NORTHSIDE PIERS
BROOKLYN, NY

Technical Assignment Three:
Lateral System Analysis and Confirmation Design

Jeremiah Ergas
AE 481W – 5th Year Thesis
Pennsylvania State University
Structural Option
December 3rd, 2007

Faculty Consultant: Dr. Ali Memari

Table of Contents

Executive Summary....................3

Introduction....................4

Loads....................6

Load Distribution....................8

Strength Check....................10

Drift....................10

Conclusions....................12

Appendix

- Foundation Plan....................13
- Summary of Moments of Inertia....................14
- Wind Pressures....................14
- Seismic Load Calculations....................15
- Member Strength Checks....................17
- Allowable Seismic Drift Calculation....................24

Executive Summary

The lateral resisting system for the building, Northside Piers, is the focus of this report. Northside Piers, a 29-story condominium tower located in Brooklyn, New York, is currently under construction utilizing a concrete structure design. It consists of two-way flat plate slabs, shear walls around the central core, and a pile foundation. There are four shear walls around the stairwell and elevator with one additional shear wall coming off of the core. The East-West walls of the core have large penetrations at every level that serve as doorways.

The wind load for the building was determined by a wind tunnel test, and the seismic load was calculated using ASCE7-'05. The wind loads were the controlling case, producing a maximum base moment of 194,000 ft-kips and a base shear of 1,140 kips. Seismic loads only create a base moment of 80,500 ft-kips and a base shear of 340 kips.

The lateral loads are distributed to the shear walls and columns through the floor slab which acts as a rigid diaphragm. The amount of load that goes to each element is a function of the element's stiffness. A simple calculation showed that the columns are expected to take less than 1% of the load, which led to the decision to model only the shear walls in the analysis.

An ETABS model was built in order to determine how much load goes to each element and how much the building drifts. It was found that the two North-South shear walls take almost 50% of the lateral load when the wind acts in the North-South direction. When the wind acts in the East-West direction, the two walls around the central core take about 30% of the load while the protruding wall takes about 40% of the load. It was also determined that torsion would not be a controlling load case in the design of the walls.

The drift was found to meet the industry standards of L/400 for wind drift and the ASCE code recommendation of L/160 for seismic drift. The building is expected to have a displacement of 3.89" in the North-South direction under full wind load.

Finally, the distributed loads were used to check the lateral system at some of its critical points. These included the shear walls at the base of the building and at the floor where the concrete changes strength. The spandrels were also checked above the penetrations in the wall. It was found that all of the members had sufficient capacity except for the elements at the base of the building. This could be due to the fact that there were a few additional openings near the base that were not included in the model, an aspect that will need to be analyzed more thoroughly.

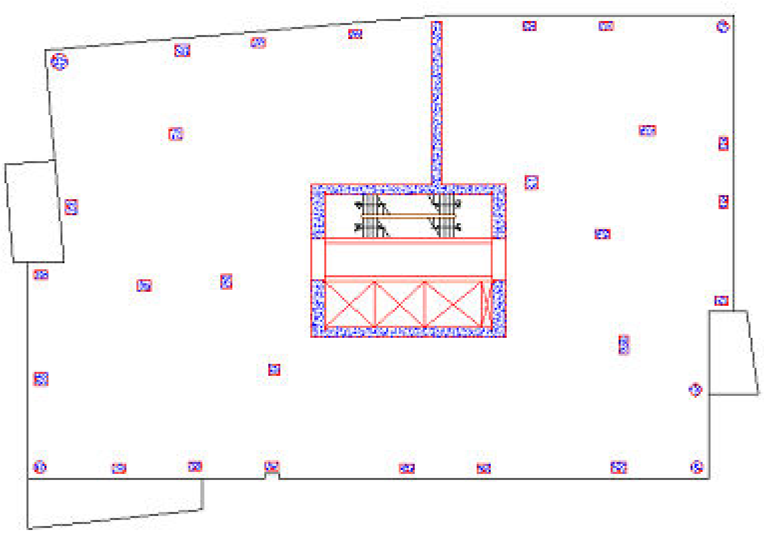
Introduction

This report will discuss issues relating to the lateral design of the North Side Piers building. It will begin with a description of the building and its structural system. The loads on the building will then be presented including the gravity, wind, and seismic loads. The distribution of these loads and their effects on structural members will be discussed. Critical members will be checked for strength, and finally the building's serviceability issues such as drift and story drift will be discussed.

Architecture
Northside Piers is a building currently under construction on 164 Kent Avenue in the Brooklyn, New York area. It is a 29-story condominium tower built directly off of the East River across from Manhattan Island that tops off at a height of 317 feet. The building features a glass cladding system that allows for floor to ceiling windows for uninhibited views of New York City. Transportation throughout the building is provided by a central elevator shaft and stairwell. The 27 floors that are dedicated to the condominium units are all very similar with only minor variations. The ground and cellar floors are used for mostly lobby, storage, and utility spaces.

Floor System
Almost the entirety of the building is designed with an 8" thick two-way flat plate slab system. Slabs consist of 6000 psi concrete with #5 reinforcing bars at typically 12"o/c or 6"o/c at the top and bottom of the slab going both directions.

Beams are only used a few times in this building. They are used on the 3rd and 26th floors as transfer beams in order to accommodate changes in the column grid.

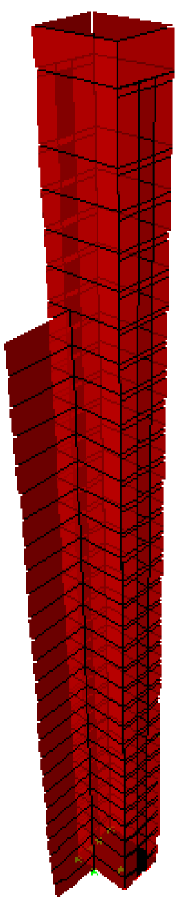


N
→

Columns
The columns in this building do not follow a consistent grid in order to accommodate the floor plans. They are mostly rectangular columns located around the perimeter of the building with a few of them on the interior to break up the large bays. Typical sizes are 20x28" until the 14th floor where they are reduced to 16x24". Almost all of the interior columns are hidden behind walls. 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 height is 9'-9". The column grid shifts twice to in order to deal with setbacks in the building plan.

Foundation
The columns sit on top of a foundation of 200 ton piles that 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. The foundation plan can be found in the appendix.

Lateral Resisting System
Lateral forces are carried in this building by the central core, which consists of concrete shear walls surrounding the elevator shaft and stairwell on all four sides. The walls are 1 ½ foot thick in the North-South direction and 2 feet thick in the East-West direction, and they extend from the foundation to the top of the building. 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.



The walls in the East-West direction contain penetrations at every level to accommodate for doorways. The wall is still continuous due to a 2 feet deep link beam reinforced with #9 and #10 bars at the top and bottom.

There is one additional shear wall in the East-West direction that extends off of the building core. It starts at the foundation of the building and goes all the way to the 25th floor.

There are some other smaller penetrations in the shear walls on various floors, which will be studied in more detail at a later time.

Loads

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

Gravity Loads Summary		
	Live Load*	**Superimposed Dead Load**
Equipment Rooms (Pumps, Boilers, Tanks, etc)	150 psf	15 psf
Light Storage Areas	100 psf	50 psf
Lobby/Public Spaces	100 psf	40 psf
Offices	50 psf	30 psf
1st Floor Elevator Lobbies	100 psf	40 psf
Multifamily Dwellings	40 psf	30 psf
EMR/Bulkhead	100 psf	5 psf
Mechanical Roof	150 psf	40 psf
Balconies (150% of serviced area)	60 psf	15 psf
*Live Loads May Be Reduced		
	Dead Load	
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	
	30 psf	

Lateral Loads
The lateral loads of the building are summarized below.

Wind loads for North Side Piers were determined using a wind tunnel test, performed by Alan G. Davenport Wind Engineering Group. The results are based on a 50-year return period and are given in the form of equivalent static loads. The wind pressures can be found in the appendix.

Seismic loads were calculated using ASCE7-05. New York City is in a relatively inactive seismic zone. Calculation details can be found in the appendix.

Lateral Loads Summary					
		Wind			Seismic
Floor	Floor Height (ft)	Along X Direction (kip)	Along Y Direction (kip)	Torsional Direction (ft-kip)	Force (kip)
Building Top	318	2	1	20	0
BULKHEAD	315	14	9	118	4
EMR FLOOR	304	27	18	353	20
ROOF	294	43	36	939	45
29	282	49	41	999	31
28	272	45	38	952	28
27	261	42	36	931	26
26	251	47	40	1,290	23
25	240	46	38	1,250	22
24	231	45	36	1,220	20
23	221	45	36	1,180	18
22	211	45	35	1,140	16
21	201	45	34	1,100	14
20	192	45	34	1,050	12
19	182	43	32	9,980	11
18	172	41	30	938	9
17	162	40	29	880	8
16	153	38	27	820	7
15	143	38	26	760	6
14	132	36	25	691	5
13	123	35	23	637	4
12	113	34	21	560	3
11	103	33	20	506	3
10	93	32	18	449	2
9	83	32	17	396	2
8	74	30	17	394	1
7	64	28	18	393	1
6	54	26	19	402	1
5	45	24	19	427	0
4	35	21	15	520	0
3	25	19	12	674	0
2	14	33	7	607	0
LOBBY	0	21	2	132	0
Total Base Shear (kip)		1,140	810	0	340
Total Base Moment (kip-ft)		194,000	149,000	23,700	80,500

Load Cases

1.4D
1.2D + 1.6L + 0.5S
1.2D + 1.6S + (1.0L or 0.8W)
1.2D + 1.6W + 1.0L + 0.5S
1.2D + 1.0E + 1.0L + 0.2S
0.9D + 1.6W
0.9D + 1.0E

Load Distribution

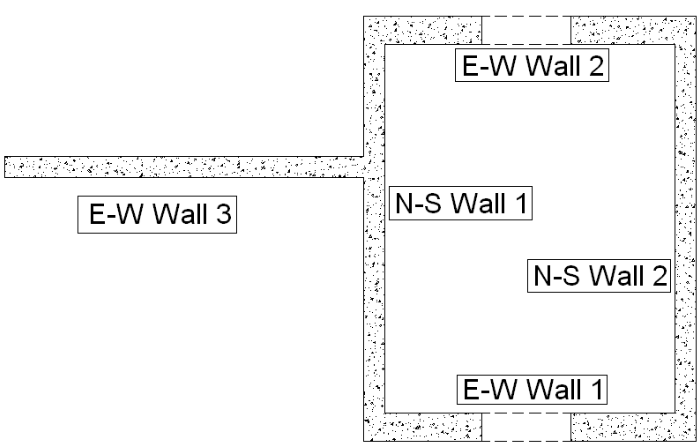
Wind loads are developed on the building as positive and negative pressures on the windward and leeward sides of the building respectively. These pressures are distributed to the slabs at each level of the building through its cladding system. The slabs then acts as a rigid diaphragm and spreads the loads into the lateral resisting system of shear walls and columns. The amount of load distributed to each lateral element is a function of its stiffness. The load is then finally taken to the foundation of the building through its respective lateral element.

Seismic loads follow the same path as the wind loads, except that they begin by acting at the center of gravity of each element of the building. The loads are then distributed through the slab and taken to the foundation by the lateral system.

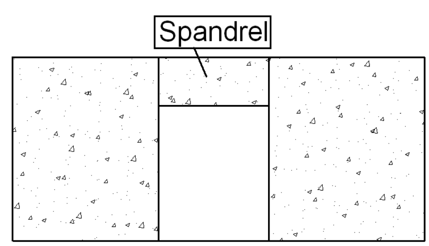
The distribution of loads to the lateral elements was first estimated by calculating the moments of inertia for the shear walls and columns. It was determined that the columns contribute to less than 0.3% of the total stiffness. This led to the decision that a model of just the concrete shear walls would be acceptable. This calculation can be found in the appendix.

The model of the building was built using ETABS, and it consisted of the shear walls of the building with a rigid diaphragm connecting them. The major openings at each floor were incorporated into the model, but some of the smaller penetrations were not. The wind and seismic loads were manually added to the model. The wind load was added at the location recommended by the wind tunnel tests, and the seismic load was placed at the center of gravity of the slab. The wind loads that were distributed to each member are listed below.

Shear Wall Plan with Labels



Shear Wall Elevation with Opening



Wind Load Distribution								
Floor	E-W Wall 1 and 2		E-W Wall 3		N-S Wall 1 and 2		E-W 1 and 2 Spandrels	
	Moment (ft-k)	Shear (k)	Moment (ft-k)	Shear (k)	Moment (ft-k)	Shear (k)	Moment (ft-k)	Shear (k)
TOP	0	1			0	1		
EMR FLOOR	12	8			7	5		
ROOF	162	22			98	14		
29	400	43			255	32	19	15
28	852	68			595	53	39	30
27	1562	90			1148	72	57	44
26	2507	111			1901	90	66	50
25	3823	125	0	19	2841	110	56	32
24	5177	139	183	37	3910	129	54	31
23	6663	152	542	55	5163	147	68	40
22	8279	166	1078	73	6594	165	87	53
21	10026	179	1787	91	8200	182	106	67
20	11903	193	2670	108	9975	199	125	80
19	13910	206	3727	126	11918	216	144	93
18	16043	219	4958	143	14025	232	161	106
17	18296	231	6355	160	16287	247	178	117
16	20667	243	7915	176	18697	262	192	128
15	23149	255	9629	191	21248	275	205	137
14	25741	266	11493	206	23932	288	215	144
13	28440	277	13503	221	26744	301	245	162
12	31239	287	15656	235	29678	312	258	170
11	34138	297	17943	248	32722	323	274	182
10	37133	307	20362	261	35869	333	291	193
9	40223	317	22909	274	39111	342	307	205
8	43405	326	25583	287	42441	350	324	217
7	46675	335	28381	299	45853	359	340	229
6	50025	344	31294	310	49349	368	357	241
5	54155	351	34936	320	53668	377	373	253
4	56933	359	37418	330	56590	386	389	265
3	60490	365	40634	338	60355	394	406	277
2	64566	371	44352	346	64687	400	423	289
LOBBY	69892	380	49193	359	70287	404	442	304
CELLAR	73758	387	52783	367	74324	404	367	252

It was determined that in the North-South direction, 50% of the load goes to each of the N-S shear walls. The East-West direction is more complicated with its three walls and many penetrations. It was determined that approximately 40% of the load goes to E-W Wall 3 and 30% goes to E-W Walls 1 and 2.

The torsional load case was also analyzed, but it was not a controlling load combination. This is due the fact that the shear walls are are balanced around the center of loading for the building.

The loads applied to the spandrel were taken from the ETABS model. Analyzing this aspect is essential because it is an obvious weak link in the lateral system and these members must be checked for capacity.

Member Checks

Now that the loads distributed to each element are known, the members must be checked to make sure they are adequate to carry the load. The areas of particular interest are locations in which the reinforcement changes and also where the concrete strength changes. Since wind acting as point loads produced higher moments and higher shear forces than the wind acting as torsional loads, this will be the primary load case that is checked Many of the walls and spandrels were checked for strength capacity at the critical points. It was found that many of the members that were checked met the load requirements, except for the elements in the Cellar. This could be due to the fact that some of the penetrations in the shear wall at the lower floors were not modeled. This will be modified in future analysis. Details of these calculations can be found in the appendix.

Overturning for the building was also checked. The wind load produces a total moment of 194,000 ft-kip in the E-W direction. The building weighs approximately 35,000 kips. This means a moment arm of 5.5ft is required to keep the building stable. The building is about 70 feet wide, so the moment arm is in the kern of the building, and the building does not need to be designed for uplift in the foundation.

Drift

For serviceability issues the drift must be controlled. This will make the building more comfortable for occupants and will prevent unwanted cracking. The values for drift were determined from the ETABS model of the building and are listed below.

Drift Summary					
Floor	Wind			Seismic	
	East-West Drift (in)	North-South Drift (in)	Rotation (rads)	East-West Drift (in)	North-South Drift (in)
Building Top	3.12	3.89	0.00131	1.43	2.30
EMR FLOOR	3.00	3.74	0.00131	1.37	2.21
ROOF	2.79	3.46	0.00130	1.27	2.04
29	2.67	3.29	0.00130	1.21	1.94
28	2.54	3.13	0.00129	1.15	1.84
27	2.42	2.96	0.00127	1.09	1.74
26	2.29	2.82	0.00126	1.03	1.64
25	2.17	2.66	0.00124	0.97	1.54
24	2.05	2.51	0.00122	0.91	1.45
23	1.94	2.36	0.00119	0.86	1.36
22	1.82	2.21	0.00116	0.80	1.27
21	1.71	2.06	0.00113	0.75	1.18
20	1.60	1.92	0.00109	0.70	1.10
19	1.49	1.77	0.00105	0.64	1.01
18	1.38	1.63	0.00101	0.59	0.93
17	1.27	1.49	0.00096	0.54	0.85
16	1.16	1.36	0.00091	0.49	0.77
15	1.05	1.23	0.00086	0.45	0.69
14	0.95	1.10	0.00081	0.40	0.61
13	0.85	0.97	0.00076	0.36	0.54
12	0.76	0.86	0.00070	0.31	0.47
11	0.66	0.74	0.00065	0.27	0.41
10	0.57	0.64	0.00059	0.23	0.35
9	0.49	0.53	0.00053	0.20	0.29
8	0.41	0.44	0.00048	0.16	0.24
7	0.33	0.35	0.00042	0.13	0.19
6	0.26	0.27	0.00036	0.10	0.15
5	0.20	0.20	0.00030	0.08	0.11
4	0.14	0.14	0.00024	0.05	0.08
3	0.09	0.09	0.00018	0.03	0.05
2	0.05	0.05	0.00012	0.02	0.02
LOBBY	0.01	0.01	0.00004	0.00	0.01
CELLAR	0.00	0.00	0.00000	0.00	0.00

The industry standard for drift from wind is L/400, so the amount of drift allowed for a 10' tall story is 0.3". All of the stories meet this requirement.

ASCE7 gives recommendations for the allowable drift due to seismic loads. It was found that a deflection of L/160 is allowable. This means a 10' tall story can drift a total of 0.75" due to seismic loads. All of the stories satisfy this requirement.

Conclusions

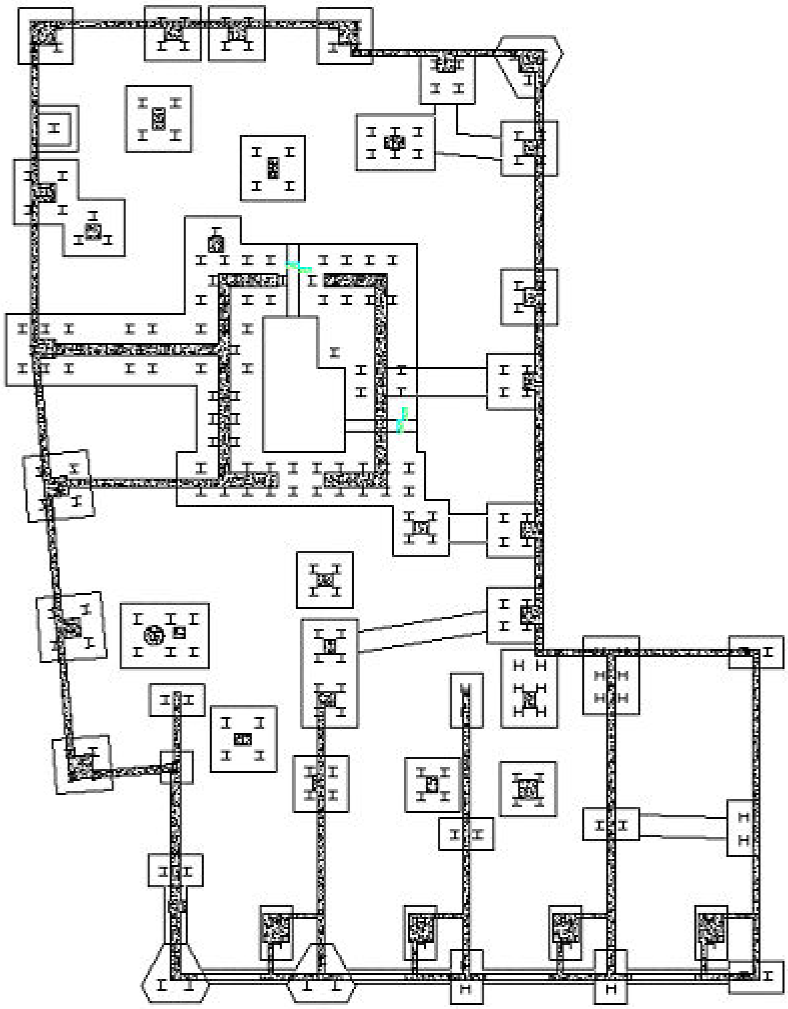
After more detailed analysis of the Northside Pier building, it was found that the loads originally determined from Technical Assignment One still appear to be close to the building's design values. Wind loads remain the controlling lateral load for this design in both terms of strength design and serviceability design.

The ETABS model proved to be helpful in realizing how much loads go to each individual wall. Due to the openings in the wall, determining the stiffness without the assistance of the model would be difficult. It clearly showed the extent to which the holes in the shear walls caused weakness. The analysis of torsion also showed that the building performed pretty well under this condition.

The new load distributions were used to check critical members in the lateral system for capacity. It was found that the original members appear to be sufficient, except for the elements near the base of the building. This discrepancy could be due to the uncertainty of how the loads are distributed at the bottom of the building where there are some additional openings. This will be something that should be looked at in the future. Also, this may be a good area to try to rework the architecture in order to avoid the problem with penetrations in the shear walls altogether.

Appendix

Foundation Plan

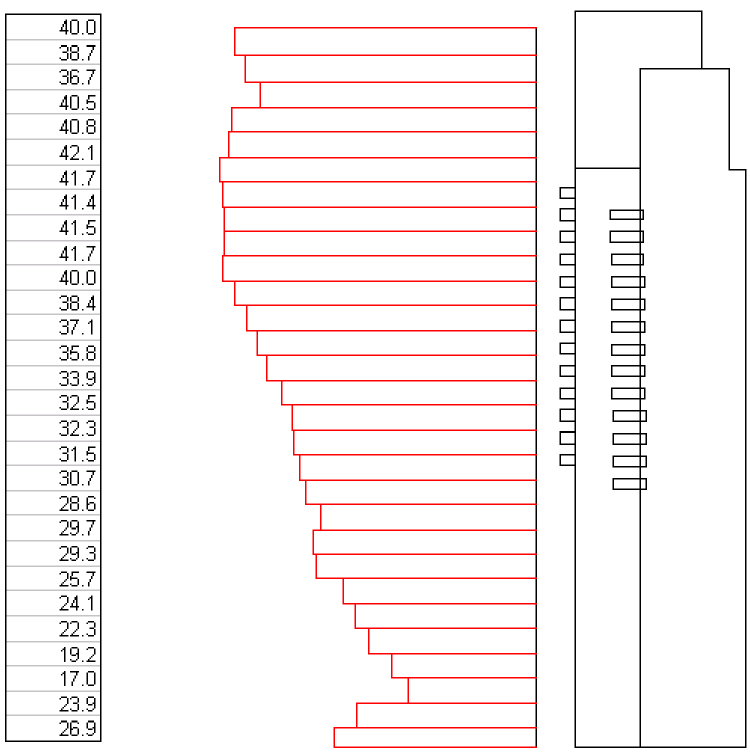


Summary of Moments of Inertia

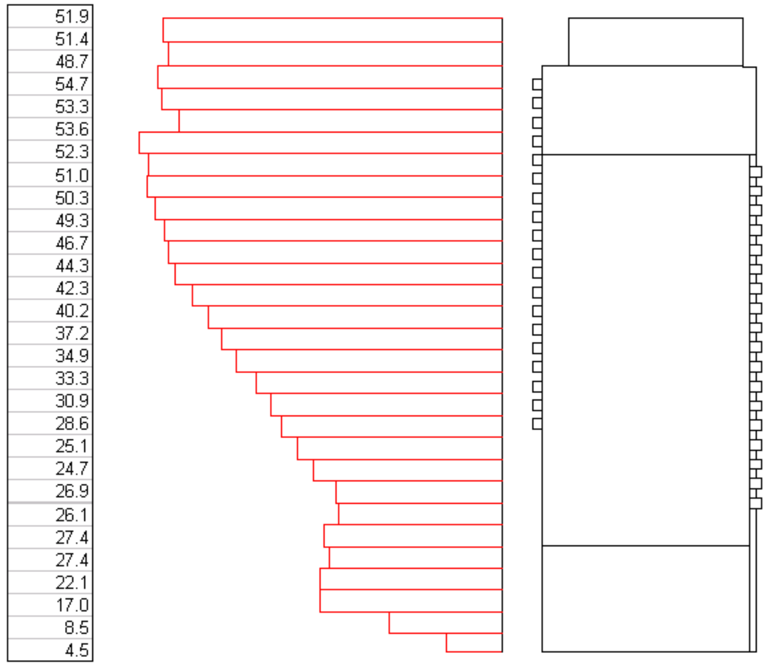
Summary of Moments of Inertia					
	Dimensions	Moment of Inertia in X-Direction (ft^4)	Percent of Total	Moment of Inertia in X-Direction (ft^4)	Percent of Total
E-W Wall 1	2'x24'	2304	34.92%	16	0.24%
E-W Wall 2	2'x24'	2304	34.92%	16	0.24%
E-W Wall 3	1.5'x25'	1953	29.60%	7.03	0.10%
N-S Wall 1	1.5'x30'	8.44	0.13%	3375	49.57%
N-S Wall 2	1.5'x30'	8.44	0.13%	3375	49.57%
(30) Typical Columns	20"x20"	0.643 (each)	0.29% (total)	0.643 (each)	0.28% (total)
Total		6597		6808	

Wind Pressures

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



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



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 11.5-1Occupancy 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

Member Strength Checks

N-S1 Shear Wall (Cellar) Spot Check

Height (ft)	10	
Length (ft)	30	
Thickness (in)	18	
Diameter bars (in)	1.125	
Spacing	12	
fy (ksi)	60	
f'c (ksi)	8	
Factored Moment (ft-k)	118918.4	
Factored Shear (k)	646.4	
Factored Axial (k)	2500	
Moment of Inertia (ft^4)	3375	
Stress (ksi)	4.056123	
0.2*f'c (ksi)	1.6	**BE Required**
2Acv*SQRT(f'c)	1159.178	**2 Curtains Not Required**
Min. Reinf.	0.0025	
Reinf. Ratio	0.009204	**OK**
hw/lw	0.333333	
alpha	3	
Vn (kip)	5317.237	Vn=Acv(alpha*SQRT(f'c)+rho*fy)
Phi*Vn (kip)	4785.513	**WALL OK**

E-W1 Shear Wall (Cellar) Spot Check

Height (ft)	8	
Length (ft)	8.5	
Thickness (in)	24	
Diameter bars (in)	0.75	
Spacing	12	
fy (ksi)	60	
f'c (ksi)	8	
Factored Moment (ft-k)	59006.4	
Factored Shear (k)	619.2	
Factored Axial (k)	2500	
Moment of Inertia (ft^4)	102.3542	
Stress (ksi)	18.03577	
0.2*f'c (ksi)	1.6	**BE Required**
2Acv*SQRT(f'c)	437.9116	**2 Curtains Required**
Min. Reinf.	0.0025	
Reinf. Ratio	0.003068	**OK**
hw/lw	0.941176	
alpha	3	
Vn (kip)	1107.49	Vn=Acv(alpha*SQRT(f'c)+rho*fy)
Phi*Vn (kip)	996.7406	**WALL OK**

E-W3 Shear Wall (Cellar) Spot Check

Height (ft)	10	
Length (ft)	25	
Thickness (in)	18	
Diameter bars (in)	1.25	
Spacing	12	
fy (ksi)	60	
f'c (ksi)	8	
Factored Moment (ft-k)	84452.8	
Factored Shear (k)	587.2	
Factored Axial (k)	2500	
Moment of Inertia (ft^4)	1953.125	
Stress (ksi)	4.216421	
0.2*f'c (ksi)	1.6	**BE Required**
2Acv*SQRT(f'c)	965.9814	**2 Curtains Not Required**
Min. Reinf.	0.0025	
Reinf. Ratio	0.011363	**OK**
hw/lw	0.4	
alpha	3	
Vn (kip)	5130.526	Vn=Acv(alpha*SQRT(f'c)+rho*fy)
Phi*Vn (kip)	4617.473	**WALL OK**

N-S1 Shear Wall (15th Floor) Spot Check

Height (ft)	10	
Length (ft)	30	
Thickness (in)	18	
Diameter bars (in)	0.625	
Spacing	12	
fy (ksi)	60	
f'c (ksi)	6	
Factored Moment (ft-k)	38291.2	
Factored Shear (k)	460.8	
Factored Axial (k)	1100	
Moment of Inertia (ft^4)	3375	
Stress (ksi)	1.35158	
0.2*f'c (ksi)	1.2	**BE Required**
2Acv*SQRT(f'c)	1003.877	**2 Curtains Not Required**
Min. Reinf.	0.0025	
Reinf. Ratio	0.002841	**OK**
hw/lw	0.333333	
alpha	3	
Vn (kip)	2610.282	Vn=Acv(alpha*SQRT(f'c)+rho*fy)
Phi*Vn (kip)	2349.254	**WALL OK**

E-W1 Shear Wall (15th Floor) Spot Check

Height (ft)	8	
Length (ft)	8.5	
Thickness (in)	24	
Diameter bars (in)	0.75	
Spacing	12	
fy (ksi)	60	
f'c (ksi)	6	
Factored Moment (ft-k)	12592.8	
Factored Shear (k)	425.6	
Factored Axial (k)	1100	
Moment of Inertia (ft^4)	102.3542	
Stress (ksi)	4.080488	
0.2*f'c (ksi)	1.2	**BE Required**
2Acv*SQRT(f'c)	379.2425	**2 Curtains Required**
Min. Reinf.	0.0025	
Reinf. Ratio	0.003068	**OK**
hw/lw	0.941176	
alpha	3	
Vn (kip)	1019.486	Vn=Acv(alpha*SQRT(f'c)+rho*fy)
Phi*Vn (kip)	917.5374	**WALL OK**

E-W3 Shear Wall (15th Floor) Spot Check

Height (ft)	10	
Length (ft)	25	
Thickness (in)	18	
Diameter bars (in)	0.875	
Spacing	12	
fy (ksi)	60	
f'c (ksi)	6	
Factored Moment (ft-k)	18388.8	
Factored Shear (k)	329.6	
Factored Axial (k)	1100	
Moment of Inertia (ft^4)	1953.125	
Stress (ksi)	1.020984	
0.2*f'c (ksi)	1.2	**BE Not Required**
2Acv*SQRT(f'c)	836.5644	**2 Curtains Not Required**
Min. Reinf.	0.0025	
Reinf. Ratio	0.005568	**OK**
hw/lw	0.4	
alpha	3	
Vn (kip)	3058.808	Vn=Acv(alpha*SQRT(f'c)+rho*fy)
Phi*Vn (kip)	2752.927	**WALL OK**

Spandrel Beam (Cellar Floor) Spot Check

Factored Shear (kip)	403.2		
Factored Moment (ft-kip)	587.2		
h (in)	24		
ln (in)	76		
d (in)	22		
b (in)	24		
f'c (ksi)	8		
fy (ksi)	60		
Area Reinforcement (in^2)	8.89	7#10's	
rho	0.016837		
Area Shear Reinf.	1.228		
Shear Reinf. Spacing (in)	5		
ln/h	3.166667	**May use diagonal or conventional reinforcement**	
rho max	0.0316		
rho min	0.0033		
phi	0.9		0.0276
R	935.2162		
Mn (ft-kip)	905.2893		
phiMn (ft-kip)	814.7603	**Beam OK in Flexure**	
Minimum Shear Reinforcing (in^2)	0.134164		
Minimum Shear Reinforcing (in^2)	0.1		
PhiAv*fy*d/s	243.144		
Vc (kip)	94.45151		
Phi Vc (kip)	70.83863		
Shear Capactiy (kip)	313.9826	**Beam not OK in Shear**	

Spandrel Beam (25th Floor) Spot Check

Factored Shear (kip)	51.2		
Factored Moment (ft-kip)	89.6		
h (in)	24		
ln (in)	76		
d (in)	22		
b (in)	24		
f'c (ksi)	6		
fy (ksi)	60		
Area Reinforcement (in^2)	5		5#9
rho	0.00947		
Area Shear Reinf.	0.8		#4@10"
Shear Reinf. Spacing (in)	10		
ln/h	3.166667		**May use diagonal or conventional reinforcement**
rho max	0.0273		
rho min	0.0033		
phi	0.9	0.0239	
R	536.5444		
Mn (ft-kip)	519.375		
phiMn (ft-kip)	467.4375		**Beam OK in Flexure**
Minimum Shear Reinforcing (in^2)	0.232379		
Minimum Shear Reinforcing (in^2)	0.2		
PhiAv*fy*d/s	79.2		
Vc (kip)	81.79741		
Phi Vc (kip)	61.34806		
Shear Capactiy (kip)	140.5481		**Beam OK in Shear**

ASCE7 – 05 Allowable Seismic Drift Calculation

I = 1.25
Cd = 4 (Table 12.2-1; Ordinary Concrete Shear Walls)
Dx = Cd * dx / I = 4*dx/1.25 = 3.2dx
Delta Max = 0.020hsx (Table 12.12-1)

Dx < Delta Max
3.2dx < 0.020hsx
dx < hsx/160