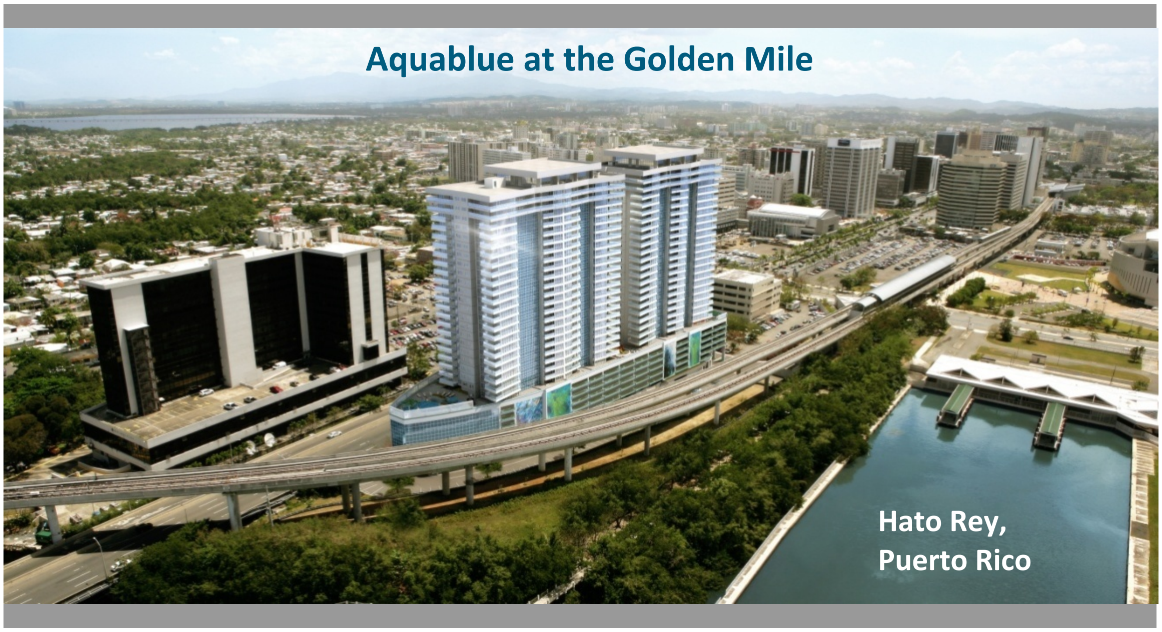
Structural Technical Report 1

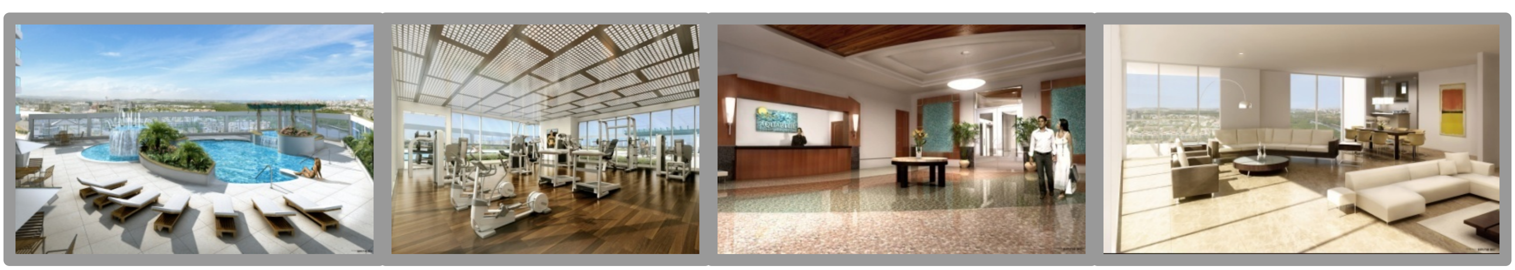


Lindsay Lynch

Structural Option

Dr. Andres Lepage

29 September 2008



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Executive Summary

The purpose of this report was to study the existing structural conditions of Aquablue at the Golden Mile. After studying the drawings, it has been determined that the primary method of resisting the lateral forces is a system of shear walls that extend from the first level through the two towers of the building. As for the transfer of gravity loads, the typical floor framing is a two-way, flat plate, post-tensioned slab supported by reinforced concrete columns. Based on the rectangular columns and the extremely long and narrow building dimensions, it has been assumed that the slab and column system also serves as a part of the lateral force resisting system.

The preliminary wind and seismic calculations have proven that the lateral loads are very severe for this building. Due to the height of the building (approximately 280'), the location (coast of Puerto Rico) and the uneven overall dimensions, the lateral forces are especially strong in the direction perpendicular to the larger face of the building. With base dimensions of 119'x487', the wind forces applied parallel to the 119' dimension could possible cause problems with overturning. The lateral system will be investigated further in the third technical assignment.

General Building Information

Aquablue at the Golden Mile is a high-rise apartment building in Hato Rey, Puerto Rico, which is a sub-district of San Juan. 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 square feet, and there are 31 stories above grade. The ground level will be developed as a commercial area, and the rest of the floors up until 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, which are separated into two towers. There is a sky lobby above the penthouse apartments.

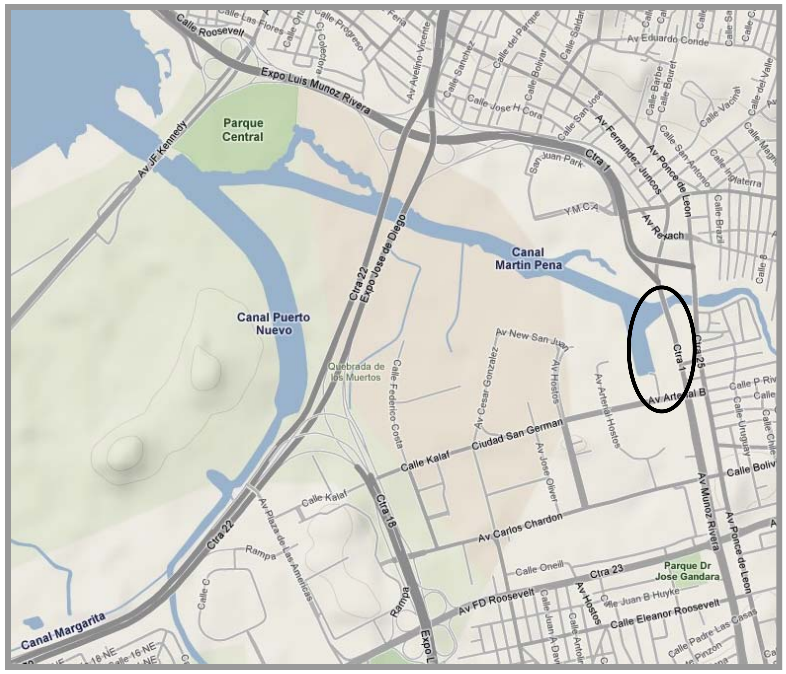


Fig. 1 – Building Site (maps.google.com – Hato Rey Central, Puerto Rico)

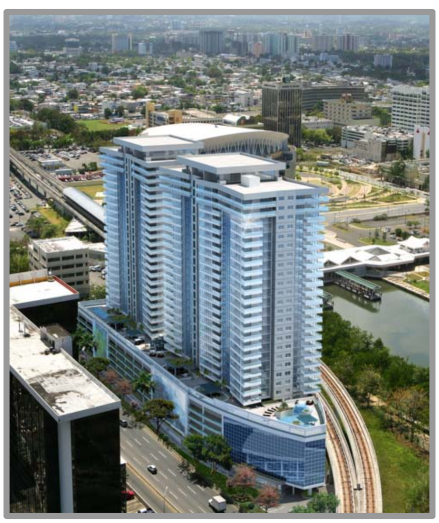


Fig. 2 – Rendering of Aquablue

The parking structure 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 at the bottom of figure 2. Above level 7, the primary 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 concrete shear walls. Each floor has a post-tensioned slab supported by columns, but there are no beams transferring the gravity loads. However, there are some beams in the building that are a part of the lateral system.

Codes

- Codes used in this report for preliminary 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)

- Codes used in this report for initial structural member 'spot checks':
    - IBC 2006
        - Section 1607 – Live Loads
    - ACI 318-08
        - Chapter 9 – Strength Reduction Factors
        - Chapter 10 – Compressive Member Design
        - Chapter 18 – Prestressed Concrete

- 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")

Typical Elevations and Framing Plans

The view of the building from the narrow end is shown in figure 3 at the left, which is the north elevation. The office areas and glass curtain wall are located on the bottom floors, and the apartment floors rise above.

The east elevation is shown in figure 4 below, where the two distinct apartment towers can easily be seen. The parking garage is the main area on the bottom floors, but the offices can be seen again in the bottom right of the figure. The drastic contrast in overall building dimensions can be seen by comparing these two figures.

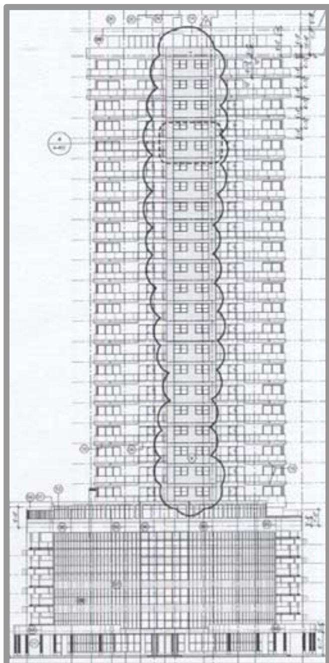


Fig. 3 – North Elevation

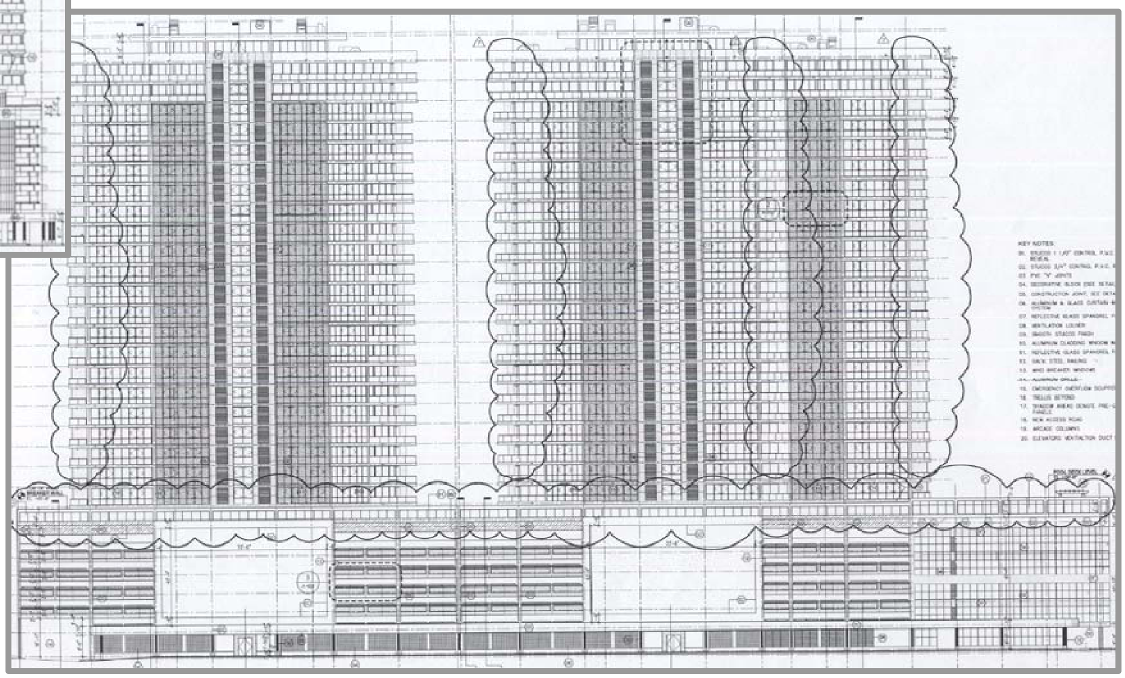


Fig. 4 – East Elevation

There are two typical floor plans in this building: one to represent the parking garage levels and one for the apartment levels. As seen below on one of the parking level plans (fig. 5), 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. There are no beams running from column to column throughout the floor plans.

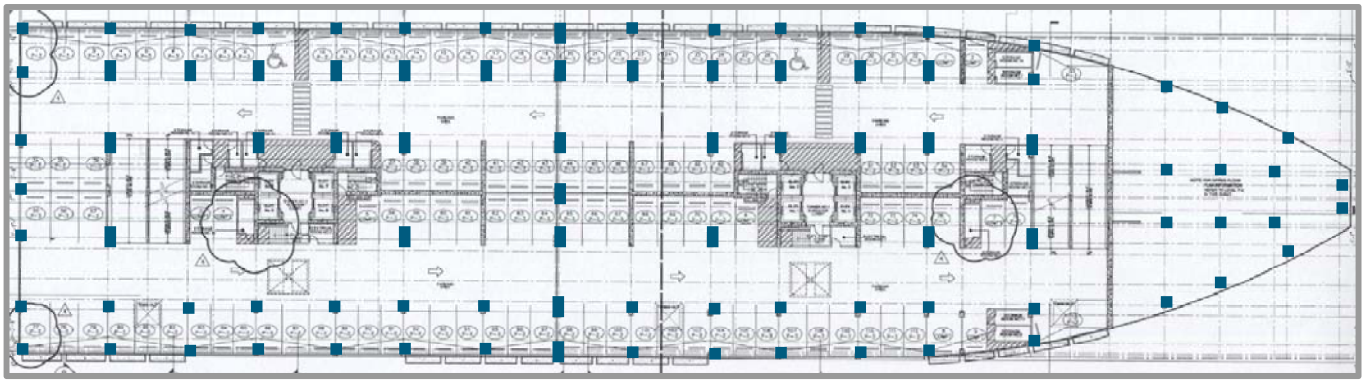


Fig. 5 – Column Layout for Typical Parking Garage Level

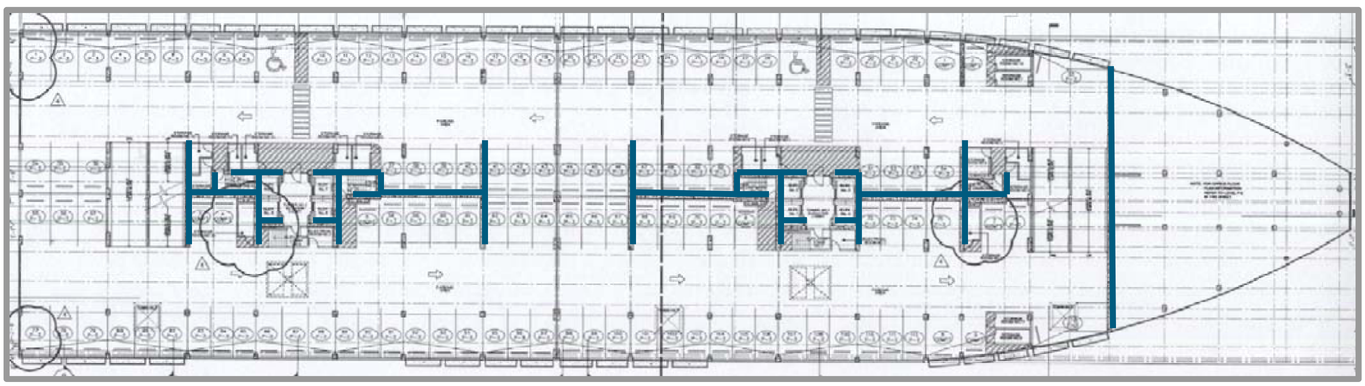


Fig. 6 – Shear Wall Layout on a Parking Level

A system of reinforced concrete shear walls concentrated toward the center of the floor plan composes the lateral force resisting system. An example of the shear wall system for one of the parking garage levels is shown above in figure 6. The most shear walls are at the base of the building, and as they move up through the floors, they become slightly less complex. The plan below (fig. 7) is a typical apartment level floor plan, and it combines the columns and shear walls into one figure.

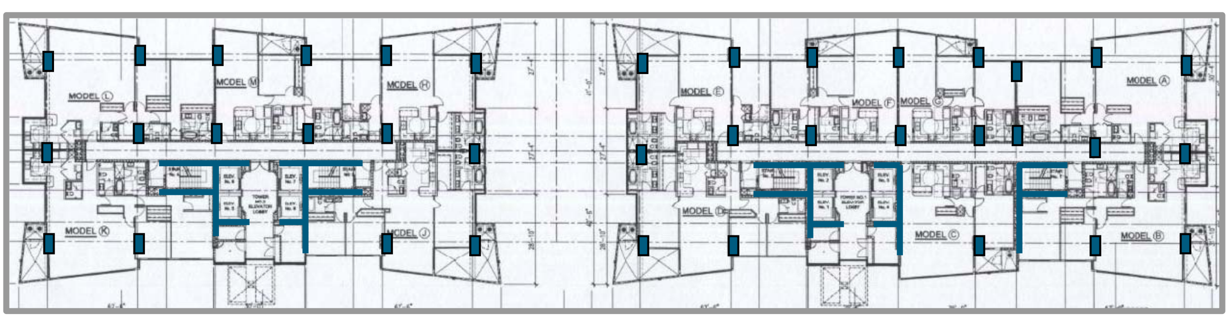


Fig. 7 – Column and Shear Wall Layout for Typical Apartment Level

Detailed Description of 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, which are all parking levels. At level 7, parts of the slab are 12" thick because the loads are heavier on this level. There is an outdoor pool, landscaping, and a finished outdoor area. For the apartment levels, the post-tensioned slabs are 8" thick.

The primary **lateral force resisting system** is a series of shear walls near the core of the building. They are 18" thick, and most of them require boundary elements. The system of shear walls is grouped into two sections, and each one extends into one of the apartment towers. Figure 8 shows an example of one section of shear walls that extends from level 7 through level 9.

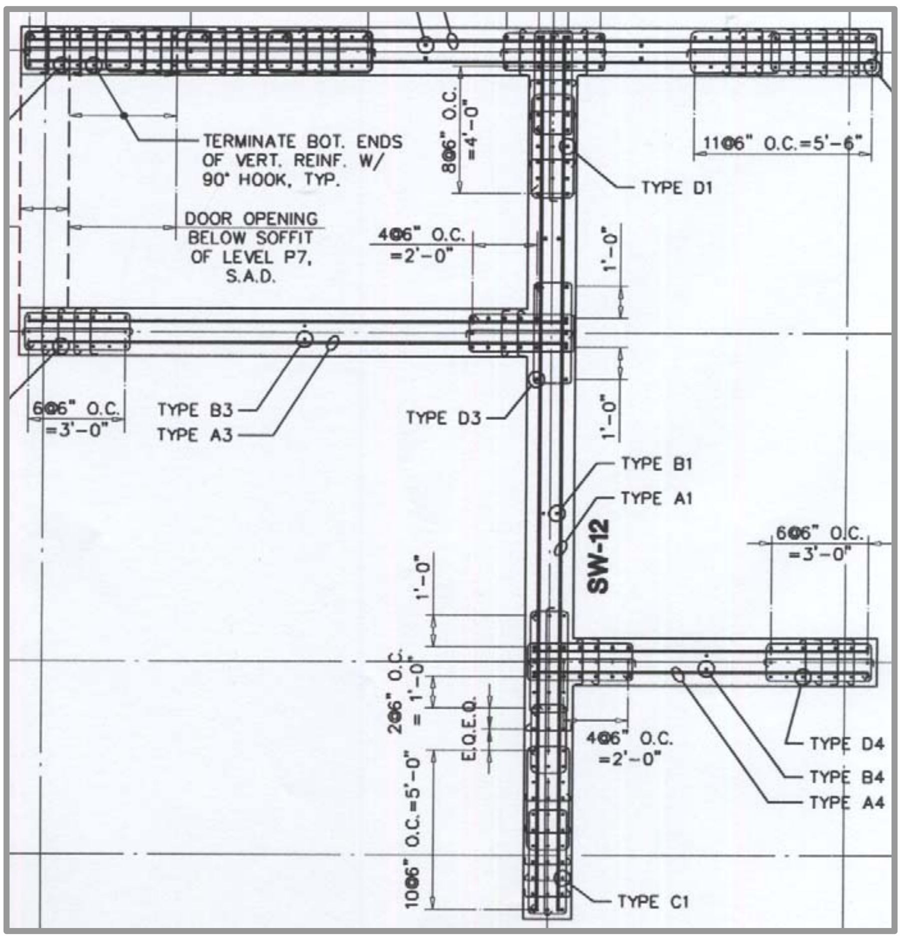


Fig. 8 – Example of Shear Wall System

The slab and column framing system also serves as a **secondary lateral force resisting system**. Together they serve as a moment frame, which can be deduced from the column dimensions. The average column is 18"x48", with the long dimension parallel to the direction of the critical lateral force. There are also beams on either end of the apartment levels, spanning the short dimension. It is assumed that they, too, are a part of the moment frame to resist the extremely high lateral forces.

There is one **expansion joint**, which breaks the building into two sections that are nearly square. It is a 5” seismic joint, and it runs parallel to the short dimension of the building.

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.

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	foundation - level 13	8
	above level 13	6

Table 1 – Concrete Strengths for Various Structural Elements

Preliminary Wind Analysis

The following analysis is based on Method 2 (analytical method) for wind design in ASCE 7-05. All variables were determined or estimated from the code (table 2). The building was assumed to be flexible based on the calculated natural frequency. For more information on the determination of variables and calculations of wind pressures, see Appendix A.

Basic Wind Speed	-	V = 145 mph
Occupancy Category	-	II
Exposure Category	-	B
Importance Factor	-	I = 1.0
Damping Ratio	-	β = 2%
Natural Frequency	-	η_1 = 0.362 Hz

Table 2 – Basic Variables for Wind Calculations

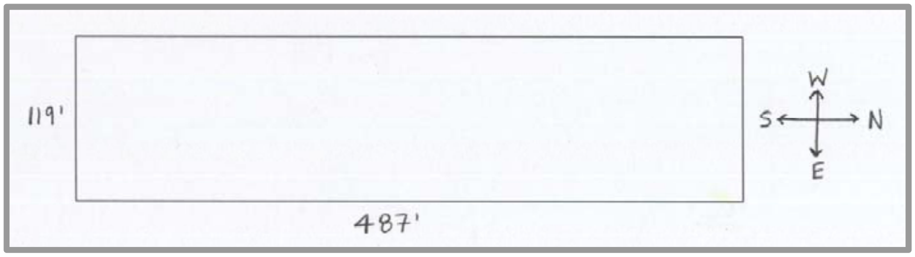


Fig. 9 – Plan Dimensions and Wind Directions

The diagrams below (figs. 10 and 11) represent the windward and leeward velocity pressures (in psf) for both the north-south and east-west directions (see fig. 9). The information is also listed in table format with the total design wind pressures (table 3). Lastly, the story forces and cumulative story shears are tabulated for each of the wind directions in table 4.

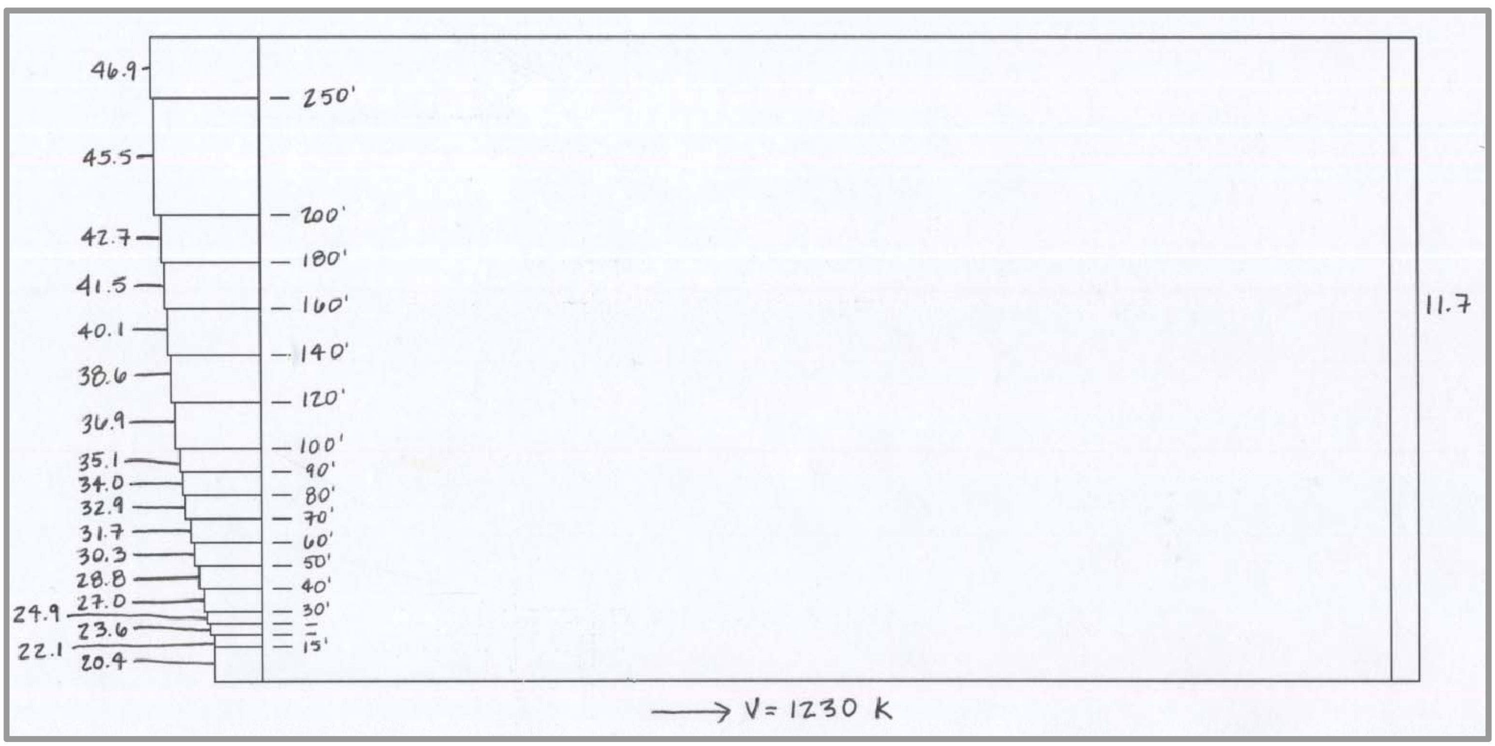


Fig. 10 – Wind Pressures (psf) for the North-South Direction

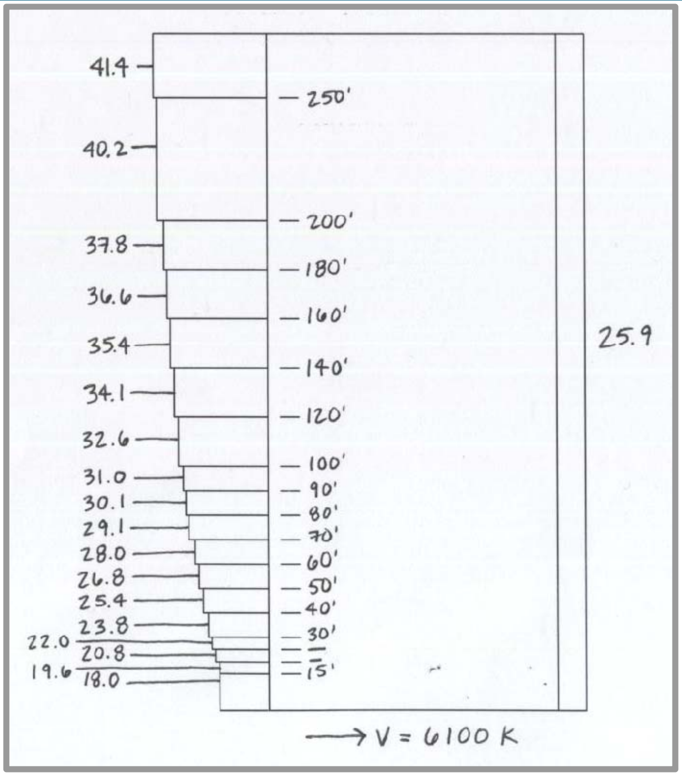


Fig. 11 – Wind Pressures (psf) for the East-West Direction

Height above ground level, z or h (ft)	North-South Direction			East-West Direction		
	Windward pressure (psf)	Leeward suction (psf)	Total wind pressure (psf)	Windward pressure (psf)	Leeward suction (psf)	Total wind pressure (psf)
15	20.38	-11.71	32.09	18.01	-25.87	43.88
20	22.13	-11.71	33.85	19.56	-25.87	45.43
25	23.59	-11.71	35.30	20.85	-25.87	46.72
30	24.85	-11.71	36.56	21.96	-25.87	47.83
40	26.98	-11.71	38.69	23.84	-25.87	49.71
50	28.76	-11.71	40.47	25.41	-25.87	51.28
60	30.29	-11.71	42.00	26.77	-25.87	52.64
70	31.65	-11.71	43.37	27.97	-25.87	53.85
80	32.89	-11.71	44.60	29.06	-25.87	54.94
90	34.01	-11.71	45.72	30.06	-25.87	55.93
100	35.05	-11.71	46.76	30.97	-25.87	56.85
120	36.93	-11.71	48.64	32.63	-25.87	58.50
140	38.59	-11.71	50.30	34.10	-25.87	59.97
160	40.09	-11.71	51.80	35.43	-25.87	61.30
180	41.46	-11.71	53.17	36.64	-25.87	62.51
200	42.73	-11.71	54.44	37.76	-25.87	63.63
250	45.54	-11.71	57.25	40.24	-25.87	66.12
276	46.85	-11.71	58.56	41.40	-25.87	67.27

* up to level 7 (parking structure)   * above level 7 (residential towers)

Table 3 – Wind Pressures (psf)

North-South Direction				
Level	Floor Elevation (ft)	Story Force (k)	Total Story Shear (k)	Overturning Moment Contribution from Each Level (ft-k)
2	10.8	57.4	1230.5	622
3	19.2	34.2	1173.0	656
4	27.5	36.5	1138.8	1005
5	35.8	38.5	1102.3	1379
6	44.2	50.4	1063.8	2225
7	56.7	56.5	1013.4	3204
8	66.6	49.1	956.8	3272
9	75.6	33.4	907.7	2522
10	84.5	34.2	874.4	2891
11	93.5	34.9	840.2	3262
12	102.5	36.1	36.1	3695
13	111.4	36.4	769.2	4054
14	120.4	37.1	732.8	4461
15	129.3	37.6	695.8	4866
16	138.3	38.0	658.1	5251
17	147.3	38.7	620.2	5706
18	156.2	38.8	581.4	6065
19	165.2	39.8	542.6	6569
20	174.1	39.8	502.8	6925
21	183.1	40.6	463.1	7428
22	192.0	40.7	422.5	7820
23	201.0	42.0	381.8	8444
24	210.0	42.8	339.7	8991
25	218.9	42.8	296.9	9375
26	227.9	42.8	254.1	9759
27	236.8	42.8	211.3	10142
28	245.8	45.7	168.5	11227
29	254.8	46.7	122.8	11894
roof level	263.7	52.7	76.1	13887
sky lobby roof level	275.0	23.4	23.4	6441

Total Overturning Moment (ft-k) = 174040

East-West Direction				
Level	Floor Elevation (ft)	Story Force (k)	Total Story Shear (k)	Overturning Moment Contribution from Each Level (ft-k)
2	10.8	320.6	6104.4	3473
3	19.2	186.4	5783.8	3573
4	27.5	194.7	5597.4	5354
5	35.8	201.7	5402.7	7228
6	44.2	260.4	5201.0	11500
7	56.7	288.8	4940.6	16365
8	66.6	248.6	4651.8	16562
9	75.6	171.3	4403.2	12947
10	84.5	174.4	4231.9	14741
11	93.5	176.9	4057.6	16542
12	102.5	181.2	181.2	18567
13	111.4	182.4	3699.4	20319
14	120.4	184.9	3517.1	22252
15	129.3	187.0	3332.2	24180
16	138.3	188.2	3145.3	26032
17	147.3	191.1	2957.0	28140
18	156.2	191.4	2765.9	29897
19	165.2	194.9	2574.5	32187
20	174.1	194.9	2379.6	33933
21	183.1	197.8	2184.8	36218
22	192.0	198.4	1987.0	38095
23	201.0	203.1	1788.6	40826
24	210.0	206.1	1585.5	43279
25	218.9	206.1	1379.3	45125
26	227.9	206.1	1173.2	46972
27	236.8	206.1	967.1	48818
28	245.8	211.6	761.0	52002
29	254.8	224.9	549.4	57288
roof level	263.7	253.6	324.5	66888
sky lobby roof level	275.0	70.9	70.9	19483

Total Overturning Moment (ft-k) = 838785

Table 4 – Total Story Shears and Overturning Moment Due to the Wind Load in Each Direction

Preliminary Seismic Analysis

The equivalent lateral force procedure from ASCE7-05 was used for the seismic analysis, and the building was assumed to be rigid. The latitude and longitude of the site were determined from an online map, and the spectral response accelerations were determined from an online calculator. (See Appendix B for the references.) The other design variables were determined from the code, although there were a few differences between the code values and design values. The design engineers used a value of 0.0389 for C_s, which would cause a large difference in the calculated base shear (see table 5 for C_s value used in this analysis). Also, the spectral response accelerations were slightly different. The design engineers used a value of 0.73 for S_s and a value of 0.41 for S_1. However, since no calculations were done in this report for the lateral force resisting system, the magnitude of these differences is yet unknown.

Site Class	-	B
Spectral Response Accelerations	-	$S_s = 0.882$
	-	$S_1 = 0.301$
	-	$S_{DS} = 0.588$
	-	$S_{D1} = 0.201$
Seismic Design Category	-	D
Response Modification Coefficient	-	R = 5.5
Period	-	T = 2.517 sec
Seismic Response Coefficient	-	$C_s = 0.0145$

Table 5 – Basic Variables for Seismic Analysis

The assumed dead load per floor was calculated based on the primary structural members and other assumed material weights (see Appendix B for the complete spreadsheet), and the values are listed in table 6. An equation from the code was used to relate the weights and heights of each level and determine the lateral seismic force per floor. These values were then added to determine the total shear forces at each story. See table 7 on the next page for a summary of these calculations.

Level(s)	Dead Load (psf)	Area (ft^2)	Weight per Floor (k)
2	186.5	51950	9689
3 to 6	166.8	51950	8665
7	231.3	51950	12016
8 to 17	196.6	26150	5141
18 to 27	194.6	26150	5089
28 to 29	189.9	28700	5450
Roof	159.7	28700	4583
Sky Lobby Roof	127.5	9480	1209

Total Building Weight, W (k) =	175357
C_s =	0.0145
Base Shear, V (k) =	2543

Table 6 – Building Weight Contribution from Each Story

Level	Weight, w_x (k)	Height, h_x (ft)	Lateral Seismic Force, F_x (k)	Seismic Design Story Shear, V_x (k)	Overturning Moment from Each Level (ft-k)
2	9689	10.833	0.8	2543.0	8
3	8665	19.167	2.1	2542.2	41
4	8665	27.500	4.4	2540.1	121
5	8665	35.833	7.5	2535.7	267
6	8665	44.167	11.3	2528.2	501
7	12016	56.667	25.9	2516.9	1466
8	5141	66.625	15.3	2491.0	1020
9	5141	75.583	19.7	2475.7	1489
10	5141	84.542	24.6	2456.0	2083
11	5141	93.500	30.1	2431.4	2818
12	5141	102.458	36.2	2401.2	3709
13	5141	111.417	42.8	2365.0	4769
14	5141	120.375	50.0	2322.2	6014
15	5141	129.333	57.7	2272.3	7459
16	5141	138.292	65.9	2214.6	9119
17	5141	147.250	74.8	2148.7	11009
18	5089	156.208	83.3	2073.9	13010
19	5089	165.167	93.1	1990.6	15379
20	5089	174.125	103.5	1897.5	18019
21	5089	183.083	114.4	1794.0	20946
22	5089	192.042	125.9	1679.6	24174
23	5089	201.000	137.9	1553.7	27717
24	5089	209.958	150.5	1415.8	31591
25	5089	218.917	163.6	1265.4	35809
26	5089	227.875	177.2	1101.8	40388
27	5089	236.833	191.4	924.6	45341
28	5450	245.792	220.8	733.1	54278
29	5450	254.750	237.2	512.3	60432
Roof	4583	263.708	213.8	275.1	56370
Sky Lobby	1209	274.958	61.3	61.3	16856

Total Overturning Moment (ft-k) = 512204

Table 7 – Story Forces and Cumulative Story Shears Based on Seismic Analysis

'Spot Check' Calculations

Various 'spot-checks' were calculated in order to compare with the existing design. For this report, an interior column and an area of a typical post-tensioned slab were analyzed. Although there are some beams throughout the structure, they were not assessed because they are not a part of the structural system that supports gravity loads. The International Building Code (2006) was used to determine the live loads, which are summarized below in table 8.

Area	Design Load Used by De-Simone Consulting Engineers	IBC 2006 Recommended Code Load
roof	20 psf (reducible)	20 psf
residential area	40 psf (reducible)	40 psf
public area	100 psf (non-reducible)	100 psf (non-reducible)
garage or driveway	50 psf (reducible)	40 psf
corridors - residential	100 psf (non-reducible)	100 psf (non-reducible)
corridors - public area	100 psf (non-reducible)	100 psf (non-reducible)
stairs	100 psf (non-reducible)	100 psf (non-reducible)
office	80 psf (reducible)	50 psf

Table 8 – Comparison of Live Loads

Many of the design live loads are the same, but there is a difference in the parking garage load and the office load. The likely reason for the difference in the parking garage load is the recent change in code. The design engineers used an older code for this project, and the garage live load was reduced from 50 psf to 40 psf a few years ago. For the office area, the engineers likely used a higher load because it is currently an open floor plan and the corridors either are non-existent or their locations are unknown.

The checks were done at level 13, which is a part of the residential section, and the primary code used was ACI 318-08. The column capacity was found to be adequate for the factored gravity load, but the simple post-tensioned slab analysis did not match up with the actual design. The slab thickness was appropriate, but the calculated number of tendons exceeded the number of tendons in the design. (The calculations in Appendix C require 23 tendons, while the design has only 15 tendons.) This inconsistency can be attributed to possible inappropriate assumptions, such as the assumed tendon force of 26.6 k or the assumed target load balance of 65% of the self weight. Also, there could be other components of the slab (such as extra reinforcement bars) that allow for fewer tendons to be used.

More in depth structural calculations will be completed in the third technical report, which will include a detailed description and analysis of the lateral system.

Appendix A – Wind Calculations

The following calculations and spreadsheets were used to provide the summarized wind data in this report. The appendix includes information on all of the intermediate variables used to calculate the wind pressures and story forces.

DETERMINING WIND LOADS

① Determine appropriate design method.
- Does the bldg. meet the conditions of 6.4.1.1?

NO (not a low-rise building, h = 280' > 60')
(cannot use method 1)

- Does the bldg. meet the conditions of 6.5.1 and 6.5.2?

regular shaped building ✓

no unusual wind effects (across wind loading, vortex shedding, channeling effects due to site location, etc.) ✓

USE METHOD 2 - ANALYTICAL PROCEDURE

Design Data
Location: Hato Rey, Puerto Rico (San Juan)
Occupancy Category: II (table 1-1, ASCE 7-05)
Surface Roughness: B (6.5.6.2 — B is assumed because the building is located in an urban area with numerous nearby obstructions)
Building Height: elevation of top of roof parapet = 276'

Horizontal Dimensions: ≈ 119' × 487'
Damping Ratio: assume $\beta = 2\%$
Parapet: neglected because only 13" above roof level

② Calculate velocity pressures q_z and q_h

basic wind speed $V = 145$ mph (fig. 6-1, ASCE7-05)

wind directionality factor $K_d = 0.85$ (table 6-4)

importance factor $I = 1.0$ (table 6-1)

exposure category = B (6.5.6.3 - The building is mostly surrounded by other structures, although it is less than 2 miles from the San Juan Bay.)

topographic factor $K_{zt} = 1.0$ (6.5.7.1 - There are no nearby hills or escarpments. The conditions of 6.5.7.1 are not met.)

velocity pressure exposure coefficient $K_h = 1.32$
(table 6-3 - exposure B, case 2, z = 280', interpolation between 1.28 and 1.35)

velocity pressure exposure coefficient $K_z = 2.01\left(\frac{z}{z_g}\right)^{2/\alpha}$
(table 6-3, note 2 - see chart below for K_z values at various heights - $\alpha = 7.0$, $z_g = 1200'$ from table 6-2)

HEIGHT	K_z	HEIGHT	K_z	HEIGHT	K_z
15'	0.575	120'	1.041	240'	1.269
20'	0.624	130'	1.065	250'	1.284
25'	0.665	140'	1.088	260'	1.298
30'	0.701	150'	1.110	270'	1.313
40'	0.761	160'	1.130	276'	1.321
50'	0.811	170'	1.150		
60'	0.854	180'	1.169		
70'	0.892	190'	1.187		
80'	0.927	200'	1.205		
90'	0.959	210'	1.222		
100'	0.988	220'	1.238		
110'	1.016	230'	1.254		

*The heights used in the spreadsheet were different, but the calcs. are the same.

Determine the velocity pressures at intermediate heights.

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$

$q_{z\,(z=15')} = 0.00256(0.575)(1.0)(0.85)(145)^2(1.0)$
$= 26.3$ psf

(sample calculation for z=15' – all values tabulated in report)

$q_h = 0.00256 K_h K_{zt} K_d V^2 I$
$= 0.00256(1.32)(1.0)(0.85)(145)^2(1.0)$
$= 60.4$ psf

(velocity pressure at mean roof height z=276')

③ Determine gust effect factors, G and G_f

$h = 276'$, assume damping ratio $\beta = 0.02$

$n_1 \cong 100/H = 100/276 = 0.362$ Hz

(eqn. C6-17, section C6.5.8 – equation is an observation from wind tunnel testing as is applicable to steel or concrete buildings less than ~400' in height)

$n_1 < 1.0$ (structure is flexible)

$g_Q = g_v = 3.4$ (given in 6.5.8.2)

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}}$$

$$= \sqrt{2\ln(3600 \times 0.362)} + \frac{0.577}{\sqrt{2\ln(3600\times 0.362)}} = 3.940$$

(equation 6-9, section 6.5.82)

$\bar{z} = 0.6h = 0.6(276') = 166'$

(equivalent height of the structure, $\geq z_{min} = 30'$) ✓OK

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{166}\right)^{1/6} = 0.229$$

(equation 6-5, section 6.5.8 – $c = 0.30$ from table 6-2)

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320'\left(\frac{166}{33}\right)^{1/3.0} = 548.3'$$

(equation 6-7 – $\ell = 320'$, $\bar{\epsilon} = 1/3.0$ from table 6-2)

≈ north-south direction ($B = 119'$) – $$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{119'+276'}{548'}\right)^{0.63}}} = 0.813$$

≈ east-west direction ($B = 487'$) – $$Q = \sqrt{\frac{1}{1+0.63\left(\frac{487'+276'}{548'}\right)^{0.63}}} = 0.750$$

(equation 6-6, where $L_{\bar{z}}$ and h are written above – B = horizontal dimension of building measured normal to wind direction, as defined in section 6.3) approximate horizontal building dimensions:

(approximate wind directions – actual 'north' is slightly NE of drawn direction)

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 0.45\left(\frac{166}{33}\right)^{1/4.0}(145)\left(\frac{88}{60}\right) = 143.3 \text{ ft/s}$$

(equation 6-14, mean hourly wind speed – $\bar{z}$ and V from before, $\bar{b}$ and $\bar{\alpha}$ from table 6-2)

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.362)(548)}{(143)} = 1.387$$

(equation 6-12)

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(1.387)}{(1+10.3\times1.387)^{5/3}} = 0.110$$

(equation 6-11)

$$\eta = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.362)(276)}{143} = 3.214$$

(section 6.5.8.2, defined when $R_\ell = R_h$)

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) = \frac{1}{3.214} - \frac{1}{2(3.214)^2}\left(1-e^{-2(3.214)}\right) = 0.263$$

(eqn. 6-13a, applied 4 more times below with different subscripts)

≈ north-south direction (B = 119', L = 487') – $\eta = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \frac{4.6(0.362)(119)}{143} = 1.386$

(section 6.5.8.2, defined when $R_\ell = R_B$)

$$- R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) = \frac{1}{1.386} - \frac{1}{2(1.386)^2}\left(1-e^{-2(1.386)}\right)$$

$$= 0.477$$

≈ north-south direction $\left(\begin{matrix}B=119'\\L=487'\end{matrix}\right)$ – $\eta = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \frac{15.4(0.362)(487)}{143} = 18.99$

(section 6.5.8.2, defined when $R_\ell = R_L$)

– $R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{18.99} - \frac{1}{2(18.99)^2}(1-e^{-2(18.99)})$

$= 0.0513$

≈ east-west direction $\left(\begin{matrix}B=487'\\L=119'\end{matrix}\right)$ – $\eta = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \frac{4.6(0.362)(487)}{143} = 5.67$

(section 6.5.8.2, defined when $R_\ell = R_B$)

– $R_B = \frac{1}{5.67} - \frac{1}{2(5.67)^2}(1-e^{-2(5.67)}) = 0.161$

– $\eta = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \frac{15.4(0.362)(119)}{143} = 4.64$

(section 6.5.8.2, defined when $R_\ell = R_L$)

– $R_L = \frac{1}{4.64} - \frac{1}{2(4.64)^2}(1-e^{-2(4.64)}) = 0.192$

≈ north-south direction – $R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$ (eqn. 6-10)

$= \sqrt{\frac{1}{0.02}(0.110)(0.263)(0.477)(0.53+0.47(0.0513))}$

$= 0.618$

– $G_f = 0.925\left[\frac{1+1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_{\bar{z}}}\right]$ (eqn. 6-8)

$= 0.925\left[\frac{1+1.7(0.229)\sqrt{(3.4)^2(0.813)^2+(3.94)^2(0.618)^2}}{1+1.7(3.4)(0.229)}\right]$

$= 0.969$

≈ east-west direction – $R = \sqrt{\frac{1}{0.02}(0.110)(0.263)(0.161)(0.53+0.47(0.192))}$

$= 0.380$

– $G_f = 0.925\left[\frac{1+1.7(0.229)\sqrt{(3.4)^2(0.75)^2+(3.94)^2(0.380)^2}}{1+1.7(3.4)(0.229)}\right]$

$= 0.856$

④ Determine the design wind pressures.

≈ north-south direction ($B = 119'$, $L = 487'$)

pressure coefficients, C_p

$L/B = 487/119 = 4.1$

windward wall — $C_p = 0.8$

leeward wall — $C_p = -0.2$

≈ east-west direction ($B = 487'$, $L = 119'$)

pressure coefficients, C_p

$L/B = 119/487 = 0.24$

windward wall — $C_p = 0.8$

leeward wall — $C_p = -0.5$

(figure 6-6)

internal pressure coefficient, GC_{pi}
- partially enclosed, $GC_{pi} = \pm 0.55$
- enclosed, $GC_{pi} = \pm 0.18$

(figure 6-5)

$$P_{windward} = q_z G_f C_p - \underbrace{\cancel{q_h (GC_{pi})}}_{\text{neglected}}$$

$$P_{leeward} = q_h G_f C_p - \underbrace{\cancel{q_h (GC_{pi})}}_{\text{neglected}}$$

* see spreadsheet for results. The pressures were determined by:
  - N-S or E-W direction
  - windward or leeward pressure

* the second term in the equation was neglected because the internal pressure/suction cancels out when looking at the windward and leeward walls together.

SUMMARY OF WIND PRESSURES

Height above ground level, z or h (ft)	K_z or K_h	q_z or q_h (psf)	North-South Direction			East-West Direction		
			windward pressure (psf)	leeward suction (psf)	total wind pressure (psf)	windward pressure (psf)	leeward suction (psf)	total wind pressure (psf)
15	0.575	26.29	20.38	-11.71	32.10	18.01	-25.87	43.89
20	0.624	28.55	22.13	-11.71	33.84	19.56	-25.87	45.43
25	0.665	30.43	23.59	-11.71	35.30	20.84	-25.87	46.72
30	0.701	32.05	24.85	-11.71	36.56	21.96	-25.87	47.83
40	0.761	34.80	26.98	-11.71	38.69	23.84	-25.87	49.71
50	0.811	37.09	28.75	-11.71	40.47	25.41	-25.87	51.28
60	0.854	39.07	30.29	-11.71	42.00	26.77	-25.87	52.64
67	0.881	40.32	31.26	-11.71	42.97	27.63	-25.87	53.50
67	0.881	40.32	31.26	-11.71	42.97	27.63	-25.87	53.50
70	0.892	40.83	31.66	-11.71	43.37	27.97	-25.87	53.85
80	0.927	42.42	32.89	-11.71	44.60	29.06	-25.87	54.94
90	0.959	43.87	34.01	-11.71	45.72	30.06	-25.87	55.93
100	0.988	45.21	35.05	-11.71	46.76	30.97	-25.87	56.85
120	1.041	47.63	36.93	-11.71	48.64	32.63	-25.87	58.50
140	1.088	49.77	38.59	-11.71	50.30	34.10	-25.87	59.97
160	1.130	51.71	40.09	-11.71	51.80	35.43	-25.87	61.30
180	1.169	53.48	41.46	-11.71	53.17	36.64	-25.87	62.51
200	1.205	55.11	42.73	-11.71	54.44	37.76	-25.87	63.63
250	1.284	58.74	45.54	-11.71	57.25	40.24	-25.87	66.12
276	1.321	60.43	46.85	-11.71	58.56	41.40	-25.87	67.27

* up to level 7 (parking structure)
* above level 7 (residential towers)

APPLICATION OF WIND LOADS — Story Forces

North-South Direction								
Level	Floor Elevation (ft)	Elevations for Tributary Height of Each Level (ft)		Tributary Width (ft)	Total Wind Pressure, windward + leeward (psf)	Story Force (k)	Total Story Shear (k)	Overturning Moment Contribution from Each Level (ft-k)
2	10.833	0.000	15.000	119.333	32.09	57.44	1230.46	622.28
3	19.167	15.000	23.333	119.333	33.84 35.30	34.23	1173.02	656.13
4	27.500	23.333	31.667	119.333	35.30 36.56 38.69	36.53	1138.79	1004.57
5	35.833	31.667	40.000	119.333	38.69	38.48	1102.26	1378.69
6	44.167	40.000	50.417	119.333	40.47 42.00	50.38	1063.78	2225.23
7	56.667	50.417	61.646	119.333	42.00 43.37	56.55	1013.40	3204.48
8	66.625	61.646	71.104	119.333	43.37 44.60	49.11	956.85	3272.19
9	75.583	71.104	80.063	83.500	44.60 45.72	33.37	907.73	2522.03
10	84.542	80.063	89.021	83.500	45.72	34.20	874.37	2891.28
11	93.500	89.021	97.979	83.500	45.72 46.76	34.89	840.17	3262.44
12	102.458	97.979	106.938	83.500	46.76 48.64	36.07	36.07	3695.31
13	111.417	106.938	115.896	83.500	48.64	36.38	769.21	4053.75
14	120.375	115.896	124.854	83.500	48.64 50.30	37.06	732.82	4460.68
15	129.333	124.854	133.813	83.500	50.30	37.63	695.77	4866.22
16	138.292	133.813	142.771	83.500	50.30 51.80	37.97	658.14	5251.28
17	147.250	142.771	151.729	83.500	51.80	38.75	620.17	5705.57
18	156.208	151.729	160.688	83.500	51.80 53.17	38.83	581.42	6064.96
19	165.167	160.688	169.646	83.500	53.17	39.77	542.60	6569.05
20	174.125	169.646	178.604	83.500	53.17	39.77	502.82	6925.35
21	183.083	178.604	187.563	83.500	53.17 54.44	40.57	463.05	7428.47
22	192.042	187.563	196.521	83.500	54.44	40.72	422.48	7820.37
23	201.000	196.521	205.479	83.500	54.44 57.25	42.01	381.76	8443.58
24	209.958	205.479	214.438	83.500	57.25	42.82	339.75	8991.30
25	218.917	214.438	223.396	83.500	57.25	42.82	296.92	9374.93
26	227.875	223.396	232.354	83.500	57.25	42.82	254.10	9758.56
27	236.833	232.354	241.313	83.500	57.25	42.82	211.28	10142.20
28	245.792	241.313	250.271	89.000	57.25 58.56	45.68	168.45	11226.91
29	254.750	250.271	259.229	89.000	58.56	46.69	122.77	11894.12
roof level	263.708	259.229	269.333	89.000	58.56	52.66	76.09	13887.22
sky lobby roof level	274.958	269.333	276.000	60.000	58.56	23.42	23.42	6440.62

Total Overturning Moment (ft-k) = 174039.79

APPLICATION OF WIND LOADS – Story Forces

East-West Direction

Level	Floor Elevation (ft)	Elevations for Tributary Height of Each Level (ft)		Tributary Width (ft)	Total Wind Pressure, windward + leeward (psf)	Story Force (k)	Total Story Shear (k)	Overturning Moment Contribution from Each Level (ft-k)
2	10.833	0.000	15.000	486.917	43.89	320.56	6104.38	3472.75
3	19.167	15.000	23.333	486.917	45.43 46.72	186.43	5783.82	3573.29
4	27.500	23.333	31.667	486.917	46.72 47.83 49.71	194.70	5597.39	5354.30
5	35.833	31.667	40.000	486.917	49.71	201.71	5402.68	7227.77
6	44.167	40.000	50.417	486.917	51.28 52.64	260.37	5200.98	11499.70
7	56.667	50.417	61.646	486.917	52.64 53.85	288.79	4940.61	16364.64
8	66.625	61.646	71.104	486.917	53.85 54.94	248.59	4651.82	16562.17
9	75.583	71.104	80.063	348.000	54.94 55.93	171.30	4403.23	12947.20
10	84.542	80.063	89.021	348.000	55.93	174.36	4231.94	14740.84
11	93.500	89.021	97.979	348.000	55.93 56.85	176.92	4057.57	16541.68
12	102.458	97.979	106.938	348.000	56.85 58.50	181.21	181.21	18566.82
13	111.417	106.938	115.896	348.000	58.50	182.37	3699.44	20319.48
14	120.375	115.896	124.854	348.000	58.50 59.97	184.86	3517.07	22252.16
15	129.333	124.854	133.813	348.000	59.97	186.96	3332.21	24179.70
16	138.292	133.813	142.771	348.000	59.97 61.30	188.24	3145.26	26031.88
17	147.250	142.771	151.729	348.000	61.30	191.10	2957.02	28139.88
18	156.208	151.729	160.688	348.000	61.30 62.51	191.39	2765.92	29897.06
19	165.167	160.688	169.646	348.000	62.51	194.87	2574.52	32186.84
20	174.125	169.646	178.604	348.000	62.51	194.87	2379.65	33932.60
21	183.083	178.604	187.563	348.000	62.51 63.63	197.82	2184.77	36218.00
22	192.042	187.563	196.521	348.000	63.63	198.37	1986.95	38094.64
23	201.000	196.521	205.479	348.000	63.63 66.12	203.11	1788.58	40825.98
24	209.958	205.479	214.438	348.000	66.12	206.13	1585.47	43278.52
25	218.917	214.438	223.396	348.000	66.12	206.13	1379.34	45125.10
26	227.875	223.396	232.354	348.000	66.12	206.13	1173.21	46971.67
27	236.833	232.354	241.313	348.000	66.12	206.13	967.08	48818.24
28	245.792	241.313	250.271	357.000	66.12 67.27	211.57	760.95	52002.44
29	254.750	250.271	259.229	373.167	67.27	224.88	549.38	57288.27
roof level	263.708	259.229	269.333	373.167	67.27	253.64	324.50	66888.06
sky lobby roof level	274.958	269.333	276.000	158.000	67.27	70.86	70.86	19482.92

Total Overturning Moment (ft-k) = 838784.58

Appendix B – Seismic Calculations

The following calculations and spreadsheets were used to provide the summarized seismic data in this report. The appendix includes information on all of the intermediate variables used to calculate the building weight, base shear, and story forces.

DETERMINING SEISMIC LOADS

Design Data

Location: Hato Rey, Puerto Rico (San Juan)
- latitude = 18.4307°
- longitude = −66.0791°

(www.satsig.net/maps/lat-long-finder.htm)

Site Class: B

Spectral response accelerations:
- $S_S = 0.882g$ ($T = 0.2$ s)
- $S_1 = 0.301g$ ($T = 1.0$ s)
- $S_{MS} = 0.882g$ ($T = 0.2$ s)
- $S_{M1} = 0.301g$ ($T = 1.0$ s)
- $S_{DS} = 0.588g$ ($T = 0.2$ s)
- $S_{D1} = 0.201g$ ($T = 1.0$ s)

(Java Ground Motion parameter calculator from earthquake.usgs.gov/research/hazmaps/design/index.php)

Seismic Design Category: D (table 11.6-1, occupancy category = II)

Response modification coefficient: $R = 5.5$
(building frame system with reinforced concrete shear walls)

Importance Factor: $I = 1.0$ (table 11.5-1)

Approximate fundamental period:
- $T_a = C_t h_n^x = 0.016(276)^{0.9} = 2.517$ s
(eqn. 12.8-7, C_t and x from table 12.8-2)
- use T_a value for T, because there's no computer analysis to compare with

Long-Period Transition Period:
- $T_L = 12$ s (figure 22-19)

Seismic Response Coefficient:
- $C_s \le S_{DS}/(R/I) = 0.588/\left(\frac{5.5}{1.0}\right) = 0.107$ (eqn. 12.8-2)

$\le S_{D1}/T(R/I) = \frac{0.201}{2.517\left(\frac{5.5}{1.0}\right)} = 0.0145$ (eqn. 12.8-3)

DETERMINATION OF BUILDING WEIGHT BY FLOOR

Level 2 (h = 10'-10", Area = 51950 ft^2)

Element	Dimensions	Load (k)	Load (psf)
concrete slab	9"	-	112.5
exterior walls	glass curtain wall (15 psf), length=160'	26.00 k	0.5
	8" CMU (55 psf), length=158'	94.14 k	1.8
	open façade with 8'-0" high concrete panel (75 psf), length=805'	483.0 k	9.3
columns	(10) 18x24	48.75 k	0.9
	(4) 24" Φ	20.42 k	0.4
	(2) 12x48	13.00 k	0.3
	(38) 18x24	185.2 k	3.6
	(18) 18x48	175.5 k	3.4
	(12) 18x60	146.2 k	2.8
	(7) 32" Φ	63.53 k	1.2
	(12) 12x48	78.00 k	1.5
beams	(negligible)		
shear walls	t=18", length=711'	1733 k	33.4
superimposed	(MEP and miscellaneous)	-	15

Total Load (psf) = 186.5

Levels 3-6 (h = 8'-4", Area = 51950 ft^2)

Element	Dimensions	Load (k)	Load (psf)
concrete slab	9"	-	112.5
exterior walls	glass curtain wall (15 psf), length=160'	20.00 k	0.4
	8" CMU (55 psf), length=158'	72.42 k	1.4
	open façade with 4'-9" high concrete panel (75 psf), length=805'	286.8 k	5.5
columns	(10) 18x24	37.50 k	0.7
	(4) 24" Φ	15.71 k	0.3
	(2) 12x48	10.00 k	0.2
	(38) 18x24	142.5 k	2.7
	(18) 18x48	135.0 k	2.6
	(12) 18x60	112.5 k	2.2
	(7) 32" Φ	48.87 k	0.9
	(12) 12x48	60.00 k	1.2
shear walls	t=18", length=586'	1099 k	21.2
superimposed	(MEP and miscellaneous)	-	15

Total Load (psf) = 166.8

DETERMINATION OF BUILDING WEIGHT BY FLOOR

Level 7 (h = 12'-6", Area = 51950 ft^2)

Element	Dimensions	Load (k)	Load (psf)
concrete slab	12"	-	150
exterior walls	glass curtain wall (15 psf), length=160'	30.00 k	0.6
	8" CMU (55 psf), length=158'	108.6 k	2.1
	open façade with 5'-9" high concrete panel (75 psf), length=805'	347.2 k	6.7
columns	(10) 18x24	56.25 k	1.1
	(4) 24" Φ	23.56 k	0.5
	(2) 12x48	15.00 k	0.3
	(38) 18x24	213.8 k	4.1
	(18) 18x48	202.5 k	3.9
	(12) 18x60	168.8 k	3.2
	(7) 32" Φ	73.30 k	1.4
	(12) 12x48	90.00 k	1.7
beams	(estimate)	-	4
shear walls	t=18", length=586'	1648 k	31.7
superimposed	(MEP and miscellaneous)	-	20

Total Load (psf) = 231.3

* Note: The concrete slab is 12" in some areas and 9" in other areas. The estimate of 12" everywhere is conservative. Also, the superimposed dead load is estimated as being higher than on other floors. These overestimates help to account for the potential loads from landscaping and heavy floor finishes.

Levels 8-17 (h = 11'-0", Area = 26150 ft^2)

Element	Dimensions	Load (k)	Load (psf)
concrete slab	8"	-	100
exterior walls	8" precast concrete panels (75 psf), perimeter=1030'	849.8 k	32.5
columns	(21) 18x48	207.9 k	8.0
beams	(4) 18x18, length =83'	112.1 k	4.3
shear walls	t=18", length=284'	702.9 k	26.9
partitions	(general allowance)	-	10
superimposed	(MEP and miscellaneous)	-	15

Total Load (psf) = 196.6

DETERMINATION OF BUILDING WEIGHT BY FLOOR

Levels 18-27 (h = 11'-0", Area = 26150 ft^2)

Element	Dimensions	Load (k)	Load (psf)
concrete slab	8"	-	100
exterior walls	8" precast concrete panels (75 psf), perimeter=1030'	849.8 k	32.5
columns	(21) 18x36	155.9 k	6.0
beams	(4) 18x18, length =83'	112.1 k	4.3
shear walls	t=18", length=284'	702.9 k	26.9
partitions	(general allowance)	-	10
superimposed	(MEP and miscellaneous)	-	15

Total Load (psf) = 194.6

Levels 28-29 (h = 11'-0", Area = 28700 ft^2)

Element	Dimensions	Load (k)	Load (psf)
concrete slab	8"	-	100
exterior walls	8" precast concrete panels (75 psf), perimeter=1070'	882.8 k	30.76
columns	(21) 18x36	155.9 k	5.4
beams	(4) 18x18, length=89'	120.2 k	4.2
shear walls	t=18", length=284'	702.9 k	24.5
partitions	(general allowance)	-	10
superimposed	(MEP and miscellaneous)	-	15

Total Load (psf) = 189.9

DETERMINATION OF BUILDING WEIGHT BY FLOOR

Roof Level (h = 9'-0", Area = 28700 ft^2)			
Element	Dimensions	Load (k)	Load (psf)
concrete slab	6" (assumed)	-	75
bitumen roofing	2-ply	-	5
rigid insulation	2"	-	3
exterior walls	8" precast concrete panels (75 psf), perimeter=1070'	722.3 k	25.2
columns	(21) 18x36	127.6 k	4.4
beams	(2) 18x18, length=88'	59.40 k	2.1
shear walls	t=18", length=284'	575.1 k	20.0
partitions	(general allowance)	-	10
superimposed	(MEP and miscellaneous)	-	15

Total Load (psf) = 159.7

Sky Lobby Roof Level (h = 11'-3", Area = 9480 ft^2)			
Element	Dimensions	Load (k)	Load (psf)
concrete slab	6" (assumed)	-	75
bitumen roofing	2-ply	-	5
rigid insulation	2"	-	3
exterior walls	glass (15 psf), perimeter=556'	93.83 k	9.9
columns	(12) 18x36	91.13 k	9.6
partitions	(general allowance)	-	10
superimposed	(MEP and miscellaneous)	-	15

Total Load (psf) = 127.5

CALCULATION OF TOTAL BUILDING WEIGHT AND BASE SHEAR

Level(s)	Load (psf)	Area (ft^2)	Weight per Floor (k)
2	186.5	51950	9689
3 to 6	166.8	51950	8665
7	231.3	51950	12016
8 to 17	196.6	26150	5141
18 to 27	194.6	26150	5089
28 to 29	189.9	28700	5450
Roof	159.7	28700	4583
Sky Lobby Roof	127.5	9480	1209

Total Building Weight, W (k) =	175357
C_s =	0.0145
Base Shear, V (k) =	2543

Summary from spreadsheet:

$W = 175{,}400$ k

$C_s = 0.0145$

$V = C_s W = 2543$ k

Exponent related to structure period $k = 2$ (12.8.3)

Determine lateral seismic force F_x at each level x:
(section 12.8.3)

(See attached spreadsheet for summary of calculations at each floor)

$$F_x = \left(\frac{w_x h_x^k}{\sum w_i h_i^k}\right) V$$

Determine the seismic design story shear V_x at each level:

(eqn. 12.8-13)

$$V_x = \sum_{i=x}^{n} F_i$$

(The story shear increases toward the base. Each story force is added from the top-down. see the attached spreadsheet.)

Calculate the overturning moment:

(see spreadsheet.)

CALCULATION OF LATERAL SEISMIC FORCES AND OVERTURNING MOMENT

Level	Weight, w_x (k)	Height, h_x (ft)	k	$w_x \cdot h_x^k$	Lateral Seismic Force, F_x (k)	Seismic Design Story Shear, V_x (k)	Overturning Moment from Each Level (ft-k)
2	9689	10.833	2	1137112	0.8	2543.0	8
3	8665	19.167	2	3183184	2.1	2542.2	41
4	8665	27.500	2	6552906	4.4	2540.1	121
5	8665	35.833	2	11125894	7.5	2535.7	267
6	8665	44.167	2	16903022	11.3	2528.2	501
7	12016	56.667	2	38585165	25.9	2516.9	1466
8	5141	66.625	2	22820337	15.3	2491.0	1020
9	5141	75.583	2	29369453	19.7	2475.7	1489
10	5141	84.542	2	36744525	24.6	2456.0	2083
11	5141	93.500	2	44943907	30.1	2431.4	2818
12	5141	102.458	2	53968376	36.2	2401.2	3709
13	5141	111.417	2	63818696	42.8	2365.0	4769
14	5141	120.375	2	74493813	50.0	2322.2	6014
15	5141	129.333	2	85993635	57.7	2272.3	7459
16	5141	138.292	2	98319966	65.9	2214.6	9119
17	5141	147.250	2	111470054	74.8	2148.7	11009
18	5089	156.208	2	124176380	83.3	2073.9	13010
19	5089	165.167	2	138828622	93.1	1990.6	15379
20	5089	174.125	2	154296015	103.5	1897.5	18019
21	5089	183.083	2	170580150	114.4	1794.0	20946
22	5089	192.042	2	187682980	125.9	1679.6	24174
23	5089	201.000	2	205600689	137.9	1553.7	27717
24	5089	209.958	2	224335851	150.5	1415.8	31591
25	5089	218.917	2	243887816	163.6	1265.4	35809
26	5089	227.875	2	264256583	177.2	1101.8	40388
27	5089	236.833	2	285442151	191.4	924.6	45341
28	5450	245.792	2	329254705	220.8	733.1	54278
29	5450	254.750	2	353691716	237.2	512.3	60432
Roof	4583	263.708	2	318710570	213.8	275.1	56370
Sky Lobby	1209	274.958	2	91402699	61.3	61.3	16856

$\sum w_i \cdot h_i^k =$ 3791576971

Total Overturning Moment (ft-k) = 512204

Appendix C – 'Spot Checks'

The following calculations were used to check the existing design of various structural members. The analysis was done based on gravity loads only for a typical interior column and a section of a post-tensioned slab.

SPOT CHECKS

(1) column at level 13, column line 8-D
(check based on axial-gravity-loads only)

Tributary area = 30'×19' = 570 ft²
Live loads = 40 psf (apartments, 16 floors)
= 100 psf (sky lobby, 1 floor)
= 20 psf (roof)
IBC 2006 table 1607.1

live load element factor, $K_{LL} = 4$
(interior column, IBC 2006 table 1607.9.1)

*The apartment live loads and roof load can be reduced ($K_{LL}A_T \geq 400$ ft²) but the live load in the lobby cannot be reduced (≥ 100 psf)
(section 1607.9.1)

Reduced apartment live load, $L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$ (eqn. 16-24

$$= 40\left(0.25 + \frac{15}{\sqrt{4 \times 570}}\right)$$

$$= 0.564\,L_o$$

$$= 22.6 \text{ psf}$$

Reduced roof live load, $L = 11.3$ psf

Dead loads (apartments and lobby):

8" slab	100 psf
ceiling	1 psf
floor finishes	3 psf
partitions	10 psf
MEP	5 psf
miscellaneous	5 psf
	124 psf

Dead loads (roof):

6" slab	75 psf
insulation	3 psf
2-ply bitumen roofing	5 psf
ceiling	1 psf
miscellaneous	5 psf
	≈90 psf

Area	# floors	live load (psf)	dead load (psf)
apartments	16	22.6	124
lobby	1	100	124
roof	1	11.3	90

The self weight of the column must also be included (changes from an 18×36 to an 18×48)

$$\left(\frac{18'' \times 36'' \times 128'}{12^2}\right)\left(\frac{150\ pcf}{1000}\right) = 86.4\ k$$

$$\left(\frac{18'' \times 48'' \times 36'}{12^2}\right)\left(\frac{150\ pcf}{1000}\right) = 32.4\ k$$

Critical load combinations:

$$1.4D = 1.4\left[86.4\ k + 32.4\ k + \frac{570\ ft^2(16 \times 124 + 124 + 90)}{1000}\right]$$

$$= 1.4\,(1372\ k)$$

$$= 1920\ k$$

$$1.2D + 1.6L = 1.2\,(1372\ k) + 1.6\left[\frac{570\ ft^2(16 \times 22.6 + 100 + 11.3)}{1000}\right]$$

$$= 2080\ k$$

$$\boxed{P_u = 2080\ k}$$

Actual design:

(12) #8 ($A_{st} = 9.5\ in^2$)
tie reinforcement

$$\phi P_{n,max} = 0.80\phi\left[0.85 f'_c (A_g - A_{st}) + f_y A_{st}\right]$$

(eqn. 10-2 ACI 318-08)

- $f_y = 60\ ksi$
- $f'_c = 6\ ksi$

$$\phi P_{n,max} = 0.80(0.65)\left[(0.85)(6)(18'' \times 48'' - 9.5) + 60(9.5)\right]$$

$$\boxed{\phi P_{n,max} = 2560\ k}$$

$$\boxed{P_u = 2080\ k < \phi P_n = 2560\ k}$$ ✓ OK

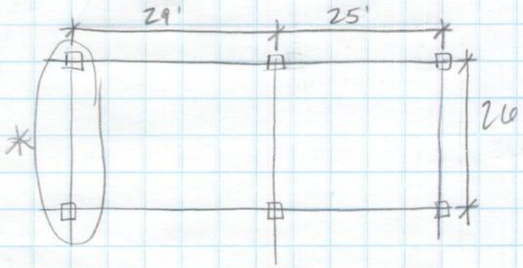
② post-tension slab, 13^{th} floor (apartments)

live load = 40 psf

reduced live load $L = 40\left(0.25 + \frac{15}{\sqrt{1(25\times 26)}}\right)$
$= 33.5$ psf

$K_{LL} = 1$ for 2 way slab (IBC 1607)

dead load $D = 124$ psf (same as load for column check)



partial floor slab

materials:
- normal weight concrete $f'_c = 5$ ksi
- rebar $f_y = 60$ ksi

max. span = 29'-0"

*Note: There are some column-to-column spans that are 34'-0", but the reinforced concrete shear walls are located along adjacent column lines to carry some of the gravity load.

Determine preliminary slab thickness:

$t = L/45$, where L = max span
$= 29\times 12/45$
$= 7.73''$

$\boxed{t = 8''}$ (preliminary thickness) ✓ This is the same thickness as the existing design.

two-way slab – class U (ACI 18.3.3)

analysis of column strip – 26' span

– $A = bh = (26' \times 12)(8'') = 2500 \text{ in}^2$

– $S = \frac{bh^2}{6} = \frac{(26' \times 12)(8)^2}{6} = 3330 \text{ in}^3$

allowable stresses for class U:

– at time of jacking (ACI 18.4.1)

- $f'_{ci} = 3000$ psi
- compression $= 0.60 f'_{ci} = 0.6(3000) = 1800$ psi
- tension $= 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164$ psi

– at service loads (ACI 18.4.2(a))

- $f'_c = 5000$ psi
- compression $= 0.45 f'_c = 0.45(5000) = 2250$ psi
- tension $= 6\sqrt{f'_c} = 6\sqrt{5000} = 424$ psi

Assume the target load balance is 65% of the structure self-weight

$0.65 w_{DL} = 0.65(124) = 81$ psf

Calculate a_{end} for the 8" slab.

$$a_{end} = \frac{(4.0'' + 8.0'')}{2} - 1.75'' = 4.25''$$

Assume the end span governs for the max required post-tensioning force.

$$W_b = 0.65 w_{DL} = (81 \text{ psf})(26') = 2100 \text{ plf}$$

Calculate the force needed in the tendons

$$P = \frac{w_b L^2}{8 a_{end}} = \frac{(2100 \text{ plf})(29')^2}{8\left(\frac{4.25}{12}\right)1000} = 623 \text{ k}$$

Assume the allowable force per tendon is 26.6 k

$$\# \text{ tendons} = \frac{623 \text{ k}}{26.6 \text{ k/tendon}} = 23.4 \longrightarrow \text{use 23 tendons}$$

Determine the actual force for the # of tendons

$$P_{actual} = (23 \text{ tendons})(26.6 \tfrac{k}{tendon}) = 612\text{ k}$$

Adjust w_b for the actual force

$$w_b = \frac{612}{623}(2100) = 2060 \text{ plf}$$

Determine the precompression stress

$$\frac{P_{actual}}{A} = \frac{612\text{ k}(1000)}{2500 \text{ in}^2} = 245 \text{ psi} > 125 \text{ psi (min, 18.12.4)}$$

* My calculations indicate that 23 tendons are required across those 2 columns, but the drawings only show a design of 15 tendons. I made some assumptions that may have been incorrect, such as the tendon force, target load balance, etc. Plus, this is my first attempt at a post-tensioned slab design, so there may be other inaccurate assumptions. Also, there could be some other component of the slab that makes up for the lack of tendons