Structural Technical Report 3
Lateral System Analysis and Confirmation Design

PricewaterhouseCoopers
Oslo, Norway

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Structural Option
AE 481W Senior Thesis
The Pennsylvania State University
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Executive Summary

Technical report three contains an analysis of the lateral force resisting system of the PricewaterhouseCoopers building. The buildings lateral force resisting system consists of cast in place concrete shear walls located at the center of each leg of the building. Concrete plank decking acts as a rigid diaphragm that transfers lateral loads to the shear walls. Shear walls are typically 400mm thick in the short direction and 300mm in the long direction.

The shear walls of the superstructure are integrated into a two story cast in place concrete substructure. The substructure acts as a base to distribute the overturning moments to the foundation. The foundation uses steel and concrete piles to transfer axial tension, axial compression and lateral loads to the ground.

To confirm the design of the existing lateral system a simplified 3-D structural model was constructed using ETABS. The model was simplified to include the lateral force resisting elements only. Lateral loads were calculated in accordance with ASCE 7-05 and applied to the model.

The simplified analysis confirmed that the existing design was adequate within the limits of the codes and reference standards. The ETABS model revealed the building is very stiff, with a maximum building deflection below one inch. Torsion is experienced in the building under both wind and seismic loads. Spot checks, using forces drawn from ETABS output, revealed only minimum reinforcement was required in shear walls.

Due to the geometry of the shear wall system, this technical report relied heavily the results obtained from the ETABS structural model. As I am still in the process of learning about the program, I am not as confident as I would like to be about the accuracy of results obtained. Some of the major concerns are smaller deflections than expected and fluctuation of in-plane and out-of-plane shears. Further investigation should be made to verify whether results obtained are accurate.

1 – Existing Conditions

1.1 Architecture

In 2003 *Oslo S Utvikling* hosted an international architecture competition for the lot located south of the *Oslo S* train lines - between the outrun of *Akerselven* and *Middeladerparken*. The competition was jointly won by *MVRDV*, *Dark Arkitekter*, and *A-lab* with their proposal for the *Barcode* development. The new *PricewaterhouseCoopers* (PwC) building is the first building to be completed in the *Barcode* strip and will be “the face” of the *Barcode* towards the west. The *Barcode* concept is based on a series of eight to ten buildings, each with their own individual form and character. The intention is to provide unique multifunctional architecture with a lot of light, variation and accessibility.

Figure 1: Barcode Concept

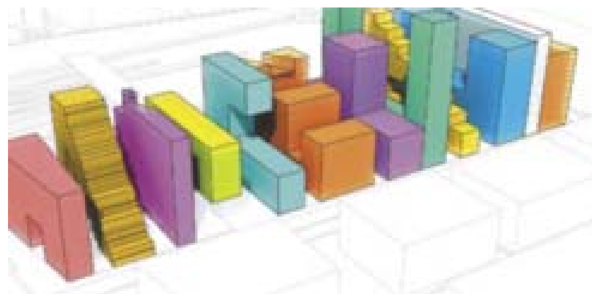


Figure 2: Image Barcode Concept

- Images courtesy of Oslo S Utvikling

The exterior shape of the PwC building is simple and defined. The east side runs perpendicular to *Nydalen Alle* and the west side follows the property line, creating a rhombus like shape in plan. There are of two stories below grade and twelve above grade with a five story opening in the center of the façade indicating the main entrance. The building envelope consists of curtainwall glazing, metal paneling and tar paper roof, intended to give off an impression of lightness, openness and technological sophistication. The story height is 12 ft which will be similar for all the buildings in the *Barcode* development.

The program inside mainly conforms to the needs of the professional services firm, PricewaterhouseCoopers. Technical rooms and parking are located on sub grade floors. The first three floors above grade contain an auditorium, a reception area, meeting rooms, and towards *Nydalen Alle*, shops and display rooms. The forth through the eleventh floors hold conference and office spaces. A grand cafeteria with spectacular views and outdoor dining options is located on the top floor. The core consists of a permanent technical zone that contains communication, technical installations and wet services, in addition to zones that can be designed differently depending on the need of the different departments.

1.2 Drawings

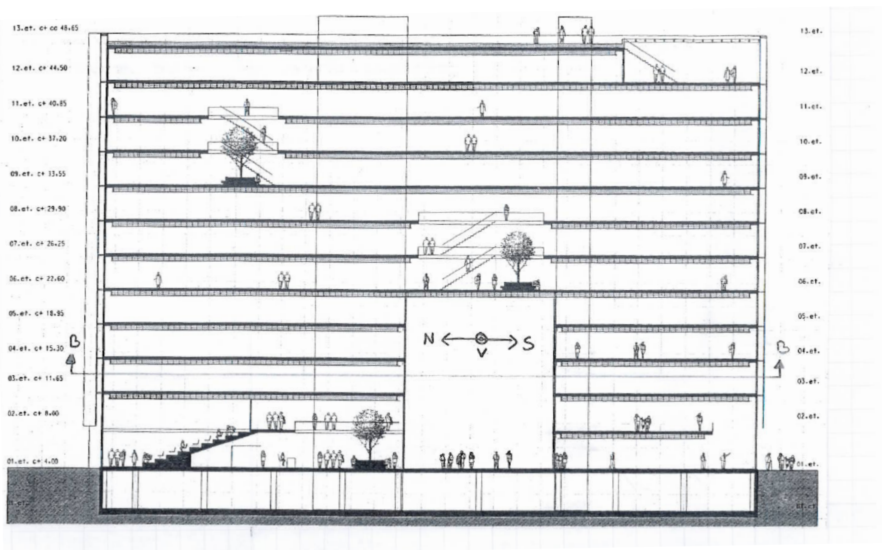


Figure 3: Building Section

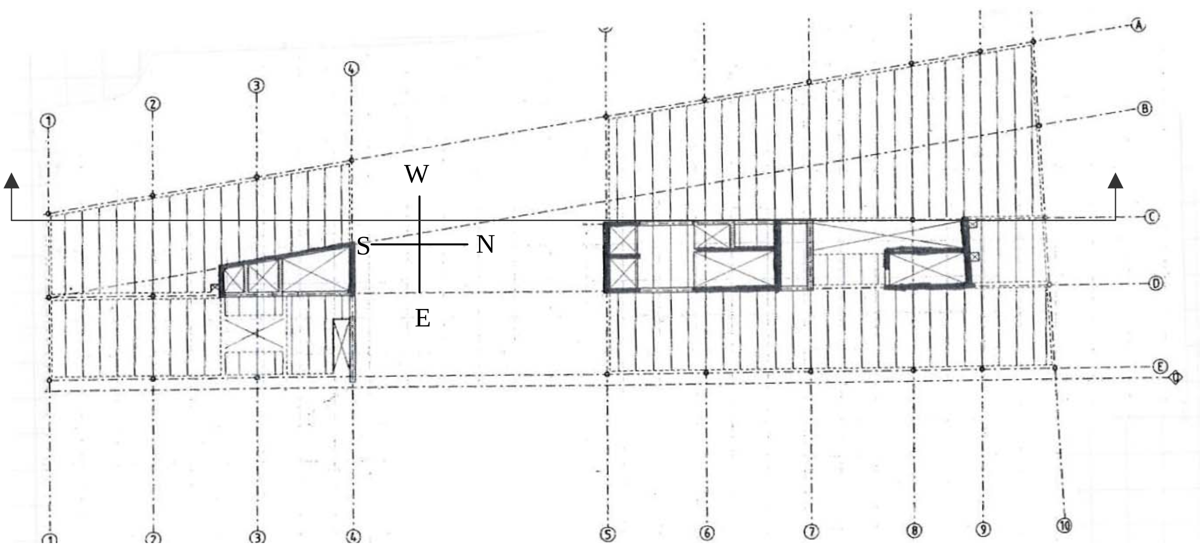


Figure 4: Typical framing plan for floors 1 – 4

1.3 Gravity System Discussion

The superstructure of the building consists of precast concrete decking on a steel frame with cast in place shear walls at the core. The decking is prestressed hollow core concrete plank with typical sections of 120cmx30cm and spans ranging from 10 to 20 meters. Along the interior of the building, planks typically rest on steel angles fastened to the concrete core (figure 6). Along the exterior, planks typically rest on the bottom flange of a special steel beam (HSQ profile, figure 5). The beams are fabricated by precast engineer and conceal the flange and web within the plane of the slab, creating extremely low structural depth. The beams are supported by circular hollow structural steel columns filled with reinforced concrete. The opening at the center of the façade is allowed through three trusses comprised of hollow circular steel tubing for diagonal/vertical members and HSQ beams for horizontal members.

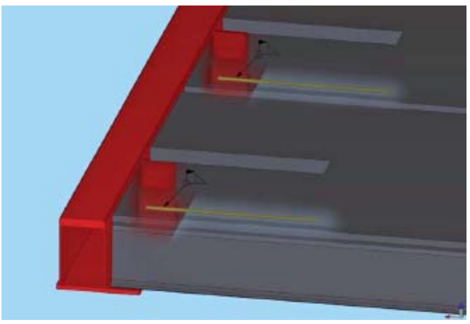


Figure 5: Principle connection of deck elements with one sided HSQ steel beam.

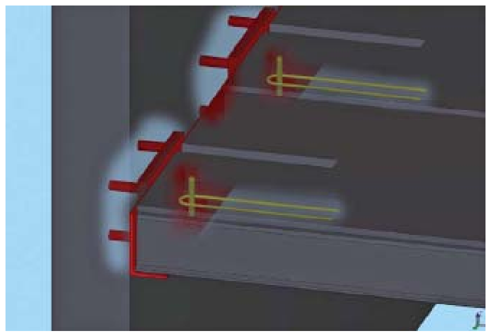


Figure 6: Principle connection of deck elements with interior concrete shear wall.

-images courtesy of Norsk Stålforbund and Betongelement Foreningen

There are two stories below grade comprised of cast in place concrete. The lowest level has a slab thickness of 500mm with recessed areas for elevator shafts. The other floor slabs below grade are 300mm thick, with exception of areas below outdoor areas where slab thickness is increased to 400mm.

1.4 Lateral System Discussion

Lateral resistance is provided by cast in place concrete shear walls located at the center of each leg of the building. Concrete plank decking acts as a rigid diaphragm that transfers loads to the shear walls. The building is tall and narrow in the short direction and therefore requires thick shear walls. Walls are typically 400mm thick in the short direction and 300mm in the long direction.

The narrow building shape also causes large overturning moments. Cores are integrated into the cast in place concrete substructure and acts as a base to distribute the overturning moments to the foundation. The foundation uses steel and concrete piles to transfer axial tension, axial compression and lateral loads to the ground. Piles are driven between 100 and 130ft to bedrock.

Material Properties of Concrete used in shear walls:

Item	Norwegian Standard	Eurocode CEN	f_{ck} (ksi)	f_{ctm} (ksi)	E_{cm} (ksi)
Cast in place concrete	B35	C35/45	5	0.46	4 850

f_{ck} - compressive cylinder strength at 28days
f_{ctm} - value of mean axial tensile strength of concrete
E_{cm} – Secant modulous of elasticity

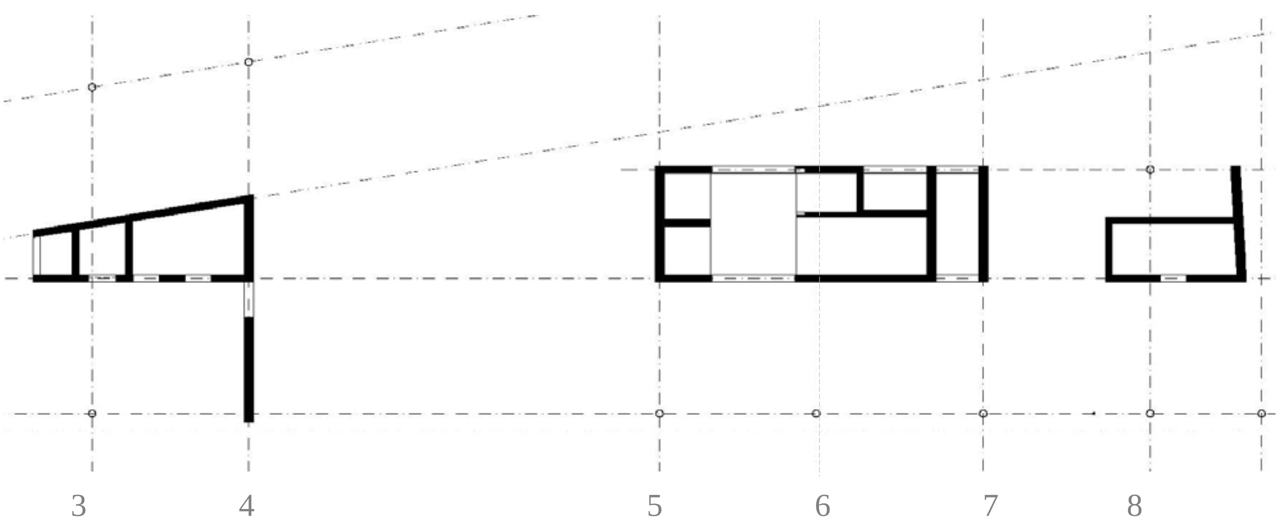


Figure 6: Typical Shear wall layout

2 – Confirmation Design

2.1 ETABS Model

To confirm the design of the existing lateral system a 3-D structural model was constructed using ETABS. Lateral loads were calculated in accordance with ASCE07-05 and applied to the model to determine reactions of the structure.

A 3-D analysis was chosen opposed to a 2-D analysis due to the configuration of shear walls. The shear walls are interconnected at the core causing them work together and contribute with out-of-plane loading. This makes the overall structure stiffer than if the walls were considered acting alone. A 3-D analysis was therefore found as the most appropriate means for determining distribution of lateral loads.

The ETABS model was simplified to a model of the superstructure lateral system only. This was done in order easily obtain direct analysis and minimize source of errors incurred with modeling gravity elements. Shear walls were modeled as membrane elements with thicknesses matching actual design. The shear walls were manually meshed to ensure meshing-joints coincided over openings and with connecting walls. The diaphragms were modeled as perfectly rigid such that the applied point loads would be distributed according to relative stiffness of the shear walls. A distributed mass including self weight and superimposed dead loads were also applied to the diaphragms. For simplicity, façade, beam and column loads were considered to be evenly distributed across the diaphragm at each story.

Lateral loads were manually applied as point loads to the rigid diaphragms at each story. Wind loads were applied at the centers of pressure for the North and West face. These faces yield the most conservative results as they create the largest eccentricity from the center of rigidity. Seismic loads were applied at the center of mass of each story.

The model assumed the shear walls of the superstructure to be fixed to a perfectly rigid substructure. Therefore there are some aspects of the lateral system not addressed in this technical report and may need further investigation. When the relatively flexible shear walls meet the rigid substructure, shear reversals occur (figure 7). Another issue is reduced drift values. Although the sub structure is relatively stiff, some increased deflection would occur if included in structural model.

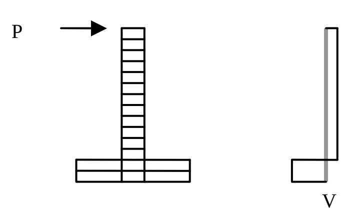


Figure 7: Concept of shear reversals

ETABS MODEL

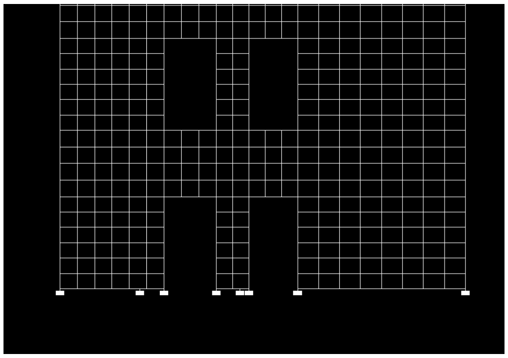


Figure 8: Elevation showing shear wall meshing

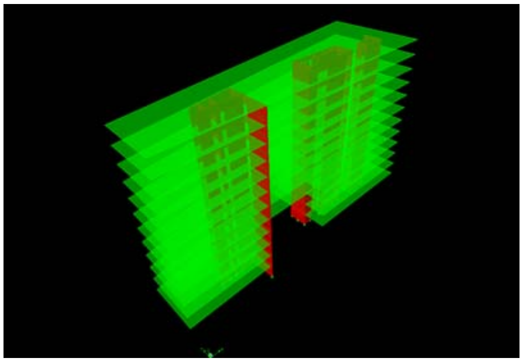


Figure 9: 3-D view

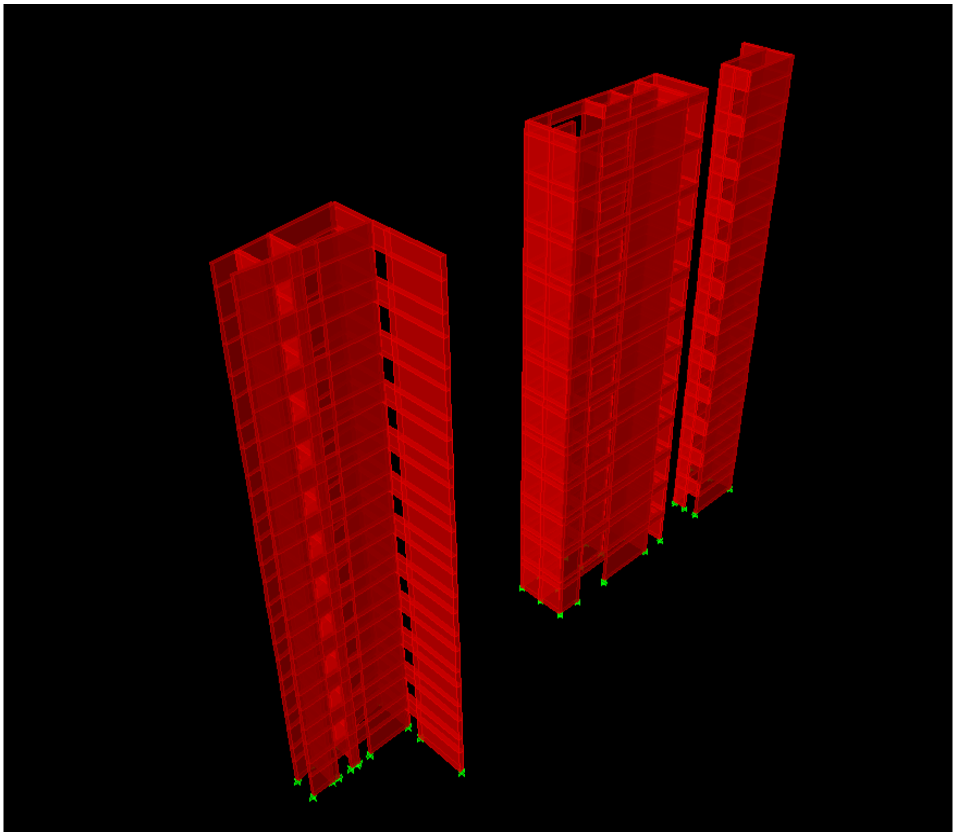


Figure 10: 3-D view showing shear wall core

2.2 Wind Loads

For the purpose of determining approximate wind loads, calculations have been made in accordance with ASCE 7 - 05. In technical report 1, Boston, MA was chosen as an equivalent building location for the US. This however yielded wind pressures considerably higher than those obtained by the design engineer. This indicated I might have been overly conservative when choosing Boson as equivalent building location. Boston has a reference wind speed of 120mph, which is relatively high due to the exposure of hurricanes along the East Coast. As Oslo does not see as high wind speeds, calculations have been revised using a reference wind speed of 100mph. The revised wind pressures are summarized in figure 11 and 12 below.

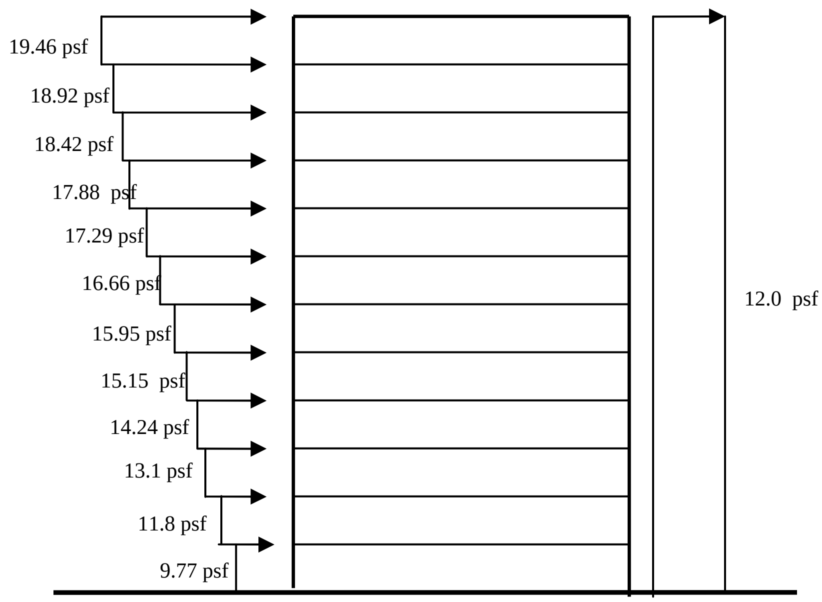


Figure 11: Wind pressure in the North / South direction.
For information on calculations see Appendix B

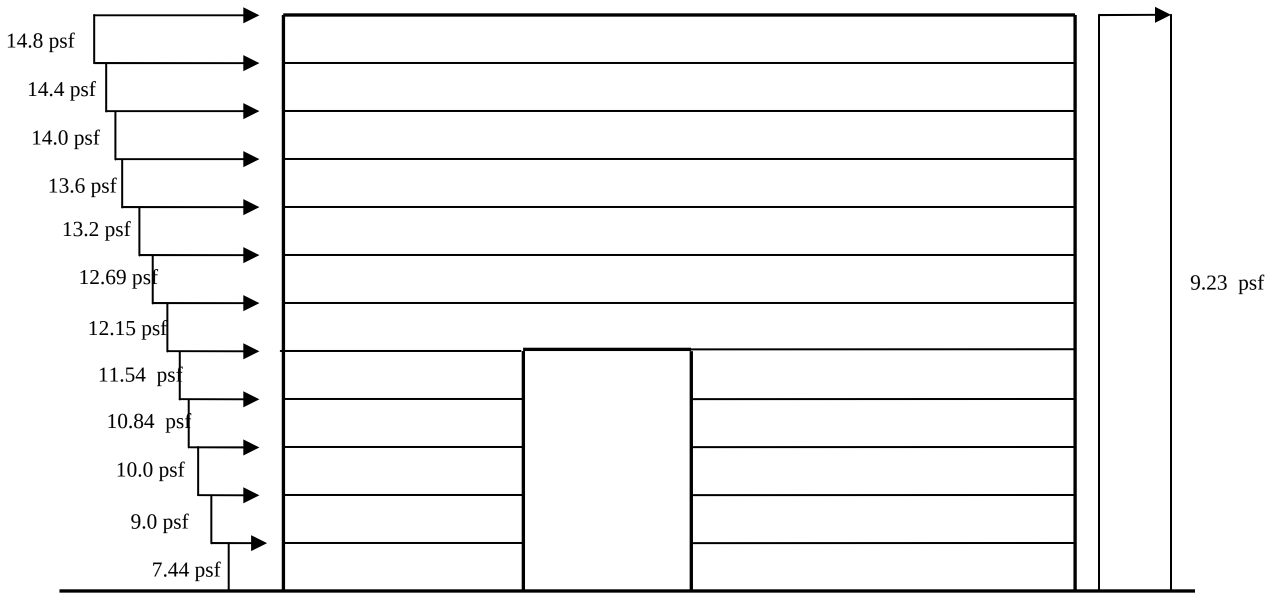


Figure 12: Wind pressure in the East / West direction
For information on calculations see Appendix B

2.3 Seismic Loads

For the purpose of determining approximate seismic loads on the structure, loads have been calculated in accordance with ASCE 7 - 05 for an equivalent US location of Boston, MA. The loads determined in technical report one were low due to calculation of a high fundamental period for the given structure type. Calculation of fundamental period was revised using equation 12.8.7 of ASCE 7, which yielded a much lower fundamental period and thus increasing Cs and shear loads. The revised seismic loads are summarized in the figure 13 below:

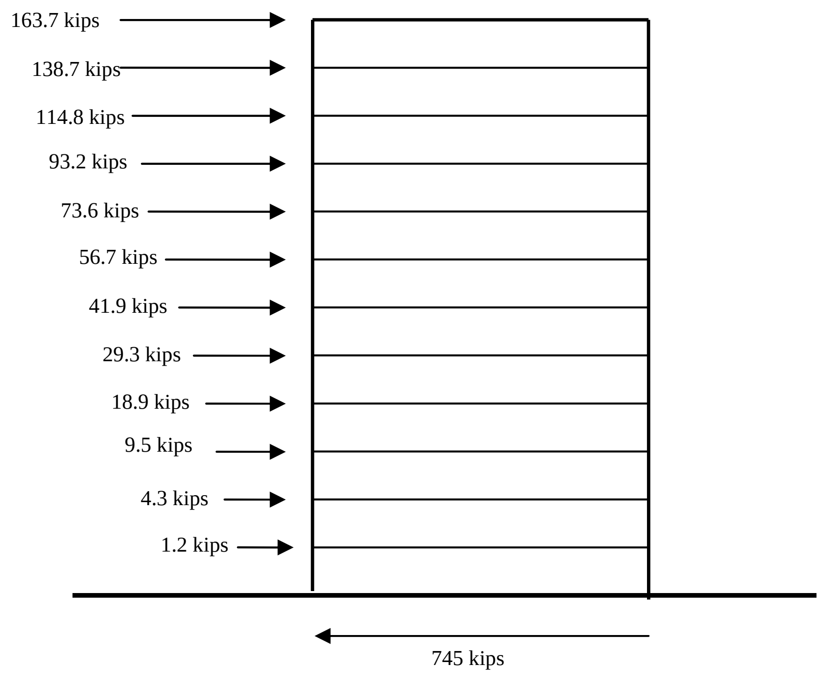


Figure 13: Distribution of lateral seismic forces on structure
For information on calculations see Appendix A

2.4 Load Combinations:

The following load combinations of ASCE 7-05, Section 2.3.2 were considered for strength design:

1. 1.4(D + F)
2. 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R)
3. 1.2D + 1.6(Lr or S or R) + (L or 0.8W)
4. **1.2D + 1.6W + L + 0.5(Lr or S or R)**
5. **1.2D + 1.0E + L + 0.2S**
6. 0.9D + 1.6W + 1.6H
7. 0.9D + 1.0E + 1.6H

The load combinations were applied to the ETABS model and resulting member forces were reviewed to determine the controlling load combination. It was determined that combination 4 controlled in the short direction and combination 5 controlled in the long direction. By inspection combinations 1 and 2 do not control for lateral loads on the structure. Load combinations 6 and 7 will also not control by inspection, due to no presence of earth pressure on the superstructure.

2.5 Load Distribution

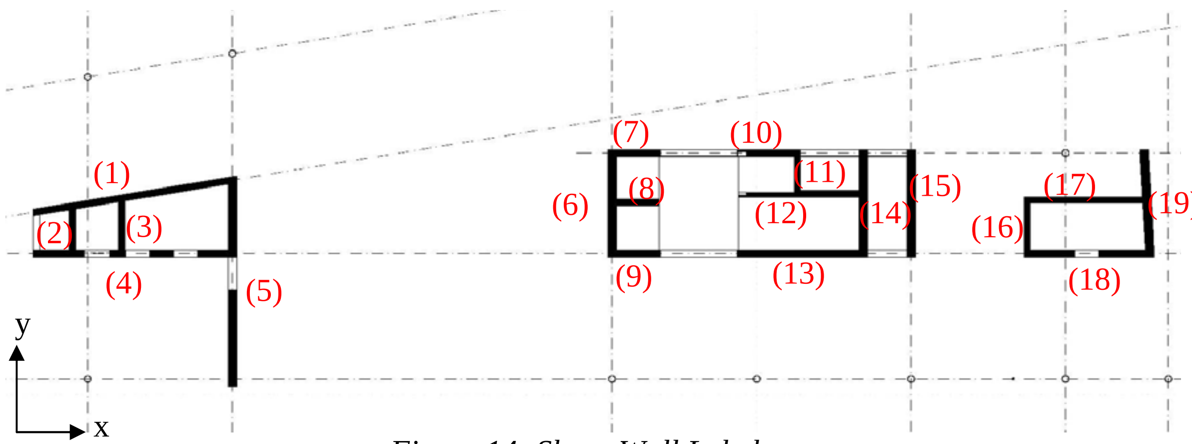


Figure 14: Shear Wall Labels

<u>ETABS RESULTS</u>: IN PLANE SHEAR FORCE DUE TO WIND LOADING

LOADING IN X DIRECTON

Shear Wall	1st Story Force (kip)	5th Story Force (kip)	12th Story Force (kip)
1	137.51	155.58	15.88
2	11.22	20.17	5.14
3	10.35	5.61	3.76
4	74.17	24.65	40.12
5	6.98	22.31	18.6
6	2.03	4.58	5.05
7	18.16	20.89	20.96
8	11.91	0.02	2.75
9	17.08	14.52	11.17
10	28.99	28.81	18.78
11	13.79	21.24	7.62
12	63.35	51.78	4.32
13	65.06	42.92	20.23
14	3.56	0.72	4.27
15	8.75	0.9 9	1.87
16	0.19	5.47	7.07
17	66.81	55.63	7.12
18	43.92	1.41	22.56
19	14.21	13	6.46

LOADING IN Y DIRECTION

Shear Wall	1st Story Force (kip)	5th Story Force (kip)	12th Story Force (kip)
1	21.57	44.54	39.45
2	43.41	22.55	6.6
3	84.86	35.44	2.92
4	13.15	41.83	19.46
5	412.75	379.21	64.56
6	156.23	105.38	-3.2
7	17.25	24.35	7.8
8	1.33	0.47	-0.74
9	19.22	13.86	9.72
10	23.54	18.35	3.7
11	31.57	31.33	1.09
12	36.97	69.86	11.94
13	46.99	114.26	40.45
14	130.21	124.18	18.47
15	82.87	36.14	0.21
16	14.66	13.47	0.6
17	3.23	8.87	3.39
18	12.7	45.98	10.53
19	112.29	82.01	8.81

ETABS RESULTS: IN PLANE SHEAR FORCE DUE TO SEISMIC LOADING

LOADING IN X DIRECTON

Shear Wall	1st Story Force (kip)	5th Story Force (kip)	12th Story Force (kip)
1	182	249.48	41.84
2	10	27.59	9.94
3	5	2.79	3.88
4	106	67.04	38.2
5	21	60.04	29.69
6	2	7.83	8.72
7	25	37.39	44.25
8	16	0.23	5.76
9	25	29.91	31.46
10	38	47.8	40.27
11	17	33.33	18.29
12	82	81.66	16.68
13	96	96.24	62.31
14	7	14.25	11.34
15	3	2.88	4.8
16	1	7.35	8.21
17	84	80.54	8.48
18	64	18.93	29.38
19	2	2.13	9.07

LOADING IN Y DIRECTION

Shear Wall	1st Story Force (kip)	5th Story Force (kip)	12th Story Force (kip)
1	1.5	77.32	77.19
2	18.0	6.74	11.53
3	40.5	18.03	4.18
4	1.4	5.61	20.05
5	226.0	297	122.55
6	95.4	84.8	4.97
7	8.4	15.95	2.59
8	1.1	0.3	0.71
9	9.8	8.03	12.05
10	20.9	24.87	-8.26
11	22.3	32.63	9.05
12	29.8	72.96	26.77
13	42.8	116.17	61.68
14	95.5	121.33	36.32
15	59.6	37.12	5.48
16	9.4	5.05	7.84
17	2.8	20.22	24.77
18	21.2	73	36.04
19	100.6	97.72	28.13

The shear walls that resist the largest amount of shear are highlighted in green and consequently oriented in the same plane as the applied lateral loads.

The fluctuation of shears pose concern to the accuracy of the model. Some shear walls show larger shears at the fifth level than at the first. Although this may be due to torsion, shear is expected to get larger towards the base as more loads are applied. I can not reason why this is happening, which indicates there might be an error in the ETABS model. Another area of concern is the large out of plane shears. Walls oriented in the weak direction are expected to have close to zero shear at the base, however it can be seen that this is not happening.

2.6 Relative stiffness

An investigation was made to determine relative stiffness of each shear wall. A 1000k fictional load was applied to the 12th floor in each direction at the center of gravity. From analysis output, load distribution to each shear wall was determined. The results indicated that critical members are shear wall (1) when loaded in the X direction and shear wall (5) when loaded in the Y direction. With exception of these members loads appear to be evenly distributed across shear walls in the strong direction.

ETABS RESULTS: SHEAR FORCE AT BASE

1000 k load in X direction			
	Shear X dir	**Shear Y dir**	**%**
1	225	-2	23
2	6.1	26	1
3	12.8	-16	1
4	132.0	1	13
5	0.0	-108	0
6	10.1	0	1
7	55.2	-6	6
8	1.8	1	0
9	52.5	7	5
10	49.6	-2	5
11	39.5	11	4
12	111.4	0	11
13	170.3	-6	17
14	0.0	45	0
15	0.0	13	0
16	1.2	6	0
17	72.9	3	7
18	59.0	-5	6
19	0.0	32	0
Total	**999.44**	**-0.18**	**99.9**

1000 k load in Y direction			
	Shear X dir	**Shear Y dir**	**%**
1	-58	14	1
2	7	21	2
3	6	51	5
4	-61	6	1
5	0	450	45
6	0	116	12
7	31	4	0
8	0	-5	-1
9	-12	-2	0
10	-30	6	1
11	-7	34	3
12	-71	-5	0
13	144	4	0
14	0	147	15
15	0	43	4
16	1	13	1
17	20	0	0
18	45	2	0
19	0	101	10
Total	**14.9**	**1000**	**100**

2.7 Torsion

The PWC building experiences torsion under wind and seismic loading in both directions. As the building is not symmetrical about either axis, the center of mass does not coincide with the center of rigidity (figure 15). Therefore under seismic loading, the building experiences torsion corresponding to its eccentricity. Torsion is greater when loaded in the long direction as the eccentricity is larger. Modal analysis from ETABS also revealed that torsional effects are an important consideration in the PWC building. The first-mode period of vibration was about the z axis. The building also experiences torsion under wind loading since the center of pressure does not coincide with the center of rigidity.

Story	Center of Mass (in)		Center of Rigidity (in)	
	X	Y	X	Y
12	1509	587	1363	478
11	1509	587	1363	478
10	1509	587	1363	478
9	1509	587	1363	478
8	1509	587	1363	478
7	1509	587	1363	478
6	1509	587	1363	478
5	1509	587	1363	478
4	1608	593	1363	478
3	1608	593	1363	478
2	1608	593	1363	478
1	1608	593	1363	478

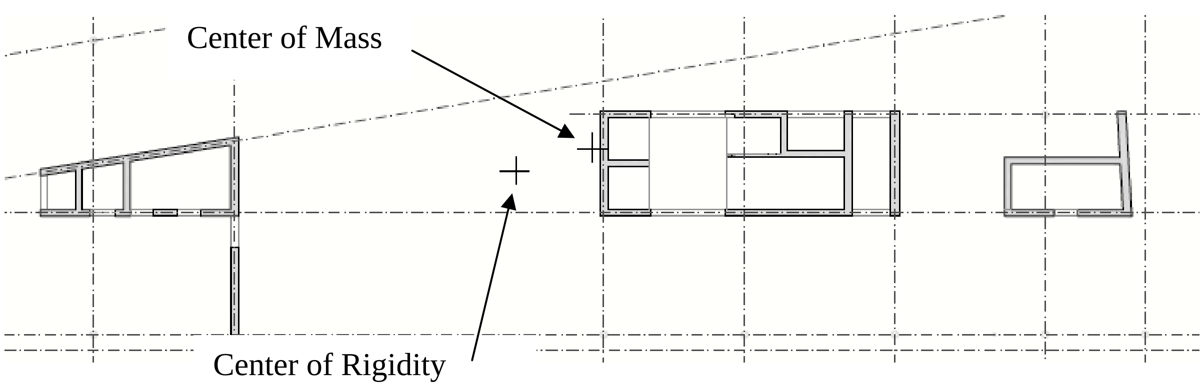


Figure 15: Location of COR and COM

2.8 Overturning

Overturning moment on the PWC needs to be considered in foundation design. The building is narrow, and the superstructure is relatively light thus causing considerable overturning moments in the short direction. Critical overturning moments were calculated by multiplying story shears by story height. The critical load combination was determined to be: 1.2D + 1.0E + L + 0.2S

Story	hx (ft)	F (k)	Overturning Moment (ft-k)
Roof	146	164	23978
12	133	139	18425
11	121	115	13879
10	109	93	10150
9	97	74	7120
8	85	57	4819
7	73	42	3055
6	61	29	1785
5	49	19	926
4	37	9	350
3	25	4	109
2	13	1	16
1	0	0	0
Totals	**-**	**746**	**84611**

2.9 Drift

The ETABS model revealed that the building is very stiff and deflections are well within the serviceability limits of ASCE 7 - 05. Wind load deflections were within the limits of h/400 and seismic deflections were within the story drift limit of $.002h_{sx}$. Seismic loads created the largest deflections at the 12th story of 0.7" in the long direction and 0.3" in the short direction. As expected from the narrow shape of the building, deflections in the long direction were larger than deflections in the short direction. Overall however, the deflections are very small.

12Th Story Displacement

Loading Type	Loading Dir.	X Disp. (in)	Y Disp. (in)	Allowable (in)
Wind	X Direction	0.104	-0.073	4.38
Wind	Y Direction	0.048	0.501	4.38
Seismic	X Direction	0.318	0.151	(drift limits)
Seismic	Y Direction	0.123	0.705	(drift limits)

2.10 Conclusion

The simplified analysis confirmed that the existing design was adequate within the limits of the codes and reference standards. The ETABS model revealed the building is very stiff, with a maximum building deflection below one inch. Torsion is experienced in the building under both wind and seismic loads. Spot checks, using forces drawn from ETABS output, revealed only minimum reinforcement was required in shear walls.

Due to the geometry of the shear wall system, this Technical Report relied heavily the results obtained from the ETABS structural model. As I am still in the process of learning the program, I am not as confident as I would like to be about the results obtained. Some of the major concerns are smaller than expected deflections and fluctuation of in plane and out of plane shears. Further investigation to verify results could include:

- Remodeling the building in ETABS using beam elements for coupling beams rather than meshed membrane elements. This should yield very similar results.
- Create another structural model using a different program such as STAAD or RAM and cross reference results.
- Include sub grade levels in structural model.

Simplification of the model to only include the superstructure left some design aspects of the lateral system unaddressed:

- Shear reversals in the shear walls where the superstructure meets the substructure.
- Foundation capacities with respect to overturning moments

A – Appendix

A.1 Seismic Loads

N/W Direction		
Location	Boston, Mass	
Latitude	42.35	
Longitude	-71.06	
Site Class	D	Table 20.3 - 1
Ss	0.28	USGA Java Motion Parameter:
S1	0.068	USGA Java Motion Parameter:
Fa	1.577	Table 11.4-1
Fv	2.4	Table 11.4-2
S_{MS}	0.44156	Eq 11.4-1
S_{M1}	0.1632	Eq 11.4-2
SDs	0.293	USGA Java Motion Parameter:
SD1	0.108	USGA Java Motion Parameter:
Occupancy Category	II	IBC Table 1604.5
T	0.84	Sec 12.8.2
SDC	B	Table 11.6-1

N/W Direction		
R	5	Table 12.2-1
I	1	Table 11.5-1
T_L	6	Figure 22-15
H	147	
Cs	0.026	

E/W Direction		
R	5	Table 12.2-1
I	1	Table 11.5-1
T_L	6	Figure 22-15
H	147	
Cs	0.026	

Story	w_x (kips)	h_x	k	$w_x h_x$^k	C_{vx}	F_x (kips)	V_x(kips)
Roof	2402	146	2	51524296	0.22	163.7	164
12	2473	133	2	43648109	0.19	138.7	302
11	2473	121	2	36135192	0.15	114.8	417
10	2473	109	2	29331316	0.12	93.2	510
9	2473	97	2	23157913	0.10	73.6	584
8	2473	85	2	17850686	0.08	56.7	641
7	2473	73	2	13173932	0.06	41.9	683
6	2473	61	2	9206218	0.04	29.3	712
5	2471	49	2	5940524	0.03	18.9	731
4	2165	37	2	2974004	0.01	9.5	740
3	2165	25	2	1363037	0.01	4.3	745
2	2165	13	2	372653	0.00	1.2	746
1	0	0	2	0	0.00	0	
Totals	28682	NA	NA	234677883		745.7	746

V = Cs*W	**745.727944**

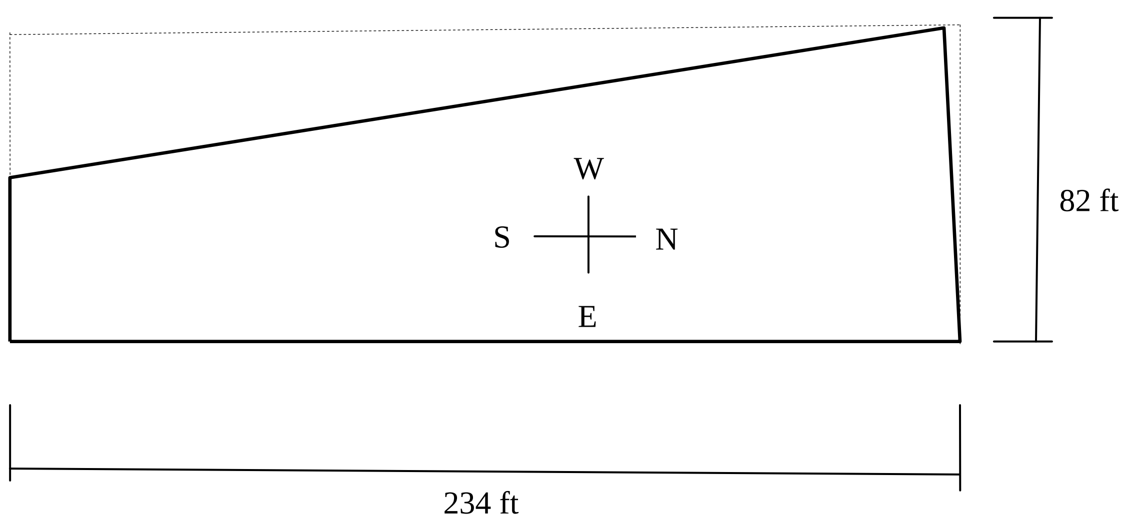
A.2 Wind Loads

Velocity Pressure	
V – Basic Wind speed	100
Occupancy Category	III
Kd	0.85
Importance Factor	1
Exposure Category B	B
Kzt	1

Pressure Coefficients	
C_p Windward wall	0.8
Cp Leeward wall	-0.5
C_p Side wall	-0.7

Internal pressure coefficients
Enclosed Structure
GC_{pi} = +/- 0.18

Gust Factor		
	N/S	E/W
L	82	234
B	234	82
h	147	147
n1	0.6	0.6
gQ, gv	3.4	3.4
gr	4.05	4.05
zhat	87.6	87.6
Iz	0.25	0.25
Lz	441.8	441.8
Q	0.72	0.84
V	100	100
Vz	83.6	83.6
N1	3.17	3.17
Rn	0.067	0.067
nh	4.9	4.9
Rh	0.18	0.18
nb	7.7	2.7
RB	0.12	0.30
nl	9.1	25.9
RL	0.10	0.04
R	0.014	0.013
G	0.772782	0.838303



North / South

Floor	height (ft)	Kz	qz	Pressure (psf)						
				N/S Windward			N/S Leeward			Total
Roof	146	1.102	23.98	16.08	+/-	4.32	-15.59	+/-	4.32	31.67
12	133	1.072	28.22	18.92	+/-	4.32	-15.59	+/-	4.32	34.51
11	121	1.043	27.47	18.42	+/-	4.32	-15.59	+/-	4.32	34.01
10	109	1.013	26.66	17.88	+/-	4.32	-15.59	+/-	4.32	33.47
9	97	0.979	25.78	17.29	+/-	4.32	-15.59	+/-	4.32	32.87
8	85	0.943	24.84	16.66	+/-	4.32	-15.59	+/-	4.32	32.24
7	73	0.903	23.78	15.95	+/-	4.32	-15.59	+/-	4.32	31.54
6	61	0.858	22.59	15.15	+/-	4.32	-15.59	+/-	4.32	30.74
5	49	0.806	21.23	14.24	+/-	4.32	-15.59	+/-	4.32	29.82
4	37	0.744	19.60	13.14	+/-	4.32	-15.59	+/-	4.32	28.73
3	25	0.666	17.53	11.76	+/-	4.32	-15.59	+/-	4.32	27.34
2	13	0.553	14.56	9.77	+/-	4.32	-15.59	+/-	4.32	25.35
1	0									0.00

East / West

Floor	hx	Kz	qz	Pressure						
				N/S Windward			N/S Leeward			Total
Roof	146	1.102	23.98	14.82	+/-	4.32	-11.31	+/-	4.32	26.13
12	133	1.072	23.32	14.42	+/-	4.32	-11.31	+/-	4.32	25.72
11	121	1.043	22.70	14.03	+/-	4.32	-11.31	+/-	4.32	25.34
10	109	1.013	22.03	13.62	+/-	4.32	-11.31	+/-	4.32	24.93
9	97	0.979	21.30	13.17	+/-	4.32	-11.31	+/-	4.32	24.48
8	85	0.943	20.52	12.69	+/-	4.32	-11.31	+/-	4.32	23.99
7	73	0.903	19.65	12.15	+/-	4.32	-11.31	+/-	4.32	23.46
6	61	0.858	18.67	11.54	+/-	4.32	-11.31	+/-	4.32	22.85
5	49	0.806	17.54	10.84	+/-	4.32	-11.31	+/-	4.32	22.15
4	37	0.744	16.19	10.01	+/-	4.32	-11.31	+/-	4.32	21.32
3	25	0.666	14.49	8.95	+/-	4.32	-11.31	+/-	4.32	20.26
2	13	0.553	12.04	7.44	+/-	4.32	-11.31	+/-	4.32	18.75
1	0		0.00							0.00

A.3 Drift Output from ETABS due to Wind Loading

Drift Due to wind in the X direction:

STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
STORY12	0.104402	-0.073343	0.000066	0.000048
STORY11	0.094907	-0.066430	0.000068	0.000049
STORY10	0.085084	-0.059423	0.000070	0.000050
STORY9	0.074996	-0.052273	0.000072	0.000051
STORY8	0.064695	-0.044992	0.000072	0.000051
STORY7	0.054280	-0.037637	0.000072	0.000051
STORY6	0.043915	-0.030319	0.000070	0.000050
STORY5	0.033823	-0.023195	0.000066	0.000047
STORY4	0.024283	-0.016469	0.000060	0.000042
STORY3	0.015645	-0.010400	0.000051	0.000035
STORY2	0.008331	-0.005316	0.000038	0.000026
STORY1	0.002854	-0.001647	0.000020	0.000011

Drift due to wind in the Y direction:

STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
STORY12	0.048420	0.508976	0.000044	0.000361
STORY11	0.042169	0.457052	0.000043	0.000365
STORY10	0.035997	0.404569	0.000042	0.000368
STORY9	0.030005	0.351732	0.000040	0.000367
STORY8	0.024256	0.298971	0.000038	0.000362
STORY7	0.018830	0.246925	0.000035	0.000351
STORY6	0.013833	0.196429	0.000031	0.000334
STORY5	0.009390	0.148501	0.000026	0.000307
STORY4	0.005647	0.104356	0.000020	0.000271
STORY3	0.002754	0.065438	0.000013	0.000222
STORY2	0.000843	0.033472	0.000006	0.000159
STORY1	-0.000013	0.010579	0.000000	0.000074

A.4 Drift Output from ETABS due to Seismic Loading

Loading in X direction:

STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
STORY12	0.318041	0.151243	0.000220	0.000121
STORY11	0.286402	0.133793	0.000227	0.000121
STORY10	0.253742	0.116425	0.000231	0.000119
STORY9	0.220489	0.099260	0.000233	0.000117
STORY8	0.187064	0.082463	0.000230	0.000113
STORY7	0.154023	0.066273	0.000223	0.000106
STORY6	0.122028	0.050992	0.000210	0.000098
STORY5	0.091830	0.036976	0.000192	0.000086
STORY4	0.064258	0.024622	0.000167	0.000071
STORY3	0.040218	0.014353	0.000136	0.000054
STORY2	0.020703	0.006602	0.000097	0.000034
STORY1	0.006809	0.001753	0.000047	0.000012

Loading in Y direction:

STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
STORY12	0.123394	0.705319	0.000103	0.000541
STORY11	0.108651	0.627552	0.000102	0.000544
STORY10	0.093938	0.549355	0.000100	0.000543
STORY9	0.079515	0.471292	0.000097	0.000535
STORY8	0.065543	0.394396	0.000093	0.000518
STORY7	0.052219	0.319929	0.000087	0.000491
STORY6	0.039787	0.249312	0.000078	0.000454
STORY5	0.028521	0.184086	0.000068	0.000405
STORY4	0.018721	0.125884	0.000056	0.000344
STORY3	0.010692	0.076436	0.000041	0.000271
STORY2	0.004744	0.037556	0.000025	0.000183
STORY1	0.001144	0.011222	0.000008	0.000078

A.5 Drift Check

PWC | DRIFT CHECK | 1/1

SEISMIC

DRIFT @ 12th STORY

12.2-1 $\delta = \frac{C_d \delta_{xe}}{I}$ — $C_d = 4.0$, $I = 1.0$, δ = From ETABS MODEL

- X DIR. $\delta = \frac{(4)(.318)}{1} = 1.27"$
- Y DIR $\delta = \frac{(4)(.705)}{1} = 2.82"$

ALLOWABLE STORY DRIFT

Table 12.12-1 $\Delta = .020 h_{sx} = (.020)(146 ft \times 12) = 35.04"$

$1.27" < 43.8"$ ∴ OK

$2.82" < 43.8"$ ∴ OK

WIND

DEFLECTION FROM ETABS

$\Delta_x = .104$ in

$\Delta_y = .501$ in

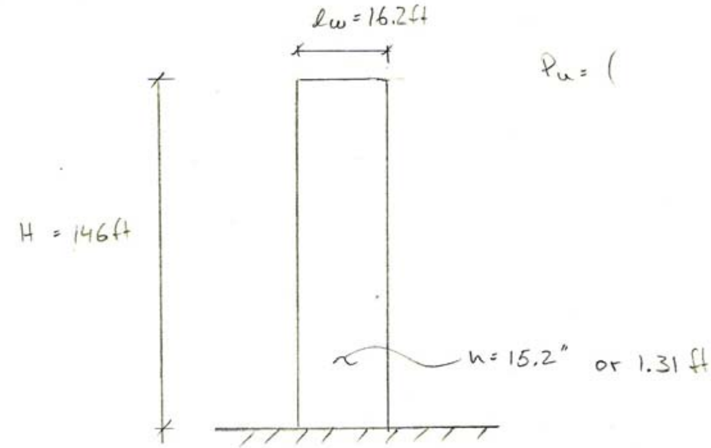
ALLOWABLE DEFLECTION

$\Delta = \frac{\ell}{400} = \frac{(146)(12)}{400} = 4.38$ in ∴ OK

A.5 Shear Wall Spot Check

PWC | SW(15) | SPOT CHECK | 1/2

SHEAR WALL (15)



FROM ETABS

CONTROLLING LOAD COMB: 1.2D + 1.6W + L + 0.5S

(1.6) WIND: $V_u = 82^k$

$M_u = 4367$

$P_u = 125^k$

MATERIALS

$f'_c = 5000$ psi

$f_y = 60\,000$ psi

CHECK NEED FOR B.E.

$$f_c = \frac{P_u}{A_g} + \frac{(M_u)\left(\frac{\ell_w}{2}\right)}{I_g} = \frac{125}{21.2} + \frac{(4367)(8.1)\left(\frac{16.2}{2}\right)}{464}$$

$$I_g = \frac{(1.31)(16.2^3)}{12} = 464 \text{ ft}^4 \qquad = 5.89 + 609$$

$$A_g = (1.31)(16.2) = 21.2 \text{ ft}^2 \qquad = 0.615 \text{ C}$$

PWC | SW(15) | SPOT CHECK | 2/2

$0.2 f'_c = (0.2)(5) = 1\text{ ksi} < 0.615\text{ ksi}$ ∴ NO B.E. Needed

DETERMINE TRANSVERSE & LONGITUDINAL REINF.

- $2 A_{cv}\sqrt{f'_c} = (2)(15.2(16.2 \times 12))\sqrt{5000}\,/1000$
  $= 417.8^k$

  $V_u = 82^k \ll 417.8^k$ ∴ Use Min Reinf

- Min Requirements

$$\rho_e, \rho_t = \frac{A_{s\ell}}{A_{cv}} \geq 0.0025$$

$$A_{sL} = (0.0025)(15.2)(12) = 0.456\ \text{in}^2/\text{ft}$$

- Try #5 bars in two curtains

$$A_{s\ell} = (2)(0.31) = 0.62\ \text{in}^2/s$$

$$\frac{0.456}{12} = \frac{0.62}{S_{req}}$$

$$S_{req} = \frac{(0.62)(12)}{0.456}$$

$$S_{req} = 16.3 \approx \text{use } 17''$$

Note per 21.9.2.1

$S_{max} = 18''$ ∴ 17" OK

CHECK SHEAR CAPACITY

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

$$= (2954.9)(2\sqrt{5000} + (.0024)(60000))/1000 = \boxed{843.4^k}$$

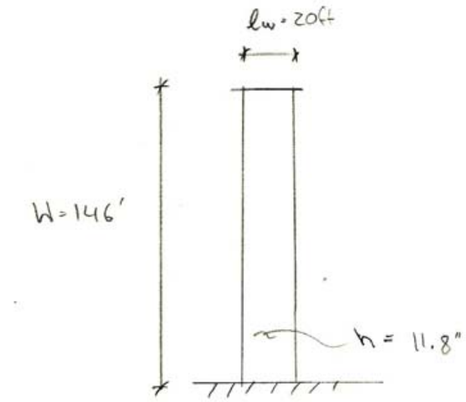
ok

$$\rho_t = \frac{(2)(0.31)}{(15.2)(17)} = .0024$$

#5 BARS @ 17" Spacing

PWC | SW (17) | SPOT CHECK | 1/2

SHEAR WALL (17)



FROM ETABS

CONTROLLING LOAD COMB: 1.2D + 1.0E + 0.2S

(1.0) E : $V_u = 84^k$

$M_u = 1173^k$

MATERIALS

$P_u = 449^k$

$f'_c = 5000$ psi

$f_y = 60\,000$ psi

CHECK NEED FOR B.E.

$$f_c = \frac{P_u}{A_g} + \frac{(M_u)(\frac{l_w}{2})}{I_g} = \frac{(1173)(\frac{20}{2})}{653} + \frac{449}{19.6}$$

$I_g = \frac{(.98)(20)^3}{12} =$ $\qquad f_c = 17.98 + 30.7$

$A_g = (.98)(20) = 19.6$ $\qquad f_c = .048$

$0.2 f'_c = (0.2)(5) = 1 ksi > 0.048$

B.E. Not needed

PWC | SW(17) | SPOT CHECK | 2/2

DETERMINE TRANSVERSE & LONGITUDINAL REINF

- $2 A_v \sqrt{f'_c} = \frac{(2)(20)(12)(11.8)}{1000}\sqrt{5000} = 400^k$

$V_u = 84^k \ll 400^k \therefore$ Use min Reinf

- Min Req Steel

$\rho_e, \rho_t = \frac{A_{se}}{A_{cv}} > 0.0025$

$A_{sL} = (0.0025)(11.8)(12) = .354 \ in^2/ft$

- Try #5 bars in two curtains

$A_{se} = (2)(0.31) = 0.62 \ in^2/s$

$\frac{0.354}{12} = \frac{0.62}{S_{req}}$

$S_{req} = \frac{(.62)(12)}{(0.354)}$

$S_{req} = 21''$

$\therefore$ Use max spacing 18"

CHECK SHEAR CAPACITY

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

$= 2832\left(\frac{2\sqrt{5000} + (.0044)(6000)}{1000}\right) = \boxed{119^k}$

$A_{cv} = (20)(12)(11.8) = 2832$ ok

$\rho_t = \frac{(2)(0.31)}{(11.8)(12)} = .0044$

#5 BARS @ 18" SPACING