longer face of the building while East-West is the short sides.

Hotel N.E.U.S. was determined to be an occupancy category II with an importance factor of 1. The exposure category was C and the topographic factor was 1 as well. Since this the Hotel is a rigid building (which was determined by having a period 1< in the seismic section), the gust factor was calculated for each direction. The values acquired were 0.8386 and 0.872 for NS and EW respectively. To be conservative, a factor of 0.85 was used for the continuation of the analysis.

The parapet pressures were designed in accordance with 6.5.11.5, where a factor of 1.5 is used for windward parapets and -1.0 for leeward parapets. The force associated with these pressures should be used in the design of the MWRFS. However, components and cladding wind loads should be used in the design of the parapet itself.

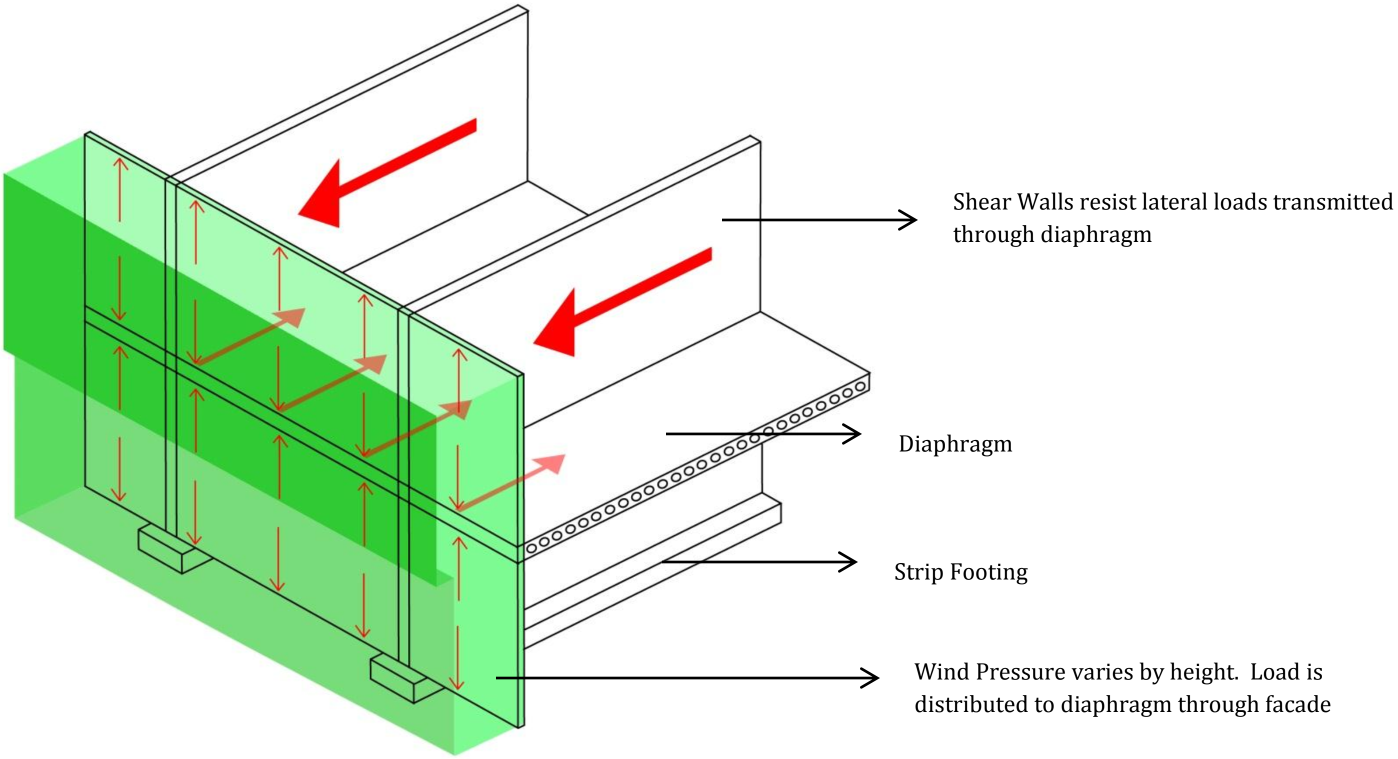


Figure 18: Load Path for Wind Loads

Wind Pressures N-S								
Location	Level	Distance (ft)	Velocity Pressure (psf)	External Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
			q_p / q_z / q_h	p_p / p_z / p_h (psf)	Positive (GCp)	Negative (GCp)	Positive	Negative
Windward		60.67	19.96	29.95	1.5		29.95	
	Parapet	52	19.35	13.16	2.70	-2.70	15.86	10.46
	5	42	18.51	12.58	2.70	-2.70	15.28	9.89
	4	32	17.48	11.89	2.70	-2.70	14.59	9.19
	3	22	16.15	10.98	2.70	-2.70	13.68	8.28
	2	12	14.98	10.19	2.70	-2.70	12.88	7.49
	Ground	0	14.98	10.19	2.70	-2.70	12.88	7.49
Leeward	Parapet	60.67	19.96	-19.96	-1.0		-19.96	
	G-4	52	14.98	-8.91	2.70	-2.70	-6.22	-11.61
Side	All	Total	14.98	-2.55	2.70	-2.70	0.15	-5.24
Roof	-	0-30.33	14.98	-11.46	2.70	-2.70	-8.76	-14.16
	-	30.33-60.67	14.98	-11.46	2.70	-2.70	-8.76	-14.16
	-	60.67-121.33	14.98	-6.37	2.70	-2.70	-3.67	-9.06
	-	>121.33	14.98	-3.82	2.70	-2.70	-1.12	-6.52

Figure 20: Wind Pressures N-S (Y direction)

Wind Pressures N-S						
Level	Elevation (ft)	Tributary Area (ft^2)		Wind Force (k)	Story Shear (k)	Overturning Moment (ft-k)
		Above	Below			
	60.67	0	1118	55.82	55.82	3386.64
Parapet	52	1118	1290	84.29	140.12	4383.34
5	42	1290	1290	56.21	196.32	2360.67
4	32	1290	1290	54.57	250.89	1746.16
3	22	1290	1290	52.50	303.39	1154.91
2	12	1290	1548	55.23	358.61	662.74
Ground	0	1548	0	0.00	358.61	0.00
						13694.46

Figure 19: Story Shears N-S (Y direction)

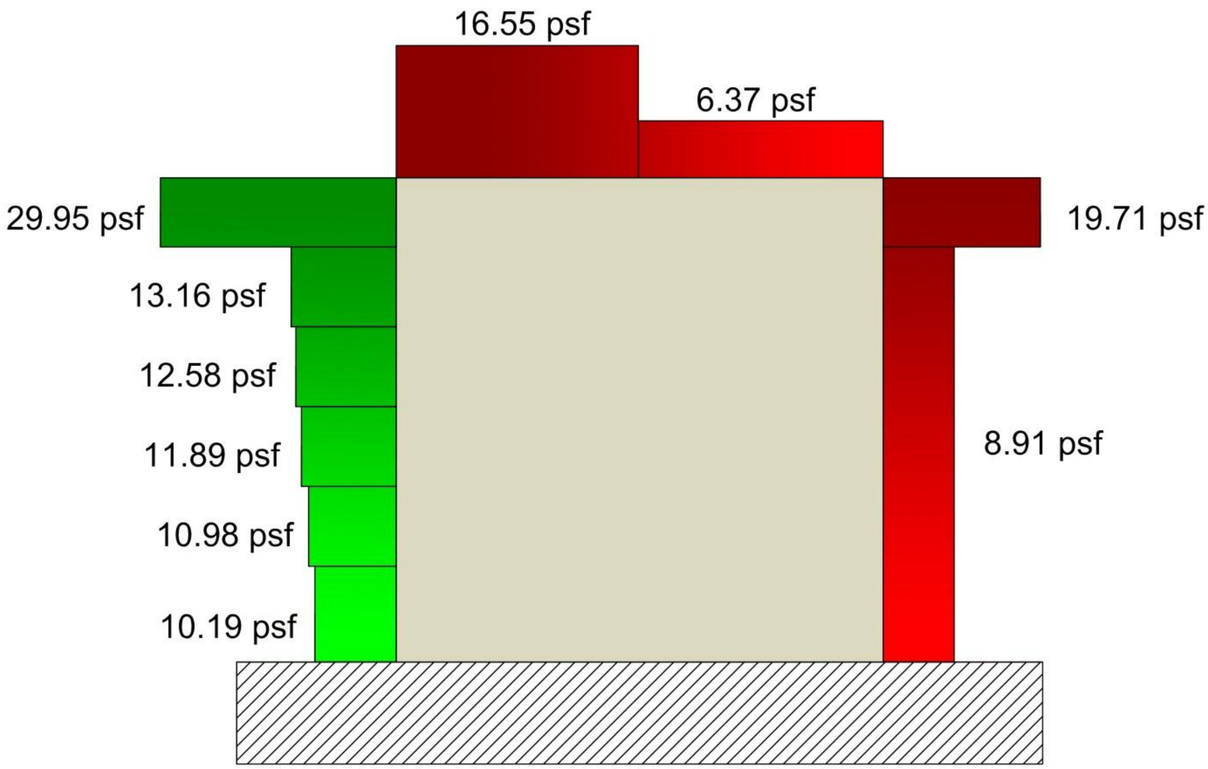


Figure 21: Wind Pressures N-S (Y direction)

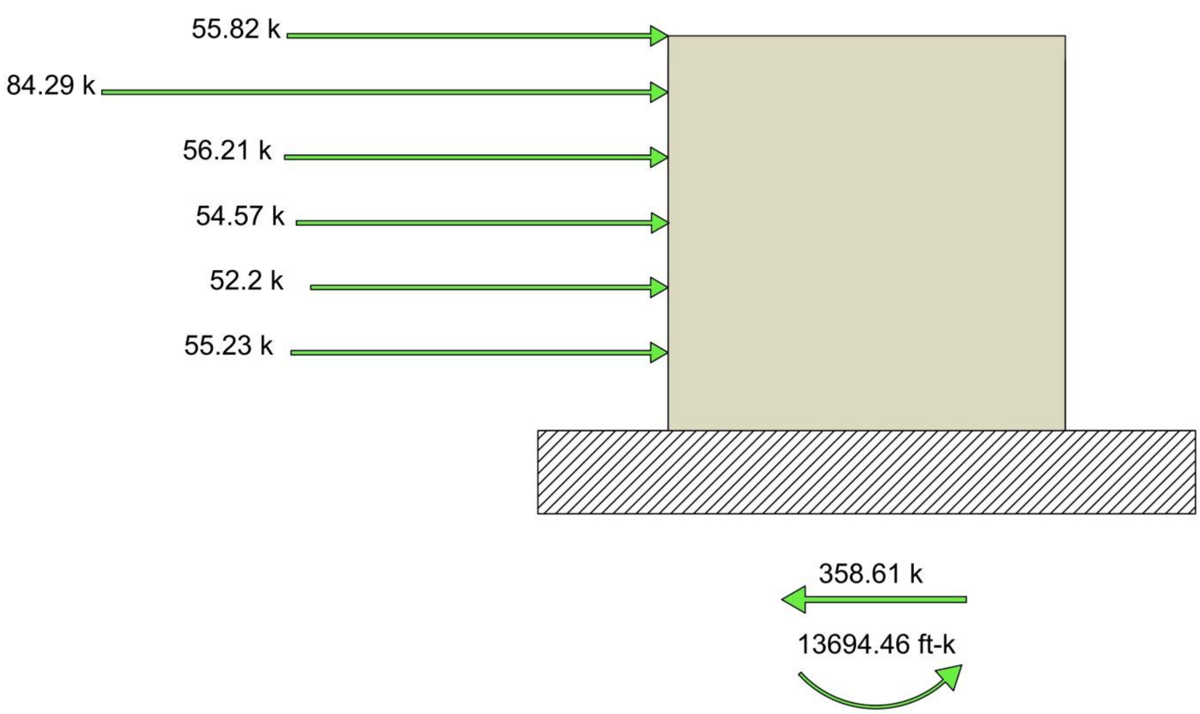


Figure 22: Wind Forces N-S (Y direction)

Wind Pressures E-W								
Location	Level	Distance (ft)	Velocity Pressure (psf)	External Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
			q_p / q_z / q_h	p_p / p_z / p_h (psf)	Positive (GCp)	Negative (GCp)	Positive	Negative
Windward		60.67	19.96	29.95	1.50		29.95	
	Parapet	52	19.35	13.16	2.70	-2.70	15.86	10.46
	5	42	18.51	12.58	2.70	-2.70	15.28	9.89
	4	32	17.48	11.89	2.70	-2.70	14.59	9.19
	3	22	16.15	10.98	2.70	-2.70	13.68	8.28
	2	12	14.98	10.19	2.70	-2.70	12.88	7.49
	Ground	0	14.98	10.19	2.70	-2.70	12.88	7.49
Leeward	Parapet	60.67	19.96	-19.96	-1.0		-19.96	
	G-4	52	14.98	-8.91	2.70	-2.70	-6.22	-11.61
Side	All	Total	14.98	-6.37	2.70	-2.70	-3.67	-9.06
Roof	-	0-28.5	14.98	-16.55	2.70	-2.70	-13.86	-19.25
	-	>h/2	14.98	-7.13	2.70	-2.70	-4.43	-9.83

Figure 23: Wind Pressures E-W (X direction)

Wind Pressures E-W						
Level	Elevation (ft)	Tributary Area (ft^2)		Wind Force (k)	Story Shear (k)	Overturning Moment (ft-k)
		Above	Below			
	60.67	0	264	13.20	13.20	800.72
Parapet	52	264	305	19.93	33.13	1036.37
5	42	305	305	13.29	46.42	558.14
4	32	305	305	12.90	59.32	412.85
3	22	305	305	12.41	71.73	273.06
2	12	305	366	13.06	84.79	156.69
Ground	0	366	0	0.00	84.79	0.00
						3237.84

Figure 24: Story Shears E-W (X direction)

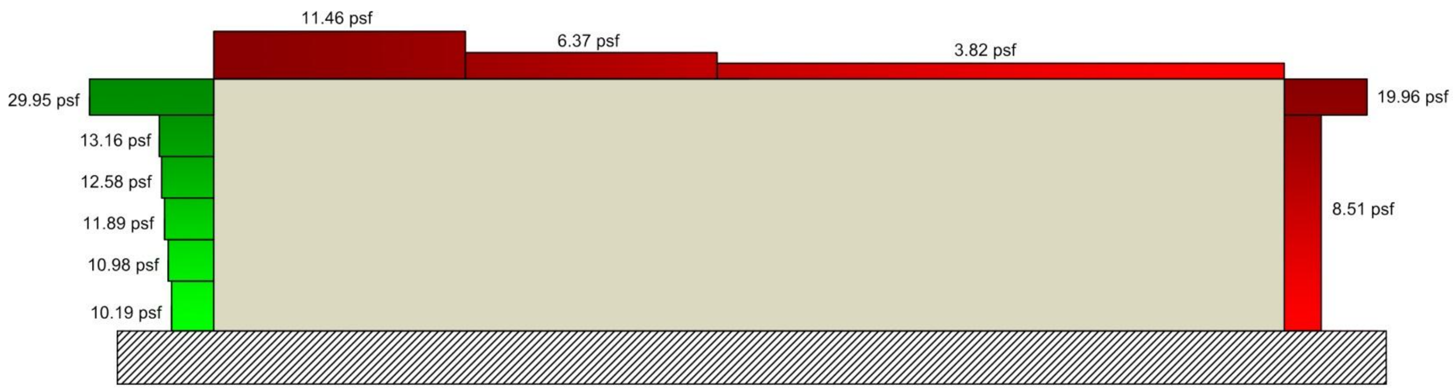


Figure 25: Wind Pressures E-W (X direction)

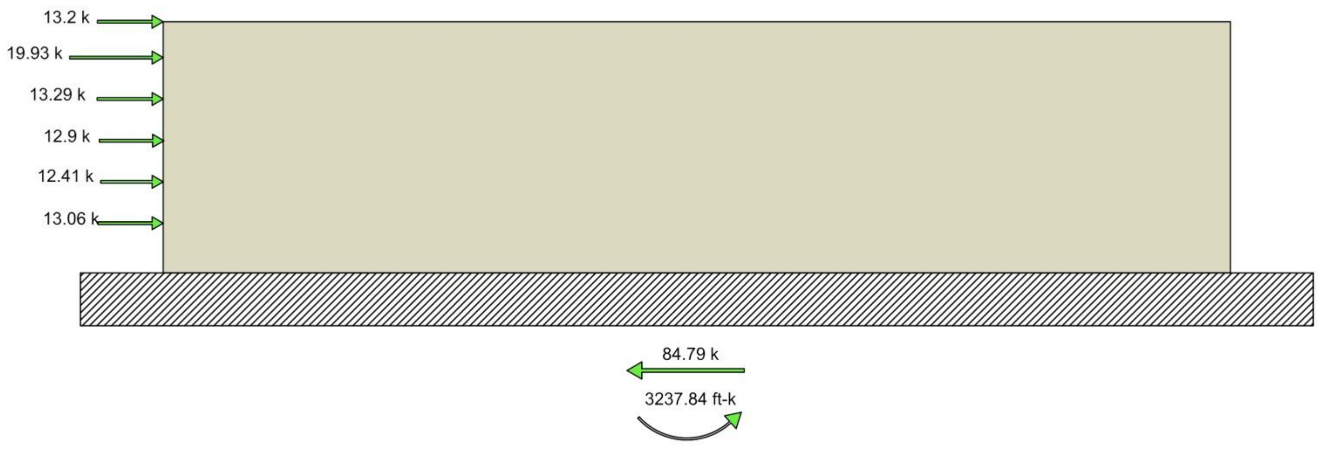


Figure 26: Wind Forces E-W (X direction)

Seismic Analysis

The Equivalent Lateral Force procedure outlined in ASCE 7-05 is used to calculate the seismic loads. The fundamental frequency was calculated for both the general equation (12.8-7) and for masonry shear walls (12.8-9). A Response Modification Coefficient of 2 was used for this system and is designated as such in the general notes. The Hotel N.E.U.S. fits into the "Other Structures" category for the general equation of frequency. The values for the N-S and E-W direction by equation 12.8-9 are much less and can be seen in Appendix C. This could likely be due to the estimates in the length of each shear wall and base area. Therefore, the general equation was used for both directions in this analysis. As stated in the wind analysis, this structure has a fundamental period that is less than one, classifying it as rigid.

The engineer of record used a coefficient of 0.67 which is from equation 12.8-2. However, by equation 12.8-3, when T is less than $T_{L,}$ the value of C_s has a maximum capped by the fundamental period and SD1 value. A value of 0.06 was found as the allowed maximum for the building and is used with the weight calculated to obtain base shear (see Appendix C). A base shear of 637 kips was about 56 kips off of the engineer of record's value on sheet S001. A 10% difference in values shows that the factors and weights used in this analysis were fairly accurate for a hand calculated base shear. The design engineer used RAM Structural to obtain these values while it is also more accurate in determining the seismic weight. The overturning moment is 25,440 foot kips and is much larger than the overturning moment due to wind. Wind generally controls in this region of the United States, but being constructed of masonry this building is heavy. The weight combined with a low R value results in a larger seismic base shear.

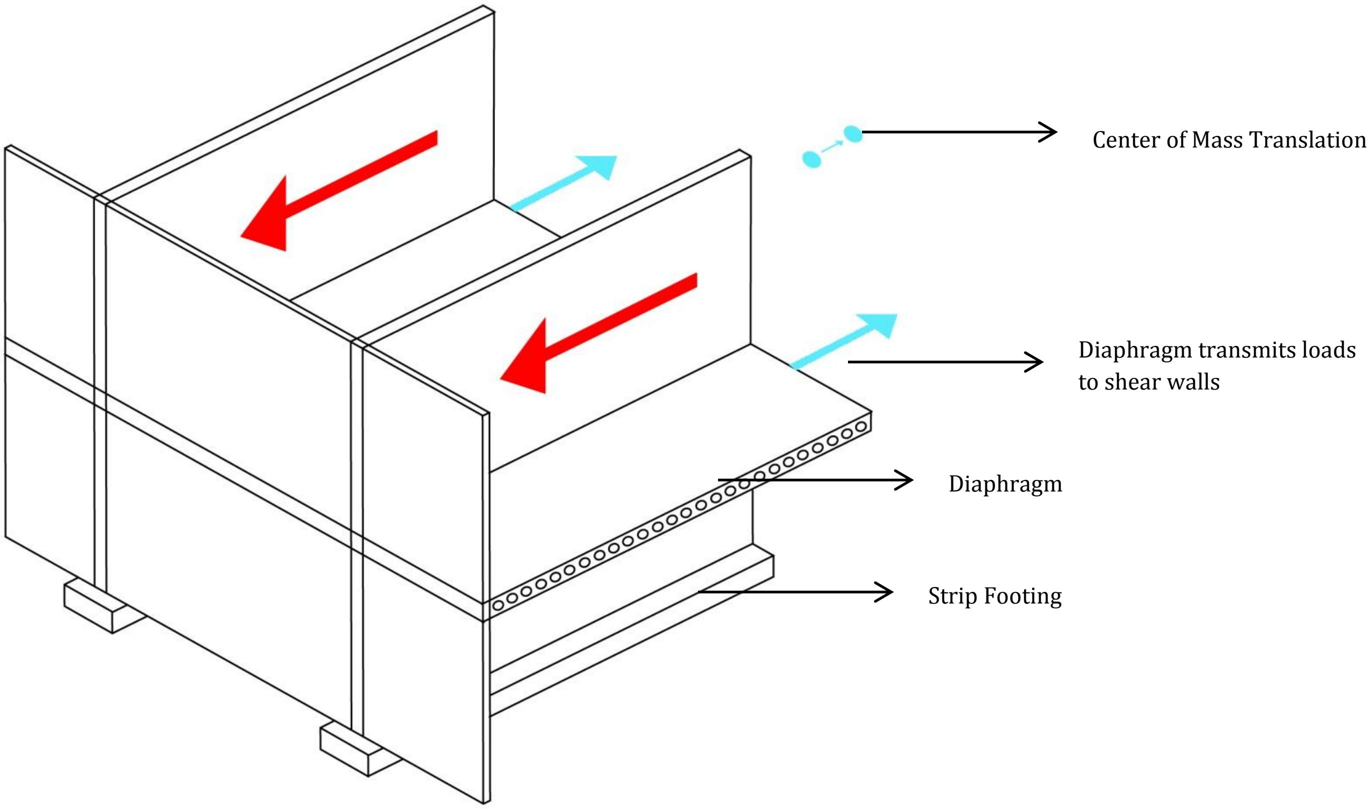


Figure 27: Seismic Load Path

Vertical Force Distribution (y)								
Level	Weight (k)	Height (ft)	k	$w_x h_x^k$	Distribution Factor	Story Force (k)	Story Shear (k)	Overturning Moment (ft-k)
	w_x	h_x			C_{vx}	$F_x=C_{vx}V$		
5	2534.45	52	1	131791.40	0.34	217.68	217.68	11319.31
4	2591.93	42	1	108861.06	0.28	179.81	397.48	7551.82
3	2592.97	32	1	82975.04	0.22	137.05	534.53	4385.58
2	2626.55	22	1	57784.10	0.15	95.44	629.98	2099.72
Ground	352.13	12	1	4225.61	0.01	6.98	636.95	83.75
				385637.21	1.00			25440.18

Figure 28: Seismic Story Shear

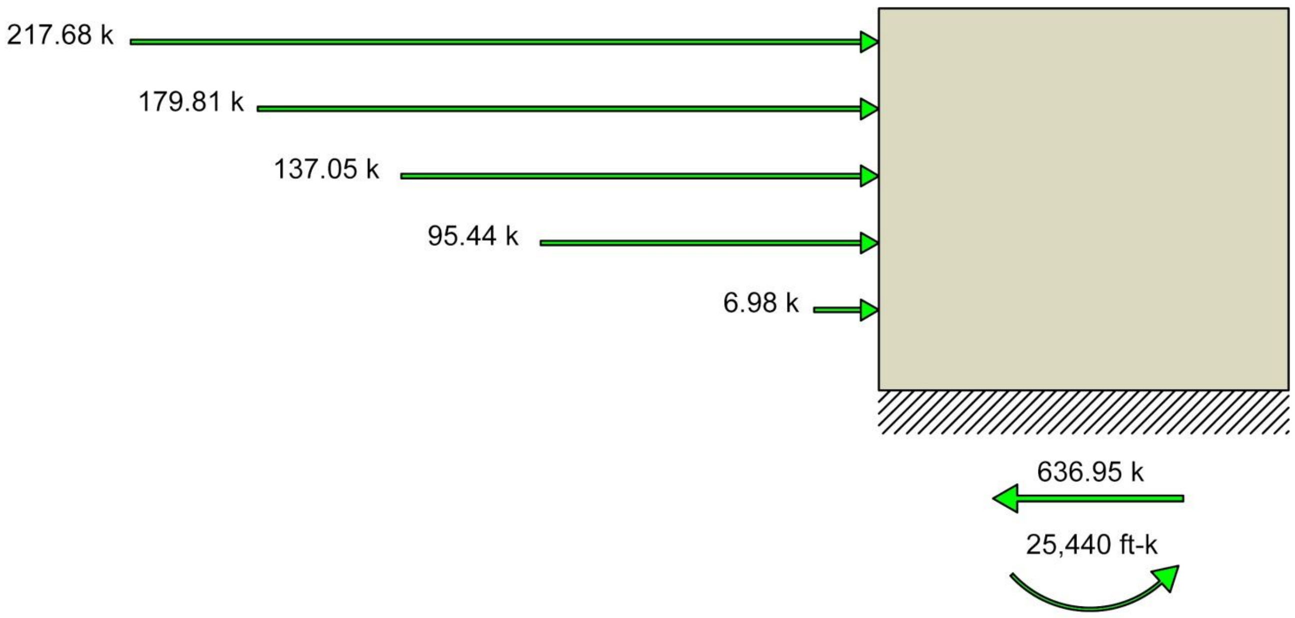


Figure 29: Story Forces for Seismic

Wall Stiffness and Center of Rigidity

A masonry shear wall is treated as a cantilever out of the ground. The following equation was used to calculate the stiffness of a wall:

$$k_{cant} = \frac{Et}{\left(\frac{h}{L}\right)\left[4\left(\frac{h}{L}\right)^2 + 3\right]}$$

Since all the shear walls in the Hotel N.E.U.S. are the same height, thickness, and have the same modulus of elasticity, the stiffness can be directly related to the length of each wall. Using the stiffness for each wall, the center of rigidity was calculated using Microsoft Excel. The Center of Rigidity is the location where a horizontal load can be applied and produce no torsion. It is the point at which the building will rotate about as well. The Hotel N.E.U.S. has an unsymmetrical shear wall layout, with more walls located on the right side plan view. Large open areas such as the pool and breakfast area prevent continuous walls in certain areas. Also, there is only one line of resistance in the X direction, meaning each wall takes approximately one third of the loads in that direction. This is an area of concern and is assessed later in this report.

In Figure 31 and 34 the stiffness per wall is calculated for each level along with the center of rigidity. The percent relative stiffness for each wall can be viewed in Figure 32 and 34 while comparison between the values obtained by hand, ETABS, and RAM is shown in Figure 30. The computer modeling programs use the diaphragm's contribution to stiffness leading to a difference between the hand calculated values. However, for the information provided it was deemed that the calculations performed were fairly close to those from the computer programs.

Center of Rigidity Comparison						
Level	X			Y		
	HAND	ETABS	RAM	HAND	ETABS	RAM
5	135.36	154.352	161.57	0.00	0.002	-5.05
4	135.36	154.025	160.71	0.00	0.034	-4.5
3	135.36	153.471	159.13	0.00	0.033	-3.52
2	135.36	152.347	156.28	0.00	0.028	-1.72
1	135.50	148.806	151.16	0.00	0.016	1.37
*For the Y direction, 0 is equal to 34.667' from Column Line C or the "bottom" of the building"						

Figure 30: Center of Rigidity Comparison

Center of Rigidity per Level (Y direction)							
E=	1800		Floor				
Wall	Length (in)	Distance to Wall (ft)	5 H=120"	4 H=120"	3 H=120"	2 H=120"	Ground H=144"
1A	238.5	0	7132.5	7132.5	7132.5	7132.5	5349.7
1B	238.5	0	7132.5	7132.5	7132.5	7132.5	5349.7
2A	238.5	9.563	7132.5	7132.5	7132.5	7132.5	5349.7
2B	238.5	9.563	7132.5	7132.5	7132.5	7132.5	5349.7
7A	92	89.897	1125.9	1125.9	1125.9	1125.9	718.8
7B	144	89.897	2990.8	2990.8	2990.8	2990.8	2057.1
7C	144	89.897	2990.8	2990.8	2990.8	2990.8	2057.1
7D	116	89.897	1911.9	1911.9	1911.9	1911.9	1265.8
9A	228	115.564	6660.1	6660.1	6660.1	6660.1	4961.3
9B	111	115.564	1735.5	1735.5	1735.5	1735.5	1140.6
10A	320	132.564	10778.9	10778.9	10778.9	10778.9	8399.0
10B	111	132.564	1735.5	1735.5	1735.5	1735.5	1140.6
14	320	185.898	10778.9	10778.9	10778.9	10778.9	8399.0
15A	284	200.898	9175.7	9175.7	9175.7	9175.7	7050.0
15B	284	200.898	9175.7	9175.7	9175.7	9175.7	7050.0
16A	238.5	223.898	7132.5	7132.5	7132.5	7132.5	5349.7
16B	238.5	223.898	7132.5	7132.5	7132.5	7132.5	5349.7
17A	238.5	246.898	7132.5	7132.5	7132.5	7132.5	5349.7
17B	238.5	246.898	7132.5	7132.5	7132.5	7132.5	5349.7
18A	238.5	256.461	7132.5	7132.5	7132.5	7132.5	5349.7
18B	238.5	256.461	7132.5	7132.5	7132.5	7132.5	5349.7
ΣR			130384.9	130384.9	130384.9	130384.9	97736.5
Center of Rigidity (ft)			135.4	135.4	135.4	135.4	135.5

Figure 31: Stiffness (k/in) and Center of Rigidity in the X direction

Percent Relative Stiffness per Floor					
	Floor				
Wall	5 H=120"	4 H=120"	3 H=120"	2 H=120"	Ground H=144"
1A	5.47	5.47	5.47	5.47	5.47
1B	5.47	5.47	5.47	5.47	5.47
2A	5.47	5.47	5.47	5.47	5.47
2B	5.47	5.47	5.47	5.47	5.47
7A	0.86	0.86	0.86	0.86	0.74
7B	2.29	2.29	2.29	2.29	2.10
7C	2.29	2.29	2.29	2.29	2.10
7D	1.47	1.47	1.47	1.47	1.30
9A	5.11	5.11	5.11	5.11	5.08
9B	1.33	1.33	1.33	1.33	1.17
10A	8.27	8.27	8.27	8.27	8.59
10B	1.33	1.33	1.33	1.33	1.17
14	8.27	8.27	8.27	8.27	8.59
15A	7.04	7.04	7.04	7.04	7.21
15B	7.04	7.04	7.04	7.04	7.21
16A	5.47	5.47	5.47	5.47	5.47
16B	5.47	5.47	5.47	5.47	5.47
17A	5.47	5.47	5.47	5.47	5.47
17B	5.47	5.47	5.47	5.47	5.47
18A	5.47	5.47	5.47	5.47	5.47
18B	5.47	5.47	5.47	5.47	5.47
ΣR%	100.0	100.0	100.0	100.0	100.0

Figure 32: Relative Stiffness per Floor

Center of Rigidity per Level (X-direction)							
E=	1800		Floor				
Wall	Length (ft)	Distance to Wall (ft)	5	4	3	2	Ground
			H=120"	H=120"	H=120"	H=120"	H=144"
F1	17.5	0	11.0	11.0	11.0	11.0	6.4
F2	18.83	0	13.7	13.7	13.7	13.7	7.9
F3	19.17	0	14.4	14.4	14.4	14.4	8.4
ΣR			39.0	39.0	39.0	39.0	22.7
Center of Rigidity			0	0	0	0	0

Figure 33: Stiffness (k/in) and Center of Rigidity in the Y direction
NOTE: Datum line is Column Line F (34'-8" from front face)

Percent Relative Stiffness per Floor					
	Floor				
Wall	5	4	3	2	Ground
	H=120"	H=120"	H=120"	H=120"	H=144"
F1	28.14	28.14	28.14	28.14	28.13
F2	34.98	34.98	34.98	34.98	34.98
F3	36.88	36.88	36.88	36.88	36.89
ΣR%	100.0	100.0	100.0	100.0	100.0

Figure 34: Relative Stiffness per Floor

Center of Mass

The Center of Mass is where the Seismic Loads will act in the diaphragm. The values for each level were very close to the center of the building which was expected due to the symmetric layout of the floors. The floor areas were broken up by bay sections. Results by hand came in very close to those of ETABS and RAM and can be seen in Figure 35. The calculations for the weight, mass, and Center of Mass for each floor can be found in Appendix E.

Center of Mass Comparison						
Level	X			Y		
	HAND	ETABS	RAM	HAND	ETABS	RAM
5	126.53	126.931	125.37	-4.24	-3.995	0.36
4	126.68	126.931	126.03	-4.22	-3.995	0.36
3	126.68	126.931	126.03	-4.22	-3.995	0.36
2	126.68	126.931	125.93	-4.22	-3.995	0.42
1	123.65	126.5	125.99	-2.79	-4.016	0.31
*For the Y direction, 0 is equal to 34.667' from Column Line C or the "bottom" of the building"						

Figure 35: Center of Mass Comparison

Computer Modeling for Lateral Analysis

Two computer models were built to understand the behavior of the Hotel N.E.U.S. when subjected to lateral loads. The programs used were ETABS and RAM Frame. The assumptions, simplifications, and process are outlined in the following sections.

ETABS Model

The overlying assumption in this model is that the lateral loads will be carried only by shear walls that run continuously to the foundation. Therefore only these walls are modeled. This is a conservative approach and forces will be resisted partially by other elements in the real building. Membrane elements were defined with an 8" membrane thickness and 0.8" for bending thickness, which is 10% of the total. This prevents warnings or huge deformations while preventing out of plane forces to be carried by the walls. These elements were meshed into a maximum size of 24" for accurate results. In Figures 37 and 38, the floor plans are shown. Walls that terminated in a connection to another wall were stopped 1' short to prevent the program from interpreting extra stiffness. These walls are not detailed to act as a group.

The mass was defined as zero for the masonry material property and the weight was calculated based off of 105 pcf masonry units from NCMA TEK 14-13B.

A rigid diaphragm was assigned to each floor. The precast plank has reinforcement grouted into the hollow sections and can transfer loads in a rigid manner (see Figure 37). An additional area mass was assigned and the calculations can be found in Appendix E. The vertical circulation shafts were not taken into account in this model.

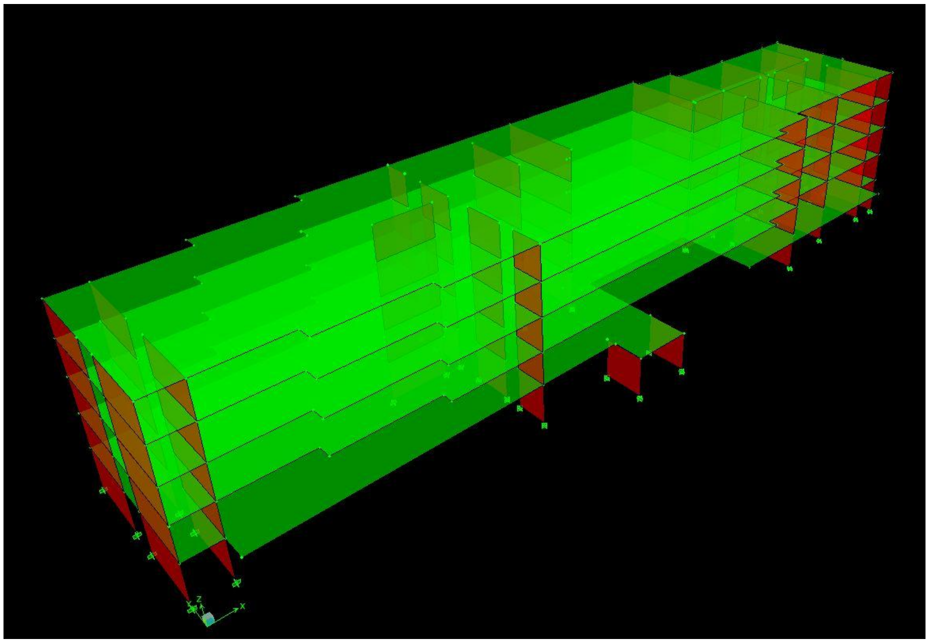


Figure 36: 3D view of ETABS model

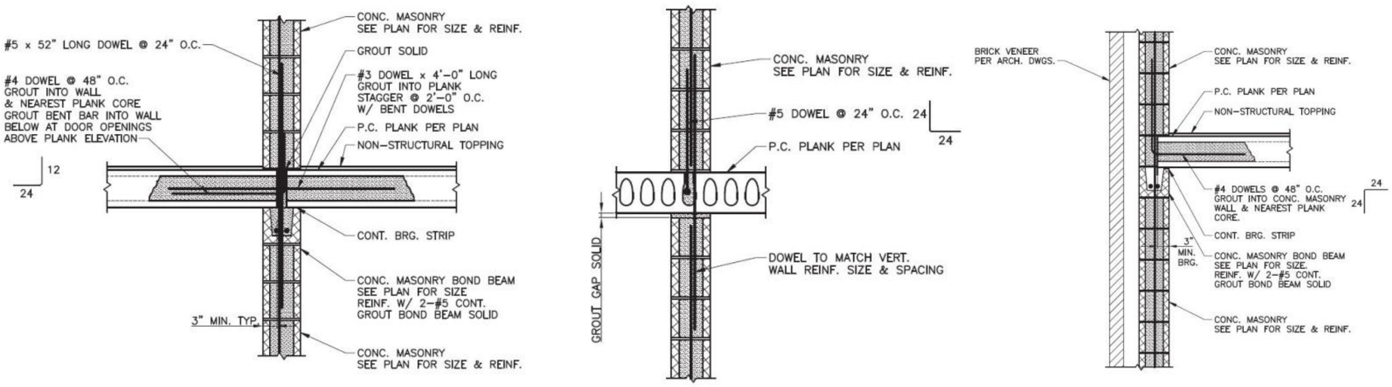


Figure 37: Details to justify rigid diaphragm

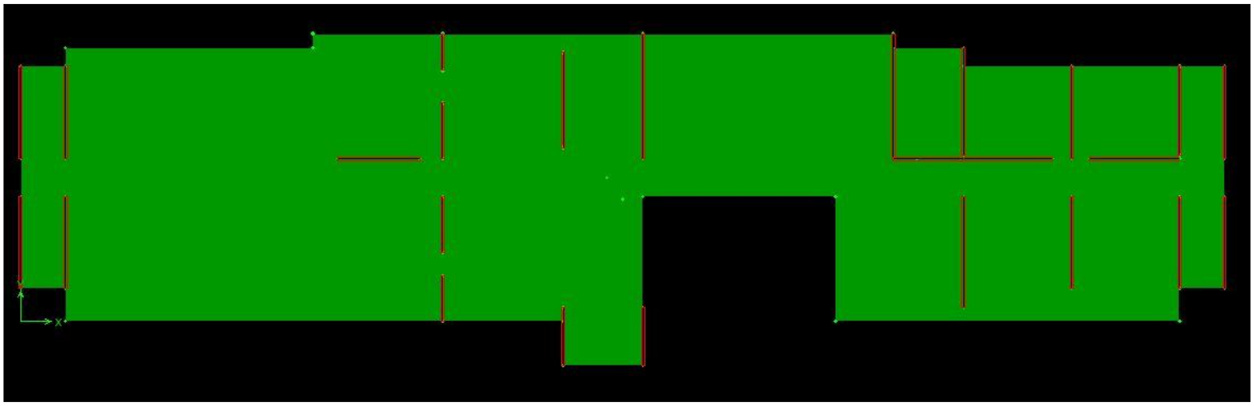


Figure 38: Second Floor Plan

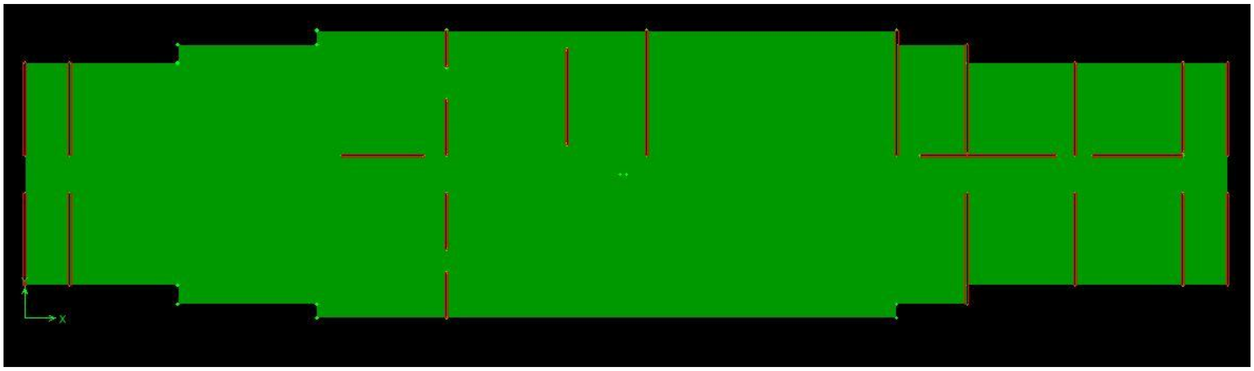


Figure 39: Third-Fifth Floor Plan

RAM Model

A RAM model was constructed to compare results with ETABS and hand analysis. Due to the ability of inputting members as gravity or lateral elements, the whole building system (excluding the foundations) was modeled. A concrete floor was designated as 8" thick and a rigid diaphragm. The loads were simplified to be the same across the entire floor and the exterior wall loads were ignored. As in the ETABS model, vertical circulation is ignored in the floor diaphragm. Walls were meshed at a maximum of 24" as well. RAM calculates lateral loads based off of ASCE 7-05 and produced slightly lower loads than those obtained by hand. This is likely due to the lack of parapet in the model and slightly more accurate weights obtained by hand methods. Figure 41 shows a 3D picture of the model while the floor plans are shown in Figures 42,43, and 44.

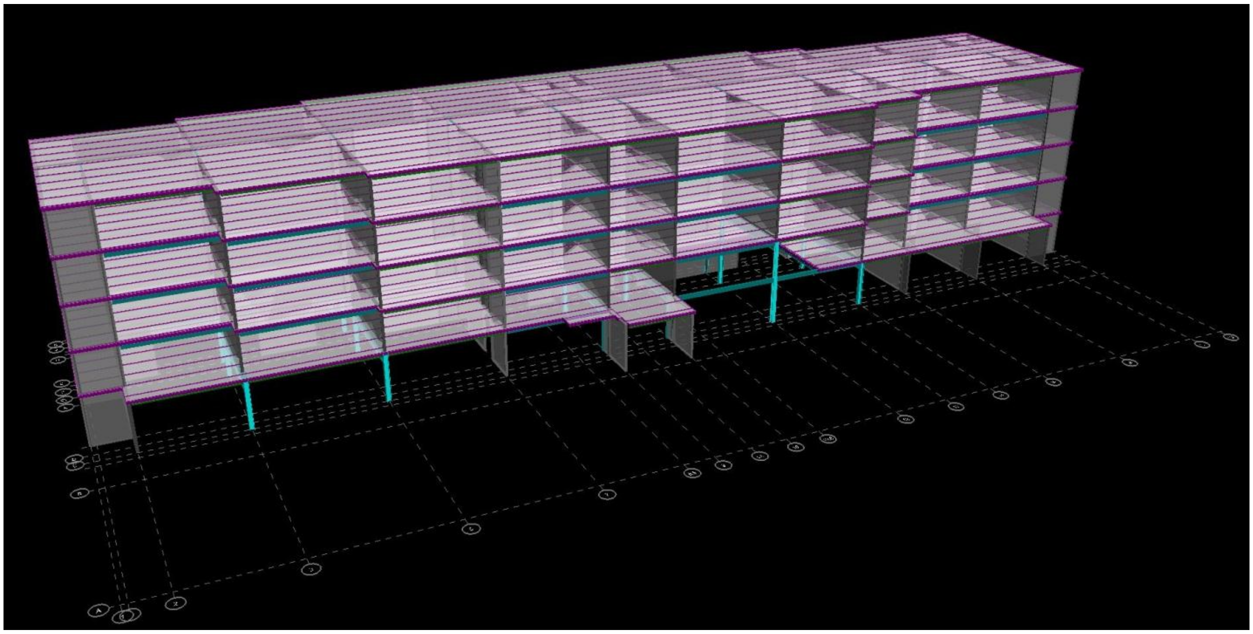


Figure 40: 3D RAM Model

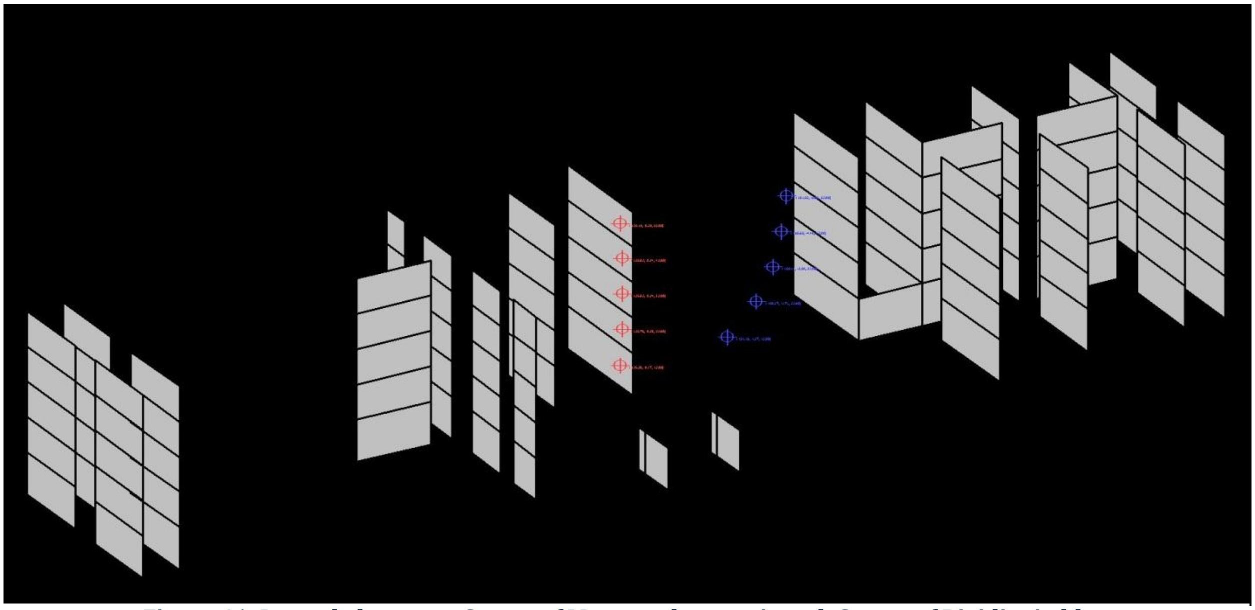


Figure 41: Lateral elements. Center of Mass can be seen in red, Center of Rigidity in blue

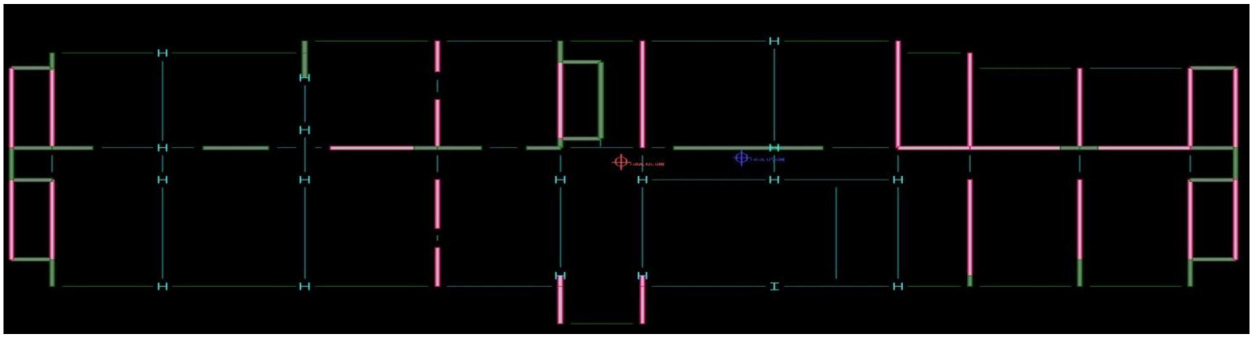


Figure 42: Level 1 floor plan

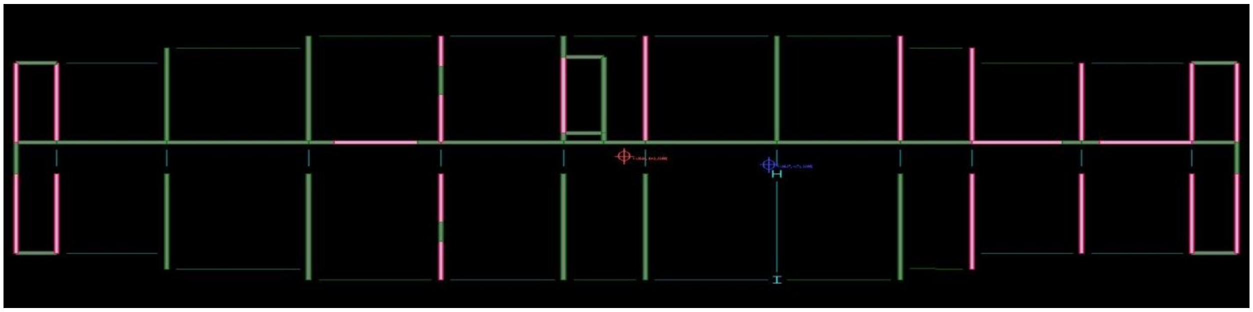


Figure 44: Level 2-4

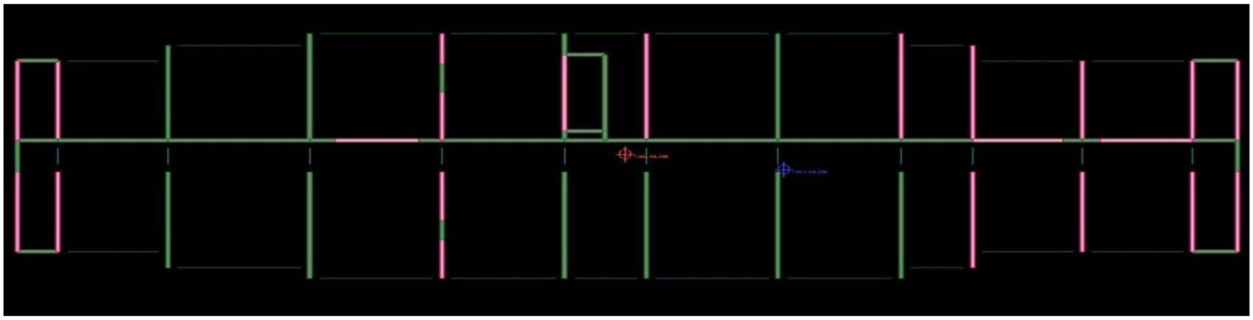


Figure 43: Level 5 floor plan

Load Cases

ASCE 7-05 provides basic load combinations for gravity and lateral loads. The Allowable Stress Design combinations were used so that ACI 530-05 shear strength checks could be used. They are as follows:

$$D + W$$

$$D + 0.7E$$

$$0.6D + W$$

$$0.6D + 0.7E$$

Since wind loads act at the Center of Pressure but the building rotates about the Center of Rigidity, lateral forces cause torsion. There are four wind cases that must be considered and can be found in the *Torsion* section of this report. A spreadsheet was developed to calculate and organize all the combinations for wind and seismic forces. These were arranged for ETABS and were confirmed with RAM containing 64 load combinations. Load cases involved in these combinations can be seen in Appendix F.

LOAD COMBO 1									
CASE 1	COMB1	1	D	+	1	XW1			
	COMB2	1	D	+	1	YW1			
CASE 2	COMB3	1	D	+	1	PXW2	+	1.6	1PXM2
	COMB4	1	D	+	1	PXW2	+	1.6	1NXM2
	COMB5	1	D	+	1	NXW2	+	1.6	2PXM2
	COMB6	1	D	+	1	NXW2	+	1.6	2NXM2
	COMB7	1	D	+	1	PYW2	+	1.6	1PYM2
	COMB8	1	D	+	1	PYW2	+	1.6	1NYM2
	COMB9	1	D	+	1	NYW2	+	1.6	2PYM2
	COMB10	1	D	+	1	NYW2	+	1.6	2NYM2
CASE 3	COMB11	1	D	+	1	PPXYW3			
	COMB12	1	D	+	1	PNXYW3			
	COMB13	1	D	+	1	NPXYW3			
	COMB14	1	D	+	1	NNXYW3			
CASE 4	COMB15	1	D	+	1	PPXYW4	+	1.6	1PPXYM4
	COMB16	1	D	+	1		+	1.6	1PNXYM4
	COMB17	1	D	+	1		+	1.6	1NPXYM4
	COMB18	1	D	+	1		+	1.6	1NNXYM4
	COMB19	1	D	+	1	PNXYW4	+	1.6	2PPXYM4
	COMB20	1	D	+	1		+	1.6	2PNXYM4
	COMB21	1	D	+	1		+	1.6	2NPXYM4
	COMB22	1	D	+	1		+	1.6	2NNXYM4
	COMB23	1	D	+	1	NPXYW4	+	1.6	3PPXYM4
	COMB24	1	D	+	1		+	1.6	3PNXYM4
	COMB25	1	D	+	1		+	1.6	3NPXYM4
	COMB26	1	D	+	1		+	1.6	3NNXYM4
	COMB27	1	D	+	1	NNXYW4	+	1.6	4PPXYM4
	COMB28	1	D	+	1		+	1.6	4PNXYM4
	COMB29	1	D	+	1		+	1.6	4NPXYM4
	COMB30	1	D	+	1		+	2.6	4NNXYM4

LOAD COMBO 3									
CASE 1	COMB31	0.6	D	+	1	XW1			
	COMB32	0.6	D	+	1	YW1			
CASE 2	COMB33	0.6	D	+	1	XW2	+	1.6	1PXM2
	COMB34	0.6	D	+	1	XW2	+	1.6	1NXM2
	COMB35	0.6	D	-	1	XW2	+	1.6	2PXM2
	COMB36	0.6	D	-	1	XW2	+	1.6	2NXM2
	COMB37	0.6	D	+	1	YW2	+	1.6	1PYM2
	COMB38	0.6	D	+	1	YW2	+	1.6	1NYM2
	COMB39	0.6	D	+	1	YW2	+	1.6	2PYM2
	COMB40	0.6	D	+	1	YW2	+	1.6	2NYM2
CASE 3	COMB41	0.6	D	+	1	PPXYW3			
	COMB42	0.6	D	+	1	PNXYW3			
	COMB43	0.6	D	+	1	NPXYW3			
	COMB44	0.6	D	+	1	NNXYW3			
CASE 4	COMB45	0.6	D	+	1	PPXYW4	+	1.6	1PPXYM4
	COMB46	0.6	D	+	1		+	1.6	1PNXYM4
	COMB47	0.6	D	+	1		+	1.6	1NPXYM4
	COMB48	0.6	D	+	1		+	1.6	1NNXYM4
	COMB49	0.6	D	+	1	PNXYW4	+	1.6	2PPXYM4
	COMB50	0.6	D	+	1		+	1.6	2PNXYM4
	COMB51	0.6	D	+	1		+	1.6	2NPXYM4
	COMB52	0.6	D	+	1		+	1.6	2NNXYM4
	COMB53	0.6	D	+	1	NPXYW4	+	1.6	3PPXYM4
	COMB54	0.6	D	+	1		+	1.6	3PNXYM4
	COMB55	0.6	D	+	1		+	1.6	3NPXYM4
	COMB56	0.6	D	+	1		+	1.6	3NNXYM4
	COMB57	0.6	D	+	1	NNXYW4	+	1.6	4PPXYM4
	COMB58	0.6	D	+	1		+	1.6	4PNXYM4
	COMB59	0.6	D	+	1		+	1.6	4NPXYM4
	COMB60	0.6	D	+	1		+	2.6	4NNXYM4
SEISMIC COMBOS									
	COMB61	1	D	+	0.7	XQUAKE			
	COMB62	1	D	+	0.7	YQUAKE			
	COMB63	0.6	D	+	0.7	XQUAKE			
	COMB64	0.6	D	+	0.7	YQUAKE			

Figure 45: Load Combinations

Modal Comparison

The first 3 modes of the Hotel N.E.U.S. from both ETABS and RAM show similar results. By ASCE 7-05, the period was calculated to be 0.658 seconds. Both models produced values below this period. This means that the shears will be larger than those calculated using the general equation and period.

The first mode for both models was the translation of the building in the X direction. This was anticipated due to the much lower number of walls and stiffness.

Mode	Period	
	ETABS	RAM
1	0.5434	0.6045
2	0.3048	0.3325
3	0.2052	0.212

Figure 46: Period in Seconds

Maximum Shear

The maximum shear for each line of action was obtained and the load combination was recorded. The shear values are for the first floor since the maximum value in the first floor means there will be max values on every level in that wall. The combinations with earthquake loads controlled every group.

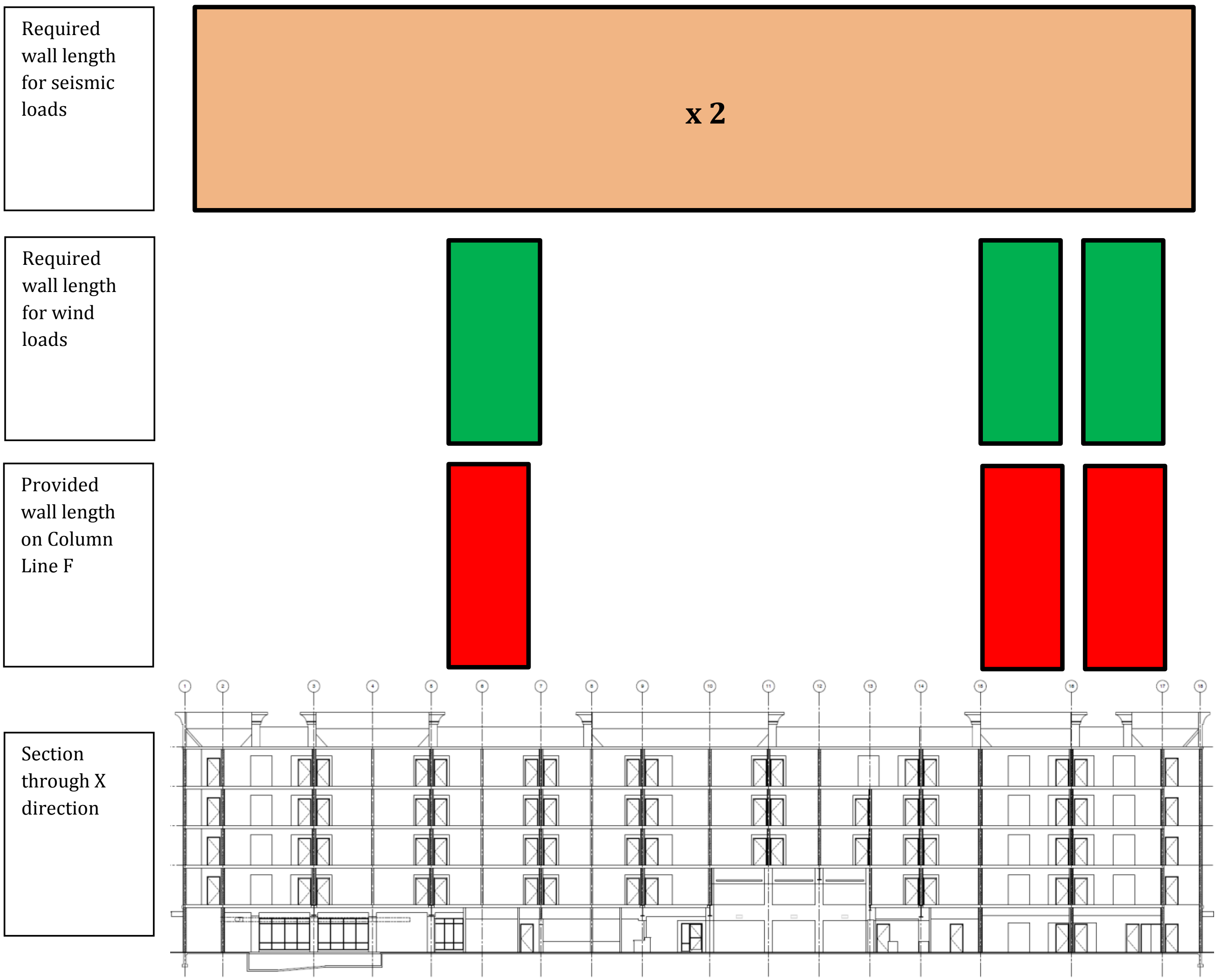
Shear Wall Line	Max Force (kips)	Combo
1	72.48	1.0D-0.7E
2	73.54	0.6D+0.7E
7	30.77	1.0D+0.7E
9	23.84	1.0D+0.7E
10	52.76	1.0D+0.7E
14	45.35	1.0D+0.7E
15	65.26	1.0D-0.7E
16	38.5	1.0D+0.7E
17	41.13	1.0D-0.7E
18	36.57	1.0D+0.7E
F1	42.12	1.0D+0.7E
F2	314.98	1.0D+0.7E
F3	82.8	1.0D-0.7E

Figure 47: Max Wall Group shears and combos

Critical Case

The shear forces that ETABS and RAM consider are a combination of direct shear and torsional shear. However, the Hotel N.E.U.S. only contains 3 shear walls along Column Line F in the X direction. Due to this, there is no ability for the building to resist torsion and is an area of concern. Therefore the shear capacity of shear walls F1,2, and 3 were calculated to compare to the applied direct forces.

It was found that the wall was over capacity with its current layout. The forces used were hand calculated, as the RAM model does not include the parapet height. Also the exterior wall weight was not included therefore the overall seismic loads were about 20 kips less. The required length of wall to resist the wind loads was only 1.5' longer. There could be aspects of the building that the design engineer assumed or knew would take loads in the X direction that would make this layout barely suitable for direct forces. For seismic loads, the required length of wall was found to be 428'. This is almost an 8 times longer than the current walls. It is likely the design engineer did not use seismic loads as the controlling lateral case. Refer to Appendix D for calculations.



Relative Stiffness Comparison

The shear per wall was assembled for the computer model output and the relative stiffness was computed for the direct wind forces at the first floor. These values were compared to the relative stiffness values obtained by hand. Values were slightly different between ETABS, RAM, and hand, with more shear being associated with the left side shear walls for Y direction forces.

In the X direction, walls F2 and F3 took much more force than was anticipated. Wall F2 has a substantially increased length on the first floor which produced these results. Both programs devoted the most shear to wall 14 in the Y direction.

Direct W	Shear		% Rel. Stiffness		
Frame	RAM	ETABS	RAM	ETABS	HAND
1A	16.81	25.19	7.84	7.76	5.47
1B	16.81	25.09	7.84	7.73	5.47
2A	16.49	24.69	7.69	7.61	5.47
2B	16.49	24.59	7.69	7.58	5.47
7AB	8.46	11.57	3.95	3.56	2.84
7CD	7.14	10.75	3.33	3.31	3.40
9A	11.61	20.14	5.42	6.20	5.08
10A	19.82	32.14	9.24	9.90	8.59
14	21.07	36.92	9.83	11.37	8.59
15A	12.94	20.33	6.04	6.26	7.21
15B	17.13	20.42	7.99	6.29	7.21
16A	8.6	13.50	4.01	4.16	5.47
16B	8.6	13.45	4.01	4.14	5.47
17A	7.75	11.58	3.61	3.57	5.47
17B	9.88	12.26	4.61	3.78	5.47
18A	7.4	10.21	3.45	3.15	5.47
18B	7.4	11.76	3.45	3.62	5.47
Total	214.4	324.59			

Direct W	Shear		% Rel. Stiffness		
Frame	RAM	ETABS	RAM	ETABS	HAND
F1	5.62	10.48	12.47	22.76	28.14
F2	30.29	23.03	67.19	50.01	34.98
F3	9.17	12.54	20.34	27.23	36.88
Total	45.08	46.05			

Figure 48: Relative Stiffness comparison

Drift and Displacement

The story displacement was checked for both the RAM and ETABS model. Being a serviceability issue, the loads used to determine these values are unfactored. Since the floor acts as a rigid diaphragm, the values are taken from the center of mass. The allowable displacement for wind loads is equal to L/400. Lateral story drifts were also determined. ASCE 7-05 states that the allowable seismic story drift is $0.010h_{sx}$ for occupancy category II. The values obtained through computer models found all values to be acceptable.

Wind Drift and Displacement					
Floor	Displacement		Drift		Allowable Displacement (in)
	X direction (in)	Y direction (in)	X direction (in)	Y direction (in)	
5	0.05930	0.07030	0.01580	0.01870	1.56
4	0.04350	0.05160	0.01440	0.01700	1.26
3	0.02910	0.03460	0.01300	0.01520	0.96
2	0.01610	0.01940	0.01020	0.01180	0.66
1	0.00590	0.00760	0.00590	0.00760	0.36

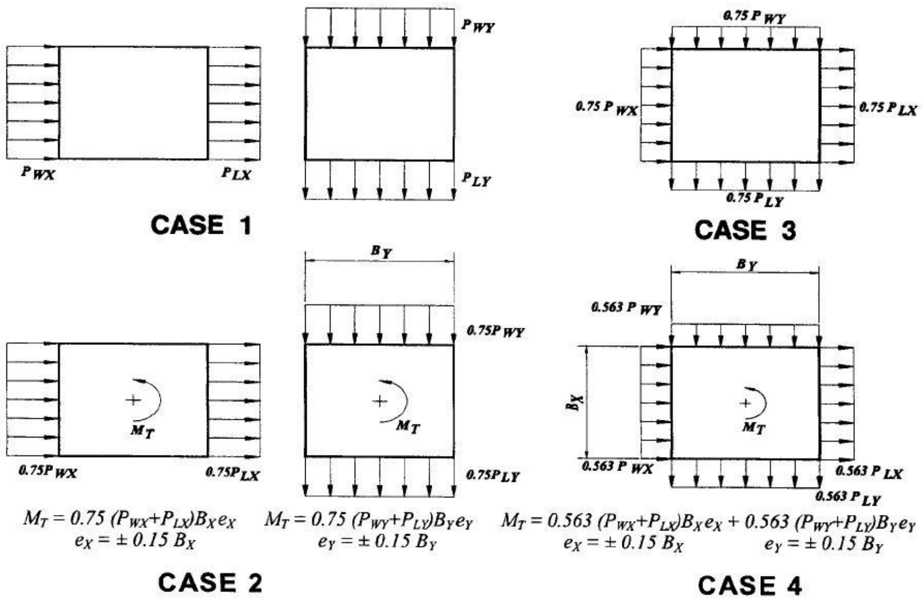
Figure 50: Drift and Displacements for Wind

Seismic Drift and Displacement					
Floor	Displacement		Drift		Allowable Drift (in)
	X direction (in)	Y direction (in)	X direction (in)	Y direction (in)	
5	0.48640	0.1310	0.12460	0.03380	1.20
4	0.36180	0.0972	0.11980	0.03190	1.20
3	0.24200	0.0653	0.10930	0.02890	1.20
2	0.13270	0.0364	0.08710	0.02290	1.20
1	0.04560	0.0135	0.04560	0.01350	1.44

Figure 49: Drift and Displacements for Seismic

Torsion

There are 4 torsional wind cases that are to be considered in ASCE 7-05. Figure 51 shows the cases along with the calculated values for the Hotel N.E.U.S. Due to the eccentricity already present in the building between the Center of Pressure and Center of Rigidity, the moment is much larger in certain cases.



Floor	Story Shear	Direction/ Length	Case 1		Case 2		Case 3		Case 4	
			Direct	Torsion	Direct	Torsion	Direct	Torsion	Direct	Torsion
5	33.13	X	33.13	0	24.85	131.60	24.85	0	18.65	98.79
				0		330.54		0		248.13
	140.12	Y	140.12	0	105.09	3257.69	105.09	0	78.89	2445.44
				0		4812.97		0		3612.94
4	13.29	X	13.29	0	9.97	52.79	9.97	0	7.48	39.63
				0		132.59		0		99.53
	56.21	Y	56.21	0	42.15	1306.80	42.15	0	31.64	980.97
				0		1930.69		0		1449.30
3	12.90	X	12.90	0	9.68	51.25	9.68	0	7.26	38.47
				0		128.73		0		96.63
	54.57	Y	54.57	0	40.93	1268.69	40.93	0	30.72	952.37
				0		1874.39		0		1407.05
2	12.41	X	12.41	0	9.31	49.30	9.31	0	6.99	37.01
				0		123.84		0		92.96
	52.50	Y	52.50	0	39.37	1220.53	39.37	0	29.56	916.21
				0		1803.23		0		1353.62
1	13.06	X	13.06	0	9.79	51.87	9.79	0	7.35	38.94
				0		130.29		0		97.80
	55.23	Y	55.23	0	41.42	1279.91	41.42	0	31.09	960.79
				0		1901.23		0		1427.19

Figure 51: Torsion for Wind Cases per ASCE 7-05

Seismic Loads on the building are applied with an eccentricity of 5% of the length, called accidental torsion. In Figure 52 the accidental torsion for each direction is calculated. However, both ETABS and RAM automatically calculate these values for the model. These values were calculated for investigative purposes. Due to the accidental torsion eccentricity being less than the natural eccentricity, the X direction has torsion in the same direction for the 5% offset (This is shown in the last column of the chart. If the direct shear is in the positive X direction, the moment will be negative when adding and subtracting 5% eccentricity from the center of mass).

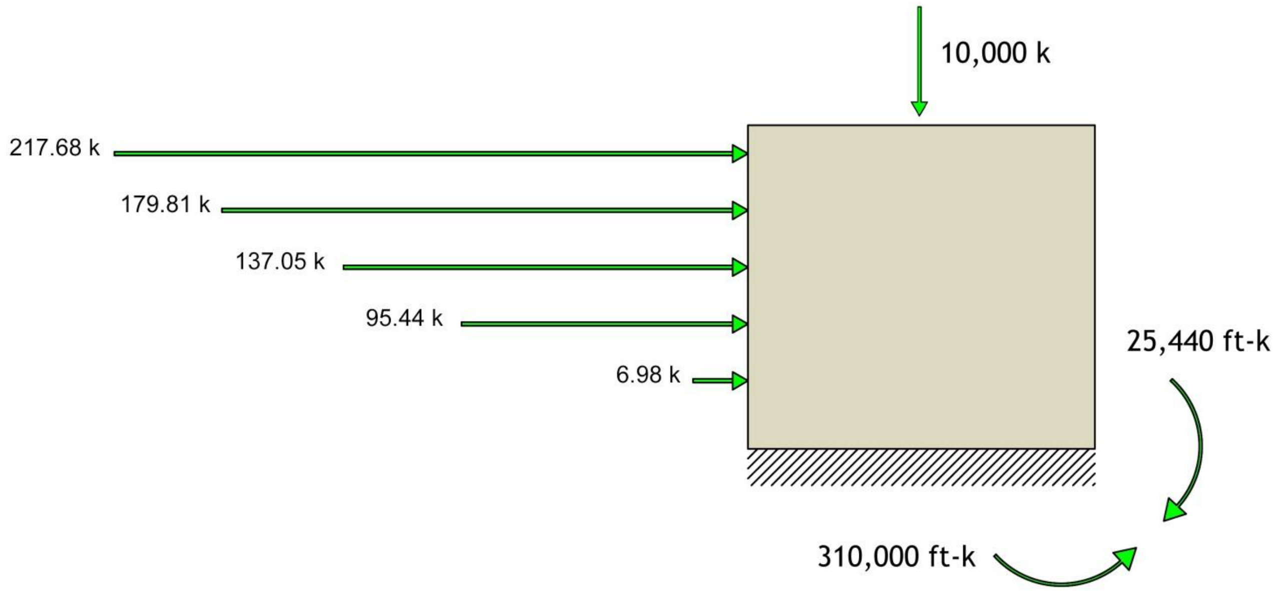
X Direction Torsion					
Floor	Story Force	e_x (ft)	M_t (ft-k)	Direct	
				POS	NEG
5	217.68	1.13	245.98	-	+
		7.33	1595.59	-	+
4	179.81	1.14	204.98	-	+
		7.34	1319.77	-	+
3	137.05	1.14	156.24	-	+
		7.34	1005.94	-	+
2	95.44	1.14	108.80	-	+
		7.34	700.54	-	+
1	6.98	1.14	7.96	-	+
		7.34	51.23	-	+

Y Direction Torsion					
Floor	Story Force	e_x (ft)	M_t (ft-k)	Direct	
				POS	NEG
5	217.68	4.05	881.60	-	+
		21.55	4690.98	+	-
4	179.81	4.05	728.21	-	+
		21.55	3874.80	+	-
3	137.05	4.05	555.05	-	+
		21.55	2953.41	+	-
2	95.44	4.05	386.54	-	+
		21.55	2056.77	+	-
1	6.98	0.90	6.28	-	+
		24.70	172.39	+	-

Figure 52: Accidental Torsion for Seismic Loads

Overturning Moment

Due to slight differences in overall weight based off of hand calculations and computer models, the weight was estimated as 10,000 kips. The worst case is seismic loading in the Y direction. The overturning moment is equal to 8.2% of the dead load resisting moment which is an acceptable amount. There is also flexural resistance provided by the shear walls.



Conclusion

This report analyzes the lateral system of the Hotel N.E.U.S. Wind and Seismic forces were calculated by ASCE 7-05. Building properties such as seismic weight and mass along with the center of mass and rigidity were evaluated by hand.

A computer model was produced in ETABS and RAM. RAM was able to calculate the load cases based on input, while load combinations were developed by hand to use in ETABS. Results from the two programs were compared to analyze the lateral system. The modes, displacements, drifts, stiffness, and shear values were gathered from the models. All drifts and displacements met code and serviceability allowances.

A spot check was performed on a critical shear wall in the X direction. This direction is an area of concern because there is only one line of resistance, meaning there is little capacity to resists torsion and it must resist a large amount of shear. It was determined that the walls did not have enough capacity to resist wind or seismic loads. For wind however, the required extra length of wall was minimal. The design engineer could have decided that wind was the controlling case and assumed shear resistance would come from other elements in the building.

Appendices

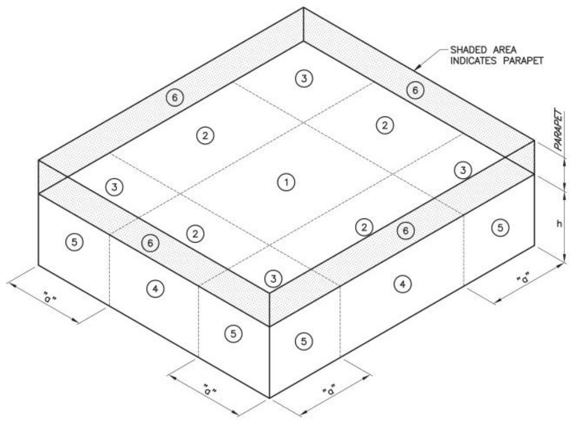
Appendix A: Plans and Sections

COMPONENT AND CLADDING WIND PRESSURES

TRIBUTARY AREA (SF)	ROOF ZONE			WALL ZONE		PARAPET
	1	2	3	4	5	6
10	−35	−54	−55	+24/−28	+24/−35	+71/−71
20	−33	−53	−52	+22/−27	+22/−32	+67/−67
50	−30	−48	−48	+21/−25	+21/−29	+62/−62
100	−28	−46	−45	+20/−24	+20/−27	+58/−58
200	−26	−43	−43	+20/−23	+20/−25	+54/−54
500	−24	−39	−39	+17/−21	+17/−21	+49/−49

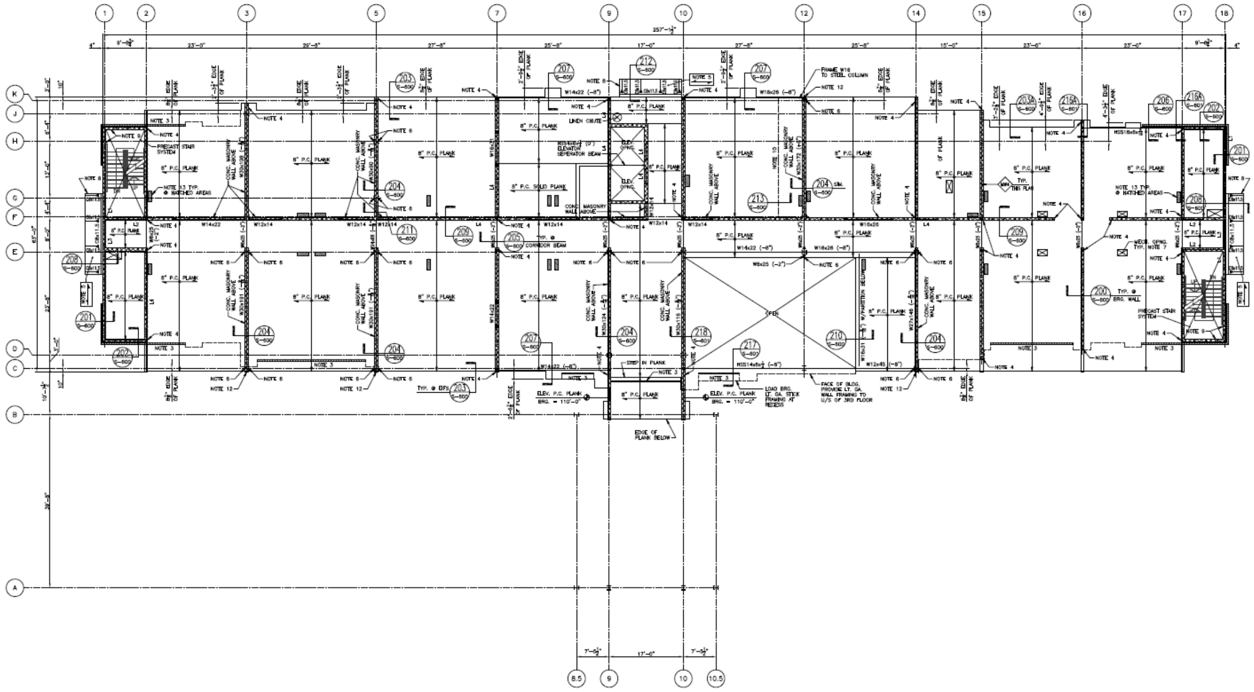
NOTES:

1. ALL LOADS ARE IN POUNDS PER SQUARE FOOT (PSF).
2. (+) DENOTES PRESSURE, (−) DENOTES SUCTIONS.
3. "a" SHALL BE 10% OF LEAST HORIZ. DIMENSION OR 0.4h, WHICHEVER IS SMALLER, BUT NOT LESS THAN 4% OF LEAST HORIZ. DIMENSION OR 3'−0".

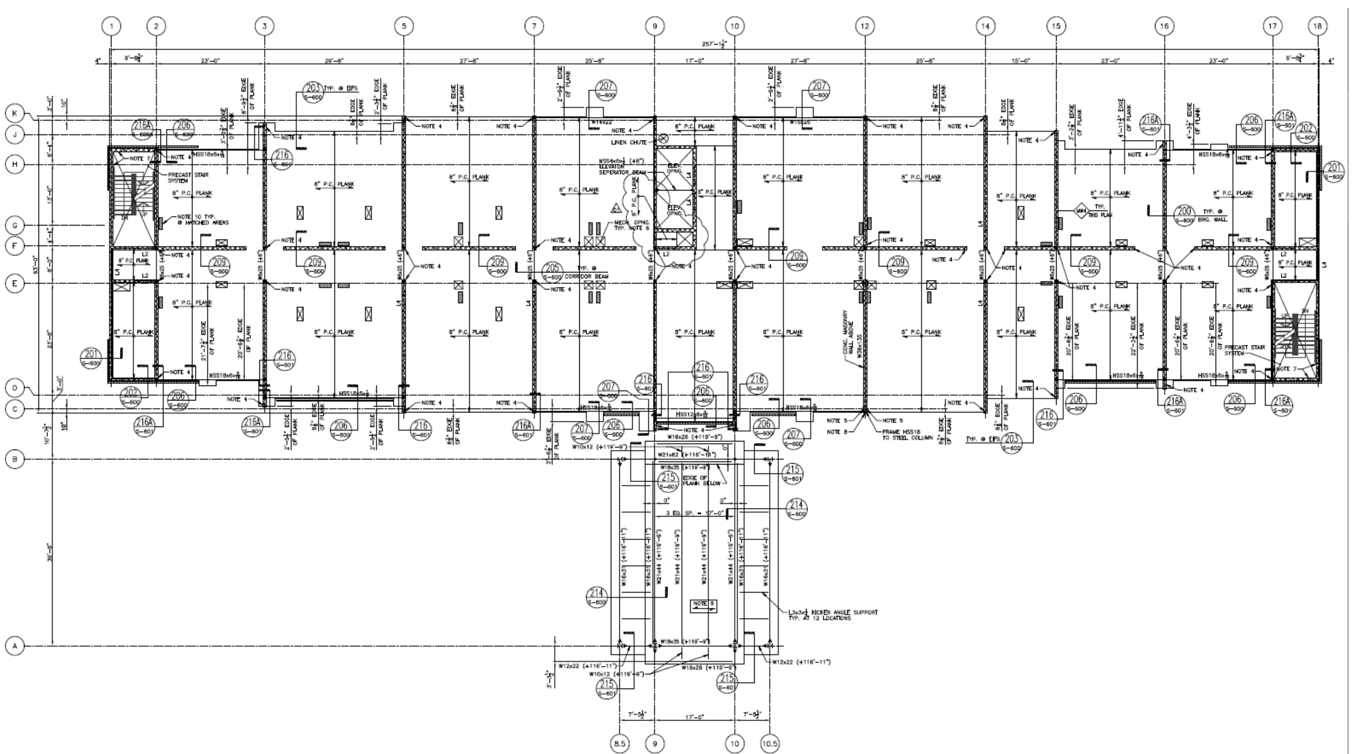


ROOF AND WALL ZONES

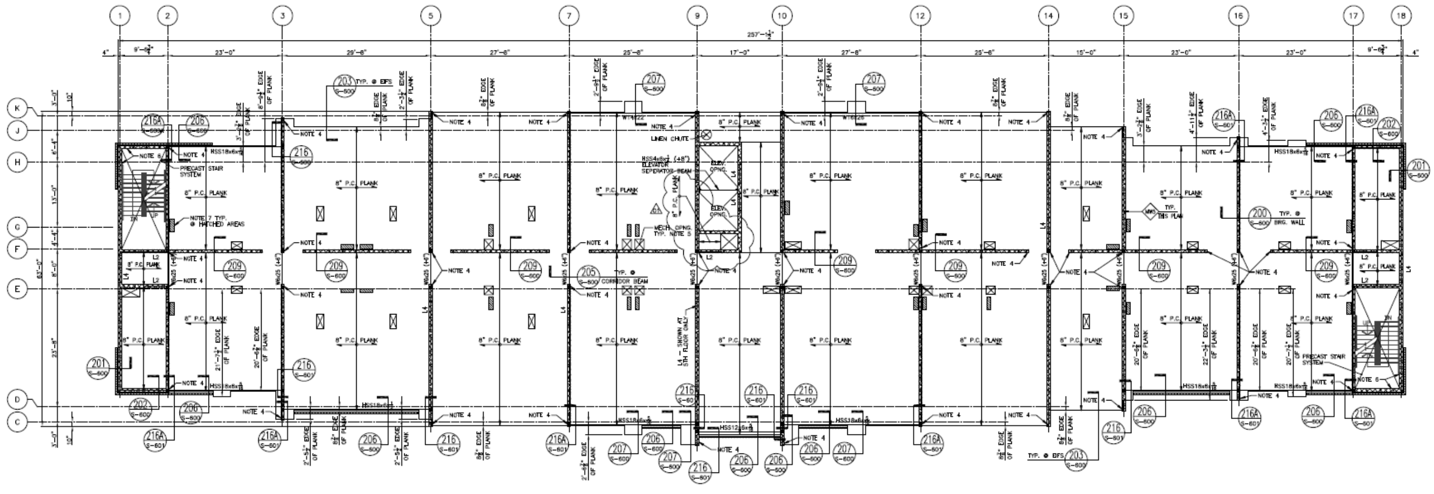
- ✓ IBC 2009
- ✓ International Mechanical Code (IMC 2009)
- ✓ International Plumbing Code (IPC 2009)
- ✓ International Fire Code (IFC 2009)
- ✓ National Fire Protection Associations (NFPA)
- ✓ ADA Accessibility Guidelines (ADAAG) and American National Standards Institute (ANSI)



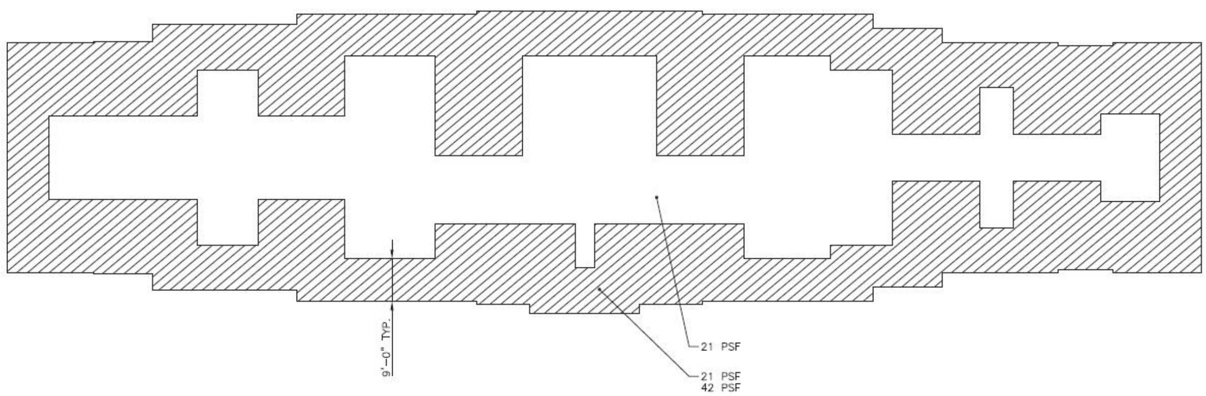
SECOND FLOOR FRAMING PLAN
SCALE: 1/8" = 1'-0"



THIRD FLOOR FRAMING PLAN
SCALE: 1/8" = 1'-0"

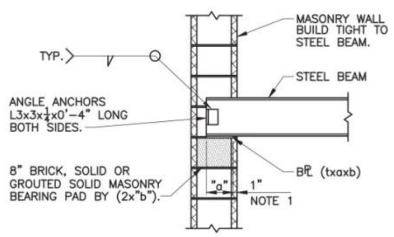


FOURTH AND FIFTH FLOOR FRAMING PLAN



ROOF SNOW LOAD DIAGRAM

SCALE: $\frac{1}{16}$" = 1'-0"

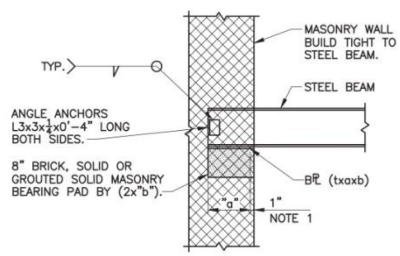


NOTES:

1. FOR BPL'S THAT ARE 1" SMALLER THAN THE MASONRY WALL, CENTER THE BPL ON THE WALL.

TYPICAL STEEL BEAM BEARING ON MASONRY WALL DETAIL

ALTERNATE DETAIL:
PROVIDE 2–½"ø ANCHOR BOLTS INTO GROUTED SOLID MASONRY BEARING W/ NO ANGLE ANCHORS.

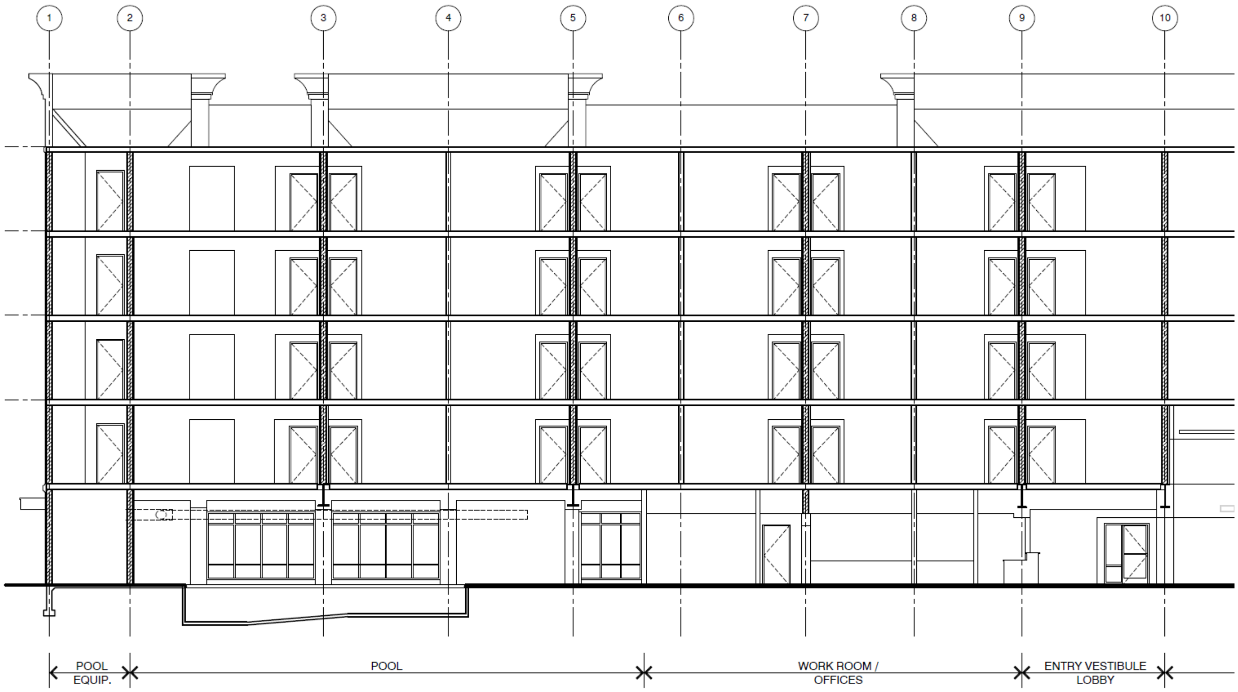


NOTES:

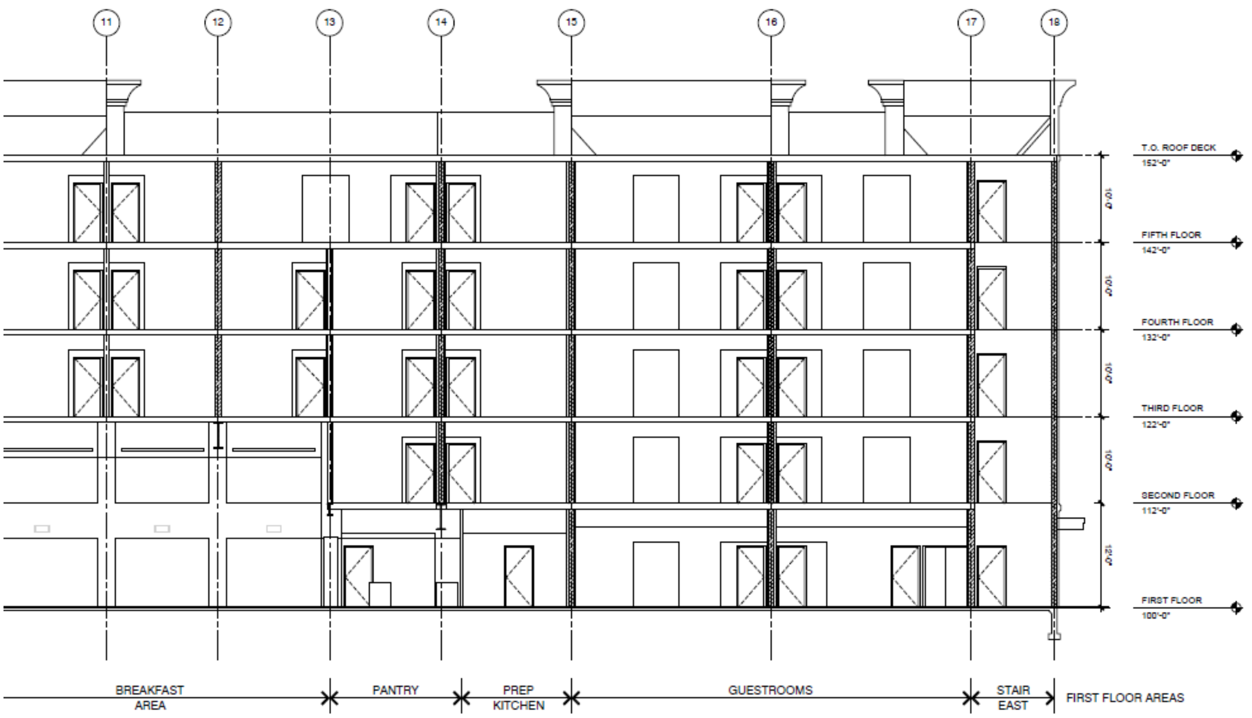
1. FOR BPL'S THAT ARE 1" SMALLER THAN THE MASONRY WALL, CENTER THE BPL ON THE WALL.

TYPICAL STEEL BEAM BEARING ON MASONRY END WALL DETAIL

ALTERNATE DETAIL:
PROVIDE 2–½"ø ANCHOR BOLTS INTO GROUTED SOLID MASONRY BEARING W/ NO ANGLE ANCHOR.

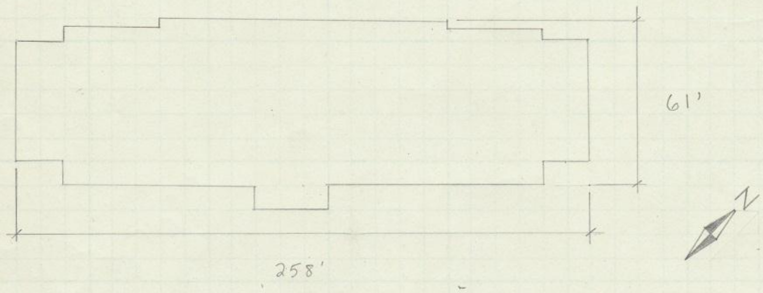


A12 **EAST-WEST BUILDING SECTION**
1/8" = 1'-0"



Appendix B: Wind Calculations

TECH 1 | WIND ANALYSIS 1 | JORDAN RUTHERFORD



PARAPET HEIGHT: 60'8" USE METHOD 2
E-W DIRECTION: 258'
N-S DIRECTION: 61'

WIND SPEED: (FIG 6-1) $V = 90$ MPH

DIRECTIONALITY FACTOR: (TBL 6-4) $K_d = 0.85$

OCCUPANCY CATEGORY: (TBL 6-1) II

IMPORTANCE FACTOR: (TBL 6-1) $I = 1.0$

EXPOSURE CATEGORY: (6.5.6.3) C

TOPOGRAPHIC FACTOR: (FIG 6-4) $K_{zt} = 1.0$

VELOCITY PRESSURE COEFFICIENTS: (TBL 6-3) VARIES W/ HEIGHT

INTERNAL PRESSURE COEFFICIENT: $GC_{pi} = \pm 0.18$

GUST FACTOR: (6.5.8.1) $G_f = 0.85$*

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{33}{0.6(60.67)}\right)^{1/6} = .197$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{0.6(60.67)}{33}\right)^{1/5} = 509.9$$

* CALCS PERFORMED BASED ON RIGID STRUCTURE, 0.85 USED TO BE CONSERVATIVE

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{258+60.67}{509.9}\right)^{.63}}} = .825 \qquad G_f = 0.925\left(\frac{1+(1.7)(3.4)(.197)(.825)}{1+(1.7)(3.4)(.197)}\right) = .8386$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{61+60.67}{509.9}\right)^{.63}}} = .892 \qquad G_f = 0.925\left(\frac{1+(1.7)(3.4)(.197)(.892)}{1+(1.7)(3.4)(.197)}\right) = .872$$

TECH 1 | WIND ANALYSIS 2 | JORDAN RUTHERFORD

BUILDING IS FULLY ENCLOSED

WALL PRESSURE COEFFICIENTS: (FIG 6-6)

	C_p		USE WITH
WINDWARD:	0.8		q_z
SIDEWALL:	-0.5	(L/B < 1) (FOR N-S)	q_h
	-0.2	(L/B > 4) (FOR E-W)	q_h
LEEWARD:	-0.7		q_h

ROOF PRESSURE COEFFICIENTS (FIG 6-6) q_h

		C_p	
WINDWARD:	0 – h/2	-0.9	2h = 121 < 258'
	h/2 – h	-0.9	∴ USE ALL
$h/L = \frac{60.67}{258} = .22 < .5$	h – 2h	-0.5	h/2 = 30.33'
	> 2h	-0.3	

		C_p	AREA	
WINDWARD:	0 – h/2	-1.3	(30.33)(258') = 7825 > 1000	X
$h/L = \frac{60.67}{61} = .99 > .5$	> h/2	-0.7 ← REDUCE x0.8	(30.67)(258') = 7912 > 1000	✓

VELOCITY PRESSURES: $q_z = 0.00256 K_z K_{zt} K_d V^2 I$

$q_h = 0.00256 K_z K_{zt} K_d V^2 I$

DESIGN WIND PRESSURES: (6.5.12.2)

WINDWARD: $p_z = q_z(GC_p) - q_h(GC_{pi})$

LEEWARD:
SIDEWAYS
ROOF

$p_h = q_h(GC_p) - q_h(GC_{pi})$

PARAPET: $p_p = q_p(GC_{pn}) = q_p(1.5)$ WINDWARD
$q_p(-1.0)$ LEEWARD

☆ FOR VALUES, SEE EXCEL TABLES

Wind Load Data		
Design Wind Speed	V	90
Directionality Factor	Kd	0.85
Occupancy Category	I	II
Importance Factor		1
Exposure Category		C
Topographic Factor	Kzt	1
Internal Pressure Coefficient	Gcpi	+/-0.18
Gust Factor	G	.85
Wall Pressure Coefficients		
Windward	Cp	0.8
Side Wall (N-S)	Cp	-0.5
Side Wall (E-W)	Cp	-0.2
Leeward	Cp	-0.7
Roof Pressure Coefficients		
Windward (E-W)	0-h/2	-0.9
	h/2-h	-0.9
	h-2h	-0.5
	>2h	-0.3
Windward (N-S)	0-h/2	-1.3
	>h/2	-0.56

Velocity Pressures							
Level	Elevation	K_z	K_{zt}	K_d	V^2	I	q_z
	60.67	1.1327	1	0.85	8100	1	19.964
Parapet	52	1.098	1	0.85	8100	1	19.3529
5	42	1.05	1	0.85	8100	1	18.5069
4	32	0.992	1	0.85	8100	1	17.4846
3	22	0.916	1	0.85	8100	1	16.145
2	12	0.85	1	0.85	8100	1	14.9818
Ground	0	0.85	1	0.85	8100	1	14.9818

Appendix C: Seismic Calculations

TECH 1 | SEISMIC ANALYSIS 1 | JORDAN RUTHERFORD

EQUIVALENT LATERAL FORCE METHOD

OCCUPANCY CATEGORY: (TBL 1-1) — II

SITE CLASS: (GEOTECH. REPORT) — D

SEISMIC LOAD IMPORTANCE FACTOR: (FIG 11.5-1) — $I_e = 1.0$

SPECTRAL RESPONSE ACCELERATIONS: (FIG 22-1,2) — $S_s = 0.125$, $S_1 = 0.049$

SITE CLASS COEFFICIENT: (TBL 11.4-1,2)

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

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

SPECTRAL RESPONSE COEFFICIENT: (TBL 11.4-3,4)

$S_{DS} = 2/3\,(0.2) = 0.1333$

$S_{D1} = 2/3\,(0.1176) = 0.0784$

SEISMIC DESIGN CATEGORY: (TBL 11.6-1,2) — B

BASE SEISMIC FORCE RESISTING SYSTEM: (TBL 12.2-1) — $R = 2$
REINFORCED MASONRY SHEAR WALLS

APPROXIMATE FUNDAMENTAL PERIOD: (12.8.2.1)

$T_a = C_t h_n^x = 0.02(52)^{0.75} = 0.387$ FOR "OTHER" SYSTEMS (TBL 12.8-1)

OR

$T_a = \frac{0.0019}{\sqrt{C_w}} h_n =$ E-W: 0.1217 | N-S: 0.08

FOR MASONRY SHEAR WALLS 12.8-9

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]} = 0.659 \;/\; 1.524$$

$A_b = 15725\ ft^2$

SEE EXCEL FOR A_i, D_i, h_i, t_i

TECH 1 | SEISMIC ANALYSIS 2 | JORDAN RUTHERFORD

SEISMIC RESPONSE COEFFICIENT: (12.8.1.1)

COEFF. FOR UPPER LIMIT ON PERIOD: (12.8-1)

$C_u = 1.7 \times T_a = 0.387 = T = 0.6579$

$T_L = 12\,s$ (FIG 22-15)

$$C_s = \frac{S_{DS}}{\frac{R}{I}} = \frac{0.1333}{\left(\frac{2}{1}\right)} = 0.067$$

$C_{s\,MAX}$:

$$\frac{S_{D1}}{T\left(\frac{R}{I}\right)} \quad \frac{0.0784}{0.6579\left(\frac{2}{1}\right)} = 0.059 \qquad \text{FOR } T < T_L \text{ OK}$$

$$\frac{S_{D1} T_L}{T^2\left(\frac{R}{I}\right)} \quad \frac{0.0784\,(12)}{(0.6579^2)\left(\frac{2}{1}\right)} = 1.087 \qquad \text{FOR } T > T_L \text{ NG}$$

$C_{s\,min}$:

$$\frac{0.5\,S_1}{\left(\frac{R}{I}\right)} = \frac{0.5\,(0.049)}{\left(\frac{2}{1}\right)} = 0.01225$$

$$0.01$$

SEE EXCEL TABLE FOR DETAILED C_s PER DIR.

TECH 1 | SEISMIC ANALYSIS 3 | JORDAN RUTHERFORD

BASE SHEAR: (12.8.1)

DEAD LOAD:

P.C. PLANK: 56 psf
3/4" TOPPING: 6 psf
PARTITIONS: 15 psf (4.2.2)
MEP/MISC: 5 psf
CEILING: 3 psf
85 psf

ROOF:

P.C. PLANK: 56 psf
MEP/MISC: 5 psf
CEILING: 3 psf
INSULATION: 12 psf
76 psf

FLOOR AREAS	WEIGHT	WALL WEIGHT
2: 14871 ft²	1264 k	COMPLETED
3: 14871 ft²	1264 k	IN
4: 14871 ft²	1264 k	EXCEL
5: 14871 ft²	1130 k	

$V = C_s W = 0.067(10907) = 649$

VERTICAL FORCE DISTRIBUTION: (12.8.3)

$F_x = C_{vx} V$

$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1} w_i h_i^k}$

CALCULATIONS DONE IN EXCEL

Seismic Load Data		
Occupancy Category	-	II
Site Class	-	D
Seismic Load Importance Factor	I_e	1
Site Class Coefficient	S_s	0.125
	S_1	0.049
Spectral Response Coefficient	F_a	1.6
	F_v	2.4
	S_{DS}	0.1333
	S_{D1}	0.0784
Seismic Design Category	-	B
Response Modification Factor	R	2
Long Period Transition Period	T_L	12
Fundamental Period	Ta	0.387

Masonry Shear Wall Data							
Type	C_u	T_a	T	C_{smin}		C_{smax}	C_s
E-W	1.7	0.122	0.207	0.012	0.010	**0.189**	**0.067**
N-S	1.7	0.080	0.136	0.012	0.010	**0.288**	**0.067**
General	1.7	0.387	0.658	0.012	0.010	**0.060**	**0.067**

Base Shear					
Type	Weight	C_s	V (k)	C_{smax}	V (k)
E-W	10698.0	0.189	**2027**	0.060	**637**
N-S	10698.0	0.288	**3083**	0.060	**637**
General	10698.0	0.067	**717**	0.060	**637**

*All controlled by Csmax

Masonry Shear Wall Data (C_w) for E-W												
Column Line	t_i (in)	D_i (ft)	A_i (ft)	h_i (ft)	h_n (ft)	Floor	Σ	A_b	100/A_b	Σ	C_w	T_a
1	8.00	40.00	26.67	52.00	52.00	1.00	11.10	15725	0.006359	103.65	0.659153	**0.1217**
2	8.00	40.00	26.67	52.00	52.00	1.00	11.10					
7	8.00	41.27	27.51	52.00	52.00	1.00	11.87					
9	8.00	30.96	20.64	52.00	52.00	1.00	6.17					
10	8.00	38.79	25.86	52.00	52.00	1.00	10.38					
14	8.00	26.67	17.78	52.00	52.00	1.00	4.28					
15	8.00	47.55	31.70	52.00	52.00	1.00	15.91					
16	8.00	39.75	26.50	52.00	52.00	1.00	10.95					
17	8.00	39.75	26.50	52.00	52.00	1.00	10.95					
18	8.00	39.75	26.50	52.00	52.00	1.00	10.95					
						Σ	103.65					

Masonry Shear Wall Data (C_w) for N-S												
Column Line	t_i (in)	D_i (ft)	A_i (ft)	h_i (ft)	h_n (ft)	Floor	Σ	A_b	100/A_b	Σ	C_w	T_a
F	8.00	70.50	47.00	52.00	52.00	1.00	32.38	15725	0.006359	239.6828	1.524215	**0.08**
						Σ	239.6828					

Masonry Shear Wall Data (C_w) for E-W												
Column Line	t_i (in)	D_i (ft)	A_i (ft)	h_i (ft)	h_n (ft)	Floor	Σ	A_b	100/A_b	Σ	C_w	T_a
1	8.00	39.75	26.50	52.00	52.00	1.00	10.95	15725	0.006359	98.97	0.629391	**0.1245**
2	8.00	39.75	26.50	52.00	52.00	1.00	10.95					
7	8.00	41.33	27.56	52.00	52.00	1.00	11.91					
9	8.00	19.00	12.67	52.00	52.00	1.00	1.76					
10	8.00	38.79	25.86	52.00	52.00	1.00	10.38					
14	8.00	26.67	17.78	52.00	52.00	1.00	4.28					
15	8.00	47.55	31.70	52.00	52.00	1.00	15.91					
16	8.00	39.75	26.50	52.00	52.00	1.00	10.95					
17	8.00	39.75	26.50	52.00	52.00	1.00	10.95					
18	8.00	39.75	26.50	52.00	52.00	1.00	10.95					
						Σ	98.97					

Masonry Shear Wall Data (C_w) for N-S												
Column Line	t_i (in)	D_i (ft)	A_i (ft)	h_i (ft)	h_n (ft)	Floor	Σ	A_b	100/A_b	Σ	C_w	T_a
F	8.00	70.50	47.00	52.00	52.00	1.00	32.38	15725	0.006359	230.3226	1.46469	**0.0816**
						Σ	230.3226					

Masonry Wall Weight (tek 14-3b)							
Type	Width	Vertical Reinforcing	Weight (psf)	Length (ft)	Height (ft)	Floor	Weight (k)
Masonry Wall 1	8"	#5 @ 24" O.C.	47	525	6	G	148.05
			47	798	10	2	1500.24
			47	721	10	3	1355.48
			47	721	10	4	1355.48
			47	721	10	5	1355.48
Masonry Wall 2	8"	#5 @ 24" O.C.	47	161	6	G	45.40
			47	161	10	2	75.67
			47	161	10	3	75.67
			47	161	10	4	75.67
			47	161	10	5	75.67
Masonry Wall 3	12"	#5 @ 48" O.C.	53	499	6	G	158.68
Masonry Wall 4	8"	#5 @ 24" O.C.	47	26	10	3	12.22
Masonry Wall 5	8"	#5 @ 32" O.C.	43	26	10	4	11.18
			43	26	10	5	11.18

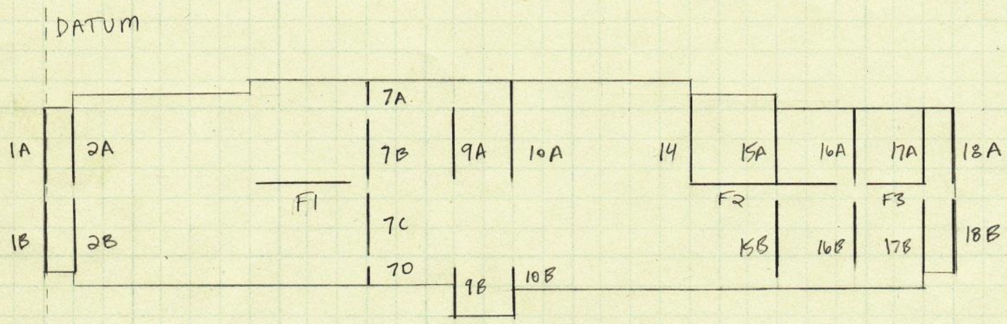
Total	G	352.13
	2	1575.91
	3	1443.37
	4	1442.33
	5	1442.33

Floor Dead Loads	Load (psf)	Reference
8" Precast Plank	56	PCI MNL 120
3/4" Topping	6	DATA FROM AES
Paritions	10	12.14.8.1
MEP/Misc.	5	
Ceiling	3	
Total	80	
Roof Dead Load	**Load (psf)**	**Reference**
8" Precast Plank	56	PCI MNL 120
MEP/Misc.	5	
Ceiling	3	
Insulation	12	DATA FROM AES
Total	76	

Total Building Weight				
Level	Area (ft^2)	Load (k)	Wall Weight (k)	Total (k)
Ground	15725	0	352.13	352.13
2	13133	1051	1575.91	2626.55
3	14370	1150	1443.37	2592.97
4	14370	1150	1442.33	2591.93
5	14370	1092	1442.33	2534.45
			Total Weight(k)	**10698.03**

Appendix D: Wall Stiffness, Deflection, Shear

TECH 3	LAYOUT, RIGIDITY	JORDAN RUTHERFORD	1



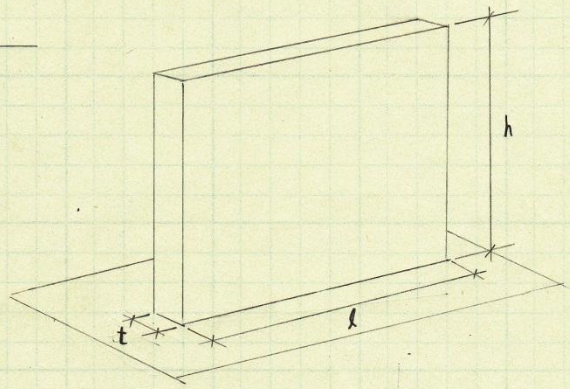
ACI 530 1.8.2.2

$E = 900\sqrt{2000} = 1800$ ksi

ALL WALLS ARE 8" THICK W/ #5's @ 24"

CANTILEVER WALL

$$k = \frac{Et}{\left(\frac{h}{L}\right)\left[4\left(\frac{h}{L}\right)^2 + 3\right]}$$



FIRST FLOOR

• WALLS : 1A, 1B, 2A, 2B, 16A, 16B, 17A, 17B, 18A, 18B

h = 12' = 144"
L = 19.875' = 238.5"

$$k = \frac{(1800 \text{ ksi})(8")}{\left(\frac{144}{238.5}\right)\left[4\left(\frac{144}{238.5}\right)^2 + 3\right]} = 5349.7 \text{ k/in}$$

• WALLS : 7A

h = 144"
L = 7'-8" = 92"

$$k = \frac{(1800 \text{ ksi})(8")}{\left(\frac{144}{92}\right)\left[4\left(\frac{144}{92}\right)^2 + 3\right]} = 718.8 \text{ k/in}$$

• WALL : 7D

h = 144"
L = 9'-8" = 116"

$$k = \frac{(1800 \text{ ksi})(8")}{\left(\frac{144}{116}\right)\left[4\left(\frac{144}{116}\right)^2 + 3\right]} = 1265.8 \text{ k/in}$$

TECH 3 | RIGIDITY | JORDAN RUTHERFORD | 2

- WALLS: 7B, 7C h = 144" L = 144"

$$k = \frac{(1800\,ksi)(8")}{\left(\frac{144}{144}\right)\left[4\left(\frac{144}{144}\right)^2 + 3\right]} = 2057.1\ k/in$$

- WALLS: 9A h = 144" L = 228"

$$k = \frac{(2208\,ksi)(8")}{\left(\frac{144}{228}\right)\left[4\left(\frac{144}{228}\right)^2 + 3\right]} = 4961.3\ k/in$$

- WALLS: 9B, 10B h = 144" L = 111"

$$k = \frac{(2208\,ksi)(8")}{\left(\frac{144}{111}\right)\left[4\left(\frac{144}{111}\right)^2 + 3\right]} = 1140.6\ k/in$$

- WALLS: 10A, 14 h = 144" L = 320"

$$k = \frac{(2208\,ksi)(8")}{\left(\frac{144}{320}\right)\left[4\left(\frac{144}{320}\right)^2 + 3\right]} = 8399\ k/in$$

- WALLS: 15A, 15B h = 144" L = 284"

$$k = \frac{(2208\,ksi)(8")}{\left(\frac{144}{284}\right)\left[4\left(\frac{144}{284}\right)^2 + 3\right]} = 7050\ k/in$$

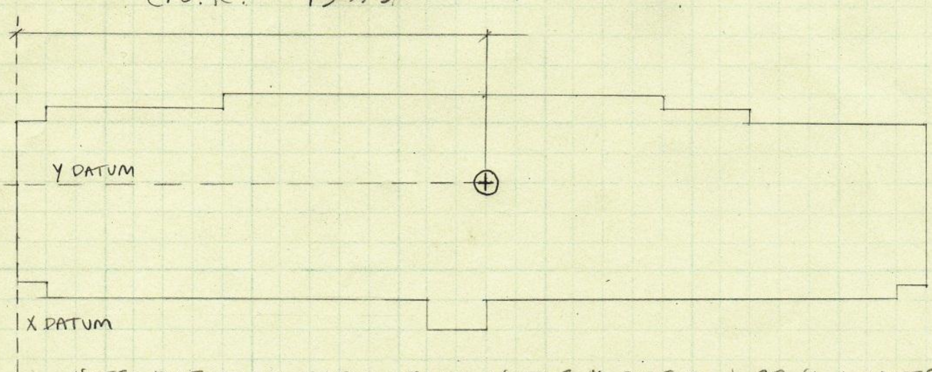
- TOTAL = (10)(5349.7) + (1)(718.8) + (1)(1265.8) + (2)(2057.1) + (2)(4961.3) + (2)(1140.6) + (2)(8399) + (2)(8648)

$\Sigma R = 97736.5$

- CENTER OF RIGIDITY (X-DIRECTION)

$$\frac{R\,d}{\Sigma R} = \frac{(2)(5349.7)(9.563) + (718.8)(89.897) + (2)(2057.1)(89.897) + (1265.8)(89.897) + (4961.3)(115.564) + (1140.6)(115.564) + (8399)(132.564) + (1140.6)(132.564) + (8399)(185.898) + (7050)(200.898)(2) + (5349.7)(203.898)(2) + (5349.7)(246.898)(2) + (5349.7)(256.461)(2)}{97736.5}$$

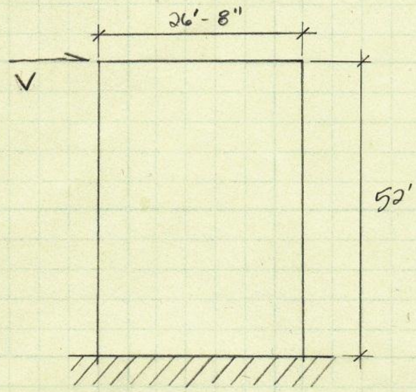
C.O.R. = 135.5'



NOTE: DUE TO ALL SHEAR WALLS IN THE X-DIRECTION BEING LOCATED ON THE SAME COL. LINE, THE Y-COMPONENT IS ON THAT LINE

TECH 3 | DEFLECTION CHECK | JORDAN RUTHERFORD | 1

SHEAR WALL 10A



$$\Delta = \Delta_{CANT} + \Delta_{FIXED}$$

$$\Delta_{CANT} = \frac{V\ell^3}{3EI}$$

$$\Delta_{FIXED} = \frac{1.2V\ell}{E_v A}$$

V = 52k (FROM ETABS)

E = 900 (2000) = 1800 ksi (MIN f'm WAS USED)

$$I = \frac{td^3}{12} = \frac{(8")(320")^3}{12} = 2.185E^7 \text{ in}^4$$

$E_v = 0.4E = 720$ ksi

$A = td = (8)(320) = 2560 \text{ in}^2$

$$\Delta_{CANT} = \frac{(52k)(624")^3}{3(1800 ksi)(2.185E7 in^4)} = 0.108"$$

$$\Delta_{FIXED} = \frac{1.2(52k)(624")}{(720 ksi)(2560 in^4)} = .0211"$$

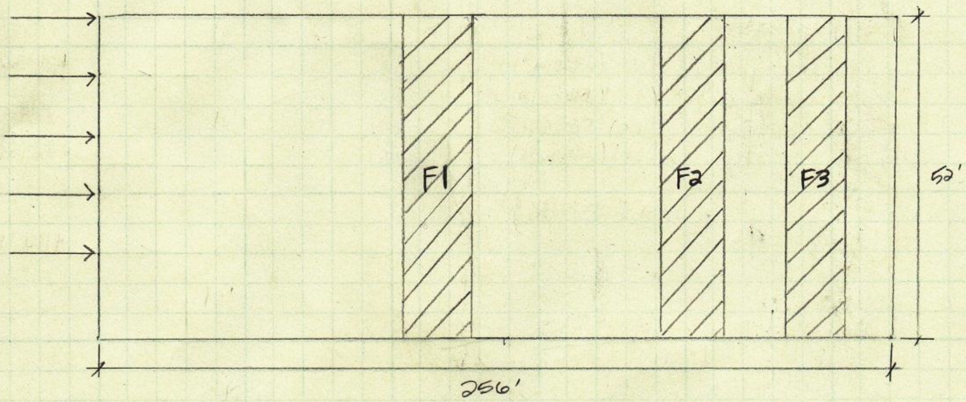
$\Delta = 0.129" < 1.56"$ OK ✓

$$\ell/400 = \frac{(52' \times 12)}{400} = 1.56"$$

TECH 3 | SHEAR CHECK | JORDAN RUTHERFORD | 1

X DIRECTION SHEAR CAPACITY: WALLS F1, F2, F3

- SINCE THERE IS ONLY ONE LINE OF ACTION → NO TORSION
- CONSIDER DIRECT SHEAR



BY ACI 530-05

$F_v = \sqrt{f'_m}$

$F_v = \sqrt{2000}\,psi = 44.7\,psi > \boxed{35\,psi}$ GOVERNS

IN WALL:

#5 @ 24" O.C. (CELLS FULLY GROUTED)

$(17.5 + 18.83 + 19.17)(12") = 666"/24" = 27.75$ GROUTED CELLS

$AREA = (1\tfrac{1}{4}")(2)(17.5+34)(12") = 1545\ in^2$

$(27.75)(25\ in^2)$ $\quad 693.75\ in^2$

$2238.75\ in^2$

CAPACITY OF ALL WALLS IN X DIRECTION

$f = (35\,psi)(2238.75\ in^2) = 78.4\,k$

$= (35\,psi)\left[(1\tfrac{1}{4}")(2)(L) + (\tfrac{L}{24"})(25\ in^2)\right]$

TECH 3	SHEAR CHECK	JORDAN RUTHERFORD	2

WIND LOADS:

HAND CALCS: 84.79 k > 78.4 k NG X

REQ L: $84.79\text{k} = (35\text{ psi})[(1\tfrac{1}{4}'')(2)(L) + (\tfrac{L}{24})(25\text{ in}^2)]$

$L = 684'' = \boxed{57'}$

EARTHQUAKE:

HAND CALCS: 636.95 k > 78.4 k

REQ L: $636.95\text{ k} = (35\text{ psi})(3.54167)L$

$L = 5138.4'' = \boxed{428'}$

AMOUNT OF SHEAR WALL NEEDED TO RESIST SEISMIC LOADS IS ALMOST 8x GREATER THAN PROVIDED!

Appendix E: Center of Mass, Weight, Mass

Level 2 Weight/Center of Mass												
	Wall/Area	Quantity		DL		Weight		Distance (x)		Distance (y)		
Walls (labels are not the same as shear walls)	1A	47.75	LF	517	PLF	24.69	K	0	ft	30.670	ft
	2A	19.88	LF	517	PLF	10.28	K	9.563	ft	44.605	ft
	2B	19.88	LF	517	PLF	10.28	K	9.563	ft	16.730	ft
	3A	23.67	LF	517	PLF	12.24	K	32.563	ft	46.500	ft
	3B	26.67	LF	517	PLF	13.79	K	32.563	ft	13.334	ft
	5A	26.67	LF	517	PLF	13.79	K	62.23	ft	46.500	ft
	5B	26.67	LF	517	PLF	13.79	K	62.23	ft	13.334	ft
	7A	26.67	LF	517	PLF	13.79	K	89.897	ft	48.000	ft
	7B	26.67	LF	517	PLF	13.79	K	89.897	ft	13.334	ft
	9A	19.00	LF	517	PLF	9.82	K	115.564	ft	48.000	ft
	9A	23.67	LF	517	PLF	12.24	K	115.564	ft	16.129	ft
	9C	9.25	LF	517	PLF	4.78	K	115.564	ft	-1.870	ft
	10A	26.67	LF	517	PLF	13.79	K	132.564	ft	48.000	ft
	10B	23.67	LF	517	PLF	12.24	K	132.564	ft	16.129	ft
	10C	9.25	LF	517	PLF	4.78	K	132.564	ft	-1.870	ft
	12	26.67	LF	517	PLF	13.79	K	160.231	ft	48.000	ft
	14A	26.67	LF	517	PLF	13.79	K	185.898	ft	48.000	ft
	14B	26.67	LF	517	PLF	13.79	K	185.898	ft	13.334	ft
	15A	23.67	LF	517	PLF	12.24	K	200.898	ft	9.938	ft
	15B	23.67	LF	517	PLF	12.24	K	200.898	ft	16.730	ft
	16A	19.88	LF	517	PLF	10.28	K	223.898	ft	44.605	ft
	16B	19.88	LF	517	PLF	10.28	K	223.898	ft	16.730	ft
	17A	19.88	LF	517	PLF	10.28	K	246.898	ft	44.605	ft
	17B	19.88	LF	517	PLF	10.28	K	246.898	ft	16.730	ft
	18A	19.88	LF	517	PLF	10.28	K	256.461	ft	44.605	ft
	18B	19.88	LF	517	PLF	10.28	K	246.461	ft	16.730	ft
	F1	28.56	LF	517	PLF	14.77	K	14.2815	ft	34.667	ft
	F2	21.67	LF	517	PLF	11.20	K	47.3965	ft	34.667	ft
	F3	19.67	LF	517	PLF	10.17	K	76.0635	ft	34.667	ft
	F4	17.67	LF	517	PLF	9.13	K	102.7305	ft	34.667	ft
	F5	9.00	LF	517	PLF	4.65	K	119.814	ft	34.667	ft
	F6	11.00	LF	517	PLF	5.69	K	138.064	ft	34.667	ft
	F7	30.67	LF	517	PLF	15.85	K	166.5645	ft	34.667	ft
	F8	27.67	LF	517	PLF	14.30	K	203.7315	ft	34.667	ft
	F9	26.25	LF	517	PLF	13.57	K	220.323	ft	34.667	ft
	F10	21.38	LF	517	PLF	11.05	K	124.064	ft	45.355	ft
	F11	9.56	LF	517	PLF	4.94	K	4.7815	ft	26.667	ft
	F12	8.50	LF	517	PLF	4.39	K	119.814	ft	37.042	ft
	F13	8.50	LF	517	PLF	4.39	K	119.814	ft	56.042	ft
	F14	9.56	LF	517	PLF	4.94	K	251.6795	ft	26.667	ft
Floor Areas	D-J, 1-2	257.29	SF	80	PSF	20.58	K	4.7815	ft	20.733	ft
	D-J, 2-3	1341.66	SF	80	PSF	107.33	K	21.063	ft	30.670	ft
	C-J, 3-5	1730.57	SF	80	PSF	138.45	K	47.3965	ft	29.167	ft
	C-K, 5-7	1696.90	SF	80	PSF	135.75	K	76.0635	ft	30.667	ft
	C-K, 7-9	1574.23	SF	80	PSF	125.94	K	102.7305	ft	30.667	ft
	C-F, 9-10	1201.34	SF	80	PSF	96.11	K	124.064	ft	30.667	ft
	F-J, 9.5-10	181.69	SF	80	PSF	14.54	K	128.314	ft	45.355	ft
	J-K, 9-10	90.02	SF	80	PSF	7.20	K	124.064	ft	58.690	ft
	F-K, 10-12	959.13	SF	80	PSF	76.73	K	146.3975	ft	44.000	ft
	F-K, 12-14	889.80	SF	80	PSF	71.18	K	173.0645	ft	44.000	ft
	C-E, 13-14	342.23	SF	80	PSF	27.38	K	179.48125	ft	13.333	ft
	C-J, 14-15	875.00	SF	80	PSF	70.00	K	193.398	ft	30.462	ft
	D-J, 15-16	1098.25	SF	80	PSF	87.86	K	212.398	ft	30.670	ft
	D-J, 16-17	1098.25	SF	80	PSF	87.86	K	235.398	ft	30.670	ft
	D-J, 17-18	257.29	SF	80	PSF	20.58	K	251.6795	ft	40.605	ft
Total Area / Total Weight								13593.63	SF	1528.10	K
Weight per Square Foot								0.1124		KSF	
Floor Mass								2.03293E-06		K-in	
Center of Mass								123.65	ft	31.87	ft

Level 3-5 Weight/Center of Mass											
	Wall/Area	Quantity		DL		Weight		Distance (x)		Distance (y)	
Walls (labels are not the same as shear walls)	1A	47.75	LF	470	PLF	22.44	K	0	ft	30.670	ft
	2A	19.88	LF	470	PLF	9.34	K	9.563	ft	44.605	ft
	2B	19.88	LF	470	PLF	9.34	K	9.563	ft	16.730	ft
	3A	23.67	LF	470	PLF	11.12	K	32.563	ft	46.500	ft
	3B	26.67	LF	470	PLF	12.53	K	32.563	ft	13.334	ft
	5A	26.67	LF	470	PLF	12.53	K	62.23	ft	46.500	ft
	5B	26.67	LF	470	PLF	12.53	K	62.23	ft	13.334	ft
	7A	26.67	LF	470	PLF	12.53	K	89.897	ft	48.000	ft
	7B	26.67	LF	470	PLF	12.53	K	89.897	ft	13.334	ft
	9A	19.00	LF	470	PLF	8.93	K	115.564	ft	48.000	ft
	9A	23.67	LF	470	PLF	11.12	K	115.564	ft	16.129	ft
	9C	9.25	LF	470	PLF	4.35	K	115.564	ft	-1.870	ft
	10A	26.67	LF	470	PLF	12.53	K	132.564	ft	48.000	ft
	10B	23.67	LF	470	PLF	11.12	K	132.564	ft	16.129	ft
	10C	9.25	LF	470	PLF	4.35	K	132.564	ft	-1.870	ft
	12A	26.67	LF	470	PLF	12.53	K	160.231	ft	48.000	ft
	12B	26.67	LF	470	PLF	12.53	K	160.231	ft	13.334	ft
	14A	26.67	LF	470	PLF	12.53	K	185.898	ft	48.000	ft
	14B	26.67	LF	470	PLF	12.53	K	185.898	ft	13.334	ft
	15A	23.67	LF	470	PLF	11.12	K	200.898	ft	9.938	ft
	15B	23.67	LF	470	PLF	11.12	K	200.898	ft	16.730	ft
	16A	19.88	LF	470	PLF	9.34	K	223.898	ft	44.605	ft
	16B	19.88	LF	470	PLF	9.34	K	223.898	ft	16.730	ft
	17A	19.88	LF	470	PLF	9.34	K	246.898	ft	44.605	ft
	17B	19.88	LF	470	PLF	9.34	K	246.898	ft	16.730	ft
	18A	19.88	LF	470	PLF	9.34	K	256.461	ft	44.605	ft
	18B	19.88	LF	470	PLF	9.34	K	246.461	ft	16.730	ft
	F1	28.56	LF	470	PLF	13.42	K	14.2815	ft	34.667	ft
	F2	21.67	LF	470	PLF	10.18	K	47.3965	ft	34.667	ft
	F3	19.67	LF	470	PLF	9.24	K	76.0635	ft	34.667	ft
	F4	17.67	LF	470	PLF	8.30	K	102.7305	ft	34.667	ft
	F5	9.00	LF	470	PLF	4.23	K	119.814	ft	34.667	ft
	F6	11.00	LF	470	PLF	5.17	K	138.064	ft	34.667	ft
	F7	30.67	LF	470	PLF	14.41	K	166.5645	ft	34.667	ft
	F8	27.67	LF	470	PLF	13.00	K	203.7315	ft	34.667	ft
	F9	26.25	LF	470	PLF	12.34	K	220.323	ft	34.667	ft
	F10	21.38	LF	470	PLF	10.05	K	124.064	ft	45.355	ft
	F11	9.56	LF	470	PLF	4.49	K	4.7815	ft	26.667	ft
	F12	8.50	LF	470	PLF	4.00	K	119.814	ft	37.042	ft
	F13	8.50	LF	470	PLF	4.00	K	119.814	ft	56.042	ft
	F14	9.56	LF	470	PLF	4.49	K	251.6795	ft	26.667	ft
Floor Areas	D-J, 1-2	257.29	SF	80	PSF	20.58	K	4.7815	ft	20.733	ft
	D-J, 2-3	1341.66	SF	80	PSF	107.33	K	21.063	ft	30.670	ft
	C-J, 3-5	1730.57	SF	80	PSF	138.45	K	47.3965	ft	29.167	ft
	C-K, 5-7	1696.90	SF	80	PSF	135.75	K	76.0635	ft	30.667	ft
	C-K, 7-9	1574.23	SF	80	PSF	125.94	K	102.7305	ft	30.667	ft
	C-F, 9-10	1042.66	SF	80	PSF	83.41	K	124.064	ft	30.667	ft
	F-J, 9.5-10	181.69	SF	80	PSF	14.54	K	128.314	ft	45.355	ft
	J-K, 9-10	90.02	SF	80	PSF	7.20	K	124.064	ft	58.690	ft
	C-K, 10-12	1696.90	SF	80	PSF	135.75	K	146.3975	ft	30.667	ft
	C-K, 12-14	1696.90	SF	80	PSF	135.75	K	173.0645	ft	30.667	ft
	C-E, 13-14	342.23	SF	80	PSF	27.38	K	179.48125	ft	13.333	ft
	C-J, 14-15	875.00	SF	80	PSF	70.00	K	193.398	ft	30.462	ft
	D-J, 15-16	1098.25	SF	80	PSF	87.86	K	212.398	ft	30.670	ft
	D-J, 16-17	1098.25	SF	80	PSF	87.86	K	235.398	ft	30.670	ft
	D-J, 17-18	257.29	SF	80	PSF	20.58	K	251.6795	ft	40.605	ft
Total Area / Total Weight								14979.82	SF	1611.47	K
Weight per Square Foot								0.1076		KSF	
Floor Mass								1.94546E-06		K-in	
Center of Mass								126.68	ft	30.44	ft

Roof Weight/Center of Mass											
	Wall/Area	Quantity		DL		Weight		Distance (x)		Distance (y)	
Walls (labels are not the same as shear walls)	1A	47.75	LF	235	PLF	11.22	K	0	ft	30.670	ft
	2A	19.88	LF	235	PLF	4.67	K	9.563	ft	44.605	ft
	2B	19.88	LF	235	PLF	4.67	K	9.563	ft	16.730	ft
	3A	23.67	LF	235	PLF	5.56	K	32.563	ft	46.500	ft
	3B	26.67	LF	235	PLF	6.27	K	32.563	ft	13.334	ft
	5A	26.67	LF	235	PLF	6.27	K	62.23	ft	46.500	ft
	5B	26.67	LF	235	PLF	6.27	K	62.23	ft	13.334	ft
	7A	26.67	LF	235	PLF	6.27	K	89.897	ft	48.000	ft
	7B	26.67	LF	235	PLF	6.27	K	89.897	ft	13.334	ft
	9A	19.00	LF	235	PLF	4.47	K	115.564	ft	48.000	ft
	9A	23.67	LF	235	PLF	5.56	K	115.564	ft	16.129	ft
	9C	9.25	LF	235	PLF	2.17	K	115.564	ft	-1.870	ft
	10A	26.67	LF	235	PLF	6.27	K	132.564	ft	48.000	ft
	10B	23.67	LF	235	PLF	5.56	K	132.564	ft	16.129	ft
	10C	9.25	LF	235	PLF	2.17	K	132.564	ft	-1.870	ft
	12A	26.67	LF	235	PLF	6.27	K	160.231	ft	48.000	ft
	12B	26.67	LF	235	PLF	6.27	K	160.231	ft	13.334	ft
	14A	26.67	LF	235	PLF	6.27	K	185.898	ft	48.000	ft
	14B	26.67	LF	235	PLF	6.27	K	185.898	ft	13.334	ft
	15A	23.67	LF	235	PLF	5.56	K	200.898	ft	9.938	ft
	15B	23.67	LF	235	PLF	5.56	K	200.898	ft	16.730	ft
	16A	19.88	LF	235	PLF	4.67	K	223.898	ft	44.605	ft
	16B	19.88	LF	235	PLF	4.67	K	223.898	ft	16.730	ft
	17A	19.88	LF	235	PLF	4.67	K	246.898	ft	44.605	ft
	17B	19.88	LF	235	PLF	4.67	K	246.898	ft	16.730	ft
	18A	19.88	LF	235	PLF	4.67	K	256.461	ft	44.605	ft
	18B	19.88	LF	235	PLF	4.67	K	246.461	ft	16.730	ft
	F1	28.56	LF	235	PLF	6.71	K	14.2815	ft	34.667	ft
	F2	21.67	LF	235	PLF	5.09	K	47.3965	ft	34.667	ft
	F3	19.67	LF	235	PLF	4.62	K	76.0635	ft	34.667	ft
	F4	17.67	LF	235	PLF	4.15	K	102.7305	ft	34.667	ft
	F5	9.00	LF	235	PLF	2.12	K	119.814	ft	34.667	ft
	F6	11.00	LF	235	PLF	2.59	K	138.064	ft	34.667	ft
	F7	30.67	LF	235	PLF	7.21	K	166.5645	ft	34.667	ft
	F8	27.67	LF	235	PLF	6.50	K	203.7315	ft	34.667	ft
	F9	26.25	LF	235	PLF	6.17	K	220.323	ft	34.667	ft
	F10	21.38	LF	235	PLF	5.02	K	124.064	ft	45.355	ft
	F11	9.56	LF	235	PLF	2.25	K	4.7815	ft	26.667	ft
	F12	8.50	LF	235	PLF	2.00	K	119.814	ft	37.042	ft
	F13	8.50	LF	235	PLF	2.00	K	119.814	ft	56.042	ft
	F14	9.56	LF	235	PLF	2.25	K	251.6795	ft	26.667	ft
Floor Areas	D-J, 1-2	257.29	SF	80	PSF	20.58	K	4.7815	ft	20.733	ft
	D-J, 2-3	1341.66	SF	76	PSF	101.97	K	21.063	ft	30.670	ft
	C-J, 3-5	1730.57	SF	76	PSF	131.52	K	47.3965	ft	29.167	ft
	C-K, 5-7	1696.90	SF	76	PSF	128.96	K	76.0635	ft	30.667	ft
	C-K, 7-9	1574.23	SF	76	PSF	119.64	K	102.7305	ft	30.667	ft
	C-F, 9-10	1042.66	SF	76	PSF	79.24	K	124.064	ft	30.667	ft
	F-J, 9.5-10	181.69	SF	76	PSF	13.81	K	128.314	ft	45.355	ft
	J-K, 9-10	90.02	SF	76	PSF	6.84	K	124.064	ft	58.690	ft
	C-K, 10-12	1696.90	SF	76	PSF	128.96	K	146.3975	ft	30.667	ft
	C-K, 12-14	1696.90	SF	76	PSF	128.96	K	173.0645	ft	30.667	ft
	C-E, 13-14	342.23	SF	76	PSF	26.01	K	179.48125	ft	13.333	ft
	C-J, 14-15	875.00	SF	76	PSF	66.50	K	193.398	ft	30.462	ft
	D-J, 15-16	1098.25	SF	76	PSF	83.47	K	212.398	ft	30.670	ft
	D-J, 16-17	1098.25	SF	76	PSF	83.47	K	235.398	ft	30.670	ft
	D-J, 17-18	257.29	SF	76	PSF	19.55	K	251.6795	ft	40.605	ft
Total Area / Total Weight								14979.82	SF	1346.04	K
Weight per Square Foot								0.0899		KSF	
Floor Mass								1.62501E-06		K-in	
Center of Mass								126.53	ft	30.43	ft