Pearl Condominiums

9th & Arch Street
Philadelphia, PA

Technical Assignment #3

Structural Concepts / Structural Existing Conditions Report

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Faculty Advisor:
Dr. Linda Hanagan

Joseph G. Lichman Jr.

Structural Option

http://www.engr.psu.edu/ae/thesis/portfolios/2008/jgl138/

Executive Summary

The purpose of this report is to analyze the lateral system implemented to resist seismic and wind activity of Pearl Condominiums which is located in Philadelphia, Pennsylvania. This was accomplished through the combination of computer analysis software (ETABS) and hand calculations.

The gravity system of this building is comprised of load bearing walls and precast concrete planks. The main component in the lateral system is the use of concrete masonry units as shear walls in the stair towers and the elevator core. The ground floor contains moment frame to transfer lateral loads from the stair tower shear walls which end on the second floor. Finally, the use of metal stud walls with metal strapping is used to help resist lateral load in the east to west direction of the building.

This report discussed the influence of the lateral load path of the building, overall building drift, and story drift. The effect of overturning and it's the impact on foundations are analyzed because of the affect on the soil that supports the foundation system. There is a brief of discussion of torsion which does not control in the design, this results from the symmetric shape of the building. Finally the center of rigidity and center of mass are analyzed and their effects on the loading.

From the process of writing the report, the findings showed that the wind controlled in the north to south direction (short direction) and the seismic loading controlled in the east to west direction (long direction). Using the loads and story deflections figured through the ETABS model prepared, the shear wall were analyzed to see if they could resist the loading and the story drifts were tested against allowable drifts by code.

The concrete masonry shear walls through hand calculations showed that theses lateral elements were able to resist the loading depicted by the two types of forces. The overall drift was acceptable by code, but some of the story drifts differed from the code requirements. The story drift will need to be addressed in the proposal, resulting in the decision of choosing a system that will be able to meet the code requirements for story drift.

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Introduction

Pearl Condominiums is located on 9th and Arch Street in Philadelphia, Pennsylvania. This structure is a mixed use development building. The building includes a retail floor at the ground level containing 10 units and five floors of housing above containing a total of 90 condominium units. The maximum height of Pearl Condominiums is 72 feet 4 inches. The building's gross floor area is 111,570 square feet.

The Gravity Framing System of Pearl Condominiums is comprised of concrete masonry units and metal stud used as load bearing walls. There is also the use of steel transfer beams on the second floor to maximize the area of retail space below by eliminating the use of load bearing walls. The upper floors consist of precast concrete hollow core planks with a ¾ inch concrete topping. The roof is made up of 1-1/2 inch steel deck with rigid insulation, and single ply- membrane which is supported by steel joists. The loads from the building are distributed into a 6 inch concrete slab on grade which then disperses these forces to the grade beams and in turn to the drilled piers.

The Lateral System of Pearl Condominiums contains several types of systems that works together to resist the lateral forces. The first is the use of concrete masonry units acting as shear walls. The second is the use of moment frames located on the first floor. The final is use of the metal wall studs and metal strapping as shear walls.

In this report, the discussion will involve the load case that controlled the design and analysis of the lateral elements. During this analysis, the distribution of the forces will be determined among the types of lateral system as they act together. Also in this report, the findings will be based off of computer analysis software and hand calculations, which will be used in member checks of certain lateral elements.

Existing Structural System

Foundations:

The primary support for the foundation is in the use of drilled piers. The drilled pier option was performed, so the loads from the building would be transferred from the piers to the soil below the SEPTA commuter train tunnel. The drilled piers range in size of diameter from 3'-0" to 3'-6" and 4'-0". They also range in depth depending on the rock elevations in the area as described in the geotechnical report.

To help distribute the load to the drilled piers the use of grade beams was employed. They range in width from 12" to 40" and in depth from 18" to 30". The slab on grade is 6" reinforced with 6x6 W2.9xW2.9 WWR over 6" crushed stone over 6 mil. Vapor retarder. This can be seen in Fig 1.

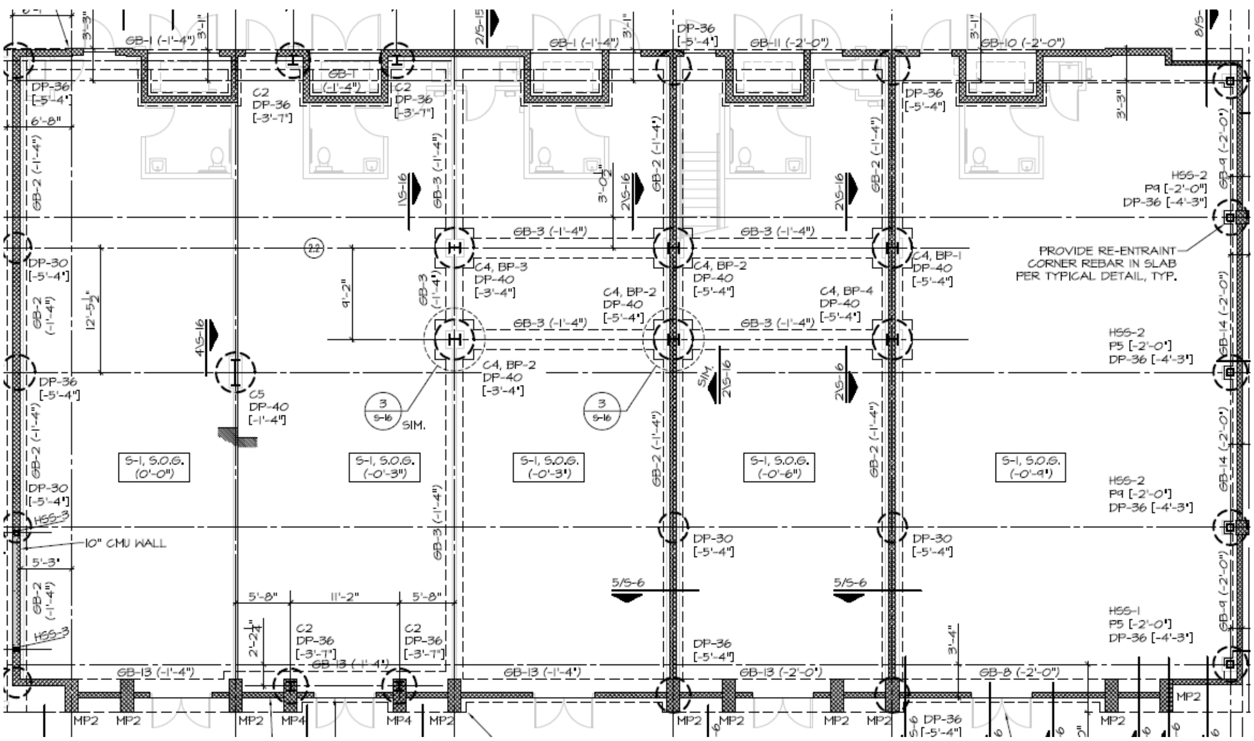


Figure 1 – South Side of Building Foundation Plan

Floor Framing Systems:

The floor system for the upper floors consists of a 10" Precast Concrete Plank with a ¾" concrete thick topping. These planks are supported by the use of 8" metal stud bearing walls and concrete masonry unit walls which are used as load bearing walls as well as shear walls. Also supporting portions of second floor, the use of steel wide flange beams and columns are used to transfer the loads from above to the foundation. This results in the maximization of retail space for the floor below. This can be seen in Fig 2. The support of the second floor employs the use of the 8" concrete masonry unit bearing walls. This can be seen in Fig 3.

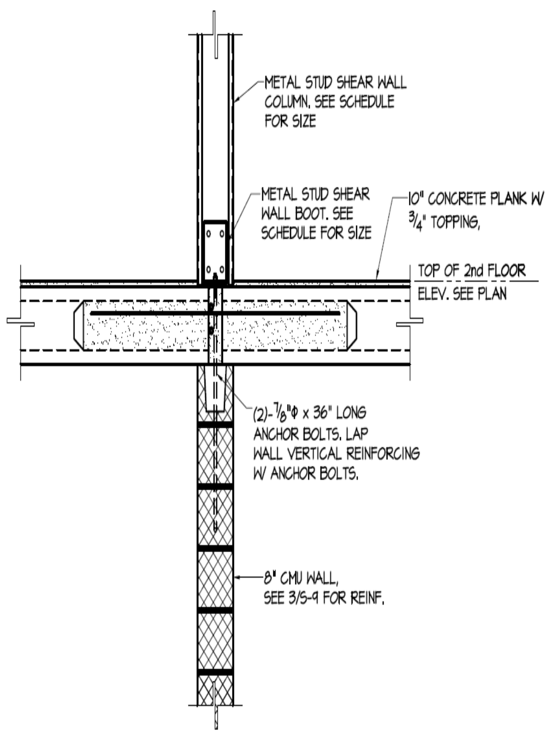


Figure 3. Second floor metal stud wall bearing on First floor CMU bearing walls

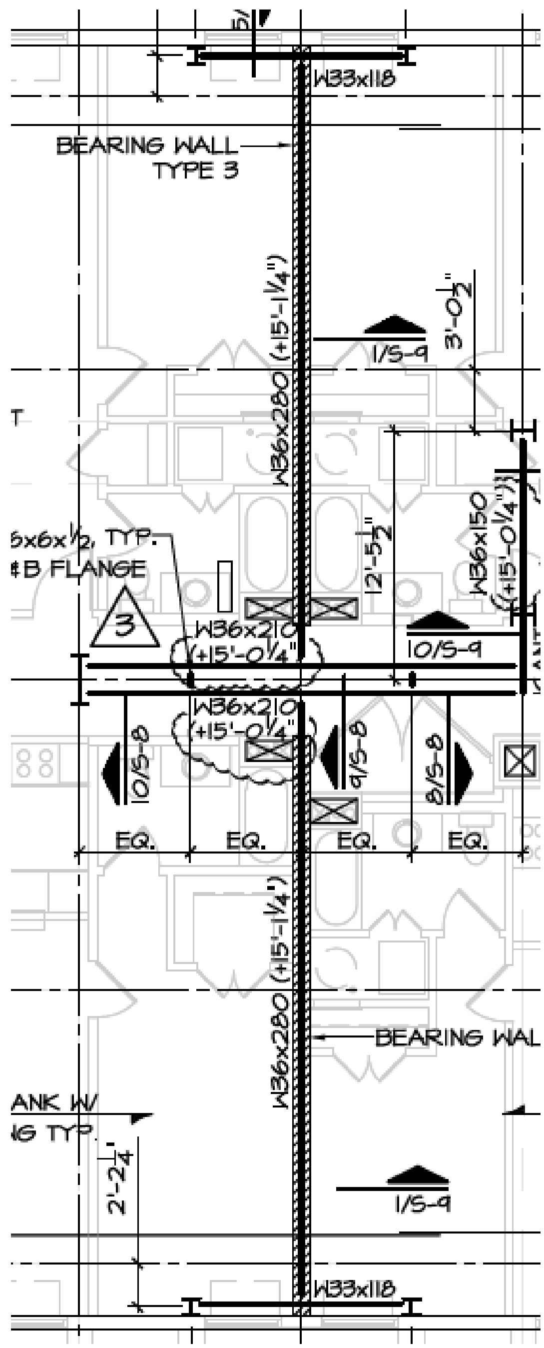


Figure 2 – Second Floor Transfer Beam Framing

Roof Framing Systems:

The main structural element in the roof system is the use of 24” deep steel joists at 48” on center. On the two ends of the building, the use of wide flange beams to transfer the load to HSS columns is implemented. The steel joists bear on the metal stud walls and the concrete masonry walls of the sixth floor. This can be seen in Fig. 4

The roof assembly is composed of:

- Single-Ply Membrane
- 5/8” Protection Board
- R-30 Rigid Insulation
- 5/8” Gypsum Wall Board
- 1-1/2” Min Steel Deck
- Steel Roof Joists
- Steel Bridging
- 5/8” Gypsum Wall Board On Suspended Ceiling Panel

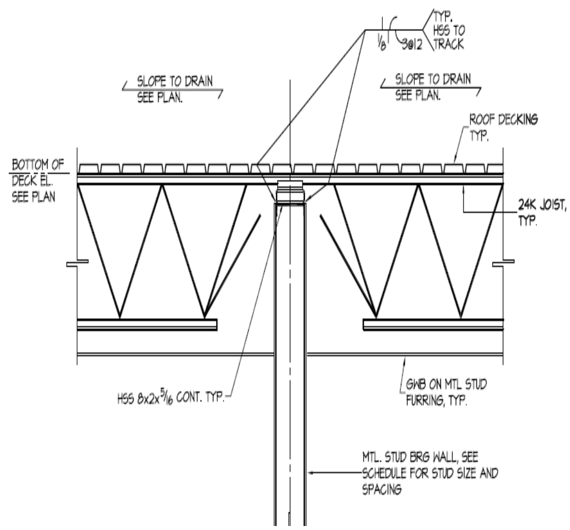


Figure 4 – Steel Joist Bearing on Metal Stud Wall

Lateral System

The Lateral system is composed to three different types of elements. For a full floor plan please see Figure A in the Appendix page A1. The first and main lateral resisting system of this building is the use of concrete masonry units (CMU) acting together as shear walls. The locations of these cmu shears walls exist at the stair towers and the elevator core. There is a difference in the type of construction of these two locations. At the two stair towers the walls are made up of 10 inch CMUs with the strength (f’m) equal to 1500 psi from the roof to the third floor. For the third to the second floor the strength is increased to 2000 psi. In the elevator core, the CMUs are 12 inches wide and have a varying strength (f’m) of 1500 psi to 3000 psi depending on the floor location of the walls.

The second element is found on the first floor, this is the result of the stair case ending on the second floor and the discontinuation of the 10 inch CMU wall. To help distribute the lateral loads from the stair tower shear walls to the foundation the use of moment frames was implemented. The moment frames main components are steel W12x120 columns and steel W36x135 beams. This can be seen in Fig 5 below.

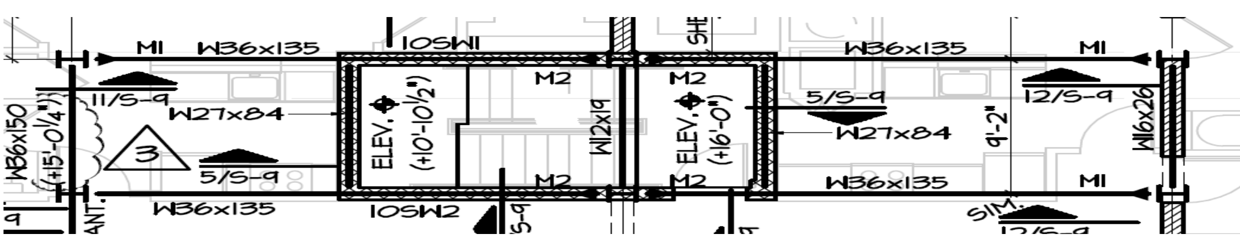


Figure 5 – South Side Stair Tower

The third and final lateral component is the combination of metal stud with metal strapping which are found on the upper floors. These shear walls resist the lateral forces by the strength of the metal strapping and connection which then transfer the forces into the metal studs and end connections of the straps. They are present to help resist lateral forces in the East to West direction of the building. These walls are comprised of 8 inch metal stud which vary on number of studs and gauge depending on location of floor level. The metal strapping and connections type vary also per floor. This can be seen in Fig. 6

METAL STUD SHEAR WALL SCHEDULE						
WALL TYPE 1 8" METAL STUD WALL (13'-8" LENGTH)						
LEVEL	COLUMN SECTION	BOOT TYPE	STRAP SIZE (EA. FACE)	STRAP CONNECTION AT EA. END	T_1 (KIPS)	ANCHOR BOLT
7th - ROOF	800C250-33	A	4" - 54 mils	4 - #12	2.8^k	(2)-7/8"Φ
6th - 7th	800C250-54	B	4" - 54 mils	8 - #12	8.2^k	(2)-7/8"Φ
5th - 6th	(2) - 800C250-54	B	6" - 54 mils	12 - #12	13.0^k	(2)-7/8"Φ
3rd - 5th	(2) - 800C250-68	C	8" - 54 mils	16 - #12	16.7^k	(2)-7/8"Φ
2nd - 3rd	(2) - 800C250-97	D	8" - 54 mils	18 - #12	19.5^k	(2)-7/8"Φ
-	-	-	-	-	-	-
WALL TYPE 2 8" METAL STUD WALL (9'-0" LENGTH)						
7th - ROOF	800C250-33	B	4" - 54 mils	4 - #12	3.2^k	(2)-7/8"Φ
6th - 7th	800C250-54	B	4" - 54 mils	8 - #12	8.6^k	(2)-7/8"Φ
5th - 6th	(2) - 800C250-54	D	6" - 54 mils	12 - #12	13.6^k	(2)-7/8"Φ
3rd - 5th	(2) - 800C250-68	D	8" - 54 mils	16 - #12	17.5^k	(2)-7/8"Φ
2nd - 3rd	(2) - 800C250-97	D	8" - 54 mils	18 - #12	20.4^k	(2)-7/8"Φ
-	-	-	-	-	-	-

Figure 6 - Metal Shear Wall Schedule

Code and Requirements

The 2006 International Building Code

Minimum Design Loads for Building and Other Structures 07-05, American Society of Civil Engineers

North American Specifications for the Design of Cold- Formed Steel Structural Members

ACI 530: Building Requirements for Masonry Structures

Steel Construction Manual, Thirteenth Edition, American Institute of Steel Construction

<u>Deflection Criteria Based on the 2006 International Building Code:</u>

Δ wind = H/400 Allowable Building Drift
Δ seismic = 0.010 h_{sx} Allowable Story Drift

Gravity Loads

The Gravity Loads that were applied during the design of the floors of Pearl Condominiums are listed below.

Floor Live Loads	
Occupancy or Use	**Uniform Live Load (psf)**
Condominium Units w\ Partitions	60
Retail Units (first floor)	100
Stairs	100
Corridor above first floor	80
Corridor at first floor	100
Roof	30

Floor Dead Loads	
Occupancy or Use	**Uniform Dead Load (psf)**
Concrete Precast Plank	66
Roof	20

Superimposed Floor Dead Loads	
Occupancy or Use	**Uniform Dead Load (psf)**
Roof	20
Condominium Units w\ Partitions	25
Corridor above first floor	25
Corridor at first floor	25
Retail Units	25

Snow Loading	
Item	**Value**
Ground Snow Load (Pg)	25 psf
Exposure Factor	B
Roof Exposure	Fully Exposed
Exposure Factor (Ce)	0.9
Thermal Factor (Ct)	1.0
Occupancy Category	II
Importance Factor (Is)	1.0
Flat-Roof Snow Load Pf = 0.7CeCtIsPg	16 psf

Lateral Loads

Wind Load:

Wind Loads were calculated using ASCE 07-05. The wind load that controlled was in the north to south direction. For the detailed calculations see the Appendix page A2 and A3. Below are the main factors used in the determination of the wind loads. Also see Fig 7 and 8 for diagram of wind lateral loads distributed by floor.

Basic Wind Speed – 90 mph
Exposure Category – B
Importance Factor – 1.0
Internal Pressure Coefficient - +/- 0.55

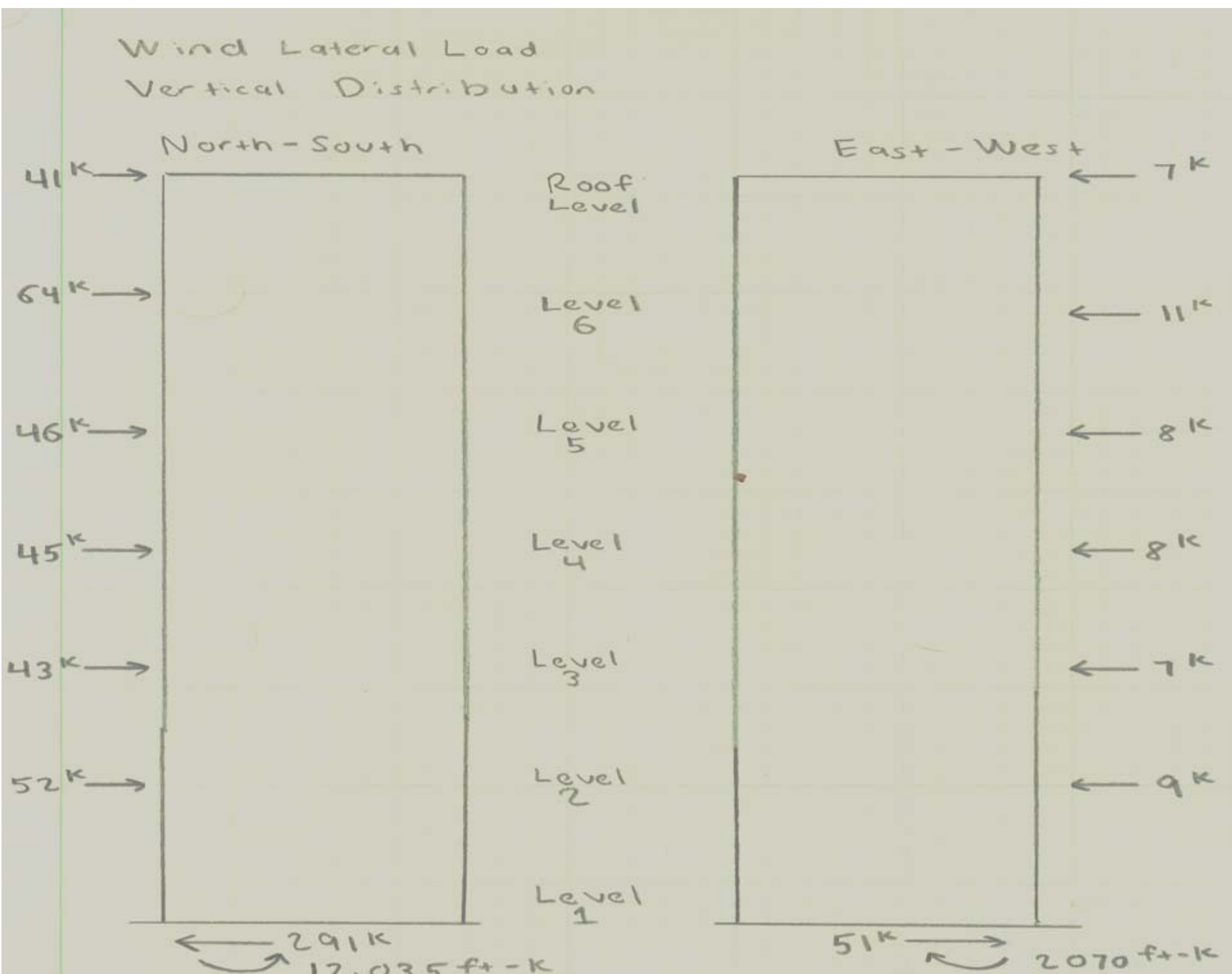


Figure 7

Figure 8

Seismic Load:

Seismic Loads were calculated using ASCE 07-05. The seismic load controlled was in the east to west direction of the building. For the detailed calculations see the Appendix page A4. Below are the main factors used in the determination of the seismic loads. Also see Fig 9 for diagram of seismic lateral loads distributed by floor.

Occupancy Category – II
Importance Factor – 1.0
Seismic Design Category – B
Response Modification Factor – 5.5
(Reinforced Masonry Shear Walls)

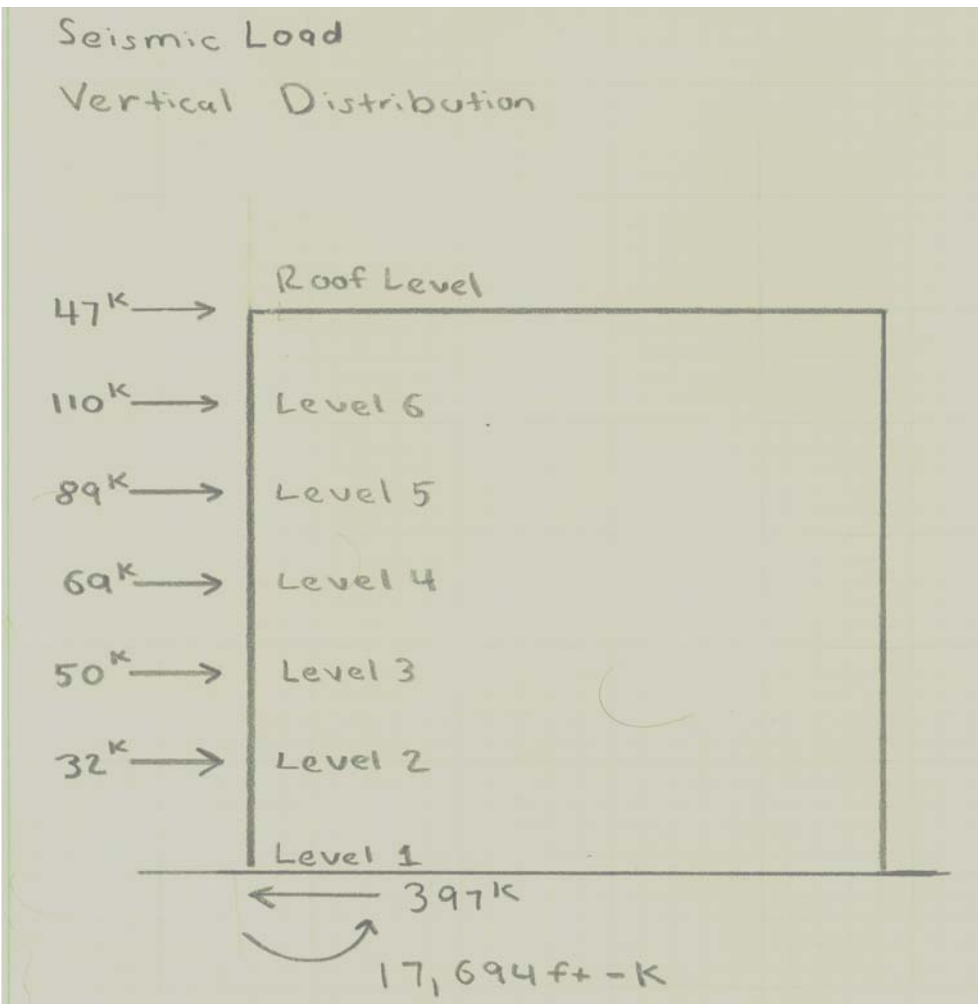


Figure 9

Lateral Load Path

The Lateral Load Path for wind forces at Pearl Condominiums begins through the exterior façade which is supported by metal studs. The forces from the exterior metal studs are transferred to the rigid diaphragm consisting of the precast concrete planks. Then the loads are distributed to the concrete masonry shear walls at the elevator core and the stair towers. At the first floor the load is transferred to the foundation by the concrete masonry shear wall of the elevator core. Similarly at the stair towers the load is then distributed to the moment frames on the first floor, result from the discontinuation of the stairs.

The seismic forces are distributed in the same load path except for the use of the exterior stud wall. The process begins by the transfer of forces by the rigid diaphragm and then follows the same path as in the wind case.

An isometric is shown below in Fig 10. The metal stud walls are left out of the modeling to simplify the computer analysis. The main focus will be on the concrete masonry shear wall and the moment frames. The blue and yellow color walls are the concrete masonry shear walls.

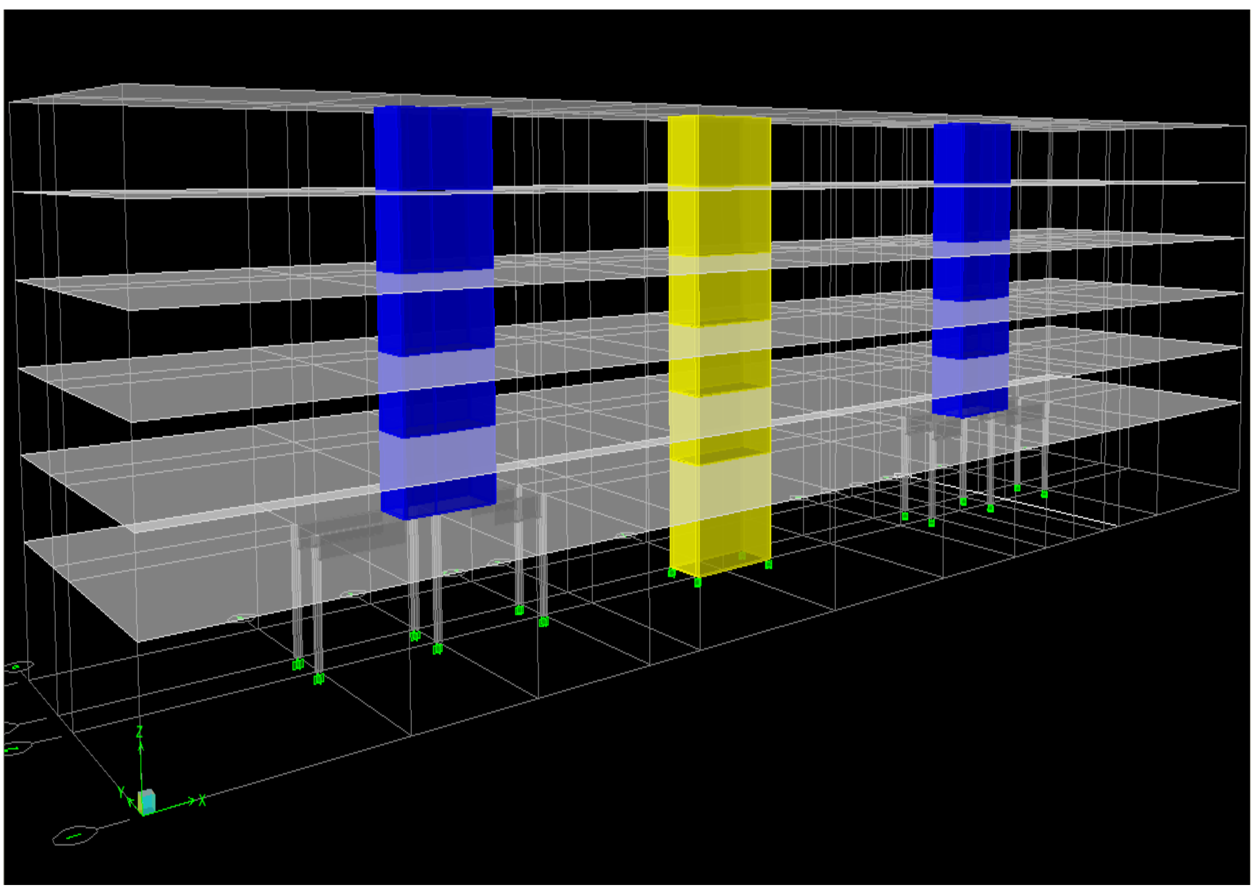


Figure 10 – Isometric of Lateral Resisting System Present in Pearl Condominiums

Analysis of Lateral System

ETABS Analysis:

A simplified model was constructed in ETABS which focused on the main lateral resisting elements: the concrete masonry units shear walls and the moment frames. As seen in Fig 11 is the labeling of the elements of the concrete masonry shear walls.

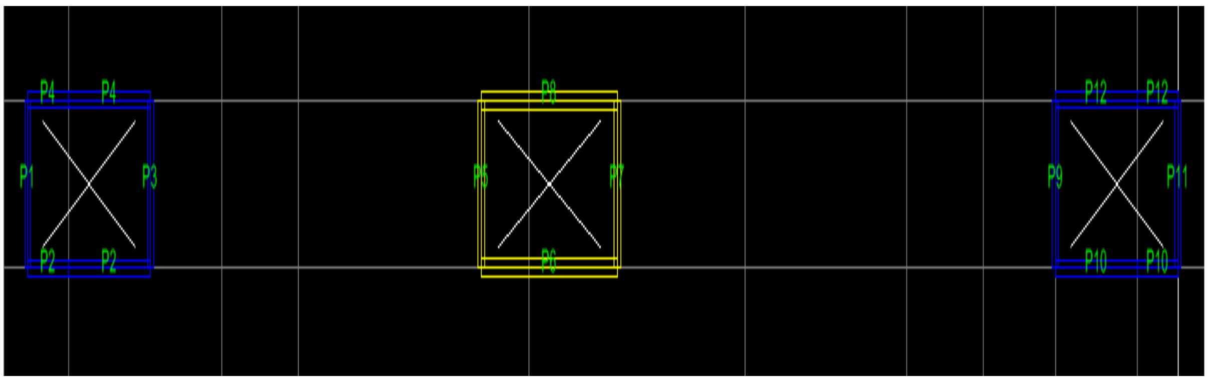


Figure 11 – Pier Labels for the Concrete Masonry Shear Walls

Distribution of Forces:

The distribution of forces will be shown for story 6 with respect to the seismic loads in Figure 12. Also for story 4 the winds load distribution of forces will be shown in Figure 13. The Pier Labels correspond to labels in Figure 11 above.

Seismic

Story	Pier	Load	Loc	Direct Shear (k)	Moment (ft-kip)
STORY6	P1	QUAKE	Bottom	4.97	66.351
STORY6	P2	QUAKE	Bottom	0.26	3.158
STORY6	P3	QUAKE	Bottom	5.49	73.32
STORY6	P4	QUAKE	Bottom	-0.26	-3.158
STORY6	P5	QUAKE	Bottom	8.31	110.952
STORY6	P6	QUAKE	Bottom	0.36	-2.069
STORY6	P7	QUAKE	Bottom	9.01	120.279
STORY6	P8	QUAKE	Bottom	-0.36	2.069
STORY6	P9	QUAKE	Bottom	9.34	124.712
STORY6	P10	QUAKE	Bottom	0.26	3.158
STORY6	P11	QUAKE	Bottom	9.87	131.685
STORY6	P12	QUAKE	Bottom	-0.26	-3.158

Figure 12

Wind

Story	Pier	Load	Loc	Direct Shear (k)	Moment (ft-kip)
STORY4	P1	WIND	Bottom	-4.09	-77.751
STORY4	P2	WIND	Bottom	22.99	436.036
STORY4	P3	WIND	Bottom	-3.19	-60.564
STORY4	P4	WIND	Bottom	22.09	419.109
STORY4	P5	WIND	Bottom	-0.91	-17.143
STORY4	P6	WIND	Bottom	31.04	490.656
STORY4	P7	WIND	Bottom	0.31	5.816
STORY4	P8	WIND	Bottom	29.78	472.569
STORY4	P9	WIND	Bottom	3.49	66.177
STORY4	P10	WIND	Bottom	22.99	436.036
STORY4	P11	WIND	Bottom	4.38	83.426
STORY4	P12	WIND	Bottom	22.09	419.109

Figure 13

As depicted in the information provided in the charts above, the seismic load is resisted by the concrete masonry shear walls in the east to west direction (ex. P1, P3). Similarly the wind load is resisted by the concrete masonry shear walls in the north to south direction (ex P2, P4). This is consistent with the direction that the two types of lateral forces are applied.

Center of Mass & Center of Rigidity

Figure 14 provides the ETABS calculated centers of rigidity of the structure. The center of rigidity is shown to be nearly in the center of the building where the elevator core is located. This can be explained because the building is almost symmetrical and the elevator core is near the center. Hand calculated Center of Mass: X– 141.8 and Y – 33.25.

CENTERS OF CUMULATIVE MASS & CENTERS OF RIGIDITY

STORY LEVEL	DIAPHRAGM NAME	/-----CENTER OF MASS MASS	ORDINATE-X	ORDINATE-Y------//	--CENTER OF RIGIDITY ORDINATE-X	--/ ORDINATE-Y
STORY6	RIGID	3.028E-01	137.432	39.620	135.710	39.620
STORY5	RIGID	9.083E-01	137.432	39.620	135.585	39.620
STORY4	RIGID	1.514E+00	137.432	39.620	135.447	39.620
STORY3	RIGID	2.119E+00	137.432	39.620	135.273	39.620
STORY2	RIGID	2.854E+00	141.359	39.620	135.328	39.620
STORY1	RIGID	4.186E+00	143.085	39.620	134.250	39.620

Figure 14 ETABS Calculated Center of Mass and Rigidity

Story Drift:

The limitations for the allowable story drift were compared to the values determined by ETABS. The Seismic story drifts were compared to the allowable story drift of $0.01h_{sx}$ and can been seen in Figure 15. The Wind drift of the overall building was compared to h/400 and can been seen in Figure 16.

Seismic Drift

Story	Story Height (ft)	Story Drift X (in)		Allowable Story Drift Δ seismic = 0.010 h_{sx}		Total Drift X (in)		Allowable Total Drift Δ seismic = 0.010 h_{sx}	
6	72.3	0.001344	<	0.1663	Acceptable	0.041136	<	0.723	Acceptable
5	55.67	0.004692	<	0.0992	Acceptable	0.039792	<	0.5567	Acceptable
4	45.75	0.007272	<	0.0992	Acceptable	0.0351	<	0.4575	Acceptable
3	35.83	0.009576	<	0.0991	Acceptable	0.027828	<	0.3583	Acceptable
2	25.92	0.011832	<	0.0992	Acceptable	0.018252	<	0.2592	Acceptable
1	16	0.00642	<	0.16	Acceptable	0.00642	<	0.16	Acceptable

Seismic Drift

Story	Story Height (ft)	Story Drift Y (in)		Allowable Story Drift Δ seismic = 0.010 h_{sx}		Total Drift Y (in)		Allowable Total Drift Δ seismic = 0.010 h_{sx}	
6	72.3	0.013476	<	0.1663	Acceptable	0.63324	<	0.723	Acceptable
5	55.67	0.0468	<	0.0992	Acceptable	0.619764	>	0.5567	Not Acceptable
4	45.75	0.072564	<	0.0992	Acceptable	0.572964	>	0.4575	Not Acceptable
3	35.83	0.094308	>	0.0991	Not Acceptable	0.5004	>	0.3583	Not Acceptable
2	25.92	0.114684	>	0.0992	Not Acceptable	0.406092	>	0.2592	Not Acceptable
1	16	0.291408	>	0.16	Not Acceptable	0.291408	>	0.16	Not Acceptable

Figure 15

Wind Drift

Story	Story Height (ft)	Story Drift X (in)		Allowable Story Drift Δ wind = H/400		Total Drift X (in)		Allowable Total Drift Δ wind = H/400	
6	72.3	0.004896	<	0.041575	Acceptable	0.128832	<	0.18075	Acceptable
5	55.67	0.011784	<	0.0248	Acceptable	0.123936	<	0.13918	Acceptable
4	45.75	0.016392	<	0.0248	Acceptable	0.112152	<	0.11438	Acceptable
3	35.83	0.021072	<	0.024775	Acceptable	0.09576	>	0.08958	Not Acceptable
2	25.92	0.028716	>	0.0248	Not Acceptable	0.074688	>	0.0648	Not Acceptable
1	16	0.045972	>	0.04	Not Acceptable	0.045972	>	0.04	Not Acceptable

Wind Drift

Story	Story Height (ft)	Story Drift Y (in)		Allowable Story Drift Δ wind = H/400		Total Drift Y (in)		Allowable Total Drift Δ wind = H/400	
6	72.3	0.002496	<	0.041575	Acceptable	0.087456	<	0.18075	Acceptable
5	55.67	0.006132	<	0.0248	Acceptable	0.08496	<	0.13918	Acceptable
4	45.75	0.008664	<	0.0248	Acceptable	0.078828	<	0.11438	Acceptable
3	35.83	0.011484	<	0.024775	Acceptable	0.070164	<	0.08958	Acceptable
2	25.92	0.0153	<	0.0248	Acceptable	0.05868	<	0.0648	Acceptable
1	16	0.04338	>	0.04	Not Acceptable	0.04338	>	0.04	Not Acceptable

Figure 16

As presented from the numbers derived from the chart, the overall drifts of the building are acceptable in both the seismic and wind loading in both directions. There is however some of the story drifts that do not meet code. This can be explained through the omission of the metal stud shear walls in the ETABS modeling to simplify the process. Therefore the resistance that is demonstrated by the metal stud shear walls will help to reduce the amount of drift per floor. This will result in distributing some of the lateral forces into the stud shear walls instead of the concrete masonry unit shear walls.

Overturning:

The Overturning moment produced by seismic loading is 17,694 ft-kips and the overturning moment produced by wind loading is 12,035 ft-kips in the north south direction, which controls. As stated previously, the foundation is comprised of drilled piers which continue past the depth of the commuter rail tunnel that runs underneath the site. By the size of the overturning moments, this will be an issue that will have to be resisted by the building weight and the foundation system.

Torsion:

Resulting from the symmetric shape of the building and the center of mass being similar in number to the center of geometry, the effect of torsion will not control in the design process. When calculating torsion; story shear (HS), eccentricity (e), relative stiffness (KSN), and distance to the shear wall (CN) all come into effect.

$$\text{Torsion} = (HS * e * KSN * CN) / \Sigma(KSN * CN2)$$

During the redesign of the building the effect of torsion will have to be calculated since the center of mass and center of geometry may vary enough to have an effect on the torsion forces.

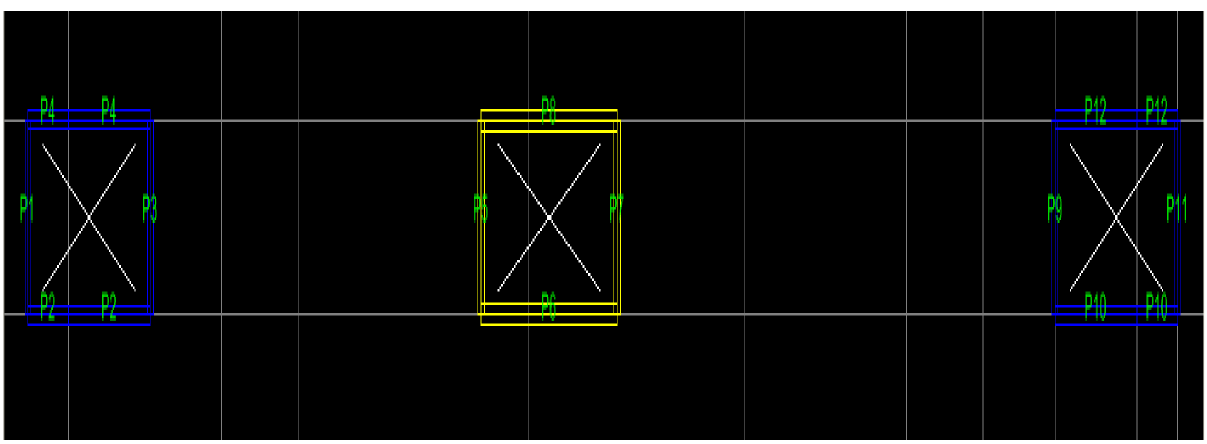
Verification of Lateral Elements

<u>Concrete Masonry Shear Wall (P1 Story 6 Seismic)</u>:
(East to West) Actual Shear Force – 4.97 k
(East to West) Actual Moment - 66.351 ft-k

Wall Thickness – 9.625 in
Wall Length – 210 in
Wall Height – 174 in
Section Modulus – (9.625*210^2)/6 = 70744 in^3
d - 210-8=202 in
f_v = (4.97*1000)/(9.625*210) = 2.46 psi
f_b = (66.351*120000)/70744 = 11.25 psi
M/V_d = (66.351*12)/(4.97*202) = 0.80
F_v max w/ reinf. = 1.5*(1500)^1/2 = 58 psi
f'_m = 1500 psi
F_b = 0.33*1500 = 495 psi
A_s (flexural in^2) = (66.351*12)/(20*0.9*202) = 0.22 in^2
Shear Spacing = 32 in
A_s (shear in^2) = (32*4.97)/(24*202) = 0.033 in^2

<u>Concrete Masonry Shear Wall (P6 Story 4 Wind)</u>:
(North to South) Actual Shear Force – 31.4 k
(North to South) Actual Moment – 490.656 ft-k

Wall Thickness – 11.625 in
Wall Length – 116.75 in
Wall Height – 123 in
Section Modulus – (11.625*116.75^2)/6 = 93312 in^3
d – 116.75-8=108.75 in
f_v = (31.4*1000)/(11.625*116.75) = 23.1 psi
f_b = (490.656*120000)/93312 = 63.1 psi
M/V_d = (490.656*12)/(31.4*116.75) = 1.61
F_v max w/ reinf. = 1.5*(2000)^1/2 = 67 psi
f'_m = 2000 psi
F_b = 0.33*2000 = 660 psi
A_s (flexural in^2) = (490.656*12)/(20*0.9*116.75) = 2.80 in^2
Shear Spacing = 32 in
A_s (shear in^2) = (32*31.4)/(24*116.75) = 0.36 in^2



From the results of the lateral member checks from above, in both case F_v was greater than f_v and F_b was greater than f_b. From this analysis, the conclusion about the stair tower wall on the fifth to sixth story and the elevator core wall on the third to fourth story will be able to handle the calculated lateral loads and moments.

Conclusion

Conclusion:

From the combination of hand calculations and computer analysis, Pearl Condominiums was analyzed for the lateral forces of wind and seismic activity. The main lateral resisting system was implemented through the use of concrete masonry unit as shear walls which also then included the addition of the moment frames on the ground floor to transfer the forces presented by the stair towers which ended on the second floor. The metal stud walls with metal strapping were omitted from this analysis, but would help with the story drift with respect to the seismic force in the east to west direction of the building.

The analysis checked the load distribution of the forces between the shear walls of the stair towers and the elevator core. The elevator received a slightly larger amount of forces than the stair tower walls because of the larger size of the concrete masonry units, which increase the stiffness of the wall. With the use of ETABS, the lateral forces were determined for the walls at each story. The shear walls were then check through the use of hand calculations to determine if these loads were acceptable to be resisted by these walls.

Also checked were the overall building drift and the story drift for seismic and wind loads. The overall building drift was acceptable for both seismic and wind. Though some of the story drift were not acceptable, this calculation could have been more precise if the model in ETABS included the metal stud shear walls. The center of rigidity was also checked and compared to the center of mass, which after calculations were similar due to the symmetric shape of the building.

All the calculations were done in accordance with the corresponding codes. The Appendix contains an enlarged second floor plan that depicts the locations of the lateral elements, and the calculations for the seismic and wind loads.

Appendix

Second Floor Plan:

The green lines represent the concrete masonry unit shear walls and the red lines represent the moment frames. The blue line represents the metal stud shear walls.

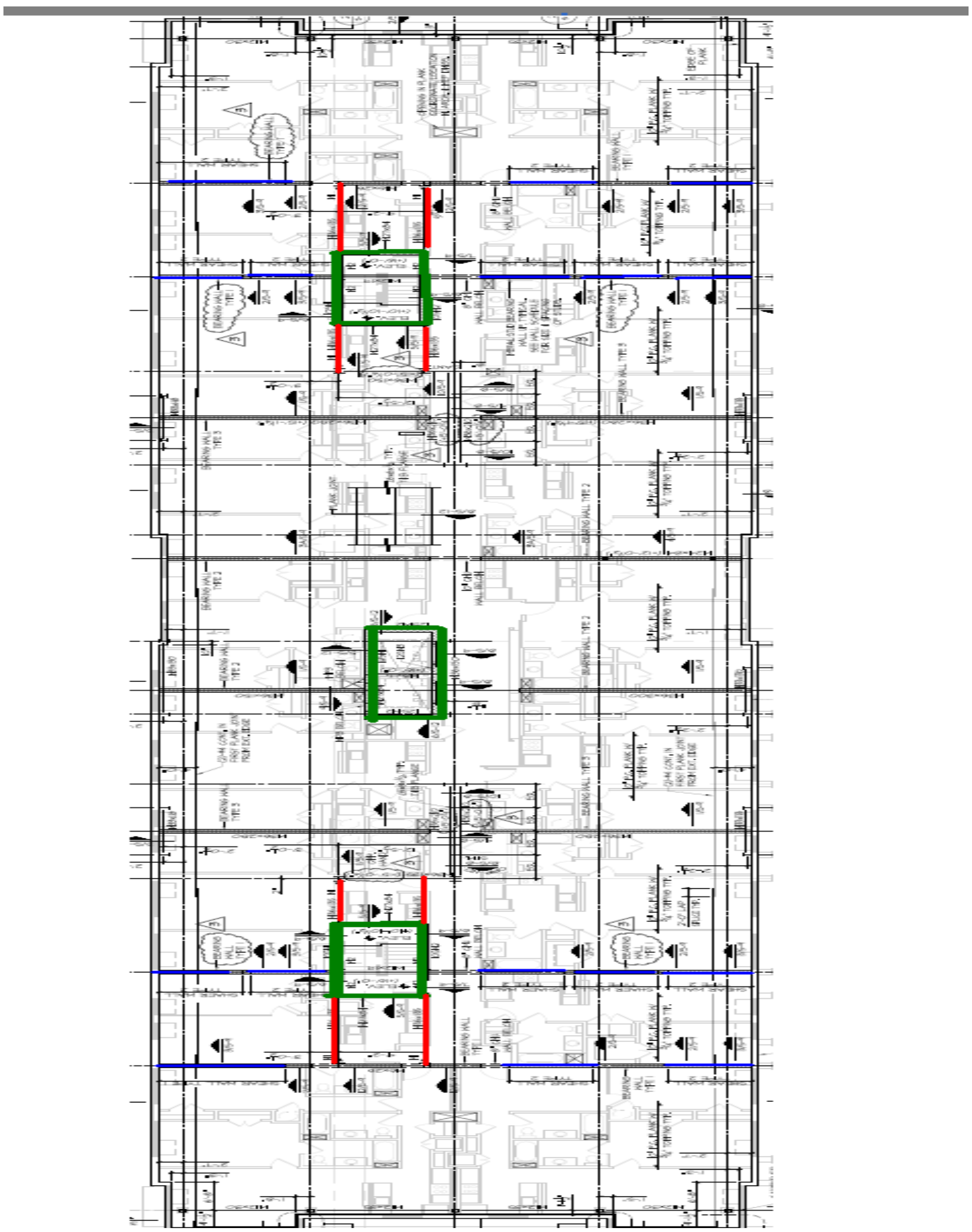


Figure A

Wind Loads:

North to South Direction

Windward Calculations							
Level	z	Kz	qz	Cp	Ext. Pressure	GCpi	P_{total} (+GC_{pi}) (psf)
1	0	0.57	10.05	0.80	6.83	+/- 0.55	-1.89
2	16	0.62	10.93	0.80	7.43	+/- 0.55	-1.29
3	25.92	0.70	12.34	0.80	8.39	+/- 0.55	-0.33
4	35.83	0.76	13.40	0.80	9.11	+/- 0.55	0.38
5	45.75	0.81	14.28	0.80	9.71	+/- 0.55	0.98
6	55.67	0.85	14.98	0.80	10.19	+/- 0.55	1.46
Roof	72.3	0.90	15.86	0.80	10.79	+/- 0.55	2.06

Leeward Calculations						
Level	z	Kz	q_h (psf)	C_p	External Pressure	P_{total} (+GC_{pi}) (psf)
1	0	0.57	15.86	-0.20	-2.70	-11.42
2	16	0.62	15.86	-0.20	-2.70	-11.42
3	25.92	0.70	15.86	-0.20	-2.70	-11.42
4	35.83	0.76	15.86	-0.20	-2.70	-11.42
5	45.75	0.81	15.86	-0.20	-2.70	-11.42
6	55.67	0.85	15.86	-0.20	-2.70	-11.42
Roof	72.3	0.90	15.86	-0.20	-2.70	-11.42

Negative Internal Pressure		
q_h (psf)	GC_{pi}	P_{neg} (psf)
15.86	- 0.55	-8.7
Positive Internal Pressure		
q_z (psf)	GC_{pi}	P_{pos} (psf)
15.86	0.55	8.7

Total		
Level	P_{total} (+GC_{pi}) (psf)	Force at Floor (kips)
1	9.53	0
2	10.13	52
3	11.09	43
4	11.81	45
5	12.40	46
6	12.88	64
Roof	13.48	41

Wind Loads:

East to West Direction

Windward Calculations							
Level	z	Kz	qz	Cp	Ext. Pressure	GCpi	P_{total} (+GC_{pi}) (psf)
1	0	0.57	10.05	0.80	6.83	+/- 0.55	-1.89
2	16	0.62	10.93	0.80	7.43	+/- 0.18	-1.29
3	25.92	0.70	12.34	0.80	8.39	+/- 0.18	-0.33
4	35.83	0.76	13.40	0.80	9.11	+/- 0.18	0.38
5	45.75	0.81	14.28	0.80	9.71	+/- 0.18	0.98
6	55.67	0.85	14.98	0.80	10.19	+/- 0.18	1.46
Roof	72.3	0.90	15.86	0.80	10.79	+/- 0.18	2.06

Leeward Calculations						
Level	z	Kz	q_h (psf)	C_p	External Pressure	P_{total} (+GC_{pi}) (psf)
1	0	0.57	15.86	-0.50	-6.74	-15.47
2	16	0.62	15.86	-0.50	-6.74	-15.47
3	25.92	0.70	15.86	-0.50	-6.74	-15.47
4	35.83	0.76	15.86	-0.50	-6.74	-15.47
5	45.75	0.81	15.86	-0.50	-6.74	-15.47
6	55.67	0.85	15.86	-0.50	-6.74	-15.47
Roof	72.3	0.90	15.86	-0.50	-6.74	-15.47

Negative Internal Pressure		
q_h (psf)	GC_{pi}	P_{neg} (psf)
15.86	-0.55	-8.7
Positive Internal Pressure		
q_z (psf)	GC_{pi}	P_{pos} (psf)
15.86	0.55	8.7

Total		
Level	P_{total} (+GC_{pi}) (psf)	Force at Floor (kips)
1	13.57	0
2	14.17	9
3	15.13	7
4	15.85	8
5	16.45	8
6	16.93	11
Roof	17.53	7

Seismic Loads:

Occupancy Category – II
Importance Factor – 1.0
Seismic Design Category – B
Response Modification Factor – 5.5
(Reinforced Masonry Shear Walls)
Site Class - D
Ss = 0.270g
S1 = 0.060g
F_a = 1.585
F_v = 2.4
S_{DS} = 0.287
S_{D1} = 0.096

Seismic Base Shear:
V = Cs*W

W = 11796 k
Cs = 0.0352
R = 5 ½ (Reinforced Masonry Shear Wall)
V = 415.2k

Vertical Distribution of Forces:
Fundamental Period:
Ta = 0.496 sec
K = 1.0

Level	wx	hx	wx*hx^k	Cvx	Fx	Mx
2	2171	16.000	34736	0.0817	32.3	516.8
3	2080	25.917	53907.36	0.127	50.1	1298.4
4	2064	35.833	73959.312	0.174	68.8	2465.3
5	2064	45.750	94428	0.222	87.8	4016.9
6	2115	55.667	117735.705	0.277	109.5	6095.5
Roof	736.26	68.500	50434	0.119	47	3219.5
				Overturning Moment =		17694