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Structural Option
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Technical Report #1

<u>Paseo Caribe Condominium Tower and Parking Garage</u>

Condado, Puerto Rico

Executive Summary

This technical assignment examines the existing conditions of Paseo Caribe Condominium Tower and Parking Garage. This structure is a 14 story cast in place concrete apartment building that sits on top of 10 story parking garage. The building is located in the northern coastline of Puerto Rico making it both a severe hurricane prone region and a high seismic zone. Due to the high price and delay of importing steel to the island, most of the architecture in the island is made out of concrete. This building takes the strength of concrete on more level by incorporating a post tensioned slab system to keep the slab thickness to a minimum, increasing floor to ceiling heights. At the same time this system allowed for large bays that are unobstructed. There are no beams or columns in the apartment layouts. This was a determining factor of the design because the condominium is located in the most expensive location of Puerto Rico's every growing tourist sector.

Due to this and other factors, the building structural design has several factors that point towards a complex analysis and design. For this preliminary analysis a simplified model of the building was developed throughout the report and is the base of the lateral and gravity analysis. One important aspect that is kept in mind through out the report but that is not directly integrated is the effect of the shape that building has on the increased stresses experienced by the lateral resisting system. Instead of behaving like a square building, like the model that has used to calculate the seismic and wind loads, the building behaves more like an H-shaped building formed by the interception of 3 rectangular planes in a central point. This shape allows for more window, but low torsion restrain.

Structurally, the building both laterally and vertically supported by two dozen concrete walls that span around the perimeter of a very thick concrete core, 10' by 162' that span the middle of the building. The framing lattices of the concrete shear and bearing walls form a building frame that work together to decrease story drift deflections and increase the torsion rigidity of the building. However, without considering those two frames, East-West & North-South, acting together the building is not as effective in lateral distribution of the loads and a computer program would be best to model its behavior.

Similarly, it is still unknown whether this structural slab was design as a one-way or two way systems. It is believed that it was originally designed to be two ways but because of the arrangement of consecutive parallel shear walls spanning next to each other to take the high lateral loads, the bay sizes end up having length to width ratios very close to or equal to two. The slab then behaves more like a one-way system with the few tendons running the transverse direction helping with slab deflections and cracking.

This report does a good attempt at layout down a simplified method of analyzing the building structure. Emphasis is given primarily to lateral resisting system. This loads are much critical than the loads of just a gravity analysis.

Part 1: Overall Structural Systems

A) General Floor Framing

The structure of the Condominium and Parking Garage is reinforced cast in place concrete. The general footprint is best described by 3 intercepting rectangles. Two of these rectangles run parallel, spanning East – West (E-W) roughly 30' apart and are 162' by 45' in dimension. The third rectangle runs North – South (N-S) and intersects the other two in the middle. It is 120' by 54' and contains the 10' wide core strip at the centerline. (Refer to Figure 1 through 4)

The floor system consists of a one way cast in place post tensioned 8" concrete slab on each floor. The floor slab in supported by the 24 staggered interior shear walls, typically 1' thick, that offset from the 10' wide central core and by 16 columns around the perimeter. The concrete columns extend from the foundation through the parking garage and range in sizes from 2' x 2' and 2' x 3'. The shear walls run parallel to each other. The largest interior span in between shear walls is 27', other interior spans are 22' and 14'. The largest exterior span between shear walls is 22'.

In the parking garage, the 8" slab is supported by columns. Columns are typically 24" x 30", with increased sizes to 24"x 56" around vehicle ramps. Spacing is 27' c/c on the E-W direction and 15' c/c on the N-S direction, skipping one every two columns. (Refer to Figure 4) Typical reinforcement for columns is 16#11.

B) Structural Slabs

The structural slab system for this building is a one way post-tensioned cast in place concrete slab. The slab is 8" thick. The slab is designed for a compressive stress of 12k/ft in both directions. This design value is increased to low 20's k/ft around the exterior and mid-span of the core. These areas experience increased loading due to location of the open terraces and core torsion stiffness. Post-tensioning tendons for this slab are 7 wire. There is post-tensioning of the concrete on both directions, N-S and E-W. However, the primary action of this one way slab spans from East to West, the short direction between shear wall supports. The tendons in this direction are placed every 14' in the interior and doubled to 7' apart on the edge of the slab and around the central core. In the transverse N-S direction, the tendons are located directly over the shear walls and are used to provide better compression capabilities and lateral stability “tie” for the shear walls. (Refer to Figure 6)

The structural slab is also reinforced in the E-W short span direction with reinforcing bars. The typical bottom reinforcement is #5 bars. Typically:

- Spans < 15’	#5@18”
- 15’-22’ Spans	#5@16”
- Spans > 22’	#5@14”
- Middle core	#5@12”
- North-South core perimeter	#5@10”

Top positive reinforcement is provided over the shear wall supports. Reinforcement extends 2/5 times the span direction on each side from the support. Typical reinforcement is #5 bars. For spans < 17’, use #5@18”. Larger spans use #5 @ 12.

C) <u>Lateral Resisting System</u>

The Condominium Tower raises 14 floors above the parking garage. The parking garage itself comprises 8 stories above grade and two below grade. This totals 22 stories above grade level. The typical story height is 9’-10” for the apartment stories, 10’ at the parking levels. The total story height of the building is 243’.

The primary lateral resisting system is formed by shear wall action of a central core and parallel offsets of that core. (Refer to Figure 4) The core is located in the center line of the building. It is 10’ wide and spans a total of 120’ N-S. It is comprised of 10 - 1’ wide wall that enclose 2 elevator cores and 3 sets of stairs along that line. There are 16 other shear wall (#1 - #16 in Figure 4) that are offsets and parallel to the core, 8 on each side. These walls are also 1’ wide, with the exception of the middle 4 shear walls (#13-#16, Figure 4) that are 10” wide.

In the opposite direction at the middle of the building, through line N’ and S’ (Refer to Figure 2), there are 4 shear walls on each side, each 1’-2” wide, that act as the lateral resisting system in the E-W direction (#17-#24, Figure 4). There are four smaller 6” walls on each side to reinforce this E-W lateral system.

All shear walls extend from the foundation and parking garage directly through the apartment building. There are some slight changes and modifications that were done to add stiffness while accommodating for the apartment's layout. Brief mentions of this for a typical apartment floor[1] are (Refer to Figure 4):

- The 2 stair enclosures that extended through the 8 levels of parking lots and form part of the core are shifted at the lobby level 30' each inward toward the center of the building. A 3rd set of stairs was added along the core line and covers the space in-between the two elevator shafts. These changes allowed for better use of the middle core space and increased stiffness at the core.
- Shear walls #15-#16 are extended 8' south over the original wall.
- Shear walls #11-#12 are extended 13' south over the original wall.
- Shear wall #21-#24 extended 8' inward over original wall.
- Column #7 is offset 2' south.

D) Foundation System

The parking garage was designed first with the notion that a condominium was to be built a later time on top of it. This is evident in the layout of the foundation system. The foundation design consists of 40 to 50 inch deep pile caps. The typical pile cap consists of 10 piles placed 3' c/c. The layout of the foundation system is in a grid following that explained for the column layout of the parking garage. Typical spacing is 15' c/c north-south and 27' c/c east-west. The building is enclosed below grade by a 2' wide L retaining slurry wall around the perimeter that goes to a maximum depth of 22' with a 2' hydrostatic slab on grade. The location of the tower is evident by replacement of columns with shear walls in the west half of the parking garage foundation layout. This foundation shear walls extend from one pile cap mat to the next. As a result of this increased load that the shear walls will be experiencing, the pile cap sizes are increased from 10 piles/ pile cap to 30 piles/ pile cap side of the building were the elevators, stairs and the tower rises there is an increased mat size to 30 piles per cap with 50" deep caps.

E) Secondary Structural System

The only mayor secondary structure that needs to be supported is the cooling tower yard located on top of the 8th level of the parking garage. The cooling yard supports two cooling towers feeding the condominium tower mechanical supply. This yard is approximately 27' x

[1] Changes occur only in the typical apartment units up to the 10th level of apartment floor. The upper 4 stories of penthouse apartments have more severe modifications to the shear wall layout. This will not be accounted for in this report but are mentioned in the Appendix and will be analyzed at a later time

27’ and is supported by six columns that form part of the parking garage frame. Two o these columns extend from the foundation through the 10 levels of parking and ends in the underside of the tower at the roof level. They are also located in the underside of the ramp. These columns see substantial load and are 24” x 56” with 16#11 reinforcement. The other four are added in the ground lobby floor. Two are continuations along the backside of the building on top of retaining wall. The other two are just added on the slab. These columns will contribute to larger punching shear stresses in the slab.

Part 2: General Provisions, Codes, and Requirements

A) Applicable Codes

a. Loads(includes wind):	ANSI/ASCE 7-95
b. Seismic:	UBC 1997
c. Reinforced Concrete:	ACI 318-95
d. Puerto Rico’s current adopted code of practice:	UBC 1997
e. Post-Tensioned Concrete two way slab system:	ACI-ASE 423
f. Steel:	AISC
g. Welding:	AWS

B) Strength Requirements

a. Concrete (28 day strength)

- Structural Slabs:	4,500psi
- Beams:	4,500psi
- Columns:	5,000psi
- Walls:	4,000psi
- Stairs:	4,000psi

b. Steel (Yield Strength, F_y)

- Reinforcement bars:	60,000psi
- Welded Wire Fabric:	50,000psi
- W Shapes – A992	50,000psi
- Plates, Channels, Angles, M, S Shapes	36,000psi
- Welding – E70xx	70,000psi
- Bolts – ASTM A 325	90,000psi

C) Minimum Required Reinforcement

a. Reinforced Concrete Walls

- 6” Thick	#4@12 E.W[2]
- 8”	#4@10 E.W
- 10”	#4@8 E.W

[2] E.W= Each way

- 12”	#4@12 E.W

b. Masonry Walls (Vertical Reinforcement)[3]
   - 6” Thick	#3@16” or #4 @ 32”
   - 8”	#5@32” or #6@48”

D) Steel Cover Requirements
   a. Footings
      - Side	3”
      - Bottom	2”

   b. Slab on Grade/Mat Foundation	1”

   c. Wall
      - Pour	3”
      - Exposed, up to #5	1-1/2”
      - Exposed, #6 or larger	2”
      - Not Exposed, up to #11	¾”

   d. Slab/Joist
      - Up to #11	¾”
      - #14 or larger	1-1/2”

   e. Beams/Columns	1-1/2”

E) Post-Tensioning
   a. Concrete
      - Compressive strength at transfer	3,000psi

   b. Steel[4]
      - Yield strength	270,000psi
      - Effective stress after losses	171,000psi
      - Preliminary long term losses	15,000psi[5]

Part 3: Framing Plans and Elevations

The following diagrams help explain the approach in which this building has been analyzed. It also lays the foundation for the for the conclusion and design analysis. They have been referenced throughout the report and are included here together for ease of location.

Figure 1 – Actual Floor Plan Layout

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[3] All Horizontal Reinforcement to be Dur-O-Wal Truss Type at every course
[4] Profiles and forces based on ½” diameter strands (#7 wire) – greased, plastic coated
[5] Value used for preliminary design only

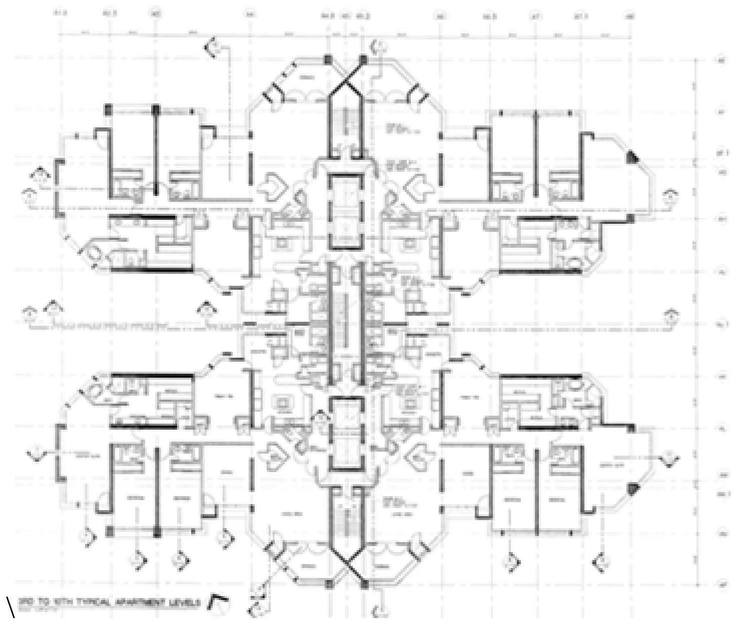
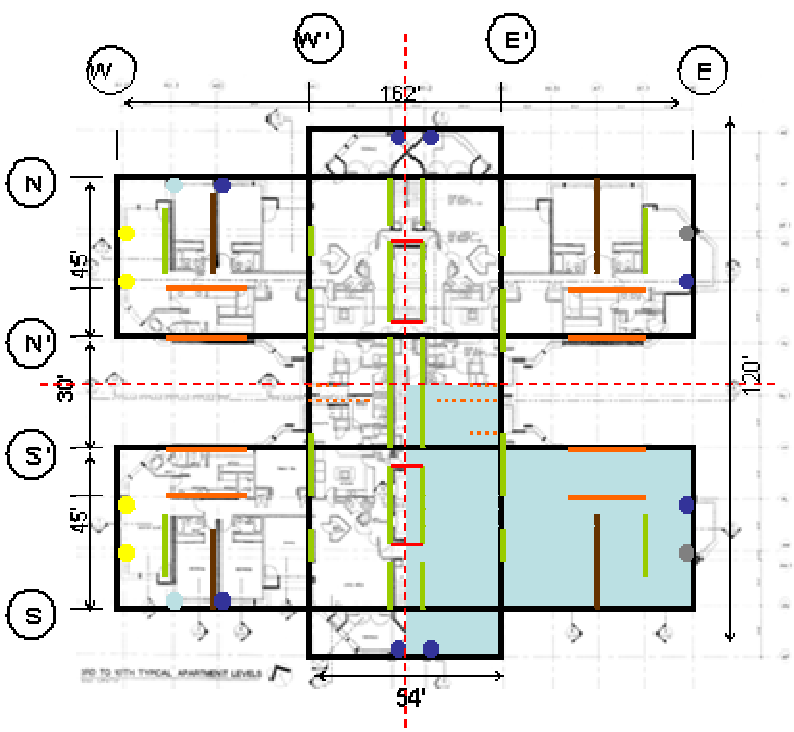


Figure 2 – Layout showing main structural slabs, columns, and shear walls



Part 4: Structural System: Conclusions and Analysis Approach

Figure 4 – Simplified Lateral Model of Structural Elements

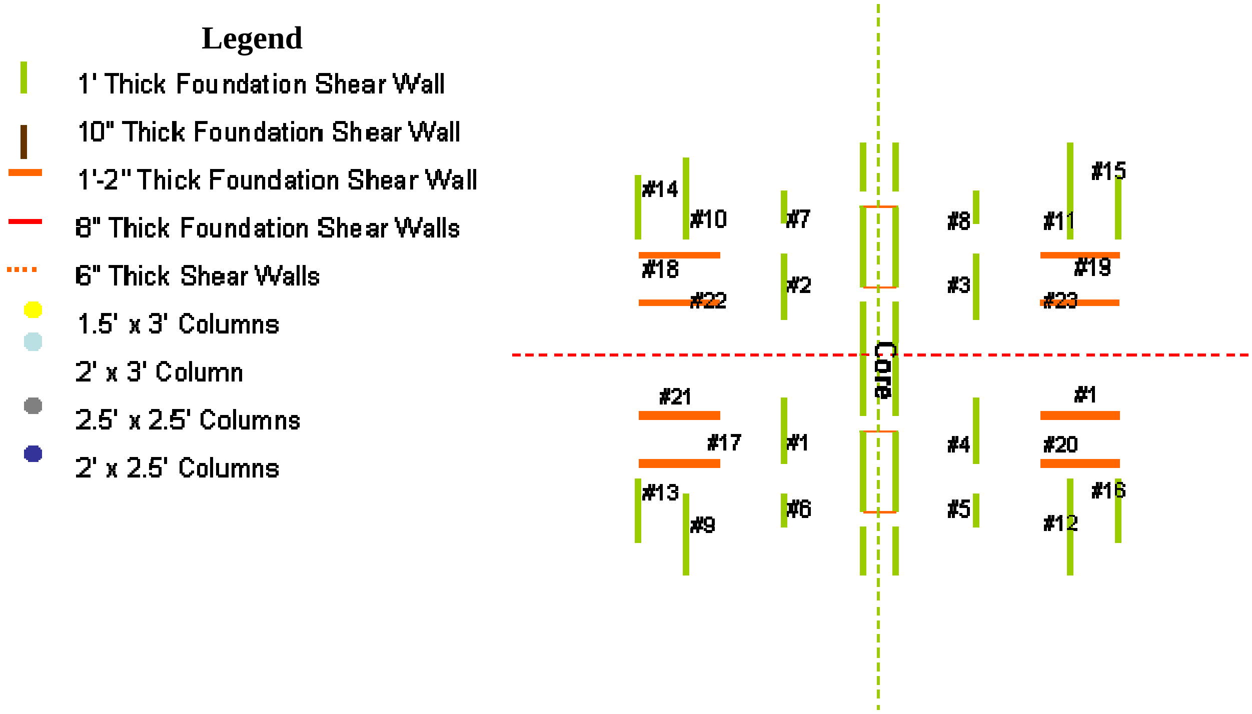
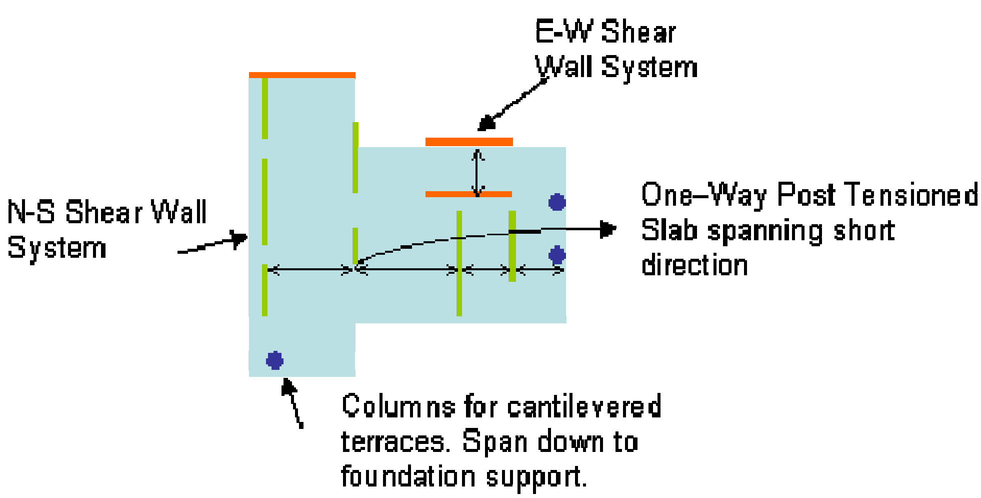


Figure 5 – Simplified Symmetric Model



Part 5: Loads and Loading Diagrams: Lateral System

A) Seismic
   a. Puerto Rico Zone Factor "2" 0.6
   b. Horizontal Force Factor "k" 1.33
   c. Building Importance Factor "I" 1.0
   d. Soil – Structure Resonance Coefficient "S" 1.3

Story lateral forces, Fx*, due to seismic loads for the existing building condition was calculated using the UBC 1997 code as per drawing recommendation.[6] Wall shear values for the E-W and N-S directions were obtained by assuming cantilevered action of the wall fixed on the foundation and experiencing the forces at each story level (Refer to Figure 5.a and 5.b) to find the maximum shear force and bending moment experienced by the wall.

Seismic Dead Load – Table 5.1

Level	Area (ft^2)	# Stories	Horizontal[7]	Vertical[8]	Total
Roof + Mech. Room	13000	1	1625	400	2025
Typ. Penthouse Floor	13000	4	1625	1100	2725
Typ. Apart Floor	16000	10	2000	1300	3300
Typ. Parking Deck	47000	9	5875	1788	7663
Lobby + Garage	60000	1	7500	1788	9288
Total Weight					124180

Factors – Table 5.2

Seismic Zone 3, Z	0.3	Soil Profile	Sd
Seismic Source Type	B	Period, T – Method A	1.3[9]
Base Shear	10250 kips	both directions	

Total Seismic Forces on Each Floor – Table 5.3

Earthquake Design Loads – UBC 1997

V = 10250 kips Ft = 932.75 kips

Level	Story Weight wx (k)	Height hx(ft)	wxhx	Lateral Force Fx* (k)	Story Shear Vx (k)	Moments
Roof	2025	244.17	494444	403	403	0
24	2725	234.33	638549	521	924	3968
23	2725	224.50	611753	499	1423	13055
22	2725	214.66	584958	477	1900	27048
21	2725	204.83	558162	455	2355	45733

[6] Puerto Rico Amendments (Dec. 1999) have not been incorporated. Calculations will be resubmitted upon arrival.
[7] Slab and superimposed dead weight
[8] Vertical shear walls. A modification includes a +20psf for concrete block walls used as partitions not part of the lateral resisting system.

20	3300	195.00	643489	525	2880	68894
19	3300	185.16	611039	498	3379	97217
18	3300	175.33	578589	472	3851	130440
17	3300	165.50	546139	445	4296	168303
16	3300	155.66	513689	419	4715	210546
15	3300	145.83	481239	393	5107	256910
14	3300	136.00	448789	366	5473	307133
13	3300	126.16	416339	340	5813	360955
12	3300	116.33	383889	313	6126	418117
11	3300	106.50	351439	287	6413	478357
10	7663	91.50	701139	572	6985	574549
9	7663	81.50	624509	509	7494	644395
8	7663	71.50	547879	447	7941	719335
7	7663	61.50	471249	384	8325	798744
6	7663	51.50	394619	322	8647	881996
5	7663	41.50	317989	259	8906	968467
4	7663	31.50	241359	197	9103	1057531
3	9288	20.00	185729	151	9255	1162219
2	7663	10.00	76604	62	9317	1254766
1	7663	0.00	0	0	**9317**	1347908
Sum	124180		11423582			**1347908**

Critical Forces on Selected Shear Walls – Table 5.4

	East - West Direction			**North - South Direction**		
	Shear Wall	**#1, #9, #13 – Ref. Fig. 4**		**Shear Wall**	**#17,#21 – Ref. Fig. 4**	
	For F_x*/x[10]	**x = 8**		**For F_x*/x**		**x = 18**
Level	Lateral Force Fx*' (k)	Story Shear Vx (k)	Moments	Lateral Force Fx*' (k)	Story Shear Vx (k)	Moments
Roof	50	50	0	22	22	0
24	65	116	496	29	51	220
23	62	178	1632	28	79	725
22	60	238	3381	27	106	1503
21	57	294	5717	25	131	2541
20	66	360	8612	29	160	3827
19	62	422	12152	28	188	5401
18	59	481	16305	26	214	7247
17	56	537	21038	25	239	9350
16	52	589	26318	23	262	11697
15	49	638	32114	22	284	14273
14	46	684	38392	20	304	17063
13	42	727	45119	19	323	20053
12	39	766	52265	17	340	23229
11	36	802	59795	16	356	26575
10	71	**873**	**71819**	32	388	31919
9	64	937	80549	28	416	35800
8	56	993	89917	25	441	39963
7	48	1041	99843	21	463	44375

[10] Fx*/x is the approximate effective load experienced by the a given shear wall forming part of the lateral resisting system in each direction. The value of x is approximated using relative wall stiffness. Look at the discussion for further details

6	40	1081	110249	18	480	49000
5	32	1113	121058	14	495	53804
4	25	1138	132191	11	506	58752
3	19	1157	145277	8	514	64568
2	8	1165	156846	3	518	69709
1	0	1165	168488	0	518	74884
Sum		1165	168488		**518**	**74884**

The values highlighted and bolded are the assumed critical values for seismic loading in each direction. As it is explained in the footnote, the total seismic lateral force per floor was distributed to the shear walls based on an approximate stiffness method. In the E-W direction, our simplified model (Figure 4) gives us 8 total shear walls or relatively the same size. Therefore, the lateral force F_x* was assumed to be distributed even among them. F_x*' = F_x*/8, where x = 8.

The N-S direction was a little more complicated as it is also our main lateral resisting system because the core spans in that direction. The assumption was made that 1/3 of the load goes into the core and the other 2/3 is distributed along the offset walls. In our simplified model (Figure 4), we have a total of 12 shear walls of approximately the same size. Therefore, each wall experiences a load F_x*' = F_x*/18, where x = 18.

The critical values I assumed for my design are highlighted. In the N-S direction it is clear that the critical shear and moment values are at the base. However, this was not my assumption for the E-W direction. The reason why I have decided to look at the 10th level in the E-W direction is because this is the transition level from the parking garage to the condominium tower. In this direction the parking garage that extends below is 100' wider and structurally it consists of a moment resisting frame comprised of the concrete columns and two-way slab system. This frame extends an extra 100' in the E-W direction is going to take some of the lateral load away from the shear walls. Therefore, I believe the critical loading condition for this wall is at the 10th level where the transition from the condominium to the parking garage occurs. This is not the case in the other direction because the parking garage doesn't extend beyond the boundaries of the condominium in the N-S direction.

B) Wind Loads

a.	Basic Wind Velocity	110mph
b.	Building Classification Category	II
c.	Building Importance Factor	1.05
d.	Method 2 – Pressure Coefficient	1.4

Wind requirements for the existing building were based on ANSI/ASCE 7-95. Calculations for this report were obtained using ANSI/ASCE 7-02. Discrepancies from the drawings and current code standards were minimal. Drawings specified a Building Classification Category I with an Importance factor of 1.05. This was not consistent with ANSI/ASCE 7-02. Therefore, a more appropriate Category II was used to model the building. Same assumptions used to the seismic analysis are applicable: cantilevered action of the shear walls and load distribution was based on relative stiffness. Also just like for seismic, in the East – West direction the critical load is considered at the 8th level, the point of transition from the condominium to the parking garage.

Wind Total Story Shear and Selected Wall Shear Values – Table 5.5

Wind Loads - ASCE 7-95

V	110	mph
kd	0.85	
Importance I	1.05	
Exposure Category	D	
Surface Roughness	D	
Kzt	1	
GCpi	0.18	
# Stories, n	22	

	N-S	**E-W**
Cp Windward	0.8	0.8
Cp Leeward	-0.5	-0.4
Gust, G	0.866	0.869
Dimensions (ft)	120	162
Shear Wall Acting/Floor (ft)	600	250
L of Shear Wall (ft)	23	23

					Resultant Pressure (psi)		**Story Forces (lb)**	
Story Level	z (ft)	Kz	qz	qh	N-S	E-W	N-S	E-W
Roof	224.17	1.65	45.62	45.62	51.35	47.57	40932	28084
22	214.33	1.63	45.06	45.62	50.97	47.18	81225	55696
21	204.50	1.61	44.51	45.62	50.59	46.80	80587	55224
20	194.66	1.61	44.51	45.62	50.59	46.80	80587	55224
19	184.83	1.59	43.96	45.62	50.21	46.42	79977	54770
18	175.00	1.57	43.40	45.62	49.82	46.03	79367	54316
17	165.16	1.56	43.13	45.62	49.63	45.84	79062	54090
16	155.33	1.54	42.57	45.62	49.25	45.45	78452	53636
15	145.50	1.53	42.30	45.62	49.06	45.26	78146	53409
14	135.66	1.51	41.75	45.62	48.67	44.88	77536	52956
13	125.83	1.49	41.19	45.62	48.29	44.49	76926	52502
12	116.00	1.46	40.36	45.62	47.72	43.92	76011	51822
11	106.16	1.44	39.81	45.62	47.33	43.53	75400	51368
10	96.33	1.42	39.26	45.62	46.95	43.15	74790	50915
9	86.50	1.4	38.70	45.62	46.57	42.76	93668	63718
8	71.50	1.35	37.32	45.62	45.61	41.80	92357	62704
7	61.50	1.32	36.49	45.62	45.03	41.23	72955	49471
6	51.50	1.28	35.39	45.62	44.27	40.46	71714	48549
5	41.50	1.23	34.00	45.62	43.31	39.50	70162	47395
4	31.50	1.17	32.35	45.62	42.16	38.34	68301	46012
3	21.50	1.09	30.13	45.62	40.63	36.81	65819	44167

2	11.50	1.03	28.48	45.62	39.48	35.65	68754	45991
1	0.00	1.03	28.48	45.62	39.48	35.65	36765	24593

Shear Wall Being Analyzed						
Lateral Forces (lb)		**Total Shear (lb)**		**Moment (lb-ft)**		
N-S	E-W	N-S	E-W	N-S	E-W	Story Level
1569	2584	0	0	0	0	Roof
3114	5124	1569	2584	15439	25424	22
3089	5081	4683	7708	61486	101218	21
3089	5081	7772	12788	137909	201563	20
3066	5039	10861	17869	244709	301481	19
3042	4997	13927	22908	381656	400988	18
3031	4976	16969	27905	548520	499675	17
3007	4935	20000	32881	745186	597746	16
2996	4914	23007	37816	971424	695202	15
2972	4872	26003	42729	1227118	792042	14
2949	4830	28975	47601	1512040	888267	13
2914	4768	31924	52431	1825958	983671	12
2890	4726	34838	57199	2168528	1078049	11
2867	4684	37728	61925	2539520	1171401	10
3591	5862	40595	66609	2938704	1263933	9
3540	5769	44186	**72471**	3601487	**1422126**	8
2797	4551	47726	78240	4078746	1538433	7
2749	4466	50523	82791	4583972	1641634	6
2690	4360	53272	87258	5116687	1731812	5
2618	4233	55961	91618	5676298	1820081	4
2523	4063	58579	95851	6262092	1906015	3
2636	4231	61102	99914	6873115	1988979	2
1409	2263	63738	104146	7606101	2084366	1
		65147	106408	**7605884**	2084345	

Even when Puerto Rico is located in an extreme hurricane prone region with five Category 4 hurricanes passing through the island in the last 25 years, seismic still controlled the design! It is no surprise that the numbers are so high.

For the wind load analysis it is important to mention that the irregularity of the building footprint has not been incorporated in the analysis. For a conservative estimate, the building was assumed to have a square shape. Later analysis will be performed using a finite element modeling program to see the actual effect of the loading conditions in the building structure. It is expected that the shear will be further resisted by the interaction of the cross formed by the concentrated core (N-S) and transverse (E-W) shear walls.

Just by looking at the Seismic and Wind values we can see that E-W shear wall experience a higher shear and moment values than N-S. This could explain why the walls in this direction are thicker. Recall from Figure 1.b, the walls in the E-W direction are 1'-2" thick, while the walls in the N-S direction are 1'.

Figure 6 – Parking Garage Foundation Layout

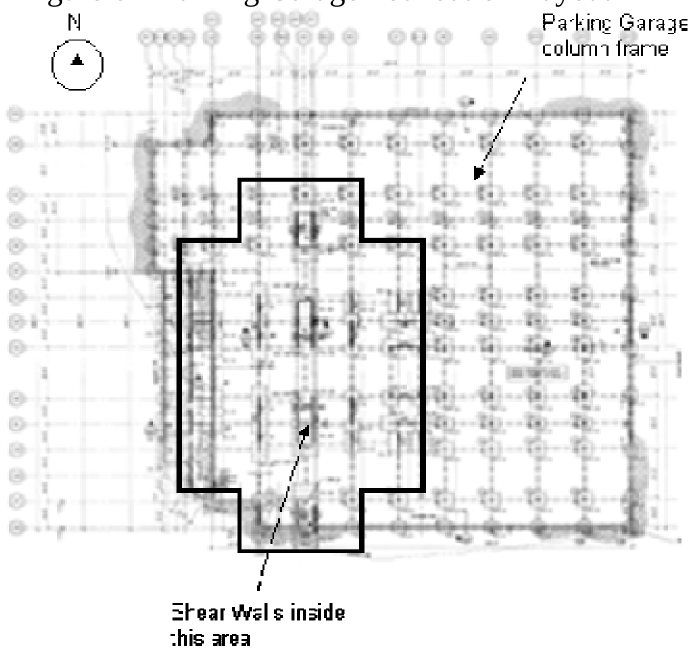


Figure 7 – Wind / Seismic Diagram

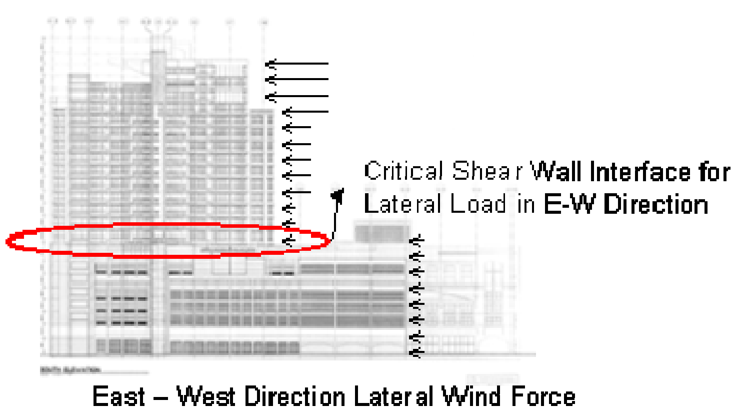


Figure 8 – Wind / Seismic Diagram

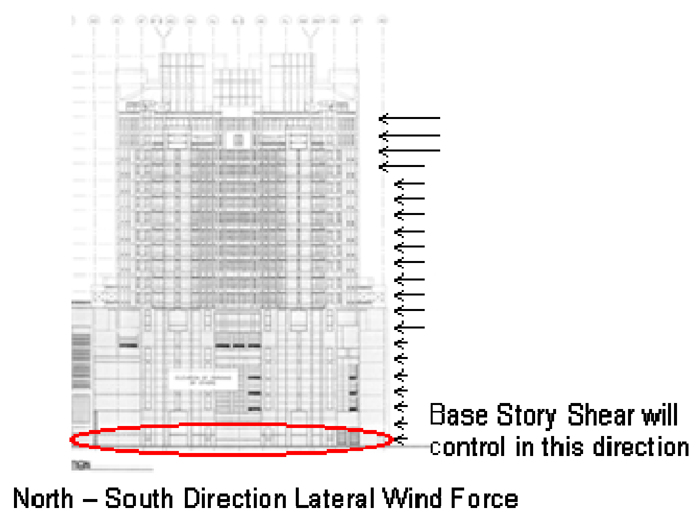
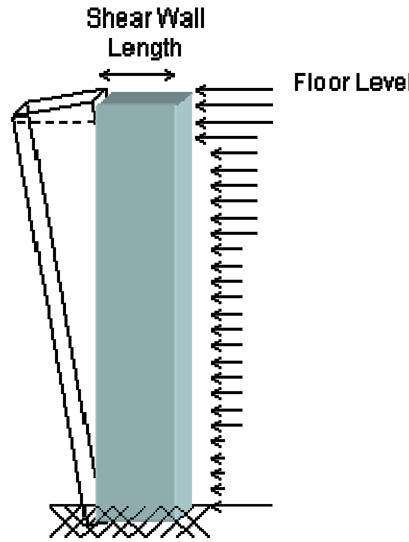


Figure 9 – Cantilevered Effect of Shear Wall



Part 6: Loads and Loading Diagram: Gravity

A) Live Loads

a.	Roof	40psf
b.	Floor	40psf
c.	Stairs	100psf
d.	Corridors	100psf
e.	Terrace	60psf
f.	Parking	50psf
g.	Storage	125psf
h.	Pool Deck	100psf

B) Dead Loads

a.	Slab – 8" thick	100psf
b.	Non – Bearing Concrete Block Walls	20 psf
c.	Superimposed MEP	25 psf
d.	Shear walls - 9' 2" High (per longitudinal area of wall)	1375 psf

The resultant service dead loads and live loads on each shear wall were obtained following UBC 1997 code references. Live and dead loads used are listed above. There were live loads reductions allowed for members carrying more than 150 ft^2. The reduction factor for members carrying only one floor is to be limited at 40% while the members carrying more floor loads can de reduced up to 60%. However, there is a note included that does not allow the reduction factor for parking garages to exceed 40% and lobbies and public spaces with live loads greater than 100 psf are not to be reduced at all. As a result live loads were reduced by 60% down to the 9^{th} level (first apartment floor), below of which lays the parking garage, reduced by 40% with the exception of level 8, 7 and 1 that are common areas for the condominium, this were not reduced at all

The major assumptions used for analysis was that the tributary area of each shear wall is half the span to each side to the next shear wall. Also, the live load was assumed to be larger, 100psf compared to 40psf, around the core because of the location of stairs and corridors in this area. Analyses were performed in walls on both N-S and E-W directions. In the N-S direction, shear wall #5 (Figure 5) was analyzed because it has one of the highest tributary areas and least wall area. On the E-W direction, shear wall # 1 was analyzed even when the tributary areas of both wall #1 and # 24 are similar (actually, #24 has slightly higher tributary area and slightly lower wall area), shear wall #1 is eccentrically loaded under this gravity load. This will further decrease the capacity of the wall.[11]

Figure 10 – Shear Wall Tributary Area, Typical Live Loading, and Slab Span

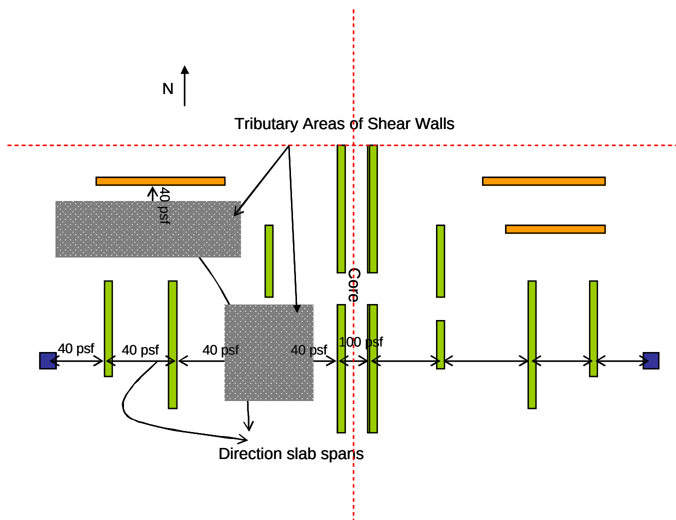


Table 6.1 North – South Shear Wall Results for Gravity Loads

Service Dead and Live Loads for Selected Shear Walls - UBC 1997						
Shear Wall #	#5	Refer to Figure 5				
Location	1	**Note:**	Select 0 for core shear walls, 1 for others			
N-S Tributary Width	27	ft				
E-W Tributary Width	24.5	ft		Interior Live Load	40	psf
Length Wall	17	ft		Core Live Load	100	psf
Thickness of Wall	1	ft		R'	1.07	

[11] This effect has not been accounted for in this preliminary calculation but it is important to consider in the final design.

Story	Dead (psf)	Live (psf)	Supported Area (ft^2)	R	Reduced Live (psi)	• Dead (kip)	• Live (kip)
Roof	100	30	661.5	0.59	18	90	12
22	145	40	1323.0	0.40	16	209	22
21	145	40	1984.5	0.40	16	328	33
20	145	40	2646.0	0.40	16	447	43
19	145	40	3307.5	0.40	16	567	54
18	145	40	3969.0	0.40	16	686	65
17	145	40	4630.5	0.40	16	805	75
16	145	40	5292.0	0.40	16	925	86
15	145	40	5953.5	0.40	16	1044	96
14	145	40	6615.0	0.40	16	1163	107
13	145	40	7276.5	0.40	16	1283	118
12	145	40	7938.0	0.40	16	1402	128
11	145	40	8599.5	0.40	16	1521	139
10	145	40	9261.0	0.40	16	1640	149
9	145	40	9922.5	0.40	16	1760	160
8	145	100	10584.0	0.60	100	1879	226
7	145	100	11245.5	0.60	100	1998	292
6	100	50	11907.0	0.60	30	2088	312
5	100	50	12568.5	0.60	30	2177	332
4	100	50	13230.0	0.60	30	2267	352
3	100	50	13891.5	0.60	30	2356	372
2	100	50	14553.0	0.60	30	2446	391
1	145	100	15214.5	0.60	100	2565	458
B1	100	50	15876.0	0.60	30	2655	477
B2	100	50	16537.5	0.60	30	**2744**	**497**

Table 6.2 East – West Shear Wall Results for Gravity Loads

Service Dead and Live Loads for Selected Shear Walls - UBC 1997

Shear Wall #	#24	Refer to Figure 5				
Location	1	**Note:**	Select 0 for core shear walls, 1 for others			
N-S Tributary Width	8	ft				
E-W Tributary Width	39	ft		Interior Live Load	40	psf
Length Wall	23	ft		Core Live Load	100	psf
Thickness of Wall	1.17	ft		R'	1.07	

Story	Dead (psf)	Live (psf)	Supported Area (ft^2)	R	Reduced Live (psi)	• Dead (kip)	• Live (kip)
Roof	100	30	312.0	0.87	26	68	8
22	145	40	624.0	0.62	25	150	16
21	145	40	936.0	0.40	16	233	21
20	145	40	1248.0	0.40	16	315	26
19	145	40	1560.0	0.40	16	397	31
18	145	40	1872.0	0.40	16	479	36
17	145	40	2184.0	0.40	16	562	41
16	145	40	2496.0	0.40	16	644	46
15	145	40	2808.0	0.40	16	726	51
14	145	40	3120.0	0.40	16	809	56

13	145	40	3432.0	0.40	16	891	61
12	145	40	3744.0	0.40	16	973	66
11	145	40	4056.0	0.40	16	1055	71
10	145	40	4368.0	0.40	16	1138	76
9	145	40	4680.0	0.40	16	1220	81
8	145	100	4992.0	0.60	100	1302	112
7	145	100	5304.0	0.60	100	1384	143
6	100	50	5616.0	0.60	30	1453	153
5	100	50	5928.0	0.60	30	1521	162
4	100	50	6240.0	0.60	30	1589	171
3	100	50	6552.0	0.60	30	1657	181
2	100	50	6864.0	0.60	30	1725	190
1	145	100	7176.0	0.60	100	1808	221
B1	100	50	7488.0	0.60	30	1876	231
B2	100	50	7800.0	0.60	30	**1944**	**240**

Part 7: Critical Loads and Spot Checks

Critical load combinations were obtained following UBC 1997. This is the current code of practice in Puerto Rico and I wanted to compare my results to this values. I discovered that the worst case is a result of D + L + Eh even with the reduction in the Live Load of 0.55. The reason is that Dead Load and Earthquake Loads are the most signification contribution to the stress of this building shear walls.

Table 7.1 Worst Load Combination in North – South Direction for Shear Wall #5

Load Case	Axial (k)	Bending (ft-k)	Shear (k)
Dead(D)	2744	0	0
Live(L)	497	0	0
Earthquakes(Eh)	-200	74884	518
	200	-74884	-518
p = redundance factor	1		
Ev =	0.2D		
Load Combination			
1.4D + 1.7L	4686.5	0	0
1.54D + 0.55L + 1.1Eh	4279	82372	570
	4719	**-82372**	**-570**
0.77D+1.1Eh	1892	82372	570
	2332	-82372	-570

Table 7.2 Worst Load Combination in East-West Direction for Shear Wall #24

Direction	E-W	Shear Wall	#24
Load Case	Axial (k)	Bending (ft-k)	Shear (k)
Dead(D)	1302	0	0
Live(L)	112	0	0
Earthquakes(Eh)	-250	71819	873
	250	-71819	-873
p = redundance factor	1		
Ev =	0.2D		
Load Combination			
1.4D + 1.7L	2013.2	0	0
1.54D + 0.55L + 1.1Eh	1792	79000	960
	2342	**-79000**	**-960**
0.77D+1.1Eh	1838	79000	960
	2388	-79000	-960

Following UBC 1997 provisions and the worst load axial, shear, and bending values obtained from the different load combinations I will check the selected shear walls for web reinforcement along the shear wall. I will also verify that the wall needs boundary zone reinforcement and an approximation of the extent of the boundary will be provided.[12]

<u>In the N-S direction: Shear Wall #5</u> (Figure 4)
L = 17’
t = 12”

d. Check web of the wall for shear reinforcement. Reinforcement most be provided if Vu > 2*Acv*• Fc’
   - Vu = 570 kips > 2*(12x17x12)*sqrt(4000)/1000 = 310 kips

   Therefore 2 curtains of reinforcement most be provided.

e. Check upper limit on strength of the reinforcement
   - • 8Acv*• Fc’ = 1054 kips

f. For hw/lw = 14.5 > 2.0
   - • Vn = • Acv*(2• Fc’ + • nfy)
   - • = 0.85
   - Drawings call for 2#4 bars @ 10”, p = 2*.2/12*10 =0 0.0033
   - **• Vn = 680 kips > 570 kips**

[12] Knowledge on how to size shear wall boundary reinforcement is not available yet. All that can be done for now is approximate where it needs it and what would be a conservative boundary length. Later reports will have more detail on this as I am aware that it is an important part of the shear wall design.

d. Vertical Reinforcement, pv
   - Hw/lw = 14.5 > 2 : Provide minimum reinforcement
   - Asv,min = 0.0025*12*12 = 0.36in^2/ft
   - Drawings call for 2#4 bars, Asv = 0.4
   - Req'd s = 0.4*12/0.36 = 13" < 18" min
   - #4@10" on drawing is appropriate

e. Boundary Design Zone: Must check for the effective boundary zone of the shear area to withstand axial loads and their eccentricities.
   - Pu = 4720 kips from critical load combinations Table 7
   - Pmax = 0.1*Ag*fc' Assuming 2' x 2' effective boundary of shear wall acting in unison with column grid on parking garage.
   - Pmax = 0.1*(13x12x12) + 2x24^2)*4 = 1210 kips << 4720 kips
   - Boundary reinforcement will be required and detailed. This can be seen in the drawing as the wall has 10#7 bars called out covering an effective boundary (B.E.) = 22". This is a significant amount of reinforcement on each side of the wall to account for the increased stress at this area due to high seismic and dead loads.

<u>In the E-W direction: Shear Wall #24</u> (Figure 4)
Recall, we are looking at the 8th level transition from condominium tower to garage.
L = 23'
t = 14"

f. Check web of the wall for shear reinforcement. Reinforcement most be provided if Vu > 2*Acv*• Fc'
   - Vu = 960 kips > 2*(14x23x12)*sqrt(4000)/1000 = 490 kips

   Therefore 2 curtains of reinforcement most be provided.

g. Check upper limit on strength of the reinforcement
   - • 8Acv*• Fc' = 1662 kips

h. For hw/lw = 10 > 2.0
   - • Vn = • Acv*(2• Fc' + • n*fy)
   - • = 0.85
   - Drawings call for 2#4 bars @ 10", p = 2*.2/14*10 =0 0.00286
   - **• Vn = 979 kips > 960 kips**

d. Vertical Reinforcement, pv
   - Hw/lw = 14.5 > 2 : Provide minimum reinforcement
   - Asv,min = 0.0025*14*12 = 0.42in^2/ft
   - Drawings call for 2#4 bars @ 10" , Asv = 0.4*10/12 = **0.48 in^2/ft**
   - #4@10" on drawing is appropriate

e. Boundary Design Zone: Must check for the effective boundary zone of the shear area to withstand axial loads and their eccentricities.
    - Pu = 2342 kips from critical load combinations Table 7
    - Pmax = 0.1*Ag*fc' Assuming 2' x 2' effective boundary of shear wall acting in unison with column grid on parking garage.
    - Pmax = 0.1*(19'x12"x14") + 2x24"^{2})*4 = 1738 kips < 2342 kips
    - Boundary reinforcement will be required and detailed. This can be seen in the drawing as the wall has 14#6 bars called out covering an effective boundary (B.E.) = 32" at the eight level.

This is a significant amount of reinforcement. It is also at the eight level, at the lower level it is even considerable larger, both the web reinforcement and the boundary zone reinforcement. It is important to point out however that this is more reinforcement than the N-S shear wall that experienced a higher load, by almost doubled. This is not too surprising after some further analysis of the structure. The structure is going to experience large torsion problem in this direction. In the N-S direction, the centroid and center of mass are located relatively close to each other. However, in the E-W direction, the center of mass will be will be offset to each side of the center line while the centroid will be somewhere at the middle. Further calculations will show how big of an impact the torsion will have on the structure's reinforcement.