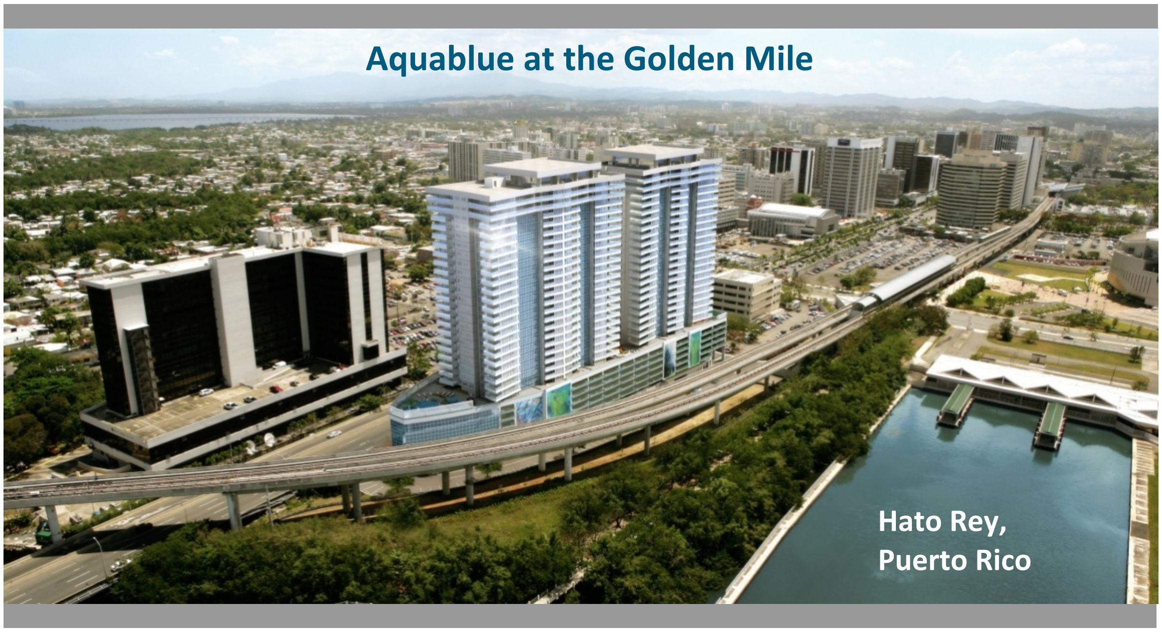
A Structural Engineering Analysis



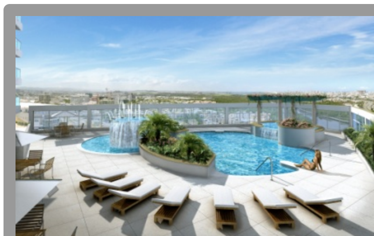
Lindsay Lynch

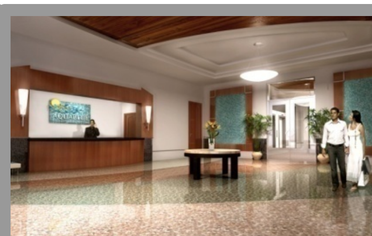
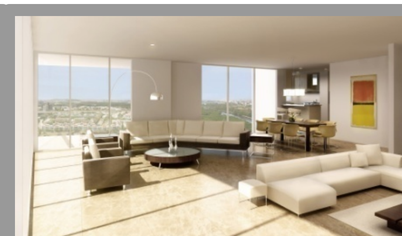
Integrated B.A.E./M.A.E. Program

Structural Option

Dr. Andres Lepage

22 April 2009



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Lindsay Lynch
Structural Option

The Pennsylvania State University

Architectural Engineering

Senior Thesis 2008 - 2009

General Building Information:

Building Name	Aquablue at the Golden Mile
Location	Hato Rey, Puerto Rico
Occupant Name	Managed by GB Realty
Occupancy Type	Luxury apartment buildings
Size	900,000 square feet
Number of Stories	31 stories above grade
Dates of Construction	February 2007 - August 2009
Estimated Cost	(not available)
Project Delivery Method	Design-Bid-Build

Primary Project Team:

Owners	Guiterrez-Latimer
	QB Construction
General Contractor	QB Construction
Construction Manager	Diaz & Associates P.S.C.
Architect	Gutierrez-Latimer C.S.P.
Landscape Architect	Raul Alvarez
Structural Engineer	DeSimone Consulting Engineers
Plumbing Engineer	Jorge Torres & Associates
Mechanical Engineer	Jose Luis Garcia & Associates
Electrical Engineer	Eduardo Chardon Casals

Architecture:

The site is long and narrow, so the overall building dimensions are about 120' x 490'. The first level will be used as a commercial area, and levels 2-6 will primarily function as a parking garage for the future residents. Level 7 is a communal space with a fitness center, meeting room, game room and outdoor pool. Above level 7, the building is separated into two roughly rectangular towers (about 90' x 180' each), and they are composed of one- and two-bedroom luxury apartments.

Structural Components:

Foundation	Drilled piles beneath a 10" reinforced concrete slab-on-grade at the ground level
Gravity System	Two-way, post-tensioned slabs at all levels supported by columns spaced anywhere from 25'-0" to 34'-0"
Lateral System	Two groups of concrete shear walls near the core of the building that extend up through the two residential towers
Roof	2-ply modified bitumen roofing system on top of 2" rigid roof insulation, supported structurally by a sloped concrete slab

CPEP Website:

www.engr.psu.edu/ae/thesis/portfolios/2009/lel145/

Executive Summary

The following report summarizes the two-semester thesis project for the 2008-2009 academic year, as required by Penn State's Architectural Engineering department. The subject of study was Aquablue at the Golden Mile, which is an actual building that is currently under construction in Hato Rey, Puerto Rico. After an initial investigation of the existing conditions, a more independent analysis was conducted for an extensive learning exercise.

In general, the project included an in-depth analysis and re-design of the structural system, as well as two smaller studies of other building related systems. After designing new shear walls for the lateral force resisting system, the most relevant topics for breadth studies were architecture and construction. The shear walls were initially proposed to be more efficient, but they had a few implications on the existing architectural design. Therefore, those changes were included in this report. Also, in order to effectively compare the original and new systems, a cost analysis was conducted to determine if any savings were to be realized.

The design process proved to be a very valuable exercise in practical engineering. In general, the project goals were met, as the analysis of the lateral force resisting system was more thorough than any other previous design experience. Also, the architectural breadth provided a better understanding of the potential difficulties of project coordination. For example, if an architect designs his/her building with a certain feature that cannot be altered, then the structural system has to work around that limitation, even if it is not the most efficient design. Based on achieving the project objectives, this independent design task was an overall success.

General Building Information

Aquablue at the Golden Mile is an approximately 280' high-rise apartment building in Hato Rey, Puerto Rico. It is located in an urban area, about two miles away from the San Juan Bay (fig. 1). The building size is about 900,000 total square feet, and there are 31 stories above grade. (Up to level 7, the typical floor area is about 51,900 ft^2. For the apartment towers, which are above level 7, the typical floor areas are 11600 and 14500 ft^2.) The ground level will be developed as a commercial area, and the rest of the floors up to level 7 will be used for both parking and office space. Level 7 is an indoor/outdoor public area for the apartment residents, and the floors above are private apartments. There is a sky lobby above the penthouse apartments.

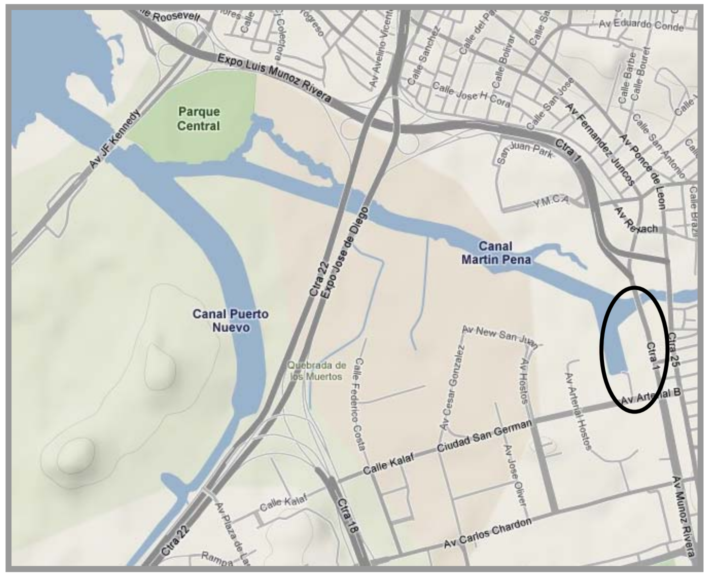

Figure 1 – Building Site (maps.google.com – Hato Rey Central, PR)

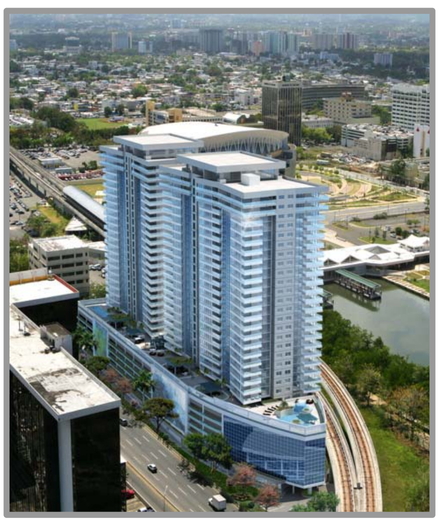

Figure 2 – Rendering of Aquablue

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

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

Description of Existing Structure

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

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

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

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

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

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

Table 1 – Concrete Strengths for Various Structural Elements

Typical Floor Framing Plans of Original Design

There are two typical floor plans in this building: one for the parking garage levels and one for the apartment levels. In figure 3 below, the gravity-based structural system for a typical parking level is highlighted in teal. The columns are supporting a two-way, flat plate, post-tensioned slab. Also shown in the figure below is the original lateral force resisting system of reinforced concrete shear walls concentrated toward the center of the floor plan. The most extensive shear wall system is at the base of the building, and the number and length of the walls decreases as the height above grade increases.

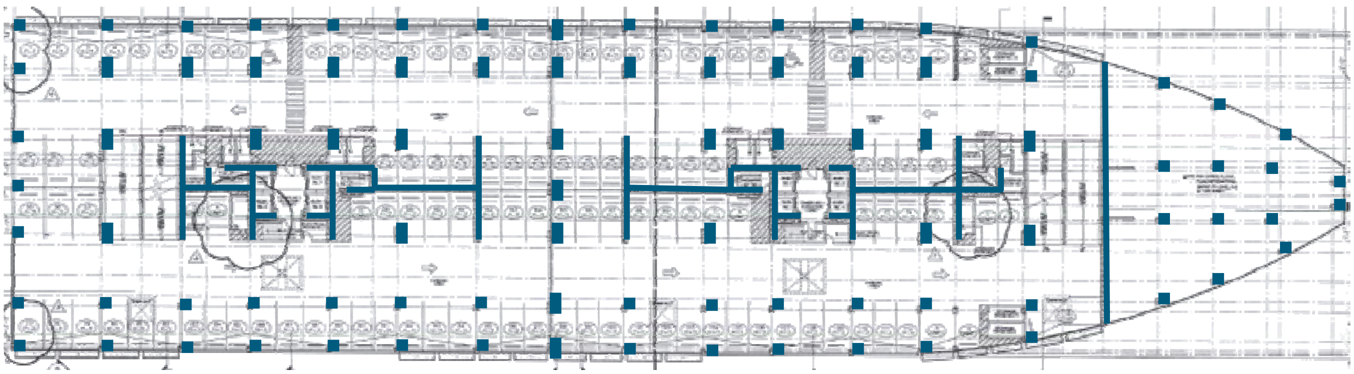

Figure 3 – Column and Shear Wall Layout for Typical Parking Garage Level

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

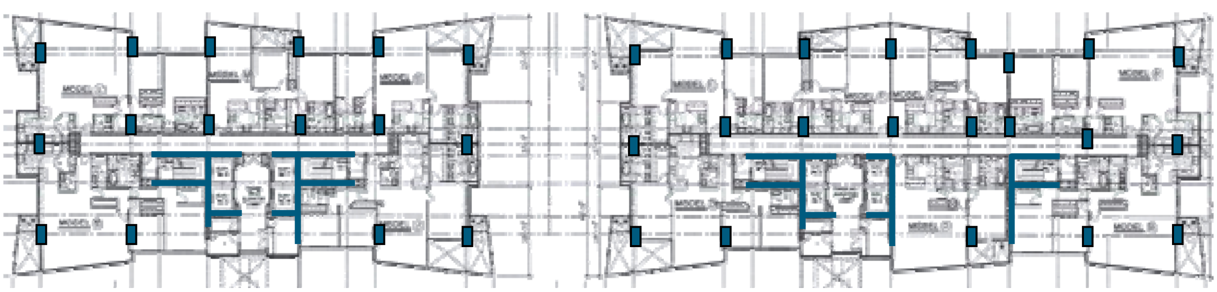

Figure 4 – Column and Shear Wall Layout for Typical Apartment Level

Description of Existing Lateral System

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

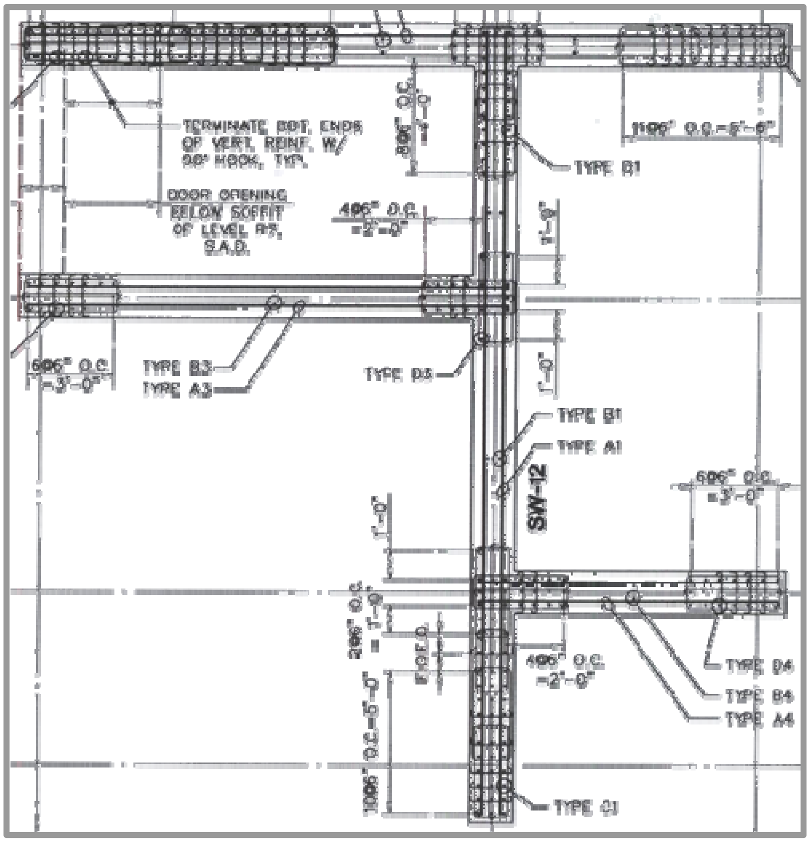

Figure 5 – Example of Shear Wall System (Levels 7-9)

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

Also, the boundary elements of the shear walls are relatively complex due to their intersection at the wall joints. Therefore, due to the multi-segment shear wall system and the difficulties it presents, the use of computer modeling is used to improve the efficiency of the design.

Codes and References

- General References:
  - ACI 318-08 (American Concrete Institute)
  - ASCE 7-05 (American Society of Civil Engineering)
  - IBC 2006 (International Building Code)

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

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

- Utilized Computer Programs
  - ETABS Nonlinear v9.2.0, copyright 2008 (Computers and Structures, Inc.)
  - pcaColumn v3.64, copyright 2005 (Portland Cement Association)

Background for Proposal and General Project Goals

The focus of this thesis project is on the lateral force resisting system of Aquablue at the Golden Mile. The location of this building in Hato Rey, Puerto Rico puts it at risk for earthquake and tsunami damage due to movement of the Puerto Rico Trench. Therefore, the seismic and wind loads are significant and provide a design challenge.

In particular, one of the difficulties in designing the structure of Aquablue occurs in the layout of the shear walls. First of all, the walls cannot be placed along the exterior of the building for architectural reasons. The tenants of this luxury apartment building need to have great views of the outside, and the shear walls would limit the amount of glass on the façade. Also, shear walls that are located at the exterior of the apartment buildings would run right through the middle of the parking garage, which would complicate its design. Therefore, the shear walls need to be located at the core of the building, specifically in two separate groups that can be continuous through each of the two towers.

Because of the specific architectural layout of apartments in the two towers of Aquablue, there is an additional challenge in the detailed layout of the shear walls. They cannot be placed just anywhere in the core of the building, so this limitation on the wall placement can result in some odd and inefficient shapes. These shear wall difficulties provide an opportunity for research and re-design.

The following is a list of general topics to be studied, and the goals for this project are to gain a more in-depth understanding and better engineering judgment in these areas:

- Detailed analysis of lateral loads
- Concrete shear wall design and the ACI 318-08 building code
- Computer modeling as a means of structural analysis
- Economic impact of structural design
- Architectural impact of structural design

Proposed Shear Wall Re-design

The shear walls will be completely re-designed to be more efficient and more cost effective. The location of the walls will remain at the core of the building (one group in each of the two towers), but the layout will be modified to be two I-shapes that are connected by coupling beams at the 'flanges'. These symmetric shapes would be much more efficient, and after a preliminary look at the existing layout, it seems like they could be incorporated into the building without too much of an impact on the architecture. The figure below gives a preliminary sketch of the potential solution for one of the towers. The dimensions are approximate in this initial sketch.

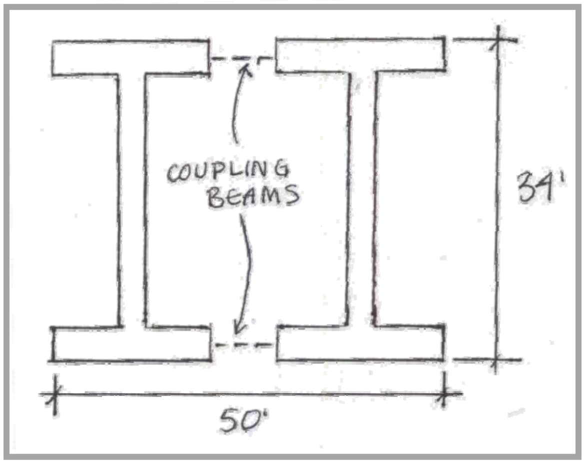

Figure 6 – Sketch of Proposed Shear Wall System

In order to achieve this goal, the use of both preliminary hand calculations as well as computer programs will be used. Once the wall layout is determined, a simple analysis based on the factored shear forces and wall lengths will give a likely solution to the required wall thicknesses. Further analysis will be done with the use of computer programs.

The building will be modeled 3-dimensionally in ETABS, with the primary elements being the shear walls, coupling beams, and floor diaphragms. The beams will have to be designed with minimal depth, because the existing gravity system is a flat plate, post-tensioned slab that is only 8" deep in the residential towers. This efficient floor system limits the floor-to-floor height of the building, so the coupling beams will have to be designed (if possible) to fit within a limited depth of about 19.5". If this design does not work, there might be some implications for the floor height and overall height of the building.

The program input will include user-defined loads based on wind and seismic shear forces calculated by hand. Inherent torsion will be included, as well as accidental torsion by applying 5% eccentricity of the loads to each floor (with potential amplification). The relevant output will include the shear/axial forces and moments in each particular wall and beam, as well as the floor displacements and story drifts.

Although the structural engineers designed this building based primarily on the Puerto Rico Building Code 1999 and the UBC 1997, the following codes will be used for this design project:

- ACI 318-08 (American Concrete Institute)
- ASCE 7-05 (American Society of Civil Engineers)
- IBC 2006 (International Building Code)

Once the shear wall layout is determined, the wall and coupling beam reinforcement will be designed according to ACI 318-08, in addition to the computer program pcaColumn. Based on the output from the model in ETABS, the overturning moment at each level for a certain I-shaped section of shear walls could be determined by assigning a pier label to that section. These moments, in addition to the axial forces in each wall based on gravity loads, would provide enough information to check the layout of the reinforcement in pcaColumn. The arrangement of the reinforcement for the boundary elements and for the wall in general will give the program the information needed to create a force-moment interaction diagram. A comparison of the diagram with the actual loads will determine if the reinforcement is adequate. It will also illustrate if the reinforcement is over-designed by giving the location of the data point(s) relative to the edge of the diagram.

Determination of Lateral Loads

For the purpose of this project, an analysis was done for about one-half of the building (which includes a residential tower and the section of parking garage up to the expansion joint). Because of the seismic joint, each tower could be treated structurally as an independent building. By narrowing the scope to just one building, a more detailed analysis could be completed because the exercises did not have to be repeated for each tower.

The sketch below (fig. 7) shows the overall building dimensions as well as the directions of the applied lateral loads. The colored rectangle (about 90' x 160') shows the approximate location of the tower above the parking garage.

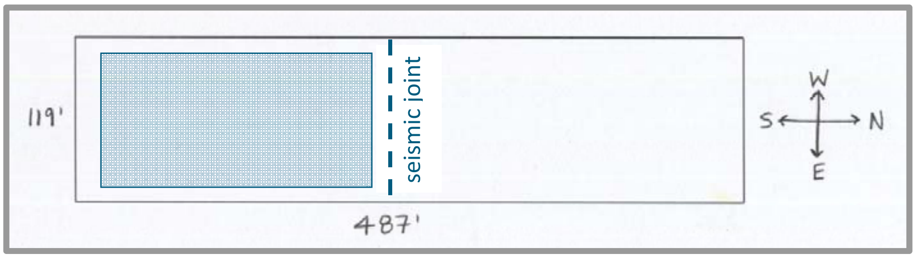

Figure 7 – Plan Dimensions (N.T.S.) and Cardinal Directions

The wind loads were calculated using the analytical procedure (method 2) in chapter 6 of ASCE 7-05. A summary of the main design variables is shown to the right in table 2, and the wind pressures for each direction are shown on the following page in table 3. For the purpose of design, the total wind pressures were used to calculate the story forces and story shears.

Basic Wind Speed	V (mph)	=	145
Wind Directionality Factor	K_d	=	0.85
Importance Factor	I	=	1.0
Exposure Category		=	B
Topographic Factor	K_{zt}	=	1.0
Velocity Pressure Exposure Coefficient	K_h	=	1.32
Velocity Pressure Exposure Coefficient	K_z	=	(varies)
Velocity Pressures	q_z, q_h (psf)	=	(varies)

Table 2 – Wind Design Variables

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	32.10	18.01	-25.87	43.89
20	22.13	-11.71	33.84	19.56	-25.87	45.43
25	23.59	-11.71	35.30	20.84	-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	28.75	-11.71	40.47	25.41	-25.87	51.28
60	30.29	-11.71	42.00	26.77	-25.87	52.64
67	31.26	-11.71	42.97	27.63	-25.87	53.50
67	31.26	-11.71	42.97	27.63	-25.87	53.50
70	31.66	-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 – Calculated Wind Pressures

A summary of the main variables used for the seismic lateral load analysis is shown to the right in table 4. A value of R=6 was used for a building frame system with special reinforced concrete shear walls (ASCE 7-05, table 12.2-1). In this study, the analysis was completed using the equivalent lateral force procedure. According to table 12.6-1 in ASCE 7-05, this type of analysis is not permitted for the building because it is in a seismic design category D and there is extreme torsional irregularity (type 1b). However, the analysis was used as a feasibility study for the new shear wall layout.

Spectral Response Acceleration (Short Periods)	S_s =	0.882
Spectral Response Acceleration (1 second)	S_1 =	0.301
Site Class	=	B
Importance Factor	I =	1.0
Short Period Site Coefficient	F_a =	1.0
Long Period Site Coefficient	F_v =	1.0
Seismic Design Category	=	D
Response Modification Coefficient	R =	6
Fundamental Period	T (sec) =	2.031
Seismic Response Coefficient	C_s =	0.0165

Table 4 – Seismic Design Variables

In table 5 below, the base shear force was found from the total building weight and the seismic response coefficient. The dead loads for each floor were calculated by doing take-offs from the original building. Allowances were included for slabs, exterior walls, shear walls, columns, partitions, roofing, and superimposed loads. The areas were found by making approximate calculations based on existing floor plan dimensions.

Level(s)	Total Dead Load (psf)	Area (ft^2)	Weight per Floor (k)
2	153	23440	3586
3 to 6	150	23440	3516
7	170	23440	3985
8 to 17	145	11060	1604
18 to 27	145	11060	1604
28 to 29	145	13490	1956
Roof	117	13490	1578
Sky Lobby Roof	115	4740	545

Total Building Weight, W (k) =	59745
C_s =	0.0165
Base Shear, V (k) =	986

Table 5 – Calculation of Base Shear

The seismic story forces were calculated by distributing the base shear to each level according to chapter 12 of ASCE 7-05. Both the seismic and wind loads for each orthogonal direction were then factored (1.6 for wind and 1.0 for earthquake), and the total story shears were calculated. The results are summarized on the next two pages in tables 6 and 7. For both the north-south and east-west directions, the factored wind loads were the controlling lateral load case in terms of story forces and base shears. However, both the wind and the seismic loading conditions were used for the design checks throughout the report.

			NORTH-SOUTH DIRECTION			
			Factored Wind Load (1.6W)		Factored Seismic Load (1.0E)	
Level	Floor Elevation (ft)	Tributary Width (ft)	Story Force (k)	Total Story Shear (k)	Story Force (k)	Total Story Shear (k)
sky lobby roof level	275.0	60.0	37.5	37.5	23.8	23.8
roof level	263.7	89.0	84.3	121.8	64.1	87.9
29	254.8	89.0	74.7	196.5	74.7	162.6
28	245.8	89.0	73.1	269.6	70.1	232.7
27	236.8	83.5	68.5	338.1	53.8	286.5
26	227.9	83.5	68.5	406.6	50.3	336.8
25	218.9	83.5	68.5	475.1	46.8	383.6
24	210.0	83.5	68.5	543.6	43.5	427.1
23	201.0	83.5	67.2	610.8	40.3	467.4
22	192.0	83.5	65.2	676.0	37.1	504.5
21	183.1	83.5	64.9	740.9	34.1	538.6
20	174.1	83.5	63.6	804.5	31.2	569.8
19	165.2	83.5	63.6	868.1	28.4	598.2
18	156.2	83.5	62.1	930.2	25.8	624.0
17	147.3	83.5	62.0	992.2	23.2	647.2
16	138.3	83.5	60.8	1053.0	20.8	668.0
15	129.3	83.5	60.2	1113.2	18.5	686.5
14	120.4	83.5	59.3	1172.5	16.2	702.7
13	111.4	83.5	58.2	1230.7	14.2	716.9
12	102.5	83.5	57.7	1288.4	12.2	729.1
11	93.5	83.5	55.8	1344.2	10.4	739.5
10	84.5	83.5	54.7	1398.9	8.7	748.2
9	75.6	83.5	53.4	1452.3	7.1	755.3
8	66.6	119.3	78.6	1530.9	5.7	761.0
7	56.7	119.3	90.5	1621.4	10.6	771.6
6	44.2	119.3	80.6	1702.0	6.0	777.6
5	35.8	119.3	61.6	1763.6	4.2	781.8
4	27.5	119.3	58.4	1822.0	2.6	784.4
3	19.2	119.3	54.8	1876.8	1.4	785.8
2	10.8	119.3	91.9	1968.7	0.5	786.3

Table 6 – Factored Wind and Seismic Loads for the North-South Direction

			EAST-WEST DIRECTION			
			Factored Wind Load (1.6W)		Factored Seismic Load (1.0E)	
Level	Floor Elevation (ft)	Tributary Width (ft)	Story Force (k)	Total Story Shear (k)	Story Force (k)	Total Story Shear (k)
sky lobby roof level	275.0	79.0	56.7	56.7	23.8	23.8
roof level	263.7	163.8	178.2	234.9	64.1	87.9
29	254.8	163.8	158.0	392.8	74.7	162.6
28	245.8	159.5	151.2	544.1	70.1	232.7
27	236.8	155.0	146.9	691.0	53.8	286.5
26	227.9	155.0	146.9	837.9	50.3	336.8
25	218.9	155.0	146.9	984.8	46.8	383.6
24	210.0	155.0	146.9	1131.7	43.5	427.1
23	201.0	155.0	144.8	1276.4	40.3	467.4
22	192.0	155.0	141.4	1417.8	37.1	504.5
21	183.1	155.0	141.0	1558.8	34.1	538.6
20	174.1	155.0	138.9	1697.7	31.2	569.8
19	165.2	155.0	138.9	1836.5	28.4	598.2
18	156.2	155.0	136.4	1972.9	25.8	624.0
17	147.3	155.0	136.2	2109.1	23.2	647.2
16	138.3	155.0	134.2	2243.3	20.8	668.0
15	129.3	155.0	133.2	2376.5	18.5	686.5
14	120.4	155.0	131.7	2508.2	16.2	702.7
13	111.4	155.0	130.0	2638.2	14.2	716.9
12	102.5	155.0	129.1	2767.3	12.2	729.1
11	93.5	155.0	126.1	2893.4	10.4	739.5
10	84.5	155.0	124.3	3017.7	8.7	748.2
9	75.6	155.0	122.1	3139.8	7.1	755.3
8	66.6	155.0	126.6	3266.4	5.7	761.0
7	56.7	196.5	186.5	3452.8	10.6	771.6
6	44.2	196.5	168.1	3621.0	6.0	777.6
5	35.8	196.5	130.2	3751.2	4.2	781.8
4	27.5	196.5	125.7	3876.9	2.6	784.4
3	19.2	196.5	120.4	3997.3	1.4	785.8
2	10.8	196.5	207.0	4204.3	0.5	786.3

Table 7 – Factored Wind and Seismic Loads for the East-West Direction

The following two tables (8 and 9) provide an analysis for the special wind load cases as defined in figure 6-9 in chapter 6 of ASCE 7-05. Some of the load cases include just story forces, while others include story forces plus an applied moment due to eccentricity of the wind load. The load cases are described below, and the calculated values for each level are shown in the tables with an assigned letter for each column.

Load Case	Column	Description
- 1a	A	Factored wind load (NS direction)
- 1b	G	Factored wind load (EW direction)
- 2a	B + E	75% of the factored wind load + the moment created by 75% of the load at 15% eccentricity (NS direction)
- 2b	H + K	75% of the factored wind load + the moment created by 75% of the load at 15% eccentricity (EW direction)
- 3	B + H	75% of the factored wind load applied in both directions simultaneously
- 4	C + F + I + L	56.25% of the factored wind loads in both directions + the moment created by 56.25% of the wind loads at 15% eccentricity in both directions

	NORTH-SOUTH DIRECTION					
	A	B	C	D	E	F
Level	Story Force, F_x (k)	$0.75F_x$ (k)	$0.5625F_x$ (k)	Building Width, B_x (ft)	Moment, $0.15B_x*0.75F_x$ (k-ft)	Moment, $0.15B_x*0.5625F_x$ (k-ft)
sky lobby roof level	37.5	28.1	21.1	60.0	253.1	189.8
roof level	84.3	63.2	47.4	89.0	844.1	633.0
29	74.7	56.0	42.0	89.0	747.9	561.0
28	73.1	54.8	41.1	89.0	731.9	548.9
27	68.5	51.4	38.5	83.5	643.5	482.6
26	68.5	51.4	38.5	83.5	643.5	482.6
25	68.5	51.4	38.5	83.5	643.5	482.6
24	68.5	51.4	38.5	83.5	643.5	482.6
23	67.2	50.4	37.8	83.5	631.3	473.4
22	65.2	48.9	36.7	83.5	612.5	459.4
21	64.9	48.7	36.5	83.5	609.7	457.2
20	63.6	47.7	35.8	83.5	597.4	448.1
19	63.6	47.7	35.8	83.5	597.4	448.1
18	62.1	46.6	34.9	83.5	583.4	437.5
17	62.0	46.5	34.9	83.5	582.4	436.8
16	60.8	45.6	34.2	83.5	571.1	428.4
15	60.2	45.2	33.9	83.5	565.5	424.1
14	59.3	44.5	33.4	83.5	557.0	417.8
13	58.2	43.7	32.7	83.5	546.7	410.0
12	57.7	43.3	32.5	83.5	542.0	406.5
11	55.8	41.9	31.4	83.5	524.2	393.1
10	54.7	41.0	30.8	83.5	513.8	385.4
9	53.4	40.1	30.0	83.5	501.6	376.2
8	78.6	59.0	44.2	119.3	1055.2	791.4
7	90.5	67.9	50.9	119.3	1215.0	911.2
6	80.6	60.5	45.3	119.3	1082.1	811.5
5	61.6	46.2	34.7	119.3	827.0	620.2
4	58.4	43.8	32.9	119.3	784.0	588.0
3	54.8	41.1	30.8	119.3	735.7	551.8
2	91.9	68.9	51.7	119.3	1233.8	925.3

Table 8 – Special Wind Load Cases for the North-South Direction

	EAST-WEST DIRECTION					
	G	H	I	J	K	L
Level	Story Force, F_x (k)	$0.75F_x$ (k)	$0.5625F_x$ (k)	Building Width, B_x (ft)	Moment, $0.15B_x*0.75F_x$ (k-ft)	Moment, $0.15B_x*0.5625F_x$ (k-ft)
sky lobby roof level	56.7	42.5	31.9	79.0	503.8	377.9
roof level	178.2	133.6	100.2	163.8	3283.9	2462.9
29	158.0	118.5	88.9	163.8	2911.6	2183.7
28	151.2	113.4	85.1	159.5	2713.8	2035.4
27	146.9	110.2	82.6	155.0	2561.6	1921.2
26	146.9	110.2	82.6	155.0	2561.6	1921.2
25	146.9	110.2	82.6	155.0	2561.6	1921.2
24	146.9	110.2	82.6	155.0	2561.6	1921.2
23	144.8	108.6	81.4	155.0	2524.1	1893.1
22	141.4	106.0	79.5	155.0	2465.0	1848.7
21	141.0	105.7	79.3	155.0	2458.3	1843.8
20	138.9	104.2	78.1	155.0	2421.7	1816.3
19	138.9	104.2	78.1	155.0	2421.7	1816.3
18	136.4	102.3	76.7	155.0	2378.3	1783.7
17	136.2	102.1	76.6	155.0	2374.8	1781.1
16	134.2	100.6	75.5	155.0	2339.2	1754.4
15	133.2	99.9	74.9	155.0	2323.2	1742.4
14	131.7	98.8	74.1	155.0	2297.2	1722.9
13	130.0	97.5	73.1	155.0	2266.4	1699.8
12	129.1	96.9	72.6	155.0	2251.9	1688.9
11	126.1	94.6	70.9	155.0	2198.5	1648.9
10	124.3	93.2	69.9	155.0	2166.8	1625.1
9	122.1	91.6	68.7	155.0	2128.6	1596.4
8	126.6	95.0	71.2	155.0	2207.8	1655.8
7	186.5	139.9	104.9	196.5	4122.2	3091.6
6	168.1	126.1	94.6	196.5	3716.5	2787.4
5	130.2	97.7	73.3	196.5	2879.1	2159.3
4	125.7	94.3	70.7	196.5	2779.2	2084.4
3	120.4	90.3	67.7	196.5	2661.2	1995.9
2	207.0	155.2	116.4	196.5	4575.8	3431.8

Table 9 – Special Wind Load Cases for the East-West Direction

Calculation of Preliminary Wall Thicknesses

For the more detailed layout of the new shear walls, the following configuration was created (fig. 8). The variation in the flange lengths was due to architectural limitations, which will be discussed in more detail later in the report.

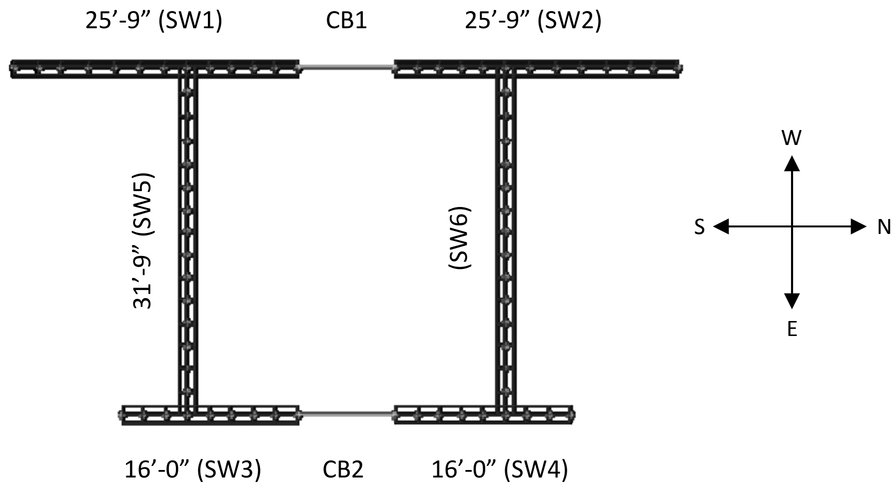

Figure 8 – Shear Wall and Coupling Beam Configuration

The basic factored load cases (1.6W and 1.0E) were used for the determination of the preliminary wall thicknesses, which were calculated with the following equation.

$$t = \frac{\rho(V_x)}{\phi\left(3\sqrt{f'_c}\right)(l_w)}$$

- t = wall thickness (in)
- ρ = fraction of the story shear force being resisted by that wall (based on number of walls and their relative lengths)
- V_x = factored total shear force at level x (multiplied by 1000 to convert from kips to pounds)
- ϕ = 0.75 for wind loads
- ϕ = 0.60 for seismic loads
- $3\sqrt{f'_c}$ = approximate shear stress of the wall (f'_c = 8000 psi up to level 13, f'_c = 6000 psi above level 13)
- l_w = length of the wall (in)

The results are shown tables 10 and 11 on following two pages.

	NORTH-SOUTH DIRECTION							
Level	Wind (factored)		Seismic (factored)		Preliminary Shear Wall Thickness (in)			
	Story Force, F_x (k)	Total Shear, V_x (k)	Story Force, F_x (k)	Total Shear, V_x (k)	SW1	SW2	SW3	SW4
sky lobby roof level	37.5	37.5	23.8	23.8	0.21	0.21	0.22	0.22
roof level	84.3	121.8	64.1	87.9	0.67	0.67	0.73	0.73
29	74.7	196.5	74.7	162.6	1.12	1.12	1.21	1.21
28	73.1	269.6	70.1	232.7	1.60	1.60	1.74	1.74
27	68.5	338.1	53.8	286.5	1.98	1.98	2.14	2.14
26	68.5	406.6	50.3	336.8	2.32	2.32	2.52	2.52
25	68.5	475.1	46.8	383.6	2.65	2.65	2.87	2.87
24	68.5	543.6	43.5	427.1	3.00	3.00	3.25	3.25
23	67.2	610.8	40.3	467.4	3.37	3.37	3.65	3.65
22	65.2	676.0	37.1	504.5	3.73	3.73	4.04	4.04
21	64.9	740.9	34.1	538.6	4.09	4.09	4.43	4.43
20	63.6	804.5	31.2	569.8	4.44	4.44	4.81	4.81
19	63.6	868.1	28.4	598.2	4.79	4.79	5.19	5.19
18	62.1	930.2	25.8	624.0	5.13	5.13	5.56	5.56
17	62.0	992.2	23.2	647.2	5.47	5.47	5.93	5.93
16	60.8	1053.0	20.8	668.0	5.81	5.81	6.29	6.29
15	60.2	1113.2	18.5	686.5	6.14	6.14	6.65	6.65
14	59.3	1172.5	16.2	702.7	6.47	6.47	7.01	7.01
13	58.2	1230.7	14.2	716.9	5.88	5.88	6.37	6.37
12	57.7	1288.4	12.2	729.1	6.16	6.16	6.67	6.67
11	55.8	1344.2	10.4	739.5	6.42	6.42	6.96	6.96
10	54.7	1398.9	8.7	748.2	6.68	6.68	7.24	7.24
9	53.4	1452.3	7.1	755.3	6.94	6.94	7.52	7.52
8	78.6	1530.9	5.7	761.0	7.31	7.31	7.92	7.92
7	90.5	1621.4	10.6	771.6	7.75	7.75	8.39	8.39
6	80.6	1702.0	6.0	777.6	8.13	8.13	8.81	8.81
5	61.6	1763.6	4.2	781.8	8.43	8.43	9.13	9.13
4	58.4	1822.0	2.6	784.4	8.71	8.71	9.43	9.43
3	54.8	1876.8	1.4	785.8	8.97	8.97	9.71	9.71
2	91.9	1968.7	0.5	786.3	9.41	9.41	10.19	10.19

Table 10 – Preliminary Thicknesses for Walls in the North-South Direction

	EAST-WEST DIRECTION					
Level	Wind (factored)		Seismic (factored)		Preliminary Shear Wall Thickness (in)	
	Story Force, F_x (k)	Total Shear, V_x (k)	Story Force, F_x (k)	Total Shear, V_x (k)	SW5	SW6
sky lobby roof level	56.7	56.7	23.8	23.8	0.44	0.44
roof level	178.2	234.9	64.1	87.9	1.81	1.81
29	158.0	392.8	74.7	162.6	3.03	3.03
28	151.2	544.1	70.1	232.7	4.20	4.20
27	146.9	691.0	53.8	286.5	5.33	5.33
26	146.9	837.9	50.3	336.8	6.46	6.46
25	146.9	984.8	46.8	383.6	7.59	7.59
24	146.9	1131.7	43.5	427.1	8.73	8.73
23	144.8	1276.4	40.3	467.4	9.84	9.84
22	141.4	1417.8	37.1	504.5	10.93	10.93
21	141.0	1558.8	34.1	538.6	12.02	12.02
20	138.9	1697.6	31.2	569.8	13.09	13.09
19	138.9	1836.5	28.4	598.2	14.16	14.16
18	136.4	1972.9	25.8	624.0	15.22	15.22
17	136.2	2109.1	23.2	647.2	16.27	16.27
16	134.2	2243.3	20.8	668.0	17.30	17.30
15	133.2	2376.5	18.5	686.5	18.33	18.33
14	131.7	2508.2	16.2	702.7	19.34	19.34
13	130.0	2638.2	14.2	716.9	17.62	17.62
12	129.1	2767.3	12.2	729.1	18.48	18.48
11	126.1	2893.4	10.4	739.5	19.32	19.32
10	124.3	3017.7	8.7	748.2	20.15	20.15
9	122.1	3139.7	7.1	755.3	20.97	20.97
8	126.6	3266.4	5.7	761.0	21.82	21.82
7	186.5	3452.8	10.6	771.6	23.06	23.06
6	168.1	3620.9	6.0	777.6	24.18	24.18
5	130.2	3751.2	4.2	781.8	25.05	25.05
4	125.7	3876.9	2.6	784.4	25.89	25.89
3	120.4	3997.3	1.4	785.8	26.70	26.70
2	207.0	4204.3	0.5	786.3	28.08	28.08

Table 11 – Preliminary Thicknesses for Walls in the East-West Direction

For the initial model, an 18" thickness was used for all shear walls oriented in the north-south direction. For the east-west direction, a 30" wall was used up to level 7, a 24" wall through level 16, and an 18" wall extending to the top floor.

ETABS Model (M.A.E. Requirement)

The computer program ETABS was used to model the shear walls 3-dimensionally. The image below (fig. 9) gives the general idea of the location of the shear wall system in this tower. In order to visualize its relation to the rest of the building, the seismic joint is labeled in the figure. The shear walls were initially modeled with the preliminary wall thicknesses calculated above. However, to help reduce the torsion and modal periods of the structure, the walls were thickened in some places. The thickness of SW1 and SW2 were increased from 18" to 24" below level 13. Also, 18" shear walls were added on either side of the shear wall core in the first 7 levels (the parking structure).

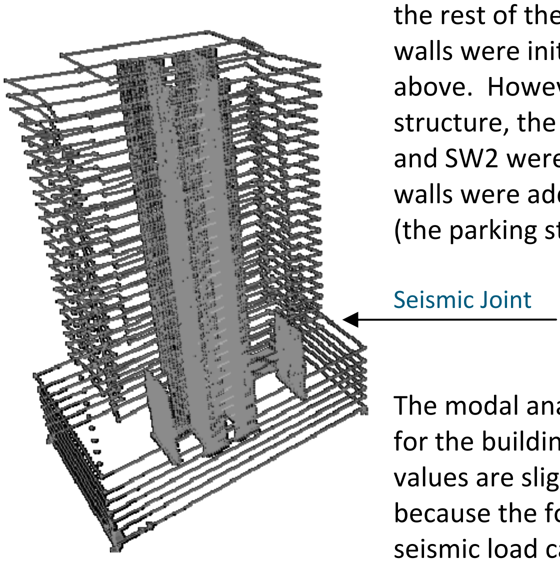

Figure 9 – 3-Dimensional Model of Shear Walls

The modal analysis of the structure (from ETABS) gave the following results for the building period: T_z = 5.75 sec, T_x = 3.91 sec, T_y = 2.31 sec. These values are slightly high for this building, but those periods are acceptable because the focus of this project is not dynamic analysis. Also, the initial seismic load calculations were based on a fundamental period of 2.031 seconds, which does not exceed the values given from ETABS. Therefore, the calculations do not need to be modified.

The same concrete strength (f'_c) as the original design was used for the analysis: 8000 psi up to level 13 and 6000 psi above level 13. The factored lateral loads, as well as the special wind load cases, were defined in the program as 'user-defined' loads to be used for analysis of the walls. (In addition, a load case for 70% of the wind load was used for drift analysis. See the following section for more details.) The floor diaphragms were drawn in the model and assigned a mass as calculated for the seismic loads.

The walls were modeled as shell elements (meshed into areas with 24" to 30" dimensions) and a 0.7 multiplier was applied to the moment of inertia (f22 in the program), which decreased the stiffness of the walls. This section property modifier was allowed according to ACI 318-08 section 10.10.4.1, which states that the "stiffnesses EI used in an analysis for strength design should represent the stiffnesses of the members immediately prior to failure" (R10.10.4).

There are two coupling beams at each level: one connecting shear walls 1 and 2, and one connecting shear walls 3 and 4. The depth of the coupling beams was limited due to the low floor-to-floor height, which was typically 8'-11 ½" for the apartment levels. After leaving a 7'-4" allowance for the door frames, only 1'-7 ½" remained for the depth of the beams. The width of the beams was 18", which did not exceed the width of the walls. For the cracked section properties, a modifier of 0.125 was applied to the moment of inertia of the beams. By making the beams less stiff in the model, the output forces in the beams were lower than the actual loads. Then, because the reinforcement was designed based on the lower forces, the beams were designed to yield and crack in an actual earthquake. Then, they would become less stiff and see the loads that resulted from $0.125I_g$.

For clarity, the elevations of each of the main shear walls are shown in figure 10 below. There are some variations and openings along the height of the building due to architectural reasons. To summarize, SW1 and SW2 are 24" thick up to level 13 and 18" for the upper floors, while SW3 and SW4 are 18" thick for the total building height. SW5 and SW6 are 30" up to level 7, 24" up to level 16, and 18" for the upper floors.

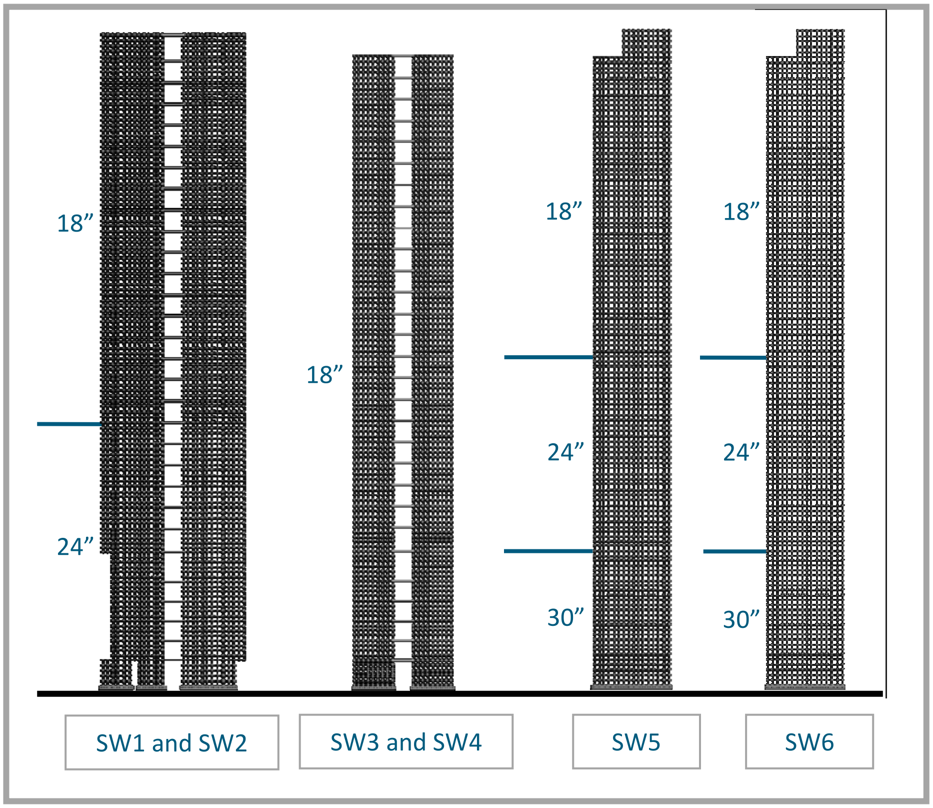

Figure 10 – Shear Wall Elevations and Thicknesses

Drift Analysis

Initially, the calculated seismic story forces were defined in the ETABS model with an automatic 5% accidental torsion. After running the model, the displacements at the roof level (table 12) were used to determine if there were any horizontal structural irregularities in the building. The maximum seismic drift for two points at opposite ends of the level was found. For the load case with the maximum displacements, the larger of the two displacements was divided by the average of the displacements at that point.

		Point 1		Point2	
		δ_x	δ_y	δ_x	δ_y
Story Drift (in)	NS Seismic	0.1737	0.2547	0.4602	0.2589
	EW Seismic	0.0216	0.0010	0.0637	0.1538

Table 12 – Displacements of two points at the roof level with 5% accidental eccentricity

The maximum drifts were in the x-direction due to seismic loads oriented in the north-south direction. The following equation was used to determine that there was extreme torsional irregularity because $\delta_{max}/\delta_{avg} > 1.4$. The types of horizontal irregularities are defined in table 12.3-1 of ASCE 7-05.

$$\frac{\delta_{max}}{\delta_{avg}} = \frac{0.4602}{\left(\frac{0.4602 + 0.1737}{2}\right)} = 1.45$$

Because of the irregularity, a torsional amplification factor (ASCE 7-05, 12.8.4.3) was calculated and applied to the accidental eccentricity ratio. This calculation for accidental torsion was only based on the displacements at the roof level, but the value was used for the entire building.

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 = \left(\frac{1.45}{1.2}\right)^2 = 1.46$$

Accidental eccentricity ratio = (0.05)(1.46) = 0.073

Table 13 on the following page summarizes the drift analyses at each floor for both wind and seismic loads. The drift limit for wind is defined as L/400, which is common practice. The ETABS output for wind is based on 70% of the calculated loads. In section CC.1.2 of ASCE 7-05, it is written that the use of factored wind loads for serviceability checks is extremely conservative. Therefore, the load combination with a 5% chance of being exceeded in a given year (0.70W) is used in this analysis. The allowable story drift due to seismic loads is based on an occupancy category of II (ASCE 7-05, table 12.12-1). The actual drifts from ETABS are multiplied by a deflection amplification factor, C_d, which is equal to 5 for special reinforced concrete shear walls (ASCE 7-05, table 12.2-1). For the new shear wall design, the story drifts do not exceed the given limits.

		Story Drift due to Wind (in)		**Story Drift due to Seismic (in)**	
Level	Story Height (ft)	ETABS Output (deflection due to 0.7W)	Code Recommendation (L/400)	ETABS Output (C_d * deflection due to 1.0E)	Code Requirement (0.020h)
sky lobby roof level	11.250	0.232	0.338	1.820	2.70
roof level	9.000	0.202	0.270	1.619	2.16
29	9.000	0.203	0.270	1.630	2.16
28	9.000	0.243	0.270	2.018	2.16
27	9.000	0.205	0.270	1.634	2.16
26	9.000	0.206	0.270	1.646	2.16
25	9.000	0.208	0.270	1.656	2.16
24	9.000	0.210	0.270	1.662	2.16
23	9.000	0.211	0.270	1.665	2.16
22	9.000	0.212	0.270	1.663	2.16
21	9.000	0.212	0.270	1.655	2.16
20	9.000	0.212	0.270	1.641	2.16
19	9.000	0.211	0.270	1.620	2.16
18	9.000	0.209	0.270	1.591	2.16
17	9.000	0.207	0.270	1.553	2.16
16	9.000	0.203	0.270	1.506	2.16
15	9.000	0.198	0.270	1.452	2.16
14	9.000	0.191	0.270	1.387	2.16
13	9.000	0.184	0.270	1.319	2.16
12	9.000	0.178	0.270	1.262	2.16
11	9.000	0.171	0.270	1.198	2.16
10	9.000	0.162	0.270	1.127	2.16
9	9.000	0.153	0.270	1.048	2.16
8	10.000	0.170	0.300	1.173	2.40
7	12.500	0.162	0.375	1.057	3.00
6	8.333	0.092	0.250	0.592	2.00
5	8.333	0.078	0.250	0.493	2.00
4	8.333	0.062	0.250	0.387	2.00
3	8.333	0.044	0.250	0.271	2.00
2	10.833	0.022	0.325	0.130	2.60

Table 13 – Story Drifts Under Wind and Seismic Loads

A brief check was conducted for the existing seismic joint (which extends from the base to the 7th floor) based on ASCE 7-05 section 12.12.3 ('Building Separation'). The maximum drift at level 7 in the north-south direction is 1.06", which is due to the seismic loading. The analysis was only completed for one of the towers, so the required separation was assumed to be twice that displacement, or 2.12". Therefore, it is assumed that the existing 5" seismic joint would suffice with the new shear wall design and no pounding would occur.

Feasibility Test for Coupling Beams

Because of the shallow depth of the coupling beams, an analysis was done to see if the beams could be designed without diagonal reinforcing. In order to avoid diagonal reinforcing (according to ASCE 7-05 section 21.9.7), the following equation had to be met.

$$\frac{V_u/\emptyset}{\lambda\sqrt{f_c'}A_{cw}} \leq \begin{cases} 12\ (for\ wind) \\ 4\ (for\ seismic) \end{cases}$$

- V_u = factored shear force in the beam based on the following load combinations
  - 1.2D + 0.5L + 1.6W
  - 0.9D + 1.6W
  - 1.32D + 0.5L + 1.0E (modified, see below)
  - 0.78D + 1.0E (modified, see below)
- ϕ = 0.75 for wind loads
- ϕ = 0.60 for seismic loads
- λ = 1 for normal weight concrete
- f'_c = 8000 psi up to level 13, f'_c = 6000 psi above level 13
- A_{cw} = cross-sectional area of the beam (18" x 19.5" = 351 in^2)

The values for V_u were found based on both hand calculations and ETABS output. Using the tributary area for each beam, the estimated dead load for each floor, and the provided live loads from IBC 2006, the V_u from gravity loads was calculated. Generally, the tributary area was found to be 183.4 ft^2 for the coupling beam labeled CB1 and 142.9 ft^2 (apartments) or 230.6 ft^2 (parking area) for the coupling beam labeled CB2. The dead loads were summarized previously for the seismic load calculations. The live loads were assumed to be 20 psf for the roof, 100 psf for the apartment towers (because the shear walls surround the elevator lobby, which is treated as a corridor), and 40 psf for the parking garage.

The shear forces in the beams due to wind were found from ETABS. All of the special load cases were considered. The seismic load combinations were modified according to section 12.4 of ASCE 7-05.

- Seismic effects, E = $E_h \pm E_v$
- $E_h = \rho Q_E$ (ρ = 1.3 for Seismic Design Category D, Q_E = seismic force)
- $E_v = (0.2S_{DS})D = 0.2(0.588)D = 0.1176D$
- Modified Dead Load Factors:
  - 1.2 + 0.1176 = 1.318
  - 0.9 – 0.1176 = 0.782

The total V_u from the gravity calculations and the ETABS output was found using the previously stated load combinations, and the maximum V_u was used in the equation to check the necessity of diagonal reinforcement. The results are summarized in the tables on the next few pages. (Note: There is only one coupling beam at the top level because the shear wall configuration changes due to the sky lobby.)

Level	V_u due to gravity loads (hand calculations)			V_u due to lateral loads (ETABS)			
				Wind Case 1	Wind Case 2	Wind Case 3	Wind Case 4
	1.2D	0.9D	0.5L	1.6W (in NS direction for beams)	0.75(1.6W) + 15% eccentricity (max. in either NS or EW direction)	0.75(1.6W) (simultaneous in both directions)	0.5625(1.6W) + 15% eccentricity (applied in both directions)
sky lobby roof level	18.47	13.86	1.34	34.99	21.76	26.04	3.24
roof level	12.87	9.65	4.58	38.35	23.84	28.55	3.53
29	15.95	11.97	4.58	38.60	23.99	28.73	3.54
28	15.95	11.97	4.58	38.94	24.21	28.99	3.58
27	15.95	11.97	4.58	39.35	24.48	29.29	3.64
26	15.95	11.97	4.58	39.79	24.77	29.62	3.73
25	15.95	11.97	4.58	40.24	25.07	29.95	3.83
24	15.95	11.97	4.58	40.68	25.37	30.27	3.96
23	15.95	11.97	4.58	41.08	25.65	30.56	4.10
22	15.95	11.97	4.58	41.40	25.90	30.80	4.26
21	15.95	11.97	4.58	41.64	26.09	30.97	4.44
20	15.95	11.97	4.58	41.75	26.22	31.05	4.64
19	15.95	11.97	4.58	41.72	26.26	31.02	4.85
18	15.95	11.97	4.58	41.52	26.21	30.86	5.09
17	15.95	11.97	4.58	41.12	26.05	30.55	5.34
16	15.95	11.97	4.58	40.49	25.75	30.07	5.61
15	15.95	11.97	4.58	39.61	25.31	29.41	5.89
14	15.95	11.97	4.58	38.48	24.71	28.55	6.15
13	15.95	11.97	4.58	43.23	27.92	32.06	7.50
12	15.95	11.97	4.58	42.48	27.57	31.50	7.83
11	15.95	11.97	4.58	40.84	26.66	30.26	8.07
10	15.95	11.97	4.58	38.91	25.58	15.98	8.33
9	15.95	11.97	4.58	36.67	24.31	27.15	8.59
8	15.95	11.97	4.58	33.87	22.70	25.06	8.80
7	18.70	14.03	4.58	30.06	20.47	22.34	9.11
6	16.50	12.38	4.58	24.89	17.24	18.75	8.81
5	16.50	12.38	4.58	21.29	14.82	16.19	7.94
4	16.50	12.38	4.58	17.34	12.13	13.35	6.79
3	16.50	12.38	4.58	12.96	9.10	10.16	5.34
2	16.83	12.63	4.58	7.82	5.50	6.25	3.34

Table 14 – **CB1** Shear Forces Due to Gravity and Lateral Loads (**Wind**)

Level	**TOTAL V_u** (kips)								$V_u/(\phi A_{cw}\sqrt{f'_c})$
	1.2D + 0.5L + Wind Case 1	1.2D + 0.5L + Wind Case 2	1.2D + 0.5L + Wind Case 3	1.2D + 0.5L + Wind Case 4	0.9D + Wind Case 1	0.9D + Wind Case 2	0.9D + Wind Case 3	0.9D + Wind Case 4	
sky lobby roof level	54.80	41.57	45.86	23.05	21.13	7.90	12.19	-10.62	3.36
roof level	55.81	41.30	46.01	20.99	28.70	14.19	18.89	-6.13	3.42
29	59.13	44.53	49.27	24.07	26.63	12.03	16.77	-8.43	3.62
28	59.48	44.75	49.53	24.12	26.98	12.25	17.02	-8.39	3.65
27	59.89	45.02	49.83	24.18	27.38	12.51	17.32	-8.32	3.67
26	60.33	45.31	50.15	24.27	27.83	12.80	17.65	-8.24	3.70
25	60.78	45.61	50.49	24.37	28.28	13.11	17.99	-8.13	3.73
24	61.22	45.91	50.81	24.50	28.72	13.41	18.31	-8.01	3.75
23	61.61	46.19	51.10	24.64	29.11	13.69	18.60	-7.86	3.78
22	61.94	46.43	51.34	24.80	29.44	13.93	18.84	-7.70	3.80
21	62.18	46.63	51.51	24.98	29.67	14.12	19.00	-7.52	3.81
20	62.29	46.76	51.59	25.18	29.79	14.25	19.08	-7.33	3.82
19	62.26	46.80	51.56	25.39	29.76	14.30	19.05	-7.11	3.82
18	62.06	46.75	51.40	25.62	29.55	14.25	18.89	-6.88	3.80
17	61.66	46.59	51.09	25.87	29.15	14.08	18.59	-6.63	3.78
16	61.02	46.29	50.61	26.15	28.52	13.78	18.11	-6.36	3.74
15	60.15	45.85	49.95	26.43	27.65	13.34	17.45	-6.08	3.69
14	59.01	45.25	49.09	26.69	26.51	12.74	16.59	-5.81	3.62
13	63.76	48.46	52.60	28.04	31.26	15.96	20.10	-4.47	3.39
12	63.02	48.11	52.03	28.37	30.51	15.61	19.53	-4.14	3.35
11	61.37	47.19	50.80	28.61	28.87	14.69	18.30	-3.89	3.26
10	59.45	46.12	36.52	28.87	26.94	13.61	4.01	-3.64	3.16
9	57.21	44.85	47.68	29.13	24.71	12.35	15.18	-3.37	3.04
8	54.41	43.23	45.60	29.34	21.91	10.73	13.09	-3.16	2.89
7	53.35	43.76	45.63	32.40	16.03	6.44	8.31	-4.92	2.83
6	45.97	38.32	39.84	29.89	12.51	4.86	6.37	-3.57	2.44
5	42.38	35.91	37.27	29.03	8.91	2.45	3.81	-4.44	2.25
4	38.43	33.22	34.44	27.88	4.96	-0.25	0.97	-5.59	2.04
3	34.05	30.18	31.25	26.42	0.58	-3.28	-2.22	-7.04	1.81
2	29.24	26.92	27.67	24.76	-4.81	-7.13	-6.38	-9.28	1.55

Table 15 – **CB1** Shear Forces Due to Specified Load Combinations (**Wind**)

Level	V_u due to gravity loads (hand calculations)			V_u due to lateral loads (ETABS)			
				Wind Case 1	Wind Case 2	Wind Case 3	Wind Case 4
	1.2D	0.9D	0.5L	1.6W (in NS direction for beams)	0.75(1.6W) + 15% eccentricity (max. in either NS or EW direction)	0.75(1.6W) (simultaneous in both directions)	0.5625(1.6W) + 15% eccentricity (applied in both directions)
roof level	10.03	7.52	3.57	16.49	18.00	11.98	26.36
29	12.43	9.32	3.57	18.08	19.72	13.14	28.88
28	12.43	9.32	3.57	18.23	19.84	13.24	29.08
27	12.43	9.32	3.57	18.43	19.98	13.39	29.33
26	12.43	9.32	3.57	18.66	20.13	13.56	29.60
25	12.43	9.32	3.57	18.92	20.27	13.74	29.87
24	12.43	9.32	3.57	19.19	20.38	13.94	30.13
23	12.43	9.32	3.57	19.47	20.46	14.14	30.36
22	12.43	9.32	3.57	19.75	20.48	14.33	30.54
21	12.43	9.32	3.57	20.00	20.44	14.50	30.64
20	12.43	9.32	3.57	20.24	20.56	14.66	30.66
19	12.43	9.32	3.57	20.44	20.65	14.78	30.56
18	12.43	9.32	3.57	20.59	20.69	14.87	30.34
17	12.43	9.32	3.57	20.70	20.65	14.91	29.96
16	12.43	9.32	3.57	20.74	20.52	14.91	29.38
15	12.43	9.32	3.57	20.71	20.30	14.85	28.63
14	12.43	9.32	3.57	20.56	19.97	14.70	27.73
13	12.43	9.32	3.57	23.41	22.50	16.67	30.64
12	12.43	9.32	3.57	23.33	22.16	16.55	29.57
11	12.43	9.32	3.57	23.06	21.62	16.29	28.14
10	12.43	9.32	3.57	22.72	20.98	15.98	26.49
9	12.43	9.32	3.57	22.29	20.21	15.60	24.55
8	12.43	9.32	3.57	21.58	19.12	15.03	22.05
7	23.52	17.64	5.77	20.84	17.71	14.45	18.45
6	20.76	15.57	5.77	18.86	15.42	13.12	14.51
5	20.76	15.57	5.77	16.39	13.35	11.53	12.47
4	20.76	15.57	5.77	13.53	10.98	9.65	10.26
3	20.76	15.57	5.77	10.24	8.28	7.45	7.79
2	21.17	15.88	5.77	6.16	4.99	4.60	4.77

Table 16 – **CB2** Shear Forces Due to Gravity and Lateral Loads (**Wind**)

Level	**TOTAL V_u** (kips)								$V_u/(\phi A_{cw}\sqrt{f'_c})$
	1.2D + 0.5L + Wind Case 1	1.2D + 0.5L + Wind Case 2	1.2D + 0.5L + Wind Case 3	1.2D + 0.5L + Wind Case 4	0.9D + Wind Case 1	0.9D + Wind Case 2	0.9D + Wind Case 3	0.9D + Wind Case 4	
roof level	30.10	31.60	25.58	39.96	8.97	10.47	4.46	18.83	2.45
29	34.08	35.72	29.14	44.88	8.76	10.39	3.81	19.56	2.75
28	34.23	35.84	29.25	45.08	8.91	10.51	3.92	19.76	2.76
27	34.43	35.98	29.39	45.33	9.11	10.66	4.07	20.00	2.78
26	34.66	36.13	29.56	45.60	9.34	10.81	4.24	20.27	2.80
25	34.92	36.27	29.75	45.87	9.60	10.95	4.42	20.55	2.81
24	35.20	36.39	29.94	46.13	9.87	11.06	4.62	20.81	2.83
23	35.47	36.46	30.14	46.36	10.15	11.14	4.81	21.04	2.84
22	35.75	36.49	30.33	46.54	10.42	11.16	5.00	21.21	2.85
21	36.01	36.44	30.50	46.64	10.68	11.12	5.18	21.32	2.86
20	36.24	36.56	30.66	46.66	10.92	11.24	5.33	21.33	2.86
19	36.44	36.66	30.78	46.57	11.12	11.33	5.46	21.24	2.85
18	36.60	36.69	30.87	46.34	11.27	11.37	5.55	21.02	2.84
17	36.70	36.65	30.92	45.97	11.37	11.33	5.59	20.64	2.82
16	36.74	36.52	30.91	45.38	11.42	11.20	5.59	20.06	2.78
15	36.71	36.30	30.85	44.63	11.38	10.98	5.52	19.31	2.74
14	36.57	35.98	30.70	43.73	11.24	10.65	5.37	18.41	2.68
13	39.42	38.50	32.67	46.64	14.09	13.18	7.35	21.32	2.48
12	39.33	38.16	32.55	45.57	14.00	12.84	7.23	20.25	2.42
11	39.06	37.63	32.29	44.15	13.73	12.30	6.97	18.82	2.34
10	38.72	36.98	31.98	42.49	13.39	11.66	6.66	17.17	2.26
9	38.29	36.21	31.60	40.56	12.96	10.89	6.28	15.23	2.15
8	37.59	35.12	31.03	38.05	12.26	9.80	5.71	12.72	2.02
7	50.13	47.00	43.73	47.73	3.20	0.07	-3.20	0.80	2.66
6	45.38	41.95	39.64	41.03	3.29	-0.14	-2.45	-1.06	2.41
5	42.91	39.87	38.05	38.99	0.83	-2.22	-4.04	-3.09	2.28
4	40.06	37.50	36.17	36.78	-2.03	-4.59	-5.92	-5.31	2.13
3	36.76	34.81	33.97	34.31	-5.33	-7.28	-8.11	-7.78	1.95
2	33.10	31.92	31.53	31.71	-9.72	-10.89	-11.28	-11.11	1.76

Table 17 – **CB2** Shear Forces Due to Specified Load Combinations (**Wind**)

Level	**V_u due to gravity loads** (hand calculations)			**V_u due to lateral loads** (ETABS)	**TOTAL V_u** (kips)		$V_u/(\phi A_{cw}\sqrt{f'_c})$
	1.32D	0.78D	0.5L	1.0E	1.32D + 0.5L + 1.0E	0.78D + 1.0E	
sky lobby roof level	20.29	12.05	1.34	33.18	54.81	21.14	3.36
roof level	14.13	8.39	4.58	36.38	55.09	27.98	3.38
29	17.52	10.40	4.58	36.61	58.72	26.21	3.60
28	17.52	10.40	4.58	36.93	59.03	26.53	3.62
27	17.52	10.40	4.58	37.28	59.38	26.88	3.64
26	17.52	10.40	4.58	37.62	59.72	27.22	3.66
25	17.52	10.40	4.58	37.93	60.03	27.53	3.68
24	17.52	10.40	4.58	38.18	60.29	27.78	3.70
23	17.52	10.40	4.58	38.36	60.46	27.95	3.71
22	17.52	10.40	4.58	38.42	60.52	28.02	3.71
21	17.52	10.40	4.58	38.36	60.46	27.96	3.71
20	17.52	10.40	4.58	38.15	60.25	27.75	3.69
19	17.52	10.40	4.58	37.77	59.88	27.37	3.67
18	17.52	10.40	4.58	37.21	59.31	26.81	3.64
17	17.52	10.40	4.58	36.45	58.55	26.04	3.59
16	17.52	10.40	4.58	35.46	57.56	25.05	3.53
15	17.52	10.40	4.58	34.24	56.34	23.84	3.45
14	17.52	10.40	4.58	32.80	54.90	22.40	3.37
13	17.52	10.40	4.58	36.29	58.39	25.89	3.10
12	17.52	10.40	4.58	35.27	57.37	24.87	3.05
11	17.52	10.40	4.58	33.51	55.61	23.11	2.95
10	17.52	10.40	4.58	31.53	53.63	21.12	2.85
9	17.52	10.40	4.58	29.31	51.41	18.91	2.73
8	17.52	10.40	4.58	26.68	48.78	16.27	2.59
7	20.54	12.20	4.58	23.21	48.33	11.01	2.57
6	18.12	10.76	4.58	18.74	41.44	7.98	2.20
5	18.12	10.76	4.58	15.81	38.51	5.05	2.04
4	18.12	10.76	4.58	12.70	35.41	1.94	1.88
3	18.12	10.76	4.58	9.37	32.07	-1.39	1.70
2	18.48	10.98	4.58	5.57	28.63	-5.41	1.52

Table 18 – **CB1** Shear Forces, Including Specified Load Combinations (**Seismic**)

Level	**V_u due to gravity loads** (hand calculations)			**V_u due to lateral loads** (ETABS)	**TOTAL V_u** (kips)		$V_u/(\phi A_{cw}\sqrt{f'_c})$
	1.32D	0.78D	0.5L	1.0E	1.32D + 0.5L + 1.0E	0.78D + 1.0E	
roof level	11.01	6.54	3.57	11.18	25.77	4.64	1.58
29	13.65	8.10	3.57	12.26	29.48	4.16	1.81
28	13.65	8.10	3.57	12.36	29.58	4.26	1.81
27	13.65	8.10	3.57	12.50	29.72	4.40	1.82
26	13.65	8.10	3.57	12.66	29.88	4.55	1.83
25	13.65	8.10	3.57	12.83	30.05	4.72	1.84
24	13.65	8.10	3.57	13.00	30.22	4.90	1.85
23	13.65	8.10	3.57	13.17	30.39	5.07	1.86
22	13.65	8.10	3.57	13.33	30.55	5.23	1.87
21	13.65	8.10	3.57	13.48	30.70	5.37	1.88
20	13.65	8.10	3.57	13.60	30.82	5.50	1.89
19	13.65	8.10	3.57	13.70	30.92	5.59	1.90
18	13.65	8.10	3.57	13.76	30.98	5.66	1.90
17	13.65	8.10	3.57	13.79	31.01	5.68	1.90
16	13.65	8.10	3.57	13.78	31.00	5.68	1.90
15	13.65	8.10	3.57	13.73	30.95	5.62	1.90
14	13.65	8.10	3.57	13.59	30.81	5.48	1.89
13	13.65	8.10	3.57	15.44	32.66	7.34	1.73
12	13.65	8.10	3.57	15.39	32.61	7.28	1.73
11	13.65	8.10	3.57	15.24	32.46	7.14	1.72
10	13.65	8.10	3.57	15.07	32.29	6.97	1.71
9	13.65	8.10	3.57	14.87	32.09	6.76	1.70
8	13.65	8.10	3.57	14.52	31.74	6.42	1.69
7	25.83	15.34	5.77	14.26	45.86	-1.07	2.43
6	22.79	13.53	5.77	13.02	41.57	-0.52	2.21
5	22.79	13.53	5.77	11.22	39.77	-2.31	2.11
4	22.79	13.53	5.77	9.17	37.73	-4.36	2.00
3	22.79	13.53	5.77	6.87	35.42	-6.67	1.88
2	23.25	13.80	5.77	4.07	33.08	-9.73	1.76

Table 19 – **CB2** Shear Forces, Including Specified Load Combinations (**Seismic**)

Because $V_u/(\phi A_{cw}\sqrt{f'_c})$ was less than 12 for the wind load cases and less than 4 for the seismic load cases, diagonal reinforcement was not required. Therefore, the analysis continued with the proposed design.

Shear Wall Reinforcement

The shear wall reinforcement was initially designed according to the seismic section 21.9 of ACI 318-08, which is entitled 'Special structural walls and coupling beams'. Then, the reinforcement was checked to see if it would be adequate under the wind loads. In some cases, the rebar size had to be increased or the spacing had to be decreased in order to have sufficient strength. Two curtains of reinforcement were used everywhere due to the thickness of the walls.

The seismic shear forces in each wall at each level were determined by ETABS, and these values were compared to $A_{cv}\lambda\sqrt{f'_c}$ (where A_{cv} was the wall length multiplied by the wall thickness) to determine if the required reinforcement ratio could be reduced. For the transverse (horizontal) reinforcement, $\rho_t \geq 0.0025$ if V_u exceeded this value, but if not, $\rho_t \geq 0.0020$. Similarly, for the longitudinal reinforcement, $\rho_l \geq 0.0025$ if V_u exceeded this value, but if not, $\rho_t \geq 0.0012$. However, the reinforcement ratio could only be reduced for rebar sizes #5 and smaller. Using the required ratio, a preliminary design for the rebar and spacing was determined for the reinforcement based on the seismic loads. The shear capacity of the wall with this preliminary design was determined by the equation from section 21.9.4 of ACE 319-08.

$$\emptyset V_n = \emptyset A_{cv}(\alpha_c \lambda \sqrt{f_c'} + \rho_t f_y)$$

- ϕ = 0.60 for seismic design
- α_c = 2.0, based on the height/length ratio of the walls
- ρ_t = transverse reinforcement ratio as calculated in the spreadsheet
- f_y = 60 ksi for grade 60 steel

In order to check the reinforcement for the wind loads, the factored shear forces were retrieved from the ETABS model. Then, a simple calculation, $V_c = 2\lambda\sqrt{f'_c}t_w d$ (ACI 318-08, eqn. 11-3), was done to check the shear capacity of the concrete. An approximation of $0.8l_w$ was made for d. In most cases, $0.5\phi V_c$ did not exceed the factored shear force, so chapter 11 of ACI 318-08 was required for the reinforcement design. The shear capacity of the steel was calculated with the steel area of the preliminary rebar design ($V_s = A_{s,t}f_y d/s$, ACI 318-08, eqn. 11-15). Then, the total capacity of the wall $\phi(V_c + V_s)$ was calculated and compared to V_u. For SW1, the transverse reinforcement only needed to be changed for the first two levels.

For the longitudinal reinforcement, the following equation was used to determine the minimum reinforcement ratio (ACI 318-08, eqn. 11-30).

$$\rho_l = 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - 0.0025)$$

This equation became the limiting factor for almost all of the walls, so the vertical reinforcement had to be increased. The calculation spreadsheets for SW1 are shown on the following pages. Table 20 shows the preliminary transverse reinforcement, and table 21 shows the preliminary longitudinal reinforcement based on the seismic loads. The last two tables (22 and 23) show the final shear wall reinforcement for SW1 after the wind load check. Similar tables for the remaining five shear walls can be found in the Appendix.

	Seismic								
Level	Seismic Shear Force (from ETABS), V_u (k)	$A_{cv}\lambda\sqrt{f'_c}$ (k)	Transverse (horizontal) Reinforcement						
			Minimum Required ρ_t	Required $A_{s,t}$ (in^2/ft)	Preliminary Rebar and Spacing		Actual $A_{s,t}$ (in^2/ft)	Actual ρ_t
sky lobby roof level	13.1	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
roof level	27.0	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
29	61.2	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
28	93.0	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
27	116.6	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
26	138.7	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
25	159.2	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
24	178.3	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
23	196.3	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
22	213.0	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
21	228.4	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
20	242.6	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
19	255.7	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
18	268.4	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
17	279.0	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
16	273.8	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
15	295.4	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
14	312.4	430.8	0.0020	0.432	(2) #5 @	12	0.62	0.0029
13	329.1	663.3	0.0020	0.576	(2) #5 @	9	0.83	0.0029
12	316.3	663.3	0.0020	0.576	(2) #5 @	9	0.83	0.0029
11	306.1	663.3	0.0020	0.576	(2) #5 @	9	0.83	0.0029
10	279.4	663.3	0.0020	0.576	(2) #5 @	9	0.83	0.0029
9	226.6	663.3	0.0020	0.576	(2) #5 @	9	0.83	0.0029
8	72.4	663.3	0.0020	0.576	(2) #5 @	9	0.83	0.0029
7	176.4	553.8	0.0020	0.576	(2) #5 @	9	0.83	0.0029
6	235.1	553.8	0.0020	0.576	(2) #5 @	9	0.83	0.0029
5	318.4	553.8	0.0020	0.576	(2) #5 @	9	0.83	0.0029
4	482.9	553.8	0.0020	0.576	(2) #5 @	9	0.83	0.0029
3	909.7	553.8	0.0025	0.720	(2) #5 @	9	0.83	0.0029
2	832.4	553.8	0.0025	0.720	(2) #5 @	9	0.83	0.0029

Table 20 – Preliminary Transverse Reinforcement for **SW1**

	Seismic						
Level	Longitudinal (vertical) Reinforcement						Shear Capacity (k), $\phi Vn = \phi A_{cv}(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y)$
	Minimum Required ρ_l	Required $A_{s,l}$ (in^2/ft)	Preliminary Rebar and Spacing		Actual $A_{s,l}$ (in^2/ft)	Actual ρ_l	
sky lobby roof level	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
roof level	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
29	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
28	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
27	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
26	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
25	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
24	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
23	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
22	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
21	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
20	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
19	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
18	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
17	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
16	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
15	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
14	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1091.7
13	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1562.3
12	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1562.3
11	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1562.3
10	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1562.3
9	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1562.3
8	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1562.3
7	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1304.4
6	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1304.4
5	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1304.4
4	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1304.4
3	0.0025	0.720	(2) #5 @	9	0.827	0.0029	1304.4
2	0.0025	0.720	(2) #5 @	9	0.827	0.0029	1304.4

Table 21 – Preliminary Longitudinal Reinforcement and Total Shear Capacity for **SW1**

	Wind								
Level	Wind Shear Force (from ETABS), Vu (k)	$V_c = 2\sqrt{f'_c}t_w d$ (k)	Transverse (horizontal) Reinforcement						
			$V_s = A_{s,t}f_y d/s$ (k)	$\phi V_n = \phi(V_c+V_s)$ (k)	Modified Rebar and Spacing		$A_{s,t}$ (in^2/ft)	NEW $\phi V_n = \phi(V_c+V_s)$ (k)	ρ_t
sky lobby roof level	20.2	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
roof level	55.3	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
29	64.8	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
28	91.4	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
27	121.0	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
26	150.5	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
25	179.8	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
24	208.9	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
23	237.4	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
22	265.1	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
21	293.3	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
20	321.1	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
19	349.0	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
18	377.2	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
17	404.5	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
16	414.8	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
15	455.2	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
14	489.9	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029
13	535.8	1061.3	1021.8	1562.3	(2) #5 @	9	0.827	1562.3	0.0029
12	534.7	1061.3	1021.8	1562.3	(2) #5 @	9	0.827	1562.3	0.0029
11	536.1	1061.3	1021.8	1562.3	(2) #5 @	9	0.827	1562.3	0.0029
10	510.2	1061.3	1021.8	1562.3	(2) #5 @	9	0.827	1562.3	0.0029
9	440.4	1061.3	1021.8	1562.3	(2) #5 @	9	0.827	1562.3	0.0029
8	290.8	1061.3	1021.8	1562.3	(2) #5 @	9	0.827	1562.3	0.0029
7	649.2	886.1	853.1	1304.4	(2) #5 @	9	0.827	1304.4	0.0029
6	524.0	886.1	853.1	1304.4	(2) #5 @	9	0.827	1304.4	0.0029
5	683.4	886.1	853.1	1304.4	(2) #5 @	9	0.827	1304.4	0.0029
4	996.9	886.1	853.1	1304.4	(2) #5 @	9	0.827	1304.4	0.0029
3	1807.1	886.1	853.1	1304.4	(2) #7 @	9	1.600	1903.0	0.0056
2	1628.3	886.1	853.1	1304.4	(2) #7 @	9	1.600	1903.0	0.0056

Table 22 – Final Transverse Reinforcement for **SW1**

	Wind				
	Longitudinal (vertical) Reinforcement				
Level	Minimum Required ρ_l	Modified Rebar and Spacing		$A_{s,l}$ (in^2/ft)	ρ_l
sky lobby roof level	0.0029	(2) #5 @	10	0.744	0.0034
roof level	0.0029	(2) #5 @	10	0.744	0.0034
29	0.0029	(2) #5 @	10	0.744	0.0034
28	0.0029	(2) #5 @	10	0.744	0.0034
27	0.0029	(2) #5 @	10	0.744	0.0034
26	0.0029	(2) #5 @	10	0.744	0.0034
25	0.0029	(2) #5 @	10	0.744	0.0034
24	0.0029	(2) #5 @	10	0.744	0.0034
23	0.0029	(2) #5 @	10	0.744	0.0034
22	0.0029	(2) #5 @	10	0.744	0.0034
21	0.0029	(2) #5 @	10	0.744	0.0034
20	0.0029	(2) #5 @	10	0.744	0.0034
19	0.0029	(2) #5 @	10	0.744	0.0034
18	0.0029	(2) #5 @	10	0.744	0.0034
17	0.0029	(2) #5 @	10	0.744	0.0034
16	0.0029	(2) #5 @	10	0.744	0.0034
15	0.0029	(2) #5 @	10	0.744	0.0034
14	0.0029	(2) #5 @	10	0.744	0.0034
13	0.0029	(2) #5 @	8	0.930	0.0032
12	0.0029	(2) #5 @	8	0.930	0.0032
11	0.0029	(2) #5 @	8	0.930	0.0032
10	0.0029	(2) #5 @	8	0.930	0.0032
9	0.0029	(2) #5 @	8	0.930	0.0032
8	0.0029	(2) #5 @	8	0.930	0.0032
7	0.0029	(2) #5 @	8	0.930	0.0032
6	0.0029	(2) #5 @	8	0.930	0.0032
5	0.0029	(2) #5 @	8	0.930	0.0032
4	0.0029	(2) #5 @	8	0.930	0.0032
3	0.0057	(2) #7 @	8	1.800	0.0063
2	0.0055	(2) #7 @	8	1.800	0.0063

Table 23 – Final Longitudinal Reinforcement for **SW1**

The computer program pcaColumn was used to do a strength check of one of the I-shaped shear walls at level 9, which is approximately at the base of the tower. The load cases for both the north-south and east-west directions were considered, and the wall section was analyzed biaxially. Based on the factored axial loads and bending moments, flexural reinforcement was required. Figure 11 below shows the final reinforcement layout for this level. The general wall reinforcement consists of #5 bars at either 8” or 12” spacing. For the boundary elements, which are indicated with teal boxes, #11 bars are used.

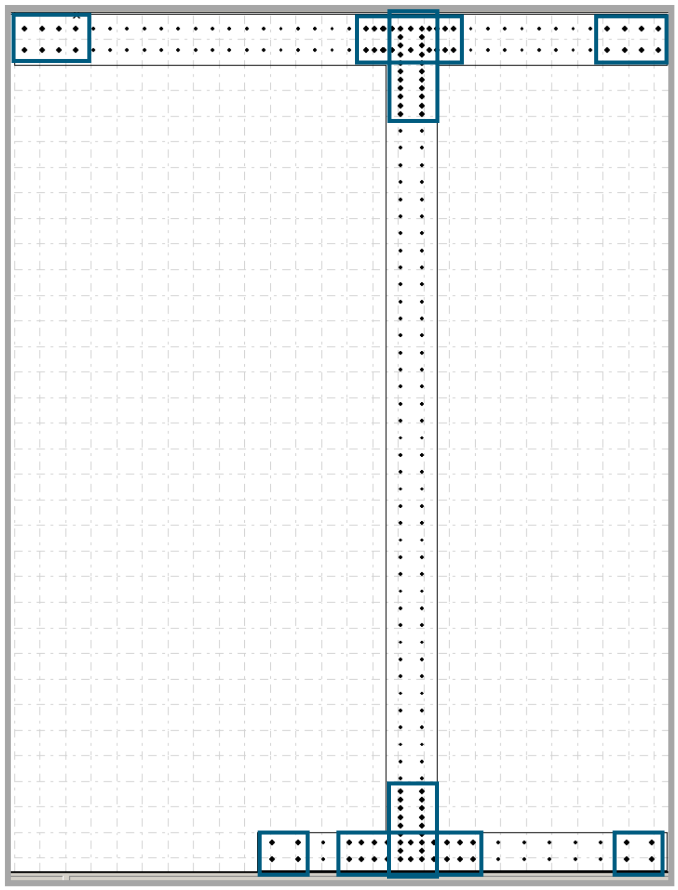

Figure 11 – Vertical Reinforcement Layout for Strength Check of Shear Wall

With the final reinforcement layout shown above in figure 11, the following moment interaction diagrams were determined through pcaColumn. The analysis was completed for the factored wind loads, which is the more critical loading condition. In each figure, the data points correspond to the axial forces and moments in the walls if both 100% of the wind load in one direction and 30% of the wind load in the other direction are applied simultaneously. In all cases, the reinforcement is adequate because the data points falls within the graph.

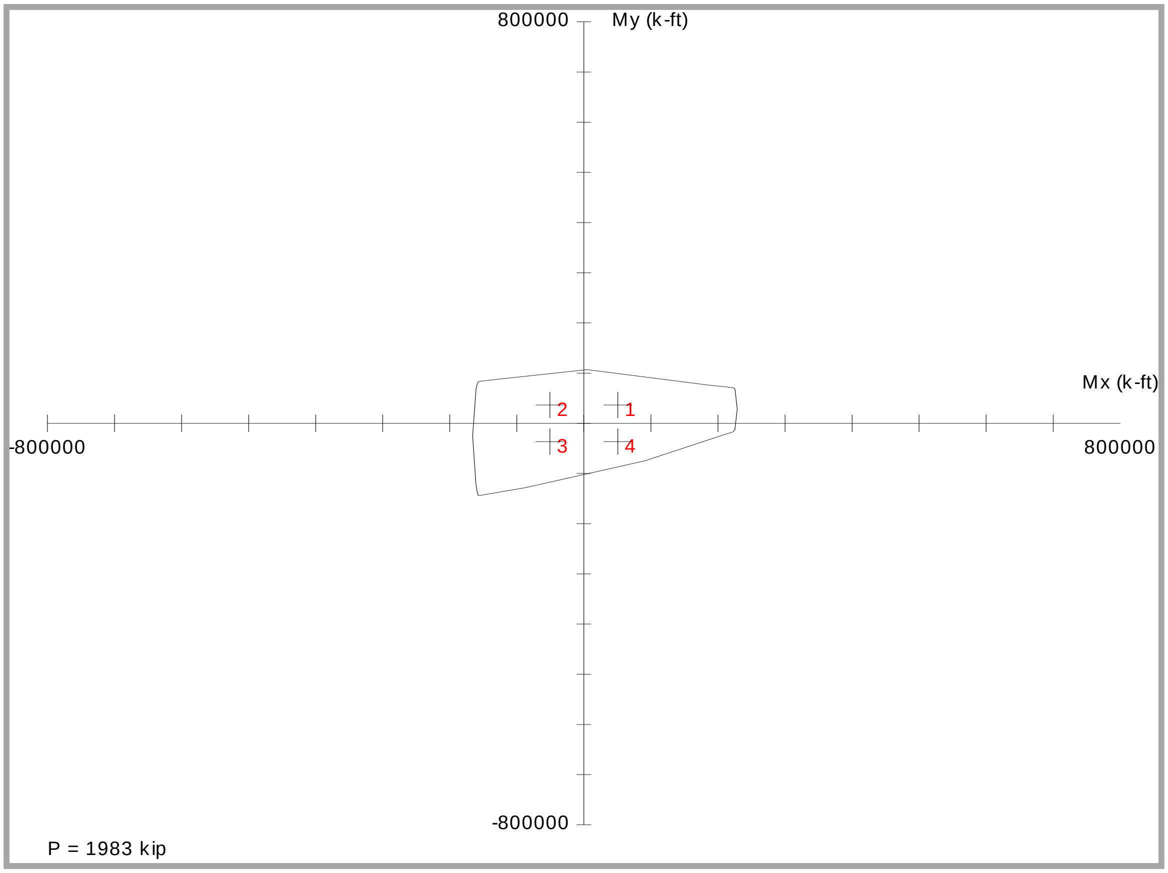

Figure 12 – Moment Interaction Diagram for Strength Check of Shear Wall
(100% Wind in North-South Direction,
30% Wind in East-West Direction)

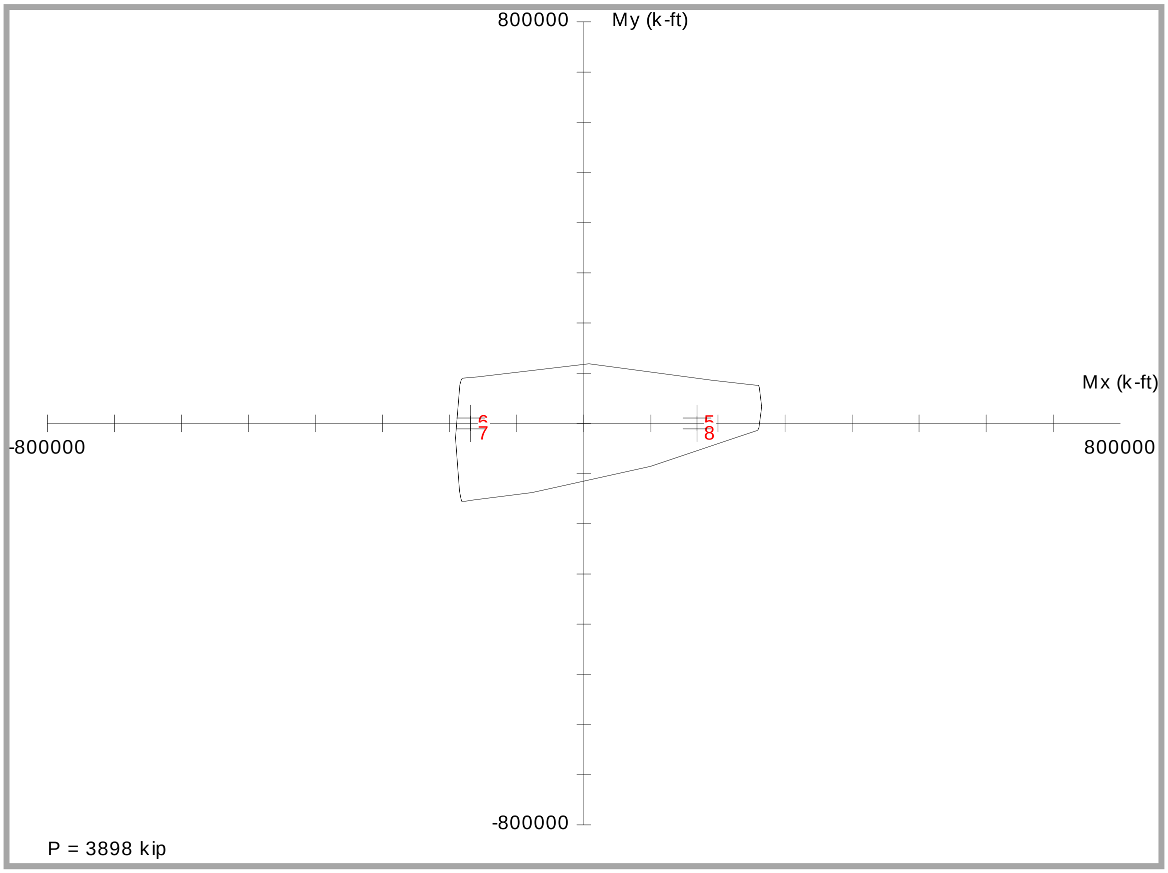

Figure 13 – Moment Interaction Diagram for Strength Check of Shear Wall
(100% Wind in East-West Direction,
30% Wind in North-South Direction)

Coupling Beam Reinforcement

The coupling beams required both longitudinal and shear (stirrup) reinforcement. For the longitudinal reinforcement, there were two stipulations for the minimum amount of required steel (for both the top and the bottom of the beam).

- $A_{s,min} = \frac{3\sqrt{f'_c}b_w d}{f_y}$ (ACI 318-08, eqn. 10-3)
- $A_s \geq \frac{200 b_w d}{f_y}$ (ACI 318-08, section 21.5.2.1)

The maximum factored moment from ETABS was used to calculate the required area of steel. A derivation (assuming A_s = 0.0125bd and ρ = 1.25%) of the two fundamental equations, $M_n = A_s f_y(d\text{-}a/2)$ and $a = A_s f_y/(0.85f'_c b)$, leads to the following equations to be used for reinforcement design.

- $A_s = \frac{M_u}{4.17d}$ (for f'_c = 6000 psi)
- $A_s = \frac{M_u}{4.25d}$ (for f'_c = 8000 psi)

For most of the floors, anywhere from (6) – (10) #7 bars were sufficient for the top and bottom of the beam, which was allowable by code. According to ACI 318-08, section 21.5.2.1, the limit for the reinforcement ratio is 0.025, and this value was not exceeded in this design. See tables 24 and 25 for the results for the longitudinal reinforcement for each coupling beam.

	Longitudinal Reinforcement						
Level	$A_{s,min} = 3\sqrt{f'_c} b_w d/f_y$ (in^2)	$A_s \geq 200 b_w d/f_y$ (in^2)	M_u, from ETABS (k-in)	$A_s \geq M_u/4.17d$ (f'_c=6 ksi) $A_s \geq M_u/4.25d$ (f'_c=8 ksi)	Required Reinforcement (top and bottom)	Actual A_s (in^2)	$\rho \leq 0.025$?
sky lobby roof level	1.088	0.936	3778.5	4.840	(10) #7	6.00	0.017
roof level	1.088	0.936	4141.8	5.306	(10) #7	6.00	0.017
29	1.088	0.936	4168.3	5.340	(10) #7	6.00	0.017
28	1.088	0.936	4205.7	5.388	(10) #7	6.00	0.017
27	1.088	0.936	4249.6	5.444	(10) #7	6.00	0.017
26	1.088	0.936	4297.4	5.505	(10) #7	6.00	0.017
25	1.088	0.936	4346.3	5.568	(10) #7	6.00	0.017
24	1.088	0.936	4393.5	5.628	(10) #7	6.00	0.017
23	1.088	0.936	4436.3	5.683	(10) #7	6.00	0.017
22	1.088	0.936	4471.6	5.728	(10) #7	6.00	0.017
21	1.088	0.936	4496.9	5.761	(10) #7	6.00	0.017
20	1.088	0.936	4509.3	5.776	(10) #7	6.00	0.017
19	1.088	0.936	4506.0	5.772	(10) #7	6.00	0.017
18	1.088	0.936	4484.2	5.744	(10) #7	6.00	0.017
17	1.088	0.936	4440.9	5.689	(10) #7	6.00	0.017
16	1.088	0.936	4372.6	5.601	(10) #7	6.00	0.017
15	1.088	0.936	4278.2	5.481	(10) #7	6.00	0.017
14	1.088	0.936	4155.4	5.323	(10) #7	6.00	0.017
13	1.256	0.936	4668.4	5.868	(10) #7	6.00	0.017
12	1.256	0.936	4587.7	5.766	(10) #7	6.00	0.017
11	1.256	0.936	4410.2	5.543	(10) #7	6.00	0.017
10	1.256	0.936	4202.2	5.282	(10) #7	6.00	0.017
9	1.256	0.936	3960.7	4.978	(10) #7	6.00	0.017
8	1.256	0.936	3658.5	4.598	(8) #7	4.80	0.014
7	1.256	0.936	3246.3	4.080	(8) #7	4.80	0.014
6	1.256	0.936	2688.1	3.379	(6) #7	3.60	0.010
5	1.256	0.936	2299.2	2.890	(6) #7	3.60	0.010
4	1.256	0.936	1873.0	2.354	(4) #7	2.40	0.007
3	1.256	0.936	1400.5	1.760	(4) #7	2.40	0.007
2	1.256	0.936	845.1	1.062	(2) #7	1.20	0.003

Table 24 – Longitudinal Reinforcement for CB1

	Longitudinal Reinforcement						
Level	$A_{s,min} = 3\sqrt{f'_c} b_w d/f_y$ (in^2)	$A_s \geq 200 b_w d/f_y$ (in^2)	M_u, from ETABS (k-in)	$A_s \geq M_u/4.17d$ (f'_c=6 ksi) $A_s \geq M_u/4.25d$ (f'_c=8 ksi)	Required Reinforcement (top and bottom)	Actual A_s (in^2)	$\rho \leq 0.025$?
roof level	1.088	0.936	2846.6	3.647	(8) #7	4.80	0.014
29	1.088	0.936	3119.3	3.996	(8) #7	4.80	0.014
28	1.088	0.936	3140.4	4.023	(8) #7	4.80	0.014
27	1.088	0.936	3167.2	4.057	(8) #7	4.80	0.014
26	1.088	0.936	3196.5	4.095	(8) #7	4.80	0.014
25	1.088	0.936	3226.1	4.133	(8) #7	4.80	0.014
24	1.088	0.936	3254.3	4.169	(8) #7	4.80	0.014
23	1.088	0.936	3278.9	4.200	(8) #7	4.80	0.014
22	1.088	0.936	3297.8	4.225	(8) #7	4.80	0.014
21	1.088	0.936	3309.1	4.239	(8) #7	4.80	0.014
20	1.088	0.936	3310.9	4.241	(8) #7	4.80	0.014
19	1.088	0.936	3300.9	4.229	(8) #7	4.80	0.014
18	1.088	0.936	3277.0	4.198	(8) #7	4.80	0.014
17	1.088	0.936	3236.2	4.146	(8) #7	4.80	0.014
16	1.088	0.936	3173.2	4.065	(8) #7	4.80	0.014
15	1.088	0.936	3091.8	3.961	(8) #7	4.80	0.014
14	1.088	0.936	2994.8	3.836	(8) #7	4.80	0.014
13	1.256	0.936	3309.2	4.159	(8) #7	4.80	0.014
12	1.256	0.936	3193.5	4.014	(8) #7	4.80	0.014
11	1.256	0.936	3039.5	3.820	(8) #7	4.80	0.014
10	1.256	0.936	2861.0	3.596	(6) #7	3.60	0.010
9	1.256	0.936	2651.9	3.333	(6) #7	3.60	0.010
8	1.256	0.936	2380.9	2.993	(6) #7	3.60	0.010
7	1.256	0.936	2251.1	2.829	(6) #7	3.60	0.010
6	1.256	0.936	2036.5	2.560	(6) #7	3.60	0.010
5	1.256	0.936	1770.4	2.225	(4) #7	2.40	0.007
4	1.256	0.936	1461.6	1.837	(4) #7	2.40	0.007
3	1.256	0.936	1105.7	1.390	(4) #7	2.40	0.007
2	1.256	0.936	669.0	0.841	(2) #7	1.20	0.003

Table 25 – Longitudinal Reinforcement for CB2

For the design of the stirrups, the strength of the concrete was neglected as a conservative assumption (based on ACI 318-08, section 21.5.4.2). The maximum V_u due to both wind and seismic loads, as previously calculated for the feasibility test of the coupling beams, was divided by the applicable ϕ factor to determine the required shear strength of the steel. The following limitations applied to the value for V_s.

- $V_s \leq 8\sqrt{f_c'}\, b_w d$ (ACI 318-08, section 11.4.7.9)

The minimum area of the shear reinforcement was determined with section 11.4.6.3 of the concrete building code.

- $A_{v,min} = \frac{0.75\sqrt{f_c'}\, b_w s}{f_y}$
- $A_{v,min} = \frac{50 b_w s}{f_y}$

The resulting steel design was (3) – (4) legs of #3 stirrups in each beam. The maximum allowable spacing was calculated (using a more accurate d = 16”), but the actual spacing was limited to 4” by section 21.5.3.2. A summary of the coupling beam reinforcement calculations can be found in the tables on the following pages. The sketches below show the transverse reinforcement for a typical coupling beam. Figure 14 shows the spacing of the reinforcement across the length of the beam, and figure 15 gives a view of a typical beam cross-section. The two hoops overlap in the beam, and they have 3” extensions where the ends of the bars extend into the interior concrete.

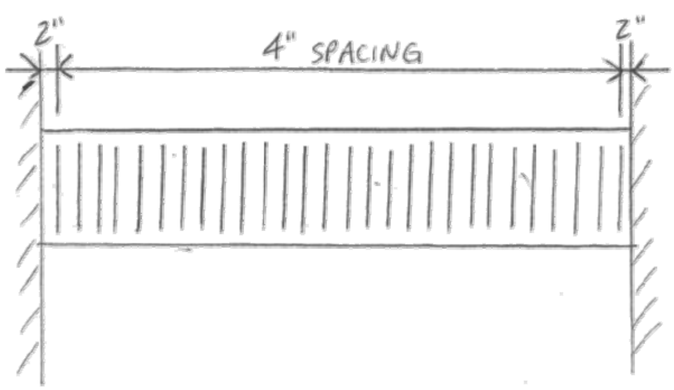

Figure 14 – Stirrup Spacing for a Coupling Beam

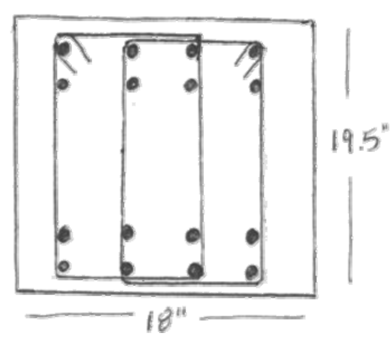

Figure 15 – Typical beam cross-section

	Shear Reinforcement (stirrups)											
Level	Wind		Seismic			$V_s \leq 8\sqrt{f'_c}b_wd$ (k)	$A_{v,min}$ (in^2)		Preliminary A_v (#3 bars, 0.11 in^2 per leg)	$s_{max} = A_vf_yd/V_s$ (in)	s_{actual} (in)	$V_{s,actual} = A_vf_yd/s$ (k)
	Max V_u (k)	$V_{s,required} \geq V_u/\phi$ (k)	Max V_u (k)	$V_{s,required} \geq V_u/\phi$ (k)	$V_s \leq 10A_{cw}\sqrt{f'_c}$ (k)		$0.75\sqrt{f'_c}b_ws/f_y$	$50b_ws/f_y$				
sky lobby roof level	54.80	73.1	46.97	78.28	271.9	174.0	0.070	0.060	0.330	4.047	4.0	79.2
roof level	55.81	74.4	46.51	77.52	271.9	174.0	0.070	0.060	0.330	4.087	4.0	79.2
29	59.13	78.8	50.07	83.45	271.9	174.0	0.070	0.060	0.440	5.062	4.0	105.6
28	59.48	79.3	50.31	83.85	271.9	174.0	0.070	0.060	0.440	5.038	4.0	105.6
27	59.89	79.8	50.58	84.30	271.9	174.0	0.070	0.060	0.440	5.011	4.0	105.6
26	60.33	80.4	50.84	84.73	271.9	174.0	0.070	0.060	0.440	4.985	4.0	105.6
25	60.78	81.0	51.08	85.13	271.9	174.0	0.070	0.060	0.440	4.962	4.0	105.6
24	61.22	81.6	51.28	85.47	271.9	174.0	0.070	0.060	0.440	4.942	4.0	105.6
23	61.61	82.2	51.42	85.70	271.9	174.0	0.070	0.060	0.440	4.929	4.0	105.6
22	61.94	82.6	51.47	85.78	271.9	174.0	0.070	0.060	0.440	4.924	4.0	105.6
21	62.18	82.9	51.43	85.72	271.9	174.0	0.070	0.060	0.440	4.928	4.0	105.6
20	62.29	83.1	51.28	85.47	271.9	174.0	0.070	0.060	0.440	4.942	4.0	105.6
19	62.26	83.0	51.00	85.00	271.9	174.0	0.070	0.060	0.440	4.969	4.0	105.6
18	62.06	82.7	50.58	84.30	271.9	174.0	0.070	0.060	0.440	5.011	4.0	105.6
17	61.66	82.2	50.00	83.33	271.9	174.0	0.070	0.060	0.440	5.069	4.0	105.6
16	61.02	81.4	49.26	82.10	271.9	174.0	0.070	0.060	0.440	5.145	4.0	105.6
15	60.15	80.2	48.34	80.57	271.9	174.0	0.070	0.060	0.440	5.243	4.0	105.6
14	59.01	78.7	47.25	78.75	271.9	174.0	0.070	0.060	0.440	5.364	4.0	105.6
13	63.76	85.0	49.95	83.25	313.9	200.9	0.080	0.060	0.440	4.968	4.0	105.6
12	63.02	84.0	49.19	81.98	313.9	200.9	0.080	0.060	0.440	5.027	4.0	105.6
11	61.37	81.8	47.85	79.75	313.9	200.9	0.080	0.060	0.440	5.162	4.0	105.6
10	59.45	79.3	46.35	77.25	313.9	200.9	0.080	0.060	0.440	5.329	4.0	105.6
9	57.21	76.3	44.67	74.45	313.9	200.9	0.080	0.060	0.330	4.153	4.0	79.2
8	54.41	72.6	42.67	71.12	313.9	200.9	0.080	0.060	0.330	4.367	4.0	79.2
7	53.35	71.1	43.05	71.75	313.9	200.9	0.080	0.060	0.330	4.415	4.0	79.2
6	45.97	61.3	37.22	62.03	313.9	200.9	0.080	0.060	0.330	5.107	4.0	79.2
5	42.38	56.5	34.96	58.27	313.9	200.9	0.080	0.060	0.330	5.437	4.0	79.2
4	38.43	51.2	32.56	54.27	313.9	200.9	0.080	0.060	0.330	5.838	4.0	79.2
3	34.05	45.4	29.98	49.97	313.9	200.9	0.080	0.060	0.330	6.340	4.0	79.2
2	29.24	39.0	27.39	45.65	313.9	200.9	0.080	0.060	0.330	6.940	4.0	79.2

Table 26 – Stirrup Reinforcement for CB1

	Shear Reinforcement (stirrups)											
Level	Wind		Seismic			$V_s \le$ $8\sqrt{f'_c}b_wd$ (k)	$A_{v,min}$ (in^2)		Preliminary A_v (#3 bars, 0.11 in^2 per leg)	s_{max} = A_vf_yd/V_s (in)	s_{actual} (in)	$V_{s,actual}$ = A_vf_yd/s (k)
	Max V_u (k)	$V_{s,required} \ge V_u/\phi$ (k)	Max V_u (k)	$V_{s,required} \ge V_u/\phi$ (k)	$V_s \le 10A_{cw}\sqrt{f'_c}$ (k)		$0.75\sqrt{f'_c}b_ws/f_y$	$50b_ws/f_y$				
roof level	39.96	53.3	24.05	40.08	271.9	174.0	0.070	0.060	0.330	5.946	4.0	79.2
29	44.88	59.8	27.59	45.98	271.9	174.0	0.070	0.060	0.330	5.294	4.0	79.2
28	45.08	60.1	27.68	46.13	271.9	174.0	0.070	0.060	0.330	5.271	4.0	79.2
27	45.33	60.4	27.79	46.32	271.9	174.0	0.070	0.060	0.330	5.242	4.0	79.2
26	45.60	60.8	27.92	46.53	271.9	174.0	0.070	0.060	0.330	5.211	4.0	79.2
25	45.87	61.2	28.06	46.77	271.9	174.0	0.070	0.060	0.330	5.179	4.0	79.2
24	46.13	61.5	28.19	46.98	271.9	174.0	0.070	0.060	0.330	5.150	4.0	79.2
23	46.36	61.8	28.33	47.22	271.9	174.0	0.070	0.060	0.330	5.125	4.0	79.2
22	46.54	62.0	28.45	47.42	271.9	174.0	0.070	0.060	0.330	5.106	4.0	79.2
21	46.64	62.2	28.56	47.60	271.9	174.0	0.070	0.060	0.330	5.094	4.0	79.2
20	46.66	62.2	28.64	47.73	271.9	174.0	0.070	0.060	0.330	5.092	4.0	79.2
19	46.57	62.1	28.70	47.83	271.9	174.0	0.070	0.060	0.330	5.102	4.0	79.2
18	46.34	61.8	28.74	47.90	271.9	174.0	0.070	0.060	0.330	5.127	4.0	79.2
17	45.97	61.3	28.73	47.88	271.9	174.0	0.070	0.060	0.330	5.169	4.0	79.2
16	45.38	60.5	28.70	47.83	271.9	174.0	0.070	0.060	0.330	5.235	4.0	79.2
15	44.63	59.5	28.62	47.70	271.9	174.0	0.070	0.060	0.330	5.324	4.0	79.2
14	43.73	58.3	28.48	47.47	271.9	174.0	0.070	0.060	0.330	5.433	4.0	79.2
13	46.64	62.2	29.98	49.97	313.9	200.9	0.080	0.060	0.330	5.094	4.0	79.2
12	45.57	60.8	29.89	49.82	313.9	200.9	0.080	0.060	0.330	5.214	4.0	79.2
11	44.15	58.9	29.73	49.55	313.9	200.9	0.080	0.060	0.330	5.382	4.0	79.2
10	42.49	56.7	29.54	49.23	313.9	200.9	0.080	0.060	0.330	5.592	4.0	79.2
9	40.56	54.1	29.31	48.85	313.9	200.9	0.080	0.060	0.330	5.858	4.0	79.2
8	38.05	50.7	28.96	48.27	313.9	200.9	0.080	0.060	0.330	6.245	4.0	79.2
7	50.13	66.8	43.03	71.72	313.9	200.9	0.080	0.060	0.330	4.417	4.0	79.2
6	45.38	60.5	38.91	64.85	313.9	200.9	0.080	0.060	0.330	4.885	4.0	79.2
5	42.91	57.2	37.47	62.45	313.9	200.9	0.080	0.060	0.330	5.073	4.0	79.2
4	40.06	53.4	35.84	59.73	313.9	200.9	0.080	0.060	0.330	5.304	4.0	79.2
3	36.76	49.0	34.01	56.68	313.9	200.9	0.080	0.060	0.330	5.589	4.0	79.2
2	33.10	44.1	32.27	53.78	313.9	200.9	0.080	0.060	0.330	5.890	4.0	79.2

Table 27 – Stirrup Reinforcement for CB2

Impact on Existing Foundation

The existing foundation consists of drilled piles, and the loads are transferred from the superstructure to the foundation via pile caps or a mat slab. Most columns are supported by a pile cap, but a mat slab is used where there are numerous columns and shear walls close together. For the existing design, there are both drilled tension piles and drilled compression piles, with the following capacities.

- Tension pile (generally 18”ϕ x 80’-0”)
    - Compression capacity = 200 tons
    - Lateral capacity = 20 tons
    - Tension capacity = 40 tons
- Compression pile (generally 18”ϕ x 80’-0”)
    - Compression capacity = 200 tons
    - Lateral capacity = 20 tons

For the mat slab supporting the area below the new shear wall design, there were 23 tension piles and 147 compression piles in the original design. For this analysis, the impact on the existing foundation was analyzed for lateral forces and overturning moments, because the focus of this structural analysis was on the lateral force resisting system of the building. Using output from ETABS, the maximum shear forces and overturning moments (based on the defined load combinations) were found at the base of the building. In table 28 below, the base shear in the two I-shaped shear walls was found in each direction for all of the load cases. Then, the forces were summed for each load case to find the base shear being transferred in the mat slab in each direction. Lastly, the maximum shear force was converted to tons and number of required piles. Because both the compression and tension piles have lateral capacity, the original design utilizes up to 170 piles to resist the base shear, which exceeds the number of required piles based on this new shear wall design.

	North-South Direction			East-West Direction		
	Shear Force (k) in I-Shape Shear Wall		Total Shear (k)	Shear Force (k) in I-Shape Shear Wall		Total Shear (k)
	Web = SW5	Web = SW6		Web = SW5	Web = SW6	
Wind Case 1a	0.4	0.4	0.8	1800.3	161.5	1961.8
Wind Case 1b	1492.9	1458.7	2951.6	155.7	148.3	304.0
Wind Case 2a	80.1	71.9	151.9	1356.6	115.9	1472.4
Wind Case 2b	827.0	1356.2	2183.1	93.3	90.6	183.9
Wind Case 3	1121.8	1092.4	2214.2	1233.6	232.5	1466.1
Wind Case 4	560.0	1070.9	1630.9	947.2	154.9	1102.0
Seismic - NS	3.5	6.2	9.7	860.3	71.8	932.1
Seismic - EW	251.8	288.9	540.7	38.9	37.0	75.9
		Total Shear (tons) =	1475.8		Total Shear (tons) =	980.9
		Required Piles =	**73.8**		Required Piles =	**49.0**

Table 28 – Number of Required Piles Based on Lateral Capacity

Similarly, an analysis was done for the overturning moment at the base of the building. The existing design has tension piles oriented to resist moments in the north-south direction, so ETABS output was used to find the N-S overturning moment in each I-shaped pier for the various load combinations. The base moment was then divided by the maximum length of the flange of the shear wall (25.75') in order to find the tension force at the extreme fiber of the wall. This tension force was then converted to tons and the corresponding number of piles. It can be seen below in table 29 that the required number of piles based on this analysis is much larger than the original design (23 tension piles).

	Web = SW5			Web = SW6			Total Tension Force (tons)	Number of Required Piles
	Base Moment (k-ft)	Tension Force (k)	Tension Force (tons)	Moment (k-ft)	Tension Force (k)	Tension Force (tons)		
Wind Case 1a	98949.1	3842.7	1921.3	83688.2	3250.0	1625.0	3546.4	88.7
Wind Case 1b	15767.9	612.3	306.2	17581.5	682.8	341.4	647.6	16.2
Wind Case 2a	74334.1	2886.8	1443.4	62585.1	2430.5	1215.2	2658.6	66.5
Wind Case 2b	11419.4	443.5	221.7	12427.9	482.6	241.3	463.1	11.6
Wind Case 3	62400.2	2423.3	1211.7	75966.0	2950.1	1475.1	2686.7	67.2
Wind Case 4	47171.5	1831.9	916.0	56244.5	2184.3	1092.1	2008.1	50.2
Seismic - NS	52404.2	2035.1	1017.6	44329.3	1721.5	860.8	1878.3	47.0
Seismic - EW	4200.3	163.1	81.6	4680.7	181.8	90.9	172.4	4.3

Table 29 – Number of Required Piles Based on Tension Capacity

The reason for the large amount of tension piles is because the ETABS model was only used for lateral loads, so there is no inclusion of gravity loads to resist the building uplift. Therefore, to account for gravity loads, the approximate dead loads for each floor (as found for the calculation of seismic base shear) were multiplied by the area of the mat slab (100' x 60'). The dead loads were summed (2600 k = 1300 tons), and converted to the number of piles corresponding to a 40 ton capacity. It was found that the loads resisting uplift, due to the weight of the building, accounted for about 33 piles. As a result, the number of tension piles required due to wind load case 1a is reduced from 89 to 56. This number could possibly be reduced even more with a more detailed analysis.

These foundation calculations were a preliminary and quick way to check the impact of the new shear wall design on the existing foundation. Based on the analysis, the base shear forces of the new shear walls would not affect the foundation. In fact, fewer piles are required to resist the base shear. As for the overturning moments, the new shear wall design requires about 2.5 times the number of existing tension piles. However, about 33 of the existing compression piles could be converted to tension piles to account for this difference in overturning moments. Because the tension piles have the same compressive strengths as the compression piles, this conversion would not alter the gravity system. However, it is likely that this would increase the foundation cost.

Architectural Breadth Study

In order for the new shear wall layout (fig. 16) to be feasible, a few architectural changes had to be made. For the new design, SW1, SW2, SW5, and SW6 were on the same column lines as before, but SW3 and SW4 shifted to the façade of the building. For a comparison to the existing design, figure 17 shows the original shear wall locations, which are highlighted in teal. For SW3 and SW4 of the new design, which are located at the building edge, the walls are shorter than SW1 and SW2 to allow for more window area. However, the new location still eliminated one set of windows on the façade. The change was deemed as acceptable because the windows were primarily chosen for exterior aesthetics. They extended the length of the tower, but they were located in an electrical room and a janitor room near the elevator core. Therefore, they were not necessary to the apartment tenants for views from the interior. The renderings on the next page (fig. 18) give an exterior view of the building. The image on the left is the original design, and the image on the right shows the new façade without the windows at the elevator core. Based on the small size of the windows, this architectural re-design is reasonable for such a large building. For aesthetic purposes, the intent of the architect is not altered drastically with the window elimination.

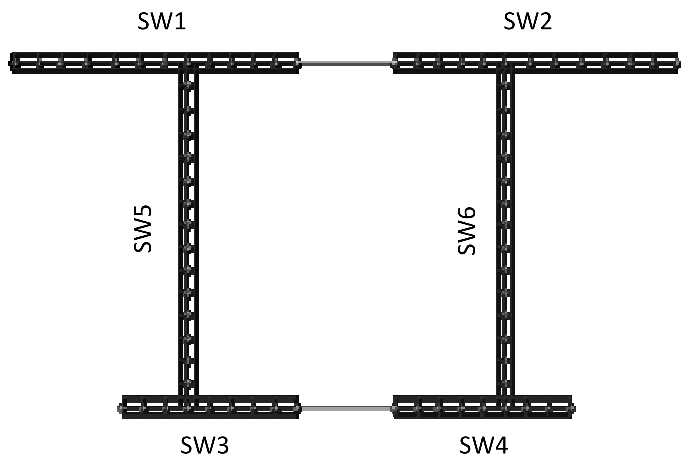

Figure 16 – Sketch of New Shear Wall Design

Figure 17 – Original Shear Wall Locations

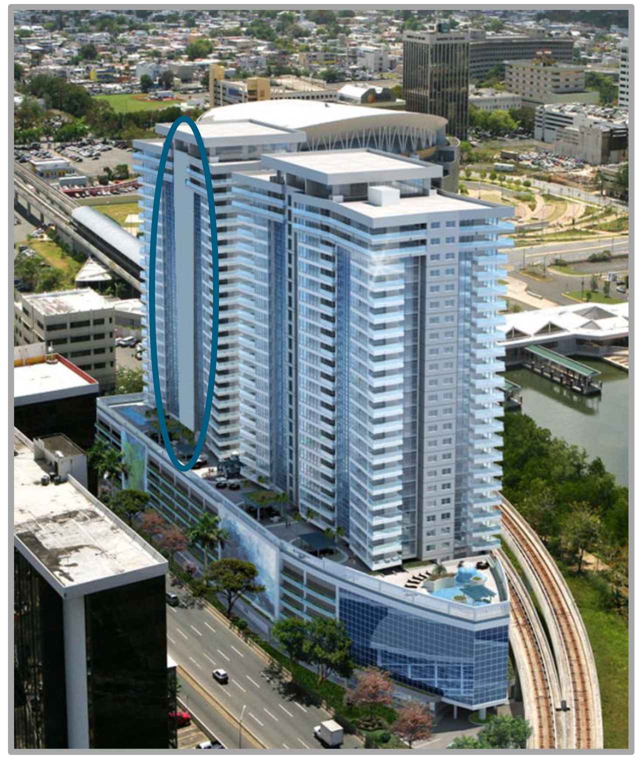

Figure 18 – Exterior Renderings of Original Building (left) and New Façade (right)

Lastly, as a comparison between the original and the new shear wall design, a study was done for the areas of the walls in plan view. At each level, the total area of the shear walls in square inches was calculated. Then, these values were converted to percentages based on the square footages of each level. The results are summarized in table 30 below. The new design reduced the area of shear walls for about 2/3 of the levels, and the floor area gained can be found in the far right column. This reduction in shear wall area allows for more floor space that can be occupied by tenants or utilized for other building functions (e.g. – mechanical or electrical rooms). If the additional floor area would be used for apartment spaces, the building owner could possibly benefit from an increase in rental rates for the tenants.

		Original Shear Wall Design		**New Shear Wall Design**		
Level	Total Floor Area (ft^2)	Plan Area of Walls (in^2)	Percentage of Floor Area (%)	Plan Area of Walls (in^2)	Percentage of Floor Area (%)	Floor Area Gained in New Design (ft^2)
sky lobby roof level	4740	30528	4.47	31104	4.56	-4.0
roof level	13490	34020	1.75	31104	1.60	20.3
29	13490	34020	1.75	31104	1.60	20.3
28	13490	34020	1.75	31104	1.60	20.3
27	11060	34020	2.14	31104	1.95	20.3
26	11060	34020	2.14	31104	1.95	20.3
25	11060	34020	2.14	31104	1.95	20.3
24	11060	34020	2.14	31104	1.95	20.3
23	11060	34020	2.14	31104	1.95	20.3
22	11060	34020	2.14	31104	1.95	20.3
21	11060	34020	2.14	31104	1.95	20.3
20	11060	34020	2.14	31104	1.95	20.3
19	11060	34020	2.14	31104	1.95	20.3
18	11060	34020	2.14	31104	1.95	20.3
17	11060	34020	2.14	31104	1.95	20.3
16	11060	34020	2.14	35460	2.23	-10.0
15	11060	34020	2.14	35460	2.23	-10.0
14	11060	34020	2.14	35460	2.23	-10.0
13	11060	34020	2.14	39024	2.45	-34.8
12	11060	34020	2.14	39024	2.45	-34.8
11	11060	34020	2.14	39024	2.45	-34.8
10	11060	34020	2.14	39024	2.45	-34.8
9	11060	34020	2.14	39024	2.45	-34.8
8	11060	34020	2.14	39024	2.45	-34.8
7	23440	63594	1.88	58032	1.72	38.6
6	23440	63594	1.88	58032	1.72	38.6
5	23440	63594	1.88	58032	1.72	38.6
4	23440	63594	1.88	58032	1.72	38.6
3	23440	63594	1.88	58032	1.72	38.6
2	23440	70110	2.08	58032	1.72	83.9

Table 30 – Comparison of Shear Wall Areas for Original and New Design

Construction Management Breadth Study

The main goal of the construction management breadth study was to compare the construction costs for the original and new shear wall designs. Because the gravity system remained the same, the focus was the lateral force resisting system. The source for the cost estimates was the 2009 edition of RS Means 'Building Construction Cost Data'. The primary expense category was the cast-in-place concrete, which included forms, reinforcing steel, concrete, placing and finishing. Due to the limitations of the book, some costs had to be extrapolated from the given data. The cost breakdown is shown below in tables 31 and 32.

		Cost Breakdown for Cast-In-Place Concrete – Original Design				
Level	Volume of Reinforced Concrete (C.Y.)	Material	Labor	Equipment	Total	Total Including O&P
		$157.00/C.Y.	$75.00/C.Y.	$7.20/C.Y.	$239.20/C.Y.	$290.00/C.Y.
sky lobby roof level	88.3	13868.33	6625.00	636.00	21129.33	25616.67
roof level	78.8	12363.75	5906.25	567.00	18837.00	22837.50
29	78.8	12363.75	5906.25	567.00	18837.00	22837.50
28	78.8	12363.75	5906.25	567.00	18837.00	22837.50
27	78.8	12363.75	5906.25	567.00	18837.00	22837.50
26	78.8	12363.75	5906.25	567.00	18837.00	22837.50
25	78.8	12363.75	5906.25	567.00	18837.00	22837.50
24	78.8	12363.75	5906.25	567.00	18837.00	22837.50
23	78.8	12363.75	5906.25	567.00	18837.00	22837.50
22	78.8	12363.75	5906.25	567.00	18837.00	22837.50
21	78.8	12363.75	5906.25	567.00	18837.00	22837.50
20	78.8	12363.75	5906.25	567.00	18837.00	22837.50
19	78.8	12363.75	5906.25	567.00	18837.00	22837.50
18	78.8	12363.75	5906.25	567.00	18837.00	22837.50
17	78.8	12363.75	5906.25	567.00	18837.00	22837.50
16	78.8	12363.75	5906.25	567.00	18837.00	22837.50
15	78.8	12363.75	5906.25	567.00	18837.00	22837.50
14	78.8	12363.75	5906.25	567.00	18837.00	22837.50
13	78.8	12363.75	5906.25	567.00	18837.00	22837.50
12	78.8	12363.75	5906.25	567.00	18837.00	22837.50
11	78.8	12363.75	5906.25	567.00	18837.00	22837.50
10	78.8	12363.75	5906.25	567.00	18837.00	22837.50
9	78.8	12363.75	5906.25	567.00	18837.00	22837.50
8	87.5	13737.50	6562.50	630.00	20930.00	25375.00
7	204.5	32099.59	15334.20	1472.08	48905.88	59292.25
6	136.3	21398.87	10222.39	981.35	32602.62	39526.58
5	136.3	21398.87	10222.39	981.35	32602.62	39526.58
4	136.3	21398.87	10222.39	981.35	32602.62	39526.58
3	136.3	21398.87	10222.39	981.35	32602.62	39526.58
2	195.3	30669.18	14650.88	1406.48	46726.54	56650.07
					Estimated Cost of Shear Walls ($) =	**827465.31**

Table 31 – Shear Wall Cost Estimate for Original Design

Level	Volume of Reinforced Concrete (C.Y.)	Cost Breakdown for Cast-In-Place Concrete - New Design				
		Material	Labor	Equipment	Total	Total Including O&P
		$157.00/C.Y.	$75.00/C.Y.	$7.20/C.Y.	$239.20/C.Y.	$290.00/C.Y.
sky lobby roof level	90.0	14130.00	6750.00	648.00	21528.00	26100.00
roof level	72.0	11304.00	5400.00	518.40	17222.40	20880.00
29	72.0	11304.00	5400.00	518.40	17222.40	20880.00
28	72.0	11304.00	5400.00	518.40	17222.40	20880.00
27	72.0	11304.00	5400.00	518.40	17222.40	20880.00
26	72.0	11304.00	5400.00	518.40	17222.40	20880.00
25	72.0	11304.00	5400.00	518.40	17222.40	20880.00
24	72.0	11304.00	5400.00	518.40	17222.40	20880.00
23	72.0	11304.00	5400.00	518.40	17222.40	20880.00
22	72.0	11304.00	5400.00	518.40	17222.40	20880.00
21	72.0	11304.00	5400.00	518.40	17222.40	20880.00
20	72.0	11304.00	5400.00	518.40	17222.40	20880.00
19	72.0	11304.00	5400.00	518.40	17222.40	20880.00
18	72.0	11304.00	5400.00	518.40	17222.40	20880.00
17	72.0	11304.00	5400.00	518.40	17222.40	20880.00
16	82.1	12887.08	6156.25	591.00	19634.33	23804.17
15	82.1	12887.08	6156.25	591.00	19634.33	23804.17
14	82.1	12887.08	6156.25	591.00	19634.33	23804.17
13	90.3	14182.33	6775.00	650.40	21607.73	26196.67
12	90.3	14182.33	6775.00	650.40	21607.73	26196.67
11	90.3	14182.33	6775.00	650.40	21607.73	26196.67
10	90.3	14182.33	6775.00	650.40	21607.73	26196.67
9	90.3	14182.33	6775.00	650.40	21607.73	26196.67
8	100.4	15758.15	7527.78	722.67	24008.59	29107.41
7	186.6	29292.13	13993.06	1343.33	44628.52	54106.48
6	124.4	19527.31	9328.33	895.52	29751.16	36069.54
5	124.4	19527.31	9328.33	895.52	29751.16	36069.54
4	124.4	19527.31	9328.33	895.52	29751.16	36069.54
3	124.4	19527.31	9328.33	895.52	29751.16	36069.54
2	161.7	25385.73	12126.94	1164.19	38676.86	46890.84

Estimated Cost of Shear Walls ($) = **795198.74**

Table 32 – Shear Wall Cost Estimate for New Design

Based on the cost analysis, the estimated cost of the shear walls decreased for the new design. The difference in cost was about $32,300, which was a 4% difference. As for the construction schedule, no information was available as a resource for the project. However, it is probable that the new shear wall design would help to tighten the schedule, especially if the shear wall erection is on the critical path. The simpler wall layout means that less formwork is necessary for the wall construction. If the shear walls are one of the critical erection tasks, then it is possible that a few days would be saved for each floor. On the other hand, if the shear walls are not on the critical path, then at the very least, the new design should not lengthen the schedule.

Conclusions and Acknowledgements

After studying Aquablue at the Golden Mile for this senior thesis project, it can be determined that the initial goals were met. A detailed lateral analysis was conducted, and some valuable design experience was gained. The concrete building code, ACI 318-08, became a much more familiar resource, and ETABS became a valuable design tool. In looking back over the course of the academic year, the project goals were accomplished.

Based on this analysis, the shear wall re-design for Aquablue at the Golden Mile was a success. The new design was feasible, and the new shear walls did not drastically alter the architecture. Even some usable floor area was gained. Also, the preliminary cost analysis showed a savings for the construction of the shear walls. In summary, this design could be an appropriate alternative to the existing lateral force resisting system.

If given more time for further study, a dynamic analysis of the structure would prove to be valuable. In fact, it was recently discovered (due to other Spring 2009 coursework) that the building has a classification of vertical irregularity type 2 due to the significant change in floor weight at the transition between the parking garage and the apartment towers (ASCE 7-05). Therefore, the equivalent lateral force procedure was not permitted for this building. Instead, a modal response spectrum analysis was required. This more detailed design approach could be the next step in studying Aquablue at the Golden Mile.

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- Family and Friends

Appendix – Shear Wall Reinforcement

The following tables summarize the shear wall reinforcement for SW2 through SW6. A description of the design process, as well as the SW1 reinforcement, can be found in the 'Shear Wall Reinforcement' section of the report.

	SEISMIC												
			Transverse (horizontal) Reinforcement					Longitudinal (vertical) Reinforcement					
Level	Seismic Shear Force (from ETABS), V_u (k)	$A_{cv}\lambda\sqrt{f'_c}$ (k)	Minimum Required ρ_t	Required $A_{s,t}$ (in^2/ft)	Preliminary Rebar and Spacing	Actual $A_{s,t}$ (in^2/ft)	Actual ρ_t	Minimum Required ρ_l	Required $A_{s,l}$ (in^2/ft)	Preliminary Rebar and Spacing	Actual $A_{s,l}$ (in^2/ft)	Actual ρ_l	Shear Capacity (k), $\phi V_n = \phi A_{cv}(\alpha_c\lambda\sqrt{f'_c} + \rho_t f_y)$
sky lobby roof level	13.1	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
roof level	27.0	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
29	61.2	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
28	93.0	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
27	116.6	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
26	138.7	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
25	159.2	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
24	178.3	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
23	196.3	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
22	213.0	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
21	228.4	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
20	242.6	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
19	255.7	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
18	268.5	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
17	279.2	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
16	274.5	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
15	296.8	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
14	315.7	430.8	0.0020	0.432	(2) #5 @ 12	0.62	0.0029	0.0012	0.259	(2) #5 @ 18	0.413	0.0019	1091.7
13	339.4	663.3	0.0020	0.576	(2) #5 @ 12	0.62	0.0022	0.0012	0.346	(2) #5 @ 18	0.413	0.0014	1370.7
12	338.1	663.3	0.0020	0.576	(2) #5 @ 12	0.62	0.0022	0.0012	0.346	(2) #5 @ 18	0.413	0.0014	1370.7
11	355.5	663.3	0.0020	0.576	(2) #5 @ 12	0.62	0.0022	0.0012	0.346	(2) #5 @ 18	0.413	0.0014	1370.7
10	392.7	663.3	0.0020	0.576	(2) #5 @ 12	0.62	0.0022	0.0012	0.346	(2) #5 @ 18	0.413	0.0014	1370.7
9	484.7	663.3	0.0020	0.576	(2) #5 @ 12	0.62	0.0022	0.0012	0.346	(2) #5 @ 18	0.413	0.0014	1370.7
8	730.8	663.3	0.0025	0.720	(2) #5 @ 9	0.83	0.0029	0.0025	0.720	(2) #5 @ 9	0.827	0.0029	1562.3
7	163.9	663.3	0.0020	0.576	(2) #5 @ 12	0.62	0.0022	0.0012	0.346	(2) #5 @ 18	0.413	0.0014	1370.7
6	322.3	663.3	0.0020	0.576	(2) #5 @ 12	0.62	0.0022	0.0012	0.346	(2) #5 @ 18	0.413	0.0014	1370.7
5	261.5	663.3	0.0020	0.576	(2) #5 @ 12	0.62	0.0022	0.0012	0.346	(2) #5 @ 18	0.413	0.0014	1370.7
4	111.0	663.3	0.0020	0.576	(2) #5 @ 12	0.62	0.0022	0.0012	0.346	(2) #5 @ 18	0.413	0.0014	1370.7
3	619.9	663.3	0.0020	0.576	(2) #5 @ 12	0.62	0.0022	0.0012	0.346	(2) #5 @ 18	0.413	0.0014	1370.7
2	162.9	553.8	0.0020	0.576	(2) #5 @ 12	0.62	0.0022	0.0012	0.346	(2) #5 @ 18	0.413	0.0014	1144.5

Table A1 – Preliminary Reinforcement for **SW2**

	WIND														
			Transverse (horizontal) Reinforcement							Longitudinal (vertical) Reinforcement					
Level	Wind Shear Force (from ETABS), Vu (k)	$V_c = 2\sqrt{f'_c}t_w d$ (k)	$V_s = A_{s,t}f_y d/s$ (k)	$\phi V_n = \phi(V_c+V_s)$ (k)	Modified Rebar and Spacing		$A_{s,t}$ (in^2/ft)	NEW $\phi V_n = \phi(V_c+V_s)$ (k)	ρ_t	Minimum Required ρ_l	Modified Rebar and Spacing		$A_{s,l}$ (in^2/ft)	ρ_l
sky lobby roof level	21.8	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
roof level	37.8	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
29	60.2	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
28	91.4	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
27	121.0	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
26	150.5	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
25	179.8	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
24	208.9	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
23	237.4	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
22	265.1	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
21	293.3	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
20	321.1	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
19	349.1	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
18	377.3	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
17	404.9	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
16	415.8	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
15	457.5	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
14	495.0	689.3	766.3	1091.7	(2) #5 @	12	0.620	1091.7	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
13	552.1	1061.3	766.3	1370.7	(2) #5 @	9	0.827	1562.3	0.0029	0.0029	(2) #5 @	8	0.930	0.0032
12	569.5	1061.3	766.3	1370.7	(2) #5 @	9	0.827	1562.3	0.0029	0.0029	(2) #5 @	8	0.930	0.0032
11	614.8	1061.3	766.3	1370.7	(2) #5 @	9	0.827	1562.3	0.0029	0.0029	(2) #5 @	8	0.930	0.0032
10	691.0	1061.3	766.3	1370.7	(2) #5 @	9	0.827	1562.3	0.0029	0.0029	(2) #5 @	8	0.930	0.0032
9	852.2	1061.3	766.3	1370.7	(2) #5 @	9	0.827	1562.3	0.0029	0.0029	(2) #5 @	8	0.930	0.0032
8	1264.0	1061.3	1021.8	1562.3	(2) #5 @	9	0.827	1562.3	0.0029	0.0029	(2) #5 @	8	0.930	0.0032
7	888.2	1061.3	766.3	1370.7	(2) #5 @	9	0.827	1562.3	0.0029	0.0029	(2) #5 @	8	0.930	0.0032
6	733.5	1061.3	766.3	1370.7	(2) #5 @	9	0.827	1562.3	0.0029	0.0029	(2) #5 @	8	0.930	0.0032
5	638.6	1061.3	766.3	1370.7	(2) #5 @	9	0.827	1562.3	0.0029	0.0029	(2) #5 @	8	0.930	0.0032
4	378.6	1061.3	766.3	1370.7	(2) #5 @	9	0.827	1562.3	0.0029	0.0029	(2) #5 @	8	0.930	0.0032
3	927.7	1061.3	766.3	1370.7	(2) #5 @	9	0.827	1562.3	0.0029	0.0029	(2) #5 @	8	0.930	0.0032
2	140.6	886.1	639.8	1144.5	(2) #5 @	9	0.827	1304.4	0.0029	0.0029	(2) #5 @	8	0.930	0.0032

Table A2 – Final Reinforcement for **SW2**

	SEISMIC															
			Transverse (horizontal) Reinforcement						Longitudinal (vertical) Reinforcement							
Level	Seismic Shear Force (from ETABS), V_u (k)	$A_{cv}\lambda\sqrt{f'_c}$ (k)	Minimum Required ρ_t	Required $A_{s,t}$ (in^2/ft)	Preliminary Rebar and Spacing		Actual $A_{s,t}$ (in^2/ft)	Actual ρ_t	Minimum Required ρ_l	Required $A_{s,l}$ (in^2/ft)	Preliminary Rebar and Spacing		Actual $A_{s,l}$ (in^2/ft)	Actual ρ_l	Shear Capacity (k), $\phi V_n = \phi A_{cv}(\alpha_c\lambda\sqrt{f'_c} + \rho_t f_y)$
roof level	19.1	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
29	19.8	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
28	22.9	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
27	25.8	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
26	28.4	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
25	30.8	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
24	33.0	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
23	35.0	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
22	36.9	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
21	38.7	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
20	40.3	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
19	41.6	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
18	42.7	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
17	41.1	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
16	54.4	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
15	49.0	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
14	36.5	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
13	30.5	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
12	40.5	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
11	44.9	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
10	44.8	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
9	39.6	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
8	41.6	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
7	327.1	309.1	0.0025	0.540	(2) #5 @	12	0.62	0.0029	0.0025	0.540	(2) #5 @	12	0.620	0.0029	728.1
6	142.9	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
5	121.3	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
4	136.2	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
3	188.2	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
2	57.2	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8

Table A3 – Preliminary Reinforcement for **SW3**

	WIND														
Level	Wind Shear Force (from ETABS), Vu (k)	$V_c = 2\sqrt{f'_c}t_wd$ (k)	Transverse (horizontal) Reinforcement							Longitudinal (vertical) Reinforcement					
			$V_s = A_{s,t}f_yd/s$ (k)	$\phi V_n = \phi(V_c+V_s)$ (k)	Modified Rebar and Spacing		$A_{s,t}$ (in^2/ft)	NEW $\phi V_n = \phi(V_c+V_s)$ (k)	ρ_t	Minimum Required ρ_l	Modified Rebar and Spacing		$A_{s,l}$ (in^2/ft)	ρ_l
roof level	62.6	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
29	66.4	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
28	74.9	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
27	81.0	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
26	87.2	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
25	93.4	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
24	99.6	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
23	105.5	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
22	111.0	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
21	116.6	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
20	122.3	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
19	129.2	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
18	139.0	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
17	155.8	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
16	122.3	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
15	138.2	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
14	146.8	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
13	120.6	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
12	142.0	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
11	157.3	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
10	177.8	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
9	232.2	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
8	330.1	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0028	(2) #5 @	12	0.620	0.0029
7	593.3	494.6	476.2	728.1	(2) #5 @	12	0.620	728.1	0.0029	0.0028	(2) #5 @	12	0.620	0.0029
6	267.5	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
5	248.3	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
4	286.0	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
3	392.7	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
2	211.7	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0028	(2) #5 @	12	0.620	0.0029

Table A4 – Final Reinforcement for **SW3**

	SEISMIC														
Level	Seismic Shear Force (from ETABS), V_u (k)	$A_{cv}\lambda\sqrt{f'_c}$ (k)	Transverse (horizontal) Reinforcement						Longitudinal (vertical) Reinforcement						Shear Capacity (k), $\phi Vn = \phi A_{cv}(\alpha_c\lambda\sqrt{f'_c} + \rho_t f_y)$
			Minimum Required ρ_t	Required $A_{s,t}$ (in^2/ft)	Preliminary Rebar and Spacing		Actual $A_{s,t}$ (in^2/ft)	Actual ρ_t	Minimum Required ρ_l	Required $A_{s,l}$ (in^2/ft)	Preliminary Rebar and Spacing		Actual $A_{s,l}$ (in^2/ft)	Actual ρ_l	
roof level	19.1	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
29	19.8	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
28	23.0	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
27	25.7	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
26	28.5	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
25	31.0	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
24	33.5	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
23	35.8	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
22	38.0	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
21	40.1	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
20	42.1	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
19	43.8	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
18	45.0	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
17	44.8	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
16	59.3	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
15	55.0	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
14	46.3	267.7	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	589.1
13	46.0	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
12	49.2	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
11	52.2	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
10	56.1	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
9	65.0	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
8	41.5	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
7	330.6	309.1	0.0025	0.540	(2) #5 @	12	0.62	0.0029	0.0025	0.540	(2) #5 @	12	0.620	0.0029	728.1
6	149.6	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
5	127.8	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
4	142.5	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
3	192.8	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8
2	55.7	309.1	0.0020	0.432	(2) #5 @	16	0.47	0.0022	0.0012	0.259	(2) #5 @	18	0.413	0.0019	638.8

Table A5 – Preliminary Reinforcement for **SW4**

	WIND													
Level	Wind Shear Force (from ETABS), Vu (k)	$V_c = 2\sqrt{f'_c}t_w d$ (k)	Transverse (horizontal) Reinforcement							Longitudinal (vertical) Reinforcement				
			$V_s = A_{s,t}f_y d/s$ (k)	$\phi V_n = \phi(V_c+V_s)$ (k)	Modified Rebar and Spacing		$A_{s,t}$ (in^2/ft)	NEW $\phi V_n = \phi(V_c+V_s)$ (k)	ρ_t	Minimum Required ρ_l	Modified Rebar and Spacing		$A_{s,l}$ (in^2/ft)	ρ_l
roof level	70.7	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
29	76.7	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
28	88.3	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
27	98.1	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
26	107.6	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
25	117.2	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
24	127.6	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
23	138.0	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
22	148.1	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
21	158.4	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
20	168.9	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
19	180.0	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
18	192.7	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
17	204.3	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
16	182.9	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
15	206.7	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
14	216.5	428.3	357.1	589.1	(2) #5 @	12	0.620	678.4	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
13	205.9	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
12	230.9	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
11	252.8	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
10	279.3	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
9	338.4	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
8	342.0	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0028	(2) #5 @	12	0.620	0.0029
7	616.3	494.6	476.2	728.1	(2) #5 @	12	0.620	728.1	0.0029	0.0028	(2) #5 @	12	0.620	0.0029
6	276.6	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
5	257.6	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
4	295.4	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
3	402.1	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0029	(2) #5 @	12	0.620	0.0029
2	237.3	494.6	357.1	638.8	(2) #5 @	12	0.620	728.1	0.0029	0.0028	(2) #5 @	12	0.620	0.0029

Table A6 – Final Reinforcement for **SW4**

Level	SEISMIC														
	Seismic Shear Force (from ETABS), V_u (k)	$A_{cv}\lambda\sqrt{f'_c}$ (k)	Transverse (horizontal) Reinforcement						Longitudinal (vertical) Reinforcement						Shear Capacity (k), $\phi V_n = \phi A_{cv}(\alpha_c\lambda\sqrt{f'_c} + \rho_t f_y)$
			Minimum Required ρ_t	Required $A_{s,t}$ (in^2/ft)	Preliminary Rebar and Spacing		Actual $A_{s,t}$ (in^2/ft)	Actual ρ_t	Minimum Required ρ_l	Required $A_{s,l}$ (in^2/ft)	Preliminary Rebar and Spacing		Actual $A_{s,l}$ (in^2/ft)	Actual ρ_l	
sky lobby roof level	21.2	320.7	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	812.6
roof level	52.7	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
29	67.3	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
28	82.3	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
27	99.9	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
26	118.8	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
25	136.2	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
24	152.3	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
23	168.8	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
22	184.8	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
21	200.1	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
20	214.5	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
19	228.0	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
18	240.2	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
17	249.4	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
16	273.2	708.3	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1558.6
15	260.9	708.3	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1558.6
14	243.6	708.3	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1558.6
13	209.1	817.9	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1690.1
12	247.7	817.9	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1690.1
11	267.2	817.9	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1690.1
10	277.6	817.9	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1690.1
9	274.7	817.9	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1690.1
8	254.6	817.9	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1690.1
7	1105.7	1022.3	0.0025	0.900	(2) #5 @	8	0.93	0.0026	0.0025	0.900	(2) #5 @	8	0.930	0.0026	2289.8
6	664.2	1022.3	0.0020	0.720	(2) #5 @	10	0.74	0.0021	0.0012	0.432	(2) #5 @	12	0.620	0.0017	2077.2
5	435.5	1022.3	0.0020	0.720	(2) #5 @	10	0.74	0.0021	0.0012	0.432	(2) #5 @	12	0.620	0.0017	2077.2
4	356.3	1022.3	0.0020	0.720	(2) #5 @	10	0.74	0.0021	0.0012	0.432	(2) #5 @	12	0.620	0.0017	2077.2
3	316.1	1022.3	0.0020	0.720	(2) #5 @	10	0.74	0.0021	0.0012	0.432	(2) #5 @	12	0.620	0.0017	2077.2
2	278.1	1022.3	0.0020	0.720	(2) #5 @	10	0.74	0.0021	0.0012	0.432	(2) #5 @	12	0.620	0.0017	2077.2

Table A7 – Preliminary Reinforcement for **SW5**

	WIND													
Level	Wind Shear Force (from ETABS), Vu (k)	$V_c = 2\sqrt{f'_c}t_wd$ (k)	Transverse (horizontal) Reinforcement							Longitudinal (vertical) Reinforcement				
			$V_s = A_{s,t}f_yd/s$ (k)	$\phi V_n = \phi(V_c+V_s)$ (k)	Modified Rebar and Spacing		$A_{s,t}$ (in^2/ft)	NEW $\phi V_n = \phi(V_c+V_s)$ (k)	ρ_t	Minimum Required ρ_l	Modified Rebar and Spacing		$A_{s,l}$ (in^2/ft)	ρ_l
sky lobby roof level	64.4	513.1	570.4	812.6	(2) #5 @	12	0.620	812.6	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
roof level	162.5	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
29	191.2	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
28	265.2	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
27	338.0	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
26	411.2	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
25	484.6	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
24	558.4	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
23	631.4	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
22	703.2	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
21	775.0	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
20	846.2	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
19	917.6	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
18	989.3	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
17	1060.1	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
16	1140.6	1133.3	944.9	1558.6	(2) #5 @	10	0.744	1700.3	0.0026	0.0026	(2) #5 @	8	0.930	0.0032
15	1198.8	1133.3	944.9	1558.6	(2) #5 @	10	0.744	1700.3	0.0026	0.0026	(2) #5 @	8	0.930	0.0032
14	1264.8	1133.3	944.9	1558.6	(2) #5 @	10	0.744	1700.3	0.0026	0.0026	(2) #5 @	8	0.930	0.0032
13	1334.5	1308.6	944.9	1690.1	(2) #5 @	10	0.744	1831.8	0.0026	0.0026	(2) #5 @	8	0.930	0.0032
12	1411.0	1308.6	944.9	1690.1	(2) #5 @	10	0.744	1831.8	0.0026	0.0026	(2) #5 @	8	0.930	0.0032
11	1475.4	1308.6	944.9	1690.1	(2) #5 @	10	0.744	1831.8	0.0026	0.0026	(2) #5 @	8	0.930	0.0032
10	1539.5	1308.6	944.9	1690.1	(2) #5 @	10	0.744	1831.8	0.0026	0.0026	(2) #5 @	8	0.930	0.0032
9	1602.5	1308.6	944.9	1690.1	(2) #5 @	10	0.744	1831.8	0.0026	0.0026	(2) #5 @	8	0.930	0.0032
8	1629.1	1308.6	944.9	1690.1	(2) #5 @	10	0.744	1831.8	0.0026	0.0026	(2) #5 @	8	0.930	0.0032
7	2421.1	1635.7	1417.3	2289.8	(2) #7 @	12	1.200	2598.4	0.0033	0.0034	(2) #5 @	6	1.240	0.0034
6	1905.0	1635.7	1133.9	2077.2	(2) #7 @	12	1.200	2598.4	0.0033	0.0034	(2) #5 @	6	1.240	0.0034
5	1536.2	1635.7	1133.9	2077.2	(2) #7 @	12	1.200	2598.4	0.0033	0.0034	(2) #5 @	6	1.240	0.0034
4	1552.5	1635.7	1133.9	2077.2	(2) #7 @	12	1.200	2598.4	0.0033	0.0034	(2) #5 @	6	1.240	0.0034
3	1586.5	1635.7	1133.9	2077.2	(2) #7 @	12	1.200	2598.4	0.0033	0.0034	(2) #5 @	6	1.240	0.0034
2	1597.3	1635.7	1133.9	2077.2	(2) #7 @	12	1.200	2598.4	0.0033	0.0034	(2) #5 @	6	1.240	0.0034

Table A8 – Final Reinforcement for **SW5**

	SEISMIC														
Level	Seismic Shear Force (from ETABS), V_u (k)	$A_{cv}\lambda\sqrt{f'_c}$ (k)	Transverse (horizontal) Reinforcement						Longitudinal (vertical) Reinforcement						Shear Capacity (k), $\phi Vn = \phi A_{cv}(\alpha_c\lambda\sqrt{f'_c} + \rho_t f_y)$
			Minimum Required ρ_t	Required $A_{s,t}$ (in^2/ft)	Preliminary Rebar and Spacing		Actual $A_{s,t}$ (in^2/ft)	Actual ρ_t	Minimum Required ρ_l	Required $A_{s,l}$ (in^2/ft)	Preliminary Rebar and Spacing		Actual $A_{s,l}$ (in^2/ft)	Actual ρ_l	
sky lobby roof level	21.2	320.7	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	812.6
roof level	36.4	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
29	94.6	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
28	148.5	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
27	191.4	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
26	232.9	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
25	271.8	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
24	308.3	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
23	342.5	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
22	374.4	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
21	404.1	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
20	431.7	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
19	456.9	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
18	479.8	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
17	498.6	531.2	0.0020	0.432	(2) #5 @	12	0.62	0.0029	0.0012	0.259	(2) #5 @	18	0.413	0.0019	1346.1
16	535.8	708.3	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1558.6
15	541.8	708.3	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1558.6
14	552.2	708.3	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1558.6
13	568.2	817.9	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1690.1
12	584.9	817.9	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1690.1
11	595.9	817.9	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1690.1
10	604.0	817.9	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1690.1
9	606.0	817.9	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1690.1
8	578.3	817.9	0.0020	0.576	(2) #5 @	12	0.62	0.0022	0.0012	0.346	(2) #5 @	18	0.413	0.0014	1690.1
7	1446.7	1022.3	0.0025	0.900	(2) #5 @	8	0.93	0.0026	0.0025	0.900	(2) #5 @	8	0.930	0.0026	2289.8
6	851.8	1022.3	0.0020	0.720	(2) #5 @	10	0.74	0.0021	0.0012	0.432	(2) #5 @	12	0.620	0.0017	2077.2
5	547.6	1022.3	0.0020	0.720	(2) #5 @	10	0.74	0.0021	0.0012	0.432	(2) #5 @	12	0.620	0.0017	2077.2
4	319.5	1022.3	0.0020	0.720	(2) #5 @	10	0.74	0.0021	0.0012	0.432	(2) #5 @	12	0.620	0.0017	2077.2
3	240.9	1022.3	0.0020	0.720	(2) #5 @	10	0.74	0.0021	0.0012	0.432	(2) #5 @	12	0.620	0.0017	2077.2
2	328.6	1022.3	0.0020	0.720	(2) #5 @	10	0.74	0.0021	0.0012	0.432	(2) #5 @	12	0.620	0.0017	2077.2

Table A9 – Preliminary Reinforcement for **SW6**

	WIND													
Level	Wind Shear Force (from ETABS), Vu (k)	$V_c = 2\sqrt{f'_c}t_w d$ (k)	Transverse (horizontal) Reinforcement							Longitudinal (vertical) Reinforcement				
			$V_s = A_{s,t}f_y d/s$ (k)	$\phi V_n = \phi(V_c+V_s)$ (k)	Modified Rebar and Spacing		$A_{s,t}$ (in^2/ft)	NEW $\phi V_n = \phi(V_c+V_s)$ (k)	ρ_t	Minimum Required ρ_l	Modified Rebar and Spacing		$A_{s,l}$ (in^2/ft)	ρ_l
sky lobby roof level	41.0	513.1	570.4	812.6	(2) #5 @	12	0.620	812.6	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
roof level	121.2	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
29	197.5	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
28	293.7	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
27	420.5	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
26	546.7	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
25	673.4	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
24	804.5	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
23	934.9	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
22	1063.8	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
21	1192.9	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
20	1321.0	849.9	944.9	1346.1	(2) #5 @	12	0.620	1346.1	0.0029	0.0029	(2) #5 @	10	0.744	0.0034
19	1447.9	849.9	944.9	1346.1	(2) #5 @	8	0.930	1700.5	0.0043	0.0045	(2) #5 @	6	1.240	0.0057
18	1570.1	849.9	944.9	1346.1	(2) #5 @	8	0.930	1700.5	0.0043	0.0045	(2) #5 @	6	1.240	0.0057
17	1678.1	849.9	944.9	1346.1	(2) #5 @	8	0.930	1700.5	0.0043	0.0045	(2) #5 @	6	1.240	0.0057
16	1886.1	1133.3	944.9	1558.6	(2) #5 @	8	0.930	1912.9	0.0032	0.0033	(2) #5 @	6	1.240	0.0043
15	1962.9	1133.3	944.9	1558.6	(2) #7 @	8	1.800	2907.3	0.0063	0.0067	(2) #7 @	6	2.400	0.0083
14	2059.5	1133.3	944.9	1558.6	(2) #7 @	8	1.800	2907.3	0.0063	0.0067	(2) #7 @	6	2.400	0.0083
13	2190.4	1308.6	944.9	1690.1	(2) #7 @	8	1.800	3038.8	0.0063	0.0067	(2) #7 @	6	2.400	0.0083
12	2318.9	1308.6	944.9	1690.1	(2) #7 @	8	1.800	3038.8	0.0063	0.0067	(2) #7 @	6	2.400	0.0083
11	2436.7	1308.6	944.9	1690.1	(2) #7 @	8	1.800	3038.8	0.0063	0.0067	(2) #7 @	6	2.400	0.0083
10	2537.5	1308.6	944.9	1690.1	(2) #7 @	8	1.800	3038.8	0.0063	0.0067	(2) #7 @	6	2.400	0.0083
9	2595.2	1308.6	944.9	1690.1	(2) #7 @	8	1.800	3038.8	0.0063	0.0067	(2) #7 @	6	2.400	0.0083
8	2440.4	1308.6	944.9	1690.1	(2) #7 @	8	1.800	3038.8	0.0063	0.0066	(2) #7 @	6	2.400	0.0083
7	3124.1	1635.7	1417.3	2289.8	(2) #7 @	8	1.800	3284.2	0.0050	0.0051	(2) #7 @	6	2.400	0.0067
6	1363.1	1635.7	1133.9	2077.2	(2) #5 @	8	0.930	2289.8	0.0026	0.0026	(2) #5 @	6	1.240	0.0034
5	1342.8	1635.7	1133.9	2077.2	(2) #5 @	8	0.930	2289.8	0.0026	0.0026	(2) #5 @	6	1.240	0.0034
4	1285.4	1635.7	1133.9	2077.2	(2) #5 @	8	0.930	2289.8	0.0026	0.0026	(2) #5 @	6	1.240	0.0034
3	1420.9	1635.7	1133.9	2077.2	(2) #5 @	8	1.800	3284.2	0.0050	0.0053	(2) #5 @	6	2.400	0.0067
2	1578.8	1635.7	1133.9	2077.2	(2) #5 @	8	1.800	3284.2	0.0050	0.0052	(2) #5 @	6	2.400	0.0067

Table A10 – Final Reinforcement for **SW6**