NORTHSIDE PIERS
BROOKLYN, NY

Technical Assignment One
Structural Concepts & Existing Conditions

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Executive Summary

A summary and detailed description of the structural system for the building "Northside Piers" is presented within this report. The building is a 29-story condominium tower located in Brooklyn, New York. Its gravity loads and seismic loads were found using the New York City code and a couple of other loads were identified. The lateral load was found using a wind tunnel test. It was determined that wind was the controlling mechanism for lateral loads in the building with a base shear of 1,140 kips in the East-West direction.

The concrete structural system consists of a pile foundation with grade beams, two-way flat plate slabs, a few beams, perimeter columns, and concrete shear walls. The shear walls create a central core around the elevator shaft and stairs. An additional wall comes off of the core in the direction that is weakened by doorway penetrations.

Spot checks were performed using the calculated loads for the slab system, a column, and the shear wall system. All of the members checked were strong enough for the given loading. There was, however, additional capacity calculated for the column, but this may be due to the fact that the lateral force was assumed to be taken entirely by the shear walls.

Introduction

Northside Piers is a building currently being constructed on 164 Kent Ave. in the Brooklyn, New York area. It is a 29-story condominium tower built directly off of the East River across from Manhattan Island. It is the first of three residential towers to be built on the site. The building is taking advantage of a recent change in zoning that now allows residential properties to be built in that area, so it is the first of its kind.

The building features a glass cladding system that allows for floor to ceiling windows for uninhibited views of New York City. The ground floor will contain the lobby which leads to the central elevator shaft that services the building. The second floor is where many of the public spaces will be including a fitness room, yoga room, sauna, media room, and children's playroom. The other twenty two levels are dedicated to the private condominium units. The mechanical equipment is located in the cellar, groud floor, and on the roof.

This report looks at the conditions and criteria of the structural design. It will check the design loads that were originally used. The structural system will then be described and discussed, and finally that system's capacity will be verified.

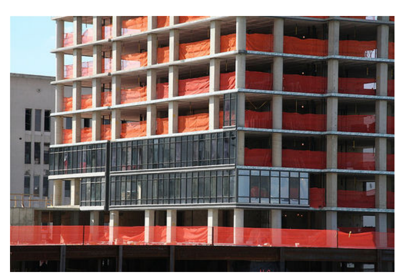
Description of Structural System

Foundation

The columns sit on top of a foundation of 200 ton piles. The pile caps are at about ten feet below grade. Grade beams run along the perimeter of the building. The highest concentration of piles is directly underneath the central core of the building in order to transfer the high moments to the ground below. This will probably be the critical portion of the foundation design. The foundation plans can be found in the appendix.

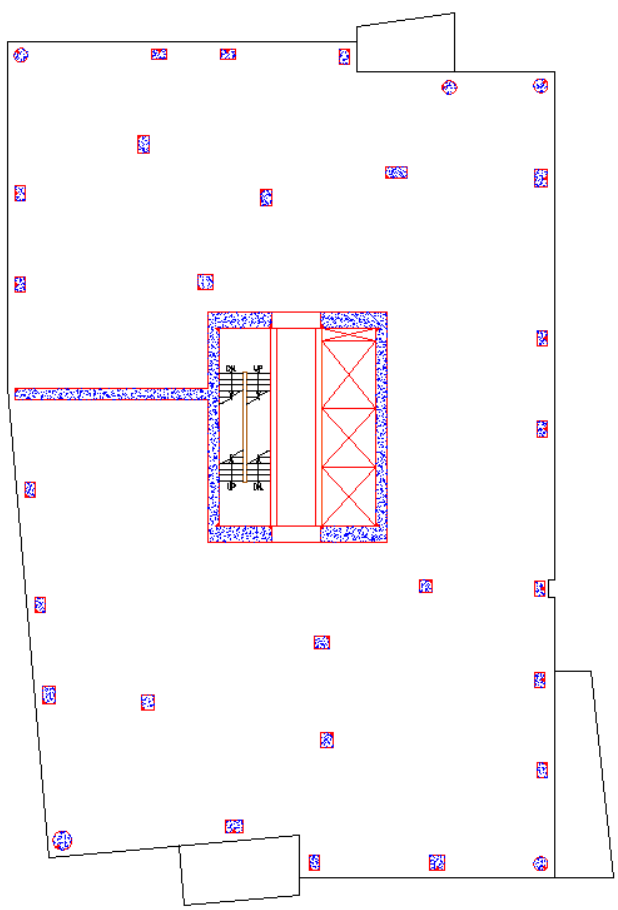
Floor System

Almost the entirety of the building is designed with a two-way flat plate slab system. This is to minimize the number of beams running along the condominium ceilings. Slabs consist of 6000 psi concrete that are



typically 8” thick except for the roof, the 2nd floor, and the lobby where they are 10”, 12”, and 10” thick respectively. While the 60,000 psi reinforcing varies size and spacing throughout the building, the most common reinforcing is #5 bars at 12” o/c at the top and bottom of the slab going both directions.

There are only a few places that additional beams are used in this building because of the flat plate slab system. On the 26th floor, they are used to help transfer loads to a shear wall and some columns that begin directly below that level. The 29th floor and the roof use beams to carry the additional weight of the equipment on top of the building.



Columns
The columns in this building do not follow a consistent grid in order to accommodate the floor plans. They are mostly rectangular columns loacted around the perimeter of the building with a few of them on the interior to break up the large bays. Columns consist of 8000 psi concrete with usually 8 rebars along their edge varying in size from #7-#11. The bars are held in place with ties. Typical floor to floor heights are 10 feet.

Lateral Resisting System
Lateral forces are carried in this building by the central core. It consists of concrete shear walls surrounding the elevator shaft. The walls are 1 ½ foot thick in the long direction and 2 feet thick in the shorter direction. The concrete strength is 8000 psi until the 14th level where it decreases to 6000 psi. The reinforcing is typically #5-#7 at 12 in. o/c. on both faces of the walls.

There is one additional shear wall in the building that extends off of the building core. It starts at the foundation of the building and goes all the way to the 25th floor. This wall makes up for the side of the central core that is weakened by doorways penetrating the shear walls. Some of the moment is also taken by the concrete columns but the extent of this cannot be determined until a model is made which is beyond the scope of this report. At this point I will assume that the shear walls carry all of the lateral load because they are much stiffer than the columns and will clearly carry the majority of it.

Code Requirements

The structural system for the building was designed using the New York City Code, 2003 edition. This code makes reference to the Uniform Building Code (UBC) on several instances with its own modifications made to it. The concrete is specified based on the American Concrete Institute (ACI) standards and the welding must be completed according to the standards of the American Welding Society.

Loads

Gravity Loads

Applicable loads were taken from the New York City Code as well as from manufacturers.

Live Loads:

Equipment Rooms (Pumps, Boilers, Tanks, etc)	150 psf
Light Storage Areas	100 psf
Lobby/Public Spaces	100 psf
Offices	50 psf
1st Floor Elevator Lobbies	100 psf
Multifamily Dwellings	40 psf
EMR/Bulkhead	100 psf
Mechanical Roof	150 psf
Balconies (150% of serviced area)	60 psf

*Live Loads may be reduced

Superimposed Dead Loads:

Equipment Rooms (Pumps, Boilers, Tanks, etc)	15 psf
Light Storage Areas	50 psf
Lobby/Public Spaces	40 psf
Offices	30 psf
1st Floor Elevator Lobbies	40 psf
Multifamily Dwellings	30 psf
EMR/Bulkhead	5 psf
Mechanical Roof	40 psf
Balconies (150% of serviced area)	15 psf

Dead Loads:

Concrete	150 pcf
Glass Cladding	8 psf
Roof Cooling Towers	16 kip each

Roof AHU1	2.8 kip
Roof AHU2	1.1 kip

Snow Load:

- Roof Load shall be 30psf
- "For valleys, loadings shall be increased to provide for accumulations of snow. The loading intensity shall be assumed to vary from forty-five psf at the low point to fifteen psf at the ridge." – New York City Code

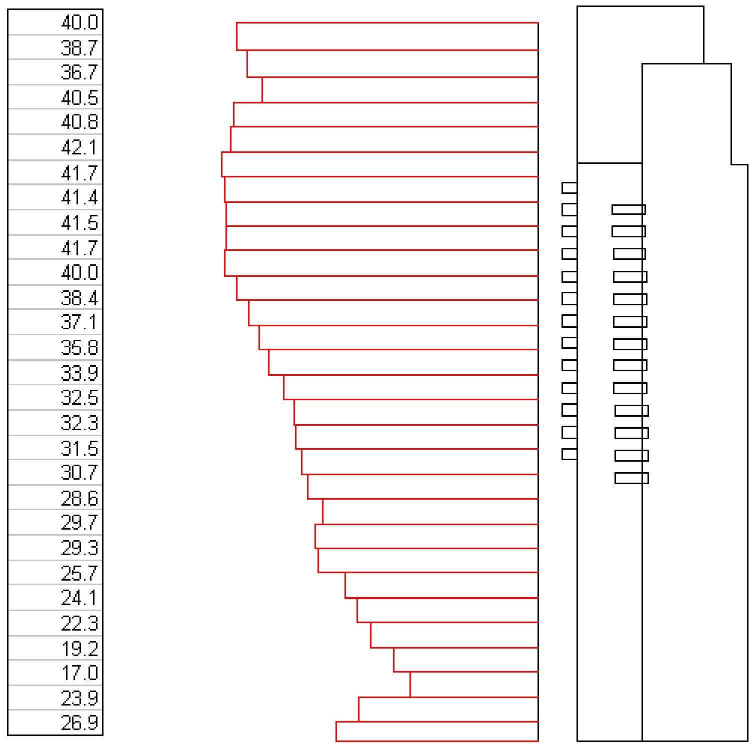
*For comparison, the ASCE7-05 load is in the appendix

Wind Loads

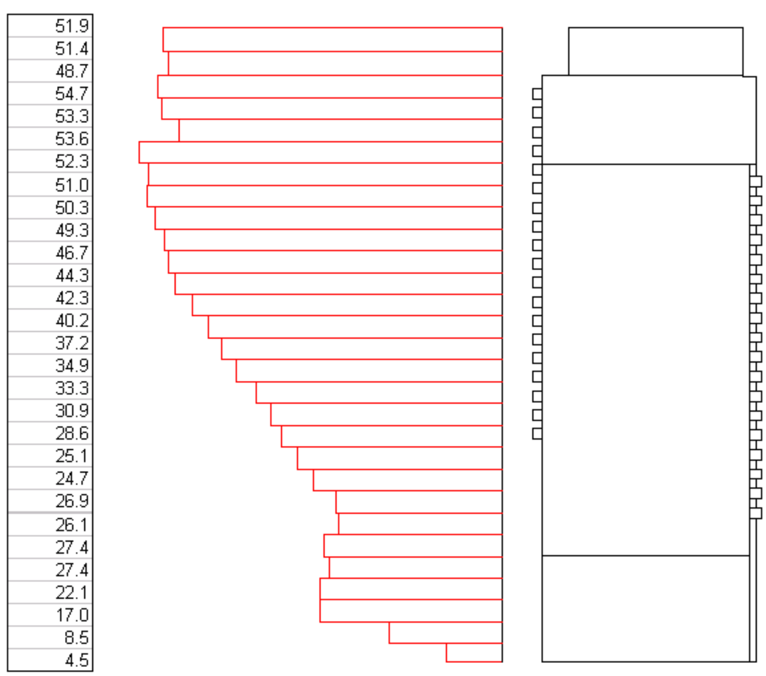
The New York City Code has a set pressure for each surface depending on its height alone rather than doing detailed analysis like is done in ASCE7. The New York City Code also allows for the use of a wind tunnel to determine loads, which is what the building's designers chose to do for accuracy and economy. The test was performed by Alan G. Davenport Wind Engineering Group, and the results are summarized below. These are the values that I will use for my analysis. The wind loads create a base shear of 1,140 kips and an overturning moment of 194,000 ft-kips in the East-West direction.

For comparison sake, I also found the loads based on ASCE7 as well as the New York City Code if you don't use a wind tunnel test. These can be found in the appendix.

Pressures in the East-West Direction, psf (Pressure + Suction)



Pressures in the East-West Direction, psf (Pressure + Suction)



Seismic Loads

Seismic loads were calculated using ASCE7-05. The results are summarized below. The base shear for the building is 340 kips which is significantly less than the 1,140 kips from the wind load. The base moment for the building is 80,500 ft-kips which is also less than the 194,000 ft-kips produced by the wind. Wind is the controlling lateral force. Calculation details can be found in the appendix.

Floor	Height, h_x (ft)	Weight, w_x (k)	$w_x h_x^{2.5}$	Cvx	Force (k)
BULKHEAD	315.2	95.8	1.69E+08	0.01113	4
EMR FLOOR	304.2	543.2	8.77E+08	0.05776	20
ROOF	294.2	1355	2.01E+09	0.13253	45
29	282.2	1033	1.38E+09	0.09104	31
28	271.8	1033	1.26E+09	0.08289	28
27	261.2	1033	1.14E+09	0.07504	26
26	250.8	1033	1.03E+09	0.06779	23
25	240.2	1086	9.71E+08	0.06398	22
24	230.5	1086	8.76E+08	0.05771	20
23	220.8	1086	7.87E+08	0.05183	18
22	211	1086	7.02E+08	0.04627	16
21	201.2	1086	6.24E+08	0.04108	14
20	191.5	1086	5.51E+08	0.03631	12
19	181.8	1086	4.84E+08	0.03188	11
18	172	1086	4.21E+08	0.02776	9
17	162.2	1086	3.64E+08	0.02397	8
16	152.5	1086	3.12E+08	0.02055	7
15	142.8	1086	2.65E+08	0.01743	6
14	132.2	1086	2.18E+08	0.01438	5
13	122.5	1086	1.80E+08	0.01188	4
12	112.8	1086	1.47E+08	0.00967	3
11	103	1086	1.17E+08	0.00770	3
10	93.2	1086	9.11E+07	0.00600	2
9	82.5	1086	6.71E+07	0.00442	2
8	73.8	1086	5.08E+07	0.00335	1
7	64	1086	3.56E+07	0.00234	1
6	54.2	1086	2.35E+07	0.00155	1
5	44.5	1086	1.43E+07	0.00095	0
4	34.8	1086	7.76E+06	0.00051	0
3	25	1086	3.39E+06	0.00022	0
2	14	1308	9.59E+05	0.00006	0
LOBBY	0	1576	0	0	0
TOTAL		33988	1.52E+10	1	340

Other Loads
While not yet considered at this point, several other loads will need to be taken into account during future analysis. In the design of the sub grade walls, the lateral soil pressure must be taken into account. Another load that may play a role into the roof design is the snow drift on the roof's mechanical equipment.

Member Spot-Checks
Since this is a concrete structure, the members must be checked based on ultimate strength design.

Column
I checked the capacity of one of the corner columns on the 14th floor. I chose this column because it is located right before a change in size so I figured it would be close to its capacity. Estimated gravity loads are from the floors above are shown in the appendix.

I found that the columns had a lot more reinforcement in it than was required by the loads I applied. This error could be because more moment is applied to the column from lateral loads than I had estimated.

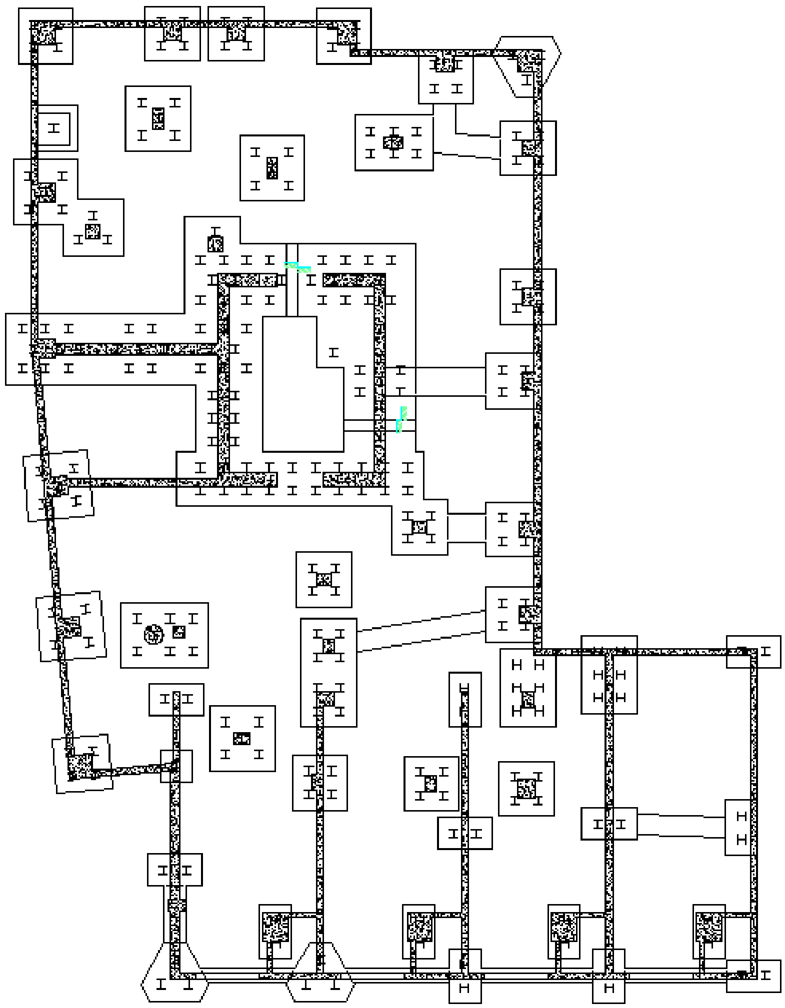
Floor System
One of the larger spans on the floor was checked because it should be close to its capacity. I checked it using PCA slab and found that the required reinforcement was very close to what was provided in the design. It appears that punching shear will be the conrolling portion of the slab design.

Shear Walls
I looked at the capacity of the shear walls around the core that are broken up by the doorways. I chose the 26th floor because this is right before the additional shear wall is added so it should be close to capacity. I assumed all of the shear was applied to the four shear walls equally. The gravity loads estimate is in the appendix before the calculations. I found that the wall had more capacity than it needed. It would have been close to capacity if one more floor of shear was added to it. The designer may have added the additional shear wall one floor earlier than it was required in order to be conservative or in order to decrease deflections.

Other Checks
These checks are by far not the only members that need to be analyzed in this design. The roof, walls, and slabs need to be designed as well as the columns and shear walls at different points throughout the building. Wind loads on the walls, uplift pressures and torsion needs to be checked at some point as well as the foundation design and the soil pressures on the structure.

Appendix

Foundation Plan



ASCE7 – 05 Snow Load

$Pf = 0.7Ce*Ct*I*Pg$

$Pg = 20psf$ *(NYC)*

$Ce = 0.9$ *(Fully exposed, Terrain B)*

$Ct = 1.0$ *(Heated building)*

$I = 1.1$ *(Category III)*

$Pf = 13.86\ psf < I*Pg = 22psf$

$Pf = 22\ psf$

ASCE7 – 05 Wind Load

Loading Summary

Height (ft)	Kz	qz (psf)	p (windward)	p (roof)
0.0	0.57	17.40	12.61	-49.03
7.0	0.57	17.40	12.61	
19.5	0.62	18.76	13.59	**p (sidewalls)**
29.9	0.70	21.19	15.36	-26.40
39.7	0.76	22.98	16.66	
49.4	0.81	24.46	17.73	**p (leeward)**
59.1	0.85	25.75	18.66	-18.86
68.9	0.89	26.90	19.50	
78.2	0.92	27.89	20.22	**p (leeward2)**
87.9	0.95	28.84	20.90	-15.08
98.1	0.98	29.76	21.57	(for wind in other dir.)
107.9	1.01	30.58	22.16	
117.7	1.04	31.35	22.72	
127.4	1.06	32.07	23.24	
137.5	1.08	32.77	23.75	
147.7	1.10	33.45	24.24	
157.4	1.12	34.06	24.69	
167.1	1.14	34.65	25.11	
176.9	1.16	35.22	25.53	
186.7	1.18	35.77	25.92	
196.4	1.20	36.29	26.30	
206.1	1.22	36.79	26.67	
215.9	1.23	37.28	27.02	
225.7	1.25	37.76	27.37	
235.4	1.26	38.21	27.70	
245.5	1.28	38.68	28.03	
256.0	1.29	39.14	28.37	
266.5	1.31	39.59	28.70	
277.0	1.32	40.03	29.02	
288.2	1.34	40.49	29.35	
299.2	1.35	40.93	29.66	
309.7	1.36	41.33	29.96	
316.4	1.37	41.58	30.14	
317.5	1.37	41.63	30.17	

Calculation Summary

Flexible Building (T>1 sec. from Wind Tunnel Test and Approximate Period Method)

h =	317.5	ft (building height)
B =	110	ft (building width)
L =	72	ft (building length)
L/B =	0.65	
B/L =	1.53	(for other direction)
h/L =	4.41	

V =	110	mph Fig 6-1 pg. 31-36
Kd =	0.85	MWFRS Table 6-4 pg. 80
I =	1.15	Category **III** Table 6-1 pg. 77
Exposure =	B	*6.5.6 (Suburban in 20*320 ft. in all directions)*
zg =	1200	*Table 6-2 pg. 78*
alpha =	7	*Table 6-2 pg. 78*
Kzt =	1	*6.5.7*
G =	0.906	*Calculations to the Right*
GCpi =	(+/-) 0.15	*Figure 6-5 pg. 47*
Cp (windward) =	0.8	*Figure 6-6 pg. 49*
Cp (sidewalls) =	-0.7	*Figure 6-6 pg. 49*
Cp (leeward) =	-0.5	*Figure 6-6 pg. 49*
Cp (leeward2) =	-0.4	*(For wind in other direction)*
Roof =	-1.3	***May be Reduced***

<u>Gust Factor For Flexible Building</u>

Gf = 0.925(1+1.7Iz*SQRT(gq^2*Q^2+gr^2*R^2))/(1+1.7*gv*Iz)

Gf = 0.906

Q = sqrt(1/(1+0.63((B+h)/Lz)^0.63))

Q = 0.810

Iz = c*(33/z)^(1/6)

Iz = 0.224

Lz = lambda*(z/33)^epsilon

Lz =	574.0	
gv =	3.4	
gq =	3.4	

gr = SQRT(2ln(3,600*n1))+0.577/SQRT(2ln(3600*n1))

gr =	3.96	
z > 0.6h =	190.5	
z > zmin =	30	*Table 6-2 pg. 78*
z =	190.5	
c =	0.3	*Table 6-2 pg. 78*
lambda =	320	*Table 6-2 pg. 78*
epsilon =	0.333	*Table 6-2 pg. 78*
n1 =	0.398	Hz

R = sqrt(Rn*Rh*Rb/Beta*(0.53+0.47RL))

R = 0.447

Rn = 7.47*N1/(1+10.3N1)^(5/3)

Rn = 0.088

N1 = n1*Lz/Vz

N1 = 2.030

Vz = b*(z/33)^alpha^*V*(88/60)

Vz = 112.5

b = 0.45

alpha^ = 0.25

Rl = 1/eta - 1/(2*eta^2)(1-e^(-2*eta)) for eta>0

Rl = 1 for eta = 0

Rh =	0.175	eta = 4.6*n1*h/Vz		
Rb =	0.407	eta = 4.6*n1*E*B/Vz	eta =	5.165
RL =	0.223	eta = 15.4*n1*L/Vz	eta =	1.790
Beta =	0.02	(From Wind Tunnel Test)	eta =	3.922

New York City Code – Wind Load

Structural Frame:

0-100'	20
101' - 300'	25
301' - 600'	30

Panel Glass:

0-300'	30
300'-600'	35

Horizontal Surfaces:

0-100'	(+/-) 8
101'-300'	(+/-) 10
301'-600'	(+/-) 12

Eaves and Cornices:

0-100'	40 (uplift)
101'-300'	50 (uplift)
301'-600'	60 (uplift)

Components and Cladding:

Wall Elements:	30/-20
Roof Elements:	
0-100'	(+/-) 12
101'-300'	(+/-) 15
301'-600'	(+/-) 18

ASCE7 – 05 Seismic Load Calculations

Ss =	0.359	g	Period = 0.2s
S1 =	0.07	g	Period = 1.0s

Fa =	1.2	11.4-1	(Ss = 0.359, Site Class C) Mapped Spectral Acceleration Values
Fv =	1.7	11.4-2	(S1 = 0.070, Site Class C)

Sms = Fa * Ss =	0.431	g
Sm1 = Fv * S1 =	0.119	g
Sds = 2/3 * Sms =	0.287	g
Sd1 = 2/3 * Sm1 =	0.079	g

Importance Factor =	1.25	11.5-1	Occupancy Category III

SDC = B 11.6-1(Sds = 0.287, Occupancy Cat. III)
New York City Code

R =	8				
Cd = 4.5					
TL =	6	pg 228 New York			
hn =	310	ft			
ct =	0.02	12.8-2 (Shear walls)			
x =	0.75	12.8-2 (Shear Walls)			
Ta =	1.48	sec	**Ta = ct * hn ^x**		
Cu =	1.7	12.8-1 (Sd1 = 0.079)			
T =	2.51	sec		F = 1/T =	0.398

T<TL

MIN	Cs < Sds/(R/I)	0.0449	
MIN	Cs < Sd1/(T*R/I)	0.0049	for T<TL
MIN	Cs < Sd1*Tl/(T^2*R/I)	0.0118	for T>TL

Cs > 0.01

Cs = 0.0100 sec

V = Cs * W

V = 340 k

W = DL + 0.25LL(Storage) + 10psf*partition floors + Permanent equipment + 0.2*snow load (if>30psf)

W = 34000 k

k = 2.5 12.8.3

Fx = Cvx*V

Cvx = wx*hx^k/(SUMwi*hi^k)

Floor Weight

	Slab (psf)	Super (psf)	Core (k)	Equip. (k)	Storage (k)	Glass (k)	ft^2	Total Weight (k)
BULKHEAD	100	16	0	3	0	0	800	95.8
EMR FLOOR	100	16	225	5	0	0	2700	543.2
ROOF	150	30	225	50	0	0	6000	1355
26 to 29	100	30	225	0	0	28	6000	1033
3 to 25	100	30	278	0	0	28	6000	1086
2	125	42	278	0	0	28	6000	1308
1	150	42	390	0	6.3	28	6000	1576.3

Wind Tunnel Equivalent Static Loads

Floor	Floor Height (ft)	Along X Direction (lb)	Along Y Direction (lb)	Torsional Direction (ft-lb)
Building Top	317.5	2.43E+03	1.48E+03	1.99E+04
BULKHEAD	315.2	1.42E+04	8.59E+03	1.18E+05
EMR FLOOR	304.2	2.67E+04	1.84E+04	3.53E+05
ROOF	294.2	4.27E+04	3.63E+04	9.39E+05
29	282.2	4.93E+04	4.07E+04	9.99E+05
28	271.8	4.47E+04	3.78E+04	9.52E+05
27	261.2	4.24E+04	3.58E+04	9.31E+05
26	250.8	4.68E+04	4.02E+04	1.29E+06
25	240.2	4.55E+04	3.79E+04	1.25E+06
24	230.5	4.49E+04	3.64E+04	1.22E+06
23	220.8	4.47E+04	3.57E+04	1.18E+06
22	211	4.46E+04	3.50E+04	1.14E+06
21	201.2	4.45E+04	3.43E+04	1.10E+06
20	191.5	4.45E+04	3.35E+04	1.05E+06
19	181.8	4.29E+04	3.19E+04	9.98E+06
18	172	4.14E+04	3.04E+04	9.38E+05
17	162.2	3.98E+04	2.89E+04	8.80E+05
16	152.5	3.82E+04	2.73E+04	8.20E+05
15	142.8	3.78E+04	2.64E+04	7.60E+05
14	132.2	3.63E+04	2.48E+04	6.91E+05
13	122.5	3.45E+04	2.26E+04	6.37E+05
12	112.8	3.38E+04	2.11E+04	5.60E+05
11	103	3.31E+04	1.96E+04	5.06E+05
10	93.2	3.23E+04	1.80E+04	4.49E+05
9	82.5	3.17E+04	1.68E+04	3.96E+05
8	73.8	2.98E+04	1.74E+04	3.94E+05
7	64	2.77E+04	1.79E+04	3.93E+05
6	54.2	2.59E+04	1.87E+04	4.02E+05
5	44.5	2.38E+04	1.86E+04	4.27E+05
4	34.8	2.06E+04	1.51E+04	5.20E+05
3	25	1.94E+04	1.24E+04	6.74E+05
2	14	3.28E+04	7.43E+03	6.07E+05
LOBBY	0	2.07E+04	2.20E+03	1.32E+05
Total Base Shear (lb)		1.14E+06	8.10E+05	
Total Moment (lb-ft)		1.94E+08	1.49E+08	2.37E+07

Load Case	X	Y	T
1	0.60	0.60	0.25
2	0.40	1.00	0.30
3	0.40	0.40	0.90
4	0.60	0.60	-0.25
5	0.40	1.00	-0.30
6	0.40	0.40	-1.00
7	0.60	-0.40	0.30
8	0.40	-0.75	0.30
9	0.40	-0.40	0.90
10	0.60	-0.40	-0.25
11	0.40	-0.75	-0.25
12	0.40	-0.40	-1.00
13	-1.00	0.40	0.25
14	-0.40	1.00	0.30
15	-0.40	0.40	0.90
16	-1.00	0.45	-0.25
17	-0.40	1.00	-0.30
18	-0.40	0.45	-1.00
19	-1.00	-0.45	0.25
20	-0.55	-0.75	0.30
21	-0.40	-0.45	0.90
22	-1.00	-0.45	-0.25
23	-0.55	-0.75	-0.25
24	-0.40	-0.45	-1.00

Member Spot-Check Calculations

Load Calculations

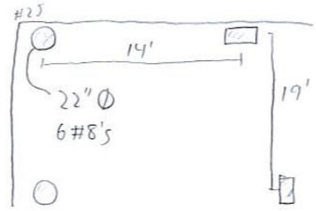
	Load on Wall slab (psf)	Shear Estimate Super (psf)	LL (psf)	Core (k)	ft^2	DL total (k)	LL Total (k)
Bulkhead	100	16	100	225	700	463.7	70
EMR	100	16	100	225	700	463.7	70
Roof	150	41	100	225	1800	973.8	180
29	100	30	40	225	1800	864	72
28	100	30	40	225	1800	864	72
27	100	30	40	225	1800	864	72
						4493.2	536

Load on Column Estimate (25 on 14th floor)

	slab (psf)	Super (psf)	LL (psf)	ft^2	DL total (k)	LL Total (k)	Reduced LL (k)
Roof	150	41	100	70	13.37	7	7
Floors 15-29	100	30	40	1050	136.5	42	29.9

Check Column #25 on the 14th Floor

Estimated Area Supporting : $A \approx 7' \cdot 10' = 70\,ft^2$



$P_{Dead} = 150^K$

$P_{Live} = 37^K$ (reduced LL)

$w_u = 220\,psf \Rightarrow M_{01} = \frac{220 \cdot 7 \cdot 19^2}{8} = 169^{1\text{-}K}$

$M_{U1} \approx 0.26\,M_{01} = 36.1^{1\text{-}K}$

$M_{02} = \frac{220 \cdot 10 \cdot 14^2}{8} = 53.9^{1\text{-}K}$

$M_{U2} \approx 0.26\,M_{02} = 14.0^{1\text{-}K}$

$P_u = 1.2 \cdot 150 + 1.6 \cdot 37 = 240^K$

$M_u = 50^{1\text{-}K}$

$\frac{Kl}{r} = \frac{120''}{0.25 \cdot 22''} = 22 \leq 22 \quad \therefore$ Not Slender

$\gamma = (22-2.5)/22 = 0.89$

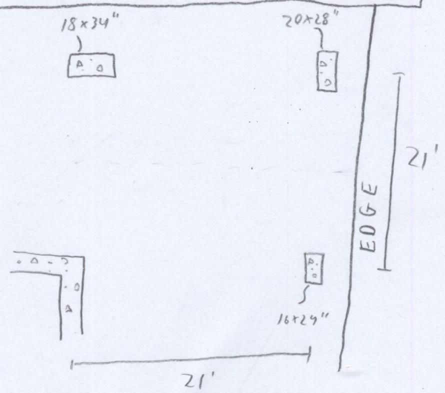
$K_n = \frac{240}{0.65 \cdot 8 \cdot 380} = 0.121$

$R_n = \frac{50^{1\text{-}K} \cdot 12}{0.65 \cdot 8 \cdot 380 \cdot 22} = 0.0138$

$\rho = \frac{4.74\,in^2}{380\,in^2} = 0.012$

$\rho_{reqd} << 0.01$ (design tables) OK

Check Slab on Levels 15 to 25



$DL = 150pcf \cdot 8''/12''/' + 30psf = 130psf$

$LL = 40psf$

$W_u = 1.2 \cdot 130psf + 1.6 \cdot 140psf = 220psf$

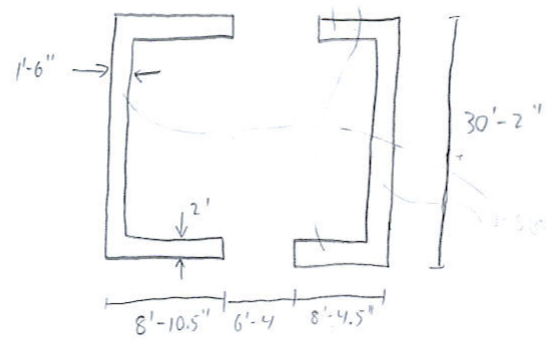
$f'_c = 6ksi$

$f_y : 60000ksi$

Span and Load information was inputed into PCA slab.
It was inputed as a 21x21' bay flat plate system with 20"x20" columns.
The results are as follows:

		Actual Spacing
Top Bars Req'd		
Edge - Columns S.	9 #5's (Spacing = 13.3")	6"
- Middle Strip	9 #5's (spacing = 13.3")	12"
Middle - Column S.		
Supports	5 #5's (spacing = 24")	6"
Span	9 #5's (spacing = 13.3")	12"
Middle Strip		
Span	9 #5's (spacing = 13.3")	12"
Bottom Bars Req'd		
Edge - Column St.	12 #5's (spacing = 10")	12"
- Middle St.	9 #5's (spacing = 13.3")	12"
Middle - Column St.	9 #5's (spacing = 13.3")	12"
- Middle St.	9 #5's (spacing = 13,3")	12"

Shear Wall @ 26th floor



$f'_c = 8\ ksi$

$f_y = 60\ ksi$

$A_{core} = 150\ ft^2$

Loads:

Wind	Dead	Live
$V_x = 269^k$	4500^k	540^k
$M_x = 7283^{'-k}$		
$V_y = 219^k$		
$M_y = 5654^{'-k}$		

Controlling Load Combination = $1.2D + 1.6W + 1.0L$

$\therefore P_u = 5940^k$

$V_x = 430^k$

$M_x = 11,650^{'-k}$

$V_y = 350^k$

$M_y = 9,050^{'-k}$

Checking X-Direction (short wall)

$I\ 0.25h = 2.5'$

- BE needed?

$A_g = 2' \cdot 8.375' + 2.5' \cdot 1.5' = 20.5\ ft^2$

$$NA = \frac{8.375 \cdot 2 \cdot (8.375/2) + 2.5 \cdot 1.5 \cdot (1.5/2)}{8.375 \cdot 2 + 2.5 \cdot 1.5} = 3.56\ ft$$

$$I_x = \tfrac{1}{12}(8.375^3)(2) + (8.375)(2)(8.375/2 - 3.56)^2 + \tfrac{1}{12}(2.5)(1.5)^3 + (2.5)(1.5)(3.56 - 1.5/2)^2 = 135\ ft^4$$

$$\frac{P_u}{A_{core}} = \frac{5940^k}{150\ ft^2} = 39.6\ ksf = 0.275\ ksi$$

$$\frac{M_x/4 \cdot \bar{y}}{I_x} = \frac{11650/4 \cdot (8.375 - 3.56)}{135} = 104\ ksf = 0.722\ ksi$$

$\sigma = 0.275\ ksi + 0.722\ ksi = 1.00\ ksi < 0.2 \cdot 8\ ksi = 1.6$ NO B.E. Req'd

$\phi P_n = 0.80\,[0.85 \cdot 8 \cdot (150)(144)] = 117,000^k$ OK

- Shear Design

$V_x/4 = 108^k < A_{cv}\sqrt{f'_c} = 2' \cdot 8.375' \cdot 144\sqrt{6000}/1000 = 187^k$

$\rho_l \geq 0.0012$

$\rho_t \geq 0.002$

$A_{cv} = 24'' \cdot 12''/1 = 288\ in^2/ft$

$As_{long} = 0.0012 \cdot 288 = 0.346\ in^2/ft$

$As_{trans} = 0.002 \cdot 288 = 0.576\ in^2/ft$

Using #6's & 2 curtains @12"

$As = 2(0.44) = 0.88\ in^2/ft > 0.576\ in^2/ft$ OK

Using #5's & 2 curtains @12"

$As = 2(0.31) = 0.62\ in^2/ft > 0.576\ in^2/ft$ OK

Using #4's & 2 curtains @12"

$As = 2(0.20) = 0.40\ in^2/ft > 0.346\ in^2/ft$ OK

- Shear capacity

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

$\frac{h_w}{l_w} = \frac{10'}{8.375'} = 1.19 \quad \therefore \alpha_c = 3$

For 2#6's @12", $\rho_t = 2 \cdot 0.44\ in^2/24''/120'' = 0.000306$

$V_n = 24'' \cdot 8.375' \cdot 12''/1 (3\sqrt{6000} + 0.000306 \cdot 60000)/1000 = 605^k$

$\phi V_n = 0.9 \cdot 605^k = 544^k > 430^k$ OK