Hyatt Place North Shore
Pittsburgh, PA

Technical Assignment #3

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Executive Summary:

In this report, the existing lateral force resisting system of the 7 story Hyatt Place North Shore are analyzed. The 70 feet tall, 108,000 square foot structure has intermediate reinforced concrete masonry bearing walls working in combination with an 8" untoped precast concrete plank floor structure to handle both gravity and lateral loads down into the soft soils along the Allegheny River and to bedrock with numerous 18" diameter auger piles.

An ETABS model was used to determine the controlling load case for the structure is *.9D + 1.0E + 1.6H*. Shear walls that were either small or have poor load paths to the foundations were excluded to simplify modeling and hand calculations. This is important to note, because semi-rigid diaphragms behave poorly when not restrained at all edges. Because of this and also the uncertainty of how rigid the diaphragm will act, there was a model created with a semi-rigid diaphragm and one with a rigid diaphragm. These were used to compare diaphragm effects and to check hand calculations of center of mass, center of rigidity, and shear based on the loads from the controlling case in each direction. The ETABS model was used exclusively to check drift and displacement against allowable code values. In this case the differences between the diaphragms became very apparent, but both rigid and semi-rigid meet code standards.

The shape of the building and layout of the lateral force resisting walls were also discussed in terms of rigidity and torsion. This leads to the determination of shear forces in walls and the deflection due to the direct and torsional shears. The walls were found to be plenty sufficient to handle the shear loads from both semi-rigid diaphragm and rigid diaphragm systems. Lastly the forces must it make safely into the foundations. The uplift forces due to overturning moment were found using ETABS and it was determined that the weight of the foundations is sufficient to keep the building firmly on the ground.

Introduction:

The Hyatt Place Hotel is part of an agreement between the Pittsburgh Steelers and Pirates that began back in 2003 with the goal to bring commercial development to the North Shore. The 108,000 SF, 178 room hotel is conveniently close to both of the teams' stadiums, Rivers Casino, and Pittsburgh in general.

Figure 1: Areal view of the North Shore courtesy of Bing.com

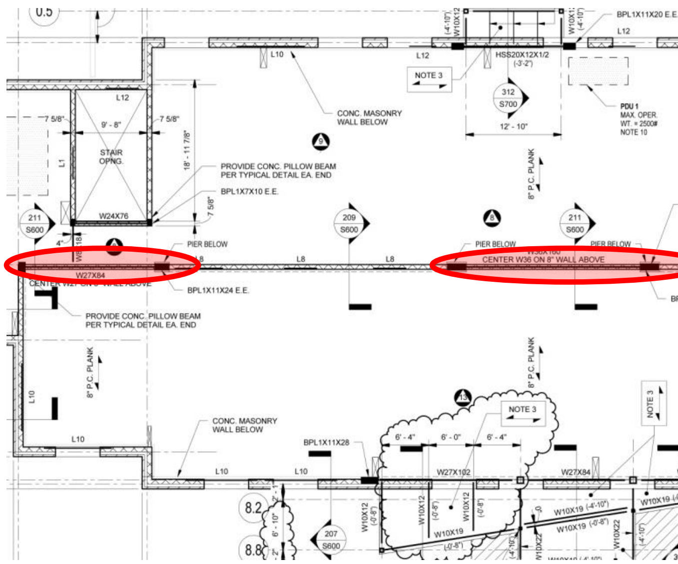
The first floor has all the expected guest amenities along with an indoor pool, lounge space, and generously sized meeting rooms. The first floor has a ceiling height of 17'-4" and the upper floors are 8'-0". Maximum floor to ceiling height is obtained with an 8 inch thick hollow core concrete plank floor system and through the use of PTACs in guestrooms. Floors 2 through 7 house 67,388 SF Net Guestroom in 178 rooms. All rooms are well sized with a partition dividing the sleeping and living spaces. Rooms are furnished with 42 inch high definition flat screen TVs and a well-designed work and entertainment center along with hotel wide Wi-Fi.

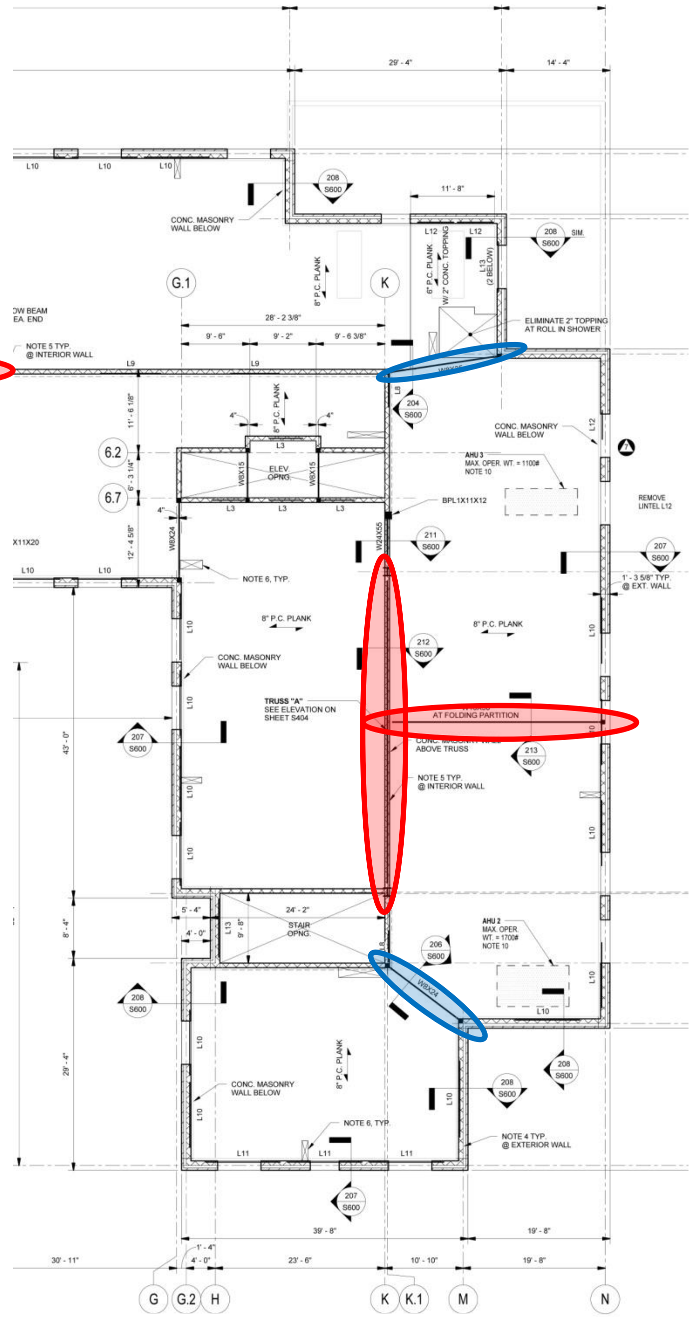
Figure 2: South Elevation

Exterior elevations are mainly comprised of brick veneer cavity wall system with rigid insulation and structural CMU backup along with cast stone window headers, some strips of aluminum, metal plates, caste stone, and polished block in a way to complement the modern look of the interior. The parapet wall also varies in height from 3 feet to 9 feet creating interesting snow and wind loadings on the roof that will be examined in the Building Load Summary section of the report on page 13. The roof is a typical TPO membrane roof system.

Structural System Overview

The Hyatt Place North Shore is a 7 story reinforced concrete masonry bearing structure located on soft soils along the Allegheny River that utilizes precast concrete planks for ease of construction and headroom. Steel beams are used to create an open space on the ground floor for a large meeting room and in other various places where the layout makes it impossible for the concrete planks to rest on the typical masonry bearing walls, shown in *Figure 3.* The reinforced concrete masonry bearing walls also serve as the lateral force resisting system with the aid of the precast concrete planks acting as a semi-rigid diaphragm.





-Typical on floors 2 through 7 for precast concrete planks to bear on

-Only on floor 2 to leave open space on floor 1

Figure 3: View of steel beams used

Foundation:

The Hyatt Place North Shore has a 15,500 SF footprint located on soil along the Allegheny River that has a maximum allowable bearing capacity of 1,500 psf. Spread footings have been provided for the front canopy, 5'-0"x5'-0"x1'-0" concrete spread footing with a maximum load of 25 kips, and site wall foundations only. There are 121 – 18" diameter end bearing 140 ton auger-cast piles that have a minimum depth of 1'-0" into bedrock to support the building. They have a 285 kip vertical capacity and a 16 kip lateral capacity. Piles are typically expected to be 70 feet deep, but this varies per pile. As shown in *Figure 4,* pile caps are 4'-0" thick. There are 2 to 4 piles supporting each pile cap. All concrete used for shallow foundations and piers have a strength of 3000 psi and the concrete for grade beams, pile caps, and slabs on grade are 4000 psi. The first floor is a 4" concrete slab on grade with W/ 6x6-W1.4xW1.4 welded wire fabric.

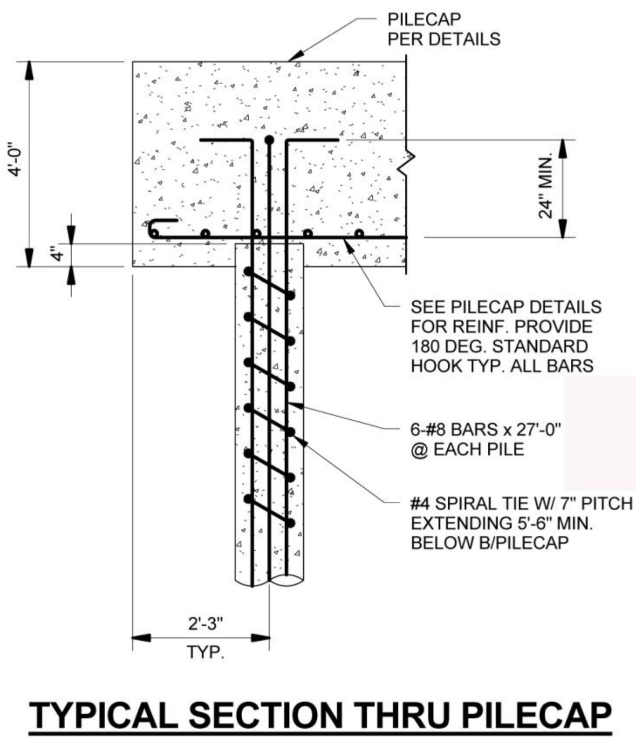


Figure 4: Section through typical pile cap

Gravity System

Walls:

Nearly all of the walls in the Hyatt Place North Shore are reinforced concrete masonry walls that resist gravity and lateral loads. The only exceptions are partition walls between the hotel rooms and other random walls not along the perimeter of the building. The walls vary in thickness and spacing of grout and reinforcing, *Table 1* shows the wall types and location. The compressive strength of the CMU units is 2800 psi and the bricks are 2500 psi, both normal weight. The grout used has a compressive strength of 3000 psi and the steel reinforcement is sized and placed as stated in *Table 1*. *Figure 5* shows the orientation of the walls on a typical upper level plan, the capacity of each of these wall types can be determined. *Table 2 & 3, and Figure 6* show the typical lintel in a masonry bearing wall.

Reinforced Concrete Masonry Bearing Wall Schedule								
						Weight (psf)		
Wall Type	Thickness	Rebar	Spacing	Grout	Floor Location	CMU & Grout	Rebar	Total
A	12"	#7	16" O.C.	All cells	1st ext.	140	1.53	141.53
B	12"	#7	32" O.C.	All cells	1st int. center	140	0.77	140.77
C	8"	#6	32" O.C.	All cells	1st int. random	92	0.56	92.56
D	8"	#6	24" O.C.	Cells w/reinforcement	2nd ext.	69	0.75	69.75
F	8"	#5	32" O.C.	All cells	2nd int. typ.	92	0.39	92.39
G	8"	#6	32" O.C.	16" O.C.	3rd - 5th ext.	75	0.56	75.56
H	8"	#6	32" O.C.	Cells w/reinforcement	5th - 7th ext.	65	0.56	65.56
I	8"	#5	32" O.C.	16" O.C.	3rd - 5th int.	75	0.39	75.39
J	8"	#5	32" O.C.	Cells w/reinforcement	5th - 7th int.	65	0.39	65.39

Table 1: Reinforced concrete masonry bearing wall schedule

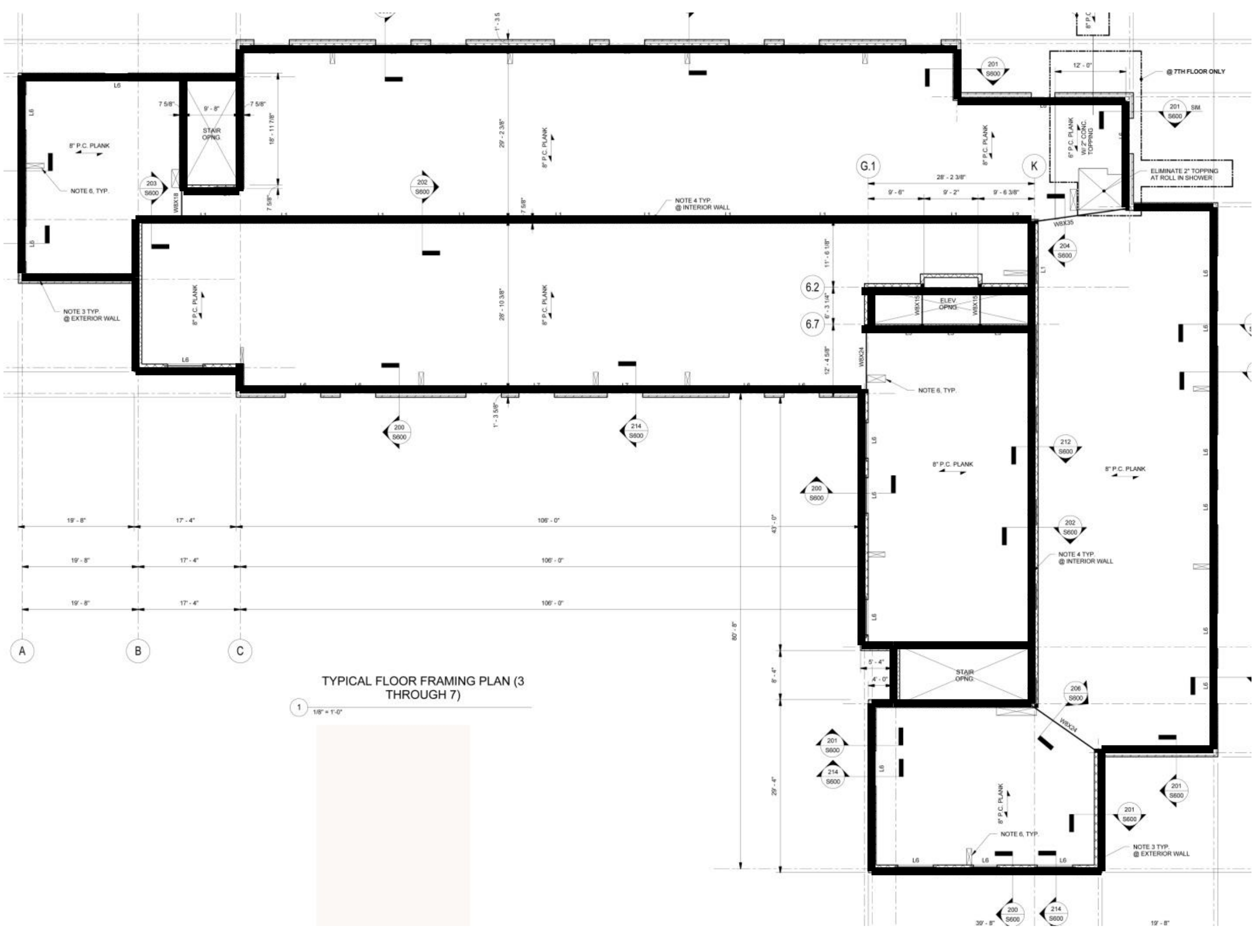


Figure 5: Typical bearing wall layout, floors 3 through 7

PRECAST LINTEL SCHEDULE F0R LOAD BEARING MASONRY WALLS						
			LOADING LBS/FT			
MARK	SIZE	MAX. M.O.	LIVE	DEAD	REMARKS	MARK
L1	8"	3'-4"	2000	1800	SEE "TYP. LINTEL DETAIL 1"	L1
L2	8'	6'-4"	2000	1800	SEE "TYP. LINTEL DETAIL 1"	L2
L3	10" VERIFY W/ELEV. MFR.	3'-6"	500	500	SEE "TYP. LINTEL DETAIL 1"	L3
L4	8"	6'-0"	1400	400	SEE "TYP. LINTEL DETAIL 2"	L4
L5	8"	6'-0"	1400	400	SEE "TYP. LINTEL DETAIL 4"	L5
L6	8"	6'-0"	1000	1000	SEE "TYP. LINTEL DETAIL 2"	L6
L7	8"	6'-0"	1000	1000	SEE "TYP. LINTEL DETAIL 4"	L7
L8	8"	6'-0"	1000	1000	SEE "TYP. LINTEL DETAIL 1"	L8
L9	8"	3'-4"	1000	1000	SEE "TYP. LINTEL DETAIL 1"	L9
L10	16"	6'-4"	2100	1000	SEE "TYP. LINTEL DETAIL 3"	L10
L11	16"	9'-4"	2100	1000	SEE "TYP. LINTEL DETAIL 3"	L11
L12	8"	5'-0"	1500	1000	SEE "TYP. LINTEL DETAIL 2"	L12
L13	16"	7'-0"	2600	1000	SEE "TYP. LINTEL DETAIL 2"	L13

PRECAST LINTEL FOR LOAD BEARING MASONRY WALLS NOTES:

1. MASONRY OPENINGS SHOWN IN SCHEDULE ARE MAXIMUM ALLOWED FOR LINTEL. SEE ARCH. DWGS. FOR ACTUAL MASONRY OPENINGS DIMENSIONS.
2. PROVIDE MIN. 8" BEARING ON BRICK OR SOLID CONC. BLOCK.
3. PRECAST LINTEL MFR. TO DESIGN PRECAST LINTELS FOR LOADS SHOWN IN SCHEDULE. SEE GENERAL NOTES FOR ADD'L INFO. LOADS ARE UNFACTORED.
4. SEE BRICK SUPPORT LINTEL SCHEDULE FOR ANGLE SIZE NEEDED FOR MASONRY OPENING.
5. LINTEL MUST BE DESIGNED FOR A MAXIMUM TOTAL LOAD DEFLECTION LESS THAN 0.3" OR SPAN/600.

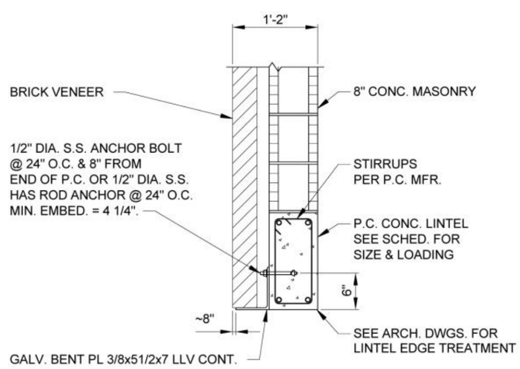
Table 2: Precast Lintel schedule for load bearing masonry walls

BRICK LINTEL SCHEDULE			
WALL THICKNESS	MASONRY OPNG. UP TO 4'-0"	MASONRY OPNG. 4'-0"+ TO 6'-0"	MASONRY OPNG. 6'-0"+ TO 8'-0"
4" WALL	BENT PL5/16x5 1/2x3 1/2 LLH	BENT PL5/16x5 1/2x4 LLH	BENT PL5/16x5 1/2x5 1/2

NOTES:

1. PROVIDE MINIMUM 6" BEARING ON BRICK.

Table 3: Brick lintel schedule



TYPICAL LINTEL DETAIL "3"

Figure 6: Typical lintel detail

Columns:

With the masonry structure, the only 2 columns in the building are W12x136s located on the first floor and are used to transfer the load in the large transfer girder down to the foundation, *Figure 7.* There are also concrete masonry piers on the first floor that support transfer beams in the lobby space and make it possible to have more window space on the first floor.

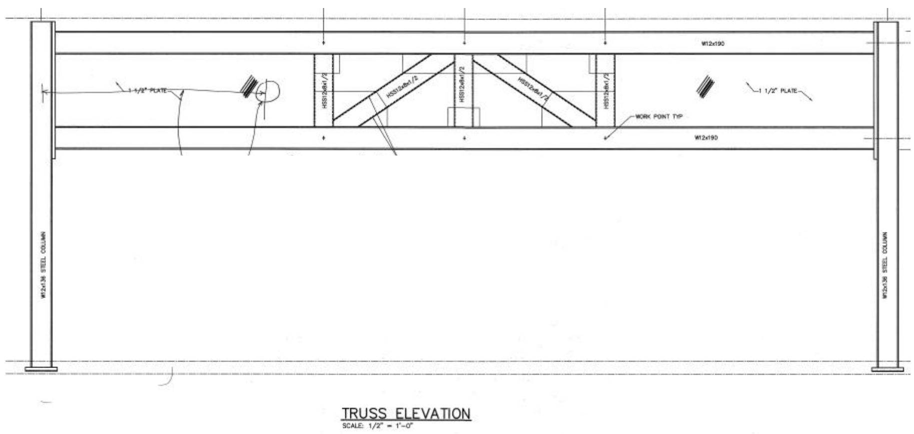


Figure 7: Transfer girder in first floor meeting space

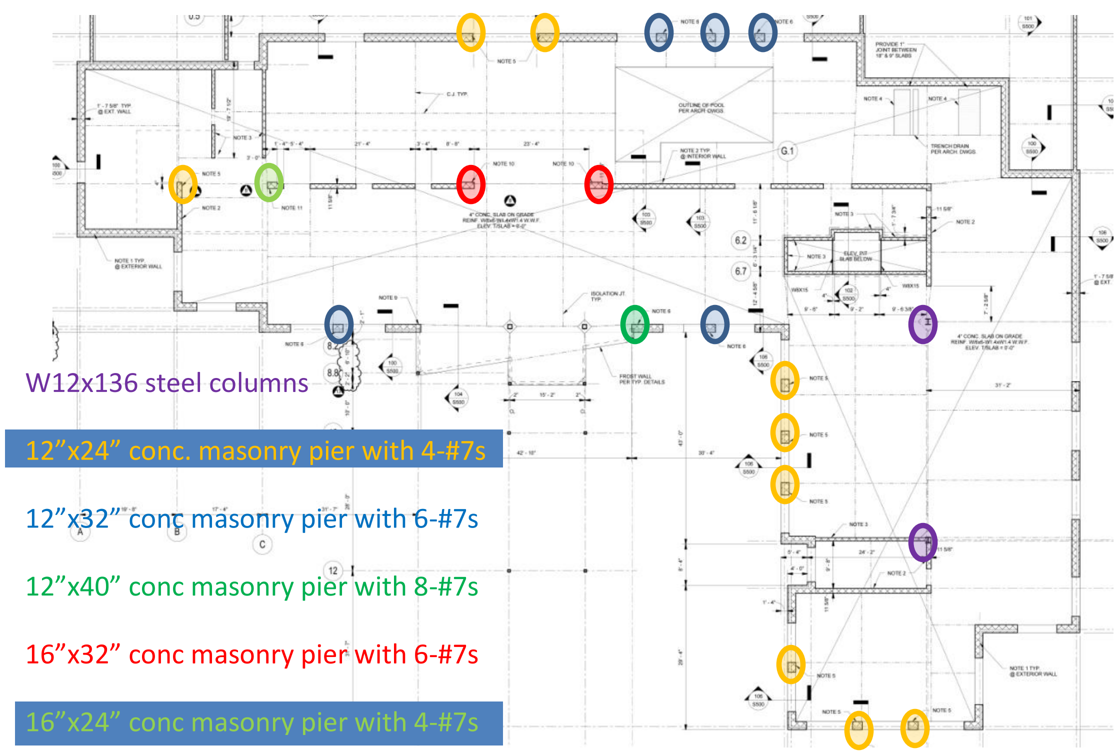


Figure 8: Location of masonry piers on first floor

Floors:

The Hyatt Place North Shore floor system is 8” thick untopped precast concrete planks. This system simplifies design and expedites construction. The system efficiently carries the loading over relatively long spans ranging from 27’-6” to 30’-6”. The concrete compressive strength of the floors is f’c=5000 psi. Extra strength is also added by prestressing the units. *Figure 12* shows a typical connection with masonry bearing walls.

The only exception to the typical concrete plank floor is on the first floor where this is a 4 inch concrete slab on grade, which was previously discussed on page 6 in the foundations section.

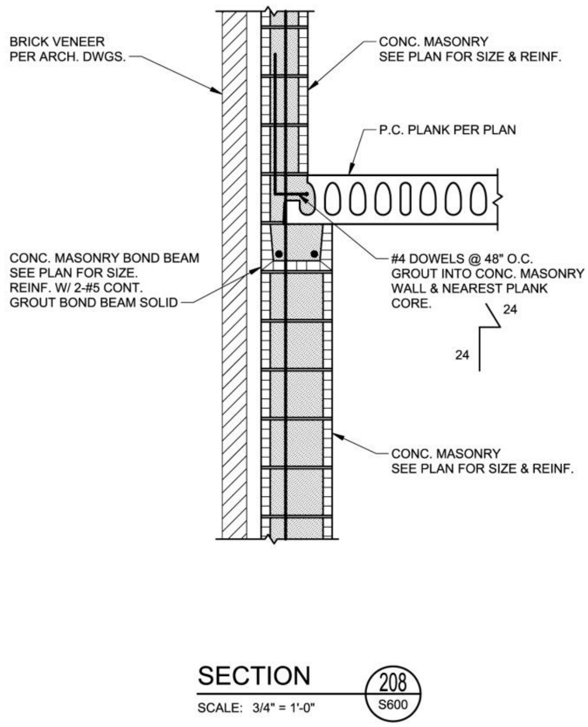


Figure 9: Typical plank and masonry wall connection

As previously stated on page 4 and denoted in *Figure 3*, steel beams are used in places where there is an opening in the interior bearing wall on the first floor and on all floors as needed for the planks to bear on. The members used are W8x18, W8x24, W8X35, W36x160, and W27x84. The large steel truss spanning 44’-4” over the meeting rooms 2 – W12x190s that are spaced 5’ apart with HSS members and 1 ½” steel plate webbing.

Lateral System

The lateral system for the structure is simply the gravity system. The reinforced masonry bearing walls depicted in *Figures 5 & 6* on page 7 act as shear walls and the precast concrete planks act as a semi-rigid diaphragm compared to cast-in-place concrete floor. The existing system only has a leveling material added, for planks to be considered fully rigid there must be a 2” structural concrete topping. The loads travel into the diaphragm and then into the bearing walls and down to the foundation and the auger piles that are capable of resisting 16 kips of lateral force per pile.

Codes and Design Standards

Codes:

The following references were used by the engineer of record at Atlantic Engineering Services to carry out the structural design of the Hyatt Place North Shore

- The International Building Code 2006 – Amendments City of Pittsburgh
- The Building Code Requirements for Structural Concrete (ACI 318-05), American Concrete Institute
- PCI MNL 120 "PCI Design Handbook – Precast and Prestressed Concrete"
- The Building Code Requirements for Masonry Structures (ACI 530), American Concrete Institute
- Specifications for Masonry Structures (ACI 530.1), American Concrete Institute
- Specifications for Structural Steel Buildings (ANSI/AISC 360-150), 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 & Structure, Inc.

Drift Criteria:

The following allowable drift criteria found in the International Building Code, 2006 edition.

- Allowable Building Drift: Δ_{wind} =H/400
- Allowable Story Drift: $\Delta_{seismic}$ = .015H_{sx}

Load Combinations:

The following load cases from ASCE 7-05 section 2.3 for factored loads using strength design; the greyed out portions don't apply in this case. These load combinations were considered in the ETABS model to determine the controlling case for the N/S and E/W directions.

- 1.4 (D + F) COMBO1
- 1.2 (D + F + T) + 1.6(L + H) + .5(L_r or S or R) COMBO2
- 1.2D + 1.6(L_r or S or R) + (L or .8W) COMBO3
- 1.2D + 1.6W + L + .5(L_r or S or R) COMBO4
- 1.2D + 1.0E + L + .2S COMBO5
- .9D + 1.6W + 1.6H COMBO6
- .9D + 1.0E + 1.6H **COMBO7 (controls for X & Y-Direction)**

Materials

Concrete:

Shallow Foundations and Piers	3000 psi
Grade Beams and Pile Caps	4000 psi
Slabs on Grade	4000 psi
Precast Concrete Planks	5000 psi

Rebar:

Deformed Bars Grade 60	ASTM A615
Welded Wire Fabric	ASTM A185

Masonry:

Concrete Masonry Units	2800 psi
Bricks	2500 psi
Grout	3000 psi

Structural Steel:

W Shapes	ASTM A992,	Fy = 50 ksi	Fu = 65 ksi
Channels	ASTM A572 Grade 50	Fy = 50 ksi	Fu = 65 ksi
Tubes (HSS Shapes)	ASTM 500 Grade B	Fy = 46 ksi	Fu = 58 ksi
Pipe (Round HSS)	ASTM 500 Grade B	Fy = 46 ksi	Fu = 58 ksi
Angles and Plates	ASTM A36	Fy = 36 ksi	Fu = 58 ksi

Gravity Loads

Load conditions determined from ASCE 7-05

Dead Loads:

Reinforced Concrete	150 pcf
Steel	490 pcf
Reinforced Masonry Walls	*Figure 5*
MEP	10 psf
Partitions	15 psf
Miscellaneous	5 psf
Roof	20 psf

Reinforced Concrete Masonry Bearing Wall Schedule								
						Weight (psf)		
Wall Type	Thickness	Rebar	Spacing	Grout	Floor Location	CMU & Grout	Rebar	Total
A	12"	#7	16" O.C.	All cells	1st ext.	140	1.53	141.53
B	12"	#7	32" O.C.	All cells	1st int. center	140	0.77	140.77
C	8"	#6	32" O.C.	All cells	1st int. random	92	0.56	92.56
D	8"	#6	24" O.C.	Cells w/reinforcement	2nd ext.	69	0.75	69.75
F	8"	#5	32" O.C.	All cells	2nd int. typ.	92	0.39	92.39
G	8"	#6	32" O.C.	16" O.C.	3rd - 5th ext.	75	0.56	75.56
H	8"	#6	32" O.C.	Cells w/reinforcement	5th - 7th ext.	65	0.56	65.56
I	8"	#5	32" O.C.	16" O.C.	3rd - 5th int.	75	0.39	75.39
J	8"	#5	32" O.C.	Cells w/reinforcement	5th - 7th int.	65	0.39	65.39

Table 1: Reinforced concrete masonry bearing wall schedule

Live Loads:

Floor Live Loads		
Area	Design Load (psf)	ASCE 7-05 Load (psf)
Public Areas	100	100
Lobbies	100	100
Public Corridors	100	100
Room Corridors	60	40
Hotel Rooms	60	40
Stairs	100	100
Mechanical*	150	125
Fitness Room	100	100
*on grade		

Table 4: Floor live loads

Snow Load:

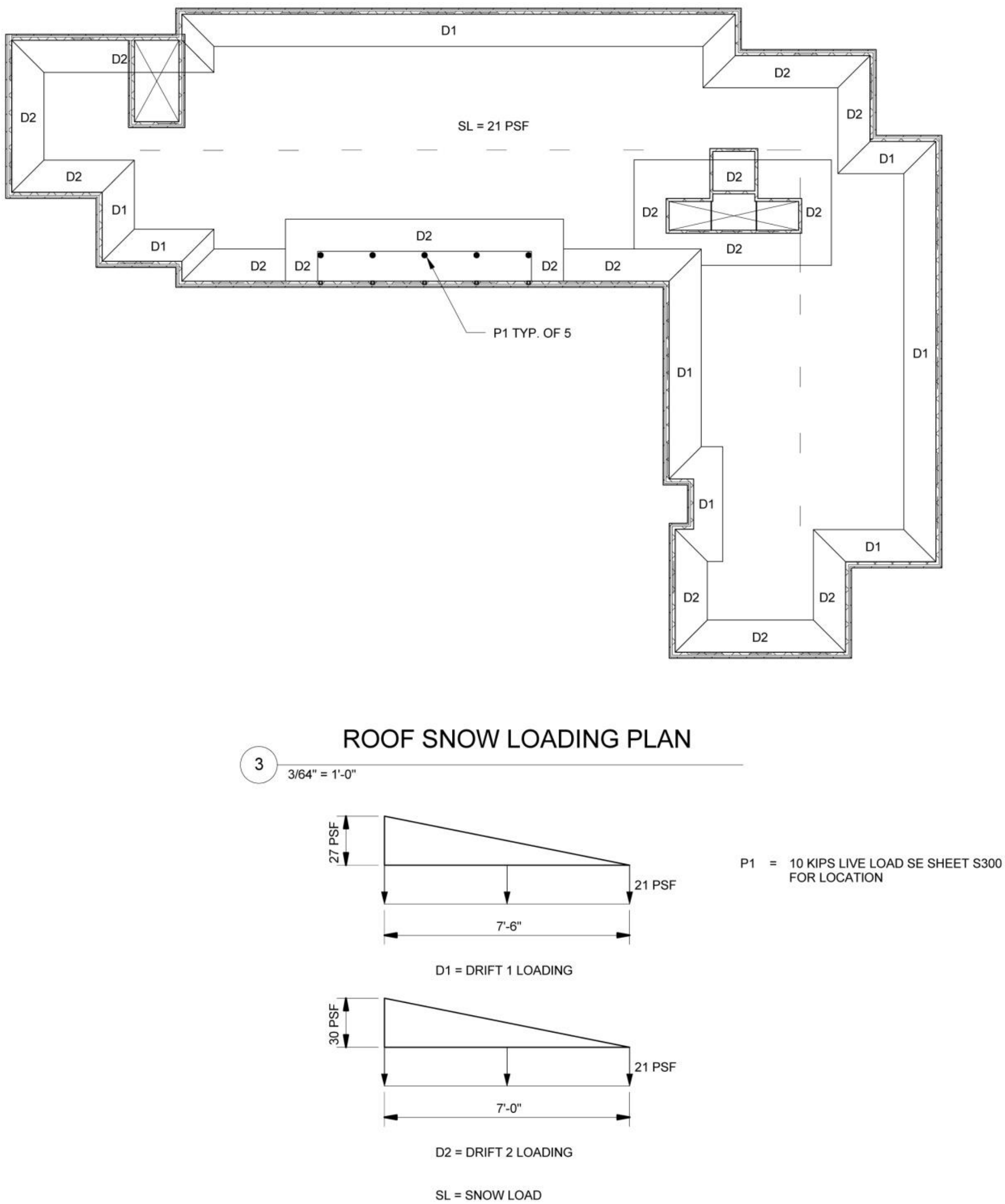


Figure 10: Roof snow loading plan as calculated by AES

Flat Roof Snow Load:

Determined using ASCE 7-05

Flat Roof Snow Load				
		AES	ASCE 7-05	
Ground Snow Load	P_g=	30	25	psf
Snow Exposure Factor	C_e=	1.0		
Snow Load Importance Factor	I_s=	1.0		
Thermal Factor	C_t=	1.0		
Flat Roof Snow Load	P_f=	21	17.5	psf

Table 5: Calculation of flat roof snow load

The roof system uses the same 8" precast concrete planks as the lower levels of the structure, therefore the roof is significantly overdesigned and can handle a much greater snow load than the tabulated value.

Drift Calculation:

Calculation of drift depth from *figure 16*

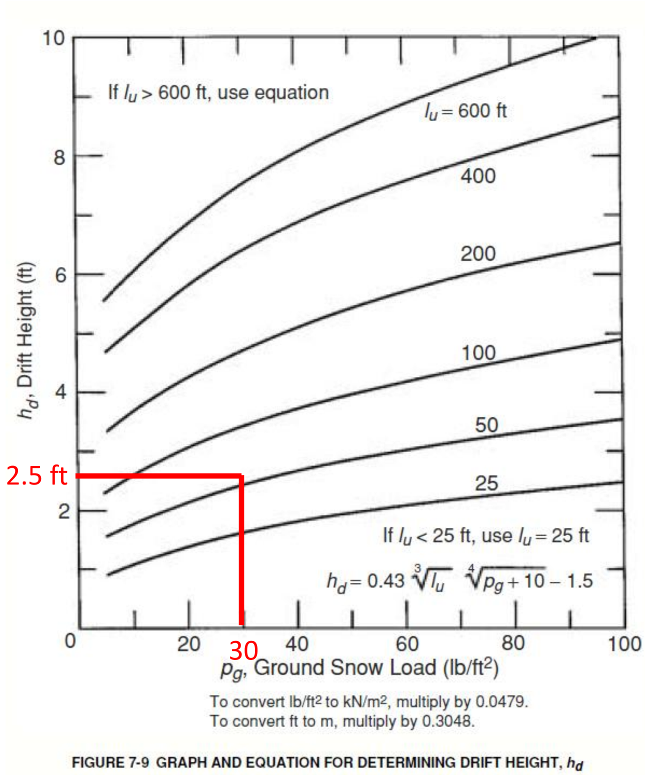


Figure 11: Graph and equation for determining drift height

Snow Density = .13(P_g) + 14

= .13(25) + 14 = **17.25 lb/ft^3**

Balanced Height= P_g/Snow Density = 25/17.25 = **1.4 ft**

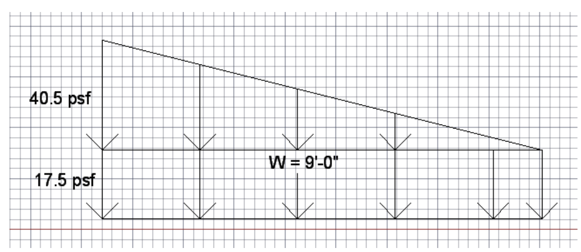
<u>Typical Parapet Wall Drift Height</u>

Drift Height = 2.5ft – from *Figure 16*

Max allowable = .75 h_d = .75*2.5 = **2.25ft**

Drift Weight = 2.25ft * 18 lb/ft^3 = **40.5 psf**

Drift Width = 4*h_d = 4*2.25 = **9 ft**



Wind Loading

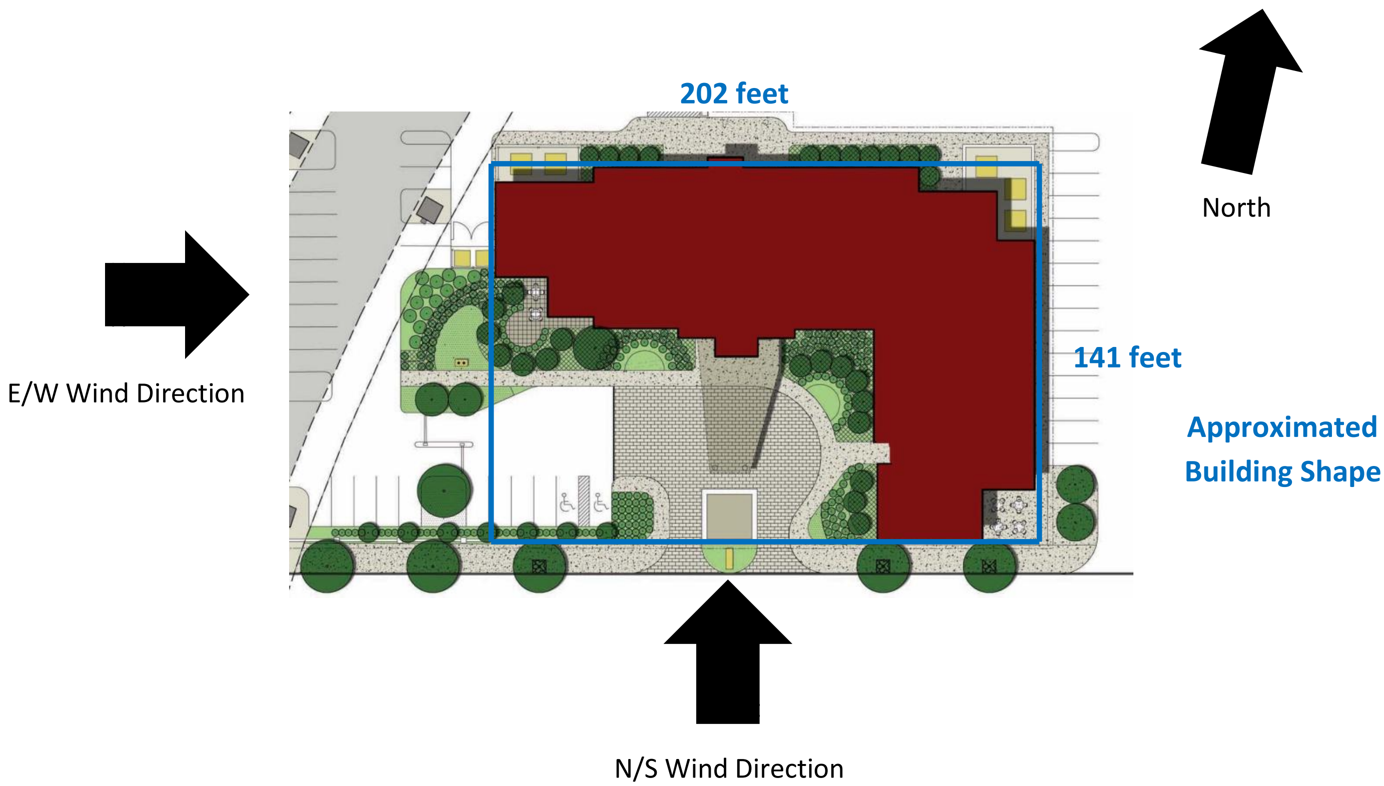


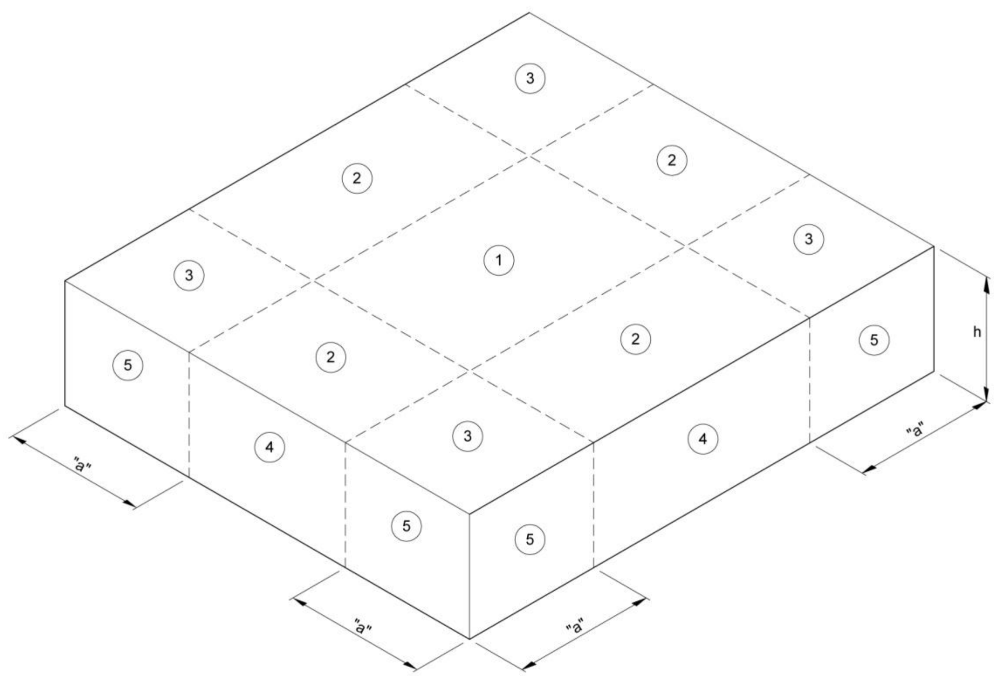
Figure 12: Wind load on building façade

Loads to be applied to the hotel's lateral system must also be determined, so that in later reports the lateral system can be studied. With the system provided, wind applies pressure to the building enclosure. The exterior walls are load bearing and begin the transfer of energy down through toward the foundation. Walls parallel to the wind direction resist the wind more efficiently than the ones perpendicular to the force. The precast concrete planks tie the wall system together and make it work as a rigid unit that directs the load down into the 141 – 18" auger piles that can resist 16 kips of lateral force each. The appropriate wind pressures to be applied to the building facade were determined from ASCE 7-05, Chapter 6. Given the height of the building, it is appropriate to use Method 2. *Figure 13* shows that Atlantic Engineering Services used Method 1 to calculate the component and cladding wind pressures, but ASCE 7-05 it states that once a building is over 60 feet tall a more complicated calculation must be done to account for the different pressures at different heights along the elevation. A clear notation of the calculations done to complete Method 2 is located in Appendix A, and a summary of the important values is found in *Table 6*. In the process of determining the wind forces the building was approximated to be rectangular in order to greatly simplify the calculations and still obtain an accurate value. If the structure was split into two parts, the wind would hit the same surface area in the end.

COMPONENT AND CLADDING WIND PRESSURES					
TRIBUTARY AREA (SF)	ZONE				
	1	2	3	4	5
10				18.6/-22.8	18.6/-29.0
20				17.6/-21.7	17.6/-26.8
50		NOTE 2		16.1/-20.2	16.1/-23.9
100				15.0/-19.9	15.0/-21.7
500				12.4/-16.6	12.4/-16.6

NOTES:

1. ALL LOADS ARE IN POUNDS PER SQUARE FOOT (PSF).
2. SEE ROOF WIND LOADING DIAGRAM ON SHEET S403 FOR UPLIFT PRESSURES.
3. + DENOTES PRESSURE
   - DENOTES SUCTION



ROOF AND WALL ZONES

Figure 13: Wind loads calculated by AES using Method 1 in ASCE 7-05

Wind Design Variables			
			ASCE Reference
Basic Wind Speed	V	90	Fig. 6-1
Wind Importance Factor	I	1.0	Table 6-1
Exposure Category		C	Sec 6.5.6.3
Directionality Factor	K_d	0.85	Table 6-4
Topographic Factor	K_{zt}	1.0	Sec 6.5.7.1
Velocity Pressure Exposure Coeficient Evaluated at Height Z	K_z	Varies (see appendix)	Table 6-3
Velocity Pressure at Height Z	q_z	Varies (see appendix)	Eq. 6-15
Velocity Pressure at Mean Roof Height	q_h	20.97	Eq. 6-15
Equivalent Height of Structure	>	48 ft	Table 6-2
Intensity of Turbulence	I_z	0.19	Eq. 6-5
Integral Length Scale of Turbulence	L_z	538.91	Eq. 6-7
Background Response Factor (East/West)	Q	0.26	Eq. 6-6
Background Response Factor (North/South)	Q	0.23	Eq. 6-7
Gust Effect Factor	G	.85 (assumed masonry was rigid)	Eq. 6-4
Internal Pressure Coeficient	GC_{pi}	.18 (enclosed building)	Fig. 6-5
External Pressure Coeficient (Windward)	C_p	0.8	Fig. 6-6
External Pressure Coeficient (N/S Leeward)	C_p	-0.5	Fig. 6-6
External Pressure Coeficient (E/W Leeward)	C_p	-0.414	Fig. 6-6
External Pressure Coeficient (Side)	C_p	-0.7	Fig. 6-6

Table 6: Wind design variables

The calculation of wind pressures and forces on the structure were done in excel for both the East/West and North/South in *Figure 19* and *20* respectively. The controlling wind direction was determined to be the North/South wind direction, due to its larger surface area. *Figure 21, 22, and 23* show how the forces from the North/South wind direction are applied to the building.

Wind Loads in the East/West Direction														
L=202' B=141' L/B = 1.43														
Level	Height Above Ground (z) (ft.)	Story Height (ft.)	K_z	q_z	q_h	Wind Pressure (psf) G = .85		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)
					h = 75 ft	Windward	Leeward							
					K_z = 1.19	C_p = .8	C_p = -.414							
PH Roof	80	10	1.2	21.15	20.97	**17.59**	**-10.59**	28.18	12.40	**19.87**	12.40	**19.87**	992.17	1589.44
Main Roof	70	10	1.17	20.62	20.97	**17.23**	**-10.59**	27.82	22.67	**36.60**	35.07	**56.47**	1586.85	2562.05
7	61.33	8.66	1.13	19.92	20.97	**16.75**	**-10.59**	27.34	20.46	**33.39**	55.53	**89.86**	1254.56	2047.62
6	52.66	8.66	1.1	19.39	20.97	**16.39**	**-10.59**	26.98	20.02	**32.95**	75.54	**122.80**	1054.09	1735.04
5	44	8.66	1.06	18.68	20.97	**15.91**	**-10.59**	26.50	19.43	**32.36**	94.98	**155.17**	854.99	1423.95
4	35.33	8.66	1.01	17.80	20.97	**15.31**	**-10.59**	25.90	18.70	**31.63**	113.68	**186.80**	660.66	1117.52
3	26.66	8.66	0.95	16.74	20.97	**14.60**	**-10.59**	25.19	17.82	**30.75**	131.50	**217.55**	475.13	819.87
2	18	18	0.88	15.51	20.97	**13.76**	**-10.59**	24.35	25.86	**45.76**	157.35	**263.31**	465.40	823.67
1	0	0	0	0.00	0.00	**0.00**	**0.00**	0.00	0.00	**0**	157.35	**263.31**	0.00	0.00
										Windward Base Shear=			**157.35**	**Kips**
										Total Base Shear=			**263.31**	**Kips**
										Sum of Windward Moment=			**7343.83**	**ft-k**
										Sum of Total Moment=			**12119.16**	**ft-k**

Table 7: Wind loads in the East/West direction

Wind Loads in the North/South Direction														
L=141' B=202' L/B = .7														
Level	Height Above Ground (z) (ft.)	Story Height (ft.)	K_z	q_z	q_h	Wind Pressure (psf) G = .85 Gcpi = +.18 -.18		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)
					h = 75 ft	Windward	Leeward							
					K_z = 1.19	C_p = .8	C_p = -.5							
PH Roof	80	10	1.2	21.15	20.97	**17.59**	**-12.12**	29.71	17.77	**30.01**	17.77	**30.01**	1421.40	2400.96
Main Roof	70	10	1.17	20.62	20.97	**17.23**	**-12.12**	29.36	32.48	**55.32**	50.24	**85.34**	2273.35	3872.73
7	61.33	8.66	1.13	19.92	20.97	**16.75**	**-12.12**	28.88	29.31	**50.51**	79.55	**135.85**	1797.32	3097.97
6	52.66	8.66	1.1	19.39	20.97	**16.39**	**-12.12**	28.52	28.68	**49.88**	108.23	**185.73**	1510.11	2626.90
5	44	8.66	1.06	18.68	20.97	**15.91**	**-12.12**	28.04	27.84	**49.05**	136.06	**234.78**	1224.87	2158.00
4	35.33	8.66	1.01	17.80	20.97	**15.31**	**-12.12**	27.44	26.79	**48.00**	162.85	**282.78**	946.48	1695.74
3	26.66	8.66	0.95	16.74	20.97	**14.60**	**-12.12**	26.72	25.53	**46.74**	188.39	**329.52**	680.68	1246.07
2	18	18	0.88	15.51	20.97	**13.76**	**-12.12**	25.88	37.04	**69.68**	225.43	**399.20**	666.74	1254.32
1	0	0	0	0.00	0.00	**0.00**	**0.00**	0.00	0.00	**0**	225.43	**399.20**	0.00	0.00
										Windward Base Shear=		**225.43**	**Kips**	
										Total Base Shear=		**399.20**	**Kips**	
										Sum of Windward Moment=		**10520.95**	**ft-k**	
										Sum of Total Moment=		**18352.68**	**ft-k**	

Table 8: Wind loads in the North/South direction

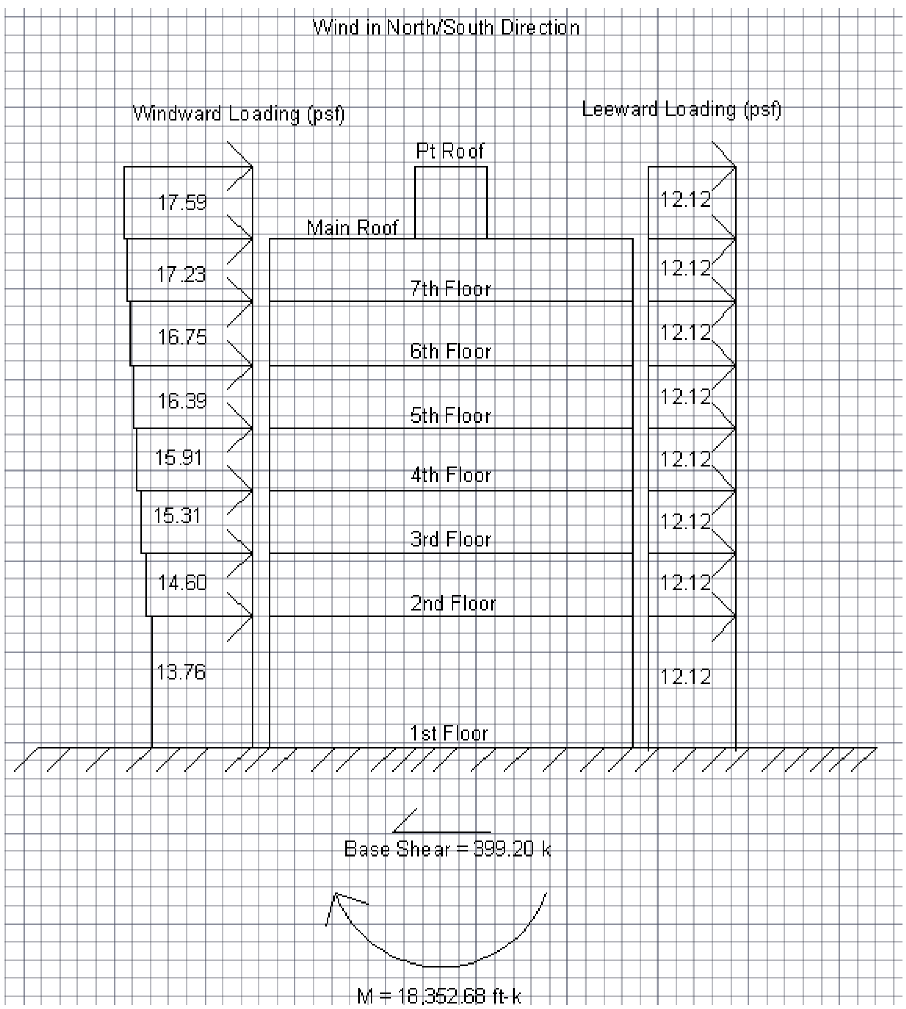


Figure 14: Wind pressures in the North/South direction

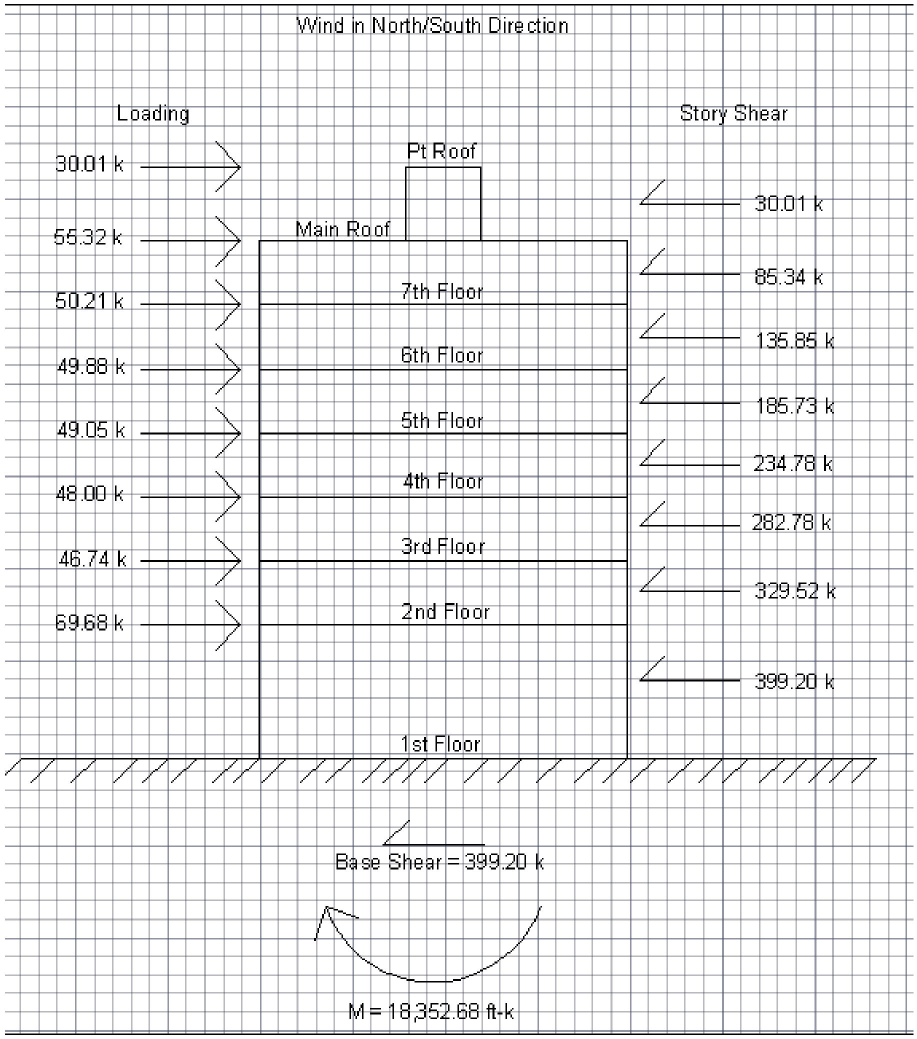


Figure 15: Wind loads in the North/South direction

Seismic Loading

Seismic loading must also be taken into consideration when checking the lateral system. In this case the seismic loading appears to control, but it may depend on the load case. Thus all load cases will be looked at when designing the lateral force resisting systems. The values in *Table 9* were obtained from ASCE 7-05, chapters 11 and 12. Calculations of the variables using the listed equations can be found in Appendix B along with building weights per floor. The variables used can be used to find the total base shear of 537.18 kips.

Seismic Design Variables			
			ASCE Reference
Soil Classification		D (stiff soil)	Table 20.3-1
Occupancy Category		II	Table 1-1
Seismic Force Resisting System		Intermediate Reinforced Masonry Shear Walls	Table 12.2-1
Response Modification Factor	R	3.5	Table 12.2-2
Seismic Importance Factor		1.0	Table 11.5-1
Spectral Response Acceleration, Short	S_s	0.125	USGS Website
Spectral Response Acceleration, 1 sec.	S_1	0.049	USGS Website
Site Coeficient	F_a	1.6	Table 11.4-1
Site Coeficient	F_v	2.4	Table 11.4-2
MCE Spectral Response Acceleraton, Short	S_{MS}	0.2	Eq. 11.4-1
MCE Spectral Response Acceleration, 1 sec	S_{M1}	0.1176	Eq. 11.4-2
Design Spectral Acceleration, Short	S_{DS}	0.13	Eq. 11.4-3
Design Spectral Acceleration, 1 sec.	S_{D1}	0.0784	Eq. 11.4-4
Approximate Period Parameter	C_t	.02 (all other systems)	Table 12.8-2
Approximate Period Parameter	x	.75 (all other systems)	Table 12.8-2
Building Height	h_n	80'-0"	
Approximate Fundamental Period	T_a	0.53 sec.	Eq. 12.8-7
Long Period Transition Period	T_L	5 sec.	Fig. 22-15
Seismic Response Coeficient	C_s	0.037	Eq. 12.8-2
Structure Period Exponent	k	1.015 (2.5 sec. > T > .5 sec.)	Sec 12.8.3
Seismic Base Shear	**V (kips)**	**537.18**	**Eq. 12.8-1**

Table 9: Seismic design variables

The next step is to distribute the forces to each level to find the story shear values and overturning moments. This was done using an excel spreadsheet shown in *Table 10,* and *Figure 16* shows how the loads are applied to the building.

Seismic Story Shear and Moment Calculations								
Level	Story Weight (K)	Height (ft)	K	$w_x h_x^k$	Vertical Distribution Factor C_{vx}	Forces (K) Fx	Story Shear (K) Vx	Moments (ft-K) Mx
Penthouse Roof	48.5	80	1.015	4140.18	0.01	3.47	3.47	277.48
Main Roof	1665.8	70	1.015	124275.59	0.19	104.11	107.58	7530.85
7th Floor	1955.5	61.33	1.015	127567.60	0.20	106.87	214.46	13152.61
6th Floor	1955.5	52.66	1.015	109283.70	0.17	91.56	306.01	16114.57
5th Floor	1956.3	44	1.015	91103.06	0.14	76.32	382.34	16822.76
4th Floor	1957.1	35.33	1.015	72940.79	0.11	61.11	443.44	15666.86
3rd Floor	1985.2	26.66	1.015	55597.21	0.09	46.58	490.02	13063.97
2nd Floor	2994.6	18	1.015	56290.31	0.09	47.16	537.18	9669.24
Total	**14518.25**			**641198.45**				**92298.34**

Table 10: Seismic story shear and moment calculations

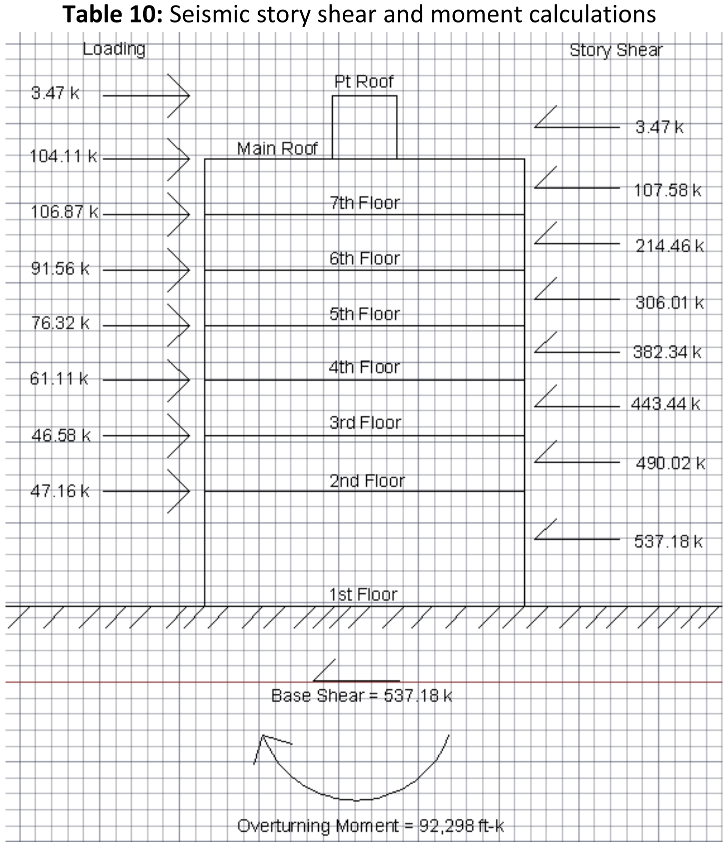


Figure 16: Seismic loading diagram

Load Distribution

Load Path

Both wind and seismic loads must be resisted by intermediate reinforced masonry shear walls and funneled down to the foundation of the building. The wind loads originate on the building façade and then the exterior walls and diaphragm distribute the forces to shear walls oriented parallel to the direction of the force. In this case the untoped precast concrete plank floor system is considered to be semi-rigid diaphragm; this means that forces are distributed to shear walls based on tributary area rather than relative rigidity. Once the forces are in the shear walls, they then travel down the walls to the foundation. Seismic forces are distributed similarly, except that they originate in the mass of the structure and then travel to the shear walls and down to the foundation. There is a difference in layout of shear walls on the ground floor and of the ones above it. The first floor has more windows and less shear wall, therefore more force has to be taken by each shear wall and the connection between shear wall or pier and diaphragm is more important. Layout of shear walls on the ground story and the typical upper stories are shown in Figure 17 and 18 respectively.

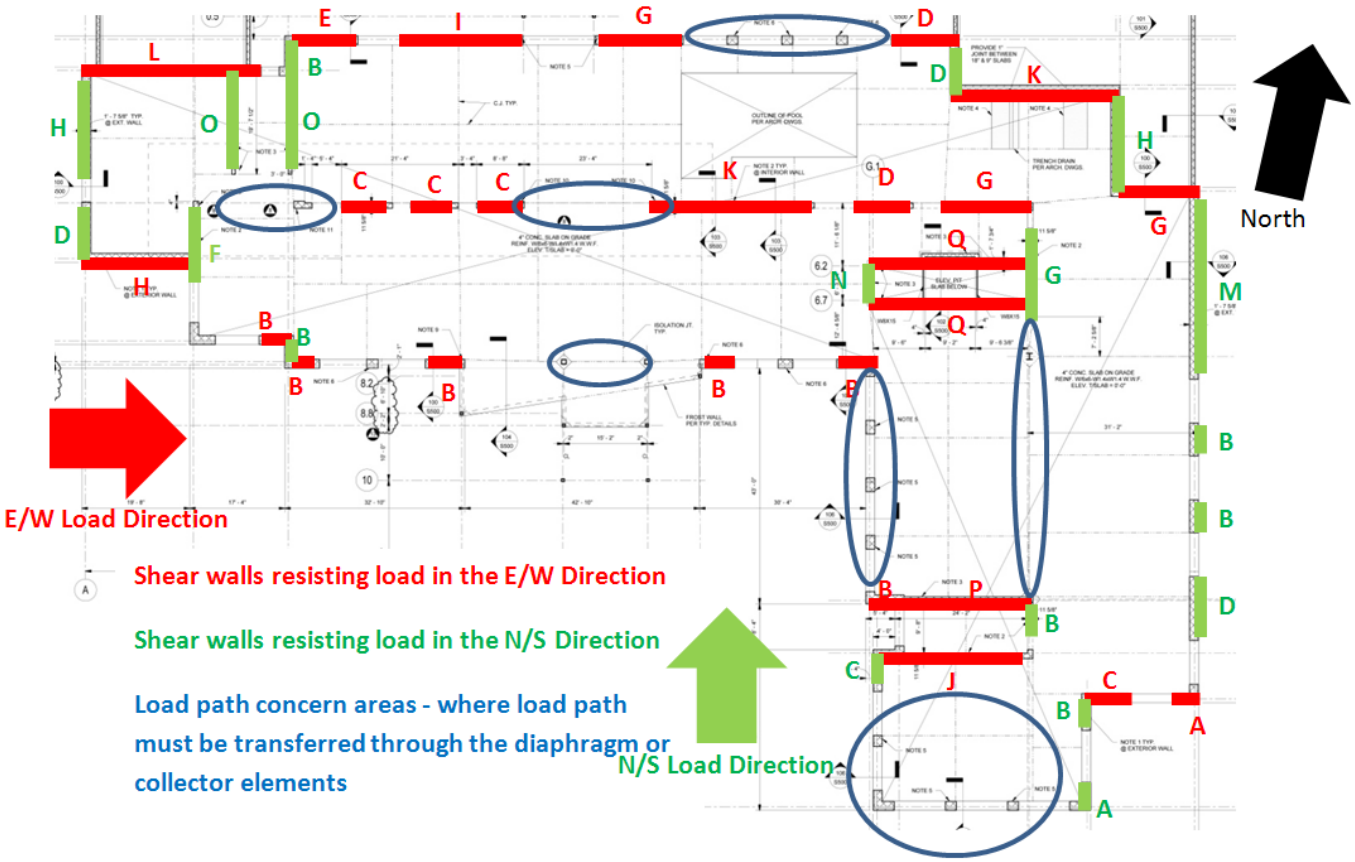


Figure 17: Shear walls on ground story

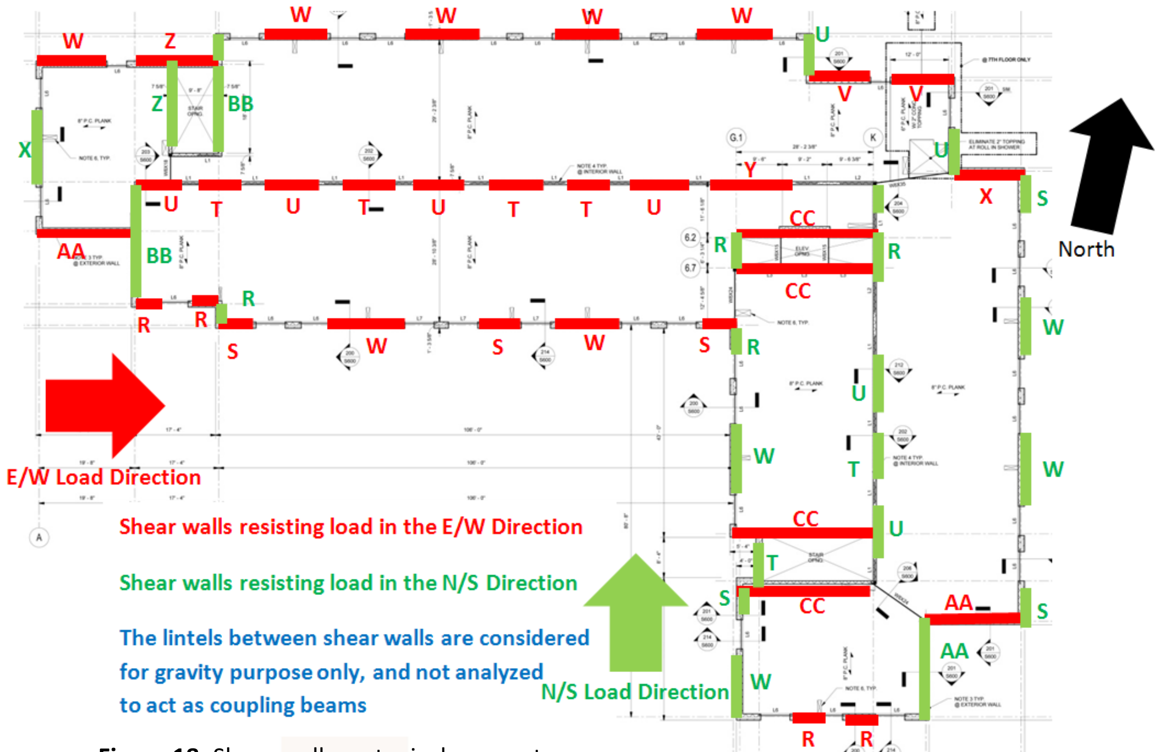


Figure 18: Shear walls on typical upper story

The layouts shown in *Figure 17 & 18* show most all of the walls that are large enough to be shear walls. In this analysis I made a few assumptions in order to simplify the layout of the walls for computer modeling and hand calculations. Load follows stiffness in the building, so I determined the most important shear walls to have in the analysis are ones that provide the most direct path for the load from the roof to the foundation. I also ignored the door holes in some shear walls, assuming things would even out because of the elimination of some other walls. In addition, reinforced masonry shear walls are often capable of holding much more lateral and shear load than needed. The final layout of shear walls I determined to analyze for adequacy are depicted in *Figure 19*. *Table 11* shows the grid coordinates used to analyze the structure. *Figure 20* shows the grid used in ETABS to layout the shear walls shown in *Figure 19*.

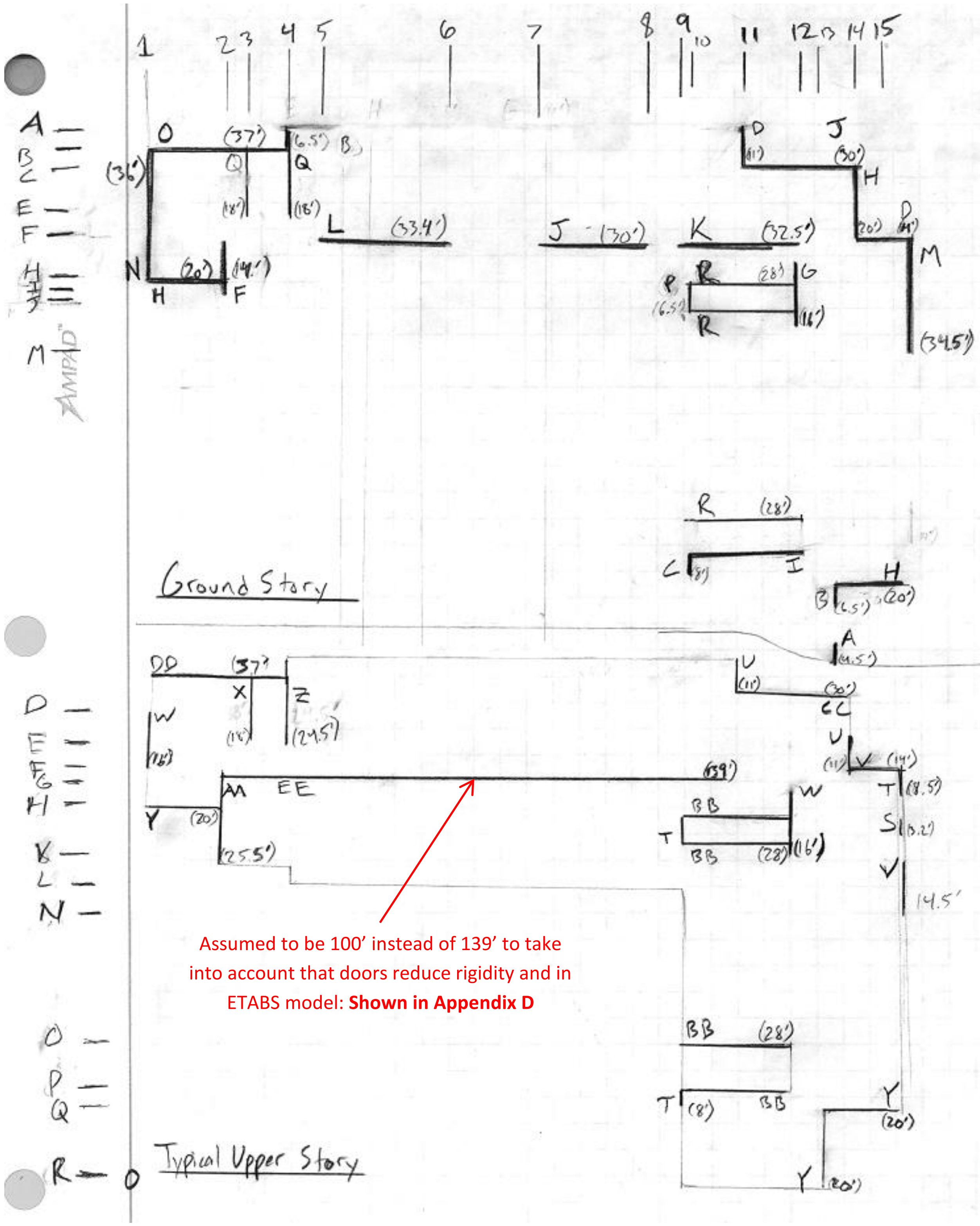


Figure 19: Simplified shear wall layout

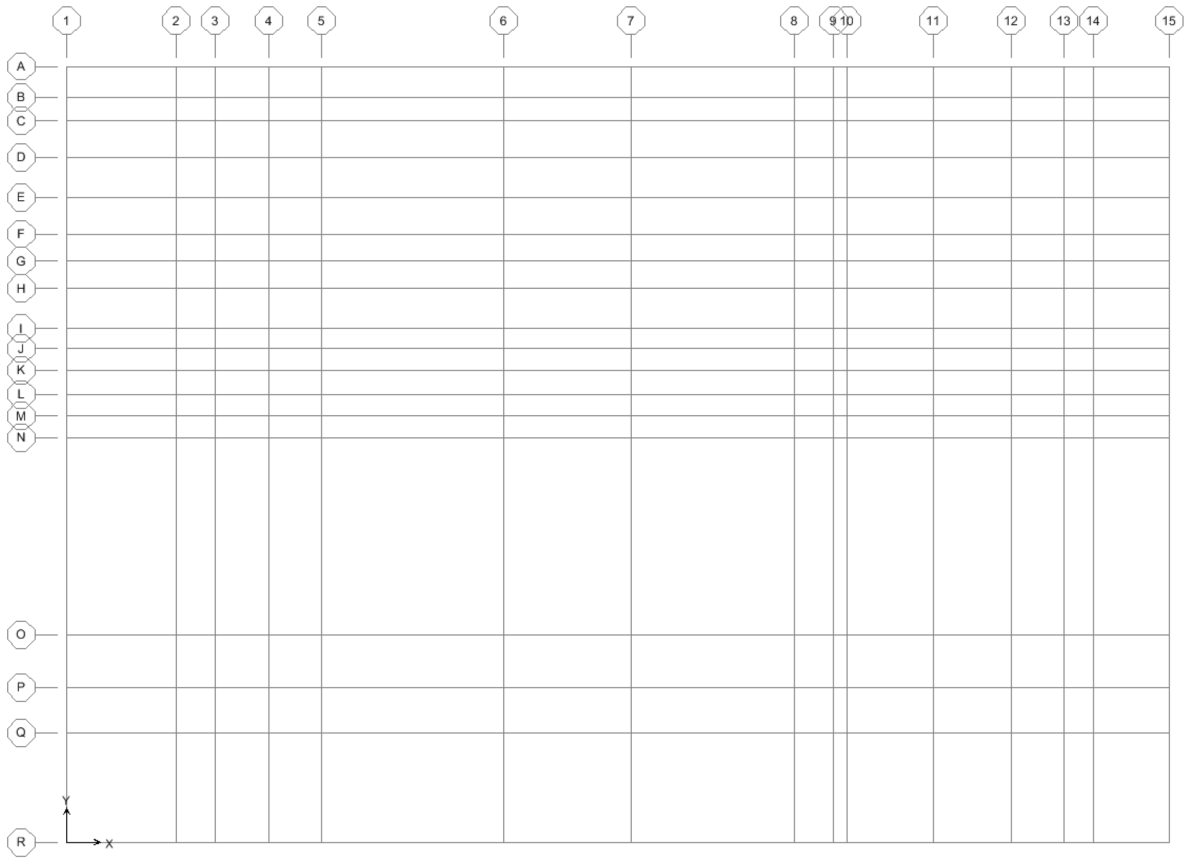


Figure 20: Grid lines

X		Y	
1	0.0	A	142.0
2	20.0	B	136.3
3	27.2	C	132.0
4	37.0	D	125.3
5	46.7	E	118.0
6	80.0	F	111.3
7	103.3	G	106.3
8	133.3	H	101.3
9	140.5	I	94.0
10	143.0	J	90.3
11	158.7	K	86.3
12	173.0	L	82.0
13	182.7	M	78.0
14	188.0	N	74.0
15	202.0	O	38.0
		P	28.3
		Q	20.0
		R	0.0

Table 11: Grid coordinates

ETABS Model

An ETABS model was developed to aid in the analysis of the structure for lateral loads, therefore the only components modeled were ones in the lateral force resisting system. Almost all of the structural components in the Hyatt Place North Shore resist lateral load, but not all were modeled. The structure consists of a large amount of reinforced masonry shear walls and untoped precast concrete planks. All of the precast concrete planks floors were modeled as semi-rigid diaphragms and had an area mass applied to each. The area mass was determined by finding the floor and wall weight for that floor and averaging it over the floor area and converting it to area mass by dividing by 32.2 and 12^3, the results are listed in *Table 12*. Because the area mass takes account for the dead load of the floors and walls, the mass of the concrete material was changed to "0" so that the mass was not double counted. As previously discussed on page 25, not all of the shear walls were modeled. All of the modeled shear walls were moment connected at the base to most accurately model the behavior of the connection to the deep foundations. Once the structure is modeled, then story forces due to wind and seismic load were applied using ASCE 7-05 load combinations for strength design in North/South and East/West directions. *Table 13* and *14* show the wind and seismic loads respectively. The center of mass, center of rigidity, story displacements, overall building drift, and controlling load cases can be determined from the analysis. *Figure 21* shows overall 3D view of the shear walls and floor diaphragms modeled, elevations views can be found in the appendix.

Building Weight			
Floor	Weight (kips)	Weight (klf)	Area Mass
1	2994.55	0.193	3.472E-06
2	1985.20	0.128	2.302E-06
3	1957.06	0.126	2.269E-06
4	1956.27	0.126	2.268E-06
5	1955.48	0.126	2.267E-06
6	1955.48	0.126	2.267E-06
7	1665.76	0.107	1.931E-06

Table 12: Area mass

Wind Load				
	X-Direction		Y-Direction	
Story	Force (k)	Moment (k-in)	Force (k)	Moment (k-in)
STORY7	36.6	30744.6	55.32	46472.76
STORY6	33.39	24571.44	50.51	37175.64
STORY5	32.95	20820.48	49.88	31522.8
STORY4	32.36	17087.4	49.05	25896
STORY3	31.63	13410.24	48	20348.88
STORY2	30.75	9838.44	46.74	14952.84
STORY1	45.76	9884.04	69.68	15051.84

Table 13: Wind story forces

Seismic Load		
Story	Force	Moment
STORY7	198.74	172503.7405
STORY6	204.01	301277.5842
STORY5	174.77	369125.0137
STORY4	145.69	385347.0081
STORY3	116.65	358869.5333
STORY2	88.91	299247.0672
STORY1	90.02	221486.4

Table 14: Seismic story forces

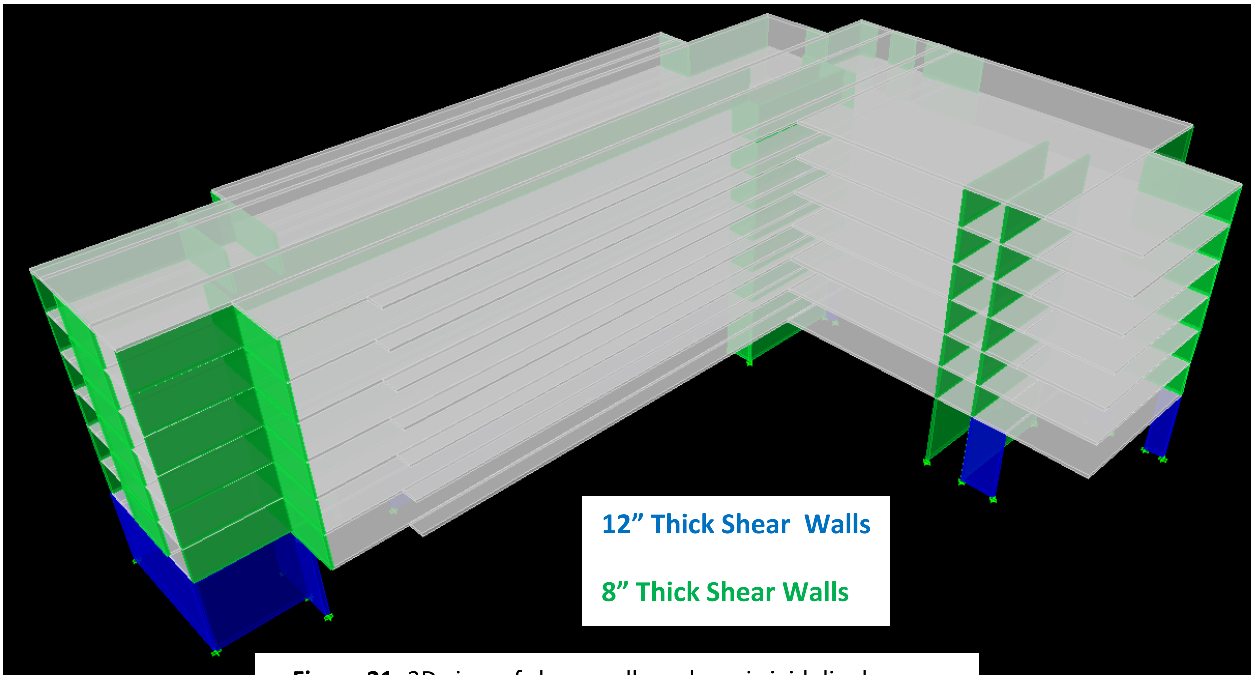


Figure 21: 3D view of shear walls and semi-rigid diaphragms

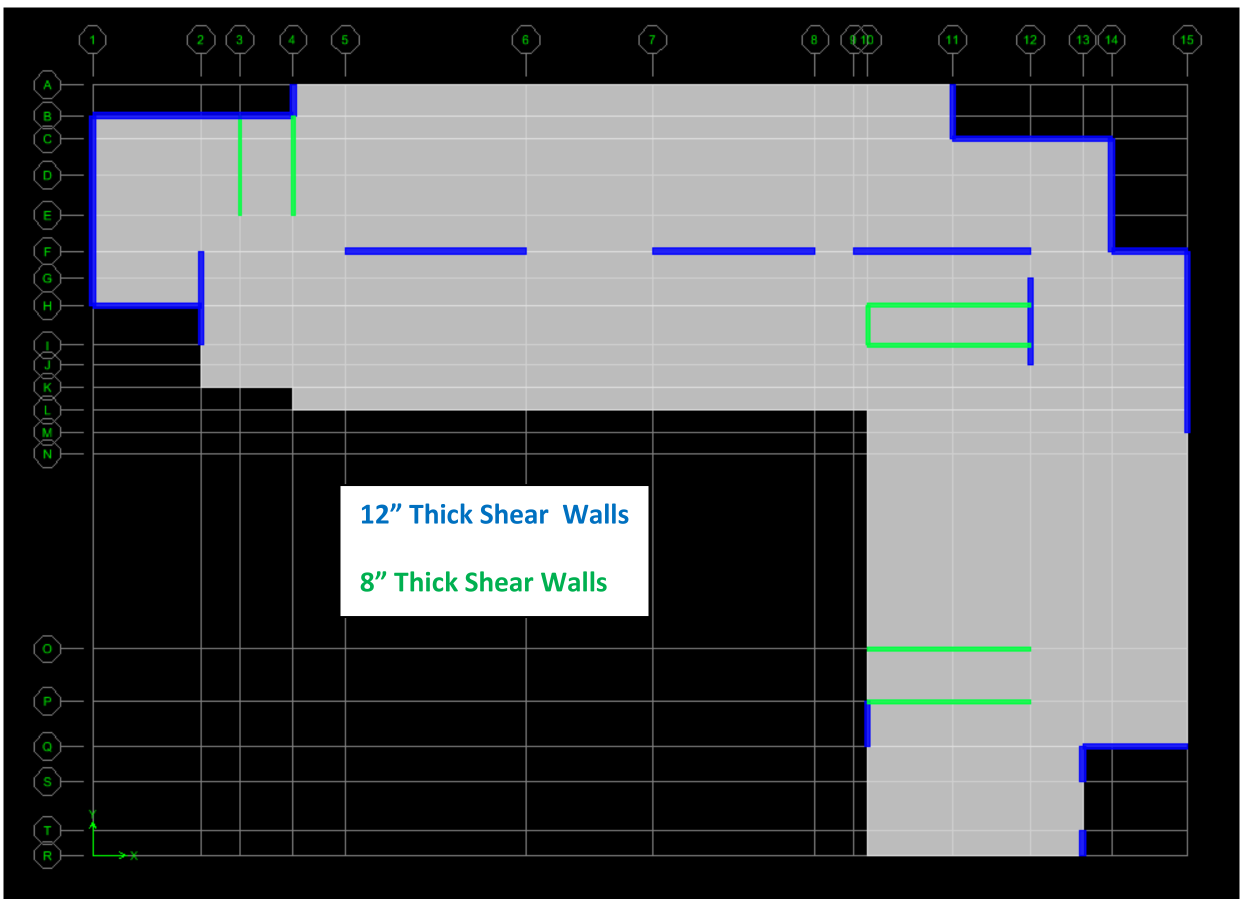


Figure 21: Plan view of ground story shear walls

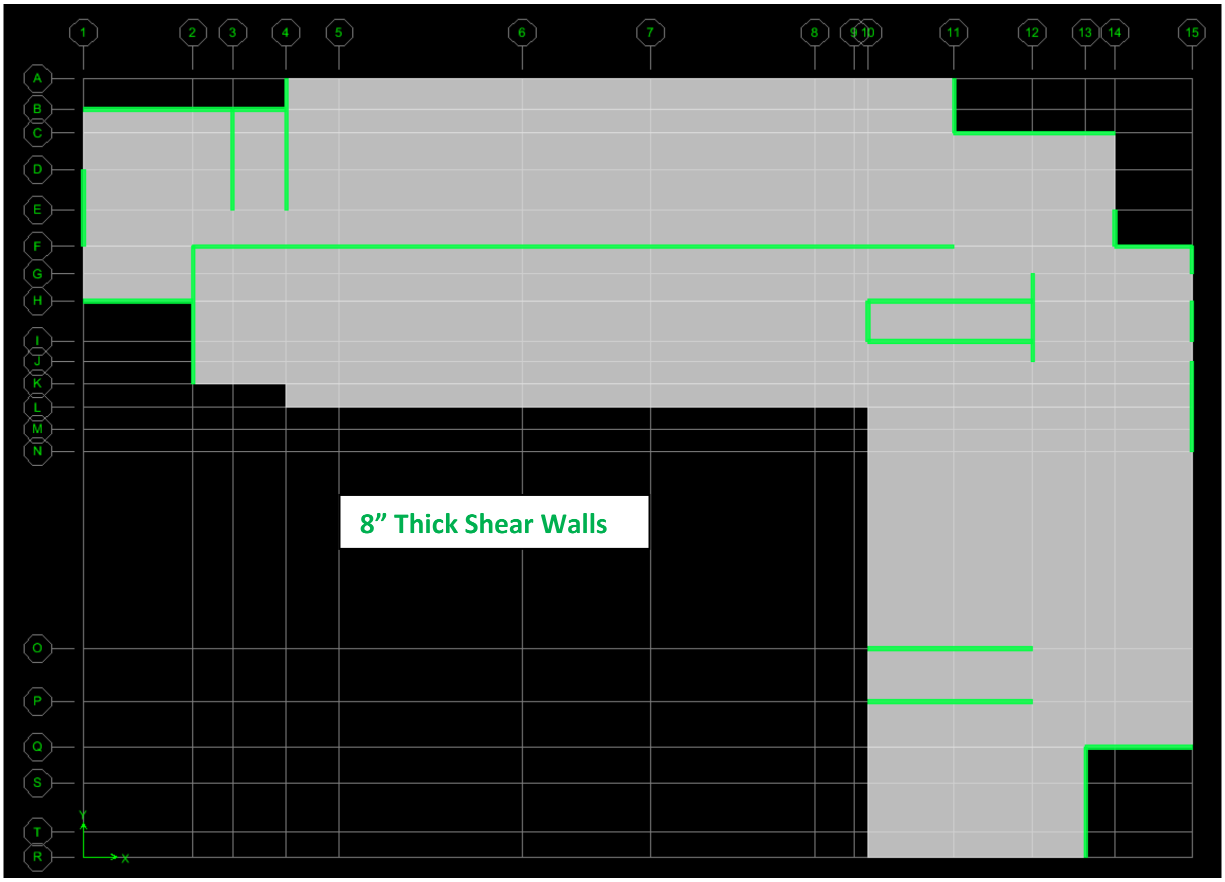


Figure 22: Plan view of story 2 through 7 shear walls

Center of Mass

The center of mass for the structure is based on the shape and mass of the slab because the slab is a large portion of the mass and the walls are even distributed around the slab. The Hyatt Place North Shore has the same shape for all floors and thus the same center of mass. The approximate hand calculation was found to be almost identical to ETABS value.

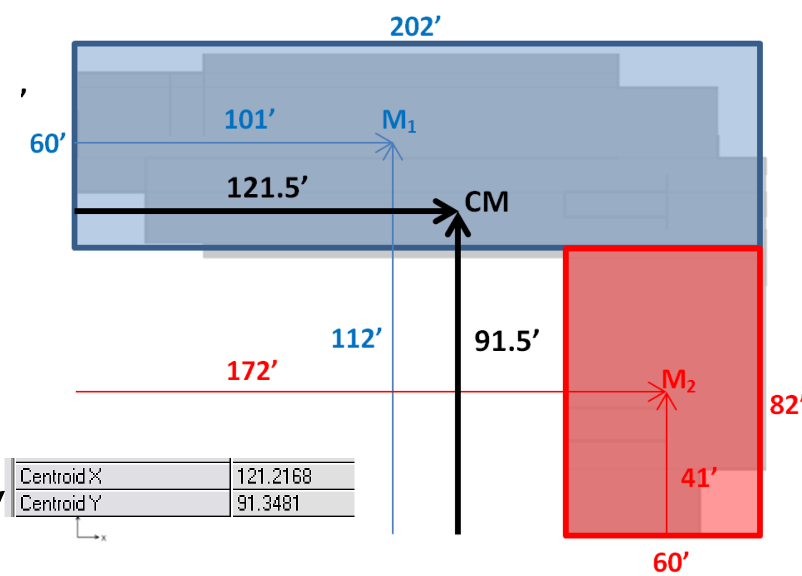


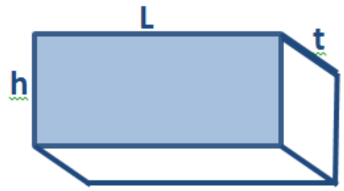
Figure 23: Center of mass diagram

$$x = \frac{12120*101 + 4920*172}{12120+4920} = 121.5'$$

$$y = \frac{12120*112 + 4920*41}{12120+4920} = 91.5'$$

Calculation of Wall Rigidity

$R = \frac{Et}{\left(4\left(\frac{h}{L}\right)^3\right)+3\left(\frac{h}{L}\right)}$ $E = 57000\sqrt{f'c}$



Example Calculation for Wall *Y*

L = 20' = 240 inches h = 8'8" = 104 inches t = 8 inches f'c = 2800 psi

$E = 57000\sqrt{2800} = \mathbf{3.016xE^6}$ $R = \frac{(3.016x10^3)(8)}{\left(4\left(\frac{104}{204}\right)^3\right)+3\left(\frac{104}{204}\right)}$ **=9114.5 kips per inch**

Wall Rigidities						
Wall Name	**Height (inches)**	**Length (inches)**	**Thickness (inches)**	**f'c (psi)**	**E (ksi)**	**Rigidity (kips/inch)**
A	216	54	12	2800	3.02E+03	**135.0**
B	216	78	12	2800	3.02E+03	**388.1**
C	216	96	12	2800	3.02E+03	**691.8**
D	216	132	12	2800	3.02E+03	**1613.1**
E	216	156	12	2800	3.02E+03	**2450.0**
F	216	168	12	2800	3.02E+03	**2928.5**
G	216	192	12	2800	3.02E+03	**3990.2**
H	216	240	12	2800	3.02E+03	**6444.4**
I	216	336	12	2800	3.02E+03	**12099.3**
J	216	360	12	2800	3.02E+03	**13585.6**
K	216	390	12	2800	3.02E+03	**15459.4**
L	216	400.8	12	2800	3.02E+03	**16136.6**
M	216	414	12	2800	3.02E+03	**16965.2**
N	216	78	12	2800	3.02E+03	**388.1**
O	216	444	12	2800	3.02E+03	**18850.0**
P	216	78	8	2800	3.02E+03	**258.7**
Q	216	216	8	2800	3.02E+03	**3446.9**
R	216	336	8	2800	3.02E+03	**8066.2**
S	104	38.4	8	2800	3.02E+03	**275.5**
T	104	102	8	2800	3.02E+03	**3305.8**
U	104	132	8	2800	3.02E+03	**5585.3**
V	104	174	8	2800	3.02E+03	**9114.5**
W	104	192	8	2800	3.02E+03	**10672.8**
X	104	216	8	2800	3.02E+03	**12759.9**
Y	104	240	8	2800	3.02E+03	**14843.6**
Z	104	294	8	2800	3.02E+03	**19485.0**
AA	104	306	8	2800	3.02E+03	**20505.8**
BB	104	336	8	2800	3.02E+03	**23040.8**
CC	104	360	8	2800	3.02E+03	**25052.3**
DD	104	444	8	2800	3.02E+03	**31995.4**
EE	104	1668	8	2800	3.02E+03	**128326.8**

Table 12: Wall rigidities

Relative Stiffness

Relative stiffness is the percent of the total stiffness per floor that one wall accounts for. The relative stiffness determines how much force a wall takes when in a structure with a rigid diaphragm. In the case of the Hyatt Place North Shore the system has a semi-rigid diaphragm that doesn't distribute the forces to walls in this manner. Instead the tributary area of the wall determines how much force goes to that wall, so relative stiffness isn't as important in the existing structure.

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

Percent Rigidity				
Wall Name	**% Rigidity 1st E/W**	**% Rigidity 1st N/S**	**% Rigidity 2nd-7th E/W**	**% Rigidity 2nd-7th N/S**
A	0.0	**0.2**	0.0	0
B	0.0	**0.7**	0.0	0
C	0.0	**1.2**	0.0	0
D	**1.3**	**2.8**	0.0	0
E	0.0	0.0	0.0	0
F	0.0	**5.0**	0.0	0
G	0.0	**6.8**	0.0	0
H	**5.0**	**11.0**	0.0	0
I	**9.4**	0.0	0.0	0
J	**10.6**	0.0	0.0	0
K	**12.0**	0.0	0.0	0
L	**12.6**	0.0	0.0	0
M	0.0	**29.0**	0.0	0
N	0.0	**30.9**	0.0	0
O	**14.7**	0.0	0.0	0
P	0.0	0.0	0.0	0
Q	0.0	**5.9**	0.0	0
R	**6.3**	0.0	0.0	0
S	0.0	0.0	0.0	**0.2**
T	0.0	0.0	0.0	**2.8**
U	0.0	0.0	0.0	**4.7**
V	0.0	0.0	**3.3**	**7.6**
W	0.0	0.0	0.0	**8.9**
X	0.0	0.0	0.0	**10.7**
Y	0.0	0.0	**5.4**	**12.4**
Z	0.0	0.0	0.0	**16.3**
AA	0.0	0.0	0.0	**17.2**
BB	0.0	0.0	**8.3**	0
CC	0.0	0.0	**9.0**	0
DD	0.0	0.0	**11.5**	0
EE	0.0	0.0	**32.1**	0

Table 13: Percent rigidity

Ground Story: $\sum R_{E/W}$ = O + H + L + J + K + J + D + R + R + R + I + H = **128416.9 k-in**

$\sum R_{N/S}$ = N + F + Q + Q + B + D + G + H + M + C + B + A = **58534.2 k-in**

Upper Stories: $\sum R_{E/W}$ = DD + Y + EE + BB + BB + CC + V + BB + BB + Y=**277080.5 k-in**

$\sum R_{N/S}$ = W + AA + X + Z + T + U + W + U + T + S + V + T + Y = **119417.7 k-in**

The structure is twice as rigid in the East/West direction due to that wing of the building being longer and better load paths to the foundations (the North/South direction has a large open space on the first floor). This confirmed in the ETABS model with the mode 1 being twice as large as mode 2, because period is related to stiffness.

The upper stories are also overall twice as rigid because there is a lower percentage of openings and the story height (h) is half of the ground story's.

Center of Rigidity and Torsion

The center of rigidity is based on the stiffness of lateral force resisting components and where and how they are oriented in the building. The Hyatt Place North Shore is a "L" shape that has an abundance of shear walls around its perimeter and along the double loaded corridor that runs down the middle of each leg, thus the center of rigidity is expected to be near the center of mass. The center of mass is where the load is considered to be applied to the structure, and the center of rigidity is where the structure wants the load to be taken at. The further the two are apart the more torsion there is on the building trying to twist it around the center of mass leading to more force in the walls to be resisted. The untoped precast concrete floor system is considered to be semi-rigid; therefore it doesn't tie the shear walls together into one unit like a rigid diaphragm would. The semi-rigid diaphragm isn't able to transmit torsional forces do to the eccentricity of the center of mass compared to the center of rigidity. But it is a good idea to determine how much of a factor torsion could be, due to the uncertainty of exactly how rigid the diaphragm will act and the shape and general layout of the shear walls. The "L" shape leads to the legs individually being better at resisting forces in a specific direction and then one side of the building could deflect a significant amount more than the other, *Figure 24*. Some of the unbalance of directional stiffness was taken out during the simplification of the shear wall layout due to the load paths in the building and partially to help the building act more uniform in the calculations. Ideally a large "L" shaped building would have an separation joint large enough to allow the two legs of the building to act independently from each other limiting the twisting action due to the major orientation of shear walls, *Figure 24*.

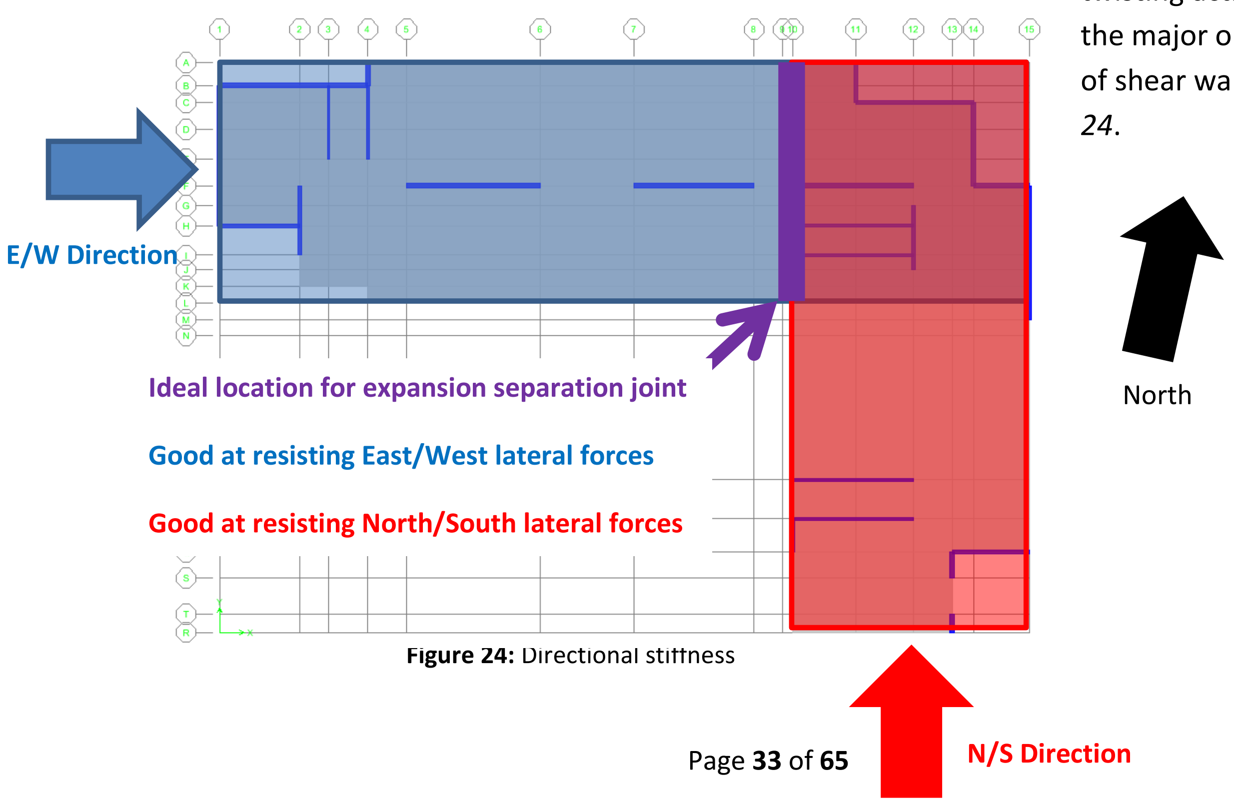


Figure 24: Directional stiffness

Calculating Center of Rigidity

$$x = \frac{\sum RiXi}{\sum Ri}$$

$$\bar{y} = \frac{\sum RiYi}{\sum Ri}$$

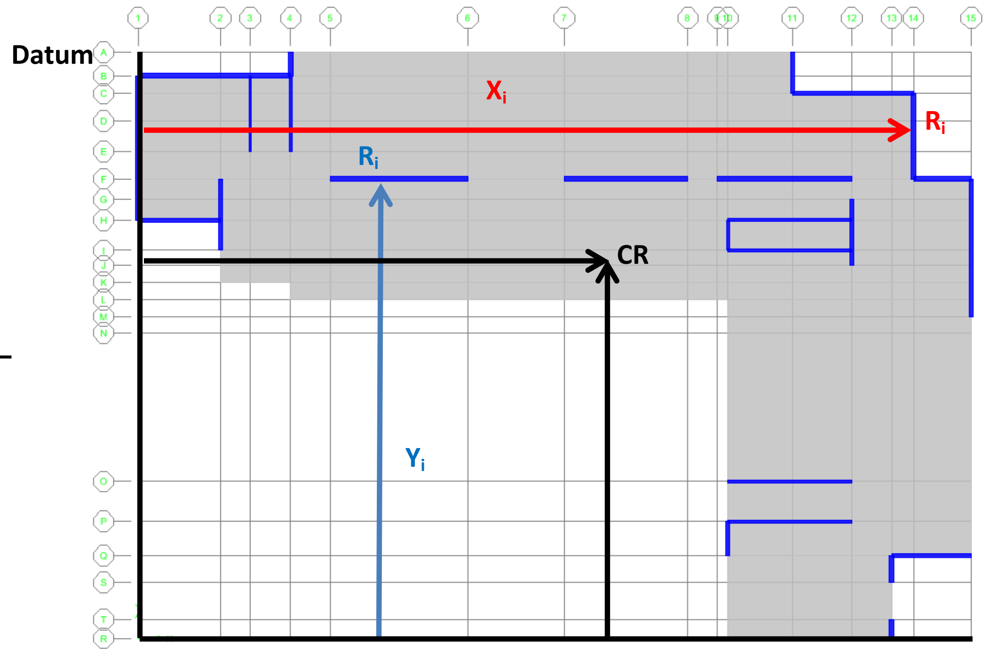


Figure 25: Center of rigidity

The center of rigidity is found in a similar way to center of mass. The center of rigidity for the two different floor layouts were found in excel and are shown in *Table 14*. The center of rigidity calculated by ETABS is slightly different because it factors in the rigidity of the diaphragm. Also noted in *Table 14* is the eccentricity between the center of mass and center of rigidity. The hand calculated center of rigidity was used to be conservative and stay consistent. The eccentricity creates torsion in buildings with rigid diaphragms, but will also be looked at in this case due to the uncertain rigidity of the diaphragm and the "L" shape of the building. Overall the eccentricity in the structure is low because of the even distribution of lateral force resisting systems throughout the layout. Some of the shear walls were not included in the calculation, so the eccentricity in the actual building may be even smaller. This should generally be the case for reinforced concrete masonry structures because all of the exterior walls that surround the layout will be lateral force resisting. One thing that seems strange is that the COR is to the East of the COM when the layout of the Western side of the building would lead to more rigidity in the N/S direction. This is because the shear wall along the corridor has a bad load path due to the open area on the ground level, thus it was omitted in the analysis. It is possible that the truss over the opening acts as a collector and directs the load to the surround shear walls and foundations, but this was not investigated.

Center of Rigidity									
Wall Type		1st Story				2nd - 7th Stories			
Name	Ri	Xi	Ri*Xi	Yi	Ri*Yi	Xi	Ri*Xi	Yi	Ri*Yi
A	135.0	2192.0	296023.6	2192.0	296018.1				
B	388.1	2192.0	850746.2	444 & 2192	1023050.2				
C	691.8	1686.0	1166446.1	1584.0	1095878.2				
D	1613.1	1904.4	3072061.6	1336.0	2155089.0				
E	2450.0								
F	2928.5	240.0	702836.9						
G	3990.2	2076.0	8283572.6						
H	6444.4	2256.0	14538666.7	1216 & 240	9383111.1				
I	12099.3			340.0	4113269.5				
J	13585.6			1336.0	18149798.9				
K	15459.4			1336.0	20653158.1				
L	16136.6			1336.0	21557787.3				
M	16965.2	2424.0	41123582.5						
N	18096.0	0.0							
O	18850.0			1636.0	30837769.9				
P	258.7	1716.0	443994.5						
Q	3446.9	326.4 & 444	2655458.7						
R	8066.2			1216 & 1128 & 456	22585313.1				
S	275.5					2424.0	8013177.7		
T	3305.8					1716 & 1716 & 2424	23603171.9		
U	5585.3					1904 & 2256	23234645.7		
V	9114.5					2424.0	22093548.6	1336.0	12176607.8
W	10672.8					2076.0	22156674.8		
X	12759.9					326.4	4164837.6		
Y	14843.6					2192.0	32537769.1	1215.96 & 240	21597440.8
Z	19485.0					444.0	8651356.5		
AA	20505.8					240.0	4921392.5		
BB	23040.8							1216 & 1128 & 456 & 340	72348024.5
CC	25052.3							1584.0	39682825.1
DD	31995.4							1636.0	52343196.1
EE	89068.0							1336.0	118991248.3
		∑Ri =	58793.0	∑Ri =	128416.9	∑Ri =	119417.7	∑Ri =	277080.5
		∑RiXi =	73133389	∑RiYi =	131850243	∑RiXi =	149376574.4	∑RiYi =	317139342.4
Hand		$\bar{x}$	1243.9	$\bar{y}$	1026.7	$\bar{x}$	1250.9	$\bar{y}$	1144.6
ETABS		$\bar{x}$	1301.8	$\bar{y}$	1229.7	$\bar{x}$	1344.8	$\bar{y}$	1207.40
Center of Mass		$\bar{x}$	1454.6	$\bar{y}$	1096.2	$\bar{x}$	1454.6	$\bar{y}$	1096.20
Ecentricity (+) Moment	(-) Moment	Ex =	-210.7	Ey =	-69.5	Ex =	-203.7	Ey =	48.4
Length Perpindicular to Load		Ly =	1704	Lx =	2424	Ly =	1704	Lx =	2424
% Eccentricity			-12.4		-2.9		-12.0		-2.0

Table 14: Center of rigidity

Torsion

Torsion is created when a structure wants to resist the applied load away from the centroid, so when the center of rigidity isn't at the same location as the center of mass there is torsion. The Hyatt Place North Shore has an overall building torsion that can be determined by taking the load times the eccentricity. There are two kinds of eccentricity, inherent and accidental. The inherent eccentricity is due to a difference in COM and COR and is calculated in *Table 14*. Accidental eccentricity is due to the possible differences in slab which would lead to added eccentricity. The accidental eccentricity is either added or subtracted to create the worst case scenario, this comes into play when finding torsional force in specific walls. Only rigid diaphragms can transmit torsion to the walls. To find total building torsion all floors are looked at individually with the largest load case in each direction and then all of the floors add up to get the total building torsion. I considered a clockwise rotation about the COM a positive moment. *Figure 26* shows a sample calculation for story 1 building torsion.

Sample calculation: Story 1 North/South & East/West Load Directions

$e_x = e_i + e_{acc} = 17.6' + .05*202 = 27.7'$

$e_y = e_i + e_{acc} = 5.8' + .05*142 = 12.9'$

Story Force:

Controling Load Case:

.9D + 1.0E + 1.6H

F_y & F_x = 47.2 kips

$M_y = F_y*e_x = 47.2*27.7 = 1307.4$ k-ft

$M_x = F_x*e_y = 47.2*12.9 = 608.3$ k-ft

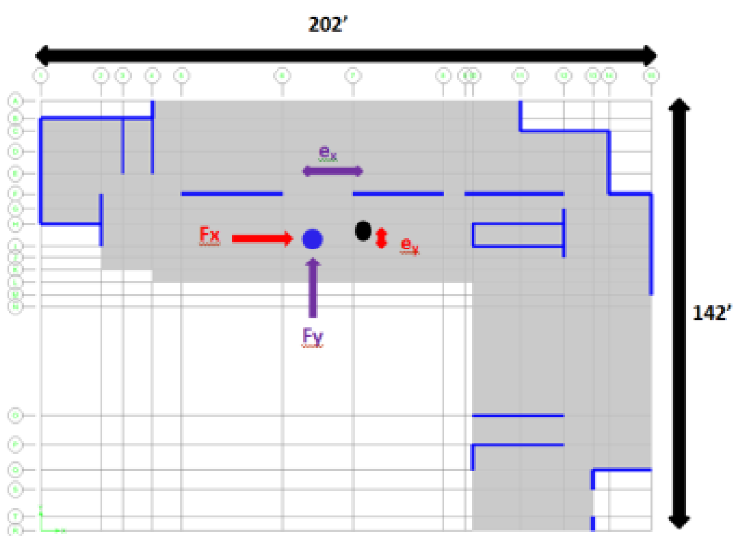


Figure 26: Building torsion

Total Building Torsion in North/South Direction						
Story	F_y (k)	L_x (ft)	e_{acc} (ft)	e_i (ft)	e_{tot} (ft)	M (k-ft)
7.0	104.1	202.0	10.1	16.9	27.0	2811.1
6.0	106.9	202.0	10.1	16.9	27.0	2885.6
5.0	91.6	202.0	10.1	16.9	27.0	2472.0
4.0	76.3	202.0	10.1	16.9	27.0	2060.7
3.0	61.1	202.0	10.1	16.9	27.0	1649.9
2.0	46.6	202.0	10.1	16.9	27.0	1257.6
1.0	47.2	202.0	10.1	17.6	27.7	1306.3
Total Building Torsion =						14443.2
Counter Clockwise			Clockwise			

Table 15: Building Torsion N/S

Total Building Torsion in East/West Direction						
Story	F_x (k)	L_y (ft)	e_{acc} (ft)	e_i (ft)	e_{tot} (ft)	M (k-ft)
7.0	104.1	142.0	-7.1	-4.0	-11.1	-1155.7
6.0	106.9	142.0	-7.1	-4.0	-11.1	-1186.3
5.0	91.6	142.0	-7.1	-4.0	-11.1	-1016.3
4.0	76.3	142.0	-7.1	-4.0	-11.1	-847.2
3.0	61.1	142.0	-7.1	-4.0	-11.1	-678.3
2.0	46.6	142.0	-7.1	-4.0	-11.1	-517.0
1.0	47.2	142.0	7.1	5.8	12.9	608.3
Total Building Torsion =						-4792.4
Counter Clockwise			Clockwise			

Table 16: Building Torsion E/W

Distribution of Lateral Shear Forces

The distribution of lateral forces to wall elements depends on the rigidity of the diaphragm. The Hyatt Place North Shore's untoped precast concrete plank system is considered to be semi-rigid diaphragm. But due to the fact that it is hard to tell specifically how rigid the diaphragm, the system will also be analyzed as a rigid diaphragm and forces compared in the walls. The system was also analyzed both ways in ETABS.

Semi-Rigid Diaphragm

When the diaphragm is flexible it acts as simple span, thus distributing load to the walls by tributary area. *Figure 27*, *28, 29, & 30* label the walls for the ground floor in the E/W and N/S directions and typical upper floors in each direction and shows their tributary area. The tributary areas were approximated and some judgment was used to determine the shape. The boxes shaded in red indicate areas that in particular are larger than that wall would need to carry because walls in the area were not modeled due to size. With a semi-rigid diaphragm it is important to have walls around its entire perimeter that take lateral load since the slab cannot transfer as well to surrounding shear walls as in a rigid diaphragm. The actual structure does have reinforced masonry shear walls around nearly all of the perimeter, so it will act better than modeled. In order to find the largest shear values in the walls the 1^{st} and 2^{nd} floor were analyzed. These floors have the largest shear forces because they have to hold the shear at that level and all of the shear action on the wall above it. The force in each wall was found by finding the percentage of total area that the wall is responsible for and multiplying that times the total lateral load needed to be resisted at that story level. Once the force is determined it is then checked to see if the wall is sufficient to carry the load.

Rigid Diaphragms

Rigid diaphragms make the walls at each floor level work as one unit and are able to transfer torsional loads. The load in each wall is the sum of direct shear (V_d) and torsional shear (V_t). The torsional shear is additive in some walls and subtractive in others. In this case the direct shear is related to relative rigidity rather than tributary area. The ETABS model of the structure with a rigid diaphragm is used to compare the difference in loads depending on the diaphragm type. Hand calculation was omitted to save time.

$$\boldsymbol{V_{di}} = \frac{\boldsymbol{R_i}}{\sum \boldsymbol{R_i}}\boldsymbol{V}$$

Direct Shear

$$\boldsymbol{J} = \sum \boldsymbol{R_i} * \mathbf{d}_{\boldsymbol{i}}^{\,2} \qquad \boldsymbol{V_{ti}} = \frac{\boldsymbol{V}\ \ \boldsymbol{e}+\boldsymbol{e_{acc}}\ \ \boldsymbol{d_i}\ \ \boldsymbol{R_i}}{\boldsymbol{J}}$$

Torsional Shear

Shear Strength Check

Also in Table # & # a shear strength check is included. Once the force that is in the wall is determined, it is necessary to check and see if the wall and its reinforcement are sufficient to carry the load, which is usually the case with reinforced masonry shear walls. The masonry shear walls are analyzed in the same way as concrete shear walls.

$$\phi V_n = \phi A_{cv} \quad \alpha_c \lambda \sqrt{f'_c} \ + \ \rho_t f_y \qquad \phi = .75 \quad A_{cv} = gross\ concrete\ area$$

$$\alpha_c = 2.0 \qquad f'_c = 2.8\ ksi \quad f_y = 60\ ksi$$

$$\rho_t = \frac{A_v}{s*h} \qquad s = reinforcement\ spacing$$

$$h = thickness\ of\ wall$$

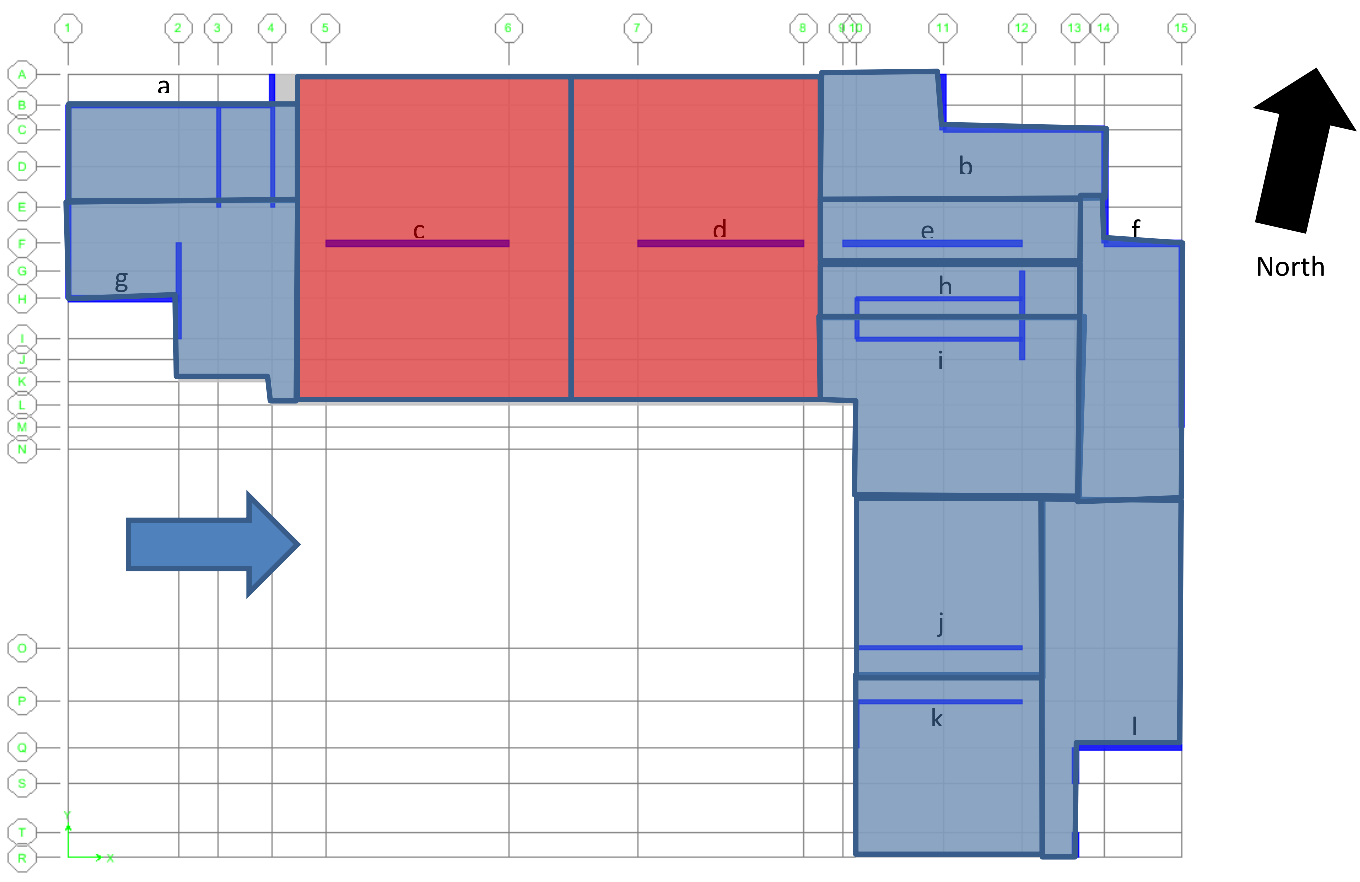


Figure 27: Shear wall tributary area for East/West walls on ground level

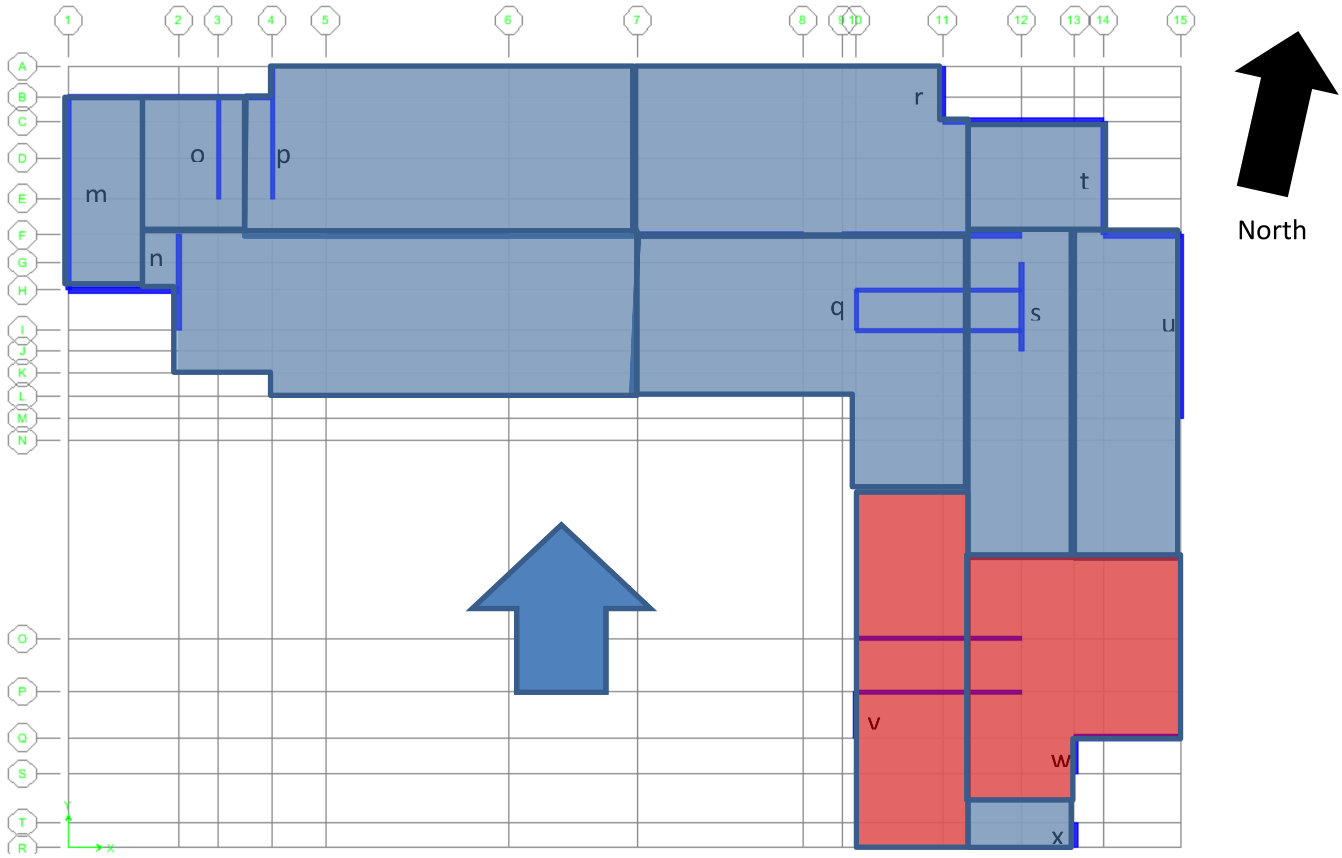


Figure 28: Shear wall tributary area for North/South walls on ground level

1st Story Shear in Walls																
				Vu (k)		Shear Strength Check										
	Wall	Area (SF)	% Tot. Area	Hand	ETABS Rigid	L_{wall} (in)	Shear Force (kli)	Vert. Reinf.	Spacing (in.)	Thickness (in)	A_{cv} (in^2)	α	f'c (ksi)	ρ_t	ΦV_n (k)	
Resists East/West Forces	a	769.9	0.0482	25.90	62.3	444.0	0.058	#7	16	12	5328	2	2.8	0.00313	14122	Works
	b	924.0	0.0579	31.09	52.8	288.0	0.108	#7	16	12	3456	2	2.8	0.00313	9160	Works
	c	2940.0	0.1841	98.92	66.10	400.0	0.247	#7	32	12	4800	2	2.8	0.00156	12384	Works
	d	2880.0	0.1804	96.90	66.20	360.0	0.269	#7	32	12	4320	2	2.8	0.00156	11147	Works
	e	540.0	0.0338	18.17	77.60	390.0	0.047	#7	32	12	4680	2	2.8	0.00156	12076	Works
	f	980.0	0.0614	32.97	17.40	168.0	0.196	#7	16	12	2016	2	2.8	0.00313	5344	Works
	g	1195.1	0.0749	40.21	29.70	240.0	0.168	#7	16	12	2880	2	2.8	0.00313	7634	Works
	h	540.0	0.0338	18.17	35.20	360.0	0.050	#6	32	8	2880	2	2.8	0.00172	7451	Works
	i	1575.0	0.0986	52.99	36.8	360.0	0.147	#6	32	8	2880	2	2.8	0.00172	7451	Works
	j	1190.0	0.0745	40.04	22.8	360.0	0.111	#6	32	8	2880	2	2.8	0.00172	7451	Works
	k	1190.0	0.0745	40.04	28.30	360.0	0.111	#7	32	12	4320	2	2.8	0.00156	11147	Works
	l	1242.0	0.0778	41.79	25.50	232.0	0.180	#7	16	12	2784	2	2.8	0.00313	7378	Works
Sum =		15966.0	1	537.18	520.7											
Resists North/South Forces	m	595.0	0.0380	20.40	190.1	420	0.049	#7	16	12	5040	2	2.8	0.00313	13359	Works
	n	2369.0	0.1512	81.22	10.1	207.96	0.391	#7	16	12	2496	2	2.8	0.00313	6615	Works
	o	459.0	0.0293	15.74	6.2	219.96	0.072	#6	32	8	1760	2	2.8	0.00172	4553	Works
	p	2880.0	0.1838	98.74	20.0	219.96	0.449	#6	32	8	1760	2	2.8	0.00172	4553	Works
	q	2010.0	0.1283	68.91	3.3	87.96	0.783	#6	32	8	704	2	2.8	0.00172	1821	Works
	r	1740.0	0.1111	59.66	2.6	120	0.497	#7	16	12	1440	2	2.8	0.00313	3817	Works
	s	1210.0	0.0772	41.49	9.0	192	0.216	#7	32	12	2304	2	2.8	0.00156	5945	Works
	t	675.0	0.0431	23.14	136.5	248.04	0.093	#7	16	12	2976	2	2.8	0.00313	7889	Works
	u	1210.0	0.0772	41.49	212.9	399.96	0.104	#7	16	12	4800	2	2.8	0.00313	12722	Works
	v	1210.0	0.0772	41.49	3.8	99.96	0.415	#7	16	12	1200	2	2.8	0.00313	3179	Works
	w	1190.0	0.0760	40.80	0.7	78	0.523	#7	16	12	936	2	2.8	0.00313	2481	Works
	x	120.0	0.0077	4.11	0.5	54	0.076	#7	16	12	648	2	2.8	0.00313	1718	Works
	Sum =	15668.0	1	537.18	595.7											

Table 17: Shear forces and checks for 1st level

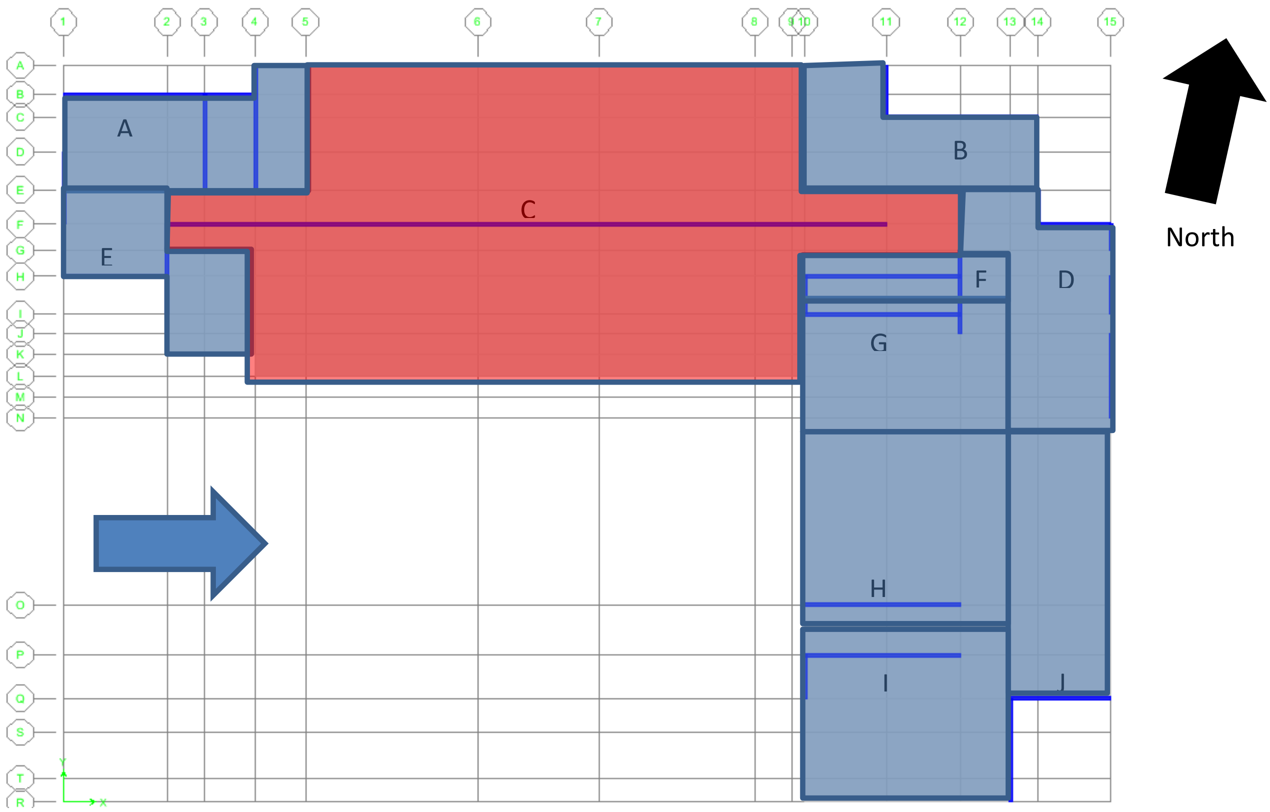


Figure 29: Shear wall tributary area for East/West walls on 2nd through 7th level

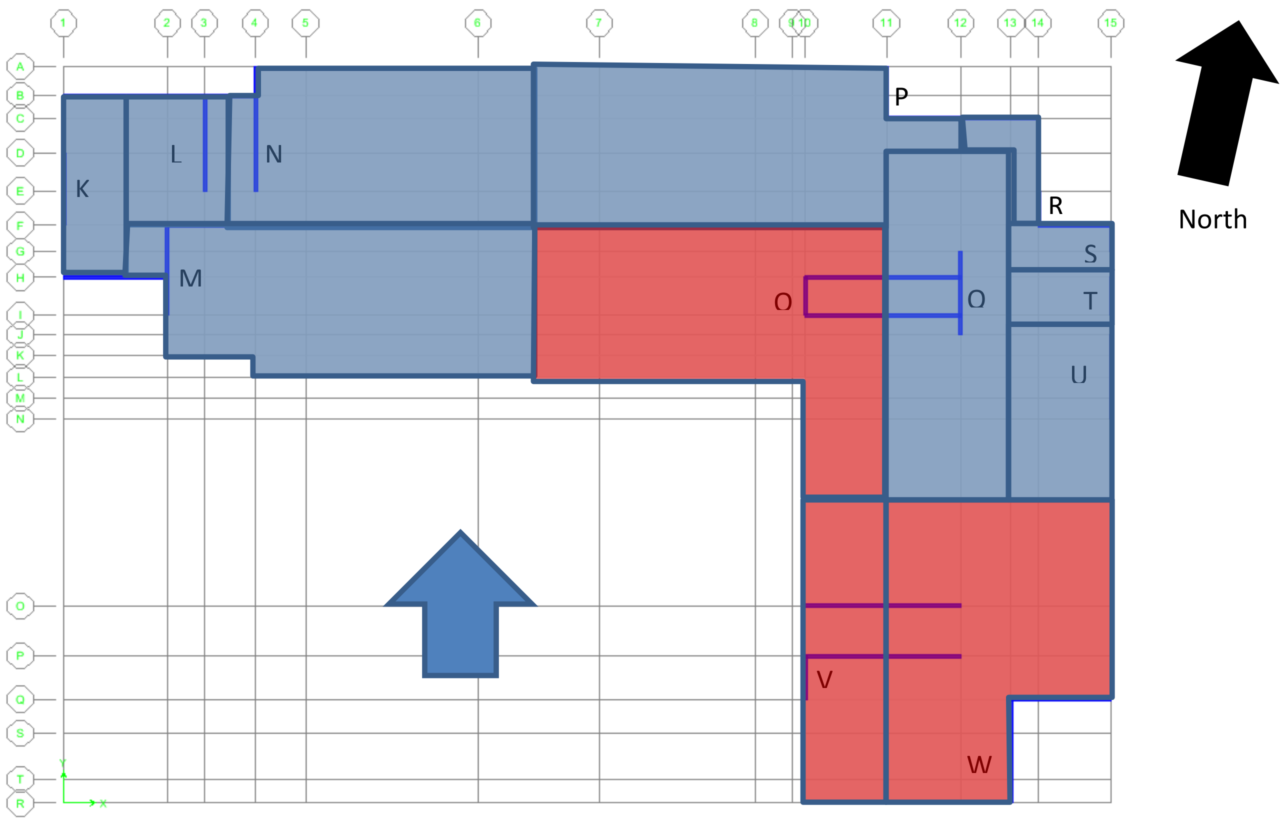


Figure 29: Shear wall tributary area for North/South walls on 2nd through 7th level

2nd Floor																
				Story Shear Force (k)		Shear Strength Check										
	Wall	Area (SF)	% Tot. Area	Hand	ETABS Rigid	L_{wall} (in)	Shear Force (kli)	Vert. Reinf.	Spacing (in.)	Thickness (in)	A_{cv} (in^2)	α	f'c (ksi)	ρ_t	ΦV_n (k)	
Resists East/West Forces	A	912.0	0.0575	28.17	45.2	444.0	0.063	#6	32	8	3552	2	2.8	0.001719	9190	Works
	B	930.0	0.0586	28.72	34.7	351.6	0.082	#6	32	8	2813	2	2.8	0.001719	7278	Works
	C	6708.0	0.4228	207.18	249.10	1664.4	0.124	#5	32	8	13315	2	2.8	0.001211	34146	Works
	D	1000.0	0.0630	30.89	9.23	168.0	0.184	#6	32	8	1344	2	2.8	0.001719	3477	Works
	E	720.0	0.0454	22.24	16.50	240.0	0.093	#6	32	8	1920	2	2.8	0.001719	4968	Works
	F	344.0	0.0217	10.62	29.70	360.0	0.030	#5	32	8	2880	2	2.8	0.001211	7386	Works
	G	1161.0	0.0732	35.86	32.86	360.0	0.100	#5	32	8	2880	2	2.8	0.001211	7386	Works
	H	1548.0	0.0976	47.81	17.80	360.0	0.133	#5	32	8	2880	2	2.8	0.001211	7386	Works
	I	1462.0	0.0922	45.15	23.7	360.0	0.125	#5	32	8	2880	2	2.8	0.001211	7386	Works
	J	1080.0	0.0681	33.36	16.5	232.0	0.144	#6	32	8	1856	2	2.8	0.001719	4801	Works
Sum =		15865.0	1.00	490.00	475.29											
Resists North/South Forces	K	420.0	0.0281	13.76	53.8	420.0	0.033	#6	32	8	3360	2	2.8	0.001719	8693	Works
	L	500.0	0.0334	16.38	47.6	300.0	0.055	#5	32	8	2400	2	2.8	0.001211	6155	Works
	M	2100.0	0.1404	68.80	17.50	220.0	0.313	#6	32	8	1760	2	2.8	0.001719	4553	Works
	N	1800.0	0.1203	58.97	69.00	220.0	0.268	#5	32	8	1760	2	2.8	0.001211	4513	Works
	O	2240.0	0.1498	73.38	16.90	88.0	0.834	#5	32	8	704	2	2.8	0.001211	1805	Works
	P	2040.0	0.1364	66.83	23.10	120.0	0.557	#6	32	8	960	2	2.8	0.001719	2484	Works
	Q	1560.0	0.1043	51.11	50.20	192.0	0.266	#5	32	8	1536	2	2.8	0.001211	3939	Works
	R	325.0	0.0217	10.65	29.10	248.0	0.043	#6	32	8	1984	2	2.8	0.001719	5134	Works
	S	176.0	0.0118	5.77	18.8	60.0	0.096	#6	32	8	480	2	2.8	0.001719	1242	Works
	T	220.0	0.0147	7.21	28.6	88.0	0.082	#6	32	8	704	2	2.8	0.001719	1821	Works
	U	616.0	0.0412	20.18	31.3	196.0	0.103	#6	32	8	1568	2	2.8	0.001719	4056	Works
	V	896.0	0.0599	29.35	14.7	99.96	0.294	#6	32	8	800	2	2.8	0.001719	2069	Works
	W	2064.0	0.1380	67.62	78.2	240	0.282	#6	32	8	1920	2	2.8	0.001719	4968	Works
Sum =		14957.0	1.0000	490.00	478.8											

Table 18: Shear forces and checks for 2nd level

ETABS Results

Controlling Load Case

When considering the controlling lateral load case you have to consider the load cases from ASCE 7-05 that take into account dead and live loads along with the lateral wind or earthquake forces. A building will never be subjected to wind loads only, there will be the weight of the building and other things in it that help to hold it down or give it weight that plays a role in the seismic load. Without the weight of the building it would be much easier for lateral loads to create an overturning moment that could cause the building to fail. The load combinations used are listed below, and values for story shear at the 7^{th} level in the semi-rigid diaphragm model are listed in *Table 19*. The other levels are listed, but they were consistent with level 7. The controlling case was determined to be combo7 for each direction, combo 5 in the X-direction gives the same values as combo7. Both load cases have a multiplier of 1.0 on the earthquake forces so combo7 was considered to control in both directions for simplicity. It is interesting that seismic controls in Pennsylvania, but the building is very massive and has a relatively low R-value. The wind load was relatively close in the Y-direction, where the wind hits the wider of the two faces of the building, so it is possible that if the floor to floor height wasn't minimal the wind would control in the Y-direction. With the controlling case determined, it is now used in the calculation of building torsion, story and wall shears, along with others.

Story	Load	Loc	P	VX	VY
STORY7	COMB1	Top	3638.6	0	0
STORY7	COMB1	Bottom	4388.79	0	0
STORY7	COMB2	Top	3508.67	0	0
STORY7	COMB2	Bottom	4151.69	0	0
STORY7	COMB3	Top	4366.4	0	0
STORY7	COMB3	Bottom	5009.42	0	0
STORY7	COMB4X	Top	3508.67	-58.56	0
STORY7	COMB4X	Bottom	4151.69	-58.56	0
STORY7	COMB4Y	Top	3508.67	0	-88.51
STORY7	COMB4Y	Bottom	4151.69	0	-88.51
STORY7	COMB5X	Top	3274.75	-104.11	0
STORY7	COMB5X	Bottom	3917.77	-104.11	0
STORY7	COMB5Y	Top	3274.75	0	-55.32
STORY7	COMB5Y	Bottom	3917.77	0	-55.32
STORY7	COMB6X	Top	2339.1	-58.56	0
STORY7	COMB6X	Bottom	2821.36	-58.56	0
STORY7	COMB6Y	Top	2339.1	0	-88.51
STORY7	COMB6Y	Bottom	2821.36	0	-88.51
STORY7	COMB7X	Top	2339.1	-104.11	0
STORY7	COMB7X	Bottom	2821.36	-104.11	0
STORY7	COMB7Y	Top	2339.1	0	-104.11
STORY7	COMB7Y	Bottom	2821.36	0	-104.11

Table 19: Determination of controlling load case

- 1.4 (D + F) COMBO1
- 1.2 (D + F + T) + 1.6(L + H) + .5(L_r or S or R) COMBO2
- 1.2D + 1.6(L_r or S or R) + (L or .8W) COMBO3
- 1.2D + 1.6W + L + .5(L_r or S or R) COMBO4
- 1.2D + 1.0E + L + .2S COMBO5
- .9D + 1.6W + 1.6H COMBO6
- **.9D + 1.0E + 1.6H** **COMBO7**

Drift and Displacement

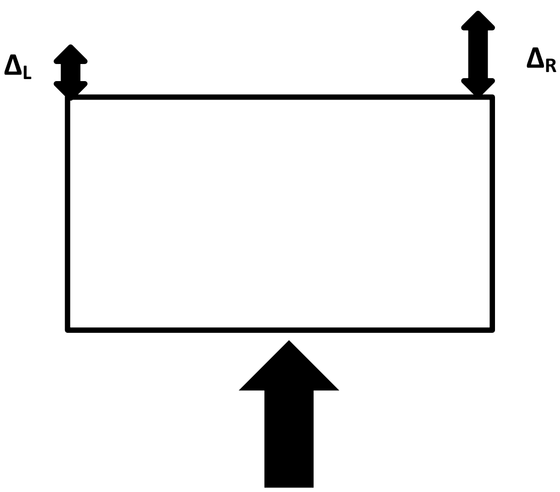
Drift and displacement are servicablility considerations to make sure that non-structural components are not damaged when the building is fully load along with making sure to not disturb the occupants of the building. Excessive drift back in forth of floor displacement maybe within the safe capacity of the structural system, but it doesn't create a good environment for its inhabitants. The code limits are as follows:

$$\Delta = l/400 \quad (wind) \qquad \qquad \Delta = .007h_{sx} \quad (seismic)$$

$$\Delta = 70*12/400 = 2.1" \qquad \qquad \Delta = .007(18*12) = 1.51" \ (ground\ level)$$

$$\Delta = .007(8*12+8) = .728" \ (upper\ levels)$$

For the Hyatt Place North Shore seismic is the controlling load in both directions. Story and total displacements were checked in the walls located on the ends of the building in ETABS to make sure they were under the code limit and to also make sure that there is no torsional irregularity.



$$\text{if } \Delta_R \geq 1.2\frac{\Delta_L+\Delta_R}{2} \quad \therefore Torsionally\ Irregular$$

Given the "L" shape of the building and layout of the lateral force resisting elements, this is a case that should be checked. *Table 20* checks the Δ limit states for seismic and torsion using data from the rigid and semi rigid diaphragm structural models. Figure 30 shows the locations where the displacements were checked.

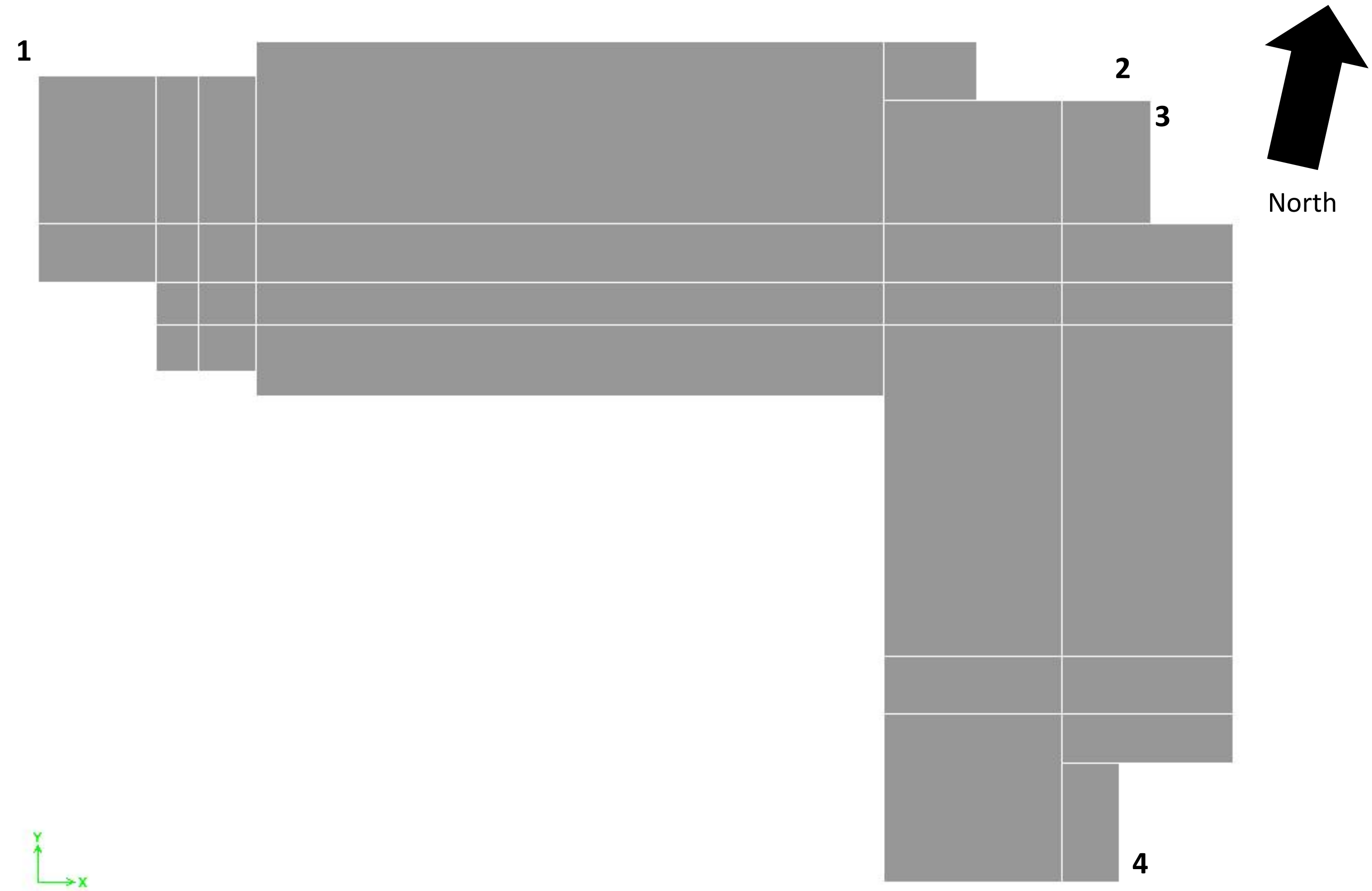


Figure 30: Locations of displacement checks

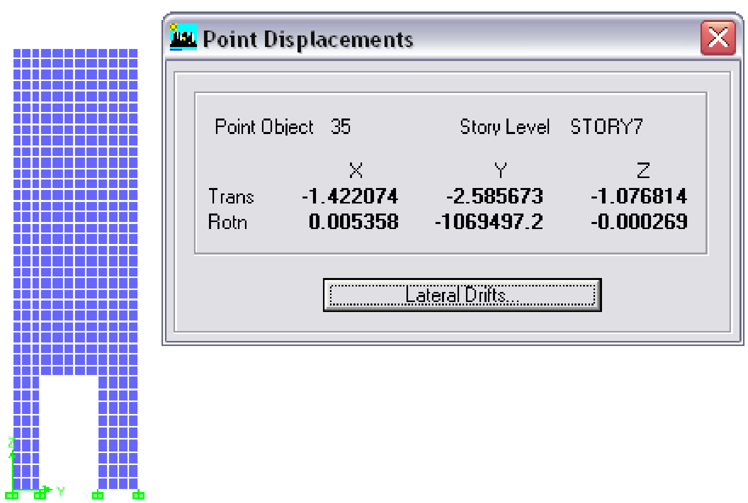


Figure 31: How displacements are checked

Deflections were checked in shear walls rather than at corners of slab to be consistent and because the semi-rigid model has large distortions in regions of the slab that are not restrained by shear walls, shown in *Figure 32* & *33*. This behavior is one reason I chose to model the with a rigid diaphragm also. It also shows that in real life that systems with semi-rigid diaphragms need to be surrounded by shear walls, as they are in the Hyatt Place North Shore.

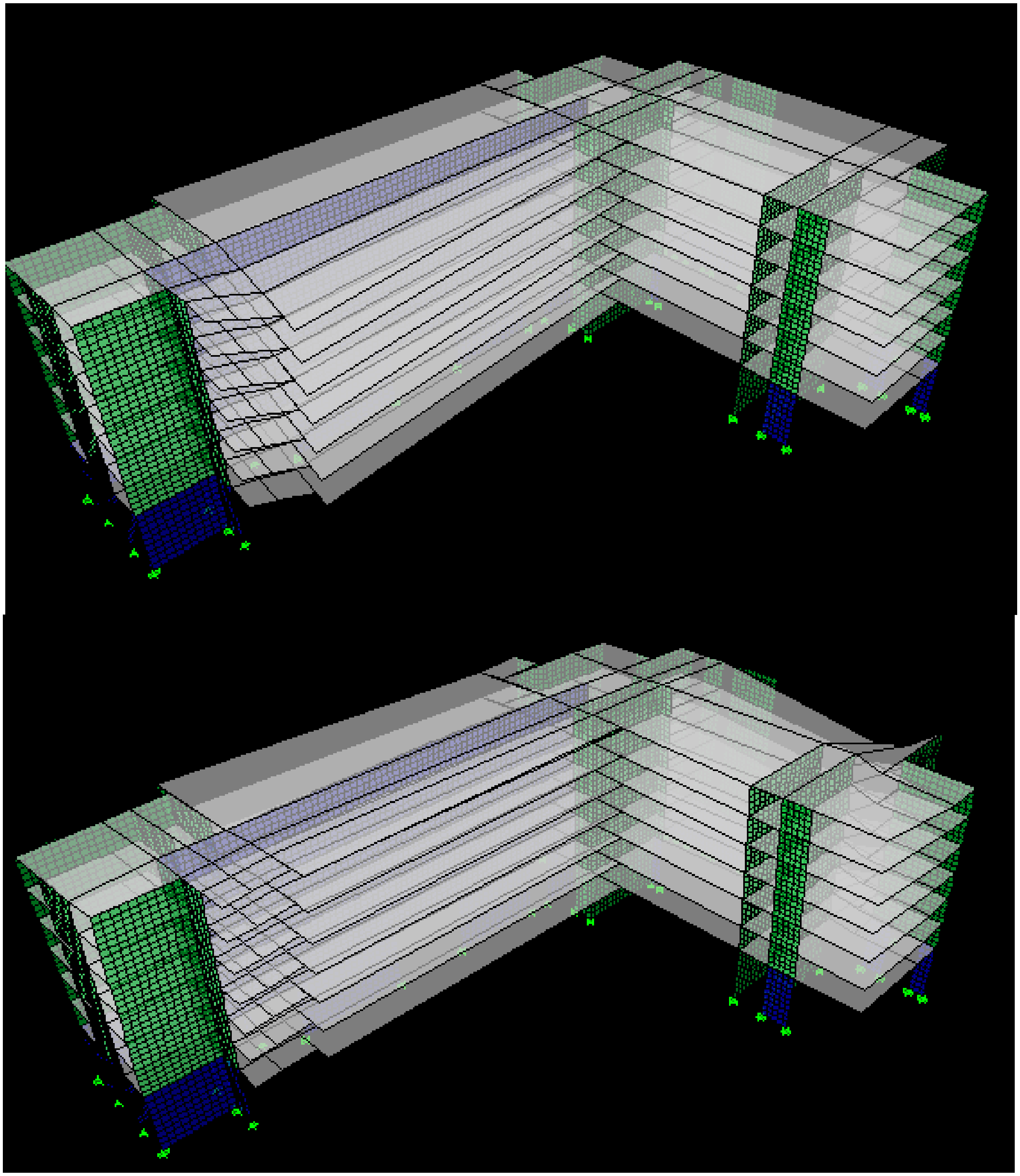


Figure 32: Semi-Rigid diaphragm behavior

Seismic Loading in Y-Direction (North/South)											Torsional Amplification Check			
			Pt 1				Pt 2				Semi-Rigid		Rigid	
			Semi-rigid		Rigid		Semi-rigid		Rigid					
Story	Height (in)	$\Delta_{max\ allow}$ (in)	Drift-Y (in)	Displ-Y (in)	Drift-Y (in)	Displ-Y (in)	Drift-Y (in)	Displ-Y (in)	Drift-Y (in)	Displ-Y (in)	Δ_{Larger}	$1.2((\Delta_L+\Delta_R)/2)$	Δ_{Larger}	$1.2((\Delta_L+\Delta_R)/2)$
7	104	0.728	0.265	1.275	0.023	0.133	-0.686	-2.892	0.029	0.167	-2.892	-0.9702	0.167	0.18
6	104	0.728	0.26	1.01	0.023	0.11	-0.656	-2.206	0.029	0.138	-2.206	-0.7176	0.138	0.1488
5	104	0.728	0.244	0.75	0.023	0.087	-0.578	-1.55	0.028	0.109	-1.55	-0.48	0.109	0.1176
4	104	0.728	0.214	0.506	0.022	0.064	-0.472	-0.972	0.027	0.081	-0.972	-0.2796	0.081	0.087
3	104	0.728	0.171	0.292	0.018	0.042	-0.335	-0.5	0.025	0.054	-0.5	-0.1248	0.054	0.0576
2	104	0.728	0.116	0.121	0.015	0.024	-0.1838	-0.165	0.018	0.029	-0.165	-0.0264	0.029	0.0318
1	216	1.512	0.005	0.005	0.009	0.009	0.0188	0.0188	0.011	0.011	0.0188	0.01428	0.011	0.012
Within Code Requirment			Pt 1				Pt 2				Semi-Rigid		Rigid	
			Semi-rigid		Rigid		Semi-rigid		Rigid					
Exceeds Allowable Code			Drift-X (in)	Displ-X (in)	Drift-Y (in)	Displ-X (in)	Drift-X (in)	Displ-X (in)	Drift-X (in)	Displ-X (in)	Δ_{Larger}	$1.2((\Delta_L+\Delta_R)/2)$	Δ_{Larger}	$1.2((\Delta_L+\Delta_R)/2)$
			0.018	-0.011	-0.0018	-0.0093	0.048	0.318	-0.0016	-0.0085	0.318	0.1535	-0.0093	-0.01068
Close to Exceeding Allowable Code			0.01	-0.029	-0.0017	-0.0075	0.06	0.27	-0.0016	-0.0069	0.27	0.1205	-0.0075	-0.00864
			-0.001	-0.039	-0.0017	-0.0058	0.062	0.21	-0.0016	-0.0053	0.21	0.0855	-0.0058	-0.00666
			-0.005	-0.038	-0.0017	-0.0041	0.055	0.148	-0.0016	-0.0037	0.148	0.055	-0.0041	-0.00468
			-0.009	-0.033	-0.0015	-0.0024	0.042	0.093	-0.0013	-0.0021	0.093	0.03	-0.0024	-0.0027
			-0.012	-0.024	-0.0009	-0.0009	0.029	0.051	-0.0008	-0.0008	0.051	0.0135	-0.0009	-0.00102
			-0.012	-0.012	0	0	0.022	0.022	0	0	0.022	0.005	0	0

Table 20: Deflection due to loading in the North/South direction

Table 20 shows that the building acts irregular with a semi-rigid diaphragm, but that the displacements are smaller and more like expected with the rigid diaphragm. Although the displacements with a rigid diaphragm are smaller, they are fairly close to being torsional irregular. This makes sense because the the rigid diaphragm transmits torsional forces to the walls. All displacements with both systems meet the code requirements for seismic loading.

Seismic Loading in X-Direction (East/West)											Torsional Amplification Check			
			Pt 3				Pt 4				Semi-Rigid		Rigid	
			Semi-rigid		Rigid		Semi-rigid		Rigid					
Story	Height (in)	$\Delta_{max\ allow}$ (in)	Drift-X (in)	Displ-X (in)	Drift-Y (in)	Displ-X (in)	Drift-X (in)	Displ-X (in)	Drift-X (in)	Displ-X (in)	Δ_{Larger}	$1.2((\Delta_L+\Delta_R)/2)$	Δ_{Larger}	$1.2((\Delta_L+\Delta_R)/2)$
7	104	0.728	0.055	0.126	0.00147	0.01447	-0.138	-1.422	0.004	0.026	-1.422	-0.7776	0.01447	0.024282
6	104	0.728	0.035	0.071	0.0016	0.013	-0.208	-1.284	0.004	0.022	-1.284	-0.7278	0.013	0.021
5	104	0.728	0.02	0.036	0.0019	0.0114	-0.234	-1.076	0.004	0.018	-1.076	-0.624	0.0114	0.01764
4	104	0.728	0.018	0.016	0.0019	0.0095	-0.234	-0.842	0.003	0.014	-0.842	-0.4956	0.0095	0.0141
3	104	0.728	0.018	-0.002	0.0021	0.0076	-0.222	-0.608	0.004	0.011	-0.608	-0.366	0.0076	0.01116
2	104	0.728	-0.007	-0.02	0.0021	0.0055	-0.209	-0.386	0.003	0.007	-0.386	-0.2436	0.0055	0.0075
1	216	1.512	-0.013	-0.013	0.0034	0.0034	-0.177	-0.177	0.004	0.004	-0.177	-0.114	0.0034	0.00444
Within Code Requirment			Pt 3				Pt 4				Semi-Rigid		Rigid	
			Semi-rigid		Rigid		Semi-rigid		Rigid					
			Drift-Y (in)	Displ-Y (in)	Drift-Y (in)	Displ-Y (in)	Drift-Y (in)	Displ-Y (in)	Drift-Y (in)	Displ-Y (in)	Δ_{Larger}	$1.2((\Delta_L+\Delta_R)/2)$	Δ_{Larger}	$1.2((\Delta_L+\Delta_R)/2)$
Exceeds Allowable Code			-0.687	-2.888	0.00089	0.00486	-0.562	-2.585	0.0008	0.0043	-2.888	-3.2838	0.00486	0.005496
Exceeds Allowable For Wind Loading			-0.654	-2.201	0.00091	0.00397	-0.552	-2.023	0.0008	0.0035	-2.201	-2.5344	0.00397	0.004482
			-0.578	-1.547	0.00086	0.00306	-0.505	-1.471	0.0008	0.0027	-1.547	-1.8108	0.00306	0.003456
			-0.472	-0.969	0.00075	0.0022	-0.425	-0.966	0.0006	0.0019	-0.969	-1.161	0.0022	0.00246
			-0.333	-0.497	0.00057	0.00145	-0.327	-0.541	0.0005	0.0013	-0.497	-0.6228	0.00145	0.00165
			-0.164	-0.164	0.00028	0.00088	-0.193	-0.214	0.0002	0.0008	-0.164	-0.2268	0.00088	0.001008
			0	0	0.0006	0.0006	-0.021	-0.021	0.0006	0.0006	0	-0.0126	0.0006	0.00072

Table 21: Deflection due to loading in the East/West direction

Table 21 shows the same information as Table 20 except for loading in the East/West direction. All code requirements are met the rigid diaphragm, and all seismic are met with the semi-rigid but the total displacement would exceed if it were a wind load displacement. Strangely there is a large deflection in the Y-direction for the semi-rigid diaphragm.

Mode Shapes

Mode	Period (sec.)	
	Semi-Rigid	Rigid
1	0.4996	0.4699
2	0.3248	0.2851
3	0.1827	0.1529

Table 22: Mode periods

Both models had reasonable periods that are close to the approximated .53 seconds that was found in the seismic calculation. The first mode shape is in the Y-Direction because the building is the least stiff in that direction based on the building shape and layout of shear walls with good load paths to the foundation. The second period is rotation about the Z-Axis, and the third period is in the X-Direction. The building is stiffest in the X-Direction with the large amount of shear walls resisting loads in that direction.

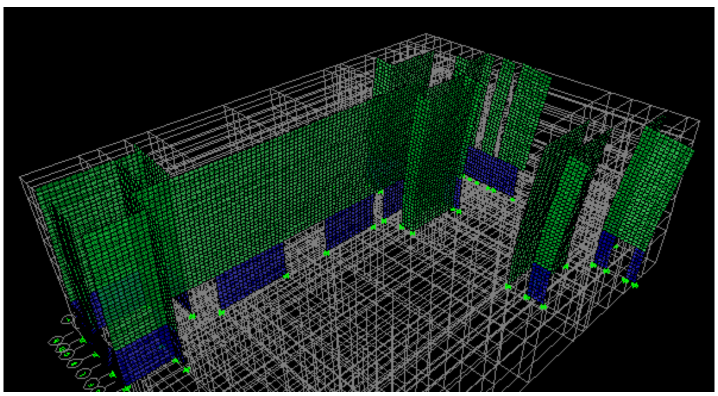


Figure 33: Mode shape 1

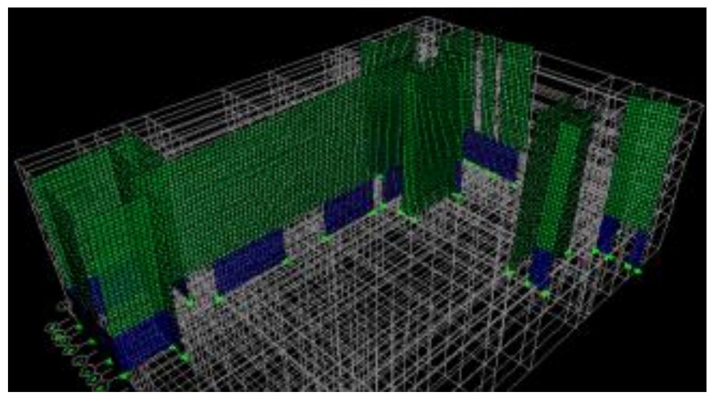


Figure 34: Mode shape 2

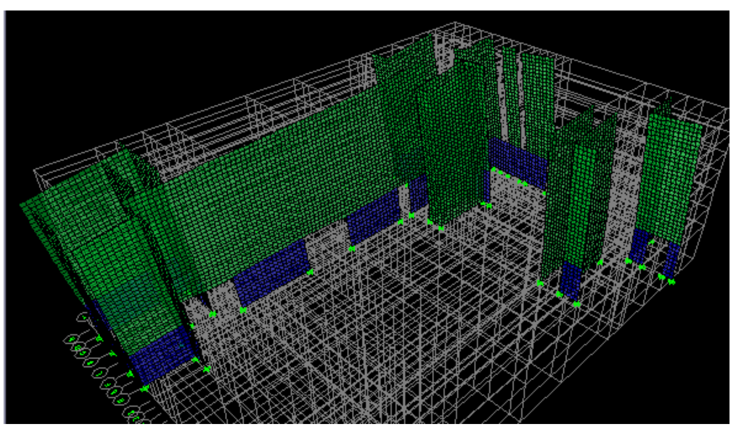


Figure 34: Mode shape 3

Overturning Moment

Overturning Moment Due to Controling Seismic Load			
Story	**Height (ft)**	**Forces (K) Fx**	**Moments (ft-K) Mx**
Penthouse Roof	80	3.47	277.48
7	70	104.11	7530.85
6	61.33	106.87	13152.61
5	52.66	91.56	16114.57
4	44	76.32	16822.76
3	35.33	61.11	15666.86
2	26.66	46.58	13063.97
1	18	47.16	9669.24
		Total =	**92298.34**

Table 23: Overturning moment

Story	Point	Load	FZ
BASE	35	COMB7Y	-27.03
BASE	36	COMB7Y	-19.5
BASE	43	COMB7Y	-16.95
BASE	16	COMB7Y	-14.76
BASE	45	COMB7Y	-9.34
BASE	38	COMB7Y	-8.06
BASE	55	COMB7Y	-7.16
BASE	14	COMB7Y	-1.32
BASE	21	COMB7Y	-0.83
BASE	22	COMB7X	0.71
BASE	20	COMB7Y	1.49
BASE	37	COMB7Y	1.75
BASE	2	COMB7X	1.91
BASE	48	COMB7Y	2.65
BASE	6	COMB7X	2.71

Table 24: Uplift force at base

Lateral forces on the building on the building at each story level create a moment based on the height above the base that they are applied. The building is acting like a cantilever beam with a fixed base. The moment has to be resisted by the foundation to prevent the building from overturning. *Figure 35* shows the uplift force in red and downward force in green. Most foundations are strictly designed to carry downward force by bearing on soil or rock, meaning that the uplift force is typically held down with dead load from the dead load of the structure. *Table 24* shows reaction forces due to the combo7 loading, which already has dead load included. The weight of the foundation is not included, so this weight will be calculated to see if it is great enough to hold the building down. The building foundation consists of 18" diameter auger piles that go down approximately 70 feet to get to bedrock. Pile caps have 2, 3, or 4 piles per cap. Weight for the minimum number of piles was calculated below.

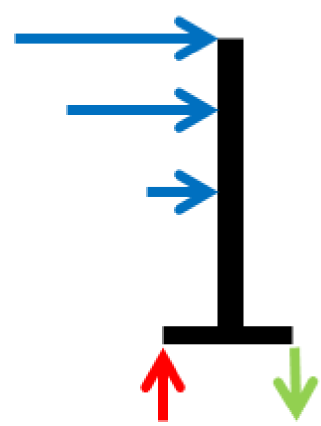


Figure 35: Overturning forces

$$A_{circle} = \pi \ \frac{9}{12}^{\ 2} = 1.76\,ft^2 \qquad A_{pile} = 1.76 \ \ 70 \ = 123.7\,ft^3$$

$$W_{per\ pile} = .150 \ \ 123.7 \ = 18.5\ kips$$

$$W_{total} = 18.5 \ \ 2 \ = \boldsymbol{37\ kips > 27\ kips \ \therefore Okay}$$

Conclusion

After analyzing the existing lateral force resisting system of the 7 story Hyatt Place North Shore it is determined that it is sufficient to carry the load and meet code standards for drift and displacement. The 70 feet tall, 108,000 square foot structure has intermediate reinforced concrete masonry bearing walls working in combination with an 8" untoped precast concrete plank floor structure to handle both gravity and lateral loads down into the soft soils along the Allegheny River and to bedrock with numerous 18" diameter auger piles.

An ETABS model was used to determine the controlling load case for the structure is *.9D + 1.0E + 1.6H*. Shear walls that were either small or have poor load paths to the foundations were excluded to simplify modeling and hand calculations, so calculated values were conservative. The added shear walls around the edges of the diaphragm will help the semi-rigid diaphragm to perform better. Because of uncertainty of how rigid the diaphragm will act, there was a model created with a semi-rigid diaphragm and one with a rigid diaphragm. These were used to compare diaphragm effects and to check hand calculations of center of mass, center of rigidity, and shear based on the loads from the controlling case in each direction. Calculated values for center of mass and center of rigidity were consistent with the ETABS model. The ETABS model was used exclusively to determine drift and displacement meet the allowable code standards. In this case the differences between the diaphragms became very apparent, but both rigid and semi-rigid meet code standards.

The shape of the building and layout of the lateral force resisting walls were also discussed in terms of rigidity and torsion. This leads to the determination of shear forces in walls. The walls were found to be plenty sufficient to handle the shear loads from both semi-rigid diaphragm and rigid diaphragm systems. Lastly the forces must make it safely into the foundations. The uplift forces due to overturning moment were found using ETABS and it was determined that the weight of the foundations is sufficient to keep the building firmly on the ground.

Appendix A

Wind Loading:

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Method 2 - analytical procedure

Basic Wind Speed (V) = 90 (Fig. 6-1)

Wind Importance Factor (I) = 1 (Table 6-1)
→ Category II → Non-hurricane

Exposure Category = C (Sec. 6.5.6.3)

Directionality Factor (K_d) = .85 (Table 6-4)
→ Buildings

Topographic Factor (K_{zt}) = 1.0 (Sec 6.5.7.1)

Velocity Pressure Exposure Coefficient Evaluated at Height Z (K_z) (Table 6-3)
→ exposure C
→ interpolated

Level	Height (ft)	K_z
1	0	0
2	18	.88
3	26.66	.95
4	35.33	1.01
5	44	1.06
6	52.66	1.10
7	61.33	1.13
Roof	70	1.17
High Roof	80	1.20

Velocity Pressure at Height Z (q_z) (Eq. 6-15)

$q_z = .00256 \boxed{K_z} K_{zt} K_d V^2 I$

Varies per level → done in excel

$q_z = .00256\, K_z (1.0)(.85)(90^2)(1.0)$

Velocity Pressure at Mean Roof Height (q_h) (Eq. 6-15)

Mean Roof Height $h = \frac{70+80}{2} = 75\text{ft} \rightarrow K_z = 1.19$

$q_z = .00256(1.19)(1.0)(.85)(90^2)(1.0) = \boxed{20.97}$

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Equivalent Height of structure ($\bar{z}$) (Table 6-2)

$$\bar{z} = .6\,h = .6(80) = \boxed{48\text{ ft}} > z_{min} \checkmark$$

Intensity Turbulence (I_z) (Eq. 6-5)

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .2\left(\frac{33}{48}\right)^{1/6} = \boxed{.19}$$

$c = .2$ (Table 6-2)

Integral Length Scale of Turbulence (L_z) (Eq. 6-7)

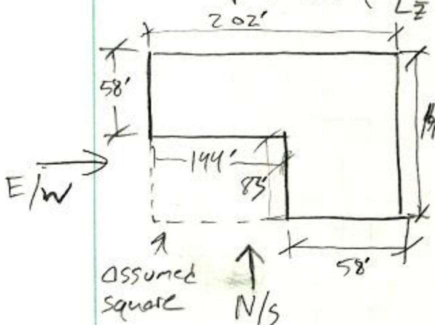
$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{48}{33}\right)^{1/5} = \boxed{538.91}$$

Table 6-2
$\ell = 500$
$\bar{\epsilon} = 1/5$

Background Response Factor (Q) (Eq 6-6)

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}}$$

B = horizontal dimension of building measured to wind direction
↓
now dependant on wind direction



North/South
$B = 202'$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{202+75}{538.91}\right)^{.63}}} = \boxed{.23}$$

East/West
$B = 141'$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{141+75}{538.91}\right)^{.63}}} = \boxed{.26}$$

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North/South

C_p (Fig. 6-6)

L = horizontal distance measured parallel to wind direction = 141'

$L/B = 141/202 = .7 < 1$

Windward Wall = .8 (with q_z)
Leeward wall = -.5 (with q_h)
Side wall = -.7 (with q_h)

East/West

C_p

$L/B = 202/141 = 1.43$

Windward = .8
Leeward = -.414 ← Linear interp
Side = -.7

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)

$GC_{pi} = +.18$, $-.18$ ← Enclosed Building

The remainder of calculations are done in an excel spreadsheet due to the fact that the are acording to height above ground.

Appendix B

Seismic Loading:

Reinforced Concrete Masonry Bearing Wall Schedule								
						Weight (psf)		
Wall Type	Thickness	Rebar	Spacing	Grout	Floor Location	CMU & Grout	Rebar	Total
A	12"	#7	16" O.C.	All cells	1st ext.	140	1.53	141.53
B	12"	#7	32" O.C.	All cells	1st int. center	140	0.77	140.77
C	8"	#6	32" O.C.	All cells	1st int. random	92	0.56	92.56
D	8"	#6	24" O.C.	Cells w/reinforcement	2nd ext.	69	0.75	69.75
F	8"	#5	32" O.C.	All cells	2nd int. typ.	92	0.39	92.39
G	8"	#6	32" O.C.	16" O.C.	3rd - 5th ext.	75	0.56	75.56
H	8"	#6	32" O.C.	Cells w/reinforcement	5th - 7th ext.	65	0.56	65.56
I	8"	#5	32" O.C.	16" O.C.	3rd - 5th int.	75	0.39	75.39
J	8"	#5	32" O.C.	Cells w/reinforcement	5th - 7th int.	65	0.39	65.39

Weight of Building							
						Weight (kips)	
Floor	Component	Weight (psf)	Height	Length	Area	Component	Total Floor
2	Wall A	141.53	9	687		875.08	2860.66
2	Wall B	140.77	9	174		220.45	
2	Wall C	92.56	9	91		75.81	
2	Wall D	69.75	4.33	687		207.49	
2	Wall E	92.39	4.33	391		156.42	
2	Steel					39.60	
2	Floor	69			13,679	943.85	
2	SDL	25			13,679	341.98	
3	Wall D	69.75	4.33	687		207.49	1985.20
3	Wall E	92.39	4.33	391		156.42	
3	Wall F	75.56	4.33	687		224.77	
3	Wall G	65.56	4.33	391		111.00	
3	Steel					1.40	
3	Floor	69			13,661	942.61	
3	SDL	25			13,661	341.53	
4	Wall F	75.56	8.66	687		449.54	1957.06
4	Wall G	65.56	8.66	391		221.99	
4	Steel					1.40	
4	Floor	69			13,661	942.61	
4	SDL	25			13,661	341.53	
5	Wall F	75.56	4.33	687		224.77	1956.27
5	Wall G	65.56	4.33	391		111.00	
5	Wall H	75.39	4.33	687		224.26	
5	Wall I	65.39	4.33	391		110.71	
5	Steel					1.40	
5	Floor	69			13,661	942.61	
5	SDL	25			13,661	341.53	

6	Wall H	75.39	8.66	687		448.53	1955.48
6	Wall I	65.39	8.66	391		221.41	
6	Steel					1.40	
6	Floor	69			13,661	942.61	
6	SDL	25			13,661	341.53	
7	Wall H	75.39	8.66	687		448.53	1955.48
7	Wall I	65.39	8.66	391		221.41	
7	Steel					1.40	
7	Floor	69			13,661	942.61	
7	SDL	25			13,661	341.53	
Roof	Wall H	75.39	4.33	687		224.26	1665.76
Roof	Wall I						
Roof	Parapet	75.39	4.33	687		224.26	
Roof	Steel					1.40	
Roof	Floor	69			13,661	942.61	
Roof	SDL	20			13,661	273.22	
Penthouse	Wall I	75.39	5	84		31.6638	48.46
Penthouse	Roof	50			240	12	
Penthouse	SDL	20			240	4.8	
						Total =	14384.37

Building Weight	
Floor	**Weight (kips)**
2	2994.55
3	1985.20
4	1957.06
5	1956.27
6	1955.48
7	1955.48
Roof	1665.76
Penthouse	48.46
Total	**14518.25**

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Seismic Ground Motion Values

$S_S = .125$

$S_1 = .049$

Soil Site Class = D

$S_{MS} = F_a S_s = 1.6(.125) = .2$ (eq 11.4-1)

$S_{M1} = F_v S_1 = 2.4(.049) = .1176$ (eq. 11.4-2)

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(.2) = .13$ (eq. 11.4-3)

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(.1176) = .0784$ (eq. 11.4-4)

Approximate Fundamental Period (T_a)

$T_a = C_t h_n^x = .02(80)^{.75} = .53\,sec$ (eq 12.8-7)

Calculate Siesmic Response Coefficient (C_s)

$C_s = \frac{S_{D1}}{T_a\left(\frac{R}{I}\right)}$ for $T_a < T_L$ or $\frac{S_{DS}}{\left(\frac{R}{I}\right)}$ pick the minimum

$\frac{.0784}{.48\left(\frac{2}{1}\right)} = .082$ $\quad \frac{.13}{\frac{2}{1}}$

Recalculation with new R-value shown on next page of calculations

Seismic Base Shear

$V = C_s W = .065(15,775\ kips) = \boxed{1,025.4\ kips}$

Story Shear Values

Shear force at level x $\rightarrow F_x = C_{vx} V$ (eq. 12.8-11)

$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k}$ $\quad k=1.015$ (eq 12.8-12)

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New Cs → R now 3.5 instead of 2.0

$$C_s = \frac{S_{DS}}{\frac{R}{I_e}} = \frac{.13}{\frac{3.5}{1}} = .037 \checkmark$$

or

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{.0784}{.53\left(\frac{3.5}{1}\right)} = .0422$$

Min

$$C_s = .044(.13)(1) \leq .01 \rightarrow .01$$

$$V = C_s W$$

$$= .037(14518.26)$$

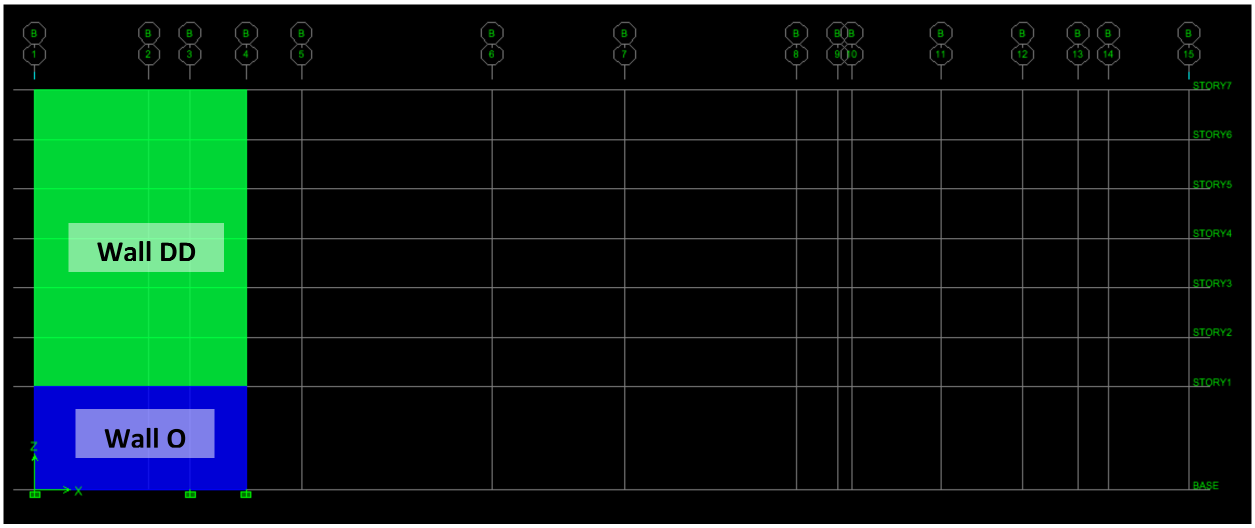
$$= \boxed{537.18 \text{ Kips}}$$

compared to 943.69 Kips with R=2.0

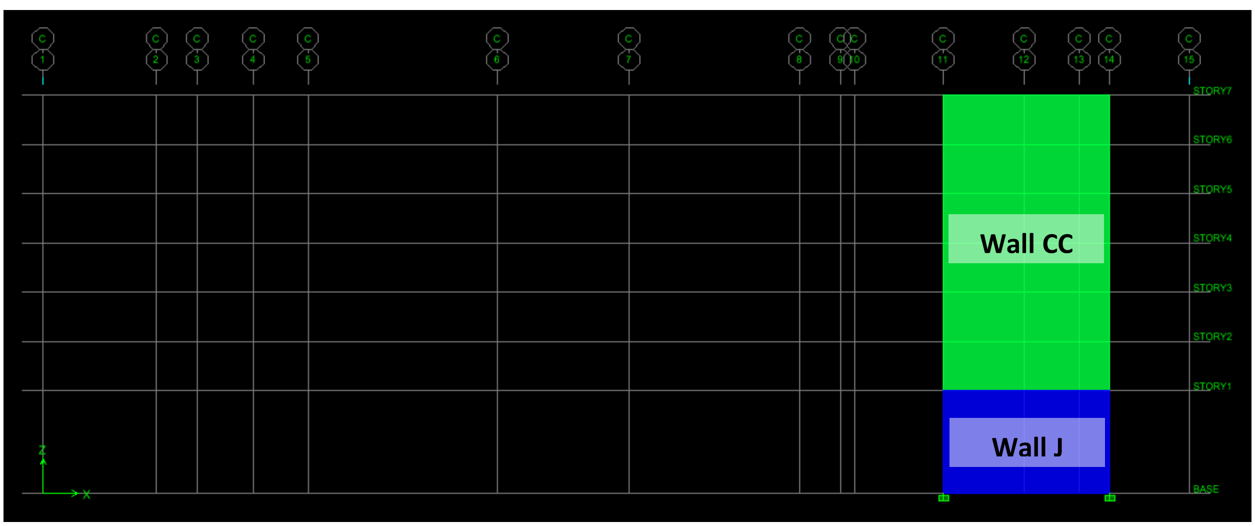
Appendix C

ETABS Model:

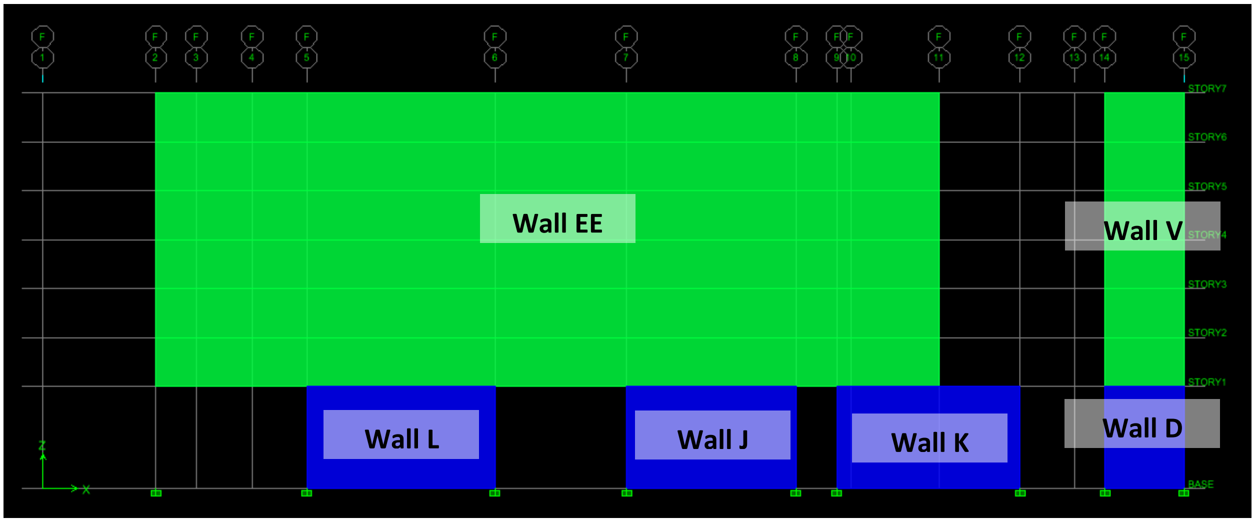
Elevation Views: Resisting forces in the X-Direction



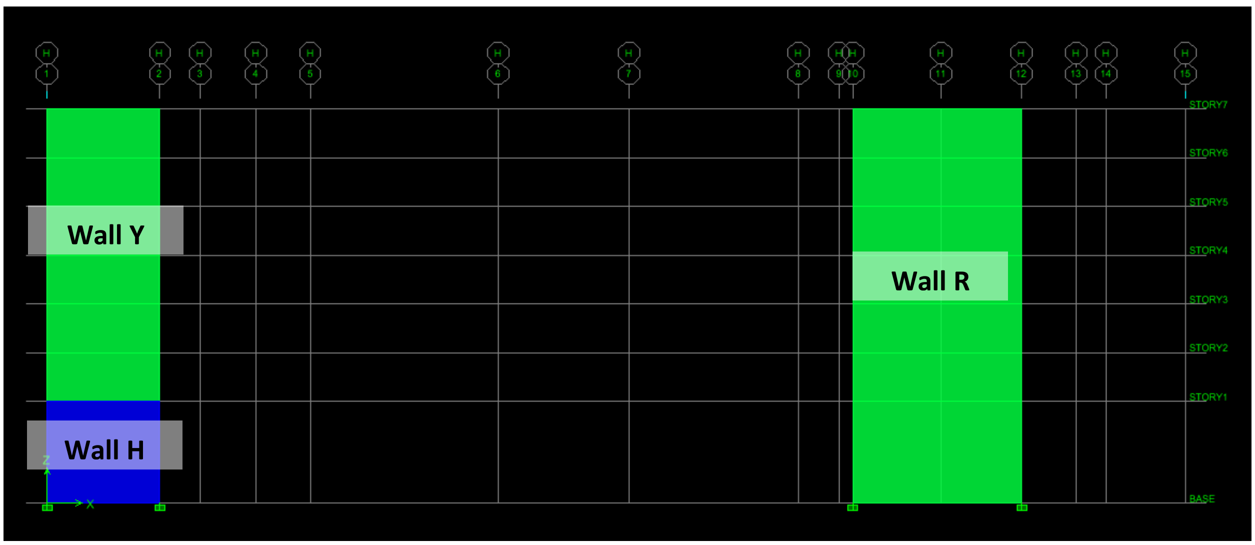
Elevation B



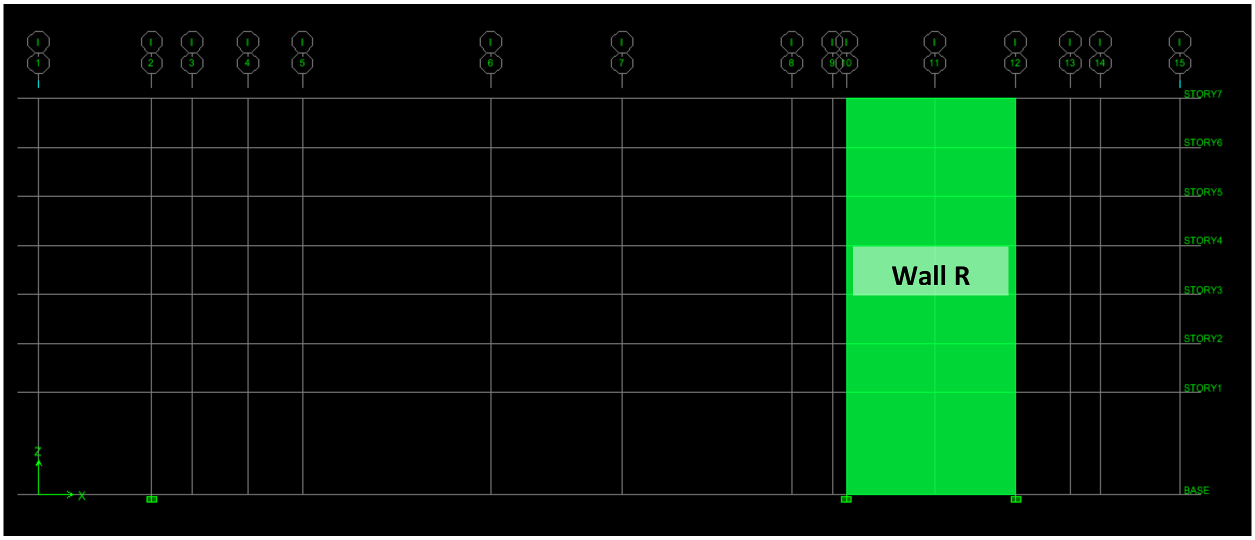
Elevation C



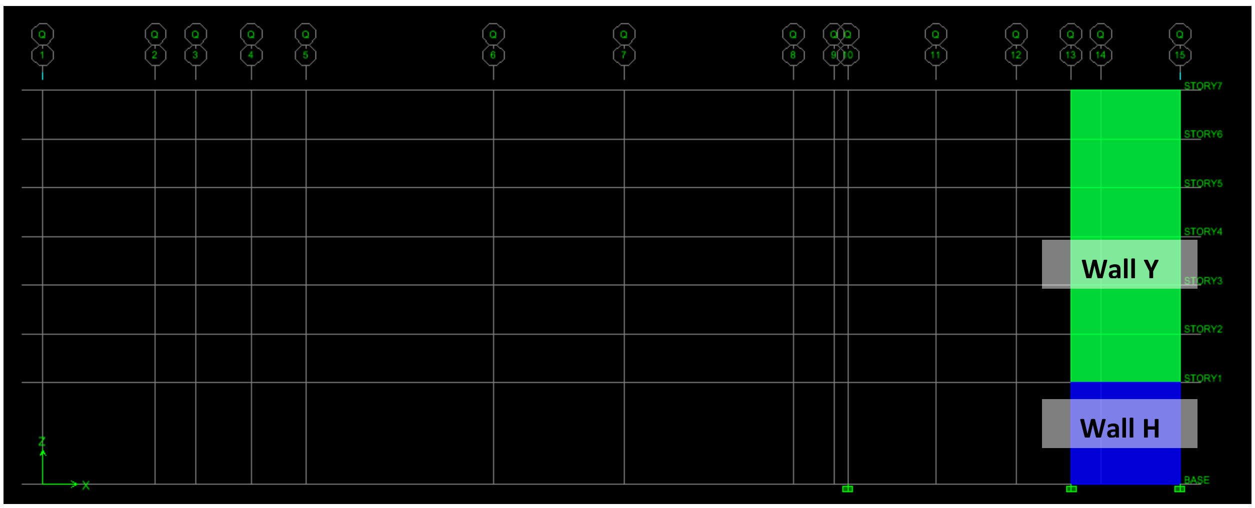
Elevation F



Elevation H

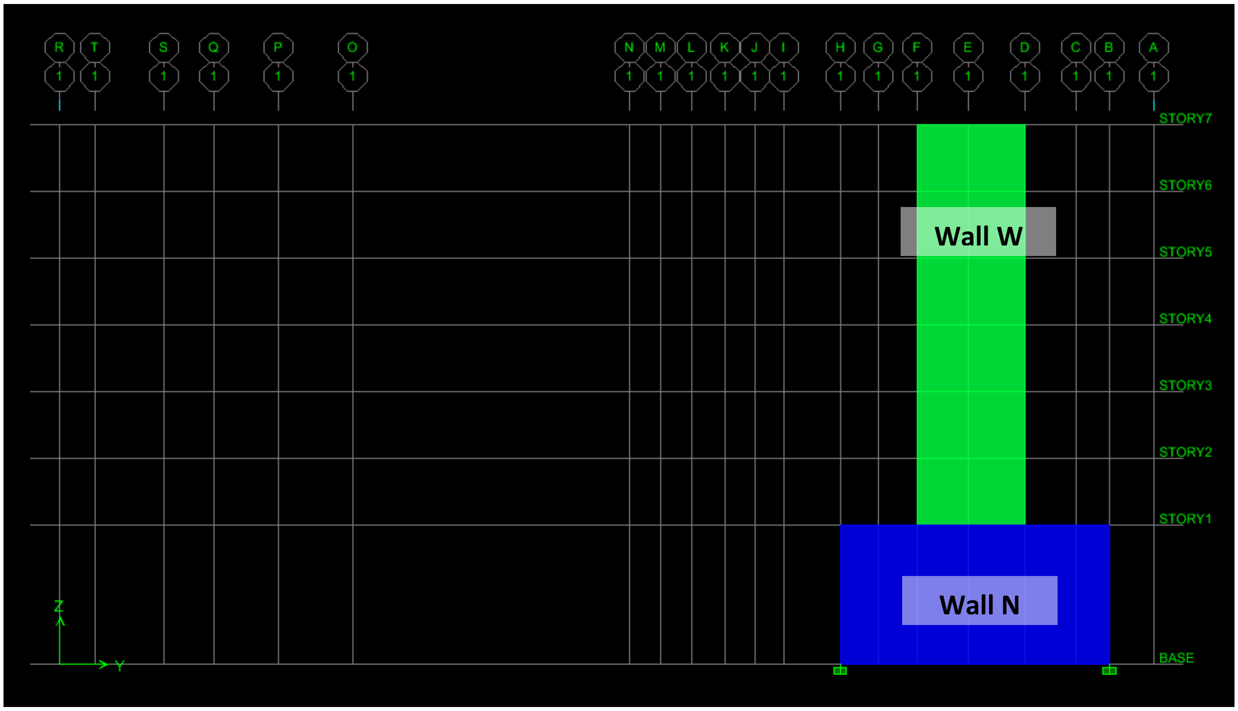


Elevation I & O & P

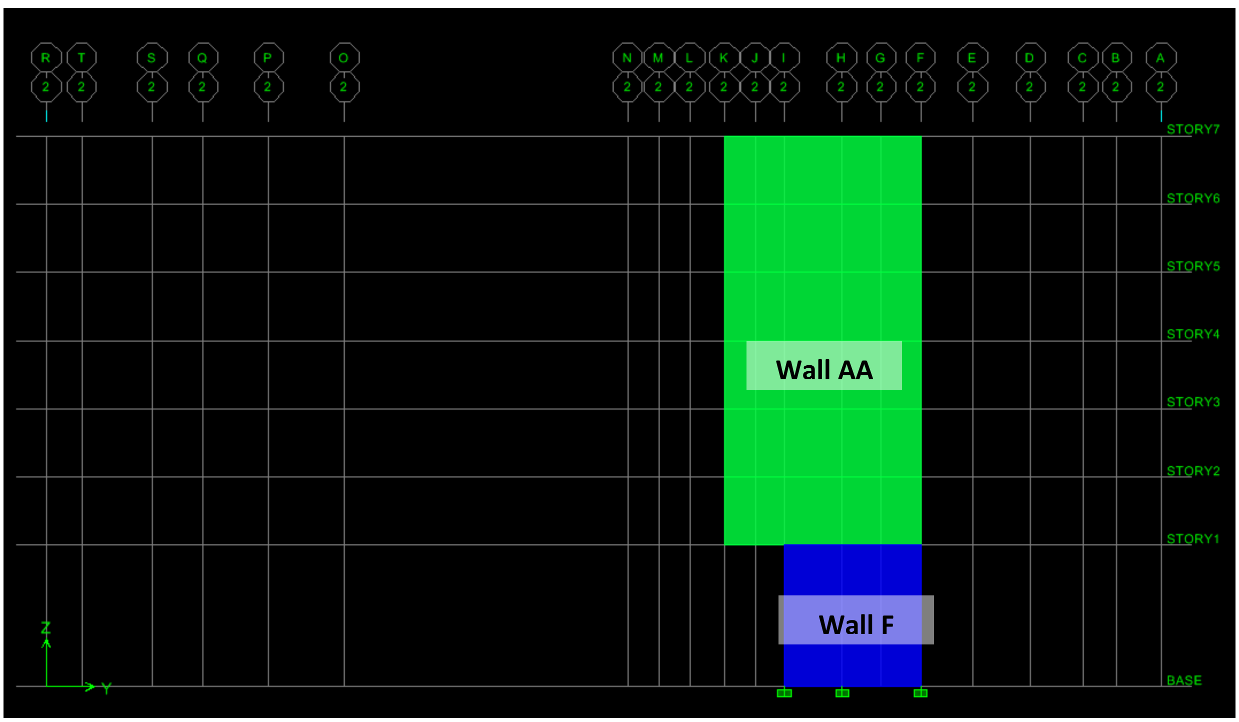


Elevation Q

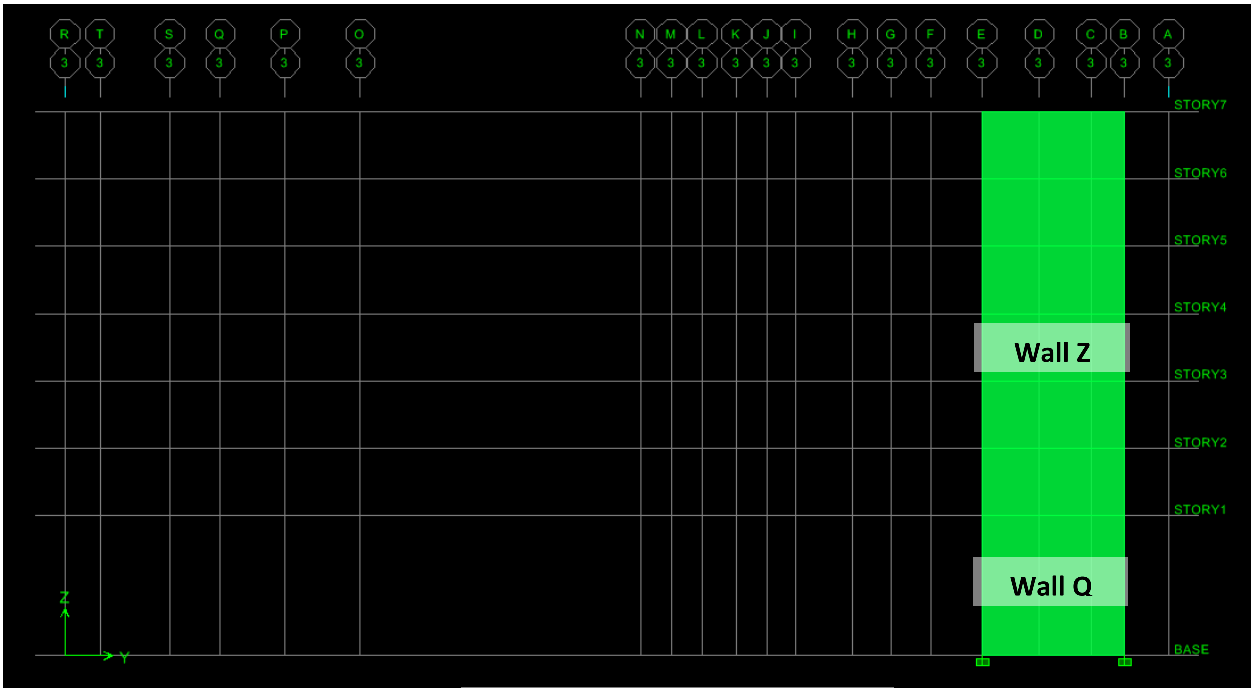
Elevation Views: Resisting forces in the Y-Direction



Elevation 1

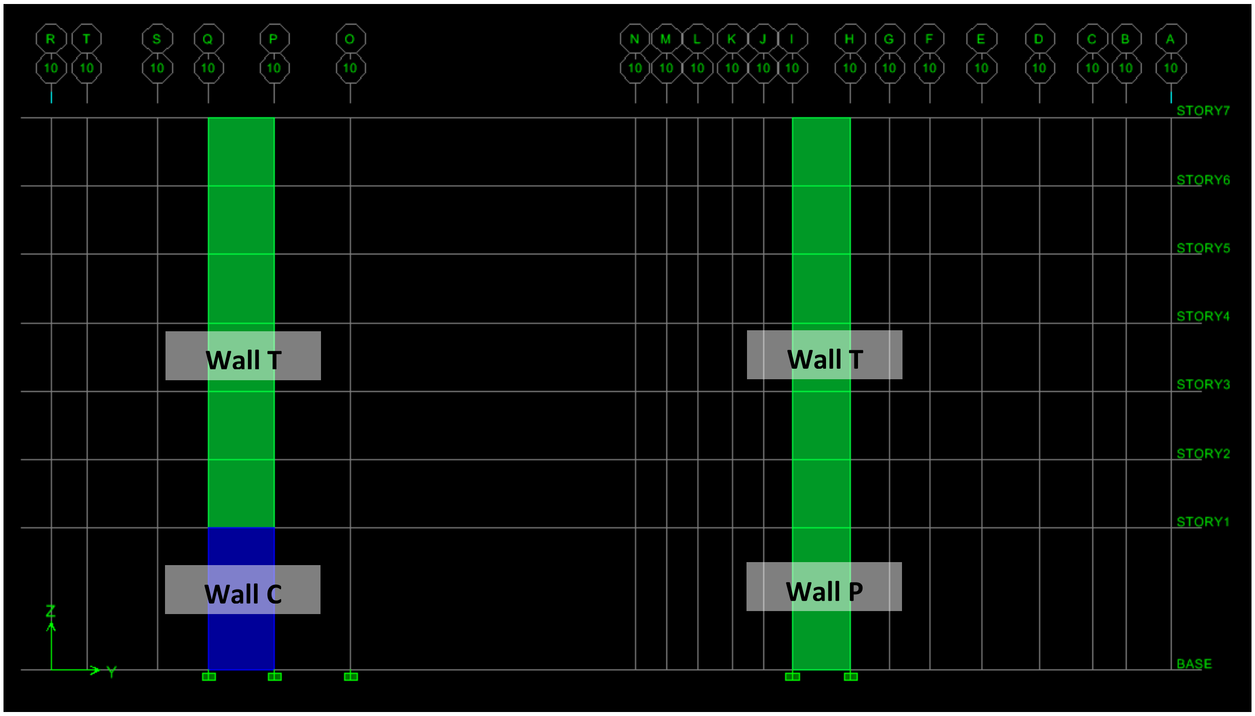


Elevation 2

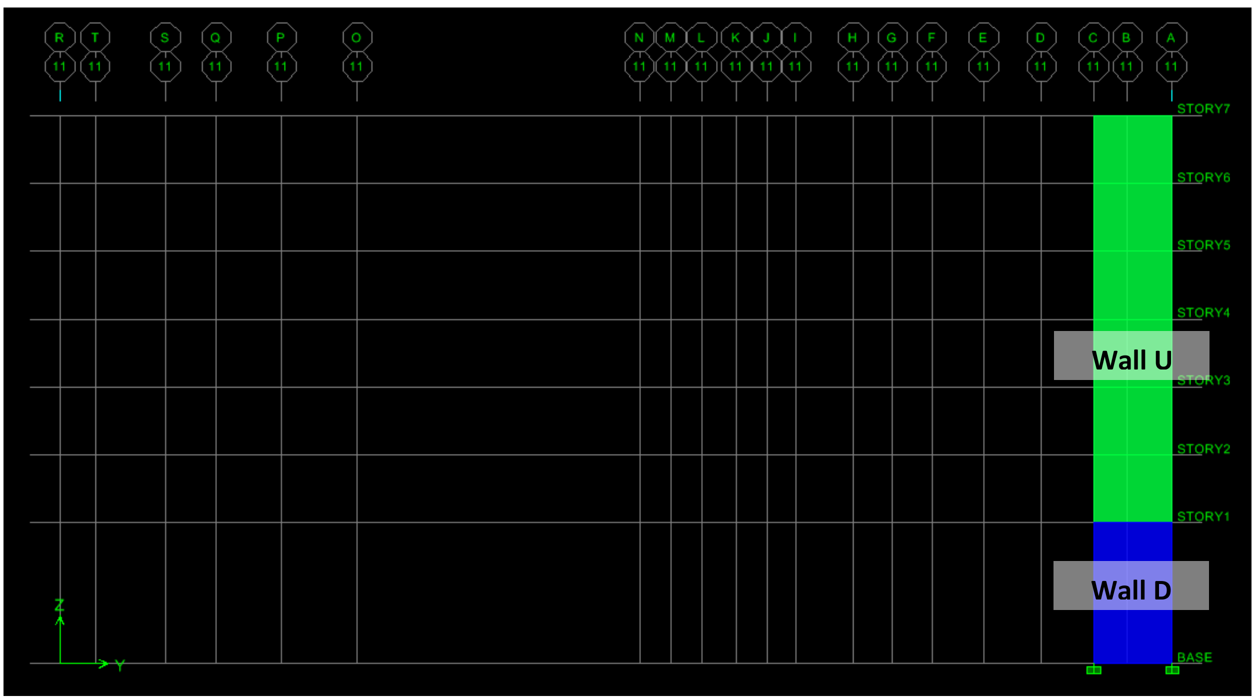


Elevation 3

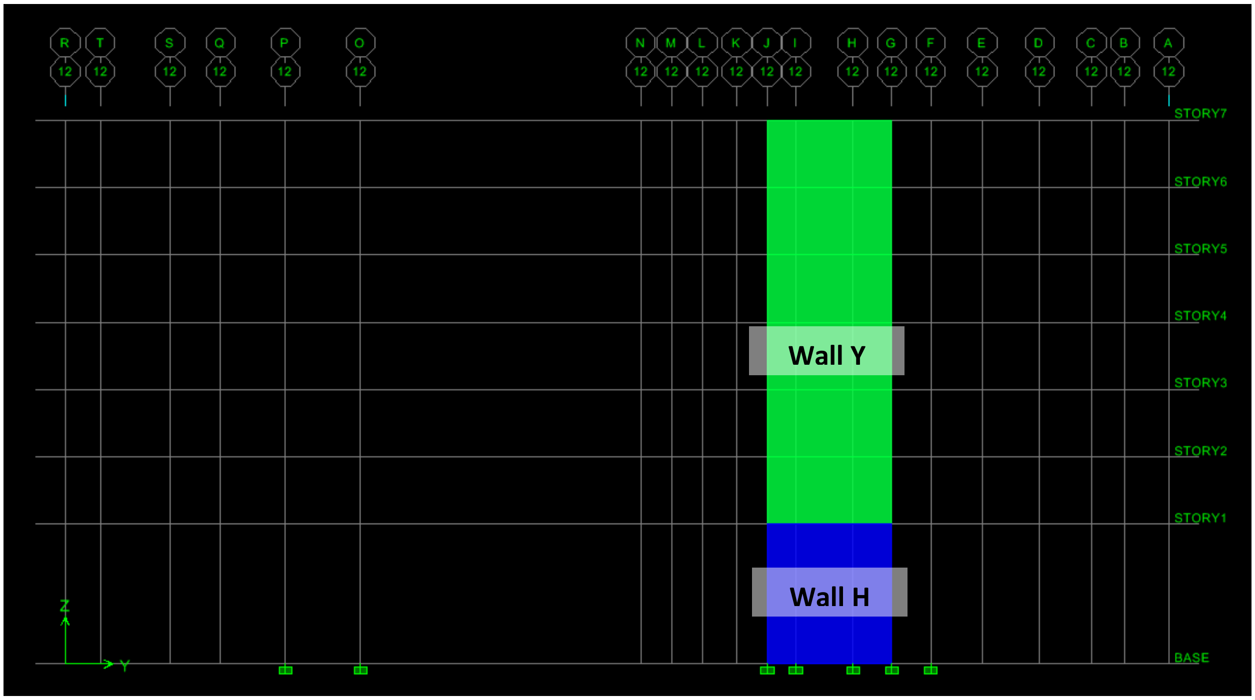
Elevation 4



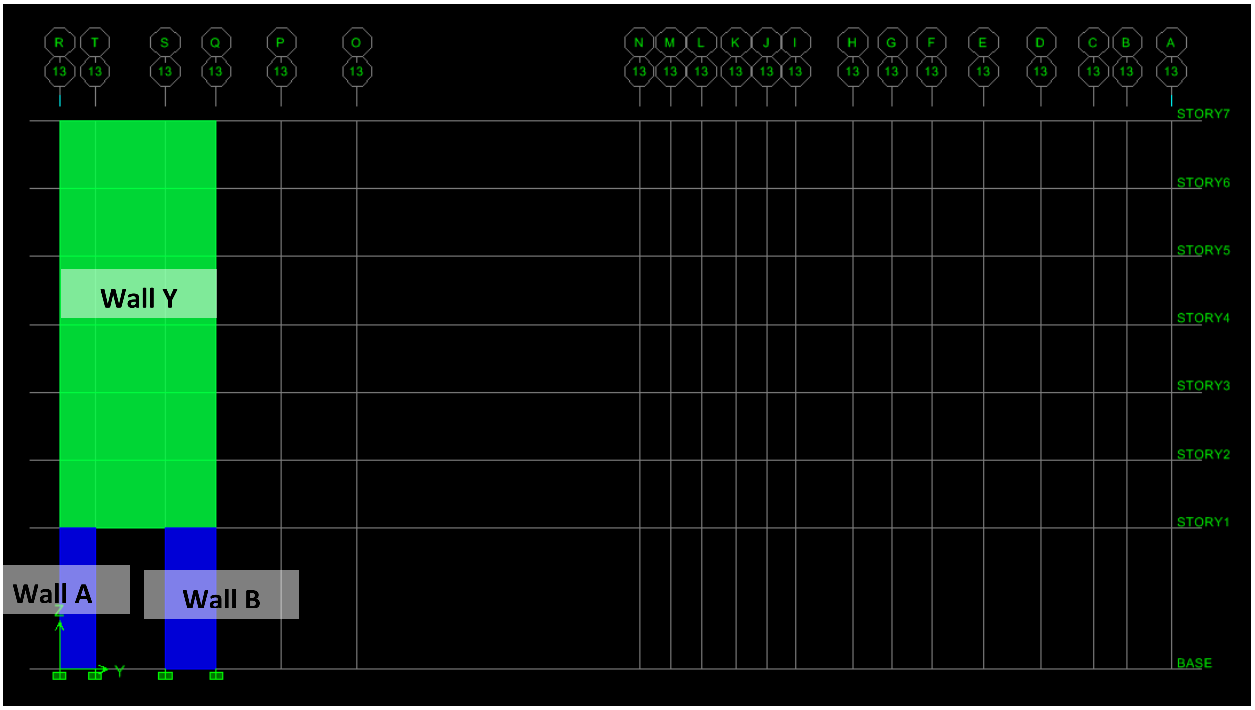
Elevation 10



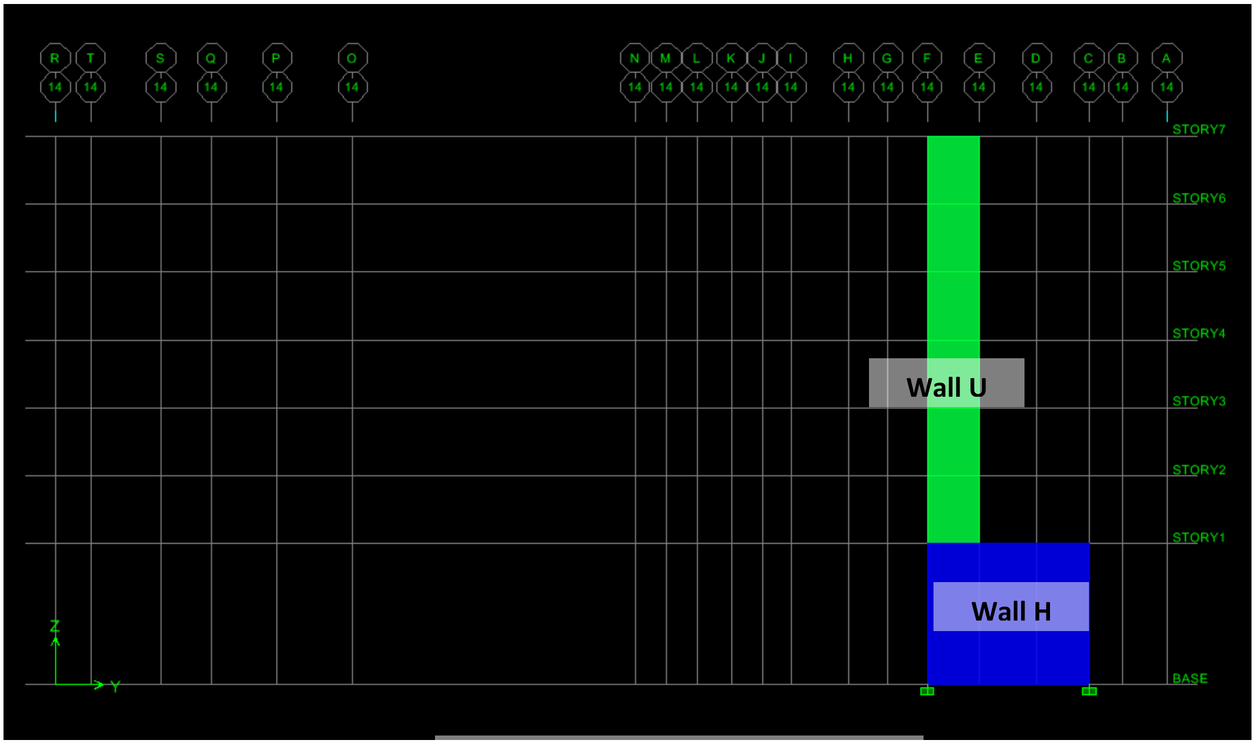
Elevation 11



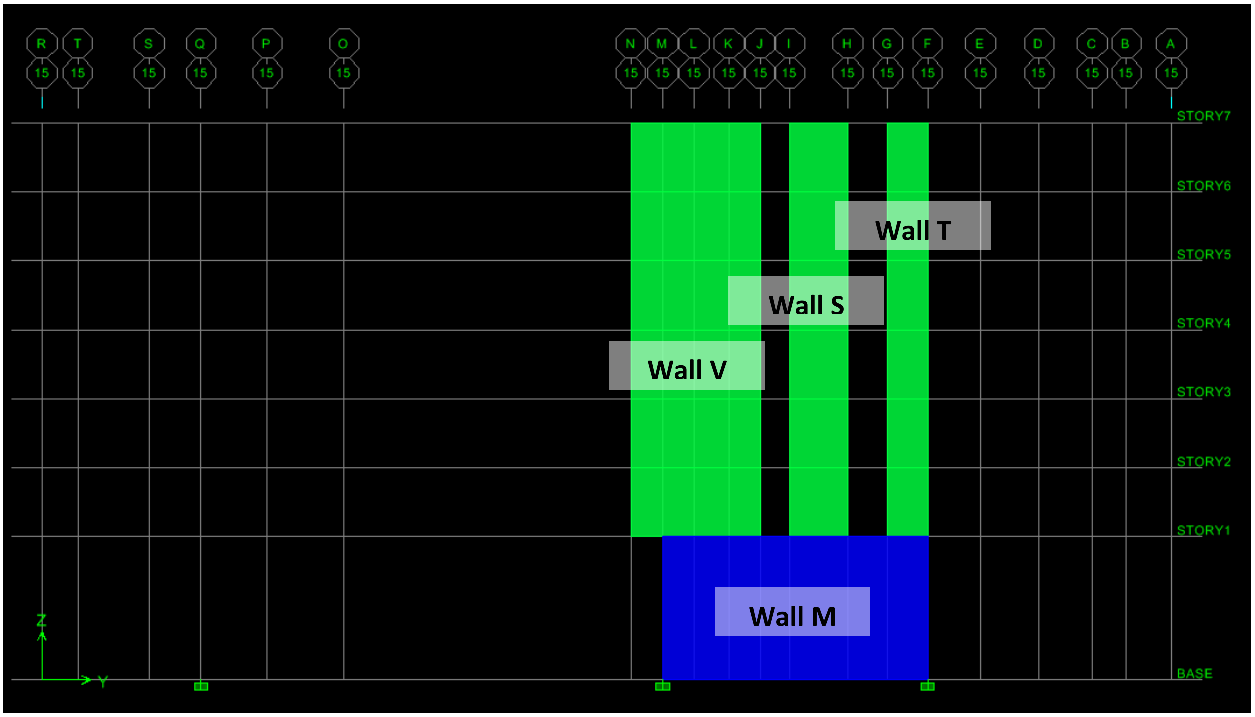
Elevation 12



Elevation 13



Elevation 14



Elevation 15