Hyatt Place North Shore
Pittsburgh, PA

Technical Assignment #1

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Executive Summary:

In this report, the existing structural conditions of the 7 story Hyatt Place North Shore are analyzed. The 178 room hotel is located on prime real estate in between Heinz Field and PNC Park and not far from the new Rivers Casino. The 70 feet tall, 108,000 square foot structure has simple reinforced concrete masonry bearing walls working in combination with an 8" 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.

A large portion of this report was dedicated to studying the loads that are on the structure. This analysis started with gravity loads. ASCE 7-05 was used to determine live loads, superimposed dead loads, and roof snow loads. Overall, most values used by the design engineer matched up directly with those of the design code.

Next in line is the determination of lateral forces. ASCE 7-05 is once again used to determine what loads need applied to the structure. It is found that seismic loading controls over wind forces with a base shear of 943.69 kips while wind forces pale in comparison with a base shear of 399.20 kips. Both of these values differ from the engineer's calculated values, seismic due to a difference in building weight and wind because of different methods being employed.

The last part of the report is a spot check of existing structural elements. In this portion a large beam on the first floor, a column carrying the weight from a 44 foot truss over meeting space, and the existing precast concrete plank structure are all determined to be adequate.

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 is 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 rigid diaphragm.

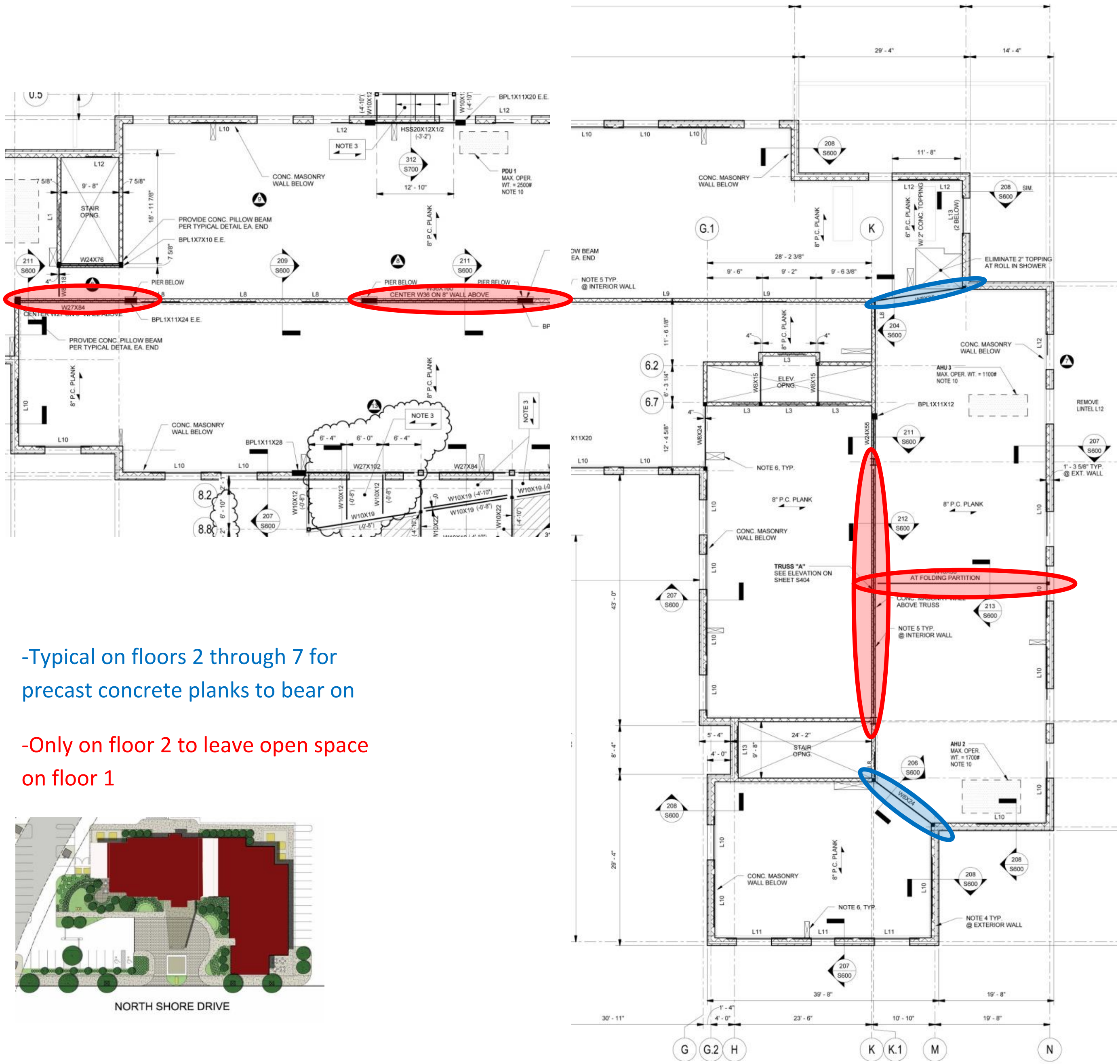


Figure 3: View of structural steel used

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 in 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.

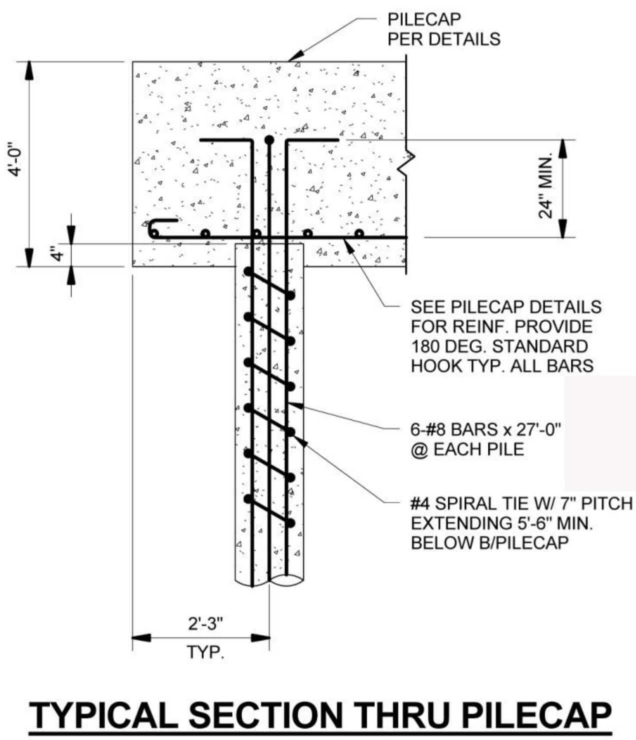


Figure 4: Section through typical pile cap

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, *Figure 5* 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 *Figure 5*. *Figure 6* shows the orientation of the walls on a typical upper level plan, the capacity of each of these wall types can be determined. *Figure 7, 8, and 9* 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

Figure 5: Reinforced concrete masonry bearing wall schedule

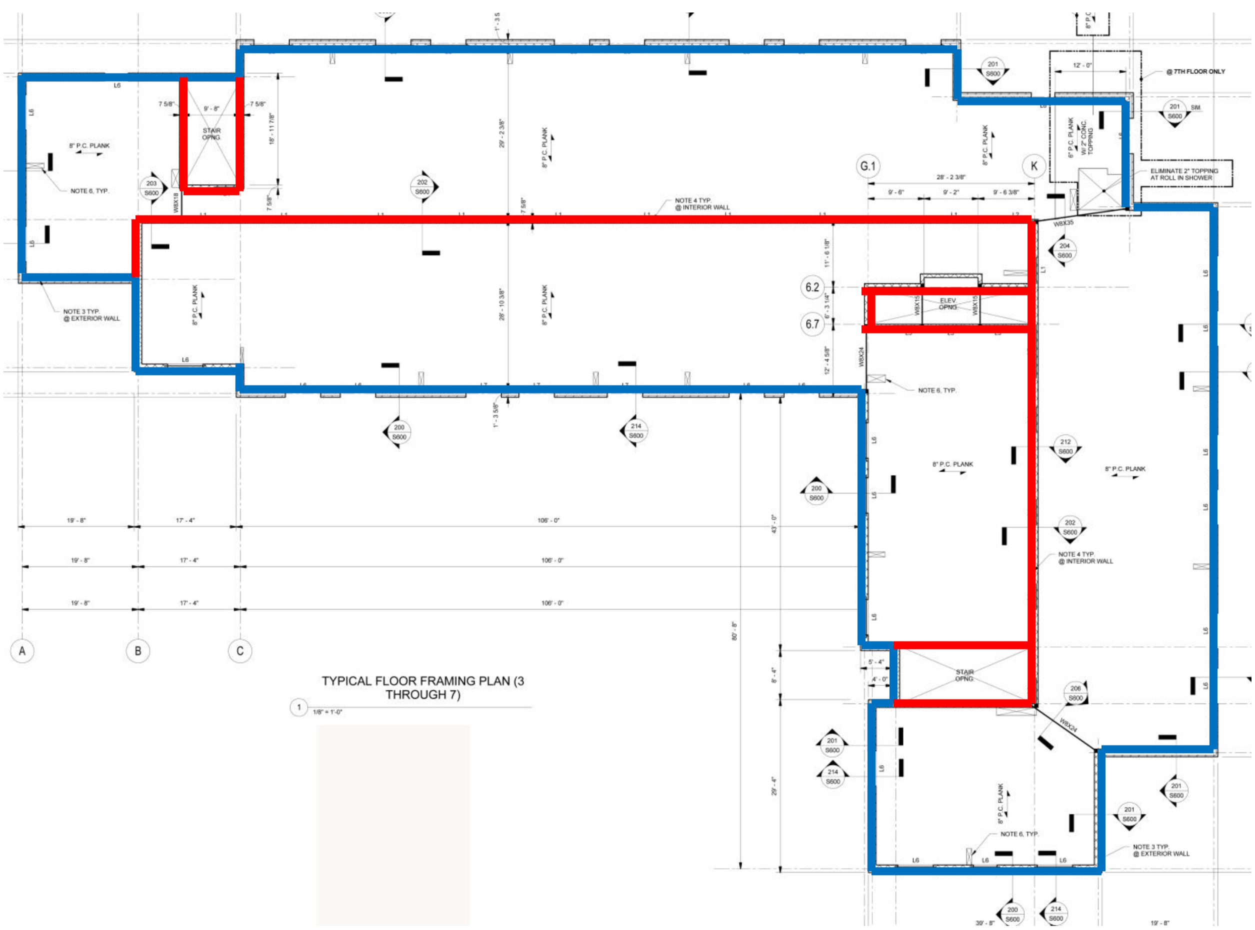


Figure 6: Typical bearing wall layout, floors 3 through 7

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.

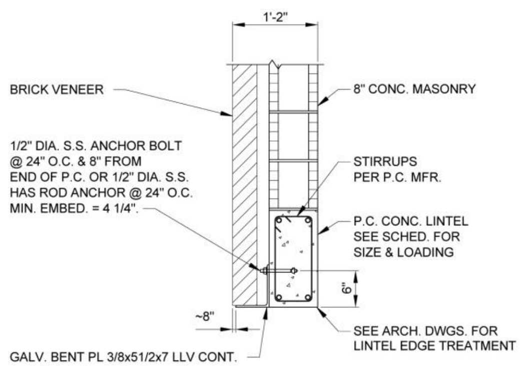
Figure 7: Precast Lintel schedule for load bearing masonry walls

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.

Figure 8: Brick lintel schedule



TYPICAL LINTEL DETAIL "3"

Figure 9: 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 10.* There are also concrete masonry piers also 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.

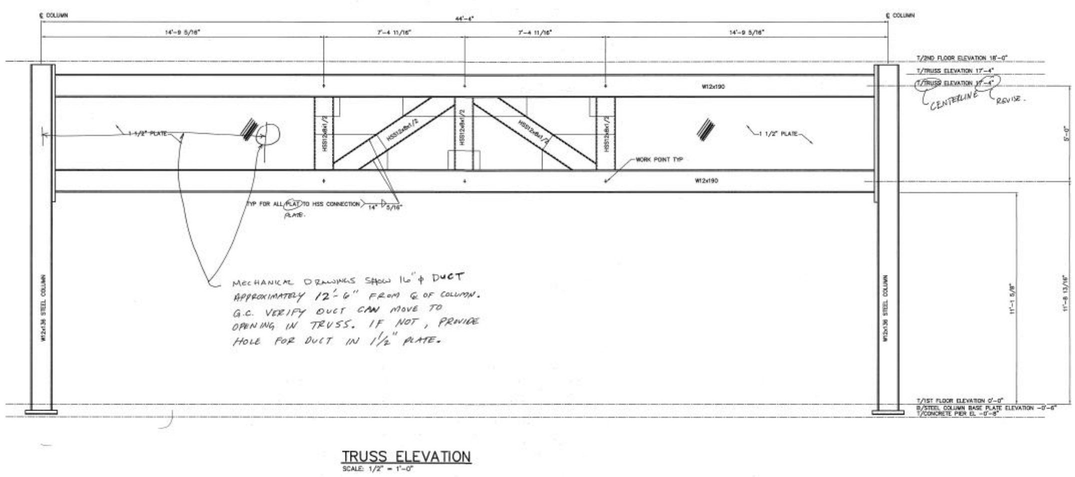


Figure 10: Transfer girder in first floor meeting space

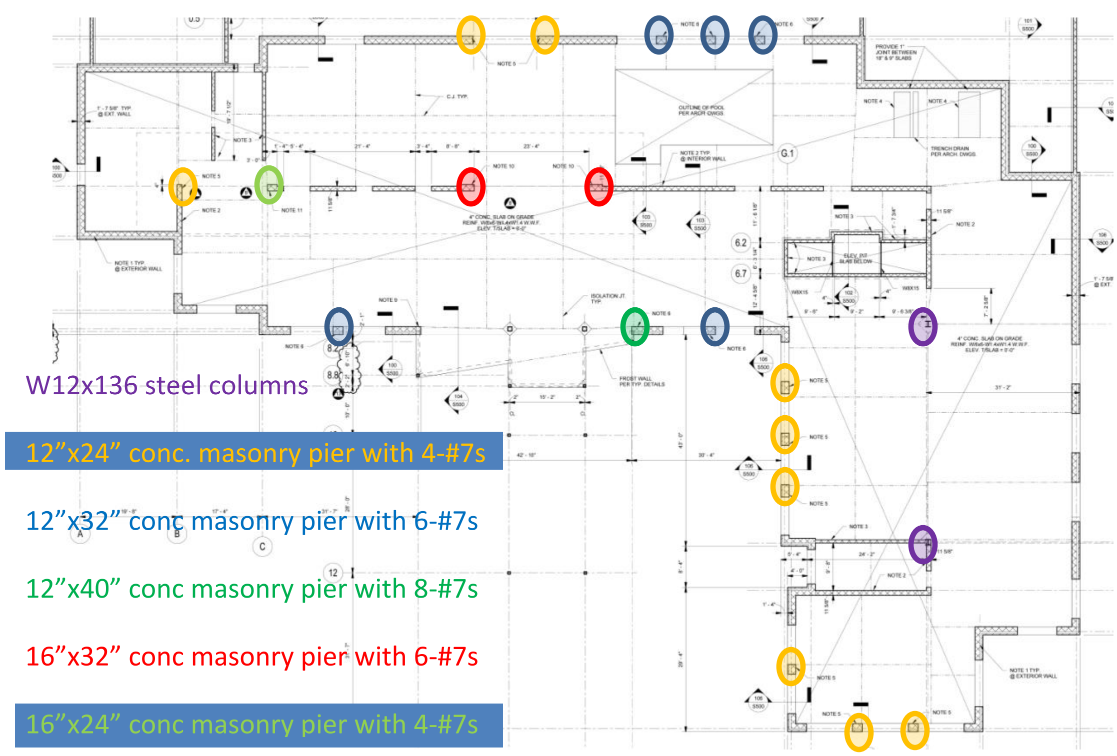


Figure 11: Location of masonry piers on first floor

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.

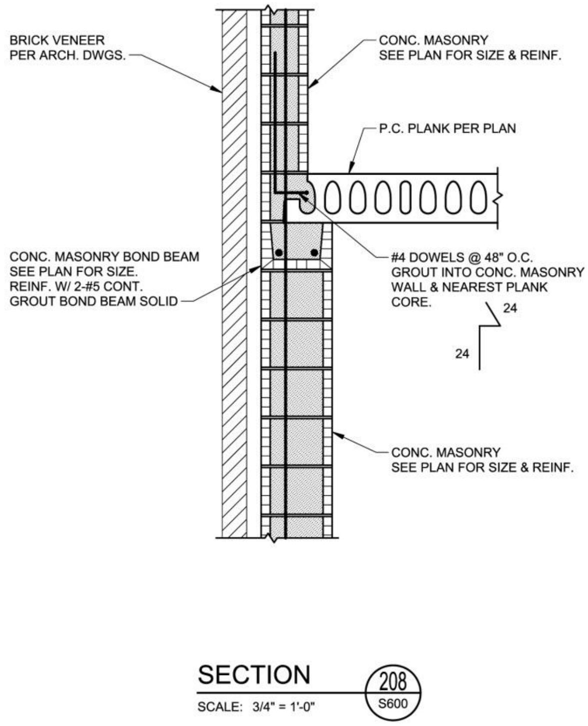


Figure 12: Typical plank and masonry wall connection

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 rigid diaphragm. So the loads travel into the rigid 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 Engieers

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)		
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Figure 5: 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		

Figure 13: Floor live loads

Snow Load:

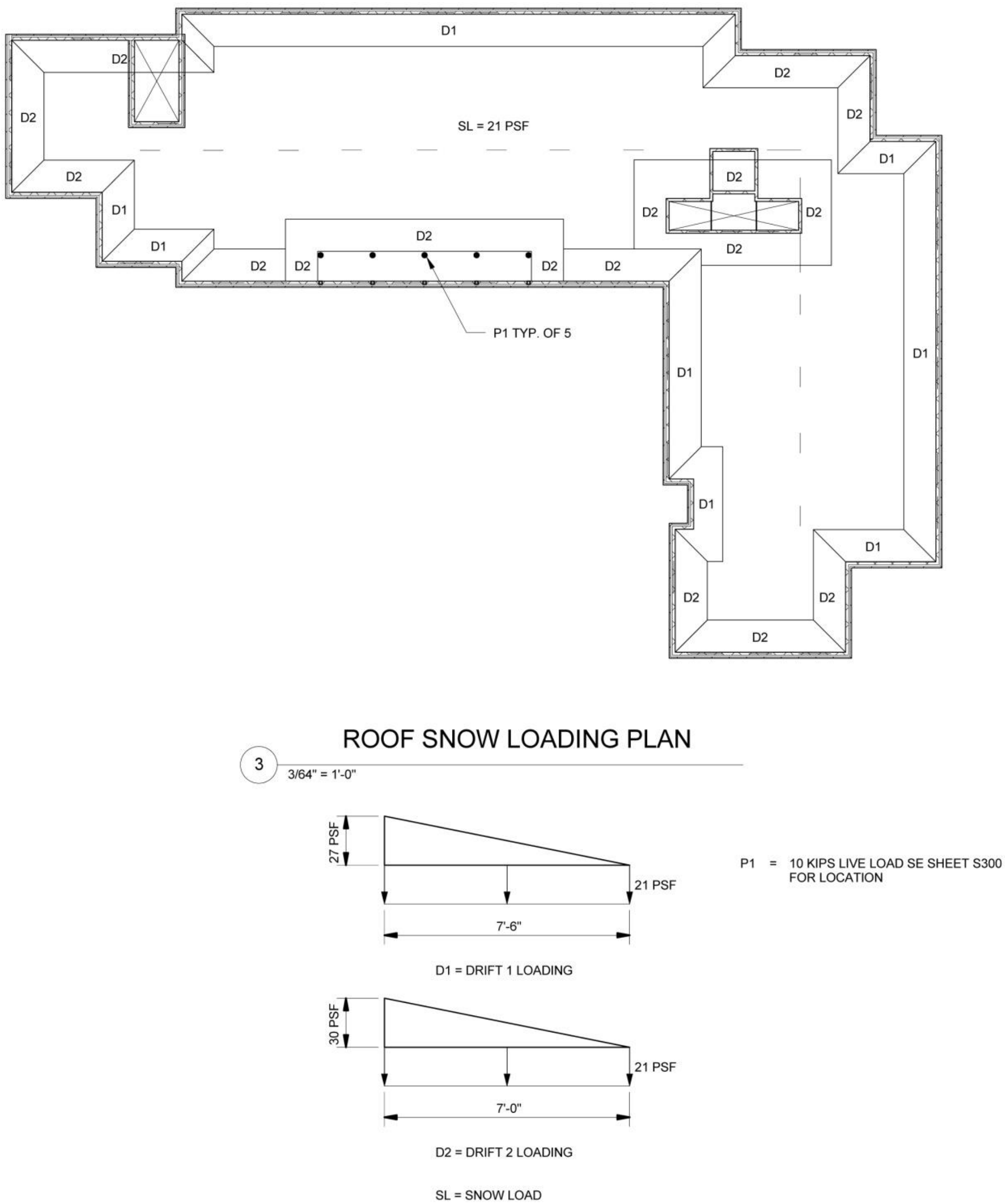


Figure 14: Roof snow loading plan as calculated by AES

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

Figure 15: 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*

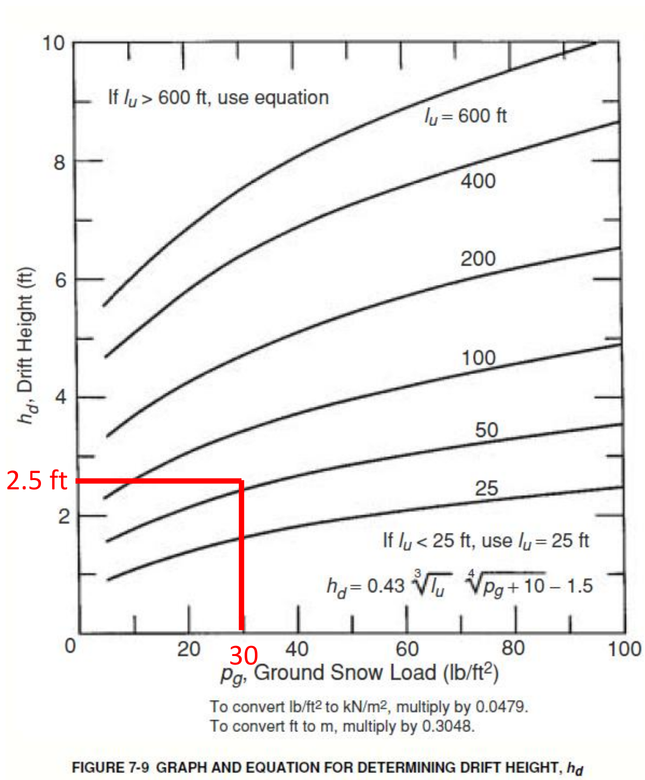


Figure 16: Graph and equation for determining drift height

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**

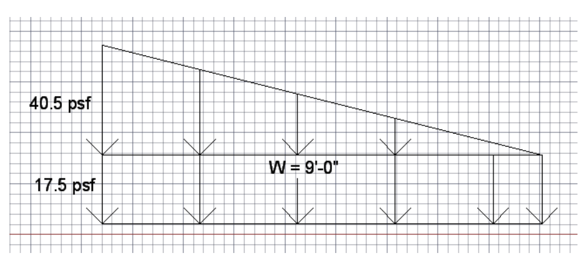
<u>Typical Parapet Wall Drift Height</u>

Drift Height = 2.5ft – from *Figure 16*

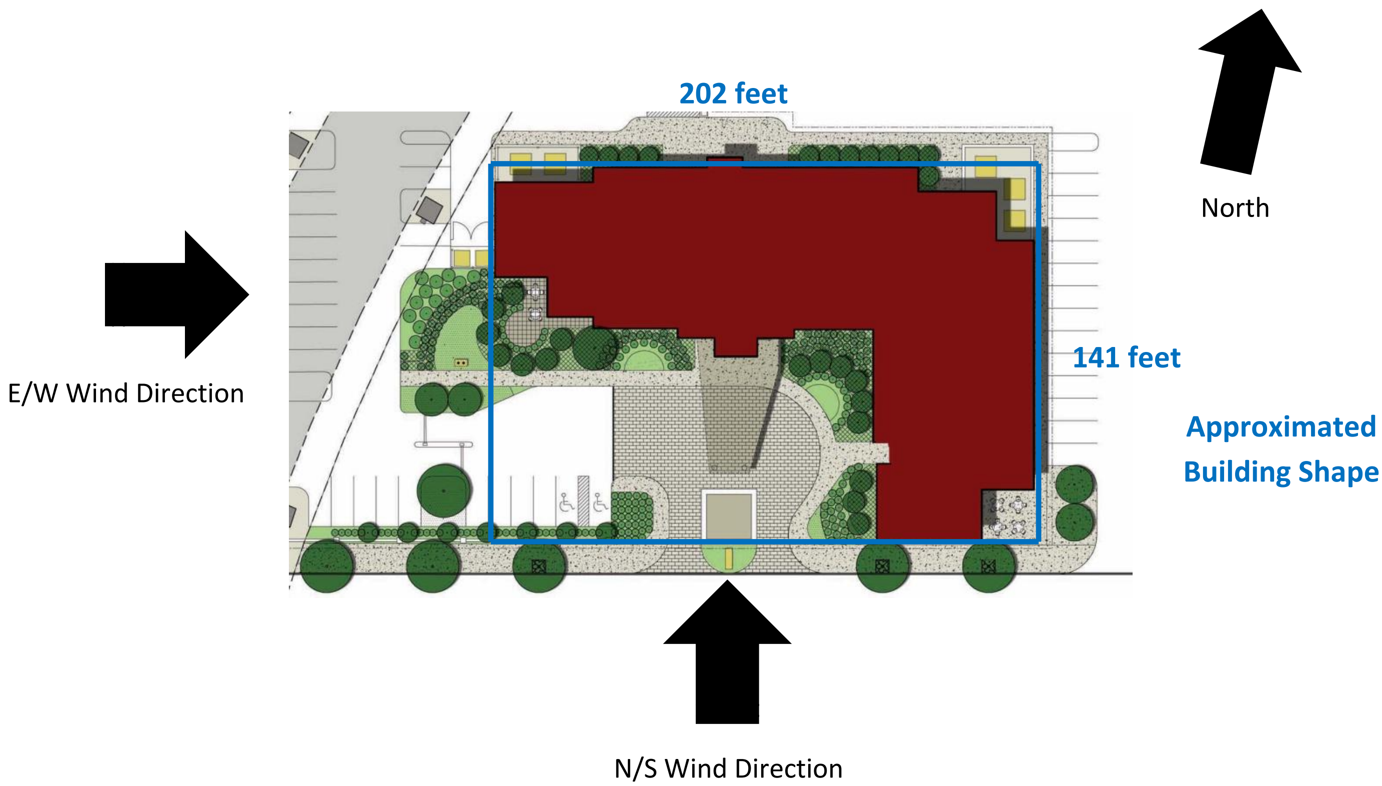
Max allowable =.75 h_d = .75*2.5 = **2.25ft**

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

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



Wind Loading



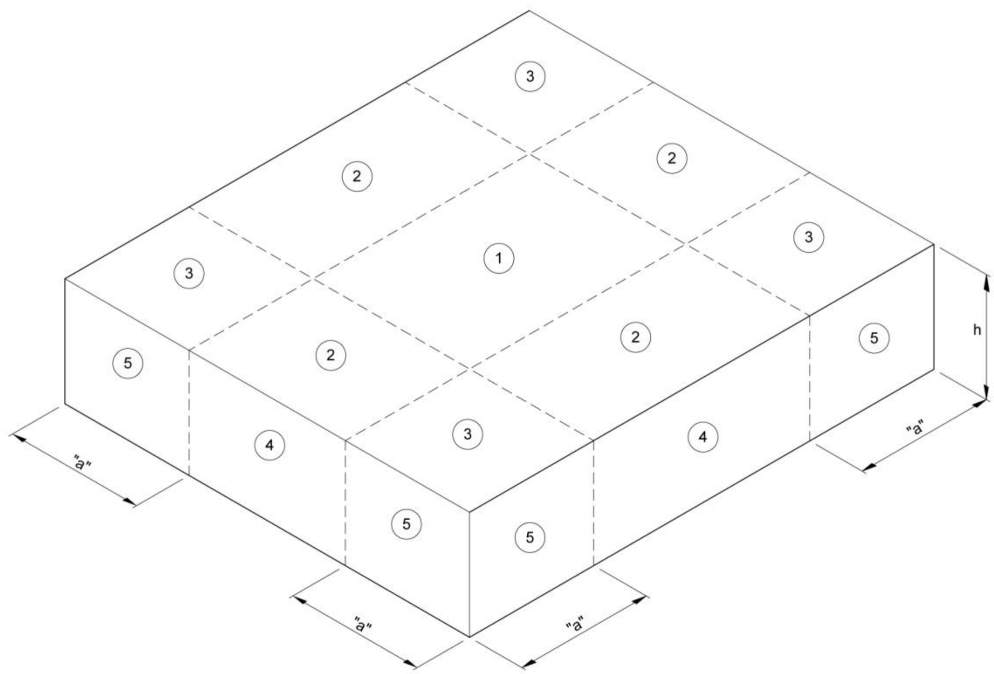
Loads to be applied to the hotel's lateral system must also be determined, so that in later reports the lateral system can be studied. With the system provided, wind applies pressure to the building enclosure. The exterior walls are load bearing and begin the transfer of energy down through toward the 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 17* 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 *Figure 18*. 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 17: 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

Figure 18: 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**

Figure 19: 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

Figure 20: Wind loads in the North/South direction

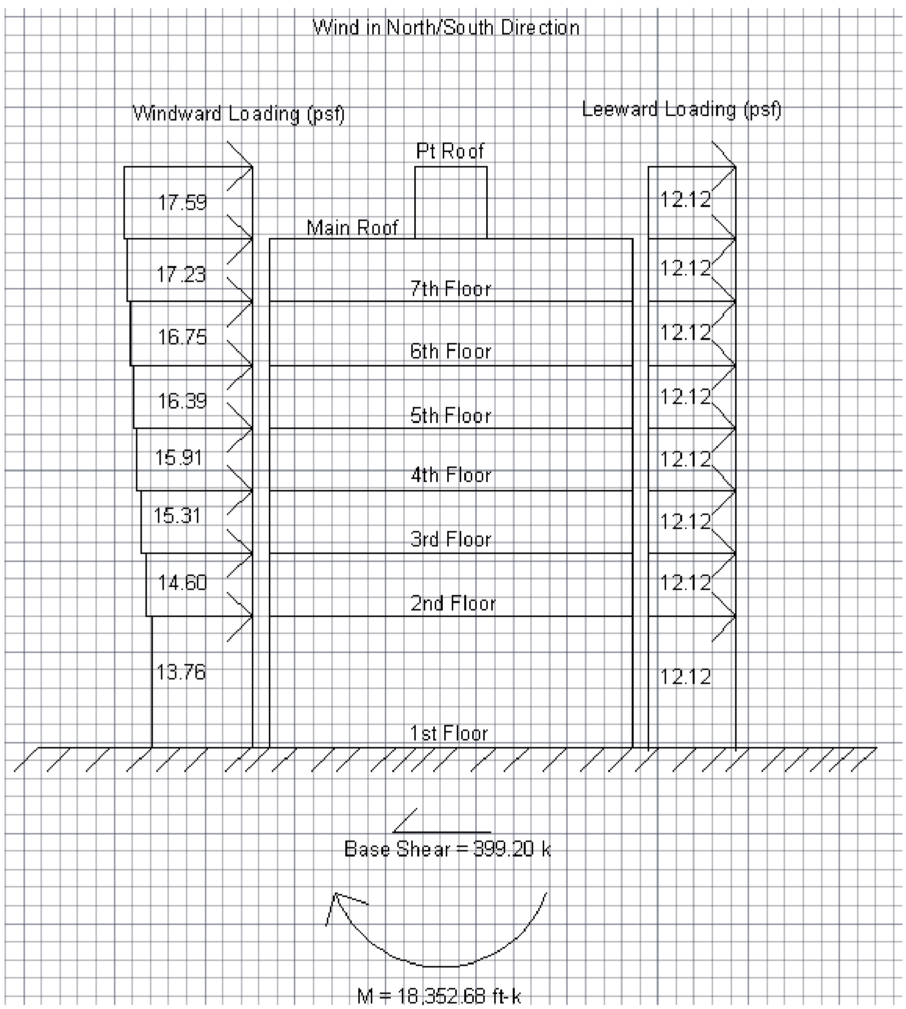


Figure 21: Wind pressures in the North/South direction

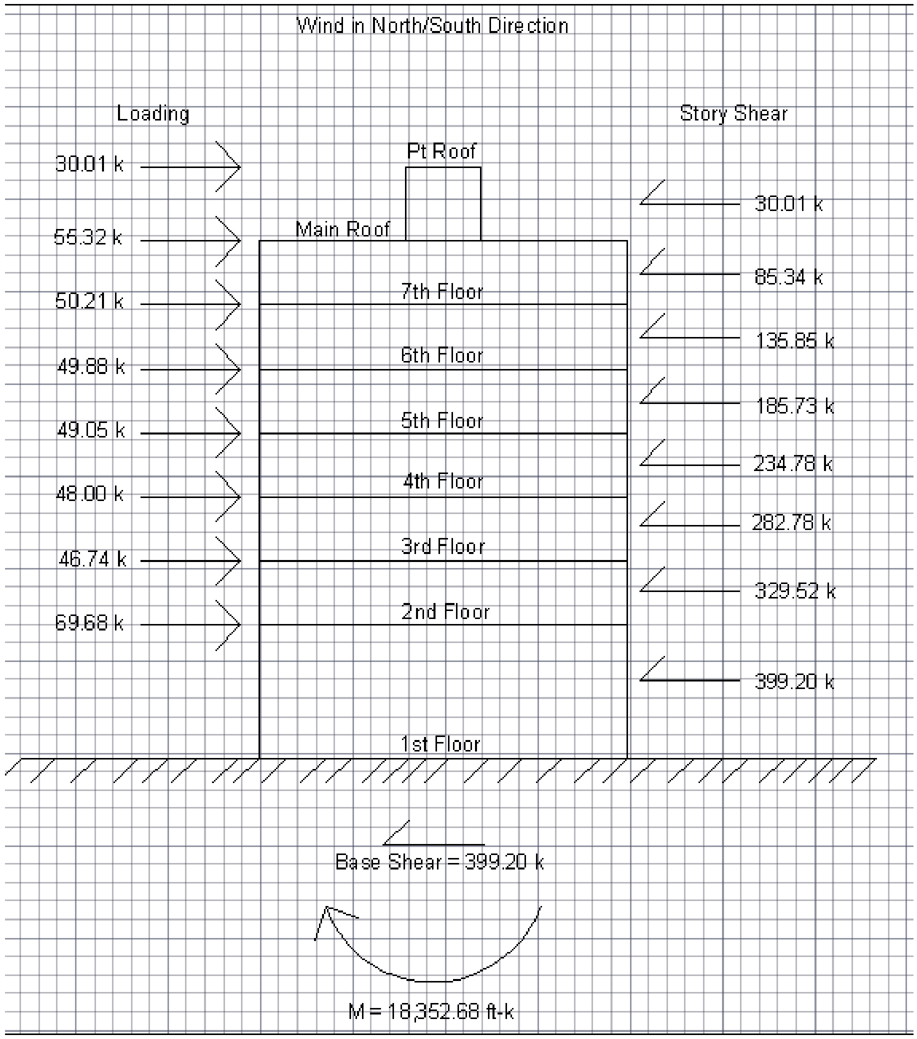


Figure 21: Wind pressures in the North/South direction

Seismic Loading

Seismic loading must also be taken into consideration when checking the lateral system. In this case the seismic loading controls and will be the load looked at when designing the lateral force resisting systems. The values in *Figure 22* 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 943.69 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		Rein. Masonry Shear Walls	Table 12.2-1
Response Modification Factor	R	2	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 12.8-3
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	0.065	Eq. 12.8-2
Structure Period Exponent	k	1.015 (2.5 sec. > T > .5 sec.)	Sec 12.8.3
Seismic Base Shear	**V**	**943.69**	**Eq. 12.8-1**

Figure 22: 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 *Figure 23,* and *Figure 24* 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	6.09	6.09	487.47
Main Roof	1665.8	70	1.015	124275.59	0.19	182.90	189.00	13229.80
7th Floor	1955.5	61.33	1.015	127567.60	0.20	187.75	376.75	23105.83
6th Floor	1955.5	52.66	1.015	109283.70	0.17	160.84	537.59	28309.24
5th Floor	1956.3	44	1.015	91103.06	0.14	134.08	671.67	29553.35
4th Floor	1957.1	35.33	1.015	72940.79	0.11	107.35	779.02	27522.72
3rd Floor	1985.2	26.66	1.015	55597.21	0.09	81.83	860.84	22950.11
2nd Floor	2994.6	18	1.015	56290.31	0.09	82.85	943.69	16986.42
Total	**14518.25**			**641198.45**				**162144.95**

Figure 23: Seismic story shear and moment calculations

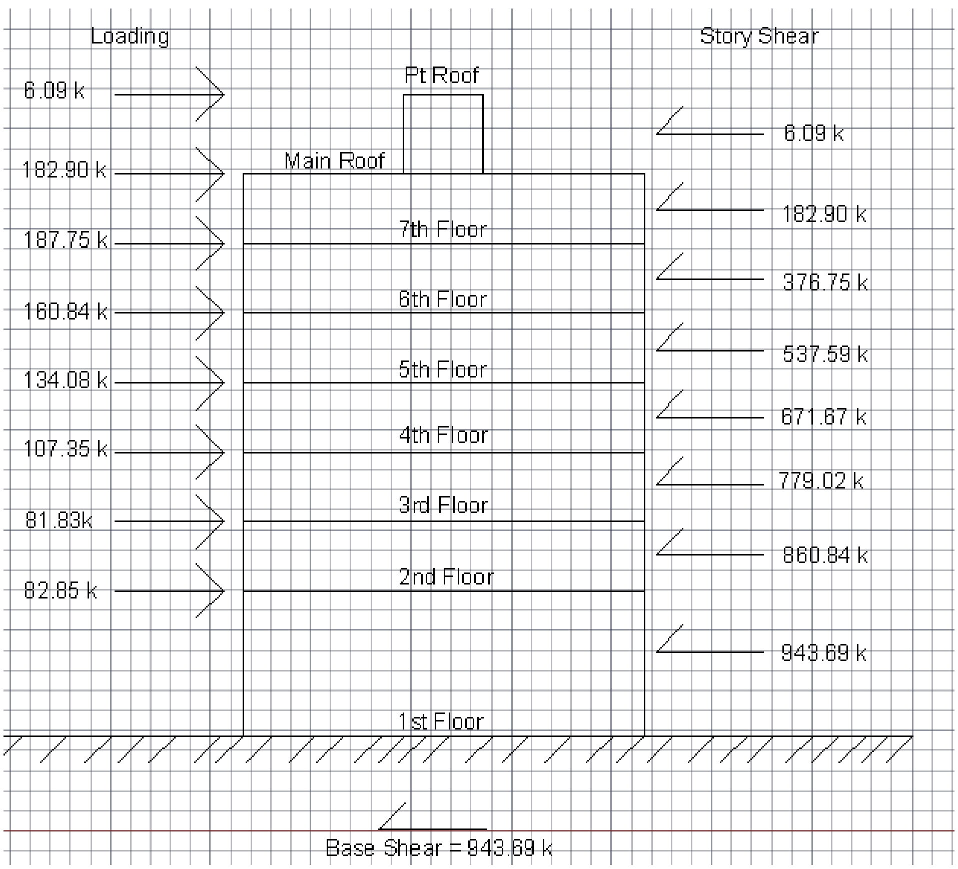


Figure 24: Seismic loading diagram

Spot Checks

Spot checks were performed on 3 different structural elements to confirm the engineer of record's design. Only gravity loads were applied in these calculations, but this is a good approximation to size the members. The first member checked was a W36x160 steel beam on the first floor allowing for larger open space, *Figure 25*, so it is sized to handle the entire load coming down from the bearing walls above it. The next element is a W12x136 steel column also on the first floor and acting to help create open space for a large meeting room, *Figure 25 and 26.* Lastly a typical section of section of the precast concrete plank floor system is checked to make sure it can handle the load over the maximum span length of 30'-6" in the East wing of the hotel tower, *Figure 27*.

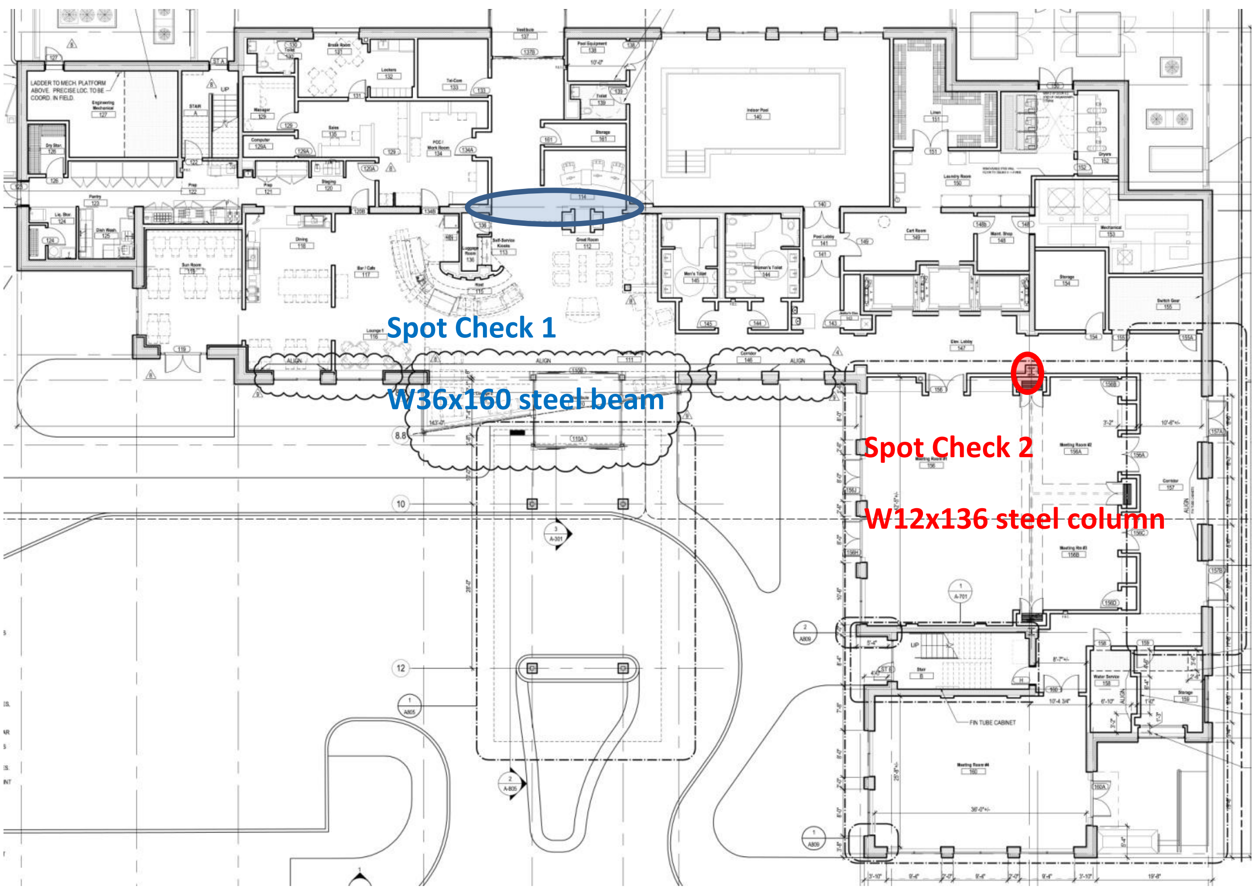


Figure 25: Architecture plan of first floor

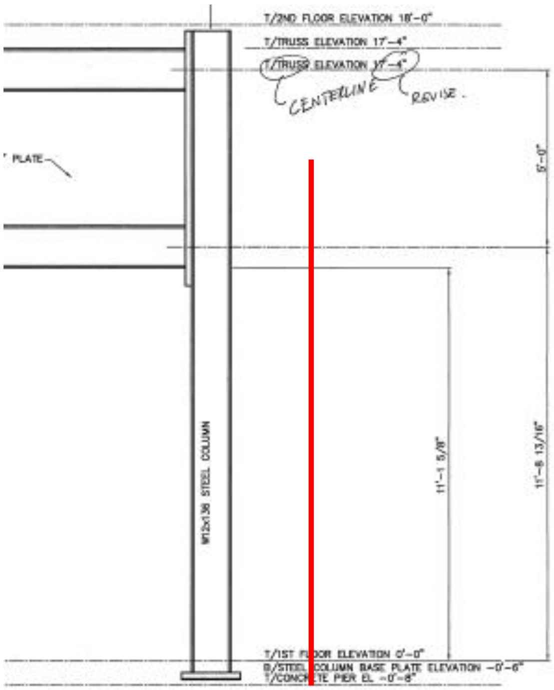


Figure 26: Elevation view of W12x136 steel column

The appropriate loads for the beam and column were determined in excel using dead loads from the seismic analysis and the proper live loads. The beam is carrying the weight that the bearing wall above it supports from the roof down, in pounds per linear foot is found in *Figure 28.* Similarly the column being analyzed is supporting a large truss that is carrying the plf found in *Figure 29.* The KL is determined from *Figure 26* from the pinned base to the center of the moment connection with the truss. Lastly the live, superimposed, and partition loads is checked against the precast plank manufacturer's table, *Figure 30*.

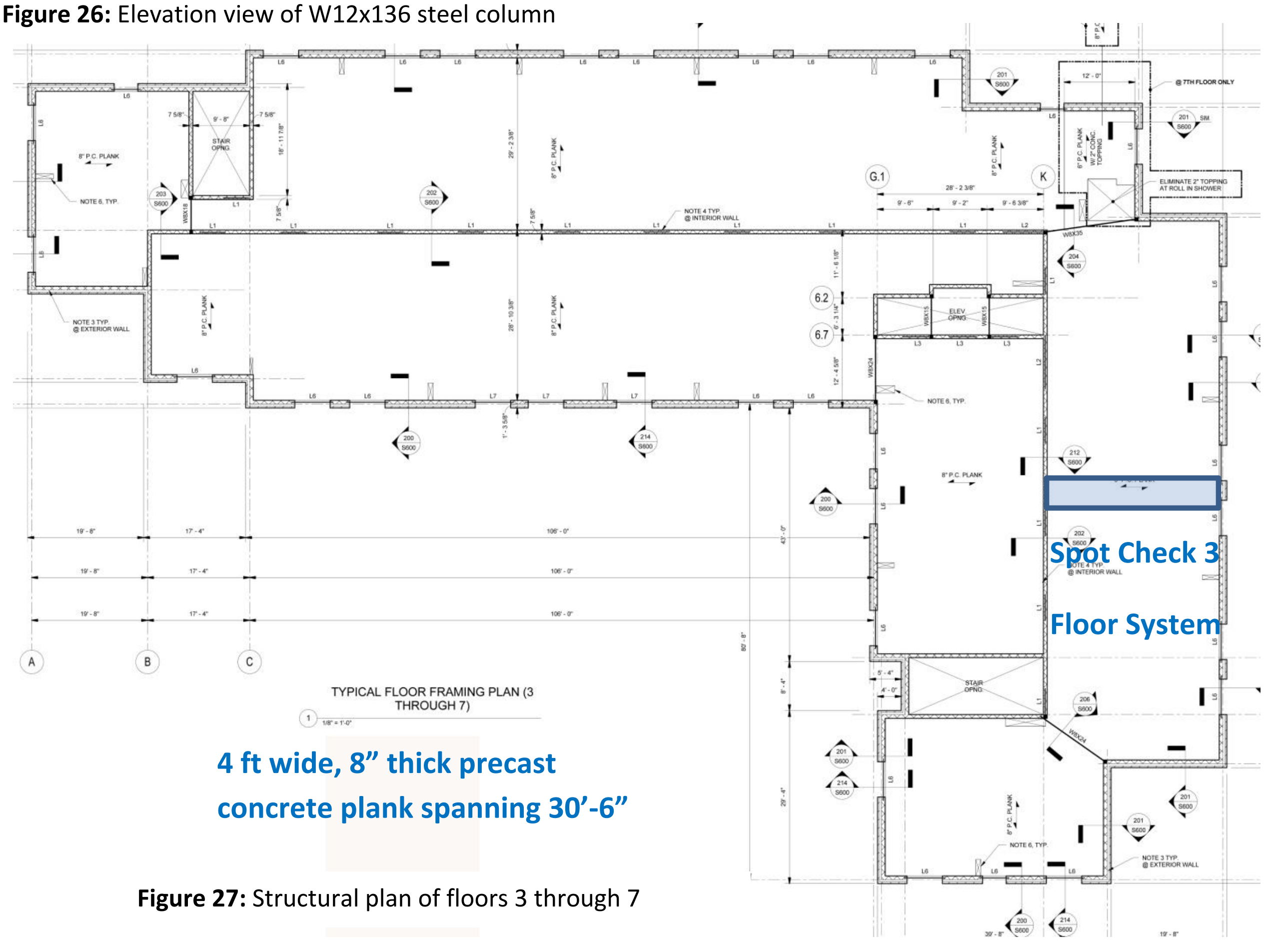


Figure 27: Structural plan of floors 3 through 7

Dead Load Due to Bearing Wall						Total DL (klf)	Total LL (klf)	Total (1.2DL+1.6LL) (klf)
Wall Type	Floor Location	Weight (psf)	Total Height		Weight (klf) Per wall			
E	2nd int. typ.	92.39	8.66		0.80			
H	3rd - 5th int.	75.39	25.98		1.96			
I	6th - 7th int.	65.39	17.32		1.13			
Area Loads								
Load Type	Weight (psf)	Tributary Width (ft) Beam	Total weight per floor (plf) Beam	# Floors	Weight (klf) Load	19.61	5.0141	**31.55**
Live Load (reduced)	25.9	29	0.75	6	4.5066			
Partions (DL)	15	29	0.44	6	2.61			
MEP (DL)	10	29	0.29	6	1.74			
Roof Snow Load	17.5	29	0.51	1	0.5075			
Floor System (DL)	56	29	1.62	7	11.368			

Figure 28: Calculation of load in kips per linear foot on beam

Spot Check 1 - The beam works for flexure and live load deflection, the calculation is located in Appendix C.

Dead Load Due to Bearing Wall						Total DL (klf)	Total LL (klf)	Total (1.2DL+1.6LL) (klf)
Wall Type	Floor Location	Weight (psf)	Total Height		Weight (klf) Per wall			
E	2nd int. typ.	92.39	8.66		0.80			
H	3rd - 5th int.	75.39	25.98		1.96			
I	6th - 7th int.	65.39	17.32		1.13			
Area Loads								
Load Type	Weight (psf)	Tributary Width (ft) Truss	Total weight per floor (plf) Truss	# Floors	Weight (klf) Load	19.61	5.15852	**31.78**
Live Load (reduced)	26.73	29	0.78	6	4.65102			
Partions (DL)	15	29	0.44	6	2.61			
MEP (DL)	10	29	0.29	6	1.74			
Roof Snow Load	17.5	29	0.51	1	0.5075			
Floor System (DL)	56	29	1.62	7	11.368			

Figure 29: Calculation of load in kips per linear foot on truss

Spot Check 2 – The column was found substantial enough to withstand the axial and bending forces that are due to the above stories, the calculation is located in Appendix C.

Spot Check 3

Find the load on the load in the given area on the precast concrete plank and check to make sure it is less than the value allowable for a 30'-6" span in *Figure 30.*

Spot Check 3	
Load	**Weight (psf)**
Live Load*	40
Partitions	15
MEP	15
Plank Topping**	13
Total	**83**
*unreducible	
**leveling material, data from AES	

Loads **83 psf < 87.75 psf** (interpolated from *Figure 30*)

Therefore the 8" precast concrete slab is sufficient for the loads applied.

PITTSBURGH FLEXICORE CO., INC.
8" x 48" Spiroll Corefloor Load Table

8" x 48" Hollowcore (Untopped)
CLEAR SPAN IN FEET

Designation	14'	16'	18'	20'	22'	24'	26'	28'	30'	32'	34'	36'	38'
8S38-1.75	257	186	137	102	75	55	40	X	X	X	X	X	X
8S48-1.75	350	258	194	148	113	87	67	51	38	X	X	X	X
8S58-1.75	369	314	241	186	146	114	90	71	55	42	32	X	X
8S68-1.75	381	325	281	232	184	146	117	94	76	60	48	37	X
8S78-1.75	393	335	290	255	214	172	140	113	92	75	61	49	38

8" x 48" Hollowcore (2" Concrete Topping)
CLEAR SPAN IN FEET

Designation	14'	16'	18'	20'	22'	24'	26'	28'	30'	32'	34'	36'	38'
T8S38-1.75	343	248	182	134	99	72	51	31	X	X	X	X	X
T8S48-1.75	451	346	260	198	151	116	88	62	38	X	X	X	X
T8S58-1.75	465	395	335	259	202	159	125	91	65	43	X	X	X
T8S68-1.75	478	406	351	307	242	193	154	120	89	64	44	X	X
T8S78-1.75	491	417	361	316	279	238	187	146	113	85	62	42	X

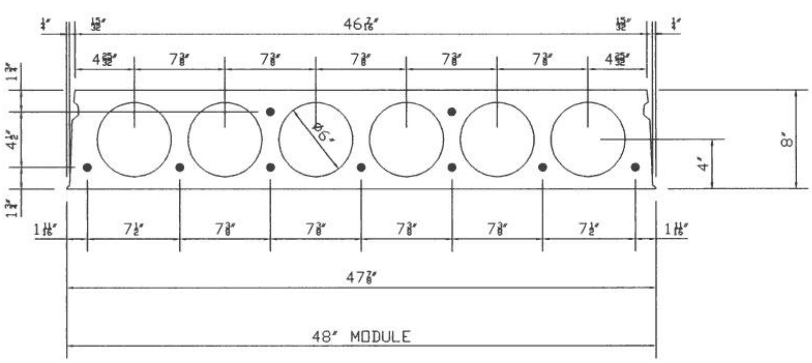


Figure 30: Precast concrete plank design values

Conclusion:

During the process of this report a much greater understanding of what the structure of the Hyatt Place North Shore is made up of and what it must withstand to remain structurally sound. It was also determined that the structure provided is sufficient at handling the gravity loads put on it.

Different portions of the structure were analyzed and found to be adequate. The steel structures on the lobby level of the building safely do their job of creating more open space for the occupants. On the upper levels the precast concrete planks take care of the loads applied in a simple and cost effective manner.

This report only analyzed members to take gravity loads. Next up will be taking the lateral loads that were determined in this report and checking to make sure that the lateral system is sufficient. The lateral system will be tested the more by wind in the North/South direction than the East/West direction. But the seismic base shear beats out the values of both wind directions, with a max base shear of 943.69 kips.

The following appendices have further details into calculations, diagrams, and tables referring to the structural analysis in this report.

Appendix A

Wind Loading:

Kyle Tennant | Wind Analysis | Tech 1 1

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 excell

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

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

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

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

Wind Loading:

Kyle Tennant | Wind Analysis | Tech 1 | 2

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

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

Intensity Turbulence ($I_{\bar{z}}$) (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 ($L_{\bar{z}}$) (Eq. 6-7)

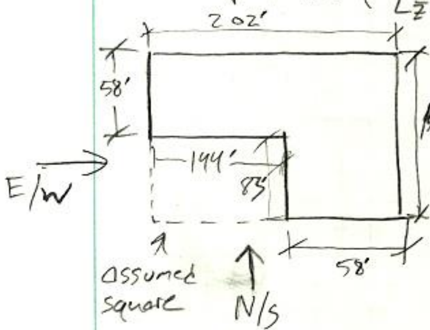
$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{48}{33}\right)^{1/5} = \boxed{538.91}$$

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

$B = 141'$

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

Wind Loading:

Kyle Tennant | Wind Analysis | Tech 1 | 3

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) $\quad GC_{pi} = +.18, -.18$

$P_h = q_h G C_p - q_h G C_{pi}$ (leeward)

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	

Seismic Loading:

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**

Seismic Loading:

Kyle Tennant | Tech 1 Seismic | 1

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)

$T_a = C_t h_n^x = .02(90)^{.75} = .53\text{sec}$ (eq 12.8-7)

Calculate Siesmic Response Coeficient (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

$\frac{.0784}{.48\left(\frac{2}{1}\right)} = .082$ $\quad \frac{.13}{\frac{2}{1}} = \boxed{.065}$

Seismic Base Shear

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

Story Shear Values

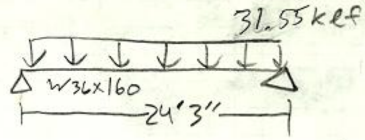
Shear force at level x $\rightarrow F_x = C_{vx} V$ (eq. 12.8-11)

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

Appendix C

Spot Check 1: Steel Beam

Kyle Tennant | Spot Calc 1 | Tech 1 (1)



$LL_{red} \Rightarrow L = 40\left(.25 + \frac{15}{\sqrt{2(24.25 \times 28)}}\right)$
$= 25.9$ psf $> .4(40)$ ✓

$L_b = 0$ (full lateral support)

$M_u = \frac{w_u (ln^2)}{8} = \frac{31.55(24.25^2)}{8} = 2319.2$ ft-k

Steel Manual Table 3-2

$\phi_b M_p = 2340$ ft-k $> M_u = 2319.2$ ✓

Check Deflection (LL) for L/360

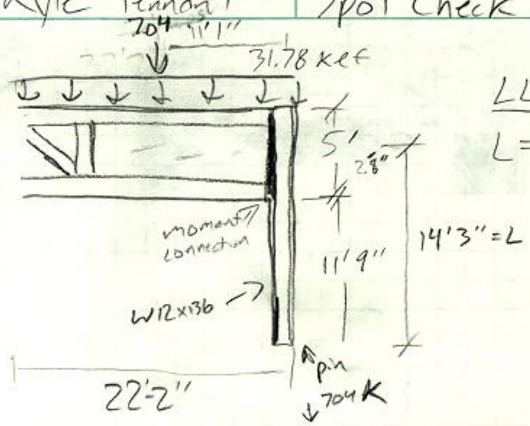
Allowable: $\frac{24.25(12)}{360} = .81$ ft vs Actual: $\Delta_{max} = \frac{5wl^4}{384EI} = \frac{5(5)(24.25^4)(12^3)}{384(29000)(9760)}$

$> = .137$ ✓

a very low percentage of the load was due to live load

Spot Check 2: Steel Column (needs further attension)

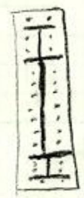
Kyle Tennant | Spot Check 2 | Tech 1 | 1



LL_{red}

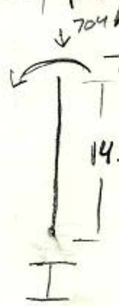
$L = 40\left(.25 + \frac{15}{\sqrt{2(22.14 \cdot 29)}}\right) = 26.73$

$> .4(40)$ ✓



→ moment connection

End view of connection



$M = 704\,(11') = 7804.75$ k-ft

Table 6-1 $KL = 15'$

$p = .706 \times 10^{-3}$ $\beta = 1.14 \times 10^{-3}$

$b_y = 2.42 \times 10^{-3}$

$.706 \times 10^{-3}(704) = .497 > .2 \rightarrow H1\text{-}1a$

$.497 + 1.14 \times 10^{-3}(7804.75) + 2.42 \times 10^{-3}(15)$

$=> 1$ ✗

X-axis bending

none about the Y-axis

Check for purely axial

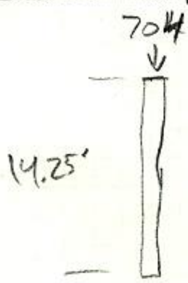


Table 4-1 $KL = 15'$

$\phi_c P_n = 1420\text{ k} >> 704\text{ k}$ ✓

Appendix D:

Photos

Steel Truss Over Meeting Room

Site Photo 1

Site Photo 2

Site Photo: Hotel Sign Framework