LETTER OF TRANSMITTAL

DATE: November 20, 2013

TO: Dr. Linda Hanagan

FROM: Alyssa Stangl

ENCLOSED: AE 481W – Senior Thesis | Structural Technical Report 4

Dear Dr. Hanagan,

This report was prepared for Technical Report 4 for AE 481W – Senior Thesis. This report includes a complete lateral analysis of La Jolla Commons Phase II Office Tower. Items included in this report are as follows:

- General Building Information
- ETABS Model Information and Verification
- Lateral Load Cases Used
- Drift Analysis
- Strength Analysis
- Overturning and Impact on Foundations
- Tech 2 Load Calculations are included in Appendix A for reference.

Thank you for your time reviewing this report. I look forward to discussing it with you in the near future.

Sincerely,

Alyssa Michelle Stangl

November 20, 2013

Technical Report 4

La Jolla Commons Phase II Office Tower

San Diego, California

Alyssa Stangl | Structural Option | Advisor: Dr. Linda Hanagan

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Executive Summary

La Jolla Commons Phase II Office Tower is a 13 story office building in San Diego, California. Each floor is about 40,320 square feet, and the structure reaches 198 feet from ground level to the top of the penthouse. With two levels of underground parking, the building extends about 20 feet below grade. Serving as an office building for LPL Financial, the building has open floor plans and large areas of glass curtain wall. La Jolla Commons Tower II received a LEED-CS Gold Certification and is the nation's largest and most advanced net-zero office building.

The building's gravity system begins with a mat foundation, two stories below grade. The mat foundation was chosen for its constructability, when compared to a system of footings and grade beams. The super structure consists of two-way, flat plate, concrete slabs on a rectangular column grid. A typical bay is 30 feet by 40 feet. Each level varies in thickness, ranging from 12 to 18 inches with reinforcing, as required, by code. Camber was used for the slab at each level, excluding Lower Level 2 where the mat foundation serves as the floor. The designers determined that the large construction loads would cause the slab to crack; therefore, slab deflections were calculated for a cracked slab section. As a result, the deflections calculated for post-construction loading were significant. The maximum camber applied to the slab is 2 ¼" at the center of a bay.

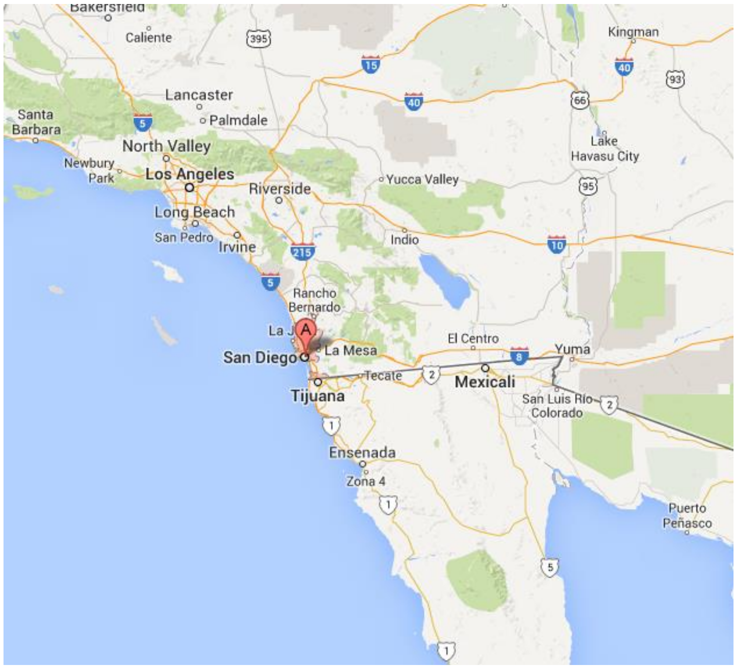
Laid out at the core of the building, the lateral system of La Jolla Commons Tower II consists of reinforced concrete shear walls. Due to the high shear forces associated with earthquake loading in this Seismic Category D structure, the diaphragm alone is not relied upon to transfer lateral loads to the shear wall system; instead, collector beams are used to aid in the transfer of lateral loads at levels below grade in the north-south direction.

La Jolla Commons Tower II has two unique structural and architectural features. The north and south sides of the building feature 15 foot cantilevers that start at Level 3 and continue up to the roof level. The structure of each cantilever is similar to that of the rest of slab; though, it does have additional reinforcement. Also, the building has a plaza area on the Ground Level which carves out a portion of Ground Level 1 and Level 2. Main building columns are exposed here, and additional 18 inch columns are added to support the slab edge above.

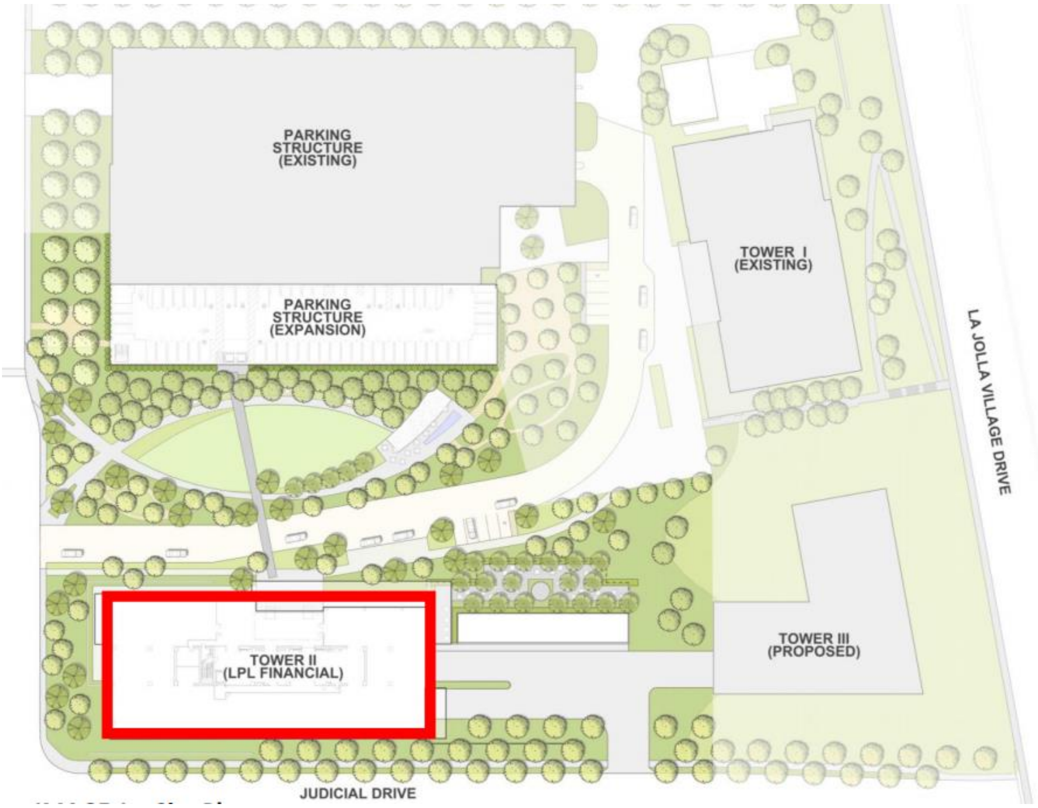
La Jolla Commons Tower II was designed using the 2010 California Building Code which corresponds to ASCE 7-05 and ACI 318-08. CBC 2010 and ASCE 7-05 were used to calculate live, wind, and earthquake loads. ACI 318 – 08, Chapter 21, references the design of concrete Earthquake-Resistant structures, and ASCE 7-05, Chapter 12, details the Seismic Design Requirements for Building Structures. Both of these documents were used heavily in the design of LJC II in order to account for seismic loading and detailing.

La Jolla Commons Phase II Office Tower is full of educational value. It has several structural challenges and unique conditions: punching shear, seismic loading and detailing, concrete shear wall design, and computer modeling. The following report explains the building structure, design codes, and design loads in more detail.

Building Site Information



San Diego California *(Google Maps)*



Building Site Plan *(Courtesy of Hines)*

La Jolla Commons Phase II Office Tower

San Diego , California | LPL Financial Office Tower

Primary Project Team

Owner | Hines
Tenant | LPL Financial
Architect | AECOM
Structural Engineer | Nabih Youssef Associates
MEP Engineer | WSP Flack + Kurtz
Civil Engineer | Leppert Engineering

General Building Data

Construction Dates | April 2012 – May 2014
Building Cost | $78,000,000
Delivery Method | Design-Bid-Build
Height| 198' – 8" | 13 Stories
2 Levels | Underground Parking
Size | 462,301 GSF

Architecture

- Modern style building with glass curtain wall
- 12 foot floor-to-floor height
- Very open and spacious office area
- Interior features and build out by tenant

Sustainability Features

- First Class A, NetZero Office Building in the USA
- Building returns more energy to the grid than it uses on an annual basis
- LEED – CS Gold Certification

Structural

- Two-way, flat plate , reinforced concrete slab
- Concrete columns on a regular column grid
- Special reinforced concrete shear walls
- Mat foundation system

Mechanical

- Chilled Water, floor-by – floor VAV Dual Path Air Handling Units
- Ventilation and cooling through underfloor air distribution, overhead air to perimeter zones.

Lighting and Electrical

- High efficiency, low glare lighting fixtures
- High power factor electronic ballasts
- Lighting control system integrated with Building Management System, local override at each floor
- Two 400 Amp, 480/277V, 3-phase, 4 wire switchboards service building
- One services the lower level bus riser and the other services the upper level bus riser
- One diesel fuel standby engine generator.

Alyssa Stangl [Structural Option] http://www.engr.psu.edu/ae/thesis/portfolios/2014/ams6158

Documents Used to Create This Report

- *American Concrete Institute*
  - ACI 318 – 11
- *International Building Code*
  - IBC 2012
- *American Society of Civil Engineers*
  - ASCE 7 – Minimum Design Loads for Buildings
- *La Jolla Commons Phase II Office Tower*
  - Construction Documents
  - Technical Specifications

Introduction

The following technical report consists of a lateral system analysis study of La Jolla Commons Phase II Office Tower. The building's lateral system was modeled using ETABS 2013 and analyzed for performance under wind and seismic loading. Checks for strength, drift, story drift, overturning and impact on foundations were performed and controlling load combinations were established. Furthermore, overall building torsion issues were investigated.

The Figures 1 through 3 show a typical floor plan, building elevation, and typical shear wall elevations. Shear walls have been highlighted on the typical floor plan for clarity.

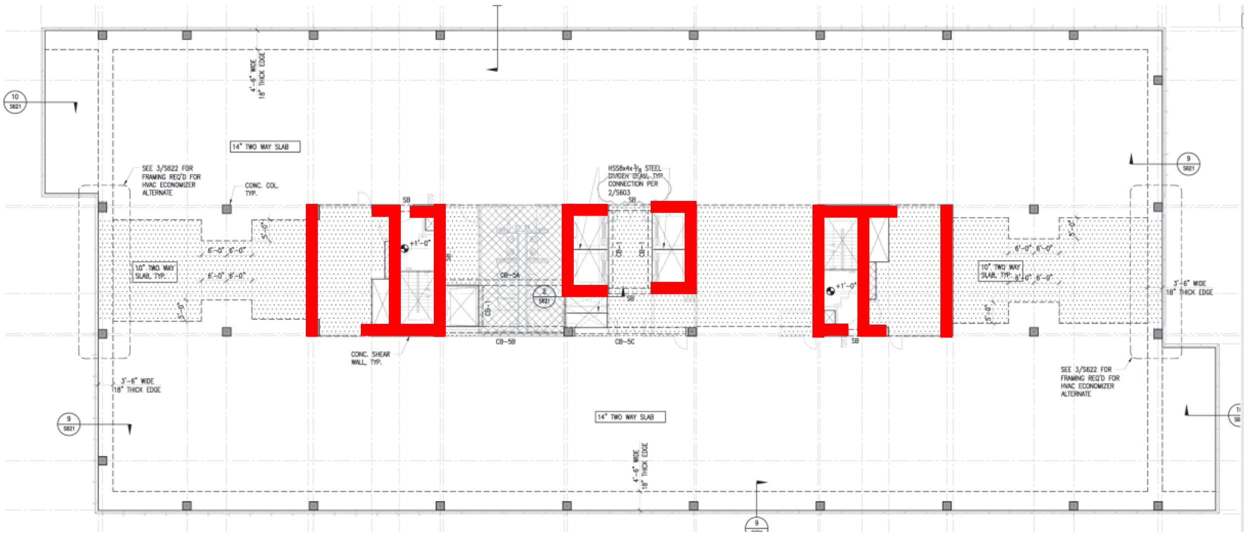


Figure 1 | Typical Shear Wall Layout – S109

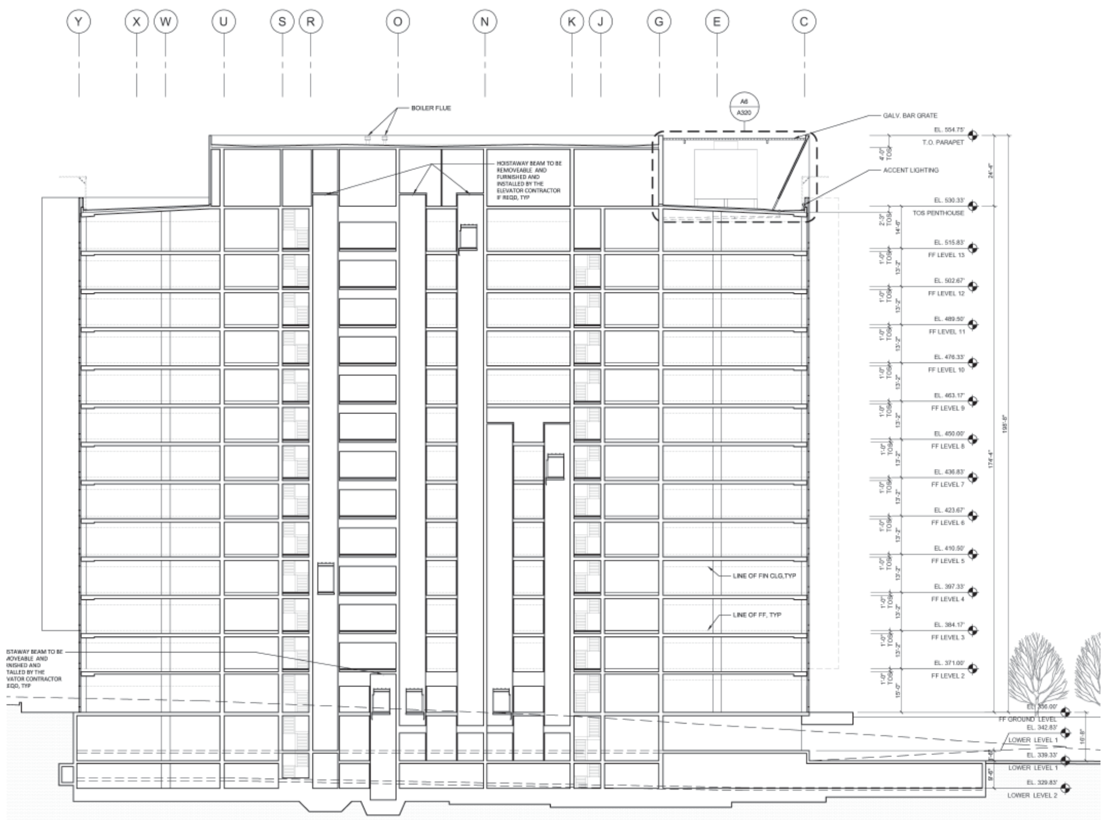


Figure 2 | Typical Shear Wall Layout – S109

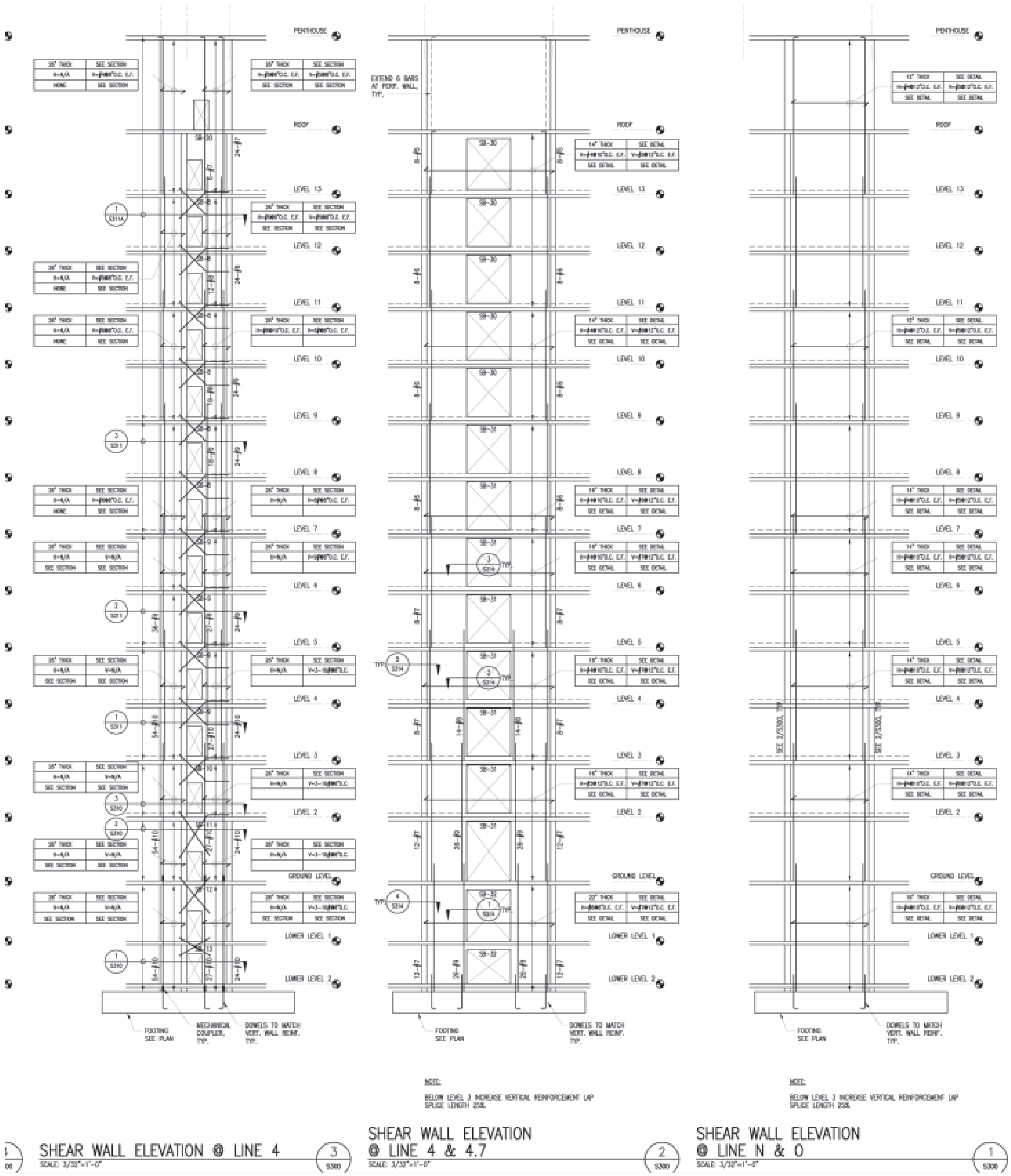


Figure 3 | Typical Shear Wall Elevations – S300

ETABS Model Assumptions and Notes

Modeling Assumptions and Discussion – ETABS 2013:

ETABS 2013 was used to model the lateral system of La Jolla Commons Phase II Office Tower. Several assumptions and decisions were made in the modeling process. These are outlined below. Figures 4 and 5 show two views of the ETABS model.

Items Included or Not Included in Lateral Model:

- All reinforced concrete shear walls were included in the model. Foundation walls from the base to ground level were included in the model. Differences in shear wall thicknesses, materials, and openings were accounted for.
- Edge beams were *not* used in this lateral model. After investigating the reinforcing and sizes of the edge beams, it was determined that the size and reinforcement did not change on any floors. As a result, the beams are most likely not intended to resist lateral forces.
- Beams called out as CB – Concrete Beams were *not* included in the lateral model. These beams seemed to be sized and reinforced for gravity loading only and are not intended to behave laterally.
- Beams called out as SB – Spandrel Beams were included in the lateral model. These beams were used to connect shear walls. They had very deep cross sections, closely spaced shear reinforcing, and some had diagonal reinforcing that was embedded in the concrete shear wall.
- Columns were not included in this lateral model. After discussions with the structural engineer, it was determined that the columns were not intended to behave laterally.

Important Model Information:

- All concrete elements were modeled as cracked sections per ACI 318. The wall stiffness was decreased by 65%.
- Mesh Size Used – 2 ft x 2 ft
- Coupling beams modeled as line elements with insertion points at the top of the beam.
- Rigid Diaphragm – Chosen because of the significantly thick, concrete flat plate slab.
- Base Constraints – All fixed connections were used. The large mat foundation, ranging from 4.5 feet to 6 feet in thickness, will effectively restrain the shear walls against moment and shear, creating a fixed condition.

Modeling Challenges:

The model used for this Technical Assignment was rather challenging to construct. When the shear walls were connected to one another, the computer output did not appear to be right and could not be verified with hand calculations. As a result, the shear walls were disconnected to produce verifiable results. This helps to create more accurate shear forces, but drift and displacement output is much more than what the building would truly experience. Further investigations will be done in order to accurately predict displacement behavior in the future.

ETABS Model Screen Shots:

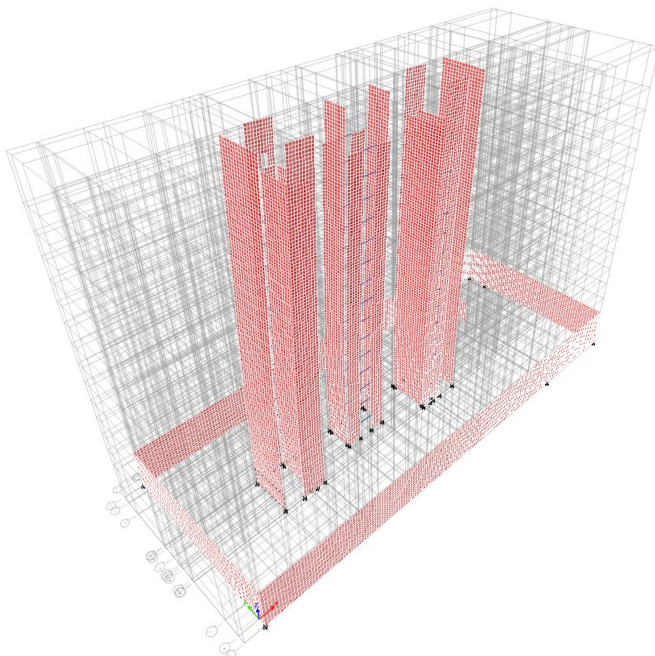


Figure 4 | Deformed Lateral System Model for LJC II *(ETABS GENERATED)*

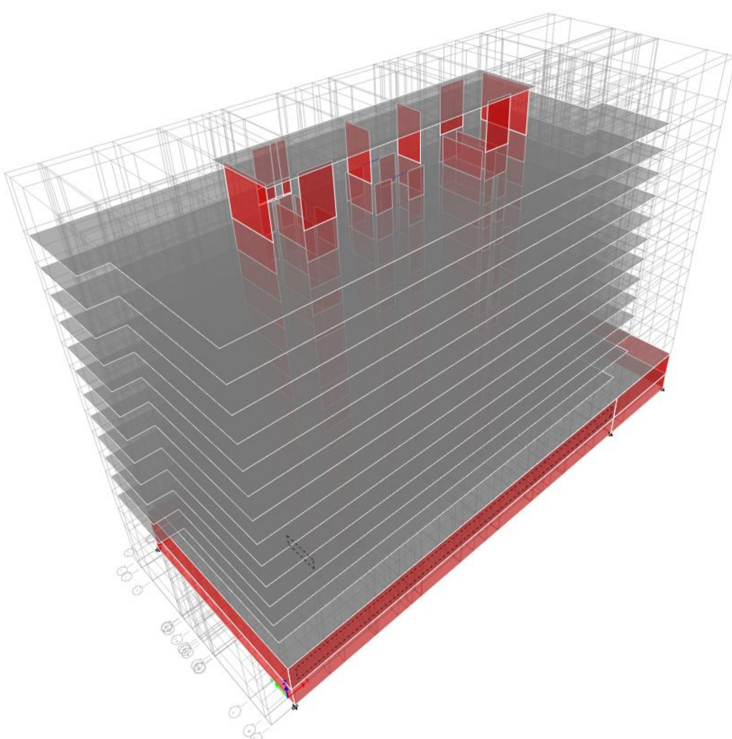


Figure 5 | Undeformed Lateral System Model for LJC II *(ETABS GENERATED)*

ETABS Model Verification

Force Distribution Diagrams and Discussion:

For diagrams were developed for Level 3 for earthquake loads in the X-direction and earthquake loads in the Y-direction. These can be viewed in Figures 6 and 7. This was done to verify that ETABS was properly distributing the direct and torsional forces to the lateral elements.

The forces seem to be distributed proportionally based on wall stiffness for direct shears. Also, the forces are distributed torsionally as expected. This is true for loads in both the X and Y directions.

At the top margin of these diagrams, forces in the X and Y directions were summed. For the EX forces diagram, the X-forces sum to 7643.49 kips which is very close to the story shear of 7646.72. Also, forces in the Y-direction sum to zero as expected.

For the EY forces diagram, the X-forces sum to zero as expected. The forces in the Y-direction sum to 7646.72 kip, which exactly matches the story shear of 7646.72 kips.

These results further verify the validity of the ETABS model.

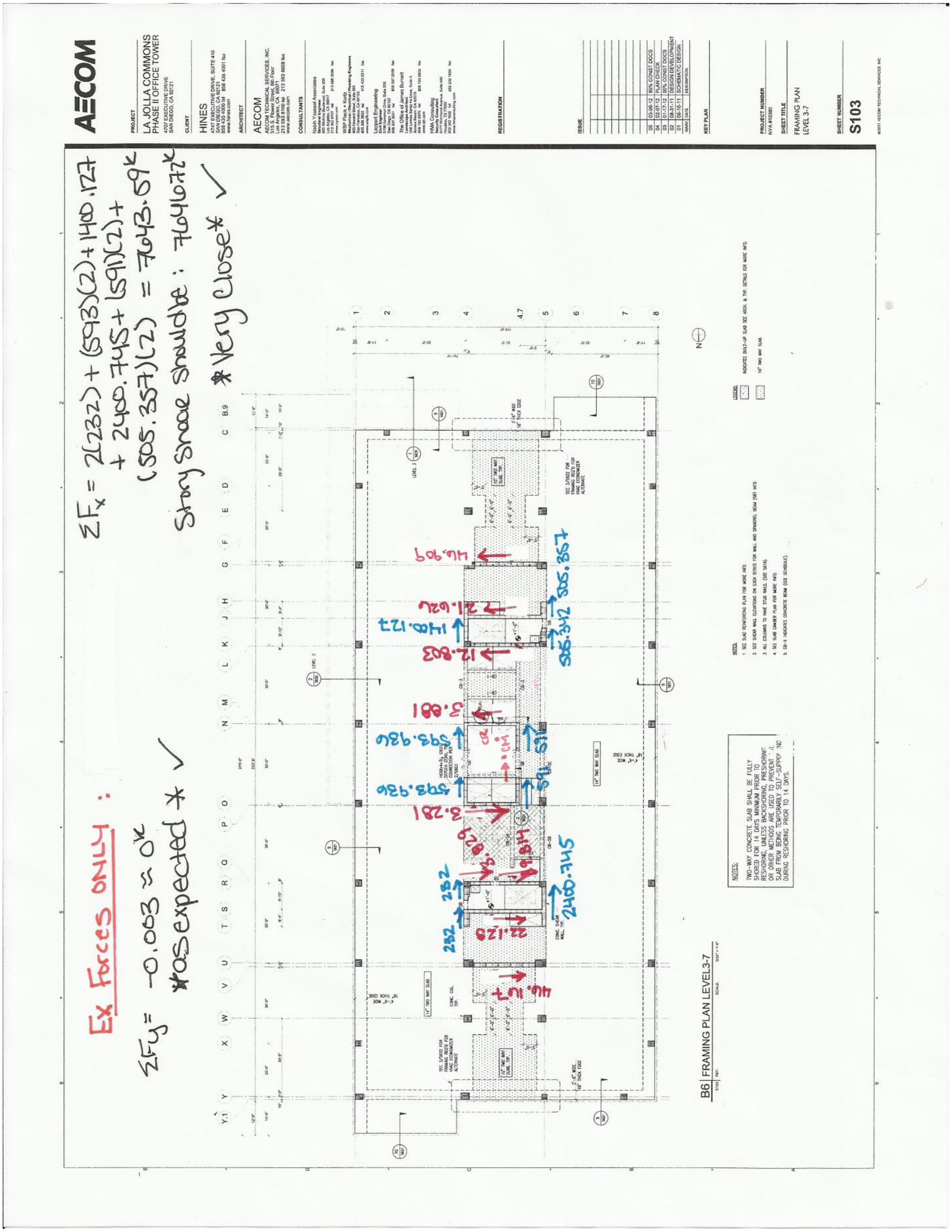


Figure 6 | Ex Force Distribution from ETABS Output

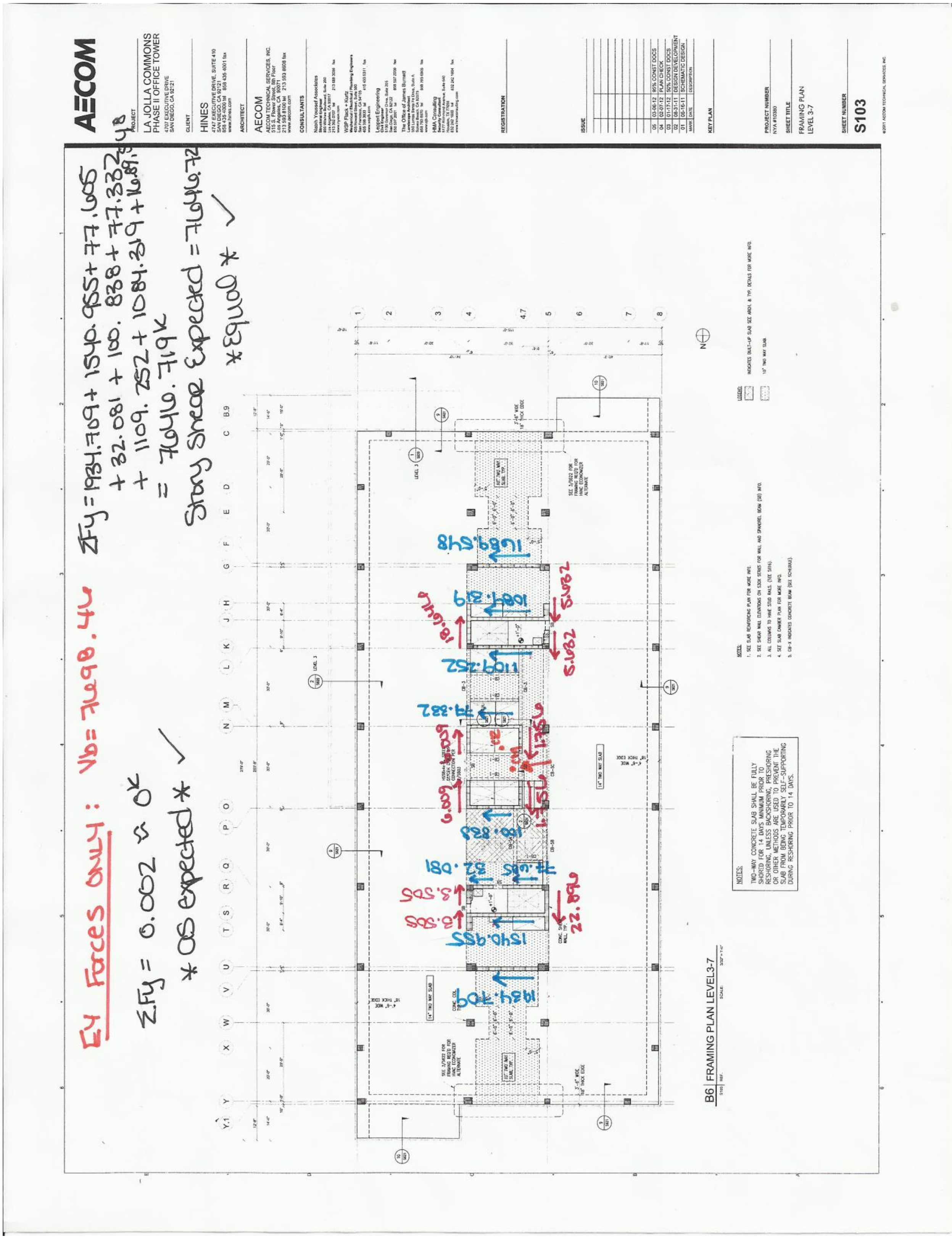


Figure 7 | Ey Force Distribution from ETABS Output

2D Analysis, COR, and COM Verification:

A 2D analysis was performed on Level 3 of LJC II under a load of 1000 kips applied at the center of mass. The results of the 2D analysis by hand can be seen in the following section. The total shears in each wall were compared to the total shears given by ETABS under the same loading.

At first glance the results seem to be pretty different; however, most forces produced by ETABS are proportionally increased or decreased. This could indicate a difference in the calculation of stiffness for the shear walls. The 2D analysis makes more assumptions about the stiffness of the shear walls than the computer model does. Therefore, although the results are different, the distribution remains pretty consistent. Therefore, these results help to verify the validity of the etabs model.

The center of rigidity and center of mass was calculated by hand and compared to the results of the ETABS model for Level 3 of the building. The results were very close, further verifying the model. See the pages to follow for calculations.

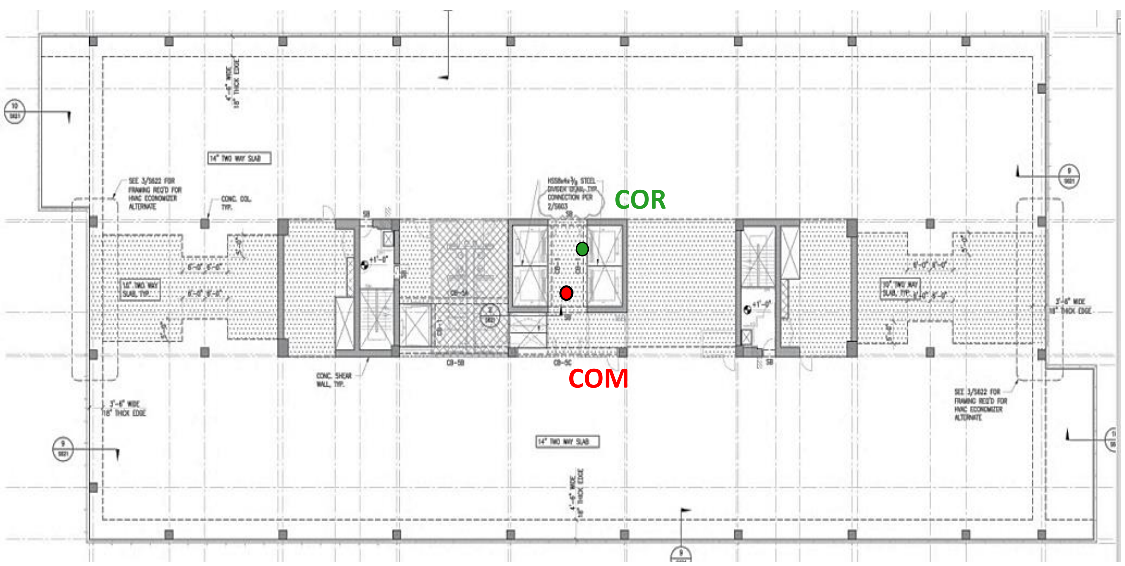


Figure 8 | Center of Mass and Center of Rigidity General Locations

Center of Mass Verification:

Levels 3-7 Center of Mass									
Item	Thickness	Length	Height	Area	Weight	X-Location (from NW corner)	Y-Location (from NW corner)	Weight * X-Location	Weight * Y-Location
10" Slab	10	253.667	83.334	21139	4227817	126.833	57.5	536226733	243099486
14" Slab	14	253.667	30	7610.01	1522002	126.833	57.5	193040080	87515115
SW from 4 to 5, line U	18	30	13.17	395	88898	50	57.5	4444875	5111606
SW from 4 to 5, Line S	16	30	13.17	395.1	79020	63.833	57.5	5044084	4543650
SW from 4 to 5, Line R	16	26.5	13.17	349	69801	80	57.5	5584080	4013558
SW from 4 to 4.7, Line O	14	20.5	13.17	269.985	47247.3	110	62	5197211	2929337
SW from 4 to 4.7, Line N	14	20.5	13.17	270	47247	140	62	6614633	2929337
SW from 4 to 5, Line K	16	30	13.17	395.1	79020	170	57.5	13433400	4543650
SW from 4 to 4.7, Line J	16	30	13.17	395	79020	179.833	57.5	14210404	4543650
SW from 4 to 4.7, Line G	18	30	13.17	395.1	88897.5	200	57.5	17779500	5111606
SW from T to R, Line 4	26	12.667	13.17	167	54218	72.333	71.667	3921745	3885636
SW from T to R, Line 5	18	16.167	13.17	212.919 39	47906.86 3	72.333	41.667	3465247	1996135
SW from O to N, Line 4	16	21	13.17	277	55314	125	71.667	6914250	3964188
SW from O to N, Line 4.7	16	21	13.17	276.57	55314	125	41.667	6914250	2304768
SW from K to H, Line 4	18	16.167	13.17	213	47907	178.5	71.667	8551375	3433341
SW from K to H, Line 5	26	12.667	13.17	166.824 39	54217.92 7	178.5	41.667	9677900	2259098
Opening 1	10	-	-	63	-12600	67.5	48.5	-850500	-611100
Opening 2	10	-	-	124	-24800	75.5	50	-1872400	-1240000
Opening 3	10	-	-	79	-15800	86	48.5	-1358800	-766300
Opening 4	10	-	-	169	-33800	117	62	-3954600	-2095600
Opening 5	10	-	-	169	-33800	117	47	-3954600	-1588600

Opening 6	10	-	-	169	-33800	136	62	-4596800	-2095600
Opening 7	10	-	-	169	-33800	146.5	62	-4951700	-2095600
Opening 8	10	-	-	169	-33800	166.5	62	-5627700	-2095600
Opening 9	10	-	-	124	-24800	178	64	-4414400	-1587200
Opening 10	10	-	-	63	-12600	185	64	-2331000	-806400
					6384247			807107266	367202164

XCM =	126.421675 9
YCM=	57.5169062

Comparison to ETABS mass output:

	ETABS	By Hand	% Difference
XCM	125.8661	126.422	0.440%
YCM	56.6799	57.517	1.466%

Note: The percent difference here is very small. However, looking at the loads used in ETABS compared to my hand calculations, ETABS is including beam weights as well as more accurate dimensions. Therefore, ETABS will be used, and these hand calculations verify that the method of modeling the masses is appropriate.

Center of Rigidity Verification:

Levels 3-7 Center of Rigidity

X-Direction

Item	Thickness, t	Length, b	Height, h	Cross Sectional Area (in^2)	Moment of Inertia (in^4)	E (ksi)	G (ksi)	K of wall	Xi	Ki * Xi
SW from 4 to 5, line U	18	360	158.04	6480	69984000	4415.2	1766.1	3171	50	158569
SW from 4 to 5, Line S	16	360	158.04	69120	62208000	4415.2	1766.1	3150	64	201081
SW from 4 to 5, Line R	16	318	158.04	61056	42876576	4415.2	1766.1	2736	80	218887
SW from 4 to 4.7, Line O	14	246	158.04	41328	17368092	4415.2	1766.1	1980	110	217756
SW from 4 to 4.7, Line N	14	246	158.04	41328	17368092	4415.2	1766.1	1980	140	277144
SW from 4 to 5, Line K	16	360	158.04	69120	62208000	4415.2	1766.1	3150	170	535518
SW from 4 to 5, Line J	16	360	158.04	69120	62208000	4415.2	1766.1	3150	180	566493
SW from 4 to 5, Line G	18	360	158.04	77760	69984000	4415.2	1766.1	3171	200	634275
								22488		2809722

Y Direction

Item	Thickness, t	Length, b	Height, h	Cross Sectional Area (in^2)	Moment of Inertia (in^4)	E (ksi)	G (ksi)	K of wall	Yi	Ki * Yi
SW from T to R, Line 4	26	152	158.04	3952	7609518	4415.2	1766.1	1159	71.667	83034
SW from T to R, Line 5	18	194.004	158.04	41904.864	10952753	4415.2	1766.1	1510	42	62908
SW from O to N, Line 4	16	252	158.04	48384	21337344	4415.2	1766.1	2075	71.667	148689
SW from O to N, Line 4.7	16	252	158.04	48384	21337344	4415.2	1766.1	2075	42	86448
SW from K to H, Line 4	18	194	158.04	41905	10952753	4415.2	1766.1	1510	71.667	108202
SW from K to H, Line 5	26	152.004	158.04	47425.248	7609518	4415.2	1766.1	1159	42	48276
								9486		537557

Ycr=	56.67
Xcr=	124.94

Comparison to ETABS mass output:

	ETABS	By Hand	% Difference
Xcr	131.912	124.941	5.428%
Ycr	56.9114	56.667	0.430%

Note: The percent difference here is very small. Therefore, the center of mass in ETABS is verified.

2D Distribution of X1000 Forces by Hand:

Done with ***1000 kip*** load applied in the **X direction** at the **center of mass**

Direct Shear				
Wall	**Stiffness**	**Relative Stiffness**	**Total Shear**	**Direct Shear in Wall**
SW from R.1 to R, Line 4	580	0.073934038	1000	73.93
SW from T to S, Line 4	580	0.073934038	1000	73.93
SW from T to R, Line 5	1510	0.19262195	1000	192.62
SW from O to O.1, Line 4	770	0.098238497	1000	98.24
SW from O.2 to N, Line 4	770	0.098238497	1000	98.24
SW from O to O.1, Line 4.7	770	0.098238497	1000	98.24
SW from O.2 to N, Line 4.7	770	0.098238497	1000	98.24
SW from K to H, Line 4	1510	0.19262195	1000	192.62
SW from K to K.1, Line 5	580	0.073934038	1000	73.93
SW from K to H, Line 5	580	0.073934038	1000	73.93
	7838			

Torsional Shear							
Wall	Rigidity, R	Distance from CR to Wall, d	R*d	d^2	$R*d^2$	Total Moment, Ve	Torsional Shear in Wall
SW from 4 to 5, line U	3171	75.72	240108.12	5733.52	18180987	7770	25.92
SW from 4 to 4.7, Line O	1980	13.887	27496.26	192.85	381841	7770	2.97
SW from 4 to 4.7, Line N	1980	16.113	31903.74	259.63	514065	7770	3.44
SW from 4 to 5, Line S	3150	57.65	181603.54	3323.52	10469444	7770	19.61
SW from 5 - 4.7, Line R	1368	46.93	64202.24	2202.42	3013011	7770	6.93
SW from 4.1 - 4, Line R	1368	46.93	64202.24	2202.42	3013011	7770	6.93
SW from 4 to 5, Line J	3150	57.65	181603.54	3323.52	10469444	7770	19.61
SW from 4 to 5, Line K	3150	46.93	147834.41	2202.42	6937869	7770	15.96
SW from 4 to 5, Line G	3171	74.08	234907.68	5487.85	17401961	7770	25.36
SW from T to S, Line 4	375	7.217	2706.38	52.09	19532	7770	0.29
SW from R.1 to R, Line 4	375	7.217	2706.38	52.09	19532	7770	0.29
SW from T to R, Line 5	1510	22.783	34397.41	519.07	783676	7770	3.71
SW from O to N, Line 4	2075	7.217	14973.30	52.09	108062	7770	1.62
SW from O to N, Line 4.7	2075	13.254	27498.42	175.67	364464	7770	2.97
SW from K to H, Line 4	1510	7.217	10896.11	52.09	78637	7770	1.18
SW from K to K.1, Line 5	368	22.783	8387.70	519.07	191097	7770	0.91
SW from J to H, Line 5	375	7.217	2706.94	52.09	19536	7770	0.29
				J=	71966169		

Total Shears				
Wall	**Direct Shear**	**Torsional Shear**	**By Hand Total Shear**	**ETABS TOTAL SHEARS**
SW from 4 to 5, line U	0	25.92	25.924	15.343
SW from 4 to 4.7, Line O	0	2.97	2.969	0.842
SW from 4 to 4.7, Line N	0	3.44	3.445	0.76
SW from 4 to 5, Line S	0	19.61	19.607	7.735
SW from 5 - 4.2, Line R	0	6.93	6.932	2.106
SW from 4.1 - 4, Line R	0	6.93	6.932	0.827
SW from 4 to 5, Line J	0	19.61	19.607	7.151
SW from 4 to 5, Line K	0	15.96	15.961	4.621
SW from 4 to 5, Line G	0	25.36	25.362	14.322
SW from T to S, Line 4	73.9	0.29	73.642	3.078
SW from R.1 to R, Line 4	73.9	0.29	73.642	3.078
SW from T to R, Line 5	192.6	3.71	196.336	401.449
SW from O to O.1, Line 4	98.2	0.81	97.430	72.94
SW from O.2 to N, Line 4	98.2	0.81	97.430	72.94
SW from O to O.1, Line 4.7	98.2	1.48	99.723	72.415
SW from O.2 to N, Line 4.7	98.2	1.48	99.723	72.415
SW from K to H, Line 4	192.6	1.18	191.446	195.017
SW from K to K.1, Line 5	73.9	0.91	74.840	53.332
SW from J to H, Line 5	73.9	0.29	74.226	53.336

Shear and Moment Diagrams and Discussion:

The following shear and moment diagrams in Figures 9 and 10 are of a typical shear wall taking direct shear from a 1000 kip load applied at the center of mass of the diaphragm in the X-direction. Please see the discussion of these results on the next page.

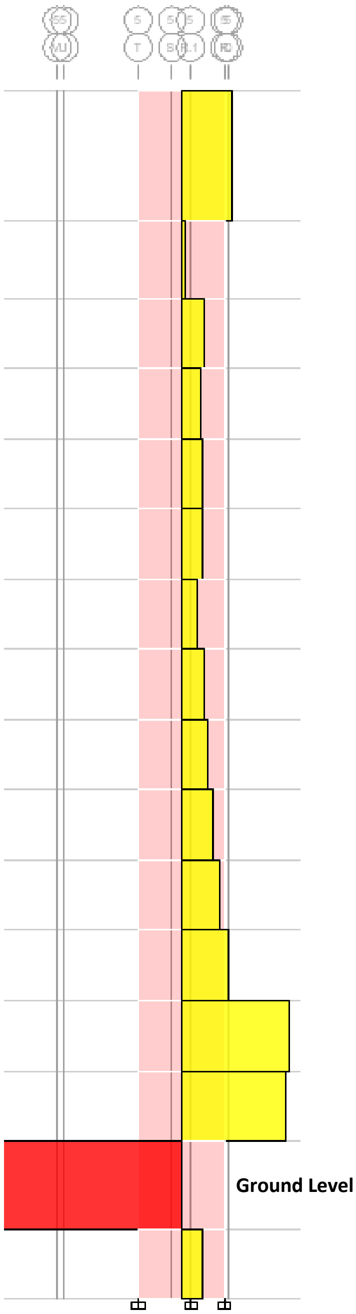


Figure 9 | Shear Diagram for Shear Wall T-R, Line 5
(ETABS GENERATED)

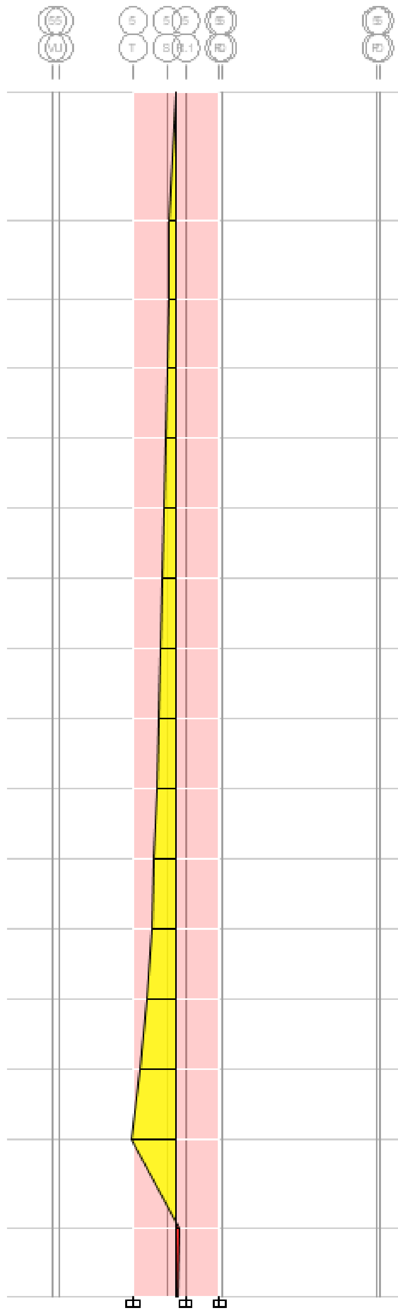


Figure 10 | Moment Diagram for Shear Wall T-R, Line 5
(ETABS GENERATED)

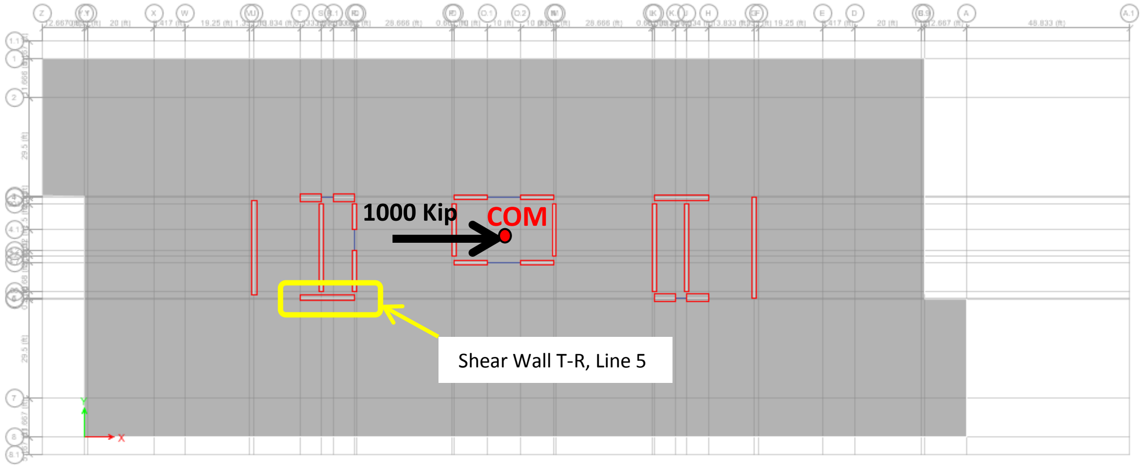


Figure 11 | Location Shear Wall T-R, Line 5 on Level 3
(ETABS GENERATED)

The shear and moment diagrams can be views for Shear Wall T-R on Line 5 on the previous page. The results are due to a 1000 kip load in the X-Direction as indicated in the floor plan above. Therefore, this is an element taking direct shear.

First, I will discuss the shear reversal at Ground Level. This is due to the introduction of foundation walls at this level. The lower two levels have foundation walls around the building perimeter. When the foundation walls are introduced, they will take more shear than is applied and cause a negative shear in the shear wall. This sudden jump in shear is due to creating a “pinned” condition between the rigid diaphragm and the shear wall that will transmit shear to the foundation walls. This affect can be seen in the shear diagram above and is an expected response.

The shear on the top most level increases drastically from the level below it. This is because at the penthouse level, several shear walls end. Therefore, there are less shear walls to take the loads. As a result, this shear wall must take more shear force.

Looking at the moment diagram, the behavior of the shear wall is as we would expect. It behaves as a cantilever until we reach the level were foundation walls begin; here the moment diagram changes due to the shear reversal. Overall, the wall is behaving as expected.

Load Cases Applied to Model

Earthquake Load Cases:

Check Torsional Amplification Factor

Load Case	δmax	δavg	Ax	Need to Adjust EQ Loads?
EX + EXT	108.489735	107.162625	0.712	No
EX - EXT	107.409049	106.866427	0.702	No
EY + EYT	49.381585	43.434636	0.898	No
EY - EYT	60.002762	44.946235	1.238	Yes

X-Direction Seismic Forces

X - Direction: Seismic Story Forces						
Floor Number	hi (ft)	Story Forces Fi (kip)	By (ft)	5% By	Ax	±Mz (ft-k)
Penthouse Roof	24.33	198.66	31.00	1.55	1.00	307.93
Penthouse Floor	14.50	927.03	115.00	5.75	1.00	5330.39
13	13.17	1127.34	115.00	5.75	1.00	6482.21
12	13.17	1010.04	115.00	5.75	1.00	5807.73
11	13.17	895.63	115.00	5.75	1.00	5149.90
10	13.17	784.33	115.00	5.75	1.00	4509.88
9	13.17	676.36	115.00	5.75	1.00	3889.07
8	13.17	572.03	115.00	5.75	1.00	3289.15
7	13.17	471.69	115.00	5.75	1.00	2712.21
6	13.17	375.81	115.00	5.75	1.00	2160.91
5	13.17	285.01	115.00	5.75	1.00	1638.80
4	13.17	200.20	115.00	5.75	1.00	1151.15
3	13.17	122.59	115.00	5.75	1.00	704.91
2	15.00	51.74	115.00	5.75	1.00	297.52
		7698.46				

Y-Direction Seismic Story Forces

Y-Direction: Seismic Story Forces						
Floor Number	hi (ft)	Story Forces Fi (kip)	By (ft)	5% By	Ax	+Mz (ft-k)
Penthouse Roof	24.33	198.66	150.00	7.50	1.00	1489.96
Penthouse Floor	14.50	927.03	254.00	12.70	1.00	11773.22
13	13.17	1127.34	254.00	12.70	1.00	14317.22
12	13.17	1010.04	254.00	12.70	1.00	12827.51
11	13.17	895.63	254.00	12.70	1.00	11374.55
10	13.17	784.33	254.00	12.70	1.00	9960.95
9	13.17	676.36	254.00	12.70	1.00	8589.78
8	13.17	572.03	254.00	12.70	1.00	7264.74
7	13.17	471.69	254.00	12.70	1.00	5990.45
6	13.17	375.81	254.00	12.70	1.00	4772.80
5	13.17	285.01	254.00	12.70	1.00	3619.61
4	13.17	200.20	254.00	12.70	1.00	2542.54
3	13.17	122.59	254.00	12.70	1.00	1556.94
2	15.00	51.74	254.00	12.70	1.00	657.14
		7698.46				

Y-Direction: Seismic Story Forces						
Floor Number	hi (ft)	Story Forces Fi (kip)	By (ft)	5% By	Ax	-Mz (ft-k)
Penthouse Roof	24.33	198.66	150.00	7.50	1.24	1844.03
Penthouse Floor	14.50	927.03	254.00	12.70	1.24	14570.97
13	13.17	1127.34	254.00	12.70	1.24	17719.53
12	13.17	1010.04	254.00	12.70	1.24	15875.80
11	13.17	895.63	254.00	12.70	1.24	14077.57
10	13.17	784.33	254.00	12.70	1.24	12328.05
9	13.17	676.36	254.00	12.70	1.24	10631.03
8	13.17	572.03	254.00	12.70	1.24	8991.11
7	13.17	471.69	254.00	12.70	1.24	7414.01
6	13.17	375.81	254.00	12.70	1.24	5906.99
5	13.17	285.01	254.00	12.70	1.24	4479.76
4	13.17	200.20	254.00	12.70	1.24	3146.74
3	13.17	122.59	254.00	12.70	1.24	1926.93
2	15.00	51.74	254.00	12.70	1.24	813.30
		7698.46				

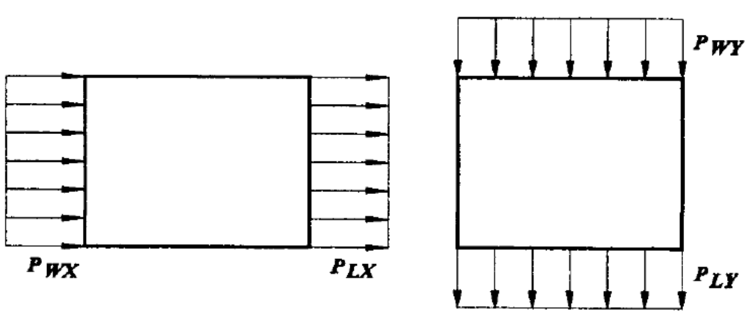
Wind Load Cases:

CASE 1 – X Direction

Wind Pressures | North-South Direction (X-Direction)								
Floor Number	(z)	q_z	q_h	(PSF)	(PSF)	Height	Trib Area (SF)	Force (k)
Ground	0	13.36	22.99	9.65	-5.78	7.50	2092.50	32.30
2	15	13.36	22.99	9.65	-5.78	14.09	1619.78	25.00
3	28.17	15.24	22.99	11.01	-5.78	13.17	1514.55	25.43
4	41.34	16.52	22.99	11.93	-5.78	13.17	1514.55	26.83
5	54.51	17.51	22.99	12.65	-5.78	13.17	1514.55	27.92
6	67.68	18.33	22.99	13.24	-5.78	13.17	1514.55	28.81
7	80.85	19.03	22.99	13.74	-5.78	13.17	1514.55	29.57
8	94.02	19.64	22.99	14.19	-5.78	13.17	1514.55	30.25
9	107.19	20.19	22.99	14.59	-5.78	13.17	1514.55	30.85
10	120.36	20.69	22.99	14.95	-5.78	13.17	1514.55	31.39
11	133.53	21.15	22.99	15.28	-5.78	13.17	1514.55	31.89
12	146.7	21.57	22.99	15.58	-5.78	13.17	1514.55	32.36
13	159.87	21.96	22.99	15.87	-5.78	13.84	1591.03	34.44
Penthouse Floor	173.04	22.33	22.99	16.13	-5.78	19.42	1196.30	26.22
Penthouse Roof	198.67	22.99	22.99	16.61	-5.78	12.17	365.10	8.17

CASE 1 – Y Direction

Wind Pressures | North-South Direction								
Floor Number	(z)	q_z	q_h	(PSF)	(PSF)	Height	Trib Area (SF)	Force (k)
Ground	0	13.36	22.99	9.12	-10.38	7.50	862.50	16.82
2	15	13.36	22.99	9.12	-10.38	14.09	3929.72	76.63
3	28.17	15.24	22.99	10.40	-10.38	13.17	3674.43	76.35
4	41.34	16.52	22.99	11.27	-10.38	13.17	3674.43	79.57
5	54.51	17.51	22.99	11.95	-10.38	13.17	3674.43	82.05
6	67.68	18.33	22.99	12.51	-10.38	13.17	3674.43	84.10
7	80.85	19.03	22.99	12.98	-10.38	13.17	3674.43	85.85
8	94.02	19.64	22.99	13.40	-10.38	13.17	3674.43	87.39
9	107.19	20.19	22.99	13.78	-10.38	13.17	3674.43	88.77
10	120.36	20.69	22.99	14.12	-10.38	13.17	3674.43	90.02
11	133.53	21.15	22.99	14.43	-10.38	13.17	3674.43	91.17
12	146.7	21.57	22.99	14.72	-10.38	13.17	3674.43	92.23
13	159.87	21.96	22.99	14.99	-10.38	13.84	3859.97	97.92
Penthouse Floor	173.04	22.33	22.99	15.24	-10.38	19.42	3300.25	84.55
Penthouse Roof	198.67	22.99	22.99	15.69	-10.38	12.17	1277.85	33.31



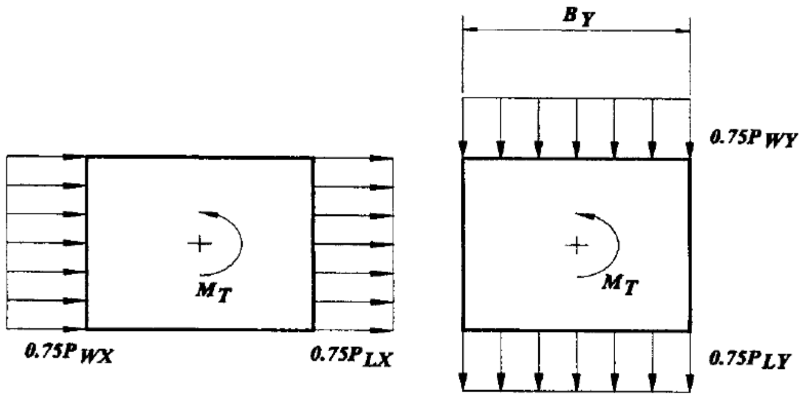
CASE 1

CASE 2 – X Direction

Wind Pressures | North-South Direction									
Floor Number	Height above	Windward	Leeward	Trib Height	Trib Area (SF)	Force (k)	75% of Force (k)	15% By (ft)	±Mz (ft-k)
Ground	0	9.65	-5.78	7.50	2092.50	32.30	24.22	4.65	112.64
2	15	9.65	-5.78	14.09	1619.78	25.00	18.75	17.25	323.47
3	28.17	11.01	-5.78	13.17	1514.55	25.43	19.07	17.25	329.01
4	41.34	11.93	-5.78	13.17	1514.55	26.83	20.12	17.25	347.15
5	54.51	12.65	-5.78	13.17	1514.55	27.92	20.94	17.25	361.17
6	67.68	13.24	-5.78	13.17	1514.55	28.81	21.61	17.25	372.72
7	80.85	13.74	-5.78	13.17	1514.55	29.57	22.18	17.25	382.62
8	94.02	14.19	-5.78	13.17	1514.55	30.25	22.68	17.25	391.31
9	107.19	14.59	-5.78	13.17	1514.55	30.85	23.14	17.25	399.09
10	120.36	14.95	-5.78	13.17	1514.55	31.39	23.55	17.25	406.15
11	133.53	15.28	-5.78	13.17	1514.55	31.89	23.92	17.25	412.62
12	146.7	15.58	-5.78	13.17	1514.55	32.36	24.27	17.25	418.61
13	159.87	15.87	-5.78	13.84	1591.03	34.44	25.83	17.25	445.61
Penthouse Floor	173.04	16.13	-5.78	19.42	1196.30	26.22	19.66	17.25	339.18
Penthouse Roof	198.67	16.61	-5.78	12.17	365.10	8.17	6.13	17.25	105.76

CASE 2 – Y Direction

Wind Pressures | North-South Direction									
Floor Number	Height above	Windward	Leeward	Trib Height	Trib Area (SF)	Force (k)	75% of Force (k)	15% Bx (ft)	±Mz (ft-k)
Ground	0	9.12	-10.38	7.50	862.50	16.82	12.61	22.50	283.81
2	15	9.12	-10.38	14.09	3929.72	76.63	57.47	38.10	2189.66
3	28.17	10.40	-10.38	13.17	3674.43	76.35	57.27	38.10	2181.82
4	41.34	11.27	-10.38	13.17	3674.43	79.57	59.68	38.10	2273.65
5	54.51	11.95	-10.38	13.17	3674.43	82.05	61.54	38.10	2344.62
6	67.68	12.51	-10.38	13.17	3674.43	84.10	63.07	38.10	2403.10
7	80.85	12.98	-10.38	13.17	3674.43	85.85	64.39	38.10	2453.19
8	94.02	13.40	-10.38	13.17	3674.43	87.39	65.54	38.10	2497.20
9	107.19	13.78	-10.38	13.17	3674.43	88.77	66.58	38.10	2536.58
10	120.36	14.12	-10.38	13.17	3674.43	90.02	67.51	38.10	2572.31
11	133.53	14.43	-10.38	13.17	3674.43	91.17	68.37	38.10	2605.07
12	146.7	14.72	-10.38	13.17	3674.43	92.23	69.17	38.10	2635.37
13	159.87	14.99	-10.38	13.84	3859.97	97.92	73.44	38.10	2798.09
Penthouse Floor	173.04	15.24	-10.38	19.42	3300.25	84.55	63.41	38.10	2416.11
Penthouse Roof	198.67	15.69	-10.38	12.17	1277.85	33.31	24.99	38.10	951.93



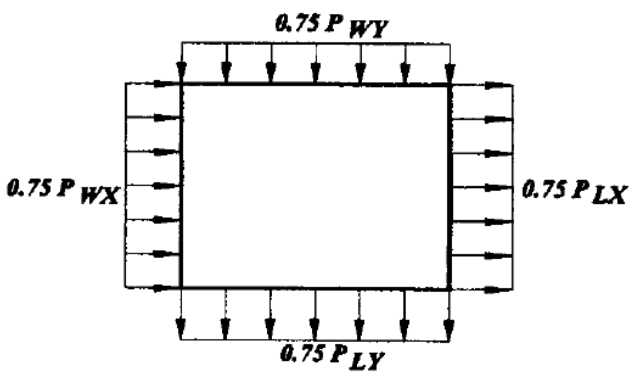
$M_T = 0.75\ (P_{WX}+P_{LX})B_X e_X$ $\quad$ $M_T = 0.75\ (P_{WY}+P_{LY})B_Y e_Y$

$e_X = \pm\ 0.15\ B_X$ $\quad$ $e_Y = \pm\ 0.15\ B_Y$

CASE 2

CASE 3

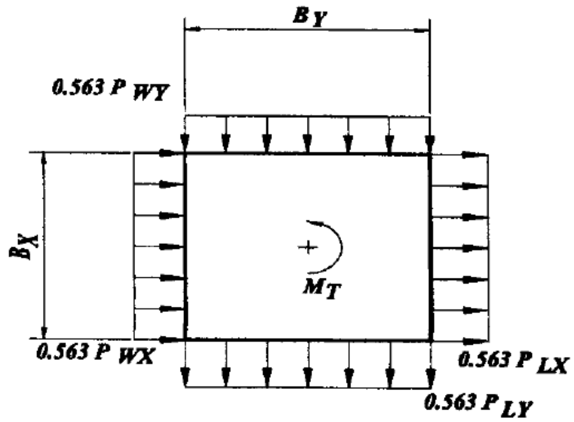
Floor Number	X-Direction Force	Y-Direction Force
Ground	24.22	18.17
2	18.75	14.06
3	19.07	14.30
4	20.12	15.09
5	20.94	15.70
6	21.61	16.21
7	22.18	16.64
8	22.68	17.01
9	23.14	17.35
10	23.55	17.66
11	23.92	17.94
12	24.27	18.20
13	25.83	19.37
Penthouse Floor	19.66	14.75
Penthouse Roof	6.13	4.60



CASE 3

CASE 4 – Moments of Same Signs

Level	15% Bx (ft)	15% By (ft)	0.563 Fx (k)	0.563 Fy (k)	±Mt (ft-k)
Ground	38.10	17.25	18.18	9.47	674.44
2	38.10	17.25	14.08	43.14	1886.52
3	38.10	17.25	14.32	42.99	1884.80
4	38.10	17.25	15.11	44.80	1967.35
5	38.10	17.25	15.72	46.19	2031.15
6	38.10	17.25	16.22	47.35	2083.72
7	38.10	17.25	16.65	48.33	2128.75
8	38.10	17.25	17.03	49.20	2168.31
9	38.10	17.25	17.37	49.98	2203.71
10	38.10	17.25	17.67	50.68	2235.83
11	38.10	17.25	17.96	51.33	2265.28
12	38.10	17.25	18.22	51.92	2292.52
13	38.10	17.25	19.39	55.13	2434.94
Penthouse Floor	38.10	17.25	14.76	47.60	2068.30
Penthouse Roof	22.50	4.65	4.60	18.76	443.40



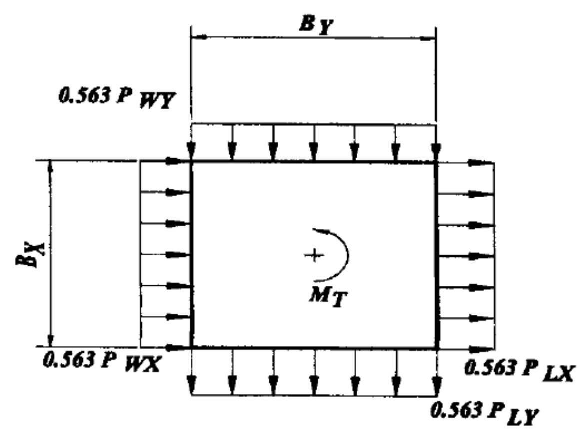
$$M_T = 0.563\,(P_{WX}+P_{LX})B_X e_X + 0.563\,(P_{WY}+P_{LY})B_Y e_Y$$

$$e_X = \pm\,0.15\,B_X \qquad e_Y = \pm\,0.15\,B_Y$$

CASE 4

CASE 4 – Moments of Opposite Signs

Level	15% Bx (ft)	15% By (ft)	0.563 Fx (k)	0.563 Fy (k)	±Mt (ft-k)
Ground	38.10	-17.25	18.18	9.47	47.08
2	38.10	-17.25	14.08	43.14	1400.89
3	38.10	-17.25	14.32	42.99	1390.85
4	38.10	-17.25	15.11	44.80	1446.16
5	38.10	-17.25	15.72	46.19	1488.91
6	38.10	-17.25	16.22	47.35	1524.14
7	38.10	-17.25	16.65	48.33	1554.31
8	38.10	-17.25	17.03	49.20	1580.82
9	38.10	-17.25	17.37	49.98	1604.54
10	38.10	-17.25	17.67	50.68	1626.06
11	38.10	-17.25	17.96	51.33	1645.79
12	38.10	-17.25	18.22	51.92	1664.05
13	38.10	-17.25	19.39	55.13	1765.93
Penthouse Floor	38.10	-17.25	14.76	47.60	1559.08
Penthouse Roof	22.50	-4.65	4.60	18.76	400.60



$$M_T = 0.563\,(P_{WX}+P_{LX})B_X e_X + 0.563\,(P_{WY}+P_{LY})B_Y e_Y$$

$$e_X = \pm\,0.15\,B_X \qquad e_Y = \pm\,0.15\,B_Y$$

CASE 4

Wind and Seismic Drift

Earthquake Drift Results and Discussion:

The drift results associated with the four earthquake load cases can be viewed in the following spreadsheets. The results, as can be seen, are not very good. Many of the drifts fail the ASCE 7-05 code minimum of 2.0% drift.

However, this is not of great concern. The model used to produce these results is not a 100% accurate reflection of the actual lateral system in La Jolla Commons Phase II Office Tower. The walls modeled do not intersect when in reality they will intersect. The walls were disconnected in the model due to inaccurate load distribution by the program.

When the walls were joined, the load distribution was incorrect. However, the overall drift of the entire building was only 54 inches, compared to the 110 inch drift without joining walls. The code drift maximum is 48 inches for a building height of 200 feet. Therefore, the building would most likely pass earthquake drift checks if the walls were modeled correctly. Further investigations will be done to create more accurate results.

Wind Drift Results and Discussion:

The drifts associated with the wind loads all pass the *L/400* industry accepted ratio for serviceability. Each wind load case was run and the highest resulting deflection was found. This deflection was the compared to the *L/400* value. The results of these checks can be viewed in the pages to follow.

It is interesting that although the walls do not connect to one another in the model, the wind drifts still pass the checks, when the earthquake cases do not. This shows that the earthquake loading is obviously the controlling load case when considering serviceability and drifts.

Once again, more investigation will be done on the lateral model to more accurately portray the drift behavior under wind loading. But for now, these results prove that the design of LJC II for serviceability was controlled by earthquake forces.

Earthquake Drift Results

TABLE 12.12-1 ALLOWABLE STORY DRIFT, Δ_a [a,b]

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$ [c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

Figure 12 | Allowable Seismic Story Drift from ASCE 7-05

Max Story Drift According to ASCE 7 - 05: 0.020 h_{sx} (From ASCE 7-05 Table 12.12-1 for All other structures)

C_d Value = 5, $\delta_x = \dfrac{C_d\,\delta_{xe}}{I}$

Earthquake: EX + EXT											
Story	**Load Combination**	**Direction of Deflection**	**Drift**	**% Drift**	**Code Allowed**	**Pass/ Fail?**	**Direction of Deflection**	**Drift**	**% Drift**	**Code Allowed**	**Pass/ Fail?**
Penthouse	EX + EXT	X	0.0550	5.504%	2.0%	Fail	Y	0.004	0.390%	2.0%	Pass
Roof	EX + EXT	X	0.0570	5.704%	2.0%	Fail	Y	0.007	0.695%	2.0%	Pass
Level 13	EX + EXT	X	0.0574	5.737%	2.0%	Fail	Y	0.007	0.694%	2.0%	Pass
Level 12	EX + EXT	X	0.0577	5.769%	2.0%	Fail	Y	0.007	0.687%	2.0%	Pass
Level 11	EX + EXT	X	0.0577	5.769%	2.0%	Fail	Y	0.007	0.673%	2.0%	Pass
Level 10	EX + EXT	X	0.0571	5.709%	2.0%	Fail	Y	0.007	0.651%	2.0%	Pass
Level 9	EX + EXT	X	0.0557	5.570%	2.0%	Fail	Y	0.006	0.620%	2.0%	Pass
Level 8	EX + EXT	X	0.0535	5.346%	2.0%	Fail	Y	0.006	0.581%	2.0%	Pass
Level 7	EX + EXT	X	0.0502	5.016%	2.0%	Fail	Y	0.005	0.532%	2.0%	Pass
Level 6	EX + EXT	X	0.0456	4.560%	2.0%	Fail	Y	0.005	0.471%	2.0%	Pass
Level 5	EX + EXT	X	0.0396	3.955%	2.0%	Fail	Y	0.004	0.398%	2.0%	Pass
Level 4	EX + EXT	X	0.0320	3.204%	2.0%	Fail	Y	0.003	0.317%	2.0%	Pass
Level 3	EX + EXT	X	0.0223	2.226%	2.0%	Fail	Y	0.002	0.187%	2.0%	Pass
Level 2	EX + EXT	X	0.0116	1.162%	2.0%	Pass	Y	0.001	0.081%	2.0%	Pass
Ground	EX + EXT	X	0.0003	0.027%	2.0%	Pass	Y	0.000	0.002%	2.0%	Pass
LL1	EX + EXT	X	0.0001	0.006%	2.0%	Pass	Y	0.000	0.000%	2.0%	Pass

Earthquake: EX - EXT											
Story	Load Combination	Direction of Deflection	Drift	% Drift	Code Allowed	Pass/ Fail?	Direction of Deflection	Drift	% Drift	Code Allowed	Pass/ Fail?
Penthouse	EX - EXT	X	0.05441	5.441%	2.0%	Fail	Y	0.00146	0.146%	2.0%	Pass
Roof	EX - EXT	X	0.05517	5.517%	2.0%	Fail	Y	0.00265	0.265%	2.0%	Pass
Level 13	EX - EXT	X	0.0555	5.550%	2.0%	Fail	Y	0.00264	0.264%	2.0%	Pass
Level 12	EX - EXT	X	0.05585	5.585%	2.0%	Fail	Y	0.00263	0.263%	2.0%	Pass
Level 11	EX - EXT	X	0.0559	5.590%	2.0%	Fail	Y	0.00262	0.262%	2.0%	Pass
Level 10	EX - EXT	X	0.05538	5.538%	2.0%	Fail	Y	0.00258	0.258%	2.0%	Pass
Level 9	EX - EXT	X	0.05409	5.409%	2.0%	Fail	Y	0.00251	0.251%	2.0%	Pass
Level 8	EX - EXT	X	0.05198	5.198%	2.0%	Fail	Y	0.00241	0.241%	2.0%	Pass
Level 7	EX - EXT	X	0.04884	4.884%	2.0%	Fail	Y	0.00227	0.227%	2.0%	Pass
Level 6	EX - EXT	X	0.04446	4.446%	2.0%	Fail	Y	0.00209	0.209%	2.0%	Pass
Level 5	EX - EXT	X	0.03863	3.863%	2.0%	Fail	Y	0.00185	0.185%	2.0%	Pass
Level 4	EX - EXT	X	0.03134	3.134%	2.0%	Fail	Y	0.00155	0.155%	2.0%	Pass
Level 3	EX - EXT	X	0.02247	2.247%	2.0%	Fail	Y	0.001	0.100%	2.0%	Pass
Level 2	EX - EXT	X	0.01155	1.155%	2.0%	Pass	Y	0.00048	0.048%	2.0%	Pass
Ground	EX - EXT	X	0.00027	0.027%	2.0%	Pass	Y	6.0E-06	0.001%	2.0%	Pass
LL1	EX - EXT	X	6.2E-05	0.006%	2.0%	Pass	Y	0.00	0.000%	2.0%	Pass

Earthquake: EY + EYT											
Story	Load Combination	Direction of Deflection	Drift	% Drift	Code Allowed	Pass/Fail ?	Direction of Deflection	Drift	% Drift	Code Allowed	Pass/Fail ?
Penthouse	EY + EYT	X	0.00079	0.079%	2.0%	Pass	Y	0.02674	2.674%	2.0%	Fail
Roof	EY + EYT	X	0.00265	0.265%	2.0%	Pass	Y	0.02952	2.952%	2.0%	Fail
Level 13	EY + EYT	X	0.00263	0.263%	2.0%	Pass	Y	0.02939	2.939%	2.0%	Fail
Level 12	EY + EYT	X	0.0026	0.260%	2.0%	Pass	Y	0.02909	2.909%	2.0%	Fail
Level 11	EY + EYT	X	0.00253	0.253%	2.0%	Pass	Y	0.02852	2.852%	2.0%	Fail
Level 10	EY + EYT	X	0.00243	0.243%	2.0%	Pass	Y	0.02763	2.763%	2.0%	Fail
Level 9	EY + EYT	X	0.0023	0.230%	2.0%	Pass	Y	0.02639	2.639%	2.0%	Fail
Level 8	EY + EYT	X	0.00213	0.213%	2.0%	Pass	Y	0.02481	2.481%	2.0%	Fail
Level 7	EY + EYT	X	0.00193	0.193%	2.0%	Pass	Y	0.02278	2.278%	2.0%	Fail
Level 6	EY + EYT	X	0.00168	0.168%	2.0%	Pass	Y	0.02028	2.028%	2.0%	Fail
Level 5	EY + EYT	X	0.00139	0.139%	2.0%	Pass	Y	0.01728	1.728%	2.0%	Pass
Level 4	EY + EYT	X	0.00107	0.107%	2.0%	Pass	Y	0.01389	1.389%	2.0%	Pass
Level 3	EY + EYT	X	0.00068	0.068%	2.0%	Pass	Y	0.00998	0.998%	2.0%	Pass
Level 2	EY + EYT	X	0.00014	0.014%	2.0%	Pass	Y	0.0061	0.610%	2.0%	Pass
Ground	EY + EYT	X	5.4E-05	0.005%	2.0%	Pass	Y	0.00055	0.055%	2.0%	Pass
LL1	EY + EYT	X	7E-06	0.001%	2.0%	Pass	Y	1.9E-05	0.002%	2.0%	Pass

Earthquake: EY - EYT											
Story	oad Combinatio	Direction of Deflection	Drift	% Drift	ode Allowe	Pass/Fail?	Direction of Deflection	Drift	% Drift	ode Allowe	Pass/Fail?
Penthouse	EY - EYT	X	0.002	0.218%	2.0%	Pass	Y	0.0335	3.354%	2.0%	Fail
Roof	EY - EYT	X	0.007	0.732%	2.0%	Pass	Y	0.0413	4.130%	2.0%	Fail
Level 13	EY - EYT	X	0.007	0.731%	2.0%	Pass	Y	0.0412	4.117%	2.0%	Fail
Level 12	EY - EYT	X	0.007	0.726%	2.0%	Pass	Y	0.0408	4.084%	2.0%	Fail
Level 11	EY - EYT	X	0.007	0.716%	2.0%	Pass	Y	0.0402	4.018%	2.0%	Fail
Level 10	EY - EYT	X	0.007	0.697%	2.0%	Pass	Y	0.0391	3.907%	2.0%	Fail
Level 9	EY - EYT	X	0.007	0.669%	2.0%	Pass	Y	0.0375	3.748%	2.0%	Fail
Level 8	EY - EYT	X	0.006	0.634%	2.0%	Pass	Y	0.0354	3.544%	2.0%	Fail
Level 7	EY - EYT	X	0.006	0.587%	2.0%	Pass	Y	0.0328	3.278%	2.0%	Fail
Level 6	EY - EYT	X	0.005	0.527%	2.0%	Pass	Y	0.0294	2.943%	2.0%	Fail
Level 5	EY - EYT	X	0.005	0.455%	2.0%	Pass	Y	0.0253	2.534%	2.0%	Fail
Level 4	EY - EYT	X	0.004	0.370%	2.0%	Pass	Y	0.0206	2.063%	2.0%	Fail
Level 3	EY - EYT	X	0.003	0.265%	2.0%	Pass	Y	0.0124	1.242%	2.0%	Pass
Level 2	EY - EYT	X	0.001	0.050%	2.0%	Pass	Y	0.0081	0.808%	2.0%	Pass
Ground	EY - EYT	X	0.000	0.007%	2.0%	Pass	Y	0.0006	0.062%	2.0%	Pass
LL1	EY - EYT	X	0.000	0.001%	2.0%	Pass	Y	0.0000	0.002%	2.0%	Pass

Horizontal and Vertical Irregularities

La Jolla Commons Phase II Office Tower was found to have a Horizontal Type 1b Extreme Torsional Irregularity, according to Table 12.3-1 from ASCE 7-05 as shown in Figure 13. Story displacements were compared, and Level 4 for load case EY – EYT, was determined to have a Maximum vs. Average displacement of 1.40. Therefore, Horizontal Type 1b exists. No other irregularities were found to exist.

TABLE 12.3-1 HORIZONTAL STRUCTURAL IRREGULARITIES

	Irregularity Type and Description	Reference Section	Seismic Design Category Application
1a.	**Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.8.4.3 12.7.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F C, D, E, and F B, C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	**Extreme Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D B, C, and D
2.	**Reentrant Corner Irregularity** is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
3.	**Diaphragm Discontinuity Irregularity** is defined to exist where there are diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
4.	**Out-of-Plane Offsets Irregularity** is defined to exist where there are discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.	12.3.3.4 12.3.3.3 12.7.3 Table 12.6-1 16.2.2	D, E, and F B, C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
5.	**Nonparallel Systems-Irregularity** is defined to exist where the vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic force–resisting system.	12.5.3 12.7.3 Table 12.6-1 Section 16.2.2	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

Figure 13 | Horizontal Irregularities from ASCE 7-05

Wind Drift Results

Maximum Drift for Entire Building Height: *L/400*
(based on a commonly used industry standard for wind deflections)

Wind Load Deflections				
Load Case	Direction of Deflection	Maximum Drift (in)	Allowable Drift (in)	Pass/Fail?
CASE 1: Wx	X	3.9968	5.94	Pass
CASE 1: Wy	Y	5.443111	5.94	Pass
CASE 2 X: Wx + Mz	Y	0.437194	5.94	Pass
CASE 2 X: Wx - Mz	Y	0.437194	5.94	Pass
CASE 2 Y: Wy - Mz	Y	2.888927	5.94	Pass
CASE 2 Y: Wy + Mz	Y	2.888927	5.94	Pass
CASE 3	X	2.992136	5.94	Pass
CASE 4: +Moments Add	Y	4.620775	5.94	Pass
CASE 4: -Moments Add	Y	5.435449	5.94	Pass
CASE 4: +Moments Opposite	Y	4.076888	5.94	Pass
CASE 4: -Moments Opposite	Y	4.806277	5.94	Pass

Overturning Moment and Impact on Foundations

Building Overturning and Maximum Base Shear:

The maximum story moment was determined to be a result of earthquake forces from load case EY + EYT, Mx = 1,175,735 ft-kip. This was then compared to the resisting moment, determined using the building weight and the appropriate moment arm for overturning. The resisting moment was compared to the overturning moment. A factor of safety of 5.23 was found to exist, which is greater than the 1.5 minimum required by code.

The seismic load cases control the building base shear with V_b=7698.46 kip.

Impact on Foundations:

The foundation for La Jolla Commons Phase II Office Tower is a concrete mat. The mat ranges from 4.5 to 6 feet in thickness. The foundation must withstand the total base shear and total moment associated with the worst case loads. See Figure 14 for a general diagram of the building foundation. The controlling load combination for the mat foundation is *1.2D + 1.0E + L + 0.2S* and, for the foundation walls, it is *0.9D + 1.0E + 0.6H.*

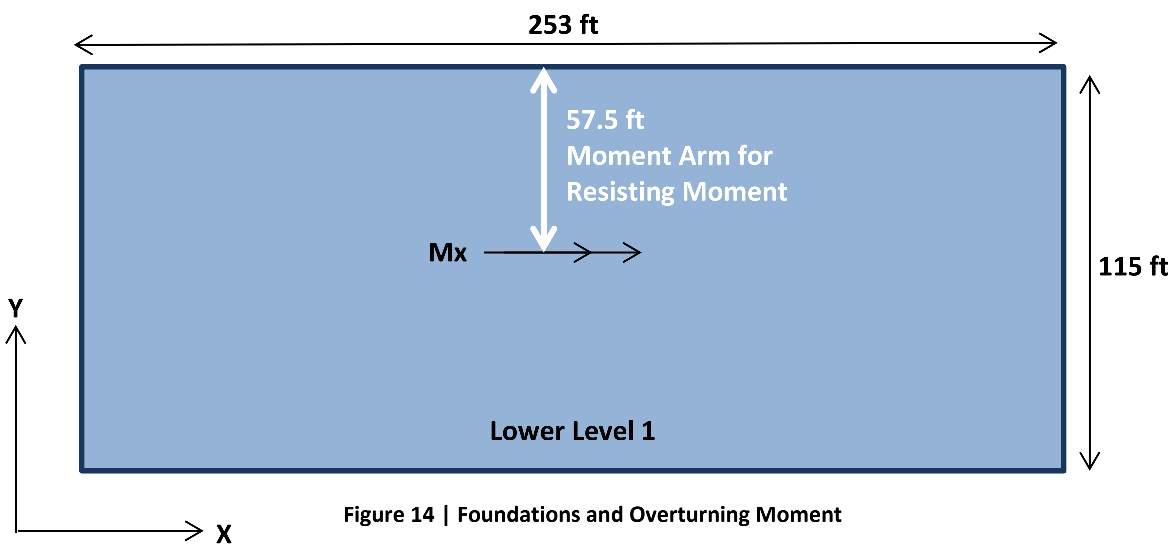


Figure 14 | Foundations and Overturning Moment

Base Shears and Overturning Moments for Seismic:

Story	Load Combo	Vx (kip)	Vy (kip)	Mx (kip-ft)	My (kip-ft)
LL1	EX + EXT	7698.46	0	0	1175735
LL1	EX - EXT	7698.46	0	0	1175735
LL1	EY + EYT	0	7698.46	1175735	0
LL1	EY - EYT	0	7698.46	1175735	0

Base Shears and Overturning Moments for Wind:

Story	Load Combo	Vx (kip)	Vy (kip)	Mx (kip-ft)	My (kip-ft)
LL1	CASE 1: Wx	421.4	0.0	0	50491
LL1	CASE 1: Wy	0.0	1166.7	147494	0
LL1	CASE 2 X: Wx - Mz	0.0	0.0	0	0
LL1	CASE 2 X: Wx + Mz	0.0	0.0	0	0
LL1	CASE 2 Y: Wy - Mz	0.0	0.0	0	0
LL1	CASE 2 Y: Wy + Mz	0.0	0.0	0	0
LL1	CASE 3	316.1	237.1	28402	37869
LL1	CASE 4: +Moments Add	237.3	656.9	83039	28427
LL1	CASE 4: +Moments Opposite	237.3	656.9	83039	28427
LL1	CASE 4: -Moments Add	237.3	656.9	83039	28427
LL1	CASE 4: -Moments Opposite	237.3	656.9	83039	28427

Check building for Overturning:

Controlling Load Case	EY + EYT
Controlling Load Combination	1.2D + 1.0E + L + S

Overturning Moment	1175735.00	ft-kip

Total Building Weight	106923.00	kips
Moment Arm	57.50	ft
$M_{resisting}$	6148072.50	ft-kip

F.S.	5.23 > 1.5
Pass/Fail	Pass

Overall Controlling Base Shear:

Seismic Loads			
Control Base Shear	Vb=	7698.46	kip

Determine Controlling Load Combination for Foundation Design:

Because both the moment and shear controlling the foundation design are a result of seismic forces, the controlling load combination for the mat foundation design will be *1.2D + 1.0E + L + 0.2S*. Load combination *0.9D + 1.0E + 1.6H* will govern the design of the foundation walls.

Check of Critical Members

Shear Wall Strength Checks:

For all the shear walls, seismic loads controlled the design. Therefore, the controlling load case was determined to be *1.2D +1.0E + L + 0.2S*, where $E = E_v + E_h$.

Three shear walls were initially investigated for shear strength. The first wall checked was Shear Wall G at Lower Level 1. This wall was selected because it took the most shear of any shear wall in the building (see Appendix B for method of determination). The wall failed to pass when checked against the maximum allowed shear strength of the wall for specially reinforced concrete shear walls. Because this wall is at a level with shear reversal effects, the shear force in the wall is not necessarily a valid value. Therefore, Shear Wall 5, T-R was checked at Level 2 (above shear reversal levels). This wall also did not pass the maximum shear capacity for special reinforced shear walls; it also did not pass the maximum shear capacity for ordinary shear walls.

In order to perform all the required checks for a typical wall, a shear wall was chosen that was expected to pass. The wall chosen was Shear Wall U at Level 2, where the controlling load case was EY + EYT (i.e. seismic forces in the Y-direction with accidental torsion). This shear wall met all the requirements for special reinforced concrete shear walls. Also, the wall was checked for axial and bending interaction and was found to be adequate for the required loads. See Figure 15 for the interaction diagram. The calculations for all three shear walls can be seen on the pages to follow.

Determination of the critical wall sections can be found in Appendix B.

Determination of axial loads on checked shear walls can be found in Appendix C.

Shear Wall Strength Check: SW G @ LL1

Earthquake loads control: $Q_E = 6444$ kip

$1.2D + 1.0E + L + \cancel{0.2S} \rightarrow 0$

$E = E_h + E_v$

$E_h = \rho Q_E$

$\rho = 1.3$ for SDC D

$E_h = (1.3)(6444) = 8377.2^k$

$E_v = 0.2 S_{DS} D$, $D = 3650^k$

$= 0.2(0.9453)(3650^k)$

$E_v = 690.07^k$

$1.0 E_h = 8377.2^k$

$1.0 E_v = 690.07^k$

See Excel sheet for axial load.

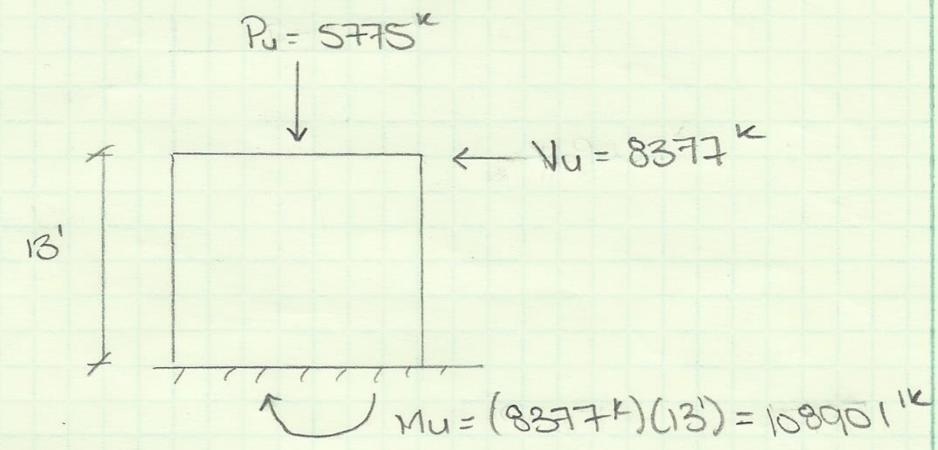
$P_{live} = 1395^k$

$P_{dead} = 3650^k$

$P = 1.2D + L + 1.0E_v = 1.2(3650^k) + 1395^k + 690.07$

$\underline{P_u = 6465^k}$

$V_u = 1.0 E_H = 8377.2^k$



Wall Information: SWG

height = 13',
length = 30'
thickness = 22"

Reinforcement: vertical #6 @ 9"
horizontal #6 @ 9"
- two curtains.

$\phi V_n \geq V_u$

$V_n = V_c + V_s$

$\phi = 0.6$ → ACI 318 § 9.3.4 for SDC D, Special conc. SW.

$V_{n_{max}} = A_{cv}(\alpha_c \lambda \sqrt{f'c} + \rho_t f_y)$ (ACI 318 § 21.9.4)

$h_w / l_w = 13'/30' = 0.433 \leq 1.5 \rightarrow \alpha_c = 3$

$\lambda = 1.0$

$f'c = 7000$ psi

$\rho_t = \dfrac{2(17 \times 0.44)}{(22'')(13 \times 12)} = 0.00436 > 0.0025$ ✓

$f_y = 60{,}000$ psi

$A_{cv} = (22'')(30' \times 12) = 7920 \text{ in}^2$

$V_{n,max} = (7920 \text{ in}^2)\left[3(1.0)\sqrt{7000} + (0.00436)(60{,}000)\right]$

$V_{n,max} = 4059.77^k$

$V_{n,max} = 4059.77^k < V_u = 8377^k$ ✗

* Because this wall is at a level experiencing shear reversal, the shears in the walls may not be accurate. Therefore, I will check a shearwall above shear reversal levels.

Shear Wall Strength Check: SW 5, T-R, level 2

Controlling Load Case: EX + EXT, $V = 2709.55^k$

Controlling Load Combination:

$1.2D + 1.0E + L + \cancel{0.2S} \rightarrow 0$

$E = E_h + E_v$

$E_h = \rho Q_E$

$= (1.3)(2709.55^k)$
$= 3522.4^k$

$E_v = 0.2 S_{DS} D$
$= 0.2(0.9453)(3092^k)$
$= 584.57^k$

$1.0 E_h = 3522.4^k$
$1.0 E_v = 584.57^k$

$P_{live} = 1269^k$
$P_{dead} = 3092^k$ } See excel sheet for calculations

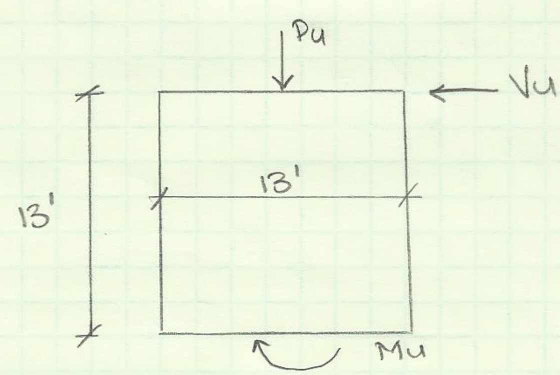
$P_u = 1.2(3092) + (1269) + 1.0(584.57)$

$P_u = 5564^k$

$V_u = 1.0 E_H = 3522.4^k$

$M_u = (3522.4^k)(13')$

$M_u = 45791^{'k}$



Length = 13'
Height = 13'
Thickness = 28"
$f'c$ = 7000 psi
Reinforcement: #7 @ 6" Horizontal } 2 curtains
#7 @ 6" Vertical }

$\phi V_n \geq V_u$

Check special shear wall:

$V_{n,max} = A_{cv} [\alpha_c \lambda \sqrt{f'c} + \rho_t f_y]$

$h_w / l_w = 1.0 \leq 1.5 \rightarrow \alpha_c = 3.0$

$\rho_t = \dfrac{2(26 \times 0.6 in^2)}{(13' \times 12)(28")} = 0.00714 > 0.0025$ ✓

$A_{cv} = (28")(13' \times 12) = 4368 \ in^2$

$V_{n,max} = (4368)[3\sqrt{7000} + 0.00714(60,000)]$

$V_{n,max} = 2967^k$

$V_{n,max} = 2967^k \leq V_u = 3522.4^k$ X

Check Regular Shear Wall Requirements:

$V_{n,max} = 10\sqrt{f'c} \ h \cdot d$

$h = 28"$
$d = 0.8 l_w = 0.8(13 \times 12) = 124.8"$

$V_{n,max} = 10\sqrt{7000}(28)(124.8)$
$= 2923.6^k$

$V_{n,max} = 2933.6^k > V_u = 3522.4^k$ X

* Try a wall that "should" pass so all checks can be performed *

Shear Wall Strength Check: SW4 @ Level 2

Controlling Load Case: EY-E4T, $V = 840.3^k$

Controlling Load Combination:

$1.2D + 1.0E + L$

$E = E_h + E_v$

$E_h = (1.3)(840) = 1092^k$

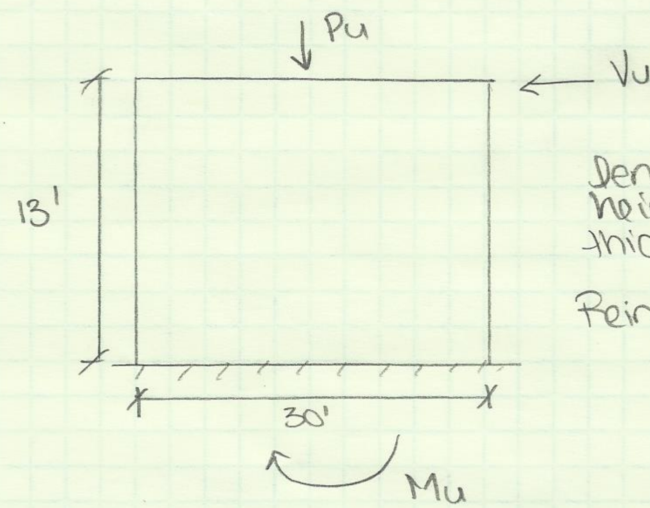
$E_v = 0.2(0.9453)(3092^k) = 584^k$

$D = 3092^k$
$L = 1269^k$

$V_u = 1.0E_H \rightarrow$ $V_u = 1092^k$

$P_u = 1.2D + 1.0L + 1.0E_v = 5563.4^k$

$M_u = (1092)(13) \rightarrow M_u = 14196^{1k}$



length = 30'
height = 13'
thickness = 18"

Reinforcement:

H-#6 @ 9"
V-#6 @ 12" } each face

Special SW Requirements:

$V_{n,max} = A_{cv}[\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y]$

$h_w/l_w = 13/30 = 0.433 < 1.5 \rightarrow \alpha_c = 3.0$

$\rho_t = \dfrac{2(17 \times 0.44)}{(13 \times 12)(18)} = 0.00533 > 0.0025$ ✓

$A_{cv} = (18")(30' \times 12) = 6480\ in^2$

$V_{n,max} = (6480)[(3)\sqrt{7000} + (0.00533)(60{,}000)) = 3699^k$

$\phi = 0.6$ (for special SW)

$\phi V_n = 0.6(3699) = 2219.4^k > 1092^k$

→ Possible for wall to Pass! Check other requirements!

$$V_c = 3.3(1.0)\sqrt{7000}\,(18'')(0.8)(30)(12) + \frac{4980 \times 1000 (288)}{4(30 \times 12)}$$

$$= 2440.3^k$$

OR

$$V_c = (18)(288)\left[(0.6)(1.0)\sqrt{7000} + \frac{(30 \times 12)\left[1.25(1.0)\sqrt{7000} + 0.2\,\frac{5564(100)}{(30\times12)(18)}\right]}{\frac{14196}{1092} - \frac{(30\times12)}{2}}\right]$$

Negative value! Eq. does not apply ×

$V_c = 2440.3^k$

$V_{c,max} = 2\lambda\sqrt{f'_c}\,h \cdot d$

$= 2(1.0)\sqrt{7000}\,(18)(0.8)(30 \times 12)$

$= \underline{867^k} < 2440.3^k$

$\underline{V_c = 867^k}$

$$V_s = \frac{A_v f_y d}{s} = \frac{(0.44\,in^2)(2)(60{,}000)(0.8)(30)(12)}{9''}$$

$\underline{V_s = 1689.6^k}$

$V_n = V_c + V_s = (867^k + 1689.6^k) = 2556.6^k$

$\phi V_n = 0.6(2556.6^k) = 1533.96^k$

$\phi V_n = 1533.96^k > V_u = 1092^k$ ✓

See the following page for the interaction diagram for shear wall U for Axial and bending.

Two Curtains Required?

$$2A_{cv}\lambda\sqrt{f'c} = 2(6480\,in^2)(1.0)\sqrt{7000}/1000$$

$$= 1084.3^k$$

$V_u = 1092^k > 1084.3^k \Rightarrow$ Two Curtains required

Two curtains provided ✓

Check Reinf. Ratios:

$$\rho_t = 0.00533 > 0.0025 ✓$$

$$\rho_\ell = \frac{(0.44\,in^2)(30)(2)}{(30\times12)(18)} = 0.00407 > 0.0025 ✓$$

Check Spacing:

Vertical: $S_{max} = \min \begin{cases} l_w/3 = 30(12)/3 = 120" \\ 3h = 3(18") = 54" \\ 18" \end{cases}$

$S_{max} = 18" > S = 12"$ ✓

Horizontal: $S_{max} = \min \begin{cases} l_w/5 = 72" \\ 3h = 54" \\ 18" \end{cases}$

$S_{max} = 18" \geq S = 9"$ ✓

Conclusion:

Shear wall U is adequate for shear axial and bending strength and meets the requirements of a special reinforced concrete shear wall.

Bending and Axial Interaction Diagram for SW-U at Level 2:

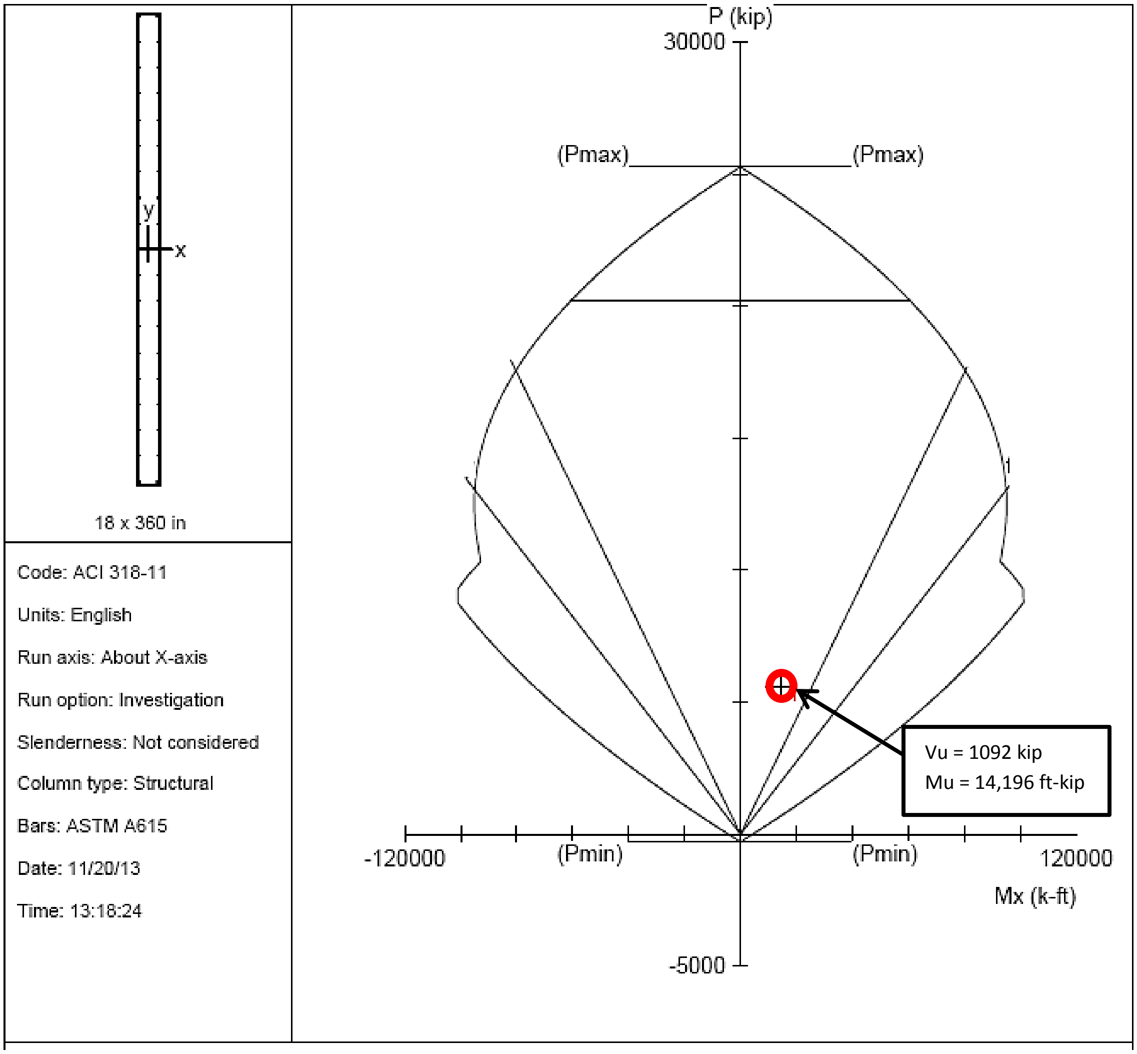


Figure 15 | Shear Wall U – Level 2 Interaction Diagram

Conclusion

This technical report has investigated the lateral system of La Jolla Commons Phase II Office Tower. LJC II uses a system of reinforced concrete shear walls. The building was analyzed for behavior under wind and seismic loading, following the requirements of ASCE 7 – 05. Being that the building is located in San Diego, California, it was not surprising to find that seismic load combinations controlled both strength and drift for the tower.

The ETABS model, as discussed previously in this report, is not 100% accurate portrayal of the tower's lateral system due to the disconnection of shear walls. As a result, the building does not pass drift requirements for seismic loading; however, if the walls were connected, it is safe to assume the building's lateral system would be able to meet the drift requirements. It is also possible that the calculated seismic loads were higher than the loads used in design. For the controlling load case in the Y-Direction, the story drift was found to be acceptable according to accepted industry standards for serviceability.

Furthermore, the building was found to have a Horizontal Irregularity Type 1B – Extreme Torsional Irregularity under earthquake loads in the Y-Direction. Therefore, a torsional amplification factor was applied to the accidental torsional moments for seismic loads in the Y-Direction. No other irregularities were found to exist.

Considering the mat foundation and overturning, LJC II will perform adequately. The building has a factor of safety of over 5 for overturning. The foundation will most likely not see any uplifting effects. However, the foundation should be investigated further for strength against punching shear and bending. The foundation walls should also be analyzed for seismic and soil loads.

Several shear walls were analyzed for strength both below and above grade. Due to shear reversal at the levels below grade, the shear walls below grade seem to take more shear than will actually be observed. Therefore, it is not surprising that these walls did not pass strength checks. Also, seismic loads may have been calculated to be more than used for design. Shear walls above grade, however, passed strength checks for shear, axial, and bending. Also, the walls met detailing requirements for special shear walls.

Overall, the lateral system for La Jolla Commons PhConase II Office Tower is adequate according to industry standards for serviceability and strength considerations.

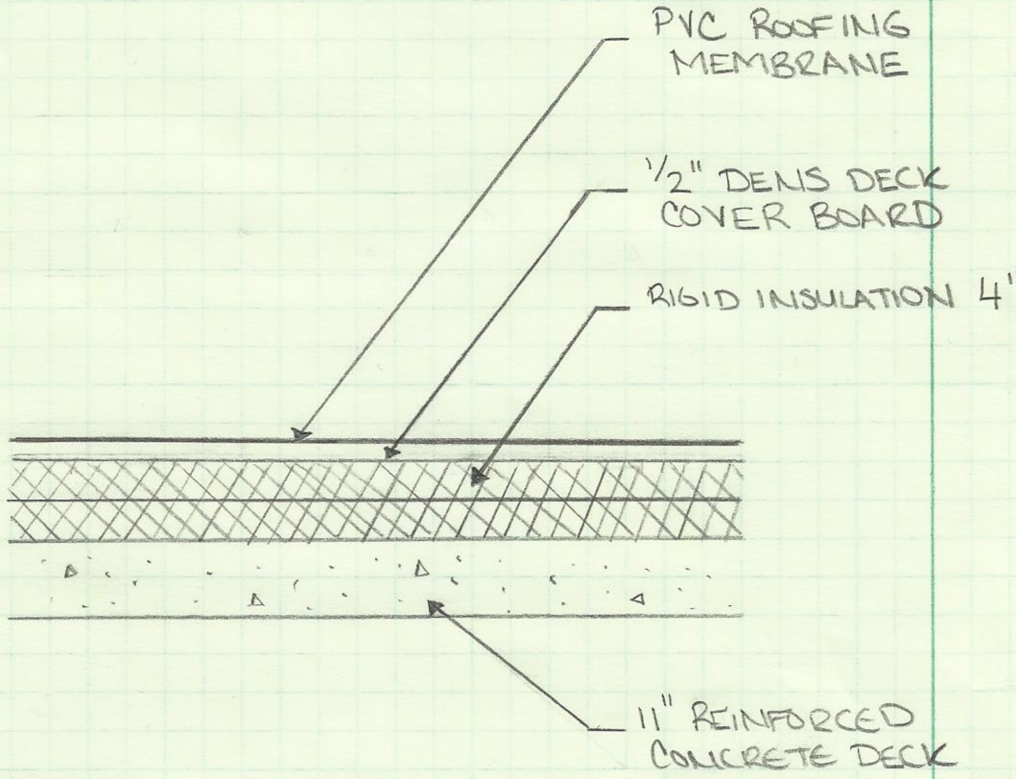
APPENDIX A

Tech 2 Load Calculations

GRAVITY LOADS

Typical Roof Bay Loading

Cross Section of Roof Construction (A470 - B2)



ROOF DEAD LOAD:

Adhered PVC Membrane = 2 PSF

1/2" Dens Deck Cover Board = 2 PSF

4" Rigid Insulation = 6 PSF

Concrete Slab, 11"

$= (150 \text{ PCF})(11"/12") = 137.5 \text{ PSF}$

Superimposed/Misc

Ceilings = 5 PSF
MEP = 15 PSF
Sprinklers = 3 psf
} = 23 PSF

+

ROOF DEAD LOAD = 171 PSF

ROOF LIVE LOAD:

ASCE 7-05 : Ch.4 Table 4-1

$L_r = 20$ PSF

Construction Documents - S001

$L_r = 20$ PSF

* Roof live load used for design is equal to the code minimum value

SNOW LOAD:

ASCE 7-05 : Ch.7

Below 1500 ft elevation → 0 PSF
Elevation from 2000-1500 ft → 5 PSF

Site elevation is about 330 ft (C103)

$P_g = 0$ PSF

$P_f = 0.7 C_e C_t I P_g$

$P_f = 0$ PSF

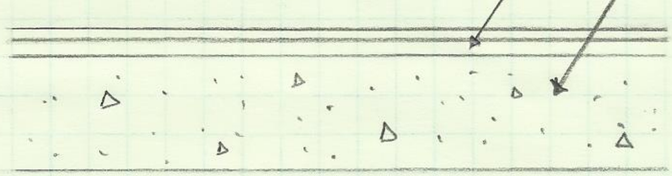
From Figure 7-9 ASCE 7-05:

$h_d = 0$ ft for $P_g = 0$ PSF

Drift Calculation will yield no drift load because $P_g = 0$ PSF

Typical Floor Bay Loading

Cross Section of floor Construction



FLOOR DEAD LOAD:

14" Concrete Slab

$= (150 \text{ PCF})(14''/12'') = 175 \text{ PSF}$

Carpet + Adhesive + Pad = 1.5 PSF

Superimposed / Misc

Ceilings = 5 PSF
MEP = 15 PSF
Fully Sprinkled = 3 PSF
} 23 PSF

Raised access floor (by tenant) = 15 PSF (allowance)

\+ ______

214.5

Typical floor bay dead load = 215 PSF

Typical Floor Bay Live load:

ASCE 7-05 Chapter 4

Office live load = 50 PSF

Interior partitions = + 20 PSF

70 PSF

Offices, Corridors above 1st floor = 80 PSF

→ Apply 80 PSF to entire office area to allow for future layout flexibility.

Typical Bay Floor Live load = 80 PSF

→ This matches the design value from sheet S001 for office spaces = 80 PSF

Non-Typical Dead Loads:

FLOORS and Roofs:

- Ground to Level 13, 18" slab edges

  $= (150\ PCF)(18''/12'') = \underline{225\ PSF}$

- Ground to Level 13, 10" core slab

  $= (150\ PCF)(10''/12'') = \underline{125\ PSF}$

- Roof / Penthouse floor, 11" slab

  $= (150\ PCF)(11''/12'') = \underline{137.5\ PSF}$

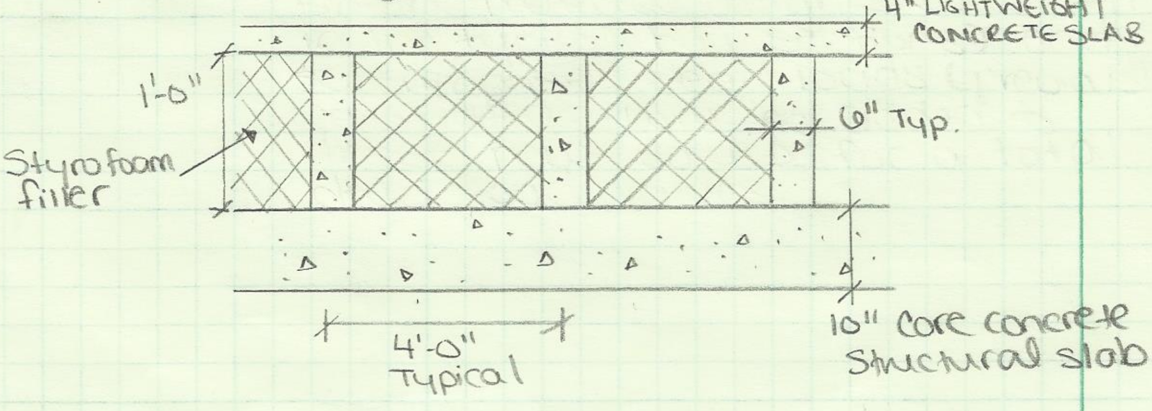
- Roof of Penthouse, 8" slab

  $= (150\ PCF)(8''/12'') = \underline{100\ PSF}$

- 1/4" x 2" - Roof metal bar grating = 15 PSF

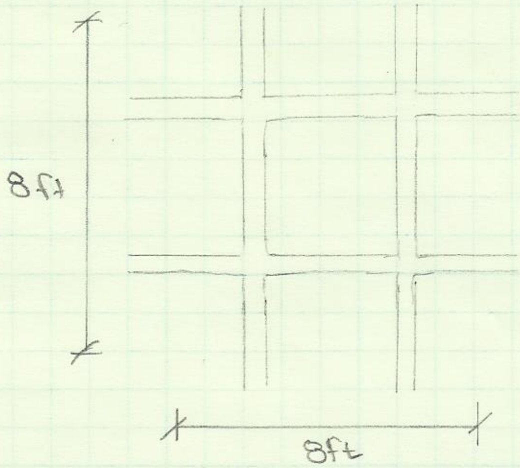
  * From Grating Pacific Catalogue dercuision was done.

- Built-up slab at several locations at building core on each level



See next page for calculation

Built up slab continued:



- Lets consider an 8ft x 8ft segment

LGTWT Concrete

$= (4''/12)(115\ PCF) = \underline{38.3\ PSF}$

Structural Slab

$= (10''/12'')(150\ PCF) = \underline{125\ PSF}$

Pedistals

$(12''/12'')(6''/12)(32')(115\ PCF) = 1840\ lb$

$1840\ lb / 64\ ft^2 = \underline{28.7\ PSF}$

TOTAL at built up slab = $\underline{192\ PSF}$

Special Note For NON-TYP. DEAD LOADS:

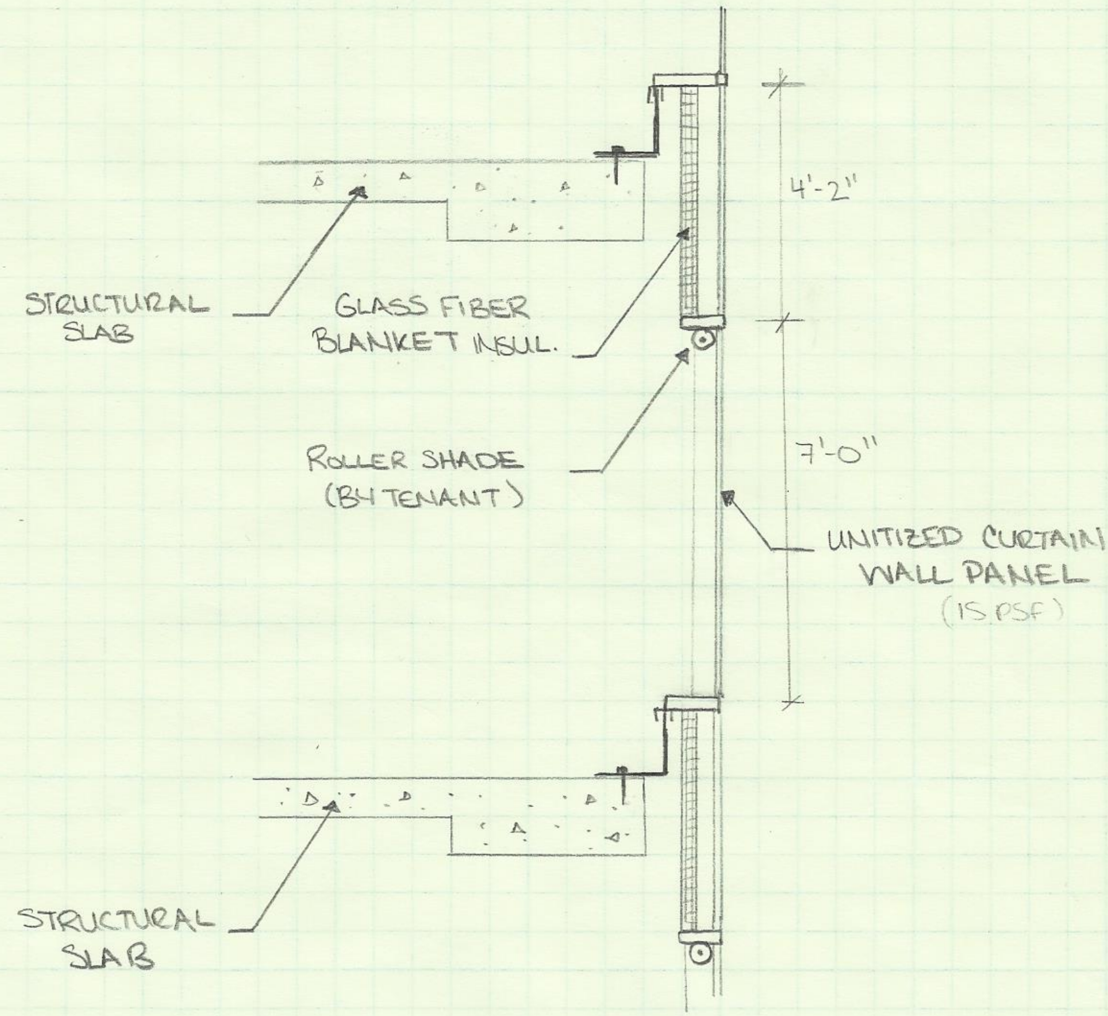
The values provided are modifications to structural components only. The finishes load and superimposed loads (calculated for a typical bay) needs to be added to these values for a total dead load value.

Non-Typical Live Loads:

Use	Location in Building	Design Value	ASCE 7-5 Value	Explaination (if necessary)
Lobby + Corridor	Ground level at core	100 PSF	100 PSF (first floor corridor)	
Lobby + Corridor	Above ground level around core	100 PSF	80 PSF	20 PSF was added for partitions in design to allow flexibility in layout for tenant
Core/ Egress	Above ground level at core	250 PSF	N/A	Value based on egress requirements for possible future multi-tenant conditions
Exit Stairs	At building core	100 PSF	100 PSF	
Cafeteria	In lease space *	100 PSF	100 PSF (Dining and resturants)	
Fitness Center	In lease space *	100 PSF	100 PSF (gymnasiums)	
Conference Center	In lease space *	100 PSF	100 PSF	ASCE 7-10 has an office for heavier anticipated occupancy
Data Center	In lease space *	250 PSF	100 PSF (computer rooms)	PSF load determined from known equipment weights
Mech. Areas	Mechanical Rooms at Core on each level	200 PSF	N/A	Value based on industry standard and actual equipment loads if known

Typical Exterior Wall Load:

Typical Curtain Wall Section



Wall Load Path - Gravity

The curtain wall is essentially hung by the top mullion of each unit from the slab edge. The unit then ties into the unit at the level below. The wall load goes into the slab which transfers the load into the edge columns. The columns will transfer this load down to the mat foundation, which will spead out the load to meet the bearing capacity of the soil.

Typical Curtain Wall Dead Load:

Line load at slab edge

Fiber blanket Insulation

1.0 PCF, 1" → 1 PCF (1"/12")(4' + 2/12) = 0.35 PLF

Roller Shade

Allowance for tenant selection = 5 PLF

Curtain Wall Units

10 PSF (4' + 2"/12" + 7') = 112 PLF
+
117.35 PLF

Curtain Wall Assembly Dead Load = 118 PLF

Adjusted Roof Dead load:

Superimposed / Misc

Ceilings = 5 PSF
MEP = 3 PSF
Sprinklers = 3 psf
} 11 PSF

Total New Roof DEAD LOAD = 156 PSF

Adjusted Floor Dead Load:

Superimposed / Misc

Ceilings = 5 PSF
MEP = 15 PSF
(includes raised floor system)
Sprinklers = 3 PSF
} 23 PSF

Typical Floor Bay New Dead Load = 200 PSF

WIND LOADS

Wind Load Calculations

- ASCE 7-05 Section 6.5 - Method 2 - Analytical Procedure

1. Occupancy Category (Table 1-1)

   II , All buildings except those in I, III, IV

2. Wind Load Importance Factor (Table 6-1, § 6.5.5)

   $I = 1.00$, for Category II Non-Hurricane prone

3. Basic Wind Speed (Figure 6-1)

   $V = 85$ mph

4. Wind Load Parameters

   a. Wind Directionality Factor, K_d (Table 6-4)

   $K_d = 0.85$

   b. Exposure Category (§ 6.5.6.3)

   Exposure C

   c. Topographic Factor, K_{zt} (Figure 6-4)

   No hill, $K_{zt} = 1.0$

d. Gust Effect Factor (§ 6.5.8)

i. Building Natural Frequency (§ C6.5.8)

- 26.9.2.1 Limitations for approx. natural frequency:

① $h = 198'\text{-}8'' < 300'$ ✓
② $4(115') = 460' > 198'\text{-}8''$ ✓

Limits are met.

- Approx. natural period for concrete shearwall systems:

$$n_1 = 385 (C_w)^{0.5} / H$$

$$C_w = \frac{100}{A_B} \sum_{i=1}^{n} \left(\frac{H}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]}$$

$H = 198.67$ ft
$A_B = (315')(123.67')$
$= 39{,}000$ SF

SW 4,6: $h_i = 198.67$ ft
$D_i = 30$ ft
$A_i = (30')(14''/12'') = 35\ ft^2$

$$\left(\frac{198.67}{198.67}\right)^2 \frac{35}{1 + 0.83\left(\frac{198.67}{30}\right)^2} = 5.60$$

SW S,R,K,J: $h_i = 174.34$ ft
$D_i = 30$ ft
$A_i = (30')(18''/12'') = 45\ ft^2$

$$\left(\frac{198.67}{174.34}\right)^2 \frac{(45)}{\left[1 + 0.83\left(\frac{174.34}{30}\right)^2\right]} = 2.01$$

SW O,N: $h_i = 198.67$ ft
$D_i = 20$ ft
$A_i = (20')(12''/12'') = 20\ ft^2$

$$\left(\frac{198.67}{198.67}\right)^2 \frac{20}{\left[1 + 0.83\left(\frac{198.67}{20}\right)^2\right]} = 0.241$$

SW 5 NORTH & 4 SOUTH:
$h_i = 198.67$ ft
$D_i = 17$ ft
$A_i = (17\text{ ft})(18''/12'') = 25.5\text{ ft}^2$

$$\left(\frac{198.67}{198.67}\right)^2 \frac{25.5}{\left[1+0.83\left(\frac{198.67}{17}\right)^2\right]} = 0.223$$

SW 5 SOUTH & 4 NORTH:
$h_i = 198.67$ ft
$D_i = 17$ ft
$A_i = (17\text{ ft})(26''/12'') = 36.83\text{ ft}^2$

$$\left(\frac{198.67}{198.67}\right)^2 \frac{36.83}{\left[1+0.83\left(\frac{198.67}{17}\right)^2\right]} = 0.322$$

SW 4 and 4.7:
$h_i = 174.34$ ft
$D_i = 30$ ft
$A_i = (30')(14''/12'') = 35\text{ ft}^2$

$$\left(\frac{198.67}{174.34}\right)^2 \frac{35}{\left[1+0.83\left(\frac{174.34}{30}\right)^2\right]} = 1.57$$

NORTH-SOUTH:

$$\Sigma\left(H/h_i\right)^2 \frac{A_i}{\left[1+0.83(h_i/D_i)^2\right]}$$

$$= 2(0.223) + 2(0.322) + 2(1.57) = 4.23$$

$$C_W = \frac{100}{39000\text{ SF}}(4.23) = 0.01085$$

$$n_{N\text{-}S} = 385(0.01085)^{0.5}/198.67$$

$$\boxed{n_{N\text{-}S} = 0.202\text{ Hz}}$$

EAST-WEST:

$$\Sigma\left(H/h_i\right)^2 \frac{A_i}{\left[1+0.83(h_i/D_i)^2\right]}$$

$$= 2(5.6) + 4(2.01) + 2(0.241) = 19.72$$

$$C_W = \frac{100}{39000\text{ SF}}(19.72) = 0.05057$$

$$n_{E\text{-}W} = 385(0.05057)^{0.5}/198.67$$

$$\boxed{n_{E\text{-}W} = 0.436\text{ Hz}}$$

∴ Flexible ($n_a < 1$ Hz) in both directions

ii. Flexible Structures (§6.5.8.2)

$$G_f = 0.925\left[\frac{1 + 1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right]$$

NORTH - South : $g_Q = g_v = 3.4$

$$\bar{z} = \max \begin{vmatrix} 0.6h = 0.6(198.67\text{ ft}) = 119.2\text{ ft} \\ z_{min} = 15\text{ ft} \end{vmatrix}$$

$\bar{z} = 119.2$ ft

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}, \quad c = 0.20$$

$$= 0.20\left(\frac{33}{119.2}\right)^{1/6}$$

$I_{\bar{z}} = 0.161$

$$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}, \quad \bar{\epsilon} = 1/5, \; l = 500\text{ ft}$$

$$= (500)\left(\frac{119.2}{33}\right)^{1/5}$$

$L_{\bar{z}} = 646.43$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right), \quad \bar{b} = 0.65, \; \bar{\alpha} = 1/6.5$$

$$= 0.65\left(\frac{119.2}{33}\right)^{1/6.5}(90)(88/60)$$

$\bar{V}_{\bar{z}} = 104.54$ ft/s

$$Q_{NS} = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}, \quad B = 115\text{ ft}, \; h = 198.67\text{ ft}$$

$$= \frac{1}{1 + 0.63\left(\frac{115 + 198.67}{646.43}\right)^{0.63}}$$

$Q_{NS} = 0.715$

North-South Gust Factor Cont.

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

$$= \sqrt{2 \ln(3600 \times 0.202)} + \frac{0.577}{\sqrt{2 \ln(3600 \times 0.202)}}$$

$$g_R = 3.79$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.202)(646.43)}{104.54}$$

$$N_1 = 1.25$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(1.25)}{(1 + 10.3(1.25))^{5/3}}$$

$$R_n = 0.117$$

$$\eta(R_h) = 4.6 n_1 h / \bar{V}_{\bar{z}} = 4.6(0.202)(198.67)/104.54$$
$$= 1.766$$

$$\eta(R_B) = 4.6 n_1 B / \bar{V}_{\bar{z}} = 4.6(0.202)(115)/104.54$$
$$= 1.022$$

$$\eta(R_L) = 15.4 n_1 L / \bar{V}_{\bar{z}} = 15.4(0.202)(279)/104.54$$
$$= 8.30$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta})$$

$$= \frac{1}{1.766} - \frac{1}{2(1.766)^2}(1 - e^{-2(1.766)})$$

$$R_h = 0.411$$

$$R_B = \frac{1}{1.022} - \frac{1}{2(1.022)^2}(1 - e^{-2(1.022)})$$

$$R_B = 0.562$$

North-South Gust Factor Cont.

$$R_L = \frac{1}{8.30} - \frac{1}{2(8.3)^2}(1-e^{-2(8.3)})$$

$R_L = 0.113$

β = damping ratio, unknown however from AE 538 most buildings have a β of 5-7%.

$\beta = 0.05$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$

$$= \sqrt{\frac{1}{0.05}(0.117)(0.411)(0.562)(0.53+0.47(0.113))}$$

$R = 0.561$

$$G_{f,N\text{-}S} = 0.925 \left[\frac{1 + 1.7(0.161)\sqrt{(3.4)^2(.715)^2 + (3.79)^2(0.561)^2}}{1 + 1.7(3.4)(0.161)} \right]$$

$G_{f,N\text{-}S} = 0.903$

EAST-WEST GUST FACTOR :

$\bar{z} = 119.2$ ft

$I_{\bar{z}} = 0.161$ ft

$L_{\bar{z}} = 646.43$

$\bar{V}_{\bar{z}} = 104.54$ ft/s

$$Q_{EW} = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

B = 279 ft
h = 198.67 ft

$$= \sqrt{\frac{1}{1+0.63\left(\frac{279+198.67}{646.43}\right)^{0.63}}}$$

$Q_{EW} = 0.811$

$$g_R = \sqrt{2\ln(3600 \times 0.436)} + \frac{0.577}{\sqrt{2\ln(3600 \times 0.436)}}$$

$g_e = 3.99$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.436)(646.43)}{104.54}$$

$N_1 = 2.70$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(2.7)}{(1+10.3(2.7))^{5/3}}$$

$R_n = 0.0745$

$\eta(R_B) = 4.6(0.436)(279) / 104.54$

$= 5.35$

$\eta(R_L) = 15.4(0.436)(115) / 104.54$

$= 7.39$

$\eta(R_h) = 4.6 n_1 h / \bar{V}_{\bar{z}} = 4.6(0.436)(198.67) / 104.54$

$= 3.81$

East west gust factor cont.

$$R_n = \frac{1}{3.81} - \frac{1}{2(3.81)^2}\left(1-e^{-2(3.81)}\right)$$

$R_n = 0.228$

$$R_B = \frac{1}{5.355} - \frac{1}{2(5.35)^2}\left(1-e^{-2(5.35)}\right)$$

$R_B = 0.169$

$$R_L = \frac{1}{7.39} - \frac{1}{2(7.39)^2}\left(1-e^{-2(7.39)}\right)$$

$R_L = 0.126$

$\beta = 0.05$ (for same reason as N/S direction)

$$R = \sqrt{\frac{1}{0.05}(0.0745)(0.228)(0.169)(0.53+0.47(0.126))}$$

$R = 0.184$

$$G_{f,E\text{-}W} = 0.925\left[\frac{1+1.7(0.161)\sqrt{(3.4)^2(0.811)^2+(3.99)^2(0.184)^2}}{1+1.7(3.4)(0.161)}\right]$$

$$\boxed{G_{f,EW} = 0.853}$$

e. Enclosure Classification (§6.5.9)

Enclosed (§6.2)

f. Internal pressure Coefficient (Fig. 6-5)

$GC_{pi} = \pm 0.18$

Interpolation for Roof Cp Values: (Fig. 6-6)

NORTH-SOUTH $h/L = 0.712$

0 to 99.34 ft:

-0.9	0.5
?	0.712
-1.3(0.8) = 1.04	1.0

(99.34)(115) = 11424.1 > 1000 → 0.8 reduction

$C_p = -1.2194$

99.34 ft to 198.67 ft:

-0.9	0.5
?	0.712
-0.7	1.0

$C_p = -0.8152$

198.67 ft to 279 ft:

-0.5	0.5
?	0.712
-0.7	1.0

$C_p = -0.5848$

EAST-WEST $h/L = 1.728 > 1.0$

0 to 99.34 ft:

$C_p = -1.3$

$A = (99.34)(279) = 27716\ ft^2 > 1000\ ft^2$

$C_p = -1.3(0.8) = \underline{-1.04}$

99.34 ft to 115 ft:

$C_p = -0.7$

Interpolation for Wall Cp Values: (Fig. 6-6)

NORTH-SOUTH

$L/B = 2.43$

-0.3	2
?	2.43
-0.2	4

$C_p = -0.279$

* No other Cp-values were interpolated

WIND LOADING CALCULATIONS

Equations Utilized:

$Kz = 2.01\ (z/z_g)^{2/\alpha}$

$q_z = 0.00256K_zK_{zt}K_dV^2I$

$p=qG_fC_p$ (MWFRS for Flexible Buildings)

Constants Previously Calculated by hand:

k_{zt} = 1.00
k_d = 0.85
V = 85.0
I = 1.00
$G_{f,\ NS}$= 0.903
$G_{f,\ EW}$= 0.853

Caclulating k_z and q_z - NS and SW

Floor Number	Height above ground (z)	z_g	α	k_z	q_z	q_h
2	15.00	900	9.5	0.85	13.36	22.99
3	28.17	900	9.5	0.97	15.24	22.99
4	41.34	900	9.5	1.05	16.52	22.99
5	54.51	900	9.5	1.11	17.51	22.99
6	67.68	900	9.5	1.17	18.33	22.99
7	80.85	900	9.5	1.21	19.03	22.99
8	94.02	900	9.5	1.25	19.64	22.99
9	107.19	900	9.5	1.28	20.19	22.99
10	120.36	900	9.5	1.32	20.69	22.99
11	133.53	900	9.5	1.35	21.15	22.99
12	146.70	900	9.5	1.37	21.57	22.99
13	159.87	900	9.5	1.40	21.96	22.99
Penthouse Floor	173.04	900	9.5	1.42	22.33	22.99
Penthouse Roof	198.67	900	9.5	1.46	22.99	22.99

Wall Pressures | NORTH-SOUTH DIRECTION

Wind Pressures | North-South Direction										
Floor Number	Height above ground (z)	q_z	q_h	Windward (PSF)	Leeward (PSF)	Trib Height	Trib Area (SF)	Force (k)	Story Shear (K)	Overturning Moment (ft-k)
Ground	0.00	13.36	22.99	9.65	-5.78	7.50	2092.50	32.30	421.44	0.00
2	15.00	13.36	22.99	9.65	-5.78	14.09	1619.78	25.00	389.14	375.03
3	28.17	15.24	22.99	11.01	-5.78	13.17	1514.55	25.43	364.14	716.38
4	41.34	16.52	22.99	11.93	-5.78	13.17	1514.55	26.83	338.71	1109.27
5	54.51	17.51	22.99	12.65	-5.78	13.17	1514.55	27.92	311.87	1521.73
6	67.68	18.33	22.99	13.24	-5.78	13.17	1514.55	28.81	283.96	1949.83
7	80.85	19.03	22.99	13.74	-5.78	13.17	1514.55	29.57	255.15	2391.09
8	94.02	19.64	22.99	14.19	-5.78	13.17	1514.55	30.25	225.57	2843.77
9	107.19	20.19	22.99	14.59	-5.78	13.17	1514.55	30.85	195.33	3306.58
10	120.36	20.69	22.99	14.95	-5.78	13.17	1514.55	31.39	164.48	3778.51
11	133.53	21.15	22.99	15.28	-5.78	13.17	1514.55	31.89	133.08	4258.76
12	146.70	21.57	22.99	15.58	-5.78	13.17	1514.55	32.36	101.19	4746.68
13	159.87	21.96	22.99	15.87	-5.78	13.84	1591.03	34.44	68.83	5506.40
Penthouse Floor	173.04	22.33	22.99	16.13	-5.78	19.42	1196.30	26.22	34.39	4536.56
Penthouse Roof	198.67	22.99	22.99	16.61	-5.78	12.17	365.10	8.17	8.17	1624.11

Base Shear [k] = 421

Total Overturning Moment [ft-k] = 38665

Windard Wall C_p= 0.800 L= 279.00

Leeward Wall C_p= -0.279 B= 115.00

(interpolate) L/B= 2.43

Wall Pressures | EAST-WEST DIRECTION

Wind Pressures | East-West Direction										
Floor Number	Height above ground (z)	q_z	q_h	Windward (PSF)	Leeward (PSF)	Trib Height	Trib Area (SF)	Force (k)	Story Shear (K)	Overturning Moment (ft-k)
1	0.00	13.36	22.99	9.12	-10.38	7.50	862.5	16.82	1166.73	0
2	15.00	13.36	22.99	9.12	-10.38	14.09	3929.7	76.63	1149.91	1149.43
3	28.17	15.24	22.99	10.40	-10.38	13.17	3674.4	76.35	1073.28	2150.90
4	41.34	16.52	22.99	11.27	-10.38	13.17	3674.4	79.57	996.93	3289.34
5	54.51	17.51	22.99	11.95	-10.38	13.17	3674.4	82.05	917.36	4472.62
6	67.68	18.33	22.99	12.51	-10.38	13.17	3674.4	84.10	835.31	5691.76
7	80.85	19.03	22.99	12.98	-10.38	13.17	3674.4	85.85	751.21	6941.05
8	94.02	19.64	22.99	13.40	-10.38	13.17	3674.4	87.39	665.36	8216.50
9	107.19	20.19	22.99	13.78	-10.38	13.17	3674.4	88.77	577.97	9515.16
10	120.36	20.69	22.99	14.12	-10.38	13.17	3674.4	90.02	489.20	10834.74
11	133.53	21.15	22.99	14.43	-10.38	13.17	3674.4	91.17	399.18	12173.40
12	146.70	21.57	22.99	14.72	-10.38	13.17	3674.4	92.23	308.01	13529.62
13	159.87	21.96	22.99	14.99	-10.38	13.84	3860.0	97.92	215.79	15654.63
Penthouse Floor	173.04	22.33	22.99	15.24	-10.38	19.42	3300.3	84.55	117.87	14631.08
Penthouse Roof	198.67	22.99	22.99	15.69	-10.38	12.17	1277.9	33.31	33.31	6618.37

Base Shear [k]= 1167

Total Overturning Moment [ft-k] = 114869

Windard Wall C_p= 0.800 L= 115.00

Leeward Wall C_p= -0.500 B= 279.00

Roof Wind Uplift | NORTH-SOUTH DIRECTION

Wind Pressures - Roof Uplift | North South					
Location on Roof	**Cp**	**G**	**q_h**	**Pressure [PSF]**	
0 to 99.34 ft	-1.2194	0.903	22.99	-25.32	<- Area Reduction Applies for Cp = 99.34*115=11424.1 ft^2
99.34 to 198.67 ft	-0.8152	0.903	22.99	-16.92	
198.67 to 279 ft	-0.5848	0.903	22.99	-12.14	

h= 198.67
L= 279
h/L= 0.712

NOTE: Interpolation between h/L=0.5 and h/L=1.0 can be seen on Page 18 of hand calculations. Also, Area reduction calculation can be seen on Page 18.

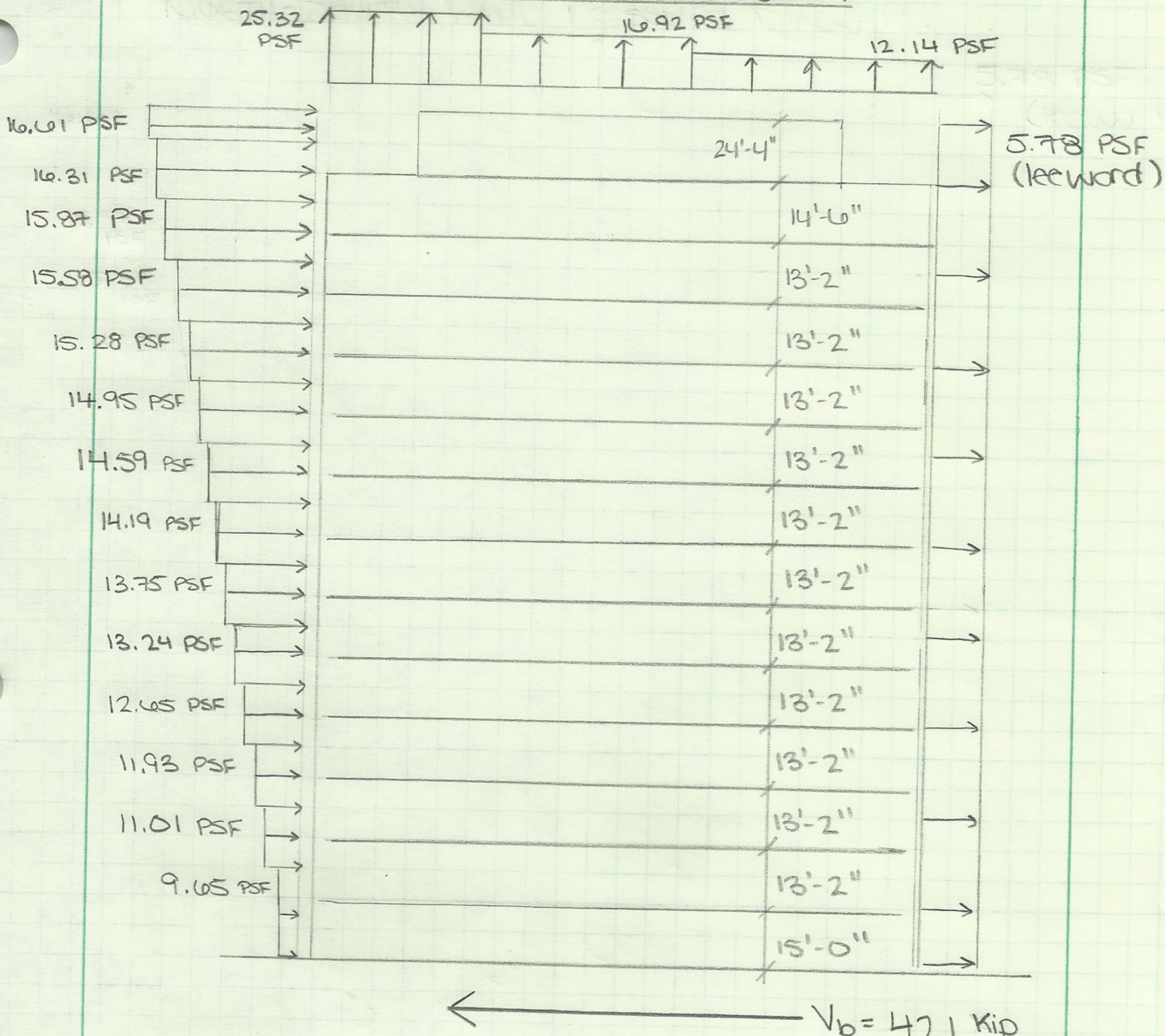
Roof Wind Uplift | EAST-WEST DIRECTION

Wind Pressures - Roof Uplift | East West					
Location on Roof	**Cp**	**G**	**q_h**	**Pressure [PSF]**	
0 to 99.34 ft:	-1.040	0.853	22.99	-20.40	<- Area Reduction Applies for Cp= 99.34*279=27716 ft^2
99.34 to 115 ft:	-0.700	0.853	22.99	-13.73	

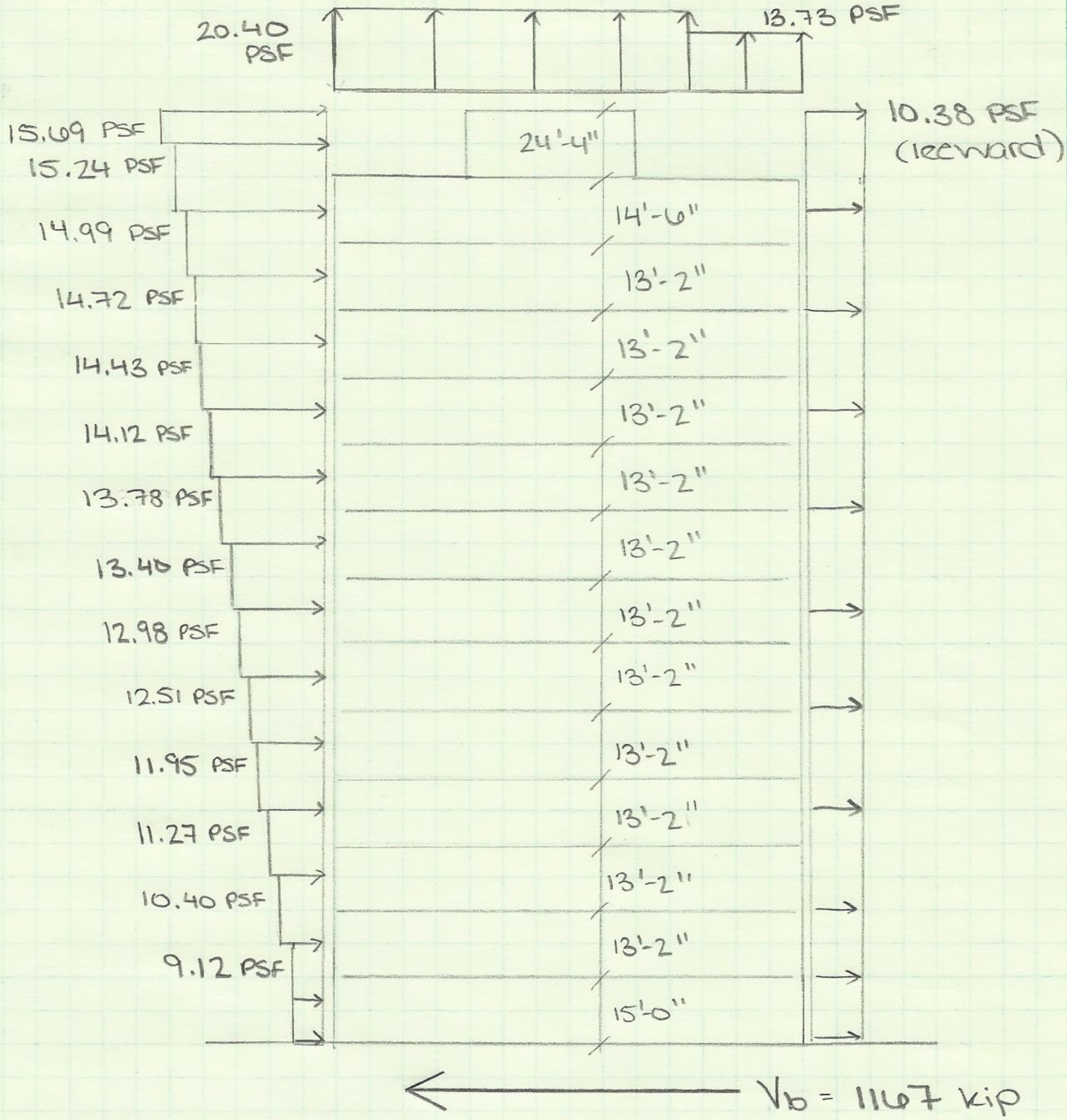
h= 198.67
L= 115
h/L= 1.728

NOTE: Area reduction application calculation can be seen on Page 19 of hand calculations.

NORTH-SOUTH WIND PRESSURE DIAGRAM



EAST-WEST WIND PRESSURE DIAGRAM



SEISMIC LOADS

SEISMIC LOAD CALCULATIONS

1. Building not exempt. (§ 11.1.2)
2. Design Spectral Response Acceleration (§11.4)
   a) Site Class Definition = C
   b) Acceleration Parameters

      $S_S = 1.418\,g$

      $S_1 = 0.527\,g$

   c) Site Class Effects (§11.4.3)

      $F_a = 1.0$
      $F_v = 1.3$

      $S_{MS} = (1.0)(1.418g) \rightarrow S_{MS} = 1.418g$

      $S_{M1} = (1.3)(0.527g) \rightarrow S_{M1} = 0.6851g$

   d) Determine Spectral Acceleration Parameters (§11.4.4)

      $S_{DS} = 2/3\, S_{MS}$
      $= 2/3\,(1.418g) \rightarrow S_{DS} = 0.9453$

      $S_{D1} = 2/3\, S_{M1}$
      $= 2/3\,(0.6851g) \rightarrow S_{D1} = 0.4567$

3. Find Seismic Design Category (§11.5 + §11.6)

   Occupancy Category = II

   $I = 1.0$

   $S_{DS} = 0.9453 \geq 0.5 \rightarrow SDC = D$

4. Analysis Procedure Selection

(Table 12.6-1)

Equivalent Lateral Force Procedure

5. Determine Response Modification Factor

B.5 - Special reinforced concrete shear walls

$R = 6$

6. Find Period (T) (§ 12.8.2)

$T_a = C_t h_n^x$

$C_t = 0.02$
$h_n = 198.17\text{ ft}$
$x = 0.75$

$T_a = 0.02(198.17)^{0.75} \rightarrow T_a = 1.056\text{ s}$

7. Determine, T_L (Fig. 22-15)

$T_L = 8\text{ s}$

8. Find C_s (seismic response coefficient) (§12.8.1.1)

$$C_s = \frac{S_{DS}}{(R/I)} = \frac{0.9453}{(6/1.0)} \rightarrow C_s = 0.1576$$

$T_a = 1.056\text{ s} < T_L = 8\text{ s}$

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.4561}{(1.056)(6/1.0)} = 0.0720$$

$$C_s = \min\begin{vmatrix} 0.1576 \\ 0.0720 \end{vmatrix} = 0.0720 > 0.01 \checkmark$$

$C_s = 0.0720$

9. Weight Calculation: See excel sheet Page. 28 of this document.

10. Calculate Base Shear (Eq. 12.8-1)

$$V = C_s W$$

$$= (0.0720)(88979\ k)$$

$$V = 6406.5\ k$$

11. Determine Story Forces

Find k:

0.5s	k=1
T=1.056s	k=?
2.5s	k=2

$$\frac{2-1}{2.5-0.5} = \frac{k-1}{1.056-0.5}$$

$$k = 1.278$$

* See excel sheet for the rest of the seismic force calculation. (Pg. 29 of this document) *

SIESMIC LOAD CALCULATIONS

Story Weights												
Floor Number	**Dead Load**	**Partition Load**	**Total Floor Weight (PSF)**	**Floor Area (ft^2)**	**Floor Weight (kip)**	**Wall Height (ft)**	**Shear Wall Length (ft)**	**Level Wall Thickness (ft)**	**Shear Wall Weight (kip)**	**Curtain Wall Length (ft)**	**Curtain Wall Weight (k)**	**Total Level Weight (kip)**
Penthouse Roof	133	0	133	6704	892	24.33	30	1.50	164	490	58	1114
Penthouse Floor	171	0	171	29703	5079	14.50	297	1.50	969	786	93	6141
13	215	20	235	29703	6980	13.17	297	2.17	1271	786	93	8344
12	215	20	235	29703	6980	13.17	297	2.17	1271	786	93	8344
11	215	20	235	29703	6980	13.17	297	2.17	1271	786	93	8344
10	215	20	235	29703	6980	13.17	297	2.17	1271	786	93	8344
9	215	20	235	29703	6980	13.17	297	2.17	1271	786	93	8344
8	215	20	235	29703	6980	13.17	297	2.17	1271	786	93	8344
7	215	20	235	29703	6980	13.17	297	2.17	1271	786	93	8344
6	215	20	235	29703	6980	13.17	297	2.17	1271	786	93	8344
5	215	20	235	29703	6980	13.17	297	2.17	1271	786	93	8344
4	215	20	235	29703	6980	13.17	297	2.17	1273	786	93	8346
3	215	20	235	29703	6980	13.17	297	2.17	1271	786	93	8344
2	215	20	235	26494	6226	15.00	297	2.33	1559	806	95	7880
				Total Weight =	**106923**							

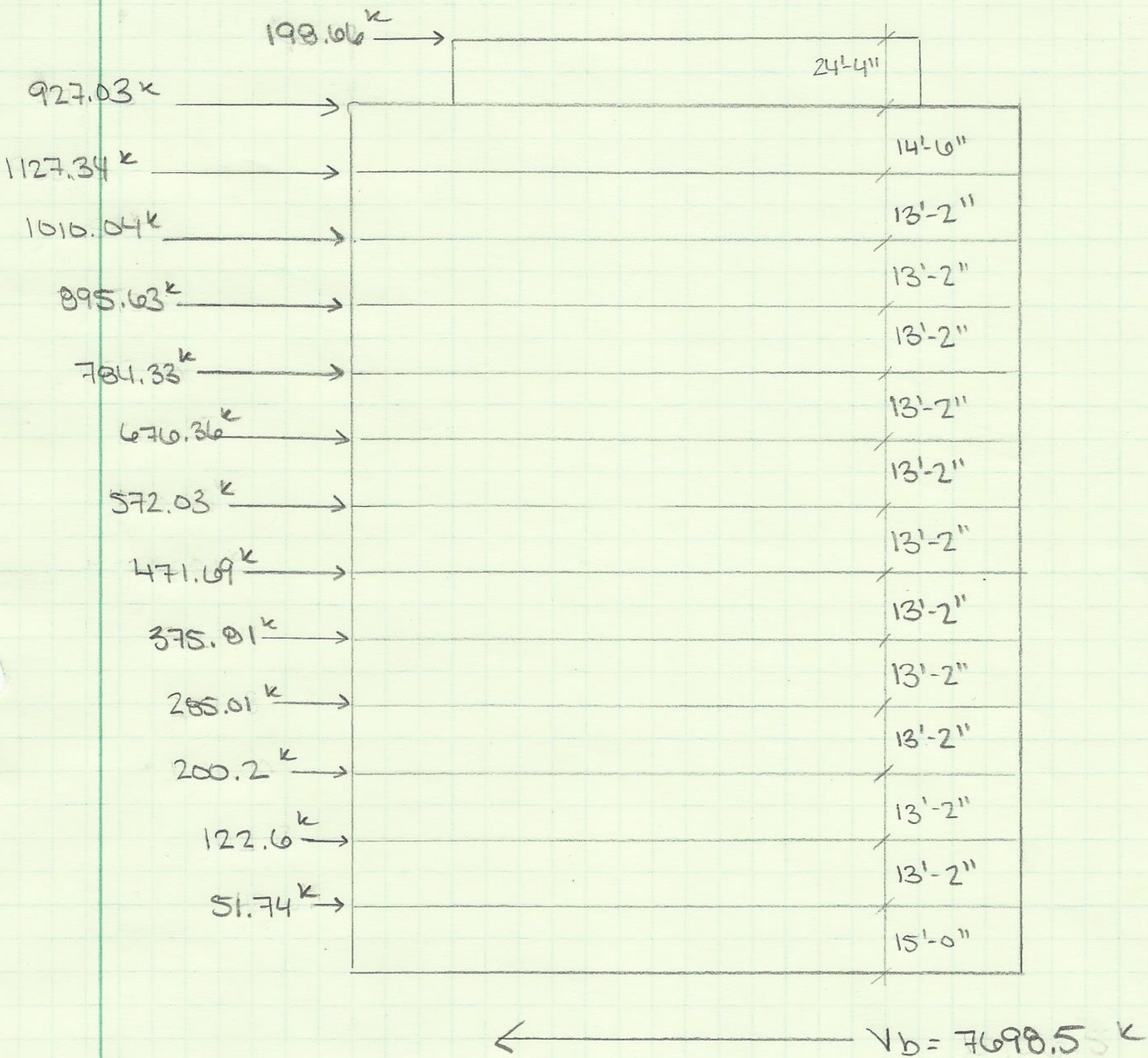
Seismic Story Forces

T= 1.056 s
k= 1.278
Vb= 7698.5 k

Story Forces | North-South						
Floor Number	**hi (ft)**	**h (ft)**	**W (kip)**	**$W*h^k$**	**Cvx**	**Story Forces Fi (kip)**
Penthouse Roof	24.33	198.70	1114	963512	0.0258	198.66
Penthouse Floor	14.50	174.37	6141	4496086	0.1204	927.03
13	13.17	159.87	8344	5467619	0.1464	1127.34
12	13.17	146.70	8344	4898710	0.1312	1010.04
11	13.17	133.53	8344	4343840	0.1163	895.63
10	13.17	120.36	8344	3803998	0.1019	784.33
9	13.17	107.19	8344	3280358	0.0879	676.36
8	13.17	94.02	8344	2774339	0.0743	572.03
7	13.17	80.85	8344	2287700	0.0613	471.69
6	13.17	67.68	8344	1822688	0.0488	375.81
5	13.17	54.51	8344	1382295	0.0370	285.01
4	13.17	41.34	8346	970973	0.0260	200.20
3	13.17	28.17	8344	594582	0.0159	122.59
2	15.00	15.00	7880	250955	0.0067	51.74
		SUM:	**106923**	**37337654**		**7698.46**

Base Shear [k] = 7698.5

Adjusted Siesmic LOAD vs. Height Diagram



APPENDIX B

Determine Worst Case Shear Wall

Determine Worst Case Shear Wall in Building: SWG – LL1

Determine Controlling Load Case and Wall Taking the Most Shear:

Story	Pier	Load Case/Combo	V2 kip	Maximum Shear and Controlling Load Case for Wall		
LL1	U - LL1	EX + EXT	647.549			
LL1	U - LL1	CASE 2 Y: Wy + Mz	309.532			
LL1	U - LL1	CASE 2 X: Wx + Mz	47.182			
LL1	U - LL1	CASE 1: Wx	5.526			
LL1	U - LL1	CASE 2 X: Wx - Mz	-47.182			
LL1	U - LL1	CASE 4: +Moments Add	-73.034			
LL1	U - LL1	CASE 3	-109.336			
LL1	U - LL1	CASE 4: +Moments Opposite	-141.727	Maximum Shear =	6405.868	
LL1	U - LL1	CASE 2 Y: Wy - Mz	-309.532	Load Case:	EY - EYT	
LL1	U - LL1	EX - EXT	-360.283			
LL1	U - LL1	CASE 4: -Moments Opposite	-526.937			
LL1	U - LL1	CASE 4: -Moments Add	-595.631			
LL1	U - LL1	CASE 1: Wy	-599.365			
LL1	U - LL1	EY + EYT	-3881.917			
LL1	U - LL1	EY - EYT	-6405.868			
LL1	S - LL1	EX + EXT	362.512			
LL1	S - LL1	EX - EXT	-202.392			
LL1	S - LL1	EY + EYT	-3041.741			
LL1	S - LL1	EY - EYT	-4456.348			
LL1	S - LL1	CASE 1: Wx	3.08			
LL1	S - LL1	CASE 1: Wy	-440.486	Maximum Shear =	4456.348	
LL1	S - LL1	CASE 2 X: Wx + Mz	26.527	Load Case:	EY - EYT	
LL1	S - LL1	CASE 2 X: Wx - Mz	-26.527			
LL1	S - LL1	CASE 2 Y: Wy - Mz	-174.007			
LL1	S - LL1	CASE 2 Y: Wy + Mz	174.007			
LL1	S - LL1	CASE 3	-81.096			
LL1	S - LL1	CASE 4: +Moments Add	-99.334			
LL1	S - LL1	CASE 4: -Moments Add	-393.186			
LL1	S - LL1	CASE 4: +Moments Opposite	-137.967			
LL1	S - LL1	CASE 4: -Moments Opposite	-354.553			
LL1	R, 5 - 4.2 - LL1	EX + EXT	-8.321			
LL1	R, 5 - 4.2 - LL1	EX - EXT	9.172			
LL1	R, 5 - 4.2 - LL1	EY + EYT	-53.348			
LL1	R, 5 - 4.2 - LL1	EY - EYT	-9.569			
LL1	R, 5 - 4.2 - LL1	CASE 1: Wx	0.016			
LL1	R, 5 - 4.2 - LL1	CASE 1: Wy	-3.738	Maximum Shear =	53.348	
LL1	R, 5 - 4.2 - LL1	CASE 2 X: Wx + Mz	-0.834	Load Case:	EY + EYT	
LL1	R, 5 - 4.2 - LL1	CASE 2 X: Wx - Mz	0.834			
LL1	R, 5 - 4.2 - LL1	CASE 2 Y: Wy - Mz	5.468			
LL1	R, 5 - 4.2 - LL1	CASE 2 Y: Wy + Mz	-5.468			
LL1	R, 5 - 4.2 - LL1	CASE 3	-0.708			
LL1	R, 5 - 4.2 - LL1	CASE 4: +Moments Add	-6.717			
LL1	R, 5 - 4.2 - LL1	CASE 4: -Moments Add	2.525			
LL1	R, 5 - 4.2 - LL1	CASE 4: +Moments Opposite	-5.503			
LL1	R, 5 - 4.2 - LL1	CASE 4: -Moments Opposite	1.312			
LL1	R, 4.1 - 4 - LL1	EX + EXT	-7.711			
LL1	R, 4.1 - 4 - LL1	EX - EXT	6.486			
LL1	R, 4.1 - 4 - LL1	EY + EYT	26.492			
LL1	R, 4.1 - 4 - LL1	EY - EYT	62.014			
LL1	R, 4.1 - 4 - LL1	CASE 1: Wx	-0.024			
LL1	R, 4.1 - 4 - LL1	CASE 1: Wy	5.448	Maximum Shear =	62.014	
LL1	R, 4.1 - 4 - LL1	CASE 2 X: Wx + Mz	-0.685	Load Case:	EY - EYT	
LL1	R, 4.1 - 4 - LL1	CASE 2 X: Wx - Mz	0.685			
LL1	R, 4.1 - 4 - LL1	CASE 2 Y: Wy - Mz	4.488			
LL1	R, 4.1 - 4 - LL1	CASE 2 Y: Wy + Mz	-4.488			
LL1	R, 4.1 - 4 - LL1	CASE 3	1.011			
LL1	R, 4.1 - 4 - LL1	CASE 4: +Moments Add	-0.743			
LL1	R, 4.1 - 4 - LL1	CASE 4: -Moments Add	6.851			
LL1	R, 4.1 - 4 - LL1	CASE 4: +Moments Opposite	0.256			
LL1	R, 4.1 - 4 - LL1	CASE 4: -Moments Opposite	5.852			
LL1	O - LL1	EX + EXT	7.732			
LL1	O - LL1	EX - EXT	-2.668			
LL1	O - LL1	EY + EYT	-245.878			
LL1	O - LL1	EY - EYT	-271.903			
LL1	O - LL1	CASE 1: Wx	0.093			
LL1	O - LL1	CASE 1: Wy	-31.173	Maximum Shear =	271.903	
LL1	O - LL1	CASE 2 X: Wx + Mz	0.502	Load Case:	EY - EYT	
LL1	O - LL1	CASE 2 X: Wx - Mz	-0.502			
LL1	O - LL1	CASE 2 Y: Wy - Mz	-3.29			
LL1	O - LL1	CASE 2 Y: Wy + Mz	3.29			
LL1	O - LL1	CASE 3	-5.85			
LL1	O - LL1	CASE 4: +Moments Add	-14.711			
LL1	O - LL1	CASE 4: -Moments Add	-20.284			
LL1	O - LL1	CASE 4: +Moments Opposite	-15.448			
LL1	O - LL1	CASE 4: -Moments Opposite	-19.548			
LL1	N - LL1	EX + EXT	5.058			

Critical Wall Conclusion	
Maximum Shear =	6444.163 kip
Load Case:	EY + EYT
Pier Label:	G - LL1

LL1	N - LL1	EX - EXT	-3.277		
LL1	N - LL1	EY + EYT	-226.233		
LL1	N - LL1	EY - EYT	-247.095		
LL1	N - LL1	CASE 1: Wx	0.027		
LL1	N - LL1	CASE 1: Wy	-28.503	Maximum Shear =	247.095
LL1	N - LL1	CASE 2 X: Wx + Mz	0.396	Load Case:	EY - EYT
LL1	N - LL1	CASE 2 X: Wx - Mz	-0.396		
LL1	N - LL1	CASE 2 Y: Wy - Mz	-2.595		
LL1	N - LL1	CASE 2 Y: Wy + Mz	2.595		
LL1	N - LL1	CASE 3	-5.393		
LL1	N - LL1	CASE 4: +Moments Add	-13.836		
LL1	N - LL1	CASE 4: -Moments Add	-18.228		
LL1	N - LL1	CASE 4: +Moments Opposite	-14.416		
LL1	N - LL1	CASE 4: -Moments Opposite	-17.648		
LL1	K - LL1	EX + EXT	-200.756		
LL1	K - LL1	EX - EXT	110.843		
LL1	K - LL1	EY + EYT	-3232.964		
LL1	K - LL1	EY - EYT	-2452.48		
LL1	K - LL1	CASE 1: Wx	-1.739		
LL1	K - LL1	CASE 1: Wy	-345.407	Maximum Shear =	3232.964
LL1	K - LL1	CASE 2 X: Wx + Mz	-14.56	Load Case:	EY + EYT
LL1	K - LL1	CASE 2 X: Wx - Mz	14.56		
LL1	K - LL1	CASE 2 Y: Wy - Mz	95.555		
LL1	K - LL1	CASE 2 Y: Wy + Mz	-95.555		
LL1	K - LL1	CASE 3	-66.681		
LL1	K - LL1	CASE 4: +Moments Add	-276.073		
LL1	K - LL1	CASE 4: -Moments Add	-114.814		
LL1	K - LL1	CASE 4: +Moments Opposite	-254.888		
LL1	K - LL1	CASE 4: -Moments Opposite	-135.999		
LL1	J - LL1	EX + EXT	-271.24		
LL1	J - LL1	EX - EXT	150.158		
LL1	J - LL1	EY + EYT	-3467.406		
LL1	J - LL1	EY - EYT	-2411.934		
LL1	J - LL1	CASE 1: Wx	-2.339		
LL1	J - LL1	CASE 1: Wy	-358.67		
LL1	J - LL1	CASE 2 X: Wx + Mz	-19.699	Maximum Shear =	3467.406
LL1	J - LL1	CASE 2 X: Wx - Mz	19.699	Load Case:	EY + EYT
LL1	J - LL1	CASE 2 Y: Wy - Mz	129.277		
LL1	J - LL1	CASE 2 Y: Wy + Mz	-129.277		
LL1	J - LL1	CASE 3	-69.639		
LL1	J - LL1	CASE 4: +Moments Add	-312.341		
LL1	J - LL1	CASE 4: -Moments Add	-94.155		
LL1	J - LL1	CASE 4: +Moments Opposite	-283.675		
LL1	J - LL1	CASE 4: -Moments Opposite	-122.822		
LL1	G - LL1	EX + EXT	-730.391		
LL1	G - LL1	EX - EXT	400.733		
LL1	G - LL1	EY + EYT	-6444.163		
LL1	G - LL1	EY - EYT	-3610.919		
LL1	G - LL1	CASE 1: Wx	-6.345		
LL1	G - LL1	CASE 1: Wy	-617.061		
LL1	G - LL1	CASE 2 X: Wx + Mz	-52.693	Maximum Shear =	6444.163
LL1	G - LL1	CASE 2 X: Wx - Mz	52.693	Load Case:	EY + EYT
LL1	G - LL1	CASE 2 Y: Wy - Mz	345.809		
LL1	G - LL1	CASE 2 Y: Wy + Mz	-345.809		
LL1	G - LL1	CASE 3	-121.508		
LL1	G - LL1	CASE 4: +Moments Add	-642.748		
LL1	G - LL1	CASE 4: -Moments Add	-59.209		
LL1	G - LL1	CASE 4: +Moments Opposite	-566.081		
LL1	G - LL1	CASE 4: -Moments Opposite	-135.876		
LL1	5, T - R - LL1	EX + EXT	443.401		
LL1	5, T - R - LL1	EX - EXT	427.168		
LL1	5, T - R - LL1	EY + EYT	33.49		
LL1	5, T - R - LL1	EY - EYT	-7.159		
LL1	5, T - R - LL1	CASE 1: Wx	18.121		
LL1	5, T - R - LL1	CASE 1: Wy	1.859	Maximum Shear =	443.401
LL1	5, T - R - LL1	CASE 2 X: Wx + Mz	0.765	Load Case:	EX + EXT
LL1	5, T - R - LL1	CASE 2 X: Wx - Mz	-0.765		
LL1	5, T - R - LL1	CASE 2 Y: Wy - Mz	-5.017		
LL1	5, T - R - LL1	CASE 2 Y: Wy + Mz	5.017		
LL1	5, T - R - LL1	CASE 3	13.945		
LL1	5, T - R - LL1	CASE 4: +Moments Add	15.484		
LL1	5, T - R - LL1	CASE 4: -Moments Add	7.013		
LL1	5, T - R - LL1	CASE 4: +Moments Opposite	14.371		
LL1	5, T - R - LL1	CASE 4: -Moments Opposite	8.126		
LL1	5, K - H - LL1	EX + EXT	316.909		
LL1	5, K - H - LL1	EX - EXT	306.823		
LL1	5, K - H - LL1	EY + EYT	20.822		
LL1	5, K - H - LL1	EY - EYT	-4.466		
LL1	5, K - H - LL1	CASE 1: Wx	11.879		
LL1	5, K - H - LL1	CASE 1: Wy	1.178		
LL1	5, K - H - LL1	CASE 2 X: Wx + Mz	0.459	Maximum Shear =	316.909
LL1	5, K - H - LL1	CASE 2 X: Wx - Mz	-0.459	Load Case:	EX + EXT
LL1	5, K - H - LL1	CASE 2 Y: Wy - Mz	-3.018		
LL1	5, K - H - LL1	CASE 2 Y: Wy + Mz	3.018		

LL1	5, K - H - LL1	CASE 3	9.134		
LL1	5, K - H - LL1	CASE 4: +Moments Add	9.891		
LL1	5, K - H - LL1	CASE 4: -Moments Add	4.812		
LL1	5, K - H - LL1	CASE 4: +Moments Opposite	9.225		
LL1	5, K - H - LL1	CASE 4: -Moments Opposite	5.477		
LL1	4.7, O - O.1 - LL1	EX + EXT	333.657		
LL1	4.7, O - O.1 - LL1	EX - EXT	330.843		
LL1	4.7, O - O.1 - LL1	EY + EYT	3.656		
LL1	4.7, O - O.1 - LL1	EY - EYT	-3.388		
LL1	4.7, O - O.1 - LL1	CASE 1: Wx	14.179		
LL1	4.7, O - O.1 - LL1	CASE 1: Wy	0.06		
LL1	4.7, O - O.1 - LL1	CASE 2 X: Wx + Mz	0.134	Maximum Shear =	333.657
LL1	4.7, O - O.1 - LL1	CASE 2 X: Wx - Mz	-0.134	Load Case:	EX + EXT
LL1	4.7, O - O.1 - LL1	CASE 2 Y: Wy - Mz	-0.879		
LL1	4.7, O - O.1 - LL1	CASE 2 Y: Wy + Mz	0.879		
LL1	4.7, O - O.1 - LL1	CASE 3	10.646		
LL1	4.7, O - O.1 - LL1	CASE 4: +Moments Add	8.76		
LL1	4.7, O - O.1 - LL1	CASE 4: -Moments Add	7.274		
LL1	4.7, O - O.1 - LL1	CASE 4: +Moments Opposite	8.564		
LL1	4.7, O - O.1 - LL1	CASE 4: -Moments Opposite	7.469		
LL1	4.7, O.2 - N - LL1	EX + EXT	333.657		
LL1	4.7, O.2 - N - LL1	EX - EXT	330.843		
LL1	4.7, O.2 - N - LL1	EY + EYT	3.656		
LL1	4.7, O.2 - N - LL1	EY - EYT	-3.388		
LL1	4.7, O.2 - N - LL1	CASE 1: Wx	14.179		
LL1	4.7, O.2 - N - LL1	CASE 1: Wy	0.06		
LL1	4.7, O.2 - N - LL1	CASE 2 X: Wx + Mz	0.134	Maximum Shear =	333.657
LL1	4.7, O.2 - N - LL1	CASE 2 X: Wx - Mz	-0.134	Load Case:	EX + EXT
LL1	4.7, O.2 - N - LL1	CASE 2 Y: Wy - Mz	-0.879		
LL1	4.7, O.2 - N - LL1	CASE 2 Y: Wy + Mz	0.879		
LL1	4.7, O.2 - N - LL1	CASE 3	10.646		
LL1	4.7, O.2 - N - LL1	CASE 4: +Moments Add	8.76		
LL1	4.7, O.2 - N - LL1	CASE 4: -Moments Add	7.274		
LL1	4.7, O.2 - N - LL1	CASE 4: +Moments Opposite	8.564		
LL1	4.7, O.2 - N - LL1	CASE 4: -Moments Opposite	7.469		
LL1	4, T - S - LL1	EX + EXT	58.298		
LL1	4, T - S - LL1	EX - EXT	61.95		
LL1	4, T - S - LL1	EY + EYT	-4.87		
LL1	4, T - S - LL1	EY - EYT	4.268		
LL1	4, T - S - LL1	CASE 1: Wx	2.99		
LL1	4, T - S - LL1	CASE 1: Wy	-0.088		
LL1	4, T - S - LL1	CASE 2 X: Wx + Mz	-0.178	Maximum Shear =	61.95
LL1	4, T - S - LL1	CASE 2 X: Wx - Mz	0.178	Load Case:	EX - EXT
LL1	4, T - S - LL1	CASE 2 Y: Wy - Mz	1.164		
LL1	4, T - S - LL1	CASE 2 Y: Wy + Mz	-1.164		
LL1	4, T - S - LL1	CASE 3	2.226		
LL1	4, T - S - LL1	CASE 4: +Moments Add	0.649		
LL1	4, T - S - LL1	CASE 4: -Moments Add	2.619		
LL1	4, T - S - LL1	CASE 4: +Moments Opposite	0.908		
LL1	4, T - S - LL1	CASE 4: -Moments Opposite	2.36		
LL1	4, R.1 - R - LL1	EX + EXT	58.298		
LL1	4, R.1 - R - LL1	EX - EXT	61.95		
LL1	4, R.1 - R - LL1	EY + EYT	-4.87		
LL1	4, R.1 - R - LL1	EY - EYT	4.268		
LL1	4, R.1 - R - LL1	CASE 1: Wx	2.99		
LL1	4, R.1 - R - LL1	CASE 1: Wy	-0.088		
LL1	4, R.1 - R - LL1	CASE 2 X: Wx + Mz	-0.178	Maximum Shear =	61.95
LL1	4, R.1 - R - LL1	CASE 2 X: Wx - Mz	0.178	Load Case:	EX - EXT
LL1	4, R.1 - R - LL1	CASE 2 Y: Wy - Mz	1.164		
LL1	4, R.1 - R - LL1	CASE 2 Y: Wy + Mz	-1.164		
LL1	4, R.1 - R - LL1	CASE 3	2.226		
LL1	4, R.1 - R - LL1	CASE 4: +Moments Add	0.649		
LL1	4, R.1 - R - LL1	CASE 4: -Moments Add	2.619		
LL1	4, R.1 - R - LL1	CASE 4: +Moments Opposite	0.908		
LL1	4, R.1 - R - LL1	CASE 4: -Moments Opposite	2.36		
LL1	4, O - O.1 - LL1	EX + EXT	326.256		
LL1	4, O - O.1 - LL1	EX - EXT	336.456		
LL1	4, O - O.1 - LL1	EY + EYT	-12.496		
LL1	4, O - O.1 - LL1	EY - EYT	13.04		
LL1	4, O - O.1 - LL1	CASE 1: Wx	14.147		
LL1	4, O - O.1 - LL1	CASE 1: Wy	-0.127		
LL1	4, O - O.1 - LL1	CASE 2 X: Wx + Mz	-0.484	Maximum Shear =	336.456
LL1	4, O - O.1 - LL1	CASE 2 X: Wx - Mz	0.484	Load Case:	EX - EXT
LL1	4, O - O.1 - LL1	CASE 2 Y: Wy - Mz	3.172		
LL1	4, O - O.1 - LL1	CASE 2 Y: Wy + Mz	-3.172		
LL1	4, O - O.1 - LL1	CASE 3	10.586		
LL1	4, O - O.1 - LL1	CASE 4: +Moments Add	5.214		
LL1	4, O - O.1 - LL1	CASE 4: -Moments Add	10.573		
LL1	4, O - O.1 - LL1	CASE 4: +Moments Opposite	5.918		
LL1	4, O - O.1 - LL1	CASE 4: -Moments Opposite	9.868		
LL1	4, O.2 - N - LL1	EX + EXT	326.256		
LL1	4, O.2 - N - LL1	EX - EXT	336.456		
LL1	4, O.2 - N - LL1	EY + EYT	-12.496		
LL1	4, O.2 - N - LL1	EY - EYT	13.04		

LL1	4, O.2 - N - LL1	CASE 1: Wx	14.147		
LL1	4, O.2 - N - LL1	CASE 1: Wy	-0.127		
LL1	4, O.2 - N - LL1	CASE 2 X: Wx + Mz	-0.484	Maximum Shear =	336.456
LL1	4, O.2 - N - LL1	CASE 2 X: Wx - Mz	0.484	Load Case:	EX - EXT
LL1	4, O.2 - N - LL1	CASE 2 Y: Wy - Mz	3.172		
LL1	4, O.2 - N - LL1	CASE 2 Y: Wy + Mz	-3.172		
LL1	4, O.2 - N - LL1	CASE 3	10.586		
LL1	4, O.2 - N - LL1	CASE 4: +Moments Add	5.214		
LL1	4, O.2 - N - LL1	CASE 4: -Moments Add	10.573		
LL1	4, O.2 - N - LL1	CASE 4: +Moments Opposite	5.918		
LL1	4, O.2 - N - LL1	CASE 4: -Moments Opposite	9.868		
LL1	4, K - H - LL1	EX + EXT	-740.307		
LL1	4, K - H - LL1	EX - EXT	-749.034		
LL1	4, K - H - LL1	EY + EYT	-13.796		
LL1	4, K - H - LL1	EY - EYT	-35.634		
LL1	4, K - H - LL1	CASE 1: Wx	-31.743		
LL1	4, K - H - LL1	CASE 1: Wy	-2.866	Maximum Shear =	749.034
LL1	4, K - H - LL1	CASE 2 X: Wx + Mz	0.423	Load Case:	EX - EXT
LL1	4, K - H - LL1	CASE 2 X: Wx - Mz	-0.423		
LL1	4, K - H - LL1	CASE 2 Y: Wy - Mz	-2.774		
LL1	4, K - H - LL1	CASE 2 Y: Wy + Mz	2.774		
LL1	4, K - H - LL1	CASE 3	-24.353		
LL1	4, K - H - LL1	CASE 4: +Moments Add	-17.137		
LL1	4, K - H - LL1	CASE 4: -Moments Add	-21.834		
LL1	4, K - H - LL1	CASE 4: +Moments Opposite	-17.756		
LL1	4, K - H - LL1	CASE 4: -Moments Opposite	-21.215		

Determine Worst Case Shear Wall at Level 2: SW 5, T-R

Determine Controlling Load Case and Wall Taking the Most Shear:

Story	Pier	Load Case/Combo	V2 kip	Maximum Shear and Controlling Load Case for Wall
Level 2	U - Level 2	EX + EXT	-17.313	
Level 2	U - Level 2	EX - EXT	186.879	
Level 2	U - Level 2	EY + EYT	333.523	
Level 2	U - Level 2	EY - EYT	840.303	
Level 2	U - Level 2	CASE 1: Wx	3.224	
Level 2	U - Level 2	CASE 1: Wy	117.718	
Level 2	U - Level 2	CASE 2 X: Wx + Mz	-13.71	
Level 2	U - Level 2	CASE 2 X: Wx - Mz	13.71	Maximum Shear = 840.303
Level 2	U - Level 2	CASE 2 Y: Wy - Mz	89.209	Load Case: EY - EYT
Level 2	U - Level 2	CASE 2 Y: Wy + Mz	-89.209	
Level 2	U - Level 2	CASE 3	24.993	
Level 2	U - Level 2	CASE 4: +Moments Add	-8.744	
Level 2	U - Level 2	CASE 4: -Moments Add	144.926	
Level 2	U - Level 2	CASE 4: +Moments Opposite	11.7	
Level 2	U - Level 2	CASE 4: -Moments Opposite	124.481	
Level 2	S - Level 2	EX + EXT	-92.503	
Level 2	S - Level 2	EX - EXT	155.851	
Level 2	S - Level 2	EY + EYT	873.984	
Level 2	S - Level 2	EY - EYT	1493.339	
Level 2	S - Level 2	CASE 1: Wx	1.211	
Level 2	S - Level 2	CASE 1: Wy	175.636	Maximum Shear = 1493.339
Level 2	S - Level 2	CASE 2 X: Wx + Mz	-14.032	Load Case: EY - EYT
Level 2	S - Level 2	CASE 2 X: Wx - Mz	14.032	
Level 2	S - Level 2	CASE 2 Y: Wy - Mz	91.651	
Level 2	S - Level 2	CASE 2 Y: Wy + Mz	-91.651	
Level 2	S - Level 2	CASE 3	34.364	
Level 2	S - Level 2	CASE 4: +Moments Add	21.322	
Level 2	S - Level 2	CASE 4: -Moments Add	177.808	
Level 2	S - Level 2	CASE 4: +Moments Opposite	42.029	
Level 2	S - Level 2	CASE 4: -Moments Opposite	157.101	
Level 2	R, 5 - 4.2 - Level 2	EX + EXT	-80.001	
Level 2	R, 5 - 4.2 - Level 2	EX - EXT	64.659	
Level 2	R, 5 - 4.2 - Level 2	EY + EYT	704.361	
Level 2	R, 5 - 4.2 - Level 2	EY - EYT	1066.076	
Level 2	R, 5 - 4.2 - Level 2	CASE 1: Wx	-0.283	
Level 2	R, 5 - 4.2 - Level 2	CASE 1: Wy	112.447	Maximum Shear = 1066.076
Level 2	R, 5 - 4.2 - Level 2	CASE 2 X: Wx + Mz	-7.258	Load Case: EY + EYT
Level 2	R, 5 - 4.2 - Level 2	CASE 2 X: Wx - Mz	7.258	
Level 2	R, 5 - 4.2 - Level 2	CASE 2 Y: Wy - Mz	47.531	
Level 2	R, 5 - 4.2 - Level 2	CASE 2 Y: Wy + Mz	-47.531	
Level 2	R, 5 - 4.2 - Level 2	CASE 3	21.115	
Level 2	R, 5 - 4.2 - Level 2	CASE 4: +Moments Add	22.842	
Level 2	R, 5 - 4.2 - Level 2	CASE 4: -Moments Add	103.454	
Level 2	R, 5 - 4.2 - Level 2	CASE 4: +Moments Opposite	33.466	
Level 2	R, 5 - 4.2 - Level 2	CASE 4: -Moments Opposite	92.831	
Level 2	R, 4.1 - 4 - Level 2	EX + EXT	-29.408	
Level 2	R, 4.1 - 4 - Level 2	EX - EXT	23.562	
Level 2	R, 4.1 - 4 - Level 2	EY + EYT	260.56	
Level 2	R, 4.1 - 4 - Level 2	EY - EYT	392.973	
Level 2	R, 4.1 - 4 - Level 2	CASE 1: Wx	-0.11	
Level 2	R, 4.1 - 4 - Level 2	CASE 1: Wy	41.593	Maximum Shear = 392.973
Level 2	R, 4.1 - 4 - Level 2	CASE 2 X: Wx + Mz	-2.662	Load Case: EY - EYT
Level 2	R, 4.1 - 4 - Level 2	CASE 2 X: Wx - Mz	2.662	
Level 2	R, 4.1 - 4 - Level 2	CASE 2 Y: Wy - Mz	17.423	
Level 2	R, 4.1 - 4 - Level 2	CASE 2 Y: Wy + Mz	-17.423	
Level 2	R, 4.1 - 4 - Level 2	CASE 3	7.813	
Level 2	R, 4.1 - 4 - Level 2	CASE 4: +Moments Add	8.571	
Level 2	R, 4.1 - 4 - Level 2	CASE 4: -Moments Add	38.139	
Level 2	R, 4.1 - 4 - Level 2	CASE 4: +Moments Opposite	12.47	
Level 2	R, 4.1 - 4 - Level 2	CASE 4: -Moments Opposite	34.24	
Level 2	O - Level 2	EX + EXT	-36.923	
Level 2	O - Level 2	EX - EXT	31.463	
Level 2	O - Level 2	EY + EYT	913.666	
Level 2	O - Level 2	EY - EYT	1084.671	
Level 2	O - Level 2	CASE 1: Wx	-0.1	
Level 2	O - Level 2	CASE 1: Wy	128.81	Maximum Shear = 1084.671
Level 2	O - Level 2	CASE 2 X: Wx + Mz	-3.449	Load Case: EY - EYT
Level 2	O - Level 2	CASE 2 X: Wx - Mz	3.449	
Level 2	O - Level 2	CASE 2 Y: Wy - Mz	22.591	
Level 2	O - Level 2	CASE 2 Y: Wy + Mz	-22.591	
Level 2	O - Level 2	CASE 3	24.339	
Level 2	O - Level 2	CASE 4: +Moments Add	53.307	
Level 2	O - Level 2	CASE 4: -Moments Add	91.62	
Level 2	O - Level 2	CASE 4: +Moments Opposite	58.355	
Level 2	O - Level 2	CASE 4: -Moments Opposite	86.572	
Level 2	N - Level 2	EX + EXT	21.706	
Level 2	N - Level 2	EX - EXT	-16.072	

Level 2	N - Level 2	EY + EYT	986.55		
Level 2	N - Level 2	EY - EYT	892.023		
Level 2	N - Level 2	CASE 1: Wx	0.098		
Level 2	N - Level 2	CASE 1: Wy	122.873	Maximum Shear =	986.55
Level 2	N - Level 2	CASE 2 X: Wx + Mz	1.871	Load Case:	EY + EYT
Level 2	N - Level 2	CASE 2 X: Wx - Mz	-1.871		
Level 2	N - Level 2	CASE 2 Y: Wy - Mz	-12.268		
Level 2	N - Level 2	CASE 2 Y: Wy + Mz	12.268		
Level 2	N - Level 2	CASE 3	23.36		
Level 2	N - Level 2	CASE 4: +Moments Add	79.62		
Level 2	N - Level 2	CASE 4: -Moments Add	58.846		
Level 2	N - Level 2	CASE 4: +Moments Opposite	76.885		
Level 2	N - Level 2	CASE 4: -Moments Opposite	61.58		
Level 2	K - Level 2	EX + EXT	83.163		
Level 2	K - Level 2	EX - EXT	-104.593		
Level 2	K - Level 2	EY + EYT	1376.97		
Level 2	K - Level 2	EY - EYT	908.21		
Level 2	K - Level 2	CASE 1: Wx	-0.418		
Level 2	K - Level 2	CASE 1: Wy	168.452	Maximum Shear =	1376.97
Level 2	K - Level 2	CASE 2 X: Wx + Mz	10.135	Load Case:	EY + EYT
Level 2	K - Level 2	CASE 2 X: Wx - Mz	-10.135		
Level 2	K - Level 2	CASE 2 Y: Wy - Mz	-66.27		
Level 2	K - Level 2	CASE 2 Y: Wy + Mz	66.27		
Level 2	K - Level 2	CASE 3	31.724		
Level 2	K - Level 2	CASE 4: +Moments Add	151.03		
Level 2	K - Level 2	CASE 4: -Moments Add	38.175		
Level 2	K - Level 2	CASE 4: +Moments Opposite	136.122		
Level 2	K - Level 2	CASE 4: -Moments Opposite	53.084		
Level 2	J - Level 2	EX + EXT	78.113		
Level 2	J - Level 2	EX - EXT	-125.775		
Level 2	J - Level 2	EY + EYT	1123.856		
Level 2	J - Level 2	EY - EYT	615.33		
Level 2	J - Level 2	CASE 1: Wx	-0.922		
Level 2	J - Level 2	CASE 1: Wy	139.199		
Level 2	J - Level 2	CASE 2 X: Wx + Mz	11.461	Maximum Shear =	1123.856
Level 2	J - Level 2	CASE 2 X: Wx - Mz	-11.461	Load Case:	EY + EYT
Level 2	J - Level 2	CASE 2 Y: Wy - Mz	-74.874		
Level 2	J - Level 2	CASE 2 Y: Wy + Mz	74.874		
Level 2	J - Level 2	CASE 3	25.821		
Level 2	J - Level 2	CASE 4: +Moments Add	141.75		
Level 2	J - Level 2	CASE 4: -Moments Add	13.949		
Level 2	J - Level 2	CASE 4: +Moments Opposite	124.844		
Level 2	J - Level 2	CASE 4: -Moments Opposite	30.856		
Level 2	G - Level 2	EX + EXT	73.165		
Level 2	G - Level 2	EX - EXT	-215.976		
Level 2	G - Level 2	EY + EYT	1124.99		
Level 2	G - Level 2	EY - EYT	405.536		
Level 2	G - Level 2	CASE 1: Wx	-2.698		
Level 2	G - Level 2	CASE 1: Wy	143.181		
Level 2	G - Level 2	CASE 2 X: Wx + Mz	17.644	Maximum Shear =	1124.99
Level 2	G - Level 2	CASE 2 X: Wx - Mz	-17.644	Load Case:	EY + EYT
Level 2	G - Level 2	CASE 2 Y: Wy - Mz	-114.994		
Level 2	G - Level 2	CASE 2 Y: Wy + Mz	114.994		
Level 2	G - Level 2	CASE 3	25.359		
Level 2	G - Level 2	CASE 4: +Moments Add	177.701		
Level 2	G - Level 2	CASE 4: -Moments Add	-19.517		
Level 2	G - Level 2	CASE 4: +Moments Opposite	151.529		
Level 2	G - Level 2	CASE 4: -Moments Opposite	6.655		
Level 2	5, T - R - Level 2	EX + EXT	2709.65		
Level 2	5, T - R - Level 2	EX - EXT	2564.744		
Level 2	5, T - R - Level 2	EY + EYT	97.517		
Level 2	5, T - R - Level 2	EY - EYT	-265.054		
Level 2	5, T - R - Level 2	CASE 1: Wx	127.959		
Level 2	5, T - R - Level 2	CASE 1: Wy	-8.18	Maximum Shear =	2709.65
Level 2	5, T - R - Level 2	CASE 2 X: Wx + Mz	7.128	Load Case:	EX + EXT
Level 2	5, T - R - Level 2	CASE 2 X: Wx - Mz	-7.128		
Level 2	5, T - R - Level 2	CASE 2 Y: Wy - Mz	-46.728		
Level 2	5, T - R - Level 2	CASE 2 Y: Wy + Mz	46.728		
Level 2	5, T - R - Level 2	CASE 3	94.419		
Level 2	5, T - R - Level 2	CASE 4: +Moments Add	106.996		
Level 2	5, T - R - Level 2	CASE 4: -Moments Add	27.879		
Level 2	5, T - R - Level 2	CASE 4: +Moments Opposite	96.582		
Level 2	5, T - R - Level 2	CASE 4: -Moments Opposite	38.293		
Level 2	5, K - K.1 - Level 2	EX + EXT	419.095		
Level 2	5, K - K.1 - Level 2	EX - EXT	396.294		
Level 2	5, K - K.1 - Level 2	EY + EYT	15.08		
Level 2	5, K - K.1 - Level 2	EY - EYT	-41.928		
Level 2	5, K - K.1 - Level 2	CASE 1: Wx	21.117		
Level 2	5, K - K.1 - Level 2	CASE 1: Wy	-1.343		
Level 2	5, K - K.1 - Level 2	CASE 2 X: Wx + Mz	1.14	Maximum Shear =	419.095
Level 2	5, K - K.1 - Level 2	CASE 2 X: Wx - Mz	-1.14	Load Case:	EX + EXT
Level 2	5, K - K.1 - Level 2	CASE 2 Y: Wy - Mz	-7.464		
Level 2	5, K - K.1 - Level 2	CASE 2 Y: Wy + Mz	7.464		
Level 2	5, K - K.1 - Level 2	CASE 3	15.582		
Level 2	5, K - K.1 - Level 2	CASE 4: +Moments Add	17.463		
Level 2	5, K - K.1 - Level 2	CASE 4: -Moments Add	4.803		

Level 2	5, K - K.1 - Level 2	CASE 4: +Moments Opposite	15.794		
Level 2	5, K - K.1 - Level 2	CASE 4: -Moments Opposite	6.472		
Level 2	5, J - H - Level 2	EX + EXT	419.137		
Level 2	5, J - H - Level 2	EX - EXT	396.333		
Level 2	5, J - H - Level 2	EY + EYT	15.082		
Level 2	5, J - H - Level 2	EY - EYT	-41.933		
Level 2	5, J - H - Level 2	CASE 1: Wx	21.119		
Level 2	5, J - H - Level 2	CASE 1: Wy	-1.343		
Level 2	5, J - H - Level 2	CASE 2 X: Wx + Mz	1.14	Maximum Shear =	419.137
Level 2	5, J - H - Level 2	CASE 2 X: Wx - Mz	-1.14	Load Case:	EX + EXT
Level 2	5, J - H - Level 2	CASE 2 Y: Wy - Mz	-7.465		
Level 2	5, J - H - Level 2	CASE 2 Y: Wy + Mz	7.465		
Level 2	5, J - H - Level 2	CASE 3	15.584		
Level 2	5, J - H - Level 2	CASE 4: +Moments Add	17.465		
Level 2	5, J - H - Level 2	CASE 4: -Moments Add	4.803		
Level 2	5, J - H - Level 2	CASE 4: +Moments Opposite	15.795		
Level 2	5, J - H - Level 2	CASE 4: -Moments Opposite	6.472		
Level 2	4.7, O - O.1 - Level 2	EX + EXT	691.99		
Level 2	4.7, O - O.1 - Level 2	EX - EXT	681.232		
Level 2	4.7, O - O.1 - Level 2	EY + EYT	7.392		
Level 2	4.7, O - O.1 - Level 2	EY - EYT	-19.519		
Level 2	4.7, O - O.1 - Level 2	CASE 1: Wx	33.331		
Level 2	4.7, O - O.1 - Level 2	CASE 1: Wy	-0.592		
Level 2	4.7, O - O.1 - Level 2	CASE 2 X: Wx + Mz	0.532	Maximum Shear =	691.99
Level 2	4.7, O - O.1 - Level 2	CASE 2 X: Wx - Mz	-0.532	Load Case:	EX + EXT
Level 2	4.7, O - O.1 - Level 2	CASE 2 Y: Wy - Mz	-3.487		
Level 2	4.7, O - O.1 - Level 2	CASE 2 Y: Wy + Mz	3.487		
Level 2	4.7, O - O.1 - Level 2	CASE 3	24.886		
Level 2	4.7, O - O.1 - Level 2	CASE 4: +Moments Add	21.386		
Level 2	4.7, O - O.1 - Level 2	CASE 4: -Moments Add	15.478		
Level 2	4.7, O - O.1 - Level 2	CASE 4: +Moments Opposite	20.608		
Level 2	4.7, O - O.1 - Level 2	CASE 4: -Moments Opposite	16.256		
Level 2	4.7, O.2 - N - Level 2	EX + EXT	691.99		
Level 2	4.7, O.2 - N - Level 2	EX - EXT	681.232		
Level 2	4.7, O.2 - N - Level 2	EY + EYT	7.392		
Level 2	4.7, O.2 - N - Level 2	EY - EYT	-19.519		
Level 2	4.7, O.2 - N - Level 2	CASE 1: Wx	33.331		
Level 2	4.7, O.2 - N - Level 2	CASE 1: Wy	-0.592		
Level 2	4.7, O.2 - N - Level 2	CASE 2 X: Wx + Mz	0.532	Maximum Shear =	691.99
Level 2	4.7, O.2 - N - Level 2	CASE 2 X: Wx - Mz	-0.532	Load Case:	EX - EXT
Level 2	4.7, O.2 - N - Level 2	CASE 2 Y: Wy - Mz	-3.487		
Level 2	4.7, O.2 - N - Level 2	CASE 2 Y: Wy + Mz	3.487		
Level 2	4.7, O.2 - N - Level 2	CASE 3	24.886		
Level 2	4.7, O.2 - N - Level 2	CASE 4: +Moments Add	21.386		
Level 2	4.7, O.2 - N - Level 2	CASE 4: -Moments Add	15.478		
Level 2	4.7, O.2 - N - Level 2	CASE 4: +Moments Opposite	20.608		
Level 2	4.7, O.2 - N - Level 2	CASE 4: -Moments Opposite	16.256		
Level 2	4, T - S - Level 2	EX + EXT	-6.903		
Level 2	4, T - S - Level 2	EX - EXT	8.212		
Level 2	4, T - S - Level 2	EY + EYT	-10.208		
Level 2	4, T - S - Level 2	EY - EYT	27.571		
Level 2	4, T - S - Level 2	CASE 1: Wx	4.08		
Level 2	4, T - S - Level 2	CASE 1: Wy	0.88		
Level 2	4, T - S - Level 2	CASE 2 X: Wx + Mz	-0.768	Maximum Shear =	27.571
Level 2	4, T - S - Level 2	CASE 2 X: Wx - Mz	0.768	Load Case:	EY - EYT
Level 2	4, T - S - Level 2	CASE 2 Y: Wy - Mz	5.025		
Level 2	4, T - S - Level 2	CASE 2 Y: Wy + Mz	-5.025		
Level 2	4, T - S - Level 2	CASE 3	3.227		
Level 2	4, T - S - Level 2	CASE 4: +Moments Add	-1.474		
Level 2	4, T - S - Level 2	CASE 4: -Moments Add	7.059		
Level 2	4, T - S - Level 2	CASE 4: +Moments Opposite	-0.348		
Level 2	4, T - S - Level 2	CASE 4: -Moments Opposite	5.933		
Level 2	4, R.1 - R - Level 2	EX + EXT	-6.903		
Level 2	4, R.1 - R - Level 2	EX - EXT	8.212		
Level 2	4, R.1 - R - Level 2	EY + EYT	-10.208		
Level 2	4, R.1 - R - Level 2	EY - EYT	27.571		
Level 2	4, R.1 - R - Level 2	CASE 1: Wx	4.08		
Level 2	4, R.1 - R - Level 2	CASE 1: Wy	0.88		
Level 2	4, R.1 - R - Level 2	CASE 2 X: Wx + Mz	-0.768	Maximum Shear =	27.571
Level 2	4, R.1 - R - Level 2	CASE 2 X: Wx - Mz	0.768	Load Case:	EY - EYT
Level 2	4, R.1 - R - Level 2	CASE 2 Y: Wy - Mz	5.025		
Level 2	4, R.1 - R - Level 2	CASE 2 Y: Wy + Mz	-5.025		
Level 2	4, R.1 - R - Level 2	CASE 3	3.227		
Level 2	4, R.1 - R - Level 2	CASE 4: +Moments Add	-1.474		
Level 2	4, R.1 - R - Level 2	CASE 4: -Moments Add	7.059		
Level 2	4, R.1 - R - Level 2	CASE 4: +Moments Opposite	-0.348		
Level 2	4, R.1 - R - Level 2	CASE 4: -Moments Opposite	5.933		
Level 2	4, O - O.1 - Level 2	EX + EXT	664.01		
Level 2	4, O - O.1 - Level 2	EX - EXT	701.412		
Level 2	4, O - O.1 - Level 2	EY + EYT	-25.383		
Level 2	4, O - O.1 - Level 2	EY - EYT	68.197		
Level 2	4, O - O.1 - Level 2	CASE 1: Wx	33.186		
Level 2	4, O - O.1 - Level 2	CASE 1: Wy	2.078		
Level 2	4, O - O.1 - Level 2	CASE 2 X: Wx + Mz	-1.834	Maximum Shear =	701.412
Level 2	4, O - O.1 - Level 2	CASE 2 X: Wx - Mz	1.834	Load Case:	EX - EXT
Level 2	4, O - O.1 - Level 2	CASE 2 Y: Wy - Mz	12.022		

Level 2	4, O - O.1 - Level 2	CASE 2 Y: Wy + Mz	-12.022		
Level 2	4, O - O.1 - Level 2	CASE 3	25.285		
Level 2	4, O - O.1 - Level 2	CASE 4: +Moments Add	9.676		
Level 2	4, O - O.1 - Level 2	CASE 4: -Moments Add	30.032		
Level 2	4, O - O.1 - Level 2	CASE 4: +Moments Opposite	12.356		
Level 2	4, O - O.1 - Level 2	CASE 4: -Moments Opposite	27.353		
Level 2	4, O.2 - N - Level 2	EX + EXT	664.01		
Level 2	4, O.2 - N - Level 2	EX - EXT	701.412		
Level 2	4, O.2 - N - Level 2	EY + EYT	-25.383		
Level 2	4, O.2 - N - Level 2	EY - EYT	68.197		
Level 2	4, O.2 - N - Level 2	CASE 1: Wx	33.186		
Level 2	4, O.2 - N - Level 2	CASE 1: Wy	2.078	Maximum Shear =	701.412
Level 2	4, O.2 - N - Level 2	CASE 2 X: Wx + Mz	-1.834	Load Case:	EX - EXT
Level 2	4, O.2 - N - Level 2	CASE 2 X: Wx - Mz	1.834		
Level 2	4, O.2 - N - Level 2	CASE 2 Y: Wy - Mz	12.022		
Level 2	4, O.2 - N - Level 2	CASE 2 Y: Wy + Mz	-12.022		
Level 2	4, O.2 - N - Level 2	CASE 3	25.285		
Level 2	4, O.2 - N - Level 2	CASE 4: +Moments Add	9.676		
Level 2	4, O.2 - N - Level 2	CASE 4: -Moments Add	30.032		
Level 2	4, O.2 - N - Level 2	CASE 4: +Moments Opposite	12.356		
Level 2	4, O.2 - N - Level 2	CASE 4: -Moments Opposite	27.353		
Level 2	4, K - H - Level 2	EX + EXT	1452.384		
Level 2	4, K - H - Level 2	EX - EXT	1559.376		
Level 2	4, K - H - Level 2	EY + EYT	-71.283		
Level 2	4, K - H - Level 2	EY - EYT	196.419		
Level 2	4, K - H - Level 2	CASE 1: Wx	77.742	Maximum Shear =	1559.376
Level 2	4, K - H - Level 2	CASE 1: Wy	6.134	Load Case:	EX - EXT
Level 2	4, K - H - Level 2	CASE 2 X: Wx + Mz	-5.268		
Level 2	4, K - H - Level 2	CASE 2 X: Wx - Mz	5.268		
Level 2	4, K - H - Level 2	CASE 2 Y: Wy - Mz	34.535		
Level 2	4, K - H - Level 2	CASE 2 Y: Wy + Mz	-34.535		
Level 2	4, K - H - Level 2	CASE 3	59.472		
Level 2	4, K - H - Level 2	CASE 4: +Moments Add	17.984		
Level 2	4, K - H - Level 2	CASE 4: -Moments Add	76.462		
Level 2	4, K - H - Level 2	CASE 4: +Moments Opposite	25.682		
Level 2	4, K - H - Level 2	CASE 4: -Moments Opposite	68.764		

APPENDIX C

Determine Axial Load on Shear Walls

Axial Loads on Shear Wall G, LL1:

Floor Loads					
Story	Tributary Area (SQ FT)	Dead Load (PSF)	Live Load (PSF)	P_{dead} (kip)	P_{live} (kip)
Penthouse	450	171	20	76.95	9
Roof/ PH Floor	900	156	200	140.4	180
Level 13	900	149.5	100	134.55	90
Level 12	900	149.5	100	134.55	90
Level 11	900	149.5	100	134.55	90
Level 10	900	149.5	100	134.55	90
Level 9	900	149.5	100	134.55	90
Level 8	900	149.5	100	134.55	90
Level 7	900	149.5	100	134.55	90
Level 6	900	149.5	100	134.55	90
Level 5	900	149.5	100	134.55	90
Level 4	900	149.5	100	134.55	90
Level 3	900	149.5	100	134.55	90
Level 2	900	149.5	100	134.55	90
Ground	900	198	100	178.2	90
LL1	900	149.5	40	134.55	36
				2144.7	**1395**

Shear Wall Self Weight					
Story	Height	Length	Thickness	Concrete Weight	P_{dead} (kip)
Penthouse	24.33	30	16	150	146
Roof/ PH Floor	14.5	30	16	150	87
Level 13	13.17	30	16	150	79
Level 12	13.17	30	16	150	79
Level 11	13.17	30	16	150	79
Level 10	13.17	30	16	150	79
Level 9	13.17	30	18	150	89
Level 8	13.17	30	18	150	89
Level 7	13.17	30	18	150	89
Level 6	13.17	30	18	150	89
Level 5	13.17	30	18	150	89
Level 4	13.17	30	18	150	89
Level 3	13.17	30	18	150	89
Level 2	13.17	30	18	150	89
Ground	16.67	30	22	150	138
LL1	13	30	22	150	107
					1505

Total Axial Loads		
P_{live} =	**1395**	**kip**
P_{dead} =	**3650**	**kip**

Axial Loads on Shear Wall 5, T-R, Level 2:

Floor Loads					
Story	Tributary Area (SQ FT)	Dead Load (PSF)	Live Load (PSF)	P_{dead} (kip)	P_{live} (kip)
Penthouse	450	171	20	76.95	9
Roof/ PH Floor	900	156	200	140.4	180
Level 13	900	149.5	100	134.55	90
Level 12	900	149.5	100	134.55	90
Level 11	900	149.5	100	134.55	90
Level 10	900	149.5	100	134.55	90
Level 9	900	149.5	100	134.55	90
Level 8	900	149.5	100	134.55	90
Level 7	900	149.5	100	134.55	90
Level 6	900	149.5	100	134.55	90
Level 5	900	149.5	100	134.55	90
Level 4	900	149.5	100	134.55	90
Level 3	900	149.5	100	134.55	90
Level 2	900	149.5	100	134.55	90
				1831.95	1269

Shear Wall Self Weight					
Story	Height	Length	Thickness	Concrete Weight	P_{dead} (kip)
Penthouse	24.33	30	16	150	146
Roof/ PH Floor	14.5	30	16	150	87
Level 13	13.17	30	16	150	79
Level 12	13.17	30	16	150	79
Level 11	13.17	30	16	150	79
Level 10	13.17	30	16	150	79
Level 9	13.17	30	18	150	89
Level 8	13.17	30	18	150	89
Level 7	13.17	30	18	150	89
Level 6	13.17	30	18	150	89
Level 5	13.17	30	18	150	89
Level 4	13.17	30	18	150	89
Level 3	13.17	30	18	150	89
Level 2	13.17	30	18	150	89
					1260

Total Axial Loads		
P_{live} =	1269	kip
P_{dead} =	3092	kip