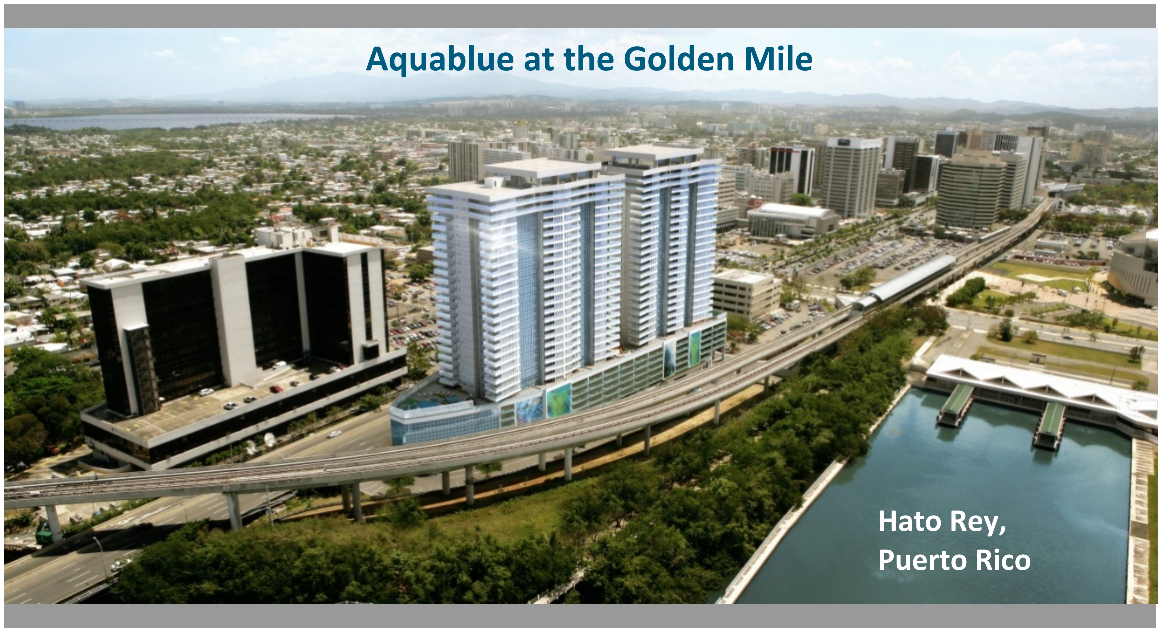
Structural Technical Report 3



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Structural Option

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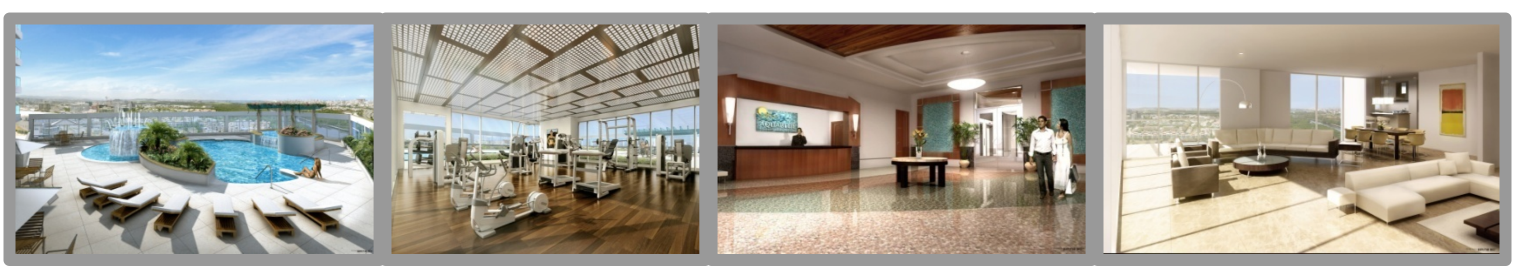
21 November 2008



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Executive Summary

Aquablue at the Golden Mile is a 31-story apartment building in Hato Rey, Puerto Rico. There is a parking garage from the ground up to level 7, and above this level, the luxury apartments are divided into two towers. The primary building material for the structure is concrete, and the existing gravity system consists of two-way, flat plate, post-tensioned slabs of varying thicknesses. The lateral system is a series of shear wall located in two groups near the core of the building.

The purpose of this report was to investigate the lateral force resisting system, specifically the following areas:

- load distribution to shear walls
- torsional effects due to eccentricity
- strength check of a typical member
- drift calculations and limitations

The wind and seismic loads were applied to one of the two towers, which could be analyzed separately due to the 5" seismic joint dividing the building in the lower floors. For the specific details of the analysis, a typical apartment floor (level 18) was the subject of investigation. The process of analyzing all aspects of the lateral system provided a deeper understanding of the existing design and the specific function of all of the structural components.

Through the analysis of the existing lateral system, some areas of the design were confirmed as appropriate while others were slightly inaccurate. For example, the analysis of the existing shear wall (with reinforcement) for axial load and overturning moment showed the load conditions a little outside of the interaction diagram. Also, the drifts calculated by the model for the service loads were higher than the recommended limitations for some floors. However, these errors can be accounted for based on minor issues with the computer modeling. Further use of computer methods to analyze Aquablue will continue with the development of the thesis proposal next semester.

General Building Information

Aquablue at the Golden Mile is a high-rise apartment building in Hato Rey, Puerto Rico. It is located in an urban area, about two miles away from the San Juan Bay (fig. 1). The building size is about 900,000 total square feet, and there are 31 stories above grade. (Up to level 7, the typical floor area is about 51,900 ft^2. For the apartment towers, which are above level 7, the typical floor areas are 11600 and 14500 ft^2.) The ground level will be developed as a commercial area, and the rest of the floors up to level 7 will be used for both parking and office space. Level 7 is an indoor/outdoor public area for the apartment residents, and the floors above are private apartments. There is a sky lobby above the penthouse apartments.

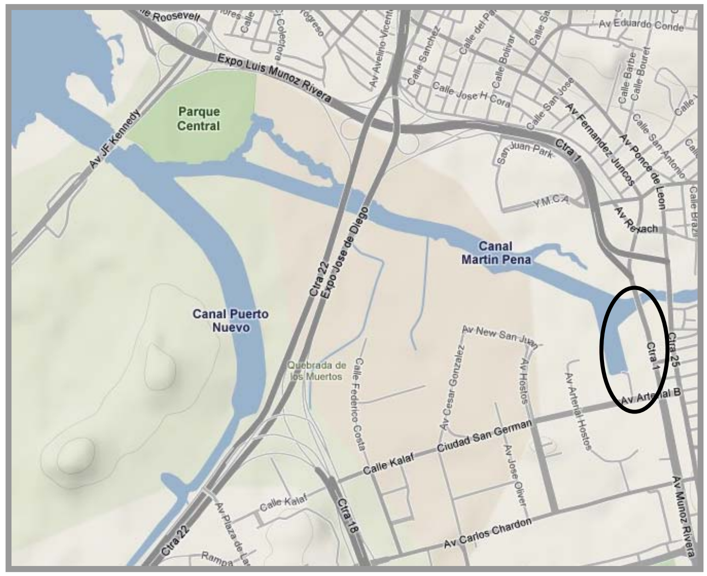


Figure 1 – Building Site (maps.google.com – Hato Rey Central, PR)

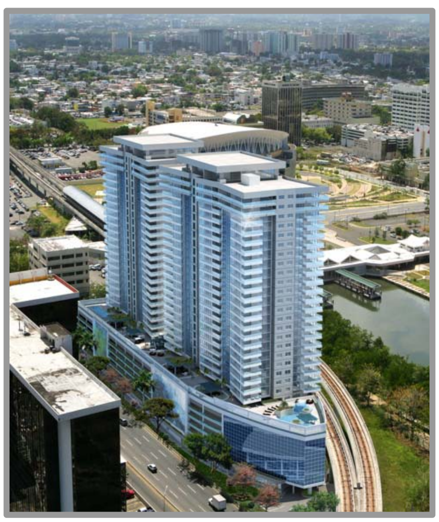


Figure 2 – Rendering of Aquablue

The parking structure (levels 2-6) is open, with concrete parapets along the exterior. As an architectural feature, there are two sections of an 8" masonry wall that extend from the ground up to level 7. The office areas of these floors are enclosed with a glass curtain wall system, as can be seen toward the bottom of figure 2. Above level 7, the façade materials are glass and concrete precast panels.

The primary building material is reinforced concrete, and the structure consists of a building frame system with shear walls. Each floor has a post-tensioned slab supported by concrete columns.

Description of Existing Structure

The **foundation** consists of drilled piles that are aligned with the columns. They are the primary foundation system, although there are some grade beams as well. (The grade beams are only used occasionally; they do not span all of the piles.) At the foundation level, there is a 10" reinforced concrete slab.

Each floor consists of a two-way, post-tensioned **structural slab** supported by reinforced columns, which span between 25'-0" and 34'-0". It is a flat plate system, so beams are not a part of the general floor framing. The slabs are 9" thick for the first six stories. At level 7, parts of the slab are 12" thick because the loads are heavier on this partially outdoor level (due to the pool and landscaping). For the apartment levels, the post-tensioned slabs are 8" thick.

The **lateral force resisting system** is a series of shear walls near the core of the building. They are 18" thick, and they require integrated boundary elements. The system of shear walls is grouped into two sections, and each one extends into one of the apartment towers.

There is one **expansion joint**, which breaks the building into two similar sections. It is a 5" seismic joint, and it runs parallel to the short dimension of the building. Because the joint falls between the two towers, it only extends from the ground to level 7. For the purpose of the structural analysis, this allows for the separation of Aquablue into two 'buildings.'

The **material strengths** of the concrete for the various structural elements are listed in table 1. The concrete strength of the slabs and columns changes around level 12. The highlighted material strengths are relevant to the analysis of the shear walls.

Concrete Material Strengths		
Structural Component		Strength, f'_c (ksi)
pile cap		4
retaining wall / basement wall		4
grade beam		4
slab on grade		5
formed slab	foundation - level 12	6
	above level 12	5
beams		5
parapet / vehicle barrier wall		5
columns / shear walls	foundation - level 13	8
	above level 13	6

Table 1 – Concrete Strengths for Various Structural Elements

Typical Floor Framing Plans

There are two typical floor plans in this building: one for the parking garage levels and one for the apartment levels. As seen below on one of the parking level plans (fig. 3), there are not a whole lot of elements in the gravity-based structural system. The columns are supporting a two-way, flat plate, post-tensioned slab. Also shown in the figure below is the lateral force resisting system of reinforced concrete shear walls concentrated toward the center of the floor plan. The most extensive shear wall system is at the base of the building, and the number and length of the walls decreases as the height above grade increases.

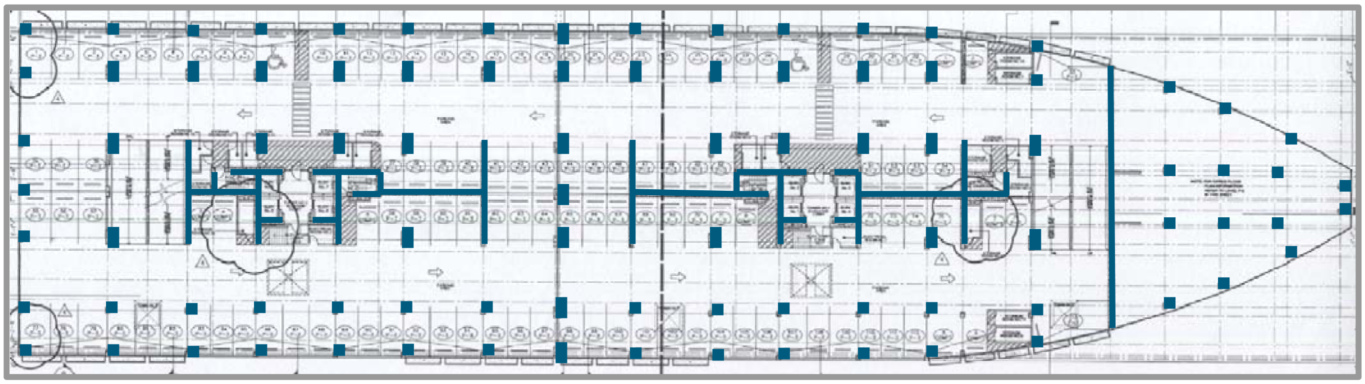


Figure 3 – Column and Shear Wall Layout for Typical Parking Garage Level

The plan below (fig. 4) is a typical apartment level floor plan. Both the columns and shear walls are shown, and the extension /simplification of the shear wall system can be seen by comparing this figure with the one above.

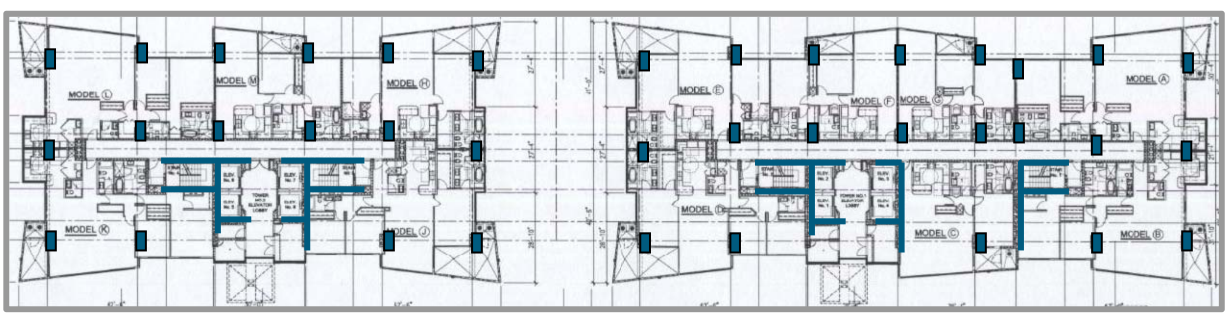


Figure 4 – Column and Shear Wall Layout for Typical Apartment Level

Description of Lateral System

The existing lateral system is composed of reinforced concrete shear walls that are concentrated toward the center of the building. As can be seen in figure 5 to the right, the walls in both the north-south and east-west directions are integrated into one multi-branch system. This detail is just one example of the general type of shear wall design. In the case of figure 5, the wall lengths and reinforcing layout represent one shear wall system between levels 7 and 9.

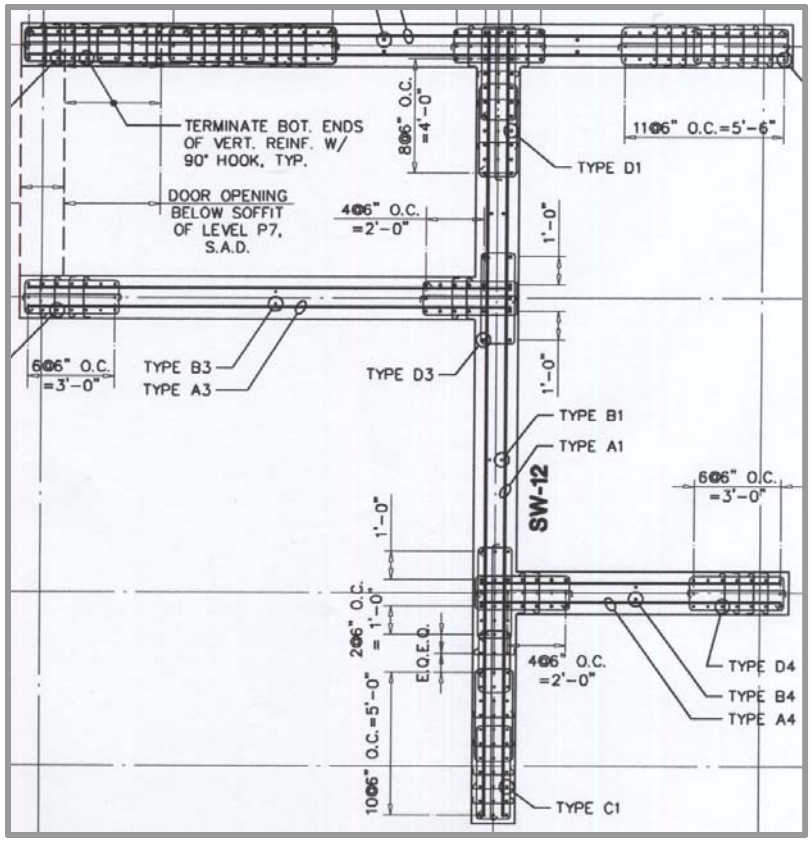
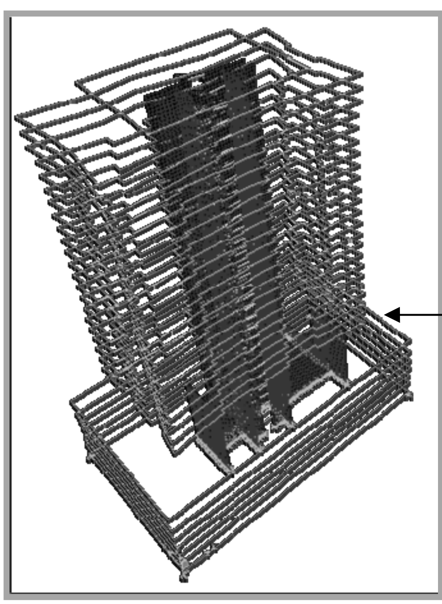


Figure 5 – Example of Shear Wall System (Levels 7-9)

The concrete strength of the shear walls changes once over the height of the building. Below level 13, $f'_c = 8$ ksi, and above level 13, $f'_c = 6$ ksi. Similarly, the reinforcement becomes less dense over the height of the building.

Also, the boundary elements of the shear walls are relatively complex due to their intersection at the wall joints. Therefore, due to the multi-branch shear wall system and the difficulties it presents, most of the analysis in this report is based on computer modeling to improve accuracy.



Seismic Joint

Figure 6 – 3-Dimensional Model of Shear Walls

In this report, the smaller of the two towers was analyzed as an independent 'building.' The image to the left (fig. 6) gives the general idea of the location and density of the shear wall system in this tower. In order to visualize its relation to the rest of the building, the seismic joint is labeled in the figure.

Codes and References

- General References:
  - IBC 2006 (International Building Code)
  - ACI 318-08 (American Concrete Institute)
  - AISC Steel Construction Manual, 13th edition (American Institute of Steel Construction)

- Code used for wind and seismic analyses:
  - ASCE 7-05 (American Society of Civil Engineers, "Minimum Design Loads for Buildings and Other Structures")
    - Chapters 6 and C6 – Wind Loads (Method 2)
    - Chapters 11 and 12 – Seismic Loads (Equivalent Lateral Force Procedure)

- Major national model codes used by De-Simone Consulting Engineers:
  - Puerto Rico Building Code 1999
  - UBC 1997 (Uniform Building Code)
  - ACI 318-99 (American Concrete Institute "Building Code Requirements for Structural Concrete")
  - ACI 530-99 (American Concrete Institute "Building Code Requirements for Masonry Structures")
  - SJI 1994 (Steel Joist Institute "Standard Specifications, Load Tables and Weight Tables for Steel Joists and Joist Girders")

- Utilized Computer Programs
  - ETABS Nonlinear, version 9.2.0 (3-dimensional building model)
  - PCA Column (wall and reinforcement layout for one group of shear walls)

Summary of Lateral Loads

The preliminary wind and seismic analyses were completed for Technical Report 1, but some modifications were made for this report. For example, the wind pressures were applied only to the tower under consideration. Also, the seismic forces were re-calculated based on the weight of one tower and the revision of the value of C_s. (See the appendix for the updated seismic information.) The sketch below (fig. 7) shows the overall building dimensions as well as the directions of the applied lateral loads. The colored rectangle shows the location of the tower studied for this report.

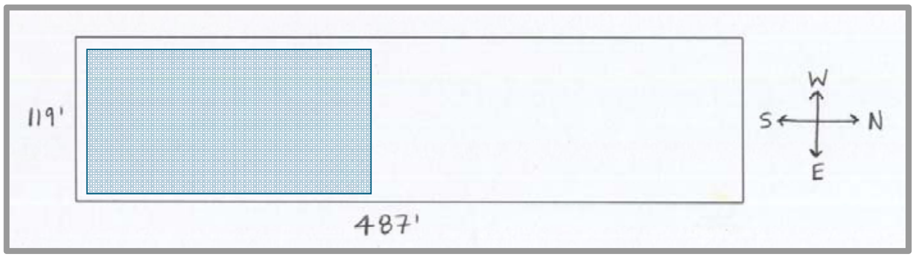


Figure 7 – Plan Dimensions and Cardinal Directions

After determining the story forces for both the north-south and east-west directions, the loads were factored (1.6 for wind and 1.0 for earthquake) to determine the controlling load case. It was found that wind primarily controlled in both directions. It was only for a few of the upper stories that earthquake loads controlled in the north-south direction. The forces and overturning moments based on the factored loads are summarized on the following pages in table 2 for the north-south direction and table 3 for the east-west direction. The overturning moments are only based on the shear force at that level. However, when the total factored moments are summed and analyzed with respect the building weight, it can be determined that overturning of the entire building is not a problem. The calculations to support the conclusion that there is no uplift are shown in the appendix. For the analysis, the shear values from the tables were applied to the computer model as 'user defined' loads, and the output was analyzed for the shear walls between levels 17 and 18. By studying one typical floor, the analysis was simplified to give a general idea of the behavior of the lateral system.

	HEIGHT, h (ft)	North-South Direction			
		WIND LOADS (k) (factor=1.6)	OVERTURNING MOMENT (k-ft)	EARTHQUAKE LOADS (k) (factor=1.0)	OVERTURNING MOMENT (k-ft)
Sky Lobby Roof Level	274.958	37.4	10294.4	45.7	12565.6
Main Roof Level	263.708	84.3	22235.9	74.9	19751.7
Level 29 (PH)	254.750	74.7	19034.9	87.3	22239.7
Level 28	245.792	73.1	17972.3	82.0	20154.9
Level 27	236.833	68.5	16218.3	65.9	15607.3
Level 26	227.875	68.5	15604.9	61.6	14037.1
Level 25	218.917	68.5	14991.4	57.4	12565.8
Level 24	209.958	68.5	14377.9	53.3	11190.8
Level 23	201.000	67.2	13507.2	49.3	9909.3
Level 22	192.042	65.1	12505.8	45.5	8737.9
Level 21	183.083	65.0	11893.1	41.8	7652.9
Level 20	174.125	63.7	11088.3	38.2	6651.6
Level 19	165.167	63.7	10517.8	34.8	5747.8
Level 18	156.208	62.1	9697.4	31.6	4936.2
Level 17	147.250	61.9	9117.7	28.4	4181.9
Level 16	138.292	60.8	8408.2	25.4	3512.6
Level 15	129.333	60.2	7780.7	22.6	2922.9
Level 14	120.375	59.4	7145.5	19.9	2395.5
Level 13	111.417	58.2	6488.9	17.4	1938.7
Level 12	102.458	57.8	5918.0	15.0	1536.9
Level 11	93.500	55.8	5221.0	12.7	1187.5
Level 10	84.542	54.7	4626.1	10.6	896.1
Level 9	75.583	53.4	4039.2	8.7	657.6
Level 8	66.625	78.6	5234.1	7.0	466.4
Level 7	56.667	90.4	5122.7	13.4	759.3
P6	44.167	80.6	3561.6	7.6	335.7
P5	35.833	61.6	2207.3	5.3	189.9
P4	27.500	58.4	1606.0	3.3	90.8
P3	19.167	54.7	1048.8	1.7	32.6
P2	10.833	91.8	994.9	0.6	6.5

Table 2 – Controlling Forces and Overturning Moments in the North-South Direction

	HEIGHT, h (ft)	**East-West Direction**			
		WIND LOADS (k) (factor=1.6)	OVERTURNING MOMENT (k-ft)	EARTHQUAKE LOADS (k) (factor=1.0)	OVERTURNING MOMENT (k-ft)
Sky Lobby Roof Level	274.958	50.5	13884.3	45.7	12565.6
Main Roof Level	263.708	180.8	47665.7	74.9	19751.7
Level 29 (PH)	254.750	160.3	40825.2	87.3	22239.7
Level 28	245.792	150.8	37057.6	82.0	20154.9
Level 27	236.833	146.9	34789.9	65.9	15607.3
Level 26	227.875	146.9	33473.9	61.6	14037.1
Level 25	218.917	146.9	32158.0	57.4	12565.8
Level 24	209.958	146.9	30842.0	53.3	11190.8
Level 23	201.000	144.8	29095.2	49.3	9909.3
Level 22	192.042	141.4	27147.1	45.5	8737.9
Level 21	183.083	141.0	25810.3	41.8	7652.9
Level 20	174.125	138.9	24182.5	38.2	6651.6
Level 19	165.167	138.9	22938.4	34.8	5747.8
Level 18	156.208	136.4	21306.8	31.6	4936.2
Level 17	147.250	136.2	20054.3	28.4	4181.9
Level 16	138.292	134.1	18551.0	25.4	3512.6
Level 15	129.333	133.2	17231.3	22.6	2922.9
Level 14	120.375	131.7	15858.7	19.9	2395.5
Level 13	111.417	130.0	14480.6	17.4	1938.7
Level 12	102.458	129.1	13231.0	15.0	1536.9
Level 11	93.500	126.1	11788.5	12.7	1187.5
Level 10	84.542	124.3	10504.9	10.6	896.1
Level 9	75.583	122.1	9227.2	8.7	657.6
Level 8	66.625	177.2	11802.8	7.0	466.4
Level 7	56.667	205.8	11662.5	13.4	759.3
P6	44.167	185.6	8195.3	7.6	335.7
P5	35.833	143.7	5150.8	5.3	189.9
P4	27.500	138.8	3815.7	3.3	90.8
P3	19.167	132.9	2546.6	1.7	32.6
P2	10.833	228.4	2474.9	0.6	6.5

Table 3 – Controlling Forces and Overturning Moments in the East-West Direction

Distribution of Lateral Loads to Shear Walls

In this building, the system of shear walls provides the means of transferring lateral loads to the foundation of the building. The concrete floor slabs serve as diaphragms in order to transfer the forces to the centrally-located walls, which extend from the top of the building all the way to the foundation system.

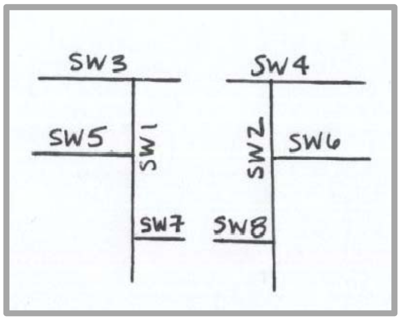


Figure 8 – Shear Wall Layout at Floor 18

As stated previously, the detailed analysis for this report is based on the shear walls between levels 17 and 18. A sketch of the shear walls at this level is shown at the left in figure 8. In the computer model, the shear walls are defined as shells, and they are meshed into areas with lengths between 24" and 30". Also, coupling beams connect shear walls 3 and 4 and shear walls 7 and 8 to increase the overall stiffness. There are no actual beams at those locations, so the beams in the model have the same thickness as the slab.

In figure 9 below, the elevations of each of the shear walls are shown. SW1 and SW2 are identical, but the rest of the walls vary slightly due to openings or coupling beams.

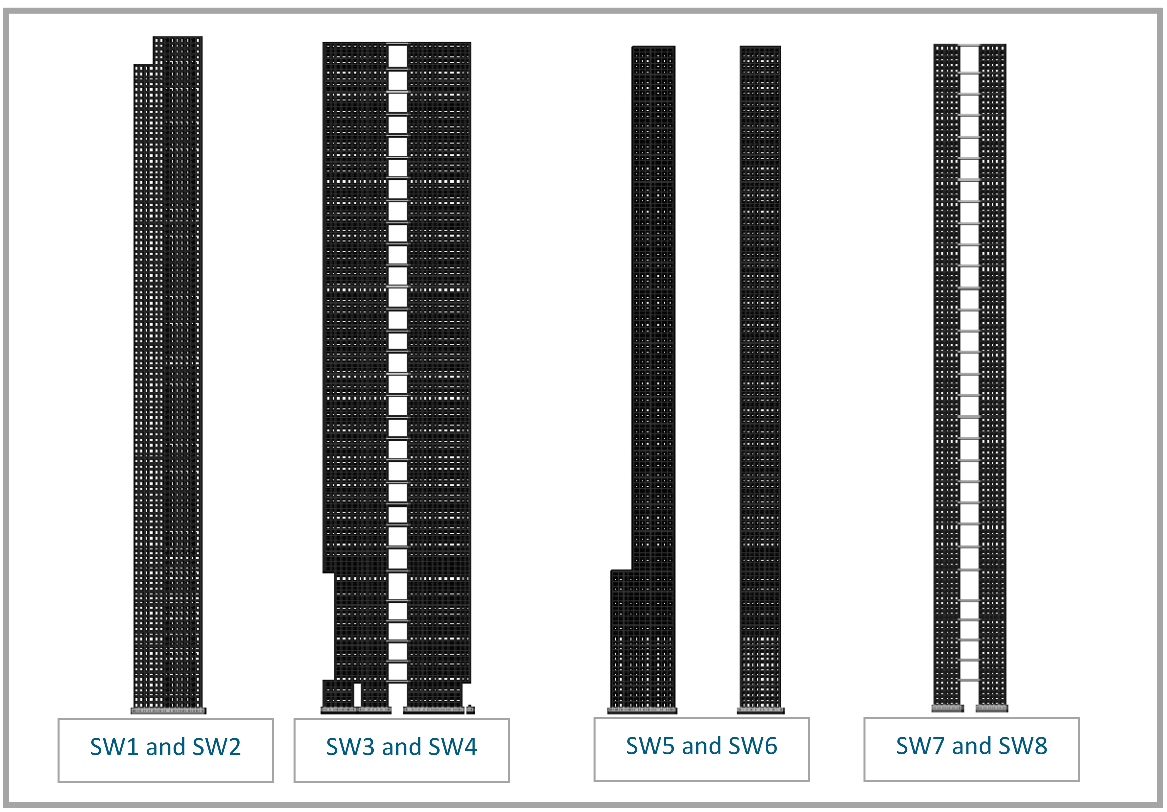


Figure 9 – Shear Wall Elevations from ETABS

After analyzing the model with the rotation locked in the z-direction, the 'section cut' function in ETABS was used to determine the forces in each shear wall. The sum of the forces in any given column adds up to the total story shear at level 18 for that load case. These shear wall forces were used to calculate the relative stiffnesses of the walls at this level. The output forces for each load case are shown in tables 4 and 5. Both load cases produced similar results for each direction, which was expected. The summary of the relative stiffnesses due to the direct shear loading is shown in table 6 below. (The cells that are not highlighted reflect out-of-plane shear forces.)

	North-South Direction			
	WIND LOADS (k) (factor=1.6)	Relative Stiffness	EARTHQUAKE LOADS (k) (factor=1.0)	Relative Stiffness
SW1	4.6	0.49	3.2	0.42
SW2	4.6	0.49	3.2	0.42
SW3	229.9	24.72	190.5	24.76
SW4	230.0	24.73	190.6	24.77
SW5	143.0	15.38	118.5	15.40
SW6	142.9	15.37	118.4	15.39
SW7	87.5	9.41	72.5	9.42
SW8	87.5	9.41	72.5	9.42
	930		769	

Table 4 – Forces in each shear wall at floor 18 due to loads in the north-south direction

	East-West Direction			
	WIND LOADS (k) (factor=1.6)	Relative Stiffness	EARTHQUAKE LOADS (k) (factor=1.0)	Relative Stiffness
SW1	955.3	48.38	370.8	48.17
SW2	948.8	48.05	371.9	48.32
SW3	15.7	0.80	6.5	0.84
SW4	15.7	0.80	6.5	0.84
SW5	10.6	0.54	4.0	0.52
SW6	9.2	0.47	3.5	0.45
SW7	9.7	0.49	2.7	0.35
SW8	9.7	0.49	3.8	0.49
	1975		770	

Table 5 – Forces in each shear wall at floor 18 due to loads in the east-west direction

	North-South Direction	East-West Direction
SW1	0.4	48.0
SW2	0.4	48.0
SW3	24.7	0.8
SW4	24.7	0.8
SW5	15.4	0.5
SW6	15.4	0.5
SW7	9.4	0.4
SW8	9.4	0.5

Table 6 – Summary of Relative Stiffnesses

After the relative stiffnesses were determined from the computer model output, the center of rigidity was calculated based on the stiffnesses and locations of the shear walls. The spreadsheet used to calculate these distances is shown below in table 7. The center of mass is shown in the table below as well in order to determine the eccentricity in each direction.

	Relative Stiffness, k	x (ft)	y (ft)
SW1	0.480	86	-
SW2	0.480	115	-
SW3	0.247	-	66.92
SW4	0.247	-	66.92
SW5	0.154	-	58.07
SW6	0.154	-	58.07
SW7	0.094	-	47.75
SW8	0.094	-	47.75
	CR (ft)	100.5	60.53
	CM (ft)	100.5	69.917

Table 7 – Center of Rigidity Calculation

Due to the partial symmetry of the shear walls, there is no eccentricity for the loads applied in the east-west direction. However, there is an eccentricity of 9.39' for the lateral loads in the north-south direction. This value creates a moment of 8730 k-ft at floor 18 due to the factored wind load (the critical load case).

Using the shear wall stiffnesses and locations relative to the center of rigidity, as well as the critical moment, the torsional forces could be calculated and added to the direct forces. The summary of these calculations are shown below in table 8. The forces for SW1 and SW 2 did not change because of the lack of eccentricity. However, the forces increase for SW3 and SW4 by about 20% and decrease for SW5 through SW8. These torsional effects would need to be considered in the design of the shear walls for two main reasons: the possible increase in lateral force (direct plus torsional force) and the resulting out-of-plane shear.

	East-West Direction		**North-South Direction**					
	SW 1	SW2	SW3	SW4	SW5	SW6	SW7	SW8
Direct Force (k)	955.3	948.8	229.9	230.0	143.0	142.9	87.5	87.5
Relative Stiffness, k_i	0.482	0.482	0.247	0.247	0.154	0.154	0.094	0.094
Distance from CR, d_i (ft)	-14.50	14.50	6.39	6.39	-2.46	-2.46	-12.78	-12.78
$k_i d_i^2$	101.3	101.3	10.1	10.1	0.9	0.9	15.4	15.4
Torsional Force (k)	0	0	53.9	53.9	-12.9	-12.9	-41.1	-41.1
Net Force (k)	955.3	948.8	283.8	283.9	130.1	130.0	46.4	46.4

Table 8 – Shear Wall Forces Including Torsional Effects

Strength Check of Typical Shear Wall

The computer program pcaColumn was used to do a strength check of one of the multi-branch shear walls between levels 17 and 18. The vertical reinforcement was entered into the program, and the layout is shown below in figure 10.

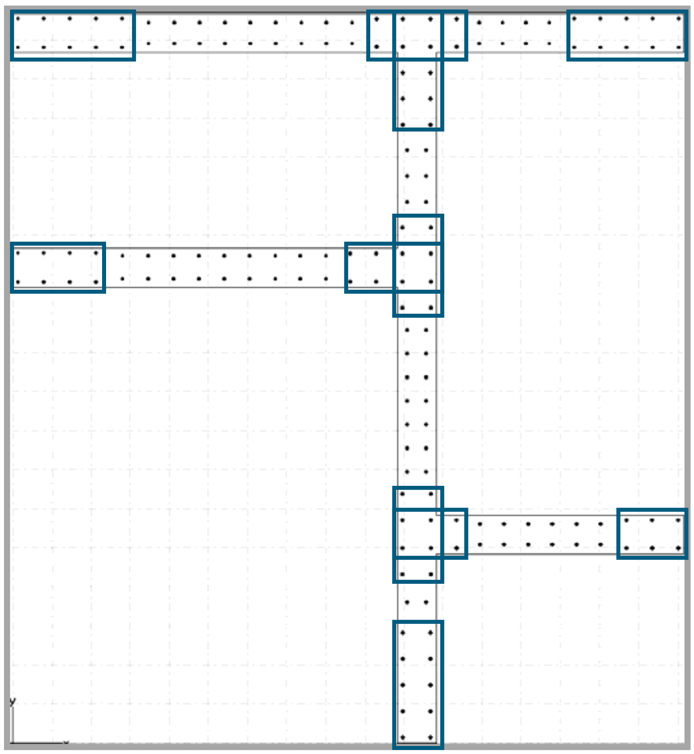


Figure 10 – Vertical Reinforcement Layout for Strength Check of Shear Wall

The groups of reinforcement that have colored boxes around them represent the boundary elements of the shear walls at this level, which have horizontal ties. The rebar sizes range from #5 bars to #10 bars, and a clear cover of 2" is applied everywhere. The spacing is 12" for all of the boundary elements and intermediate wall reinforcement. The loads applied in the program are from the load case 1.2D + 1.6W + 1.0L. For the gravity load, an area of 1680 ft^2 (multiplied by 14 floors) was used with a 145 psf dead load and 40 psf live load for a total axial load of 5030 k. The moment applied to the model was determined from the story height and total story shear (from the factored wind load) in the east-west direction. The value for the overturning moment was then divided by two because there is an identical shear wall to the right of the one shown above.

Based on the section drawn in the model, the following interaction diagram resulted (shown below in fig. 11). The calculated loads fell slightly outside of the diagram, as can be seen in the bottom right corner of the figure. This discrepancy is possibly due to the fact that the horizontal wall reinforcement was not modeled in the program. (Horizontal reinforcement cannot be modeled in pcaColumn. Further investigation of the wall using pcaWall could be done in the future.) In any case, because the point is so close to the edge of the diagram, it is possible that the wall strength limits the design, not the deflection or drift limitations. However, a drift analysis is shown in the next section.

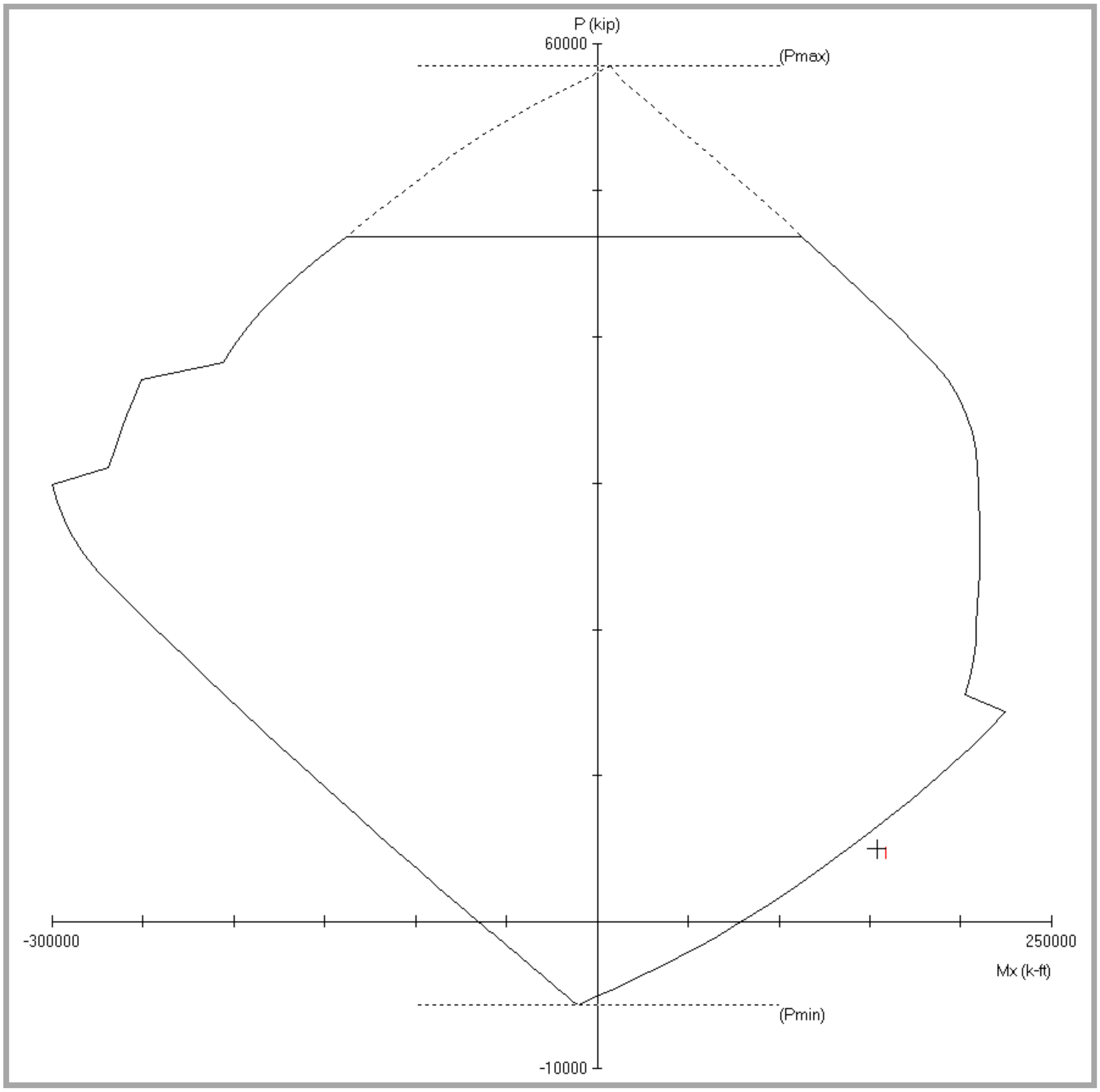


Figure 11 – Resulting Axial / Moment Interaction Diagram

Drift Analysis

The ETABS model was used to find the drift ratios for each level and for each load case. Table 9 below summarizes the drift analysis for seismic loads. The 'maximum drift ratio' at each level could be from either the seismic load in the x-direction or the y-direction. This ratio was multiplied by the story height in order to find the story drift. These values were then compared with the seismic story drift limitations, which were calculated from the equation from table 12.12-1 in ASCE 7-05. (The higher values are highlighted in the table.) As it turns out, the story drift was too high above level 14. All of the story drifts were then added to find the total building drift at each level. These values were acceptable when being compared with the drift limitation of h/500 (recommended by the structural engineer).

	Height, ft	Maximum Drift Ratio	Story Drift, in	Seismic Story Drift Limitation, in (= 0.020h)	Total Building Drift at Given Story, in	Seismic Drift Limitation, in (= h/500)
Sky Lobby Roof Level	274.96	0.002222	0.300	0.225	5.709	6.599
Main Roof Level	263.71	0.002552	0.274	0.179	5.409	6.329
Level 29 (PH)	254.75	0.002559	0.275	0.179	5.134	6.114
Level 28	245.79	0.002227	0.239	0.179	4.859	5.899
Level 27	236.83	0.002550	0.274	0.179	4.620	5.684
Level 26	227.88	0.002550	0.274	0.179	4.346	5.469
Level 25	218.92	0.002544	0.273	0.179	4.072	5.254
Level 24	209.96	0.002531	0.272	0.179	3.798	5.039
Level 23	201.00	0.002508	0.270	0.179	3.526	4.824
Level 22	192.04	0.002475	0.266	0.179	3.257	4.609
Level 21	183.08	0.002431	0.261	0.179	2.990	4.394
Level 20	174.13	0.002373	0.255	0.179	2.729	4.179
Level 19	165.17	0.002301	0.247	0.179	2.474	3.964
Level 18	156.21	0.002214	0.238	0.179	2.227	3.749
Level 17	147.25	0.002111	0.227	0.179	1.989	3.534
Level 16	138.29	0.001990	0.214	0.179	1.762	3.319
Level 15	129.33	0.001851	0.199	0.179	1.548	3.104
Level 14	120.38	0.001693	0.182	0.179	1.349	2.889
Level 13	111.42	0.001580	0.170	0.179	1.167	2.674
Level 12	102.46	0.001482	0.159	0.179	0.997	2.459
Level 11	93.50	0.001376	0.148	0.179	0.838	2.244
Level 10	84.54	0.001259	0.135	0.179	0.690	2.029
Level 9	75.58	0.001131	0.122	0.179	0.554	1.814
Level 8	66.63	0.000899	0.107	0.199	0.433	1.599
Level 7	56.67	0.000777	0.117	0.250	0.325	1.360
P6	44.17	0.000646	0.065	0.167	0.209	1.060
P5	35.83	0.000536	0.054	0.167	0.144	0.860
P4	27.50	0.000417	0.042	0.167	0.091	0.660
P3	19.17	0.000292	0.029	0.167	0.049	0.460
P2	10.83	0.000152	0.020	0.217	0.020	0.260

Table 9 – Drift analysis due to seismic loads

The following chart (table 10) summarizes the drift analysis for the wind loads. The process for determining all of the drifts is similar to that described above, and in the case of wind, the total building drift is too high again at the upper stories. These problems can be justified by the inaccuracy of the building model in ETABS. After initially running the analysis, the period of the building was too high for the height and structural system. A few changes were made (decreasing the masses of the diaphragms and adding 'coupling beams' between a few of the shear walls to increase stiffness), but the period only decreased by about two seconds. This issue with the model resulted in a building with too much flexibility. The over-estimation of flexibility then caused too much drift in the computer output .

	Height, ft	Maximum Drift Ratio	Story Drift, in	Total Building Drift at Given Story, in	Wind Drift Limitation, in (= h/500)
Sky Lobby Roof Level	274.958	0.003528	0.476280	9.017386	6.599
Main Roof Level	263.708	0.003696	0.397305	8.541106	6.329
Level 29 (PH)	254.750	0.003718	0.399670	8.143800	6.114
Level 28	245.792	0.003534	0.379919	7.744130	5.899
Level 27	236.833	0.003664	0.393880	7.364211	5.684
Level 26	227.875	0.003687	0.396353	6.970331	5.469
Level 25	218.917	0.003705	0.398287	6.573979	5.254
Level 24	209.958	0.003716	0.399470	6.175691	5.039
Level 23	201.000	0.003717	0.399563	5.776221	4.824
Level 22	192.042	0.003706	0.398425	5.376658	4.609
Level 21	183.083	0.003681	0.395693	4.978234	4.394
Level 20	174.125	0.003638	0.391070	4.582541	4.179
Level 19	165.167	0.003577	0.384556	4.191471	3.964
Level 18	156.208	0.003493	0.375484	3.806914	3.749
Level 17	147.250	0.003385	0.363874	3.431431	3.534
Level 16	138.292	0.003249	0.349293	3.067557	3.319
Level 15	129.333	0.003084	0.331518	2.718264	3.104
Level 14	120.375	0.002886	0.310245	2.386746	2.889
Level 13	111.417	0.002663	0.286283	2.076501	2.674
Level 12	102.458	0.002521	0.270997	1.790218	2.459
Level 11	93.500	0.002363	0.254013	1.519220	2.244
Level 10	84.542	0.002188	0.235228	1.265207	2.029
Level 9	75.583	0.001989	0.213810	1.029980	1.814
Level 8	66.625	0.001618	0.193345	0.816170	1.599
Level 7	56.667	0.001445	0.216750	0.622826	1.360
P6	44.167	0.001221	0.122110	0.406076	1.060
P5	35.833	0.001028	0.102796	0.283966	0.860
P4	27.500	0.000814	0.081400	0.181170	0.660
P3	19.167	0.000583	0.058300	0.099770	0.460
P2	10.833	0.000319	0.041470	0.041470	0.260

Table 10 – Drift analysis due to wind loads

Conclusions

The analysis of the lateral system of Aquablue proved to be beneficial in further understanding the existing conditions of the building. It was concluded that the wind loads primarily controlled over seismic loads in both the north-south and east-west directions. As for the relative stiffness, torsion, and drift analyses, the use of computer programs provided an in-depth understanding of the behavior of the structure and fairly accurate results. However, there is room for improvement in the future in the development of computer models to ensure even more precise results.

The relative stiffnesses found from the resulting forces in each shear wall due to direct shear seemed to be reasonable based on their locations and lengths. The longer walls were stiffer than the shorter walls in the same direction. Also, it was found that there are some torsional effects in the building due to eccentricity between the center of rigidity and center of mass. This was expected due to the centralized location of the shear walls. If the walls had been more spread out toward the edges of the slab, torsion would not have had the same impact.

Although level 7 was not studied in this report, this could be a potential area of weakness in the lateral system. Because of the huge change in diaphragm size and weight, there is a possibility for shear reversals at the base of the two towers. The sudden shift in shear forces could cause cracking and other damage at the wall/slab intersection. This level (and possibly the levels below) would have to be given special consideration, and in the existing design, there is a large increase in shear wall length at this transition zone. If the lateral system is changed in any way for this thesis project, this unique area would have to be given extra attention.

Appendix

UPDATED WIND CALCULATIONS

In general, there were no changes in the calculations of the wind pressures for this technical report. However, the pressures were applied to a smaller area, because this report focuses on just one of the towers. The resulting story forces are shown below in table A1.

	North-South Direction	East-West Direction
Sky Lobby Roof Level	23.4	31.6
Main Roof Level	52.7	113.0
Level 29 (PH)	46.7	100.2
Level 28	45.7	94.2
Level 27	42.8	91.8
Level 26	42.8	91.8
Level 25	42.8	91.8
Level 24	42.8	91.8
Level 23	42.0	90.5
Level 22	40.7	88.4
Level 21	40.6	88.1
Level 20	39.8	86.8
Level 19	39.8	86.8
Level 18	38.8	85.3
Level 17	38.7	85.1
Level 16	38.0	83.8
Level 15	37.6	83.3
Level 14	37.1	82.3
Level 13	36.4	81.2
Level 12	36.1	80.7
Level 11	34.9	78.8
Level 10	34.2	77.7
Level 9	33.4	76.3
Level 8	49.1	110.7
Level 7	56.5	128.6
P6	50.4	116.0
P5	38.5	89.8
P4	36.5	86.7
P3	34.2	83.0
P2	57.4	142.8

Table A1 – Story forces due to lateral wind forces

UPDATED SEISMIC CALCULATIONS

The new seismic calculations are shown below. Many variables remained the same, but there was a change in the value of R and the fundamental period. Because of these changes, C_s increased from 0.0145 to 0.0165.

$S_s = 0.882$
$S_1 = 0.301$ — latitude = 18.4307°, longitude = −66.0791° (Hato Rey, Puerto Rico)
site class: B

$F_a = 1.0$ (table 11.4-1)
$F_v = 1.0$ (table 11.4-2)

$S_{MS} = F_a S_s = 1.0(0.882) = 0.882$
$S_{M1} = F_v S_1 = 1.0(0.301) = 0.301$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.882) = 0.588$
$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.301) = 0.201$

seismic design category: D

$R = 6$ (table 12.2-1 — building frame system with special reinforced concrete shear walls)

*Note: According to the code, this building type has a height limit of 160', but it best described the existing system. Also, the engineers used a value of 5.5 for R, which is close to R=6.

$T_a = C_t h_n^x = 0.02(276)^{0.75} = 1.354$ sec
- $C_t = 0.02$
- $x = 0.75$
(table 12.8-2)

T = fundamental period $= C_u T_a = 1.5(1.354 \text{ sec}) = 2.031$ sec
- $C_u = 1.5$ (based on $S_{D1} = 0.201$, table 12.8-1)

$$C_s = \frac{S_{DS}}{(R/I)} = \frac{0.588}{(6/1.0)} = 0.098$$

$$\leq \frac{S_{D1}}{T(R/I)} = \frac{0.201}{2.031(6/1.0)} = 0.0165$$

$\boxed{C_s = 0.0165}$

Level(s)	Total Dead Load (psf)	Area (ft^2)	Weight per Floor (k)
2	153	23444	3587
3 to 6	150	23444	3517
7	170	23444	3985
8 to 17	145	10744	1558
18 to 27	145	10744	1558
28 to 29	145	12508	1814
Roof	117	12508	1463
Sky Lobby	115	7205	829

Total Building Weight, W (k) =	58716
C_s =	0.0165
Base Shear, V (k) =	969

Table A2 – Total dead load per floor

Also, the dead load at each floor was slightly decreased to more accurately reflect the actual weight of the building. The new loads (based on one tower) and resulting base shear are shown to the left in table A2.

The chart below (table A3) summarizes the new lateral seismic forces.

Level	Weight, w_x (k)	Height, h_x (ft)	k	$w_x.h_x^k$	Lateral Seismic Force, F_x (k)	Total Story Shear, V_x (k)
2	3587	10.833	1.77	243366	0.6	969.0
3	3517	19.167	1.77	655058	1.7	968.4
4	3517	27.500	1.77	1241056	3.3	966.6
5	3517	35.833	1.77	1982684	5.3	963.3
6	3517	44.167	1.77	2870752	7.6	958.1
7	3985	56.667	1.77	5056183	13.4	950.4
8	1558	66.625	1.77	2632724	7.0	937.0
9	1558	75.583	1.77	3291381	8.7	930.0
10	1558	84.542	1.77	4013154	10.6	921.3
11	1558	93.500	1.77	4796274	12.7	910.7
12	1558	102.458	1.77	5639410	15.0	897.9
13	1558	111.417	1.77	6541378	17.4	883.0
14	1558	120.375	1.77	7500957	19.9	865.6
15	1558	129.333	1.77	8517125	22.6	845.7
16	1558	138.292	1.77	9589109	25.4	823.1
17	1558	147.250	1.77	10715818	28.4	797.7
18	1558	156.208	1.77	11896583	31.6	769.3
19	1558	165.167	1.77	13130814	34.8	737.7
20	1558	174.125	1.77	14417556	38.2	702.9
21	1558	183.083	1.77	15756308	41.8	664.6
22	1558	192.042	1.77	17146629	45.5	622.8
23	1558	201.000	1.77	18587650	49.3	577.3
24	1558	209.958	1.77	20079046	53.3	528.0
25	1558	218.917	1.77	21620263	57.4	474.7
26	1558	227.875	1.77	23210825	61.6	417.4
27	1558	236.833	1.77	24850278	65.9	355.8
28	1814	245.792	1.77	30898841	82.0	289.9
29	1814	254.750	1.77	32919968	87.3	207.9
Roof	1463	263.708	1.77	28224911	74.9	120.6
Sky Lobby	829	274.958	1.77	17220906	45.7	45.7

$\sum w_i.h_i^k$ = 365247007

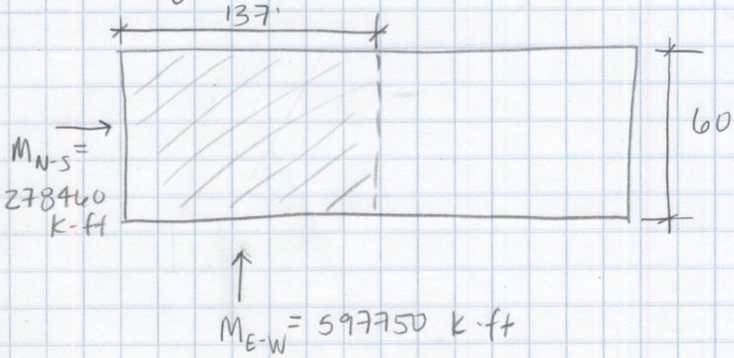
Table A3 – New lateral forces based on the weight of one tower

CHECK FOR OVERTURNING

Overturning at foundations check

Weight = 58716 k
½ weight = 29358 k

factored (0.9D) = 52840 k
= 26420 k



— Dimensions are from exterior column to exterior column (where the loads are transferred to the foundations)

— Wind loads are factored (1.6)

north-south direction

$$\frac{278460\ \text{k·ft}}{137'} = 2030\ \text{k}$$ (tension and compression on either side of building)

$\frac{1}{2}W = 26420\ \text{k} > 2030\ \text{k}$

∴ no uplift

east-west direction

$$\frac{597750\ \text{k-ft}}{60'} = 9960\ \text{k}$$ (tension and compression)

$\frac{1}{2}W = 26420\ \text{k} > 9960\ \text{k}$

∴ no uplift