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Paseo Caribe Condominium Tower and Parking Garage
San Juan, Puerto Rico

Technical Assignment 3

Lateral Systems Analysis & Confirmation Design

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1.Building Description

1.1 Introduction

This technical assignment examines the lateral system of Paseo Caribe Condominium Tower and Parking Garage. This structure is a 14 story cast in place concrete luxurious 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.

1.2 Lateral System

Lateral forces due to wind and seismic on the building are designed to be sustained by a shearwall/ bearing frame system in both north-south and east-west directions. The walls act as a cantilever, resisting the applied lateral loads at each level through deflection. In the north-south direction there are a total of 28 shearwalls. In this direction the shear walls are 10" wide and they cover a total distance of approximately 629 linear feet per floor. In the east-west direction, there are 8 resisting lateral walls, each 12" wide. They are located in the center of the building spanning that direction and cover approximately 145 linear feet.

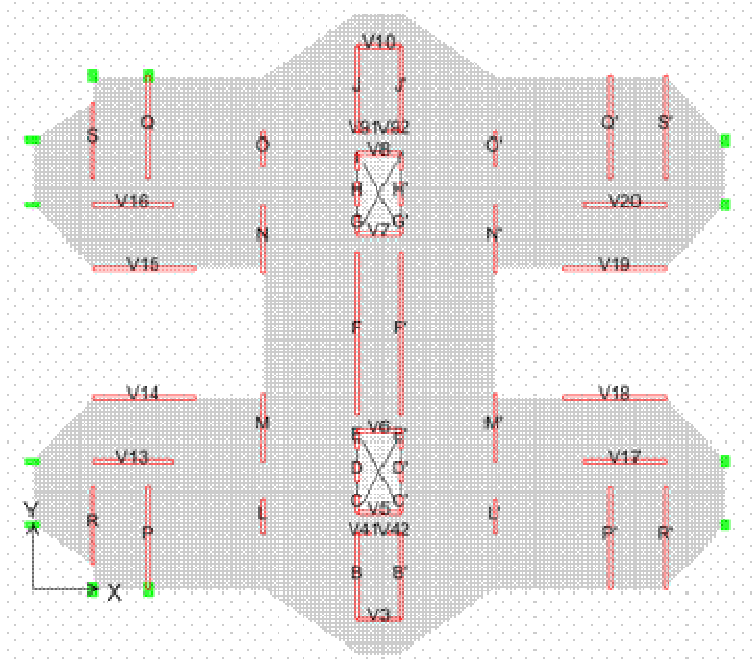

Figure 1: Apartment shearwall system and labeling

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 are (Refer to Figure 1)

- 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 L, O are extended 8’ south over the original wall.
- Shear walls M, N are extended 13’ south over the original wall.
- Shear wall V14-V18 extended 8’ inward over original wall.
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1.3 Lateral System Considerations

Seismic and wind forces cause lateral forces to develop at each story height. These lateral forces are assumed to be acting on the center of mass of each story’s diaphragm. The 8” concrete slab acts as a rigid shell and transfers the forces into each wall. Based on the fundamental principle that load follows stiffness, the shear walls are going to resist a portion of the lateral force in proportion to their relative rigidity. The relative rigidity of each wall depends on a ratio that relate wall thickness, length and height. Detailed calculations are provided and discussed later in the report The lateral load is then transferred through shear and bending of the shearwall out of the building into the foundation. The primary function of the shearwall system is to resist and transfer lateral forces due to shear and bending. Therefore, shear strength calculations are critical in the wall selection. However, other considerations must also me evaluated when designing a shearwall lateral system:

1. Load combinations: Even when the lateral forces will not cause significant axial forces in the shearwalls, bearing wall concrete buildings experience large axial dead loads that must be considered along with lateral bending strength in the wall.
2. Overturning: It is important to ensure that the moment caused by the lateral loads in the whole building and in each wall can be resisted by a "resisting" moment. Adequate support must be provided at the foundation. For purposes of this report, the resisting moment will be conservatively assumed to be provided by the dead weight of the building.
3. Torsion: Differences between the center of mass of a building story and the center of rigidity can cause torsion to develop in the building. This torsional moment is caused by the lateral force, applied at the center of mass, trying to reach equilibrium by effectively "twisting" the building toward the center of rigidity in the horizontal plan. The vertical elements/shear walls will resists this twisting through shear. Increasing the shear design value, Vu.

2. Design Consideration

2.1 Load Combinations

The design code provision using for Paseo Caribe and that hold as current practice in Puerto is UBC 1997. Following code provisions 1612.2.1 for concrete and masonry with a 1.1 multiplier if seismic forces are to be considered, yields the following load combinations:

1. 1.4D + 1.7L
2. 0.75(1.4D + 1.7L + 1.7W)
3. 0.9D + 1.3W
4. 1.1(1.2D + f_1L + f_2S + 1.1E),   f1 = 0.5 for live loads < 100psf, S = 0 in Puerto Rico
5. 1.1(0.9D + 1.0E)

2.2 Axial Loads

The shearwalls also act as the bearing walls for gravity loads transfer to the foundation. This 24 story high concrete building has very high dead loads when you consider a 16000sqft of 8" slab per floor (100psf) and as was mentioned earlier, over 775 linear feet of 12" concrete wall per floor, spanning 222 feet in the air ! This load will not only affect the axial forces on the walls but are also very important in the determination of seismic forces on the building since they are directly proportional. V= CvIW/RT. The total dead weight of the building was calculated to be 95132 kips:

Table 1: Dead Weight Calculations

Floor Description	# Stories	Floor Area (ft^2)	Story Height (ft)	10" Wall (lf)	12" Wall (lf)	Column Area (ft^2)	Slab Load	Wall/Col Load (k)	Total Load(k)
Penthouse	2	10200	9.83	168	217.5	30	2040.00	1061.64	3101.64
Typical Apartments	12	15870	9.83	336	435	60	19044.00	12739.68	31783.68
Common Area	1	63084	15	404	502	199	6308.40	1924.31	8232.71
Parking Garage	7	63084	10	384	392	434	44158.80	7855.75	52014.55
Total Weight							71551.20	23581.38	**95132.58**

Detailed calculations were performed using a spreadsheet to find the total axial loads on each shearwall due to dead, superimposed, and live loads. Reductions factors recommended by UBC 97 code provisions 1607.5 were used for shearwall live loads. This provision states that live load reductions can be applied to members carrying more than 150 square feet. The maximum reduction to members carrying load in multiple stories should not be greater than 60% unless live loads are greater than 100psf, in which case it should not be less greater than 40%. Example of axial load on critical shearwall M with tributary area of 750 square feet:

Table 1

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

Shear Wall #	M	Refer to Figure 5				
Location	1	**Note:**	Select 0 for core shear walls, 1 for others			
N-S Tributary Width	30	ft				
E-W Tributary Width	25	ft		Interior Live Load	60	psf
Length Wall	24	ft		Core Live Load	100	psf
Thickness of Wall	0.83	ft		R'	0.69	

Story	Floor (psf)	Live (psf)	Supported Area (ft^2)	R	Reduced Live (psf)	? Dead (kip)	? Live (kip)
Roof	100	30	750.0	0.52	16	102	12
21	120	60	1500.0	0.40	24	220	30
20	120	60	2250.0	0.40	24	337	48
19	120	60	3000.0	0.40	24	455	66
18	120	60	3750.0	0.40	24	572	84
17	120	60	4500.0	0.40	24	689	102
16	120	60	5250.0	0.40	24	807	120
15	120	60	6000.0	0.40	24	924	138
14	120	60	6750.0	0.40	24	1042	156
13	120	60	7500.0	0.40	24	1159	174
12	120	60	8250.0	0.40	24	1276	192
11	120	60	9000.0	0.40	24	1394	210
10	120	60	9750.0	0.40	24	1511	228
9	120	60	10500.0	0.40	24	1629	246
8	120	60	11250.0	0.40	24	1746	264
7	120	100	12000.0	0.60	100	1863	339
6	120	50	12750.0	0.60	30	1981	361
5	120	50	13500.0	0.60	30	2098	384
4	120	50	14250.0	0.60	30	2216	406
3	120	50	15000.0	0.60	30	2333	429
2	120	50	15750.0	0.60	30	2450	451
1	120	50	16500.0	0.60	30	2568	474
0	120	100	17250.0	0.60	100	2685	549
B1	120	50	18000.0	0.60	30	2803	571
B2	120	50	18750.0	0.60	30	**2920**	**594**

2.3 Lateral Forces

2.3-1 Wind Loads

Preliminary calculations were performed using a spreadsheet for wind lateral and shear forces on Paseo Caribe following ANSI/ASCE 7-95 per drawing recommendations. Located in the Caribbean Sea and in a very hurricane prone region with five Category IV Hurricanes (wind speeds > 125 mph) directly hitting the island in the last 25 years and personally experiencing a couple of them, I was very concerned about lateral wind forces in my design. Paseo Caribe is not a typical square building. It has plenty of discontinuities in its “flower” shape arrangement. For my preliminary calculations I decided to conservatively make the building a square box with boundaries representing the largest dimensions of the building, 190’ x 162’. This is conservative because the width represented by this dimensions (190’) only occurs in about 20% the length of the building. The rest is much narrower, about 60' to 140' wide. The parameters used for the analysis were provided by the structural drawings:

Basic Wind Velocity	100mph
Building Classification	II
Importance Factor	1.05
Pressure Coefficient-Method 2	1.4

Table 2

Wind Loads - ASCE 7-95

V	110 mph						**N-S**	**E-W**
kd	0.85				Cp Windward		0.8	0.8
Importance I	1.05				Cp Leeward		-0.5	-0.4
Exposure Category	D				Gust, G		0.866	0.869
Surface Roughness	D				Dimensions (ft)		120	162
Kzt	1				Shear Wall Acting/Floor (ft)		600	250
GCpi	0.18				L of Shear Wall (ft)		23	23
Number of Stories, r	22							
					Resultant Pressure (psi)		**Story Forces (K)**	
Story Level	**z (ft)**	**Kz**	**qz**	**qh**	**N-S**	**E-W**	**N-S**	**E-W**
Roof	222.62	1.65	45.62	45.62	51.35	47.57	41	28
21	212.79	1.63	45.06	45.62	50.97	47.18	81	56
20	202.96	1.61	44.51	45.62	50.59	46.80	81	55
19	193.13	1.61	44.51	45.62	50.59	46.80	81	55
18	183.30	1.59	43.96	45.62	50.21	46.42	80	55
17	173.47	1.57	43.40	45.62	49.82	46.03	79	54
16	163.64	1.56	43.13	45.62	49.63	45.84	79	54
15	153.81	1.54	42.57	45.62	49.25	45.45	78	54
14	143.98	1.53	42.30	45.62	49.06	45.26	78	53
13	134.15	1.51	41.75	45.62	48.67	44.88	78	53
12	124.32	1.49	41.19	45.62	48.29	44.49	77	52
11	114.49	1.46	40.36	45.62	47.72	43.92	76	52
10	104.66	1.44	39.81	45.62	47.33	43.53	75	51
9	94.83	1.42	39.26	45.62	46.95	43.15	75	51
8	85.00	1.4	38.70	45.62	46.57	42.76	94	64
7	70.00	1.35	37.32	45.62	45.61	41.80	92	63
6	60.00	1.32	36.49	45.62	45.03	41.23	73	49
5	50.00	1.28	35.39	45.62	44.27	40.46	72	49
4	40.00	1.23	34.00	45.62	43.31	39.50	70	47
3	30.00	1.17	32.35	45.62	42.16	38.34	68	46
2	20.00	1.09	30.13	45.62	40.63	36.81	66	44
1	10.00	1.03	28.48	45.62	39.48	35.65	64	43
0	0.00	1.03	28.48	45.62	39.48	35.65	32	21

2.3-2Seismic Loads

Seismic forces were calculated based on UBC 1997 provisions. The building and soil classification parameters obtained from the structural drawing specify:

Seismic Zone 3, Z = 0.3

Seismic Type B

Soil Profile Sd

Period T = 1.35 (Method A)

Table 3: Seismic Design Parameters and Loads

Calculated Parameters - UBC 1997	
W	95132.58
Cv	0.54
Ca	0.36
R	4.50
T	1.35
I	1.00
V = 2.5CaIW/R	19026.52
V = CvIW/RT	8456.23
V = 0.11CaIW	3767.25

Earthquake Design Loads - UBC 1997							
	V =	8456.23 kips			Ft = 0.7TV	799.11 kips	
	Level	Story Weight, wx (k)	Height, hx(ft)	wxhx	Lateral Force, Fx* (k)	Story Shear, Vx (k)	Moments (FT-K)
PENT4	22	1550.82	222.62	345244	1122	**1122**	0
PENT3	21	1550.82	212.79	329999	309	**1431**	11029
PENT2	20	2648.64	202.96	537568	503	**1933**	25092
PENT1	19	2648.64	193.13	511532	478	**2412**	44096
10TH	18	2648.64	183.30	485496	454	**2866**	67802
9TH	17	2648.64	173.47	459460	430	**3295**	95972
8TH	16	2648.64	163.64	433423	405	**3701**	128365
7TH	15	2648.64	153.81	407387	381	**4082**	164742
6TH	14	2648.64	143.98	381351	357	**4438**	204865
5TH	13	2648.64	134.15	355315	332	**4771**	248493
4TH	12	2648.64	124.32	329279	308	**5078**	295387
3RD	11	2648.64	114.49	303243	284	**5362**	345308
2ND	10	2648.64	104.66	277207	259	**5621**	398017
1ST	9	2648.64	94.83	251171	235	**5856**	453274
P8	8	8232.71	85.00	699781	654	**6511**	510840
P7	7	7430.65	70.00	520146	486	**6997**	608498
P6	6	7430.65	60.00	445839	417	**7414**	678468
P5	5	7430.65	50.00	371533	347	**7761**	752607
P4	4	7430.65	40.00	297226	278	**8039**	830220
P3	3	7430.65	30.00	222920	208	**8248**	910613
P2	2	7430.65	20.00	148613	139	**8387**	993091
LOBBY	1	7430.65	10.00	74307	69	**8456**	1076958
	VALUES	**95132.58**		**8188036**		**8456**	**1076958**

If we compare these results with the wind forces, it is clear that seismic forces control de lateral system design. The primary concern of high wind forces being inappropriately modeled as too conservative because of the larger area used was minimized when the seismic results were obtained. The maximum story shears due to seismic is about 5 times larger than that due to wind

Seismic: 8456 kips

Wind: 1678 kips

This can be explained by many factors including location close to a fault line, bad soil characteristics, and a very large building dead weight!

3. Lateral Design Model

Paseo Caribe is a complex structure to analyze for lateral loads because of the discontinuities throughout the building's stories as it changes from a parking garage (levels P2-P7) into an common recreation area with a gymnasium and pool (P8), and then into a 14 story apartment complex. The first 8 stories above ground are parking facilities. Throughout these levels the structure is a joint column frame system and shearwalls. There are 82 columns on each level (green on Figure 1). The grid is made of a uniform 27' east-west x 15' north to south spacing. Typical column sizes are 24"x 24", 24"x 30", 24"x 36", and 24"x 54".

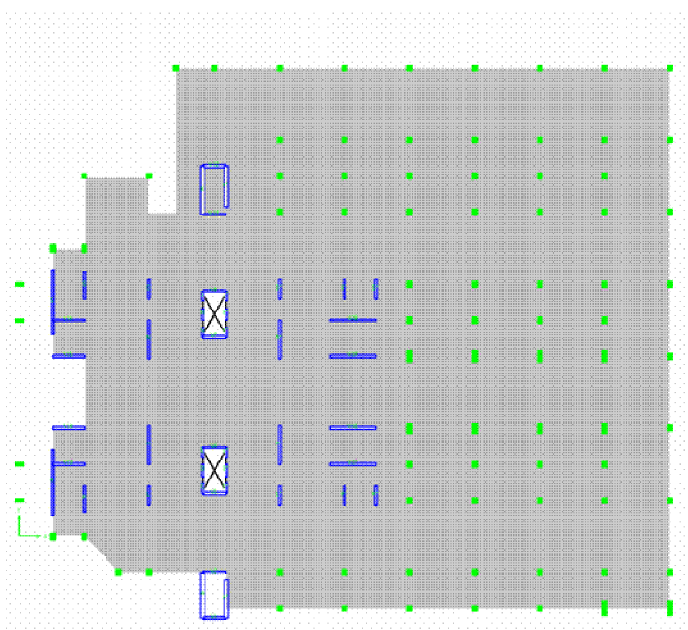

Figure 2: Typical Parking Layout

At the P8 level, or common area level, there is a transition. The building floor area and the configuration of the shear walls in the east- west direction change at this level. This is where the building goes from being a parking garage to an apartment condominium.

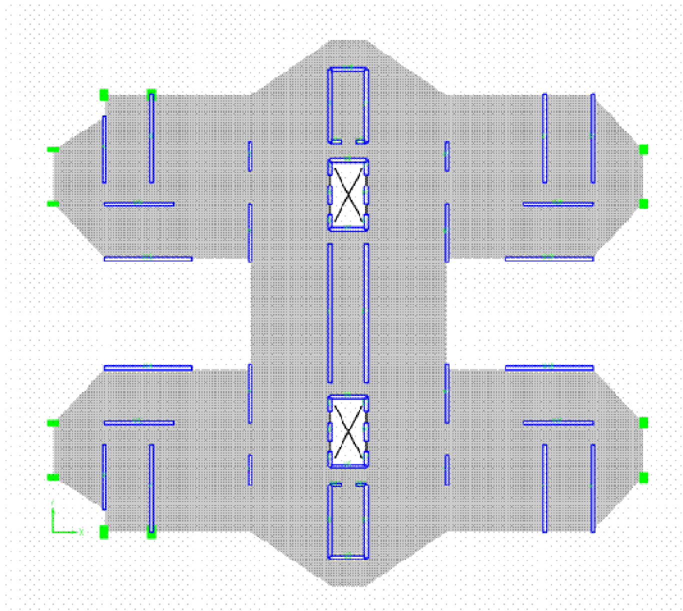

Figure 3: Typical apartment floor plan showing added core shearwalls and reduced floor area

From Figure 2 and Figure 3 above we can observe the changes taking place. First, the dimension of the building is reduced by 25%, from 190' to less than 150' in the east – west direction. There is one more change in the structure that affects the lateral analysis. There is a change in shearwall configuration. Two core staircases are removed and two others are introduced at different locations. At this level we also have that every shear wall is elongated from the original shearwall length by 4' to 8'. This creates a change (increase) in stiffness and rigidity in this level that is hard to account for by simple hand calculations. Changes in floor dimension and shearwall configuration do not take place for the lateral system in the north-south direction. This will have an impact in analysis results.

The lateral system was analyzed by two methods. The first is a simplified model used for hand calculation and spreadsheet outputs. The assumptions and simplifications of this model are mentioned in the next section. The second method uses Etabs software modeling tool in an attempt to more accurately model the buildings and reduce the number of assumptions.

3.1 Simplified Model

Throughout the next sections you will see copies of the hand calculations I performed on this building. I created multiple spreadsheets to calculate lateral seismic forces, story shears, wall rigidities, shear distributions and torsion effects two critical walls in each direction. However, due to the complexity of the lateral system I had to make some assumptions on my analysis. These are:

1. The parking garage is not considered in the lower 10 levels of the structure for wind and seismic loads
2. Walls are assumed to be continuos and uniform from the basement up with no changes in dimensions or configuration.
3. Square box instead of flower floor arrangement was used. Change is floor dimensions at 8^{th} level ignored.

3.2 Etabs Model

I believed that a more accurate model was needed that would account for the transition level, the columns frame in the parking garage levels, and the change in wall dimensions and rigidities at each level. I wanted to an actual model of my building and see the difference between my assumptions and more realistic results. I decided to use Etabs to model my building. I created a complete model of the whole building, taking into account the different walls at each level and modeling the parking garage structure.

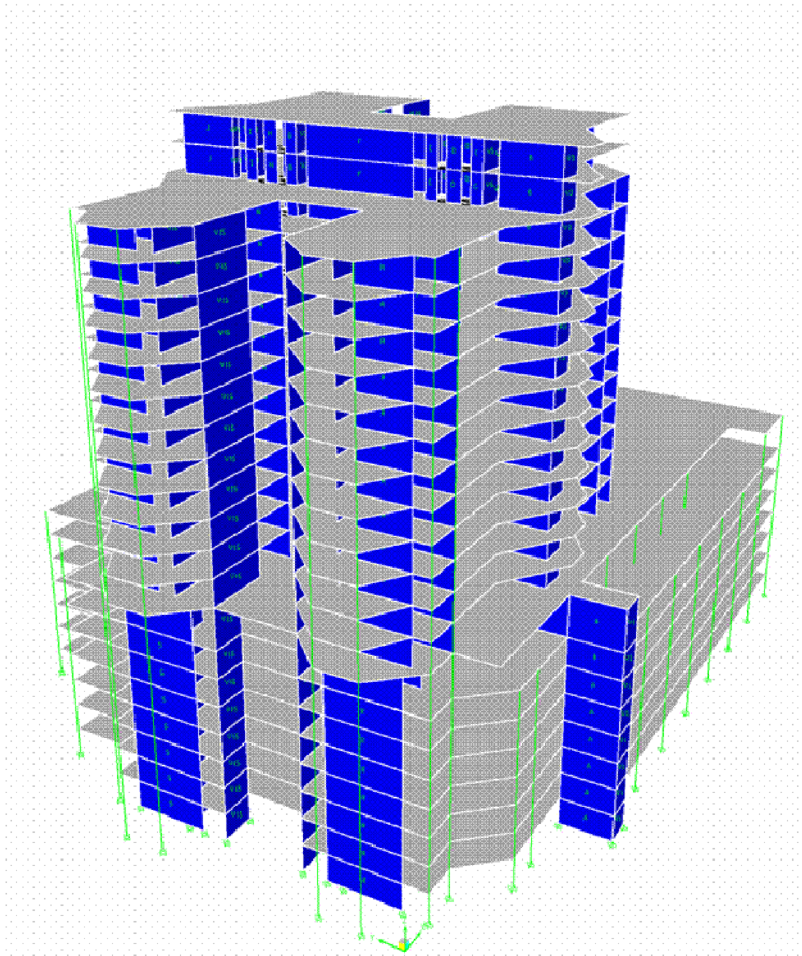

Figure 4: Etabs Paseo Caribe and Parking Model

To simplify the model and the post processing procedure, I only used my model for lateral load analysis. I had already performed a detail calculation of gravity live and dead load on my building. (See Table 2) Therefore, I assigned my floor to be a rigid diaphragm, only transferring lateral loads. The parameters inputted were the same as those provided in the drawing and used in the hand calculations following UBC 1997. To be more accurate, I allowed the program to calculate the Period T of the building, instead of the simplified Method-A I had previously used in my calculations.

4. Lateral System Results and Comparison

This section looks in detail at the results obtained for shear and bending through my simplified hand calculations and the Etabs results. The comparison starts with the total load and lateral story forces of each model and follows with a close examination of two shear walls in each direction.

4.1 Lateral Forces and Story Shear

A comparison of the lateral forces and story shear forces show that my initial forces were conservative. The forces obtained through the program are smaller than mine calculated. The reason is a lower calculated weight for the building and I higher calculated Period resulting in a lower overall Base Shear. Recall that I allowed the program to calculate the Period T of the building, instead of the simplified Method-A I had previously used in my calculations.

Table 4: Seismic Lateral and Story Forces

	Seismic UBC 1997 Lateral Story Forces (k)		
Story	Etabs F(x,y)	Calc F(x,y)	Etabs Story Shear (k)
PENT4	195.91	323.25	962.81
PENT3	238.94	308.98	1201.75
PENT2	396.48	503.33	1598.23
PENT1	413.17	478.95	2011.40
10TH	392.14	454.57	2403.54
9TH	371.11	430.19	2774.65
8TH	350.08	405.82	3124.73
7TH	329.05	381.44	3453.78
6TH	308.02	357.06	3761.80
5TH	286.99	332.68	4048.79
4TH	265.96	308.31	4314.75
3RD	244.93	283.93	4559.68
2ND	223.90	259.55	4783.58
1ST	202.87	235.17	4986.45
P8	382.49	655.21	5368.94
P7	387.60	487.01	5756.54
P6	320.02	417.44	6076.56
P5	266.69	347.87	6343.25
P4	213.35	278.29	6556.60
P3	160.01	208.72	6716.61
P2	106.67	139.15	6823.28
LOBBY	53.34	69.57	**6876.62**

Recall from Table 2, my previous calculated V = 8456 kips. A summary of the different parameters are:

Table 5: Hand Calculation and Etabs Difference Summary

	Etabs	Calc
Weight (k)	88652.29	95132.58
Period T	1.547	1.35
V (k)	6876.51	8456.23

4.2 Rigidity Calculations

To distribute the calculated story shear forces to each of the 28 walls, relative rigidity of each wall was calculated in both directions. Because of symmetry in the floor plan above the 8th level I was able reduce the model to 10 walls and just account for the number of wall, N, per floor represented by each label. This was allowed to be done for each wall that had the same dimensional properties and length. Also, the walls had to be at the same distance from the center of mass of the floor for later calculations of torsional shear. Crossed-out values in the table show that the wall is not present in that floor. This was accounted for in rigidity calculations. However, changes in length per floor were not accounted for. Conservatively, the shorter length was used.

Table 6: Wall Rigidities North – South Direction

R = $Et/(4*(H/L)^3 + 3(H/L))$						N-S DIRECTION						
R/E	**WALL Label**	A	B	C	D	E	F	L	M	P	R	N-S SUM
	N (per floor)	4.00000	4.00000	4.00000	4.00000	4.00000	2.00000	4.00000	4.00000	2.00000	2.00000	
	t(in)	12.00000	12.00000	12.00000	12.00000	12.00000	12.00000	10.00000	10.00000	10.00000	10.00000	
LEVEL	**H(FT)/L (FT)**	34.00000	13.00000	4.00000	5.00000	4.00000	38.00000	10.00000	18.00000	25.00000	20.00000	
PENT4	222.62000	~~0.00000~~	0.00005	0.00000	0.00000	0.00000	0.00122	0.00002	0.00011	0.00029	0.00015	**0.00405**
PENT3	212.79000	~~0.00000~~	0.00006	0.00000	0.00000	0.00000	0.00139	0.00002	0.00013	0.00033	0.00017	**0.00464**
PENT2	202.96000	~~0.00000~~	0.00007	0.00000	0.00000	0.00000	0.00160	0.00002	0.00014	0.00038	0.00020	**0.00533**
PENT1	193.13000	~~0.00000~~	0.00008	0.00000	0.00000	0.00000	0.00185	0.00003	0.00017	0.00045	0.00023	**0.00618**
10TH	183.30000	~~0.00000~~	0.00009	0.00000	0.00001	0.00000	0.00216	0.00003	0.00020	0.00052	0.00027	**0.00721**
9TH	173.47000	~~0.00000~~	0.00010	0.00000	0.00001	0.00000	0.00254	0.00004	0.00023	0.00061	0.00032	**0.00848**
8TH	163.64000	~~0.00000~~	0.00012	0.00000	0.00001	0.00000	0.00301	0.00005	0.00027	0.00073	0.00038	**0.01008**
7TH	153.81000	~~0.00000~~	0.00015	0.00000	0.00001	0.00000	0.00360	0.00006	0.00033	0.00088	0.00045	**0.01209**
6TH	143.98000	~~0.00000~~	0.00018	0.00001	0.00001	0.00001	0.00437	0.00007	0.00040	0.00107	0.00055	**0.01467**
5TH	134.15000	~~0.00000~~	0.00023	0.00001	0.00001	0.00001	0.00536	0.00009	0.00050	0.00131	0.00068	**0.01804**
4TH	124.32000	~~0.00000~~	0.00028	0.00001	0.00002	0.00001	0.00667	0.00011	0.00062	0.00164	0.00085	**0.02252**
3RD	114.49000	~~0.00000~~	0.00036	0.00001	0.00002	0.00001	0.00844	0.00014	0.00079	0.00209	0.00109	**0.02860**
2ND	104.66000	~~0.00000~~	0.00047	0.00001	0.00003	0.00001	0.01089	0.00018	0.00104	0.00272	0.00142	**0.03704**
1ST	94.83000	~~0.00000~~	0.00064	0.00002	0.00004	0.00002	0.01436	0.00024	0.00139	0.00363	0.00189	**0.04911**
P8	85.00000	0.01429	0.00088	0.00003	0.00005	0.00003	0.01943	0.00034	0.00191	0.00498	0.00261	**0.12409**
P7	70.00000	0.02434	~~0.00000~~	0.00005	0.00009	0.00005	~~0.00000~~	0.00060	0.00337	0.00866	0.00458	**0.14047**
P6	60.00000	0.03666	~~0.00000~~	0.00007	0.00014	0.00007	~~0.00000~~	0.00094	0.00527	0.01333	0.00712	**0.21358**
P5	50.00000	0.05837	~~0.00000~~	0.00013	0.00025	0.00013	~~0.00000~~	0.00162	0.00886	0.02193	0.01190	**0.34505**
P4	40.00000	0.09957	~~0.00000~~	0.00025	0.00048	0.00025	~~0.00000~~	0.00311	0.01648	0.03934	0.02193	**0.60311**
P3	30.00000	0.18536	~~0.00000~~	0.00058	0.00113	0.00058	~~0.00000~~	0.00712	0.03543	0.07927	0.04630	**1.17202**
P2	20.00000	0.38777	~~0.00000~~	0.00194	0.00373	0.00194	~~0.00000~~	0.02193	0.09448	0.18735	0.11905	**2.65996**
LOBBY	10.00000	1.01613	~~0.00000~~	0.01429	0.02632	0.01429	~~0.00000~~	0.11905	0.35423	0.57234	0.41667	**8.15520**

Table 7: Shearwall Rigidity Calculation for East-West Direction

R = Et/(4*(H/L)^3 + 3(H/L))		EAST-WEST DIRECTION								
R/E	WALL Label	V1	V2	V3	V4	V5	V6	V17	V18	E-W SUM
	N (per floor)	2.00000	4.00000	4.00000	2.00000	2.00000	2.00000	4.00000	4.00000	
	t(in)	10.00000	10.00000	10.00000	10.00000	10.00000	10.00000	12.00000	12.00000	
LEVEL	H(FT)/L (FT)	10.00000	3.50000	3.50000	10.00000	10.00000	10.00000	18.00000	24.00000	
PENT4	222.62000	0.00000	0.00000	0.00000	0.00002	0.00002	0.00002	0.00013	0.00031	**0.00100**
PENT3	212.79000	0.00000	0.00000	0.00000	0.00002	0.00002	0.00002	0.00015	0.00036	**0.00114**
PENT2	202.96000	0.00000	0.00000	0.00000	0.00002	0.00002	0.00002	0.00017	0.00041	**0.00248**
PENT1	193.13000	0.00000	0.00000	0.00000	0.00003	0.00003	0.00003	0.00020	0.00047	**0.00288**
10TH	183.30000	0.00000	0.00000	0.00000	0.00003	0.00003	0.00003	0.00024	0.00055	**0.00336**
9TH	173.47000	0.00000	0.00000	0.00000	0.00004	0.00004	0.00004	0.00028	0.00065	**0.00396**
8TH	163.64000	0.00000	0.00000	0.00000	0.00005	0.00005	0.00005	0.00033	0.00078	**0.00472**
7TH	153.81000	0.00000	0.00000	0.00000	0.00006	0.00006	0.00006	0.00040	0.00093	**0.00567**
6TH	143.98000	0.00000	0.00000	0.00000	0.00007	0.00007	0.00007	0.00048	0.00113	**0.00690**
5TH	134.15000	0.00000	0.00000	0.00000	0.00009	0.00009	0.00009	0.00060	0.00140	**0.00851**
4TH	124.32000	0.00000	0.00000	0.00000	0.00011	0.00011	0.00011	0.00075	0.00175	**0.01065**
3RD	114.49000	0.00000	0.00000	0.00001	0.00014	0.00014	0.00014	0.00095	0.00223	**0.01358**
2ND	104.66000	0.00000	0.00000	0.00001	0.00018	0.00018	0.00018	0.00124	0.00290	**0.01769**
1ST	94.83000	0.00000	0.00000	0.00001	0.00024	0.00024	0.00024	0.00166	0.00387	**0.02362**
P8	85.00000	0.00034	0.00001	0.00001	0.00034	0.00034	0.00034	0.00230	0.00531	**0.03323**
P7	70.00000	0.00060	0.00003	0.00000	0.00000	0.00060	0.00060	0.00405	0.00926	**0.05693**
P6	60.00000	0.00094	0.00004	0.00000	0.00000	0.00094	0.00094	0.00632	0.01429	**0.08827**
P5	50.00000	0.00162	0.00007	0.00000	0.00000	0.00162	0.00162	0.01063	0.02357	**0.14681**
P4	40.00000	0.00311	0.00014	0.00000	0.00000	0.00311	0.00311	0.01978	0.04252	**0.26840**
P3	30.00000	0.00712	0.00033	0.00000	0.00000	0.00712	0.00712	0.04252	0.08649	**0.56007**
P2	20.00000	0.02193	0.00109	0.00000	0.00000	0.02193	0.02193	0.11337	0.20769	**1.42021**
LOBBY	10.00000	0.11905	0.00818	0.00000	0.00000	0.11905	0.11905	0.42507	0.64962	**5.04580**

4.3 Distribution to Shearwall System

The Etabs model was originally intended to more accurately portray the shear and bending stress in each wall. I was especially interested in the P8 level transitions floor results that I had failed to model in my hand calculation. These results proved to be a challenge even for modeling software. The discontinuity problem at the P8 level is still a problem even when the computer model was used. I expected an increase in shear stresses at this P8 level. This was expected for the following reasons:

1. Increase stiffness at this level by the added walls and
2. The need to transfer stresses of 8 different 135’ high, 20’ long walls into reduced 14’ long walls.
3. Twice the increase in building story area at this level and below. This will have an effect and increase the relative weight and seismic forces from this level and below.

4.3-1 Wall Shear Results and Comparison North-South Direction, Case Study M

When we look at the results, the shear stresses spike considerably. In the north-south direction, where there is no change in building width the spikes are very realistic. The numbers are much higher at the transition level than my calculated value. However, the overall maximum shear design value is just a little higher than my calculated values. They can be explained by the transition from 24' long wall above this level into 18' long walls below this level.

Table 8: Shearwall shear stress values hand calculation vs Etabs at each level

N-S WALL LABEL		M				
STORY	STORY SHEAR (K)	WALL R	STORY R	R PROPORTION	CALC WALL SHEAR (K)	ETABS OUTPUT (K)
PENT4	963	0.00011	0.00405	0.02703	26.03	21.78
PENT3	1202	0.00013	0.00464	0.02706	32.52	25.49
PENT2	1598	0.00014	0.00533	0.02709	**43.30**	**-6.88**
PENT1	2011	0.00017	0.00618	0.02713	54.56	17.11
10TH	2404	0.00020	0.00721	0.02717	65.30	24.85
9TH	2775	0.00023	0.00848	0.02722	75.51	29.03
8TH	3125	0.00027	0.01008	0.02727	85.22	32.54
7TH	3454	0.00033	0.01209	0.02734	94.42	35.73
6TH	3762	0.00040	0.01467	0.02742	103.15	38.96
5TH	4049	0.00050	0.01804	0.02752	111.42	41.91
4TH	4315	0.00062	0.02252	0.02764	119.27	46.7
3RD	4560	0.00079	0.02860	0.02780	126.74	51.13
2ND	4784	0.00104	0.03704	0.02799	133.91	86.57
1ST	4986	0.00139	0.04911	0.02825	140.86	138.68
P8	5369	0.00191	0.12409	0.01543	**82.82**	**265.78**
P7	5757	0.00337	0.14047	0.02403	**138.30**	**30.66**
P6	6077	0.00527	0.21358	0.02467	149.92	13.02
P5	6343	0.00886	0.34505	0.02567	162.86	-4.67
P4	6557	0.01648	0.60311	0.02733	179.17	-3.33
P3	6717	0.03543	1.17202	0.03023	203.06	-3.09
P2	6823	0.09448	2.65996	0.03552	242.36	-3.09
LOBBY	6877	0.35423	8.15520	0.04344	298.69	-4.69
				MAX SHEAR VALUE	**298.69**	**265.78**
	FLOOR WITH DISCONTINUITIES IN WALL LAYOUT AND/OR FLOOR DIAGRAPHM AREA - AFFECTING OVERALL STIFFNESS IN THIS DIRECTION					

.The overall maximum stress stays relatively the same for both methods (green in Table 8). I believe this is because is this direction the building width stays the same and the amount of shearwalls, 28 total, is enough to properly redistribute the changes is shear stress.. The large jump at the P8 transition level can also result from the two added 38' long, 12" wide wall in the core that terminate at this level from 222' feet in the air. These walls carry shear stresses from the top 14 stories and at this level they must be transmitted through the other shear walls into the lower 10 stories to the foundation. This increases the shear stresses in the surrounding walls, like this one (See Figure 5).

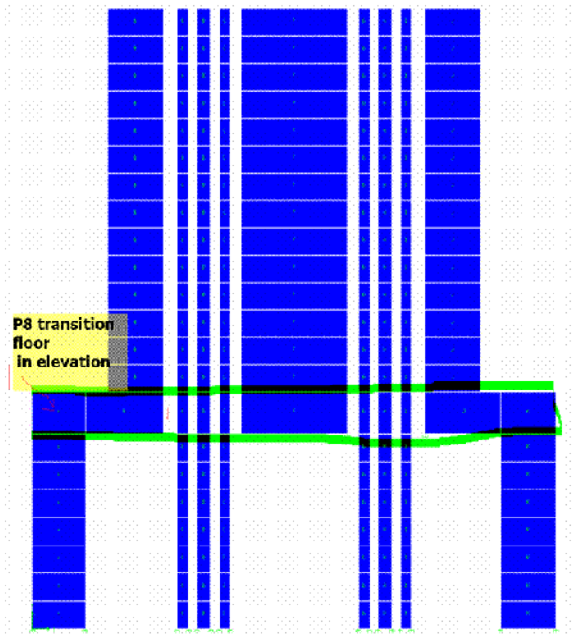

Figure 5: Elevation through building's core shear wall system transition

4.3-2 Wall Shear Results and Comparison East-West Direction, Case Study V18

When we take a look at the east-west direction shear results the spikes are a little higher than credible. The shear values are very large in the P8 transition level. It is in this direction that we have the increased building depth. We also only have 4 walls acting to resist lateral load in this direction. The increased stresses must be taken directly by these 4 walls.

Table 9: Shear stress at critical wall in East – West Direction. Hand calculated values compared to Etabs

E-W WALL LABEL		V18				
STORY	STORY SHEAR (K)	WALL R	STORY R	R PROPORTION	CALC WALL SHEAR (K)	ETABS OUTPUT (K)
PENT4	963	0.00031	0.00100	0.31040	298.85	69.48
PENT3	1202	0.00036	0.00114	0.31034	372.96	106.56
PENT2	1598	0.00041	0.00248	0.16473	263.28	107.91
PENT1	2011	0.00047	0.00288	0.16470	331.28	141.44
10TH	2404	0.00055	0.00336	0.16467	395.78	187.27
9TH	2775	0.00065	0.00396	0.16462	456.77	223.03
8TH	3125	0.00078	0.00472	0.16457	514.25	257.01
7TH	3454	0.00093	0.00567	0.16451	568.20	288.51
6TH	3762	0.00113	0.00690	0.16444	618.60	319.25
5TH	4049	0.00140	0.00851	0.16435	665.44	349.32
4TH	4315	0.00175	0.01065	0.16425	708.68	380.08
3RD	4560	0.00223	0.01358	0.16411	748.28	405.51
2ND	4784	0.00290	0.01769	0.16393	784.18	425.64
1ST	4986	0.00387	0.02362	0.16370	816.28	337.42
P8	5369	0.00531	0.03323	0.15980	**857.94**	**-2034.60**
P7	5757	0.00926	0.05693	0.16264	**936.27**	**-206.1**
P6	6077	0.01429	0.08827	0.16184	983.44	-97.73
P5	6343	0.02357	0.14681	0.16057	1018.56	-72.38
P4	6557	0.04252	0.26840	0.15842	1038.69	-58.68
P3	6717	0.08649	0.56007	0.15442	1037.19	-52.27
P2	6823	0.20769	1.42021	0.14624	997.84	-48.36
LOBBY	6877	0.64962	5.04580	0.12875	885.33	-42.34
				OVERALL MAX SHEAR VALUE	**1038.69**	**2034.6**
	MAX SHEAR VALUE IGNORING DISCONTINUITY (ABOVE P8)				**816.28**	**425.64**
	FLOOR WITH DISCONTINUITIES IN WALL LAYOUT AND/OR FLOOR DIAGRAPHM AREA - AFFECTING OVERALL STIFFNESS IN THIS DIRECTION					

It is in this direction that the structural concerns are present. As can be seen, there is a very substantial spike at the P8 level in the shear experienced by this wall. I expected the shear to be larger at this level, but these values are a little too high. I looked at two walls in detail; the one we looked at is V18 and is in the perimeter of the building. The second wall, V7 is in the core of the building.

4.4 Simplified model calculations and Etabs Results Conclusion

As a structural designer I am very confident in that my hand calculations are at least conservative for every level constituting the apartment complex. This includes every level above the P8 transition level. I did them and verified them. When I tried to get more accurate results by taking into account the levels at and below the parking garage by using Etabs, I am not sure that the model is properly distributing the stresses in the east-west direction at the transition level. I devoted much time in trying to adjust the model to get better results. The model was also looked at without the parking slab extension at the P8 level by assuming the there was an expansion joint at the wall. The results did not improve. For purposes of this assignment, the rest of the report I will only look at the results above the P8 level. I will perform design checks from the drawing just of the condominium tower. That is to say, I will not consider the stories below the P8 level for design until a more accurate model is obtained.

5. Design Checks

5.1 Summary of Results and Critical Load Combination

Table 10: Service Shear, Bending and Axial Loads for Seismic, Dead and Live and Load Combinations North-South Direction

North - South Direction	PIER M'				PIER P'			
Story	V (K)	M (FT-k)	Pdead (K)	Plive (k)	V (K)	M (FT-k)	Pdead (K)	Plive (k)
PENT4	21.78	0.00	93.33	11.70	0.96	0.00	85.62	11.70
PENT3	25.49	214.08	201.67	29.70	23.94	9.40	184.25	27.30
PENT2	-6.88	464.67	310.00	47.70	114.56	244.74	282.87	42.90
PENT1	17.11	397.04	418.33	65.70	99.67	1370.83	381.50	58.50
10TH	24.85	565.24	526.66	83.70	106.99	2350.58	480.12	74.10
9TH	29.03	809.56	635.00	101.70	116.40	3402.26	578.75	89.70
8TH	32.54	1094.89	743.33	119.70	127.64	4546.48	677.37	105.30
7TH	35.73	1414.73	851.66	137.70	138.89	5801.22	775.99	120.90
6TH	38.96	1765.94	959.99	155.70	150.28	7166.54	874.62	136.50
5TH	41.91	2148.92	1068.33	173.70	161.29	8643.82	973.24	152.10
4TH	46.70	2560.89	1176.66	191.70	173.40	10229.31	1071.87	167.70
3RD	51.13	3019.96	1284.99	209.70	181.98	11933.80	1170.49	183.30
2ND	86.57	3522.53	1393.32	227.70	201.45	13722.62	1269.12	198.90
1ST	**138.68**	**4373.51**	1501.66	245.70	**130.43**	**15702.86**	1367.74	214.50
MAX ABOVE P8	**139**	**4374**	**1502**	**246**	**201**	**15703**	**1368**	**215**
P8	**265.78**	**5736.73**	1609.99	263.70	-441.13	5002.93	1466.36	230.10
P7	30.66	-330.10	1718.32	338.70	177.60	-1614.02	1564.99	295.10
P6	13.02	-23.55	1826.66	361.20	-28.10	161.94	1663.61	314.60
P5	-4.67	106.62	1934.99	383.70	6.33	-119.03	1762.24	334.10
P4	-3.33	59.92	2043.32	406.20	1.45	-55.69	1860.86	353.60
P3	-3.09	26.63	2151.65	428.70	3.49	-41.22	1959.48	373.10
P2	-3.09	-4.22	2259.99	451.20	4.54	-6.33	2058.11	392.60
LOBBY	-4.69	-35.11	2368.32	473.70	8.54	39.08	2156.73	412.10
ABSMAX	**266**	**5737**	**2368**	**474**	**441**	**13723**	**2157**	**412**
LOAD COMBINATIONS	**Shear (k)**	**Bending (ft-k)**	**Axial (k)**		**Shear (k)**	**Bending (ft-k)**	**Axial (k)**	
1.4D + 1.7L	0	0	4121		0	0	3720	
1.50D + 0.55L + 1.32Eh	**351**	**4650**	**2388**		**582**	**18114**	**2170**	
1.50D + 0.55L -1.32Eh	-351	-4650	2388		-582	-18114	2170	
0.79D + 1.32Eh	351	4650	1186		582	18114	1704	
0.79D - 1.32Eh	-351	-4650	1186		-582	-18114	1704	

Table 11: Service Shear, Bending and Axial Loads due to Seismic, Dead and Live and Load Combination East-West Direction

East - West Direction	PIER V17				PIER V18			
Story	V (K)	M (FT-K)	Pdead (K)	Plive (k)	V (K)	M (FT-k)	Pdead (K)	Plive (k)
PENT4	65.51	0.00	65.26	9.67	69.48	0.00	53.26	5.82
PENT3	85.06	643.95	138.62	21.14	106.56	683.03	110.57	15.49
PENT2	-3.47	1480.10	211.98	30.86	107.91	1730.52	167.89	23.19
PENT1	59.82	1445.95	285.34	40.58	141.44	2791.27	225.20	28.93
10TH	90.07	2033.96	358.70	50.30	187.27	4181.63	282.51	33.79
9TH	110.66	2919.33	432.05	60.02	223.03	6022.47	339.82	38.65
8TH	127.82	4007.16	505.41	69.74	257.01	8214.81	397.13	43.51
7TH	143.49	5263.65	578.77	79.46	288.51	10741.20	454.45	48.37
6TH	158.98	6674.19	652.13	89.18	319.25	13577.28	511.76	53.23
5TH	174.62	8236.93	725.49	98.90	349.32	16715.53	569.07	58.09
4TH	194.60	9953.48	798.85	108.62	380.08	20149.33	626.38	62.95
3RD	218.70	11866.37	872.21	118.34	405.51	23885.52	683.69	67.81
2ND	284.60	14016.18	945.57	128.06	**425.64**	27871.73	741.01	72.67
1ST	**379.50**	**16813.83**	1018.93	137.78	337.42	**32055.77**	798.32	77.53
MAX ABOVE P8	**380**	**16814**	**1019**	**138**	**426**	**32056**	**798**	**78**
P8	**-1356.13**	**20544.35**	1092.29	147.50	**-2034.60**	**35372.58**	855.63	82.39
P7	48.47	202.34	1165.64	188.00	-206.10	4853.58	912.94	102.64
P6	0.30	686.99	1239.00	200.15	-97.73	2792.56	970.25	108.71
P5	-22.64	690.04	1312.36	212.30	-72.38	1815.28	1027.57	114.79
P4	-24.61	463.65	1385.72	224.45	-58.68	1091.52	1084.88	120.86
P3	-24.98	217.54	1459.08	236.60	-52.27	504.68	1142.19	126.94
P2	-25.61	-32.21	1532.44	248.75	-48.36	-18.05	1199.50	133.01
LOBBY	-28.05	-288.35	1605.80	260.90	-42.34	-501.66	1256.81	139.09
ABSMAX	**1356**	**20544**	**1606**	**261**	**2035**	**35373**	**1257**	**139**
LOAD COMBINATIONS	**Shear (k)**	**Bending (ft-k)**	**Axial (k)**		**Shear (k)**	**Bending (ft-k)**	**Axial (k)**	
1.4D + 1.7L	0	0	1661		0	0	1249	
1.50D + 0.55L + 1.32Eh	**501**	**22194**	**1604**		**562**	**42314**	**1240**	
1.50D + 0.55L -1.32Eh	-501	-22194	1604		-562	-42314	1240	
0.79D + 1.32Eh	501	22194	805		562	42314	631	
0.79D - 1.32Eh	-501	-22194	805		-562	-42314	631	

From the results we can see that the controlling load case (green on Tables 10 and 11) is

1.50D + 0.55L + 1.32Eh.

This combination results from UBC 1997 Load Combination Equation 4:

$1.1(1.2D + f_1L + f_2S + 1.1E)$,

Where, f1 = 0.5 for live loads < 100psf

S = 0 in Puerto Rico

E = pEh + Ev,

$p = 2 – 20 / (r_{max}* \bullet Ag) = 1.2$,

for $r_{max} = 0.2$

$Ag = 15870\ ft^2$ à ground floor area

Ev = 0.5CaID = 0.18D

5.2 Shear Check

Once the design Vu was obtained from the critical load combinations above (Tables 10 and 11), shear was strength was checked based on UBC 1997 and ACI 318 – 95 as specified in the drawings.

$$V_c = 2*A_{cv} \bullet fc'$$

Acv – net area bounded by web thickness and the length in the direction of analysis

If • Vc < Vu, two curtain of web reinforcement are required

If reinforcement is provided,

$$\bullet\ V_n = V_c + V_s$$

The upper shear strength of the wall is given by:

$$\bullet(8)(A_{cv})\bullet fc', \text{ where } \bullet = 0.85$$

Minimum reinforcement is given by,

$$\bullet \text{min} = 0.0025$$

5.2-1 North – South (Refer to Figure 1, pg 2, for wall label M', P' references)

Table 12: Shear Strength Check for North-South Walls

Shear Design Check	**M'**	**P'**
Lwall (ft)	18	25
twall (in)	10	10
Reinf Ratio Prov, pn	0.0028	0.0028
fc' (psi)	4000	4000
pn > pmin = 0.0025	Yes	Yes
Acv (in2)	2160	3000
Vu (k)	**351**	**582**
Lower Strenght Vc	273	379
Shear Reinforcement	Required	Required
Max øVu	929	1290
øVn	**541**	**751**

Greater than Vu à Good!

5.2-2East – West (Refer to Figure 1, pg 2, for wall label V17, V18 references)

Table 13: Shear Strength Check for East-West Walls

Shear Design Check	V17	V18
Lwall (ft)	18	24
twall (in)	12	12
Reinf Ratio Prov, pn	0.0028	0.0028
fc' (psi)	4000	4000
pn > pmin = 0.0025	Yes	Yes
Acv (in2)	2592	3456
Vu (k)	**501**	**562**
Lower Strenght Vc	328	437
Shear Reinforcement	Required	Required
Max øVu	1115	1486
øVn	**649**	**865**

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Minimum reinforcement provided in each wall is adequate for strength and meet temperature and shrinkage requirements.

5.3 Overturning Moment

It is important that the moment created by the lateral forces on the base of the building can be resisted to prevent the building from uplift. The method used compares the overturning moment created by the lateral force to the axial force on the foundation by the building weight.

Maximum overturning moment = 1076958 ft-k (Table 3)

M / Minimum Width building = 1076958 / 120' = 8974.56 kips

Weight building / 4 (assumes 1 support at each corner) = 95132.58 k / 4 = 23783 kips

23783 kips >> 8974.56 kips à Overturning is not a concern as expected due to the large weight of the building

5.4 Bending and Axial

The bending moment caused by the lateral forces in the shearwalls must be resisted by compression in one side of the wall and tension in the other, much like a beam would. Therefore, the bending moment can be effectively converted into a couple by placing an axial load at each end of the wall. The magnitude of the axial load = Mu / L_{wall} . This side that is in compression will have an added axial force from the dead and live axial loads plus this axial load due to the bending on the wall ends. We design this wall ends as the Boundary Zone. If the total compressive load in the wall is larger than

$$0.1(f_c')A_g$$

reinforcement must be provided in each wall

The calculated maximum compressive strength of the wall is given by:

$$P_o = 0.85(f_c')(A_g - A_{st}) + f_y A_{st}$$

A_{st} is the amount of reinforcement provided in the boundary zone.

The bearing length of the boundary zone can be approximated by a ratio of the total load Pu in the wall to the maximum compressive strength, Po.

In accordance with code reference 1921.6.6.6:

Boundary zones must be provided at each end a distance varying linearly from $0.25l_w$ to $0.15l_w$ for Pu between 0.35Po and 0.15Po. The boundary zone shall have a minimum length of $0.15l_w$.

For each two shear walls in each direction, the required Boundary Zone length at each end of the wall was calculated and compared to the actual design by using the given amount of reinforcement as A_{st}.

5.4-1 North – South (Refer to Figure 1, pg 2, for wall label references)

Bending and Axial	**M'**	**P'**
Pbending	258.32	724.55
Paxial	2387.62	2169.58
Putotal	2645.94	2894.14
0.1f'c*Ag	864.00	1200.00
Boundary (in)	32.00	32.00
As prov (in^2)	16.84	16.84
Total As	22.888	25.24
Po max	8639.46	11628.58
Pu/Po	0.31	0.25
Pu/Po*lw (in)	66.15	74.66
Req B.Z. side (in)	33	37

Boundary Zone not adequate

5.4-2 East – West (Refer to Figure 1, pg 2, for wall label references)

Bending and Axial	V17	V18
Pbending	1233.01	1763.07
Paxial	1604.17	1240.12
Putotal	2837.18	3003.19
0.1f'c*Ag	1036.80	1382.40
Boundary (in)	32.00	37.00
As prov (in^2)	12.32	12.32
Total As	19.5776	21.9968
Po max	9920.89	12995.42
Pu/Po	0.29	0.23
Pu/Po*lw (in)	61.77	66.56
Req B.Z. side (in)	31	33

Boundary Zone is adequate

5.5 Torsion

Torsional shear was checked for all walls in the ground floor label. The rigidities and wall configurations at that label were used. As was expected, the walls that are farthest apart from the core experience greater torsional shear. However, the torsional shear is small compared to the direct shear values. This is specially true in the north-south direction where there are 28 shear walls and they are arranged symmetrically around the core.

5.5-1 North – South

Table 14

N-S DIRECTION

WALL Label	A	B	C	D	E	F	L	M	P	R
Base Shear (k)	6880.00									
C.G (ft)	81.00	81.00	81.00	81.00	81.00	81.00	81.00	81.00	81.00	81.00
Wall x (ft)	76.00	76.00	76.00	76.00	76.00	76.00	54.00	54.00	27.00	27.00
x (C.G. to Wall) ft	5.00	5.00	5.00	5.00	5.00	5.00	27.00	27.00	54.00	54.00
R	1.02	0.00	0.01	0.03	0.01	0.00	0.12	0.35	0.57	0.42
Rx	5.08	0.00	0.07	0.13	0.07	0.00	3.21	9.56	30.91	22.50
Center of Rigidity (x)	52.76									
Torsional Moment	30329.57	ft-k								
x (C.R to wall) ft	23.24	23.24	23.24	23.24	23.24	23.24	1.24	1.24	25.76	25.76
Rx2	2744.01	0.00	38.58	71.06	38.58	0.00	4.94	14.70	20509.20	14930.69
Rx	23.61	0.00	0.33	0.61	0.33	0.00	0.15	0.44	14.74	10.73
Rx/SumRx2	0.00064	0.00000	0.00001	0.00002	0.00001	0.00000	0.00000	0.00001	0.00040	0.00029
Torsional Shear	**19.29**	0.00	0.27	0.50	0.27	0.00	0.12	0.36	**12.04**	**8.77**

Largest torsional shear experienced by wall A, the longest wall and farthest from the centroid.

5.5-2 East – West

In the east-west direction, the walls labels as V1 – V 17 in Table 15 all represent that walls vertical walls around the core. As it is expected, these walls are close to the center and experience low torsional shear. Wall V18 located farther away and one of the 4 other main resisting lateral walls in this direction experiences the largest torsional shear.

Table 15

				EAST-WEST DIRECTION					
WALL Label		**V1**	**V2**	**V3**	**V4**	**V5**	**V6**	**V17**	**V18**
Base Shear (k)									
C.G (ft)		60.00	60.00	60.00	60.00	60.00	60.00	60.00	60.00
Wall x (ft)		0.00	12.00	16.00	28.00	32.00	52.00	30.00	45.00
x (C.G. to Wall) ft		60.00	48.00	44.00	32.00	28.00	8.00	30.00	15.00
R		0.12	0.01	0.00	0.00	0.12	0.12	0.43	0.65
Rx	71.54	7.14	0.39	0.00	0.00	3.33	0.95	12.75	9.74
Center of Rigidity (x)		36.17							
Torsional Moment									
x (C.R to wall) ft		36.17	24.17	20.17	8.17	4.17	15.83	6.17	45.00
Rx2		155.73	4.78	0.00	0.00	2.07	29.84	16.17	1315.49
Rx		4.31	0.20	0.00	0.00	0.50	1.88	2.62	29.23
Rx/SumRx2		0.00012	0.00001	0.00000	0.00000	0.00001	0.00005	0.00007	0.00079
Torsional Shear		3.52	0.16	0.00	0.00	0.41	1.54	2.14	**23.88**

5.6 Deflections Building

Drift was calculated based on the Etabs outputs for story drifts. The critical drift was caused by seismic along the east-west direction. Each story drift was calculated and added. The overall drift is 7.9”. This is a little higher than the H / 400 limit of 7”.

Story	Item	Load	DriftX	Story Height (in)	Story Displacement (in)
PENT4	Max Drift X	EQX	0.005653	117.96	0.66682788
PENT3	Max Drift X	EQX	0.005628	117.96	0.66387888
PENT2	Max Drift X	EQX	0.005547	117.96	0.65432412
PENT1	Max Drift X	EQX	0.005494	117.96	0.64807224
10TH	Max Drift X	EQX	0.005406	117.96	0.63769176
9TH	Max Drift X	EQX	0.005277	117.96	0.62247492
8TH	Max Drift X	EQX	0.005099	117.96	0.60147804
7TH	Max Drift X	EQX	0.004867	117.96	0.57411132
6TH	Max Drift X	EQX	0.004574	117.96	0.53954904
5TH	Max Drift X	EQX	0.004213	117.96	0.49696548
4TH	Max Drift X	EQX	0.003782	117.96	0.44612472
3RD	Max Drift X	EQX	0.00327	117.96	0.3857292
2ND	Max Drift X	EQX	0.002675	117.96	0.315543
1ST	Max Drift X	EQX	0.001961	117.96	0.23131956
P8	Max Drift X	EQX	0.000991	180	0.17838
P7	Max Drift X	EQX	0.000284	120	0.03408
P6	Max Drift X	EQX	0.000092	120	0.01104
P5	Max Drift X	EQX	0.000329	120	0.03948
P4	Max Drift X	EQX	0.000441	120	0.05292
P3	Max Drift X	EQX	0.00044	120	0.0528
P2	Max Drift X	EQX	0.000336	120	0.04032
LOBBY	Max Drift X	EQX	0.000134	120	0.01608
Total Displacement					**7.90919** in
H	2820 in			H/400	**7.05** in