SENIOR THESIS

TECHNICAL REPORT 3: LATERAL SYSTEM ANALYSIS

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Date Submitted: November 17th, 2011

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Executive Summary

The purpose of this Technical Report was to do an in depth analysis of the lateral system of the Judicial Center Annex. The lateral system consists of five reinforced concrete shear walls and was analyzed by creating a finite element model in the structural program E-Tabs.

The computer model was made with several assumptions. Floors were modeled as rigid diaphragms, while beams used centerline modeling. A rigid end factor of 0.5 was applied to all frame elements to account for rigid joint behavior, and the Modulus of Elasticity for the concrete elements was cut in half to account for cracked sections and reduced stiffness. In addition to these assumptions the building was modeled down through the basement in an effort to create more accurate results.

Lateral loads were determined for wind and seismic based upon Chapters 6 and 12 of ASCE 7-05 respectively, with the seismic loads adjusted based upon the periods determined from the structures modal analysis. Load cases were determined from ASCE 7-05 and were applied to the model.

Hand checks were performed on the centers of mass and rigidity to confirm the accuracy of the computer model. A hand check was also performed to observe the load path incorporating torsion due to the eccentricity between the center of rigidity and center of mass. The relative stiffness' of the lateral resisting members were identified to help locate the center of rigidity which led to the revelation that in the North-South direction 20% of the load is transferred by the frame action of the wide/shallow beams.

Displacements and story drifts from the model for both wind and seismic were compared to H/400 and .015hsx respectively and found to pass easily even in the worst case loading. It is hypothesized that because the JCA is being attached to an existing building controlling drift drove the design and led to a very stiff building.

Strength checks were performed on Column D4 at the basement level and Shear Wall 4 at Level 1 to determine their combined flexural and axial strength. Both elements were found to be adequate.

Overturning was considered using seismic as the worst case, with a reduced dead load and over strength factor according to ASCE 7-05 12.4.3.2. The building was found to have a Factor of Safety of 17 against overturning.

Building Introduction

The Judicial Center Annex (JCA) is a modern addition to the existing Montgomery County Judicial Center. Located on the corners of Maryland Avenue and East Jefferson Street in downtown Rockville, MD the JCA is set provide a bold statement through both its architecture and engineering. Construction on the addition began this past April and is projected to take two years to complete.

Figure 1: Site Location, Bing.com

The JCA will stand 114' tall at the crest of each of the four lanterns located on top of the building, so tall that local building codes needed waved for overall building height. Six stories rise above the ground, with garage and terrace levels located below grade, adding approximately 210,000 sq ft to the Judicial Center that will add 10 more courtrooms and administrative spaces among other spaces.

The project team, led by AECOM who provided both architectural and the majority of building engineering services, was able to achieve a unique look through both form and material. The East and West Elevations (Figure 2) are dominated by glazing, with the curtain wall that covers the East wrapping around the South corner. This curtain wall system is unique in that it uses glass stabilizing fins instead of traditional aluminum mullions, which enables an all glass look that when combined with the way the slab cantilevers out from the structure gives the illusion of the floors floating without structure. On the North the addition abuts against the original Judicial Center. The elements of the façade not covered in glass are sheathed in either a powder coated aluminum that has a reddish hue or architectural pre-cast panels that are more reminiscent of the exterior of the original building.

Figure 2: West Elevation

From the roof projects four lanterns which have a translucent linear glazing system allowing them to light up the night sky in a truly dramatic manner. The roof is also the site of two of the JCA's sustainable features that enabled it to achieve a LEED Gold Rating. The tops of each of the four lanterns are covered in photovoltaic panels, while green roofs cover much of the remaining roof.

Structural Overview

The JCA sits atop core-drilled concrete piers due to the rather poor soil conditions, all columns coming to bear atop a pier. The floor systems are post-tensioned slabs, with wide-shallow beams running one-way on the typical levels framing into cast-in-place concrete columns. The lateral system consists of five concrete shear walls, which rise continuously to the penthouse level, with some continuing to support the roof.

This building was designed as Occupancy III according to Sheet 1.S001. The reason for this is thought that the holding cells in the building subject it to the "Jail and detention facilities" clause or perhaps a courtroom has the ability for "more than 300 people to congregate." This Occupancy was assumed due to one of the previously mentioned reasons for purposes of coming up with importance factors in later calculations.

Foundations

Schnabel Engineering performed the geotechnical services on the JCA project. Reports indicated that for the purposes of shallow continuous wall footings the soil has a bearing capacity of 2000 psi, with any unsuitable conditions requiring excavation and replacement with lean concrete. Core-drilled piers ranging in diameter from 2.5' to 7' are located beneath every column and support much of the shallow wall footings. Grade beams are also used in several locations, specifically beneath the five shear walls. The usage of grade beams beneath the continuous shear walls is due to the extremely large concentration of forces that need transferred into the soil as a result of both the shear walls own weight and the lateral forces that are being transferred through them. Tying into the Grade beams would help against uplift which will be investigated further in Technical Report 3. Grade beams vary from 24" to 42" in width and 36" to 72" in depth. The slab on grade is 5" thick and reinforced with WWF.

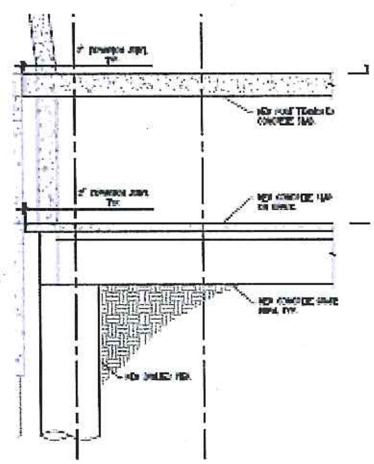


Figure 3: Column adjacent to existing Judicial Center resting on pier foundation

The garage level of the JCA is located 25' below grade. Though soil pressures on basement walls were not considered in this report they are a possible point of investigation in the future.

Framing Systems

Cast-in-place columns rise from the garage level to the roof, with the four lanterns extending the extra fourteen feet with steel framing. The column concrete has a compressive strength of 7000 psi at the base, which is reduced to 5000 psi at level 2. Typical column sizes are 24"x24"

Each lantern has a flat roof framed in structural steel with a slight slope on the edges. HSS tubes make up the columns, with the majority of the framing being small steel shapes with spans in the range of 5' and typical sizes of L3x3x1/4, HSS4x4x1/4, and C6x13. In the center of the roof are several W12x40 girders with a maximum span of 33' that are famed into by smaller wide flange shapes. These heavier shapes are intended to carry the photovoltaic panels mounted on top of the lanterns. Several HSS brace frames provide lateral stability for the lanterns. The lanterns were given an assumed weight of 30 psf in the center section to account photovoltaic panels, leading to a total weight of approximately 50 kips per lantern.

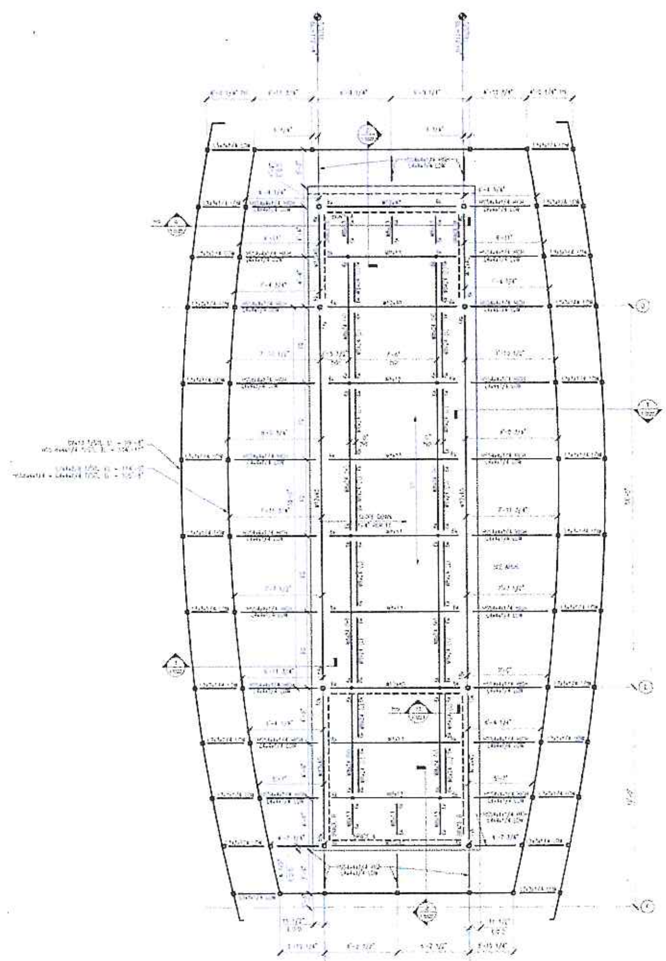


Figure 4: Lantern Framing Plan, larger plan found in Appendix A

Floor Systems

The current floor system of the JCA is a post tensioned slab that ranges in depth from 8" to 9" on a typical floor.PT slabs are used to achieve greater economy over longer spans as the moment balancing allows for a shallower slab depth. The plans denote continuous drop panels which are also referred to as slab bands in the design narrative that run in the North-South direction and are approximately 8' in width with a depth of 8" beyond the adjacent slab. These are interpreted as wide-shallow beams as it is thought they may prove beneficial with regards to reducing positive moment reinforcement. According to ACI 318-08 section 13.2.5 a drop panel that is used to reduce negative moment reinforcement or a minimum slab thickness will meet two requirements: project beneath the slab at least one quarter of the adjacent slab distance and extend in each direction from the centerline of support a distance greater than one sixth the span length measured from center to center. The wide-shallow beams meet these requirements and therefore may be called continuous drop panels, though because it is assumed that they are providing aid to the positive moment they will be referred to as beams from here on out.

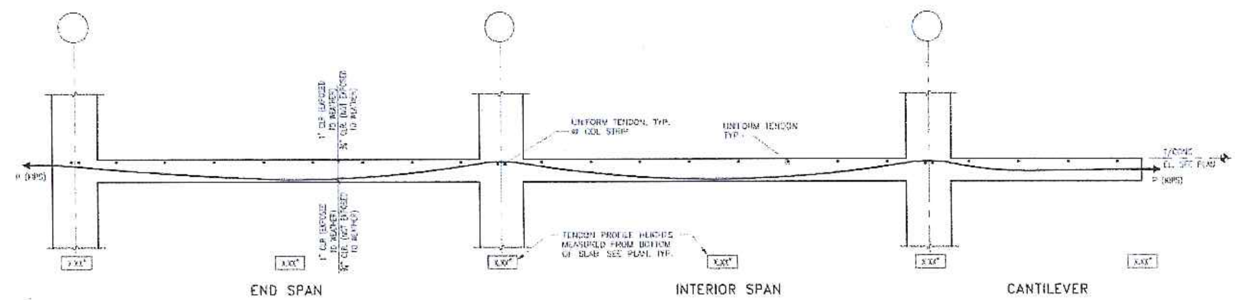


Figure 5: Section of Post-Tensioned Slab

The penthouse slab is 11" thick due to the larger loads present on this floor. There is an unreducible 150 psf mechanical live load present, as well as a 55 psf green roof dead load in several areas. The mechanical floor also features a "floating" four inch light weight concrete on metal deck isolation slab, that is isolated from the slab it rests upon by dampers to prevent mechanical equipment vibrations from affecting other parts of the building. The roof slab is 10" and features several large voids. This slab has post tensioned beams 36" x 24" typical for additional span stiffness in lieu of the wide-shallow beams.

Figure 7: Shear Wall Layout

Roof Systems

The roof varies in height in several locations with the floor slabs described earlier in *Floor Systems*. The varying heights made snow drift a concern, and the large loads associated with the penthouse floor, which is the heaviest floor on the building, add a significant contribution to both seismic base shear and overturning. The green roof and pavers on the penthouse and upper roof levels lay overtop a hot applied fluid membrane.

Design Codes

The list of Major Codes and Standards on Sheet 1.S001 is as follows:

- 2009 International Building Code
- ACI 318-08
- AISC LRFD, 13th Edition, 2005
- AWS D1.1, D1.3, D1.4, Current Edition
- ASTM, Current Edition
- Steel Deck Institute Design Manual for Composite Deck, Form Decks and Roof Decks., 2007

These are the codes being used to complete the analyses performed in this report, with heavy usage of ASCE 7-05 (Minimum Design Loads).

Materials Used

Sheet 1.S001 was used as the reference for materials used in the construction of this project and summarized in Figure 6.

Concrete		
Usage	Weight	f'c (psi)
Column (Levels 2-Rf)	Normal	5000
Column (Levels G1-1)	Normal	7000
Floor Slab	Normal	5000
Wall Footings	Normal	3000
Beams	Normal	5000
Slab on Grade	Normal	4500
Walls, Piers, & Pilasters	Normal	5000
Drilled Piers	Normal	4000
LW Concrete Fill on Deck	Light	4000
Isolation Slab @ Penthouse	Light	4000

Steel		
Type	ASTM Standard	Grade
W Shapes	A992	
Plates, Angles, Channels	A36	
High-Strength Bolts	A325 or A490	
Anchor Rods	F1554	36
Tubes	A500	B
Pipes	A53 E or S	B
Reinforcing Steel	A615	60
Reinforcing Steel, Welded	A706	60
Roof Deck	A653	A - F
Floor Deck	A653	C, D, or E
Post-Tensioned Reinforcment	A416-96	

Masonry		
Type	ASTM Standard	F'm (psi)
CMU	C90	1500
Masonry Mortar	C270	
Grout	C476	
Aggregate	C404	

Figure 6: Summary of Materials Used

Gravity Loads

This section will describe how dead, live, and snow loads were calculated and compared to loadings given on the structural drawings. Three gravity checks were performed once the loadings were determined for an interior column, the typical long span for the post tensioned slab, and a doubly reinforced beam with full hand calculations available in Appendix A.

Dead and Live Loads

Dead Loads		
	Design	Student
Vegetated Roof	55	55
MEP/Celing	15	15
CMU Partitions	Actual Weight	91 pcf (Fully Grouted Assumption)

Figure 7: Summary of Dead Loads

The dead loads listed on 1.S001 shown in Figure 7 were used for the purposes of analyses. The non-load-bearing CMU walls were assumed to be fully grouted for the purposes of worst-case load calculations. The weight of the building was calculated neglecting voids in slabs and with an assumption of 10 psf for the steel lantern framing, which would not have much effect on the building weight were it too small an assumption. The total building weight which was used for the seismic calculations was in the order of 40000 kips when accounting for floors at or below grade.

Based upon ASCE 7-05 the 100 psf typical live load was found to be correct, possibly for different reasons than the designer decided for, and the 40 psf holding cell load was neglected in favor of using the 100 psf live load in all locations except for the mechanical penthouse and the roof loading.

Live Loads		
	Design	ASCE 7-05
Typical	100	80 (Corrider Above First Floor) + 20 (Partition) = 100
Holding Cells	40	-
Mechanical Penthouse	150	150
Roof	-	20

Figure 8: Summary of Live Loads

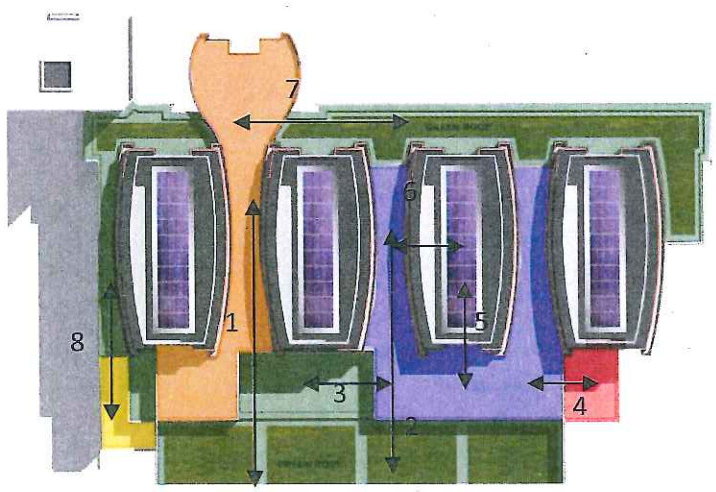
Snow Loads

The flat roof snow load was calculated via the method outlined in Chapter 7 of ASCE 7-05. A discrepancy arose as the importance factor, I, listed on the drawings had a value of 1.0, whereas the appropriate importance factor for an Occupancy III building is 1.1. This led to flat roof snow load value of 22 psf which differs from the calculated value of 23.1 psf. Curiously the design load is higher despite the lower importance factor which may be a result of a higher design ground snow load, though this isn't available on the drawings.

Flat Roof Snow Load		
pf = .7 CeCtIpg > 20*I		
Ce	1	ASCE 7-05 Tab. 7-2
Ct	1	ASCE 7-05 Tab. 7-3
pg	25	ASCE 7-05 Fig. 7-1
I	1.1	ASCE 7-05 Tab. 7-4
pf =	0	
20*I=	500	
pf =	22	

Figure 9: Snow Load Parameters and Flat Roof Calculation

The varying roof levels led to eight different drift calculations. The calculations can be see viewing Figure 10 and 11, with an accompanying hand check for one of the drifts performed in Appendix A.



Snow Drift		γ=	17.25						
	Lu	Ll	hc	hd Lee	hd Wind		hd (ft)	w (ft)	Max psf
Drift 1	130	50	16	3.79826	1.764815	3.79826	3.79826	15.19	65.52
Drift 2	93	30.33	18	3.238561	1.321269	3.238561	3.238561	12.95	55.87
Drift 3	70	50	18	2.810406	1.764815	2.810406	2.810406	11.24	48.48
Drift 4	70	20	21	2.810406	1.004234	2.810406	2.810406	11.24	48.48
Drift 5	70	20	14	2.810406	1.004234	2.810406	2.810406	11.24	48.48
Drift 6	38	12	14	2.016252	0.670866	2.016252	2.016252	8.07	34.78
Drift 7	21	147	16	1.385528	3.014862	3.014862	3.014862	12.06	52.01
Drift 8	83	24	52	3.06224	1.137649	3.06224	3.06224	12.25	52.82

Figure 10: Drift Diagram and Spreadsheet

Lateral System Analysis

The purpose of this report is to analyze the existing lateral system and confirm whether it is adequacy with regards to the calculated seismic and wind forces. To accomplish this, the building was modeled in the computer analysis program E-Tabs.

The lateral system of the JCA is comprised of five shear walls, highlighted in red Figure 11. The walls rise continuously, with shear walls 1-3 extending to the roof while shear walls 4 and 5 end at the penthouse level. The walls are all 12" thick, and have several large openings. These openings will be critical areas for further investigation as their presence creates link beams that must be able to transfer the shear load to maintain the load path.

The wide/shallow beams in the floor slab, highlighted in green, are also believed to create a frame action running in the North/South direction that will be addressed later in the report. Finally there are several small concrete frames, those in the North/South direction are highlighted in orange and those in the East/West direction are highlighted in blue. The lateral elements are also given a label which they will be referred to by later in the report.

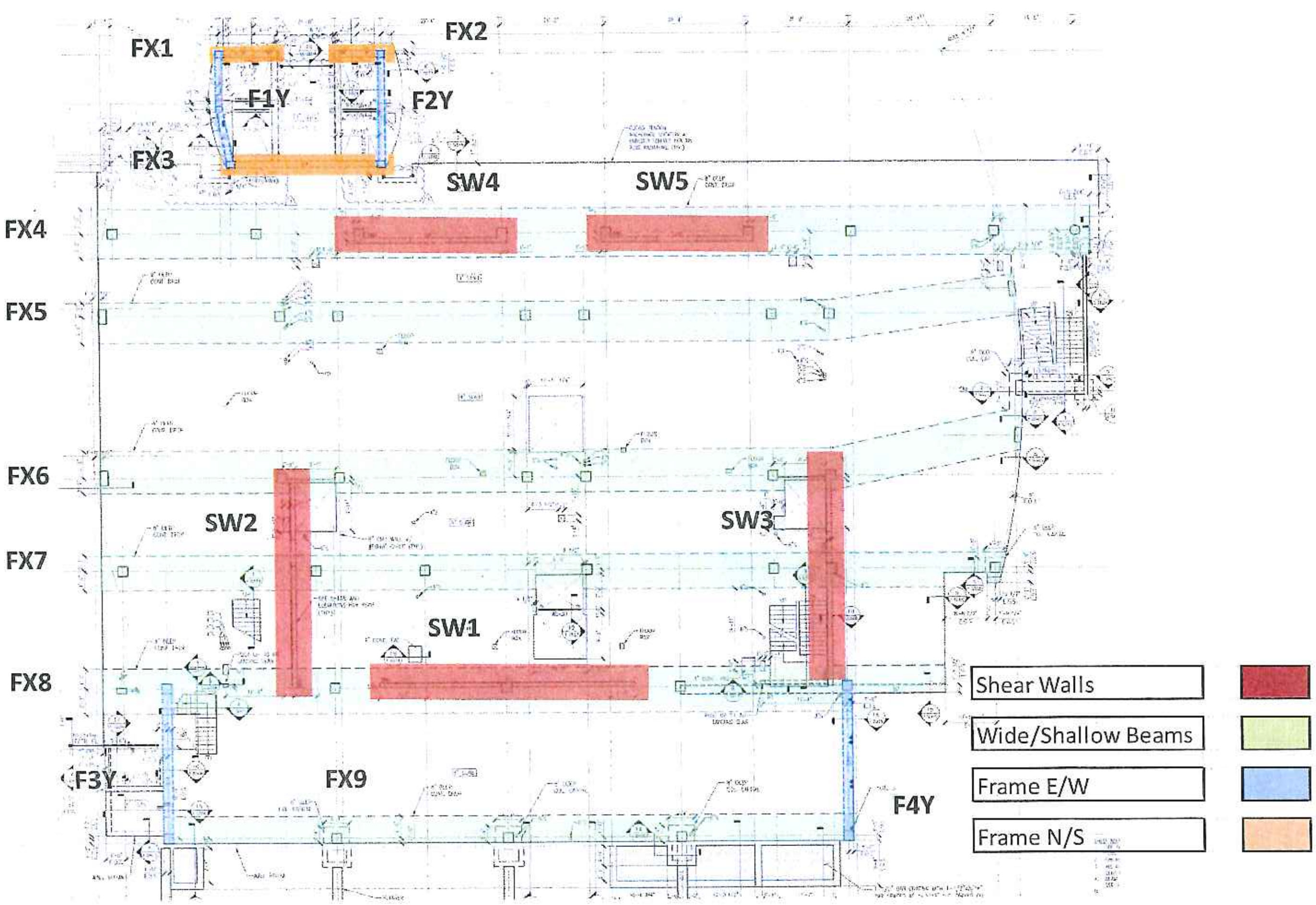


Figure 11: Lateral System, Typical Floor

Computer Model

To perform a more accurate and sophisticated analysis the lateral system the JCA was explicitly modeled using the structural analysis program E-Tabs.

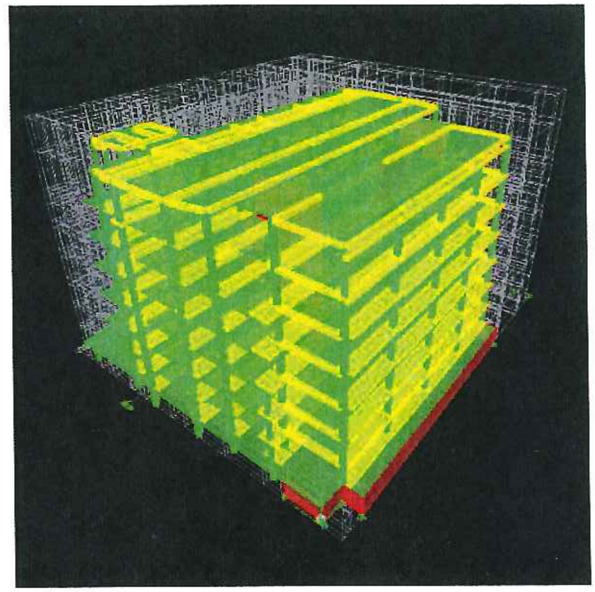


Figure 12: 3D View of Finite Element Model in E-Tabs

The model went through several iterations, each becoming slightly more sophisticated. The first model was modeled ignoring the two floors below grade, with only the shear walls. It was hypothesized however that the frame elements would contribute to the system, and to see the extent to which they contributed it was necessary to add the columns and wide/shallow beams. Finally the two floors below grade were added for a more accurate model. This was also done due to the nature of the terrace level not being entirely below grade for the entire building, thus making it a more conservative approach to model this floor and account for the loading it would receive.

Several assumptions were made in the modeling process. Floors were modeling as rigid diaphragms, which causes all connected nodes to displace together but ignores any in-plane stiffness. A more accurate representation would've been using a shell element which could be explored in future iterations, but as the floor slab is relatively complex with regards to changing depths it was deemed a valid simplification. The mass was lumped on the floor diaphragms, with the mass of the lanterns lumped onto the roof below as the total weight they represented was less than 20% of the average floor weight.

Beams were modeled using centerline modeling, with insertion points a consideration for the future. Adhering to section 12.7 of ASCE 7-05 the foundations were modeled as fixed, and the stiffness properties of cracked concrete were considered by reducing the Modulus of Elasticity by half. ACI 8.8.2 allows for the stiffness of all elements to be reduced by 50% based upon their gross section properties in lieu of section 10.10.4.1 which provides inertia factors for various elements. Rigid joint effects were accounted for using a standard of care value of 0.5 as panel zones were not employed.

In modeling a basement wall soil springs were considered. E-Tabs does not provide an area spring element that only takes compression however, so these were not employed. It is conservative to assume non cohesive soil, which cannot take tension, so this short coming in the area spring would likely have hurt the accuracy of the model rather than improve it. A possible way around this would be to create a number of nodes along the basement wall and attach links with the 'Gap' property that is designed to act in compression only. Research on the subject seemed to indicate it may not work as intended, though it is an option to explore in future models.

One of the first checks of the computer model to confirm accuracy was against the centers of mass and rigidity that it derived.

The 1st Level was chosen, as it is a fairly representative floor for the building. Hand calculations of the center of mass gave a 5% error in the X-direction and an error of 0.1% in the Y-direction, indicating that the model was accurate with regards to this. To check the center of rigidity relative stiffness values were determined by applying a 1000 kip load in the direction of interest. As the floors were modeled as rigid diaphragms the load each element takes will be proportional to the stiffness of the element. To avoid any torsional effects which could skew the accuracy of results the 1000 kip load was applied at the center of rigidity as a separate load case per level. The hand calculations for the center of rigidity using the relative stiffness values pictured to the right had an error of 3.8% in the X-Direction and 0.4% in the Y-Direction indicating that the computer model was accurate for the center of rigidity as well.

Etabs Centers of Mass and Rigidity

Story	X Center of Mass	Y Center of Mass	X Center of Rigidity	Y Center of Rigidity
Roof	1059.672	839.61	1132.941	673.658
Penthouse	1105.524	889.647	1118.572	675.523
Level 5	1107.132	895.846	1109.757	686.891
Level 4	1107.939	898.953	1101.768	701.845
Level 3	1108.423	900.821	1090.231	722.559
Level 2	1108.746	902.067	1086.859	709.648
Level 1	1108.272	902.563	1142.5	754.843
Terrace	1103.936	912.659	1690.37	812.459

Level 1 Relative Stiffness

X Direction			Y Direction		
Lateral Element	Shear	%	Lateral Element	Shear	%
SW1	569.47	56.90%	SW2	448.41	50.83%
SW4	148.46	14.84%	SW3	410.66	46.55%
SW5	141.83	14.17%	F1Y	4.92	0.56%
F1X	1.62	0.16%	F2Y	3.12	0.35%
F3X	1.54	0.15%	F6Y	8.04	0.91%
F2X	1.71	0.17%	F5Y	7.06	0.80%
F4X	27.07	2.70%		882.21	
F5X	22.76	2.27%			
F6X	23.61	2.36%			
F7X	18.72	1.87%			
F8X	31.69	3.17%			
F9X	12.26	1.23%			
	1000.74				

Figures 13 and 14: Center of Mass and Rigidity/Relative Stiffness of Level 1

Lateral Loads

Wind and seismic loads were calculated using the prescribed methods from ASCE 7-05 so that they could be applied in the computer model.

Wind Loads

Method 2 Main Wind Force Resisting System (MWRFS) procedure from ASCE 7-05 chapter 6 was used in the calculation of the wind forces the building will be subjected to. To simplify the calculations, the maximum roof height was made 115'. This ignores the lanterns, as they have a small surface area that would not result in much load accumulation and accounts for the inclusion of the Terrace Level (the roof is listed at 100' above grade). As mentioned before it is more conservative to take the entire Terrace above grade so that both Windward and Leeward forces can be applied, which will also give a more 'apples to apples' comparison with the seismic forces that are taking into account the additional mass of the Terrace level that was originally not accounted for. Additionally the floor plan was assumed rectangular and an idealized building width and length were determined to get values of L and B.

Wind loads originate as a pressure on the building enclosure which creates a force that moves through the slab to the lateral elements and from there into the foundation system.

Design Wind Pressure E/W							
		Distance	Wind Pressure	Internal Pressure		Net Pressure	
				(+) Gcpi	(-) Gcpi	(+) Gcpi	(-) Gcpi
Windward	Terrace	0	7.86	3.70	-3.70	4.15	11.56
	1st	15	7.86	3.70	-3.70	4.15	11.56
	2nd	29	9.54	3.70	-3.70	5.83	13.24
	3rd	44.5	10.79	3.70	-3.70	7.08	14.49
	4th	61	11.77	3.70	-3.70	8.07	15.47
	5th	77.5	12.68	3.70	-3.70	8.98	16.38
	Penthouse	94	13.40	3.70	-3.70	9.69	17.10
	Roof	115	13.99	3.70	-3.70	10.29	17.69
Leeward	All	-	-8.04	3.70	-3.70	-11.75	-4.34
Side Walls	All	-	-12.24	3.70	-3.70	-15.94	-8.54
Roof		0 - 50	-16.58	3.70	-3.70	-20.28	-12.87
		> 50	-15.32	3.70	-3.70	-19.02	-11.62

Figure 15: Design Wind Pressures in the E/W Direction

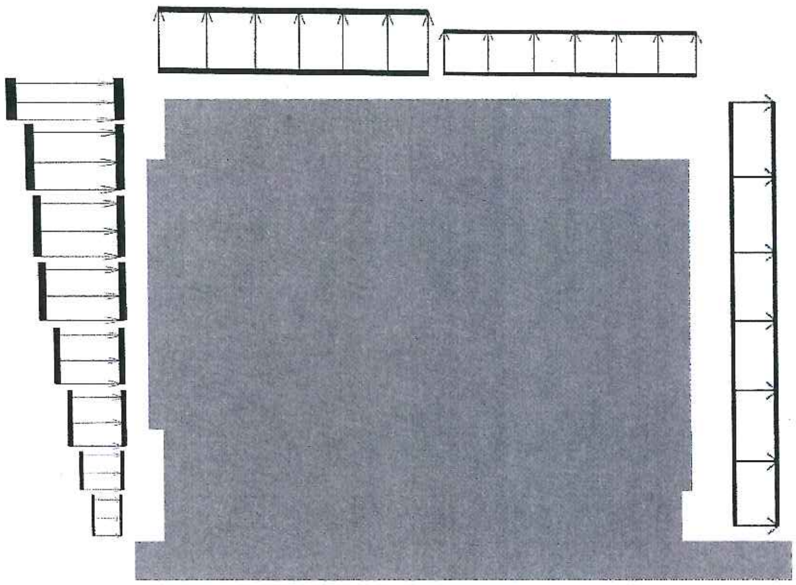


Figure 16: Pressure distribution in E/W direction corresponding to Figure 15

Wind Force (EW)								
		Trib Below		Trib Above		Story Force	Story Shear	Overturning Moment
	Height	Ht	Area	Ht	Area			
Terrace	0	0	0	7.5	1350.00	21.47	398.29	0.00
1st	15	7.5	1350	7	1260.00	41.50	376.82	622.51
2nd	14	7	1260	7.75	1395.00	46.68	335.32	1353.74
3rd	15.5	7.75	1395	8.25	1485.00	54.23	288.64	2413.20
4th	16.5	8.25	1485	8.25	1485.00	58.85	234.41	3589.90
5th	16.5	8.25	1485	8.25	1485.00	61.55	175.56	4770.33
Penthouse	16.5	8.25	1485	10.5	1890.00	72.37	114.01	6802.32
Roof	21	10.5	1890	0	0.00	41.64	41.64	4789.13
							Base Shear (k)	398.29
							Total Overturning Moment (k-ft)	24739.43

Figure 17: Wind Forces in E/W Direction

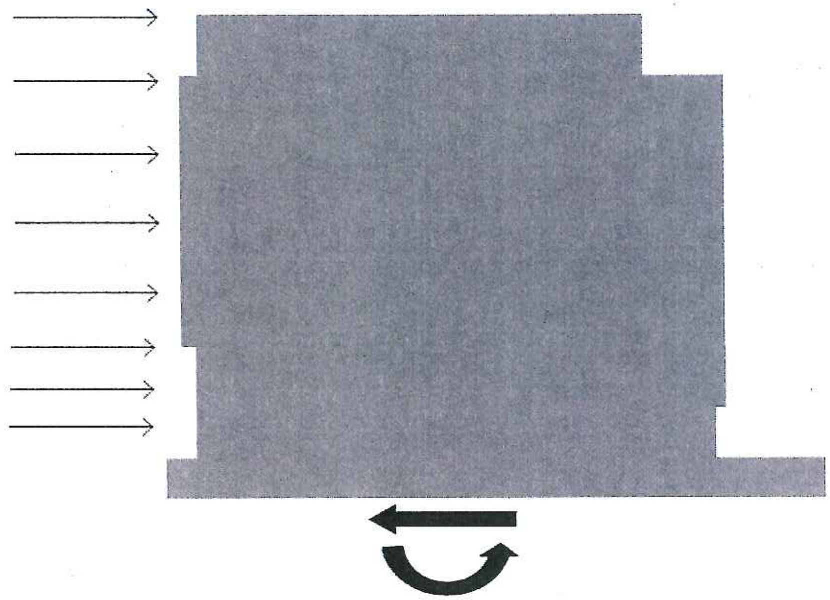


Figure 18: Wind force distribution corresponding to Figure 17

Seismic Loads

The seismic loads were calculated based upon the Equivalent Lateral Force Method outlined in ASCE 7-05 Chapters 11 and 12. The fundamental period was calculated via equation 12.8-7 in ASCE 7-05 and modified by the Cu coefficient found in Table 12.8-1 as the low base shear listed on the drawings did not seem achievable given the weight of the structure if the period was calculated via the method for shear walls. As the building was being modeled including the levels below grade it was thought that the mass of these floors should be accounted for. This increased the height of the building as well making the period Ta=0.81 and CuTa=1.38. Using R=5 (ASCE 7-05 Table 12.2-1) due to the ordinary shear wall lateral system and I=1.25 due to the Occupancy Category of 3 the resulting base shear was 591 kips, which adds weight to the assumptions made with regards to calculating this value as the drawings list a base shear of 560 kips.

The first three modes determined by E-tabs were assumed to equal either the X-Translational, Y-Translational, or Z-Rotational directions based upon their modal participation factors listed. This resulted in periods for the X and Y directions, which correspond to the E/W and N/S directions respectively, that fell within the upper bound envelope of CuTa calculated according to ASCE 7-05 section 12.8.2. Smaller periods result in larger loads, so the seismic forces in the X and Y directions were recalculated with these new values. These results were vetted by applying E-Tabs automatic seismic loads in both the X and Y directions. The base shear calculated by hand was 884 kips and 658 kips compared to the values of 907 kips and 681 kips that E-Tabs determined, indicating that I had assigned the appropriate mode shapes to the directions of motion and that these values were more accurate.

Modal Participation Factors										
	Mode	Period	UX	UY	UZ	RX	RY	RZ	ModalMass	ModalStiff
Y Translation	1	1.24107	2.528019	7.428368	0	-7777.2	2661.389	-1718.16	1	25.631081
Z Rotational	2	1.200028	-5.26118	3.465123	0	-3583.26	-5508.37	4052.284	1	27.414301
X Translational	3	0.923084	5.655301	-0.13946	0	170.2161	5970.554	4717.421	1	46.33164
	4	0.315555	1.717248	3.479439	0	-722.318	433.1006	-1197.28	1	396.469509
	5	0.308806	2.671268	-2.27397	0	416.2821	641.6679	-1582.69	1	413.988171
	6	0.24414	2.043353	-0.03195	0	21.46404	439.1207	2630.373	1	662.343176
	7	0.201336	-2.64765	-0.41627	0	78.74266	-566.525	233.416	1	973.901248
	8	0.167458	-0.50477	2.119451	0	-425.628	-97.7315	-348.924	1	1407.827744
	9	0.140215	1.024335	0.260947	0	-68.1638	194.9841	1470.274	1	2008.03532
	10	0.125913	0.709123	1.440351	0	-190.708	103.4811	-550.72	1	2490.125854
	11	0.121002	1.443691	-0.68252	0	90.98848	208.2465	-607.199	1	2696.344878
	12	0.09717	-1.10029	0.891087	0	-106.546	-151.299	-869.373	1	4181.172707

Figure 19: Modal Participation Factors

Seismic Forces N/S (X) Direction						
Level	Story Ht (ft)	Story Weight (k)	Cvx	Story Force (k)	Shear Shear (k)	Overturning Moment (k-ft)
G1	0	0	0	0.00	884.00	0.00
Terrace	15	5809.9676	0.034742	30.71	884.00	460.67
1	25	4421.0536	0.042873	37.90	853.29	947.50
2	39	4868.4042	0.074363	65.74	815.39	2563.75
3	54.5	4954.1477	0.105933	93.64	749.65	5103.63
4	71	4977.945	0.138734	122.64	656.01	8707.48
5	87.5	4967.07	0.170564	150.78	533.37	13193.11
PentHouse	104	6902.0272	0.291123	257.35	382.59	26764.66
Roof	123	3078.675	0.141669	125.24	125.24	15403.92
					Base Shear (k)	884.00
					Total Overturning Moment (k-ft)	72684.05

Figure 20: Seismic Forces in the N/S Direction

Seismic Forces E/W (Y) Direction						
Level	Story Ht (ft)	Story Weight (k)	Cvx	Story Force (k)	Shear Shear (k)	Overturning Moment (k-ft)
G1	0	0	0	0.00	658.00	0.00
Terrace	15	5809.9676	0.034742	22.86	658.00	342.90
1	25	4421.0536	0.042873	28.21	635.14	705.27
2	39	4868.4042	0.074363	48.93	606.93	1908.31
3	54.5	4954.1477	0.105933	69.70	558.00	3798.86
4	71	4977.945	0.138734	91.29	488.29	6481.36
5	87.5	4967.07	0.170564	112.23	397.01	9820.21
PentHouse	104	6902.0272	0.291123	191.56	284.78	19922.11
Roof	123	3078.675	0.141669	93.22	93.22	11465.81
					Base Shear (k)	658.00
					Total Overturning Moment (k-ft)	54101.93

Figure 21: Seismic Forces in the E/W Direction

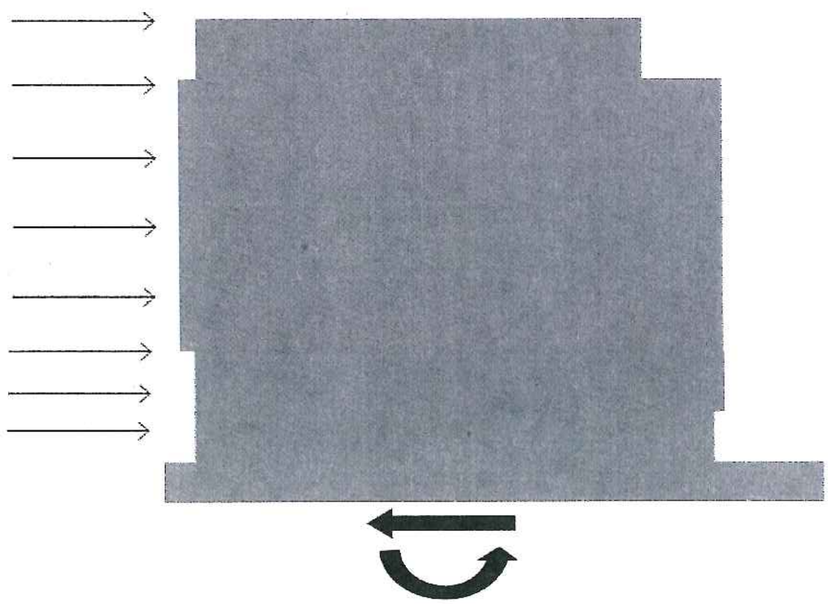


Figure 22: Seismic force distribution corresponding to Figure 21

Load Cases and Paths

The purpose of this analysis was a lateral force check. Therefore, gravity loads were not applied to the model. ASCE 7-05 section 2.3.2 prescribes the Basic Load Combinations for strength design which never have seismic and wind acting concurrently. However there are different load cases within seismic and wind prescribed in Chapters 12 and 6 respectively of ASCE 7-05 that would each need investigated to see which load case controls. The load cases were created manually to ensure accuracy and are summarized in the table below. Eccentricities were calculated by hand based upon the idealized dimensions of 150' x 180'. ASCE 7-05 Section 12.8.4.2 prescribes that in cases where diaphragms are not flexible an accidental moment due to inherent torsion be accounted for by applying the load at an eccentricity of 5% from the center of mass which was taken into account. This accidental torsional moment is not amplified by Ax as required by Section 12.8.4.3 because the SDC is B. Section 12.5.3a which describes the Orthogonal Combination Procedure was followed to create the final two seismic load cases though the JCA is in an SDC B and this section is not required.

Load Cases			
Type			User Designation
Wind	Case 1	100% X	WC1X
		100% Y	WC1Y
	Case 2	100% X + e_x	WC2X+
		100% X - e_x	WC2X-
		100% Y + e_y	WC2Y+
		100% y - e_y	WC2Y-
	Case 3	75% X + 75% Y	WC3
	Case 4	56.3% X + 56.3% Y + e_x + e_y	WC4EX+EY+
		56.3% X + 56.3% Y + e_x - e_y	WC4EX+EY-
		56.3% X + 56.3% Y - e_x + e_y	WC4EX-EY+
		56.3% X + 56.3% Y - e_x - e_y	WC4EX-EY-
Seismic		100% X	QCX
		100% Y	QCY
		100% X + 30% Y	QCX.3Y
		100% Y + 30% X	QC.3XY

Figure 23: Load Case Table

In the computer model section Figures 13 and 14 were presented. As can be seen the center of masses and center of rigidities are not at the same point, which means that the systems will see torsional effects in addition to the eccentricities required by code. An example of the story shear distribution is shown in Appendix C for the lateral system supporting the 1st Level.

Earlier it was mentioned that an area of investigation for this report was how much the concrete frames are involved in the lateral load path. In an earlier section the calculation for the center of rigidity was performed by determining the relative stiffness of the various lateral elements. According to this floor by floor break down in the Y-Direction the shear walls take almost all of the force, accounting for on average 95% of the direct shear transfer from floor to floor. In the X-Direction the shear walls on average accounted for 80% of the direct shear transfer, which means the wide/shallow beams do form moment frames which are important to the shear load path, so it was important that they were included in the building model.

Displacement and Story Drift

The total level displacements and story drifts were computed for each level using E-Tabs and the service loads for the given load cases. For lateral systems it is not the overall displacement so much as the relative displacement of stories, story drift, that is an indication of damage, so this was the parameter focused on. The allowable seismic story drift was taken as 0.015hsx from ASCE 7-05 Table 12.12-1 and the seismic drifts were amplified according to Equation 12.8-15. The rule of thumb for story drift due to wind is H/400 and was used to evaluate the drifts due to wiind loading. All story drifts were found to be well within the necessary bounds. The lateral system for this building may have been created to control drift as it will be abutting a current structure, which may explain the small values for displacement and story drift. Shown below are the controlling wind and seismic load cases for the X and Y directions.

Load Case: QCX+.3Y								
				Amplified by Cd/I				Δa =.015sx
Story	Height	δxe	δye	δx	δy	Δx	Δy	
Roof	19	0.613247	0.246227	2.207689	0.886417	0.3918	0.1501	3.42
Penthouse	16.5	0.504422	0.204539	1.815919	0.73634	0.3763	0.1524	2.97
Level 5	16.5	0.399897	0.162199	1.439629	0.583916	0.3891	0.1591	2.97
Level 4	16.5	0.291809	0.118001	1.050512	0.424804	0.3748	0.1542	2.97
Level 3	15.4	0.187685	0.075155	0.675666	0.270558	0.2890	0.1183	2.772
Level 2	14	0.107401	0.042281	0.386644	0.152212	0.2305	0.0941	2.52
Level 1	15	0.043377	0.016145	0.156157	0.058122	0.1352	0.0520	2.7
Terrace	10	0.005833	0.001708	0.020999	0.006149	0.0210	0.0061	1.8

Load Case: QCY								
				Amplified by Cd/I				Δa =.015sx
Story	Height	δxe	δye	δx	δy	Δx	Δy	
Roof	19	-0.0238	0.5574	-0.0858	2.0067	-0.0002	0.3118	3.42
Penthouse	16.5	-0.0238	0.4708	-0.0855	1.6949	-0.0109	0.3132	2.97
Level 5	16.5	-0.0207	0.3838	-0.0747	1.3817	-0.0149	0.3389	2.97
Level 4	16.5	-0.0166	0.2897	-0.0598	1.0428	-0.0166	0.3391	2.97
Level 3	15.4	-0.0120	0.1955	-0.0432	0.7037	-0.0154	0.2970	2.772
Level 2	14	-0.0077	0.1130	-0.0278	0.4067	-0.0183	0.2422	2.52
Level 1	15	-0.0026	0.0457	-0.0095	0.1645	-0.0117	0.1553	2.7
Terrace	10	0.0006	0.0025	0.0022	0.0091	0.0022	0.0091	1.8

Load Case: WC2XE-						
Story	Height	δxw	δyw	Δx	Δy	Δa =H/400
Roof	19	0.2268	0.0777	0.0366	0.0124	0.5700
Penthouse	16.5	0.1902	0.0652	0.0366	0.0129	0.4950
Level 5	16.5	0.1537	0.0523	0.0388	0.0136	0.4950
Level 4	16.5	0.1149	0.0387	0.0386	0.0135	0.4950
Level 3	15.4	0.0763	0.0252	0.0312	0.0106	0.4620
Level 2	14	0.0451	0.0146	0.0265	0.0088	0.4200
Level 1	15	0.0186	0.0057	0.0166	0.0052	0.4500
Terrace	10	0.0020	0.0005	0.0020	0.0005	0.3000

Load Case: WC2YE-						
Story	Height	δxw	δyw	Δx	Δy	Δa =H/400
Roof	19	0.0492	0.2658	0.0084	0.0441	0.5700
Penthouse	16.5	0.0408	0.2216	0.0084	0.0411	0.4950
Level 5	16.5	0.0324	0.1805	0.0087	0.0437	0.4950
Level 4	16.5	0.0237	0.1368	0.0085	0.0442	0.4950
Level 3	15.4	0.0152	0.0925	0.0066	0.0386	0.4620
Level 2	14	0.0086	0.0539	0.0051	0.0304	0.4200
Level 1	15	0.0035	0.0235	0.0030	0.0202	0.4500
Terrace	10	0.0005	0.0034	0.0005	0.0034	0.3000

Figure 24: Controlling Load Case Story Drifts

Strength Checks

Two strength spot checks were performed. Column D4 which was spot checked in Technical Report 1 for gravity loads was checked for combined flexure and axial now that the lateral loads have been determined. Using spColumn to create an interaction diagram the worst case design moment caused by seismic forces was applied in addition to the gravity loads. The column was found to be more than adequate. Shear wall number 4 was also checked for combined flexural and axial and found to be adequate, with the assumption that only self-weight would contribute to the axial load. Calculations for the strength checks can be found in Appendix D.

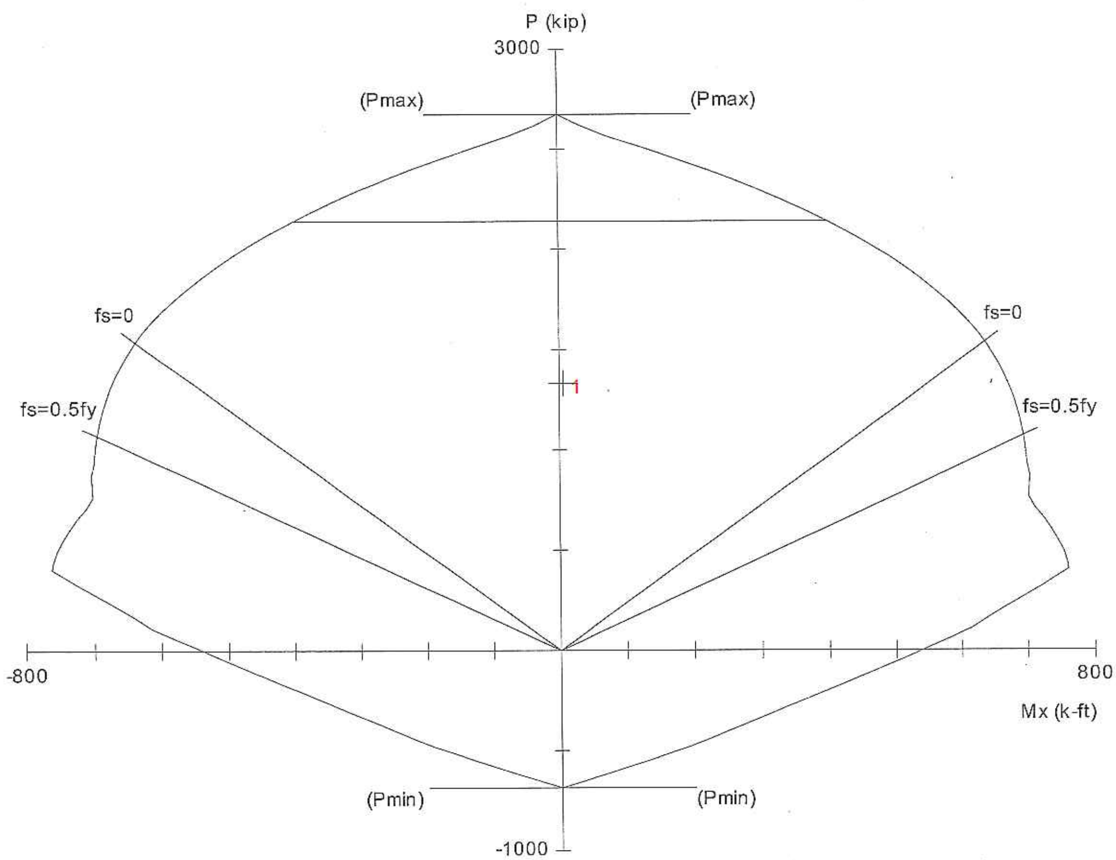


Figure 25: spColumn Interaction Diagram

Overturning Moment

Lateral loads create an overturning moment which is resisted by the building weight. If the stabilizing moment due to the building weight is not adequate to resist the overturning moment the foundations will see uplift forces that will need to be dealt with. The seismic forces control with respect to overturning in both directions. Under section 12.4.3.2 the worst case load combination will be;

$$(0.9 - 0.2S_{DS})D + \Omega_O Q_E$$

Even with the reduction in dead load available to resist the overturning moment and the over strength factor of 2.5 for ordinary reinforced shear walls the stabilizing moment was more than adequate to resist the overturning moment with a factor of safety of 17 in the X-direction and 19 in the Y-direction.

Conclusion

In conclusion the analysis of the lateral system the JCA was found to be adequately designed for strength, story drift, and overturning.

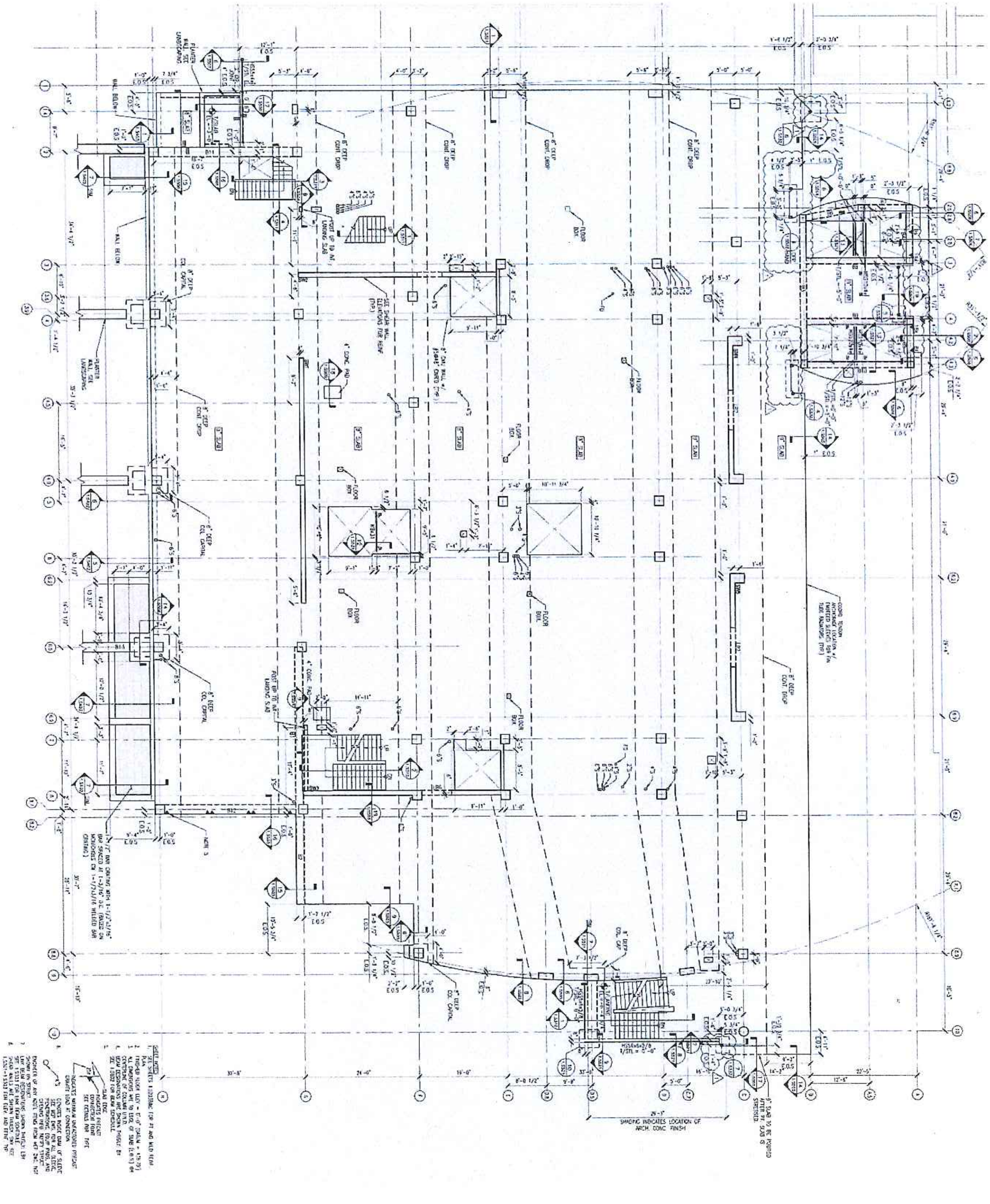
The finite element model created in E-Tabs was determined accurate through hand calculations for the center of mass and rigidity. Relative stiffness of the lateral elements was determined by applying forces at the given center of rigidity on each level to avoid torsional effects, with the result that the wide/shallow beams transfer an average of 20% of the shear force per level, indicating that it was worth modeling the columns and beams.

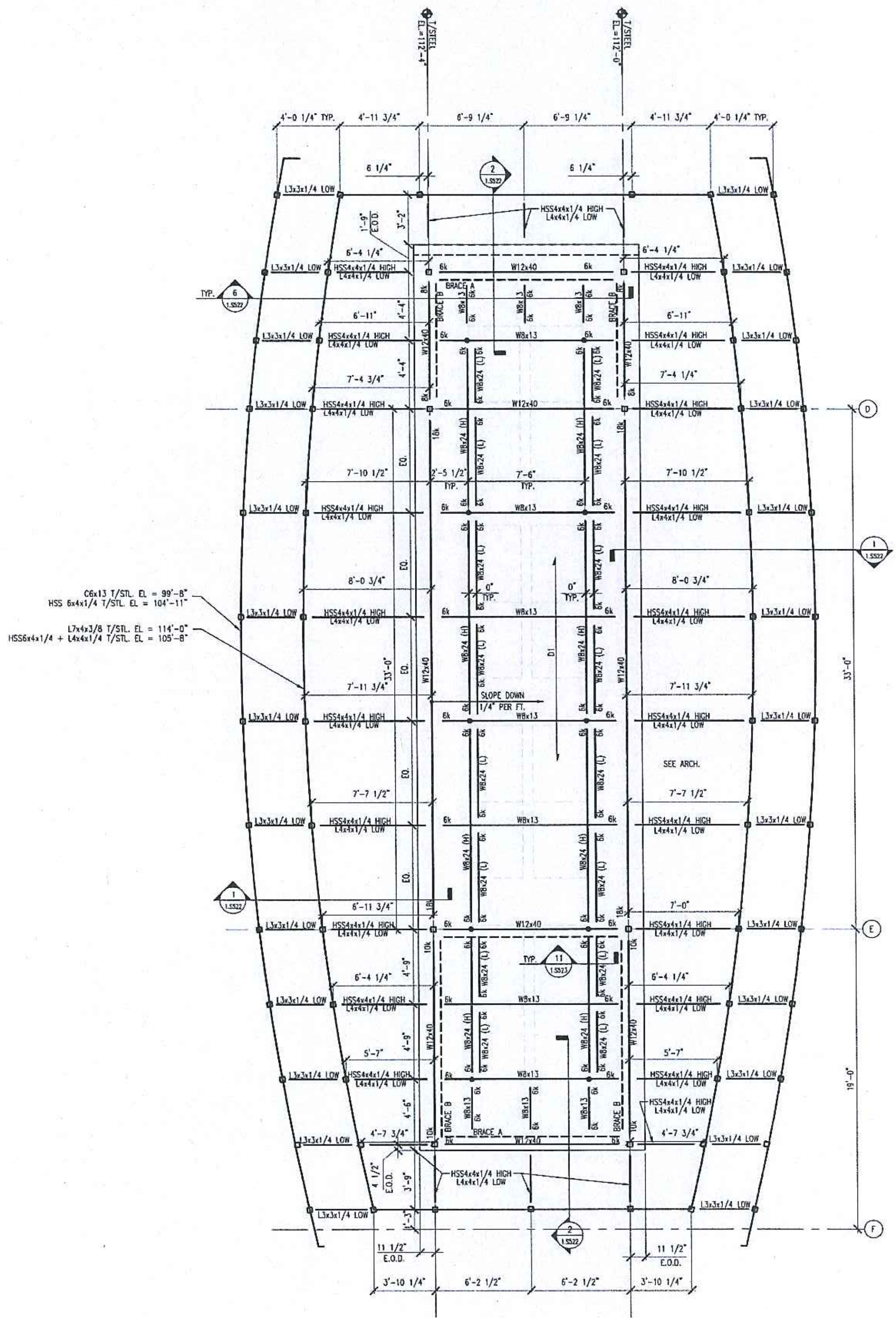
Wind and seismic forces were determined similarly to Technical Report 1 and applied in several different load cases. The controlling drift cases were Seismic QCY and QCX.3Y and Wind WC2XE- and WC2YE-, with the seismic cases providing the greatest overall displacement. All drifts were found to fall within code and prescribed limitations by a large margin lending weight to the idea that perhaps drift controlled the design as this building is abutted against another existing building.

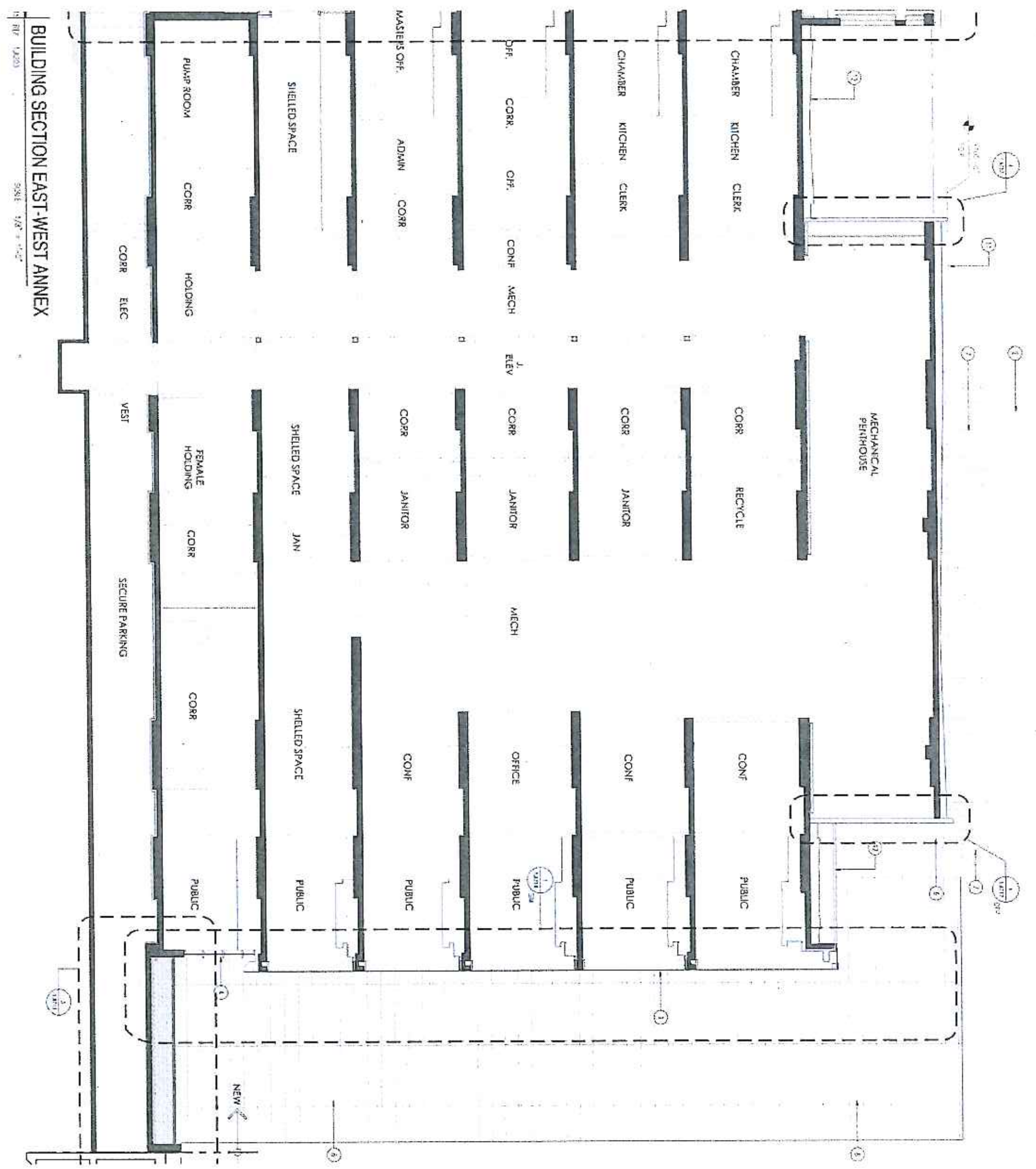
Strength checks on Column D4 and Shear Wall 4 proved members to be adequate to the worst case loading. In addition the overturning moment for seismic was determined to control but was resisted by the stabilizing moment with a Factor of Safety of 17.

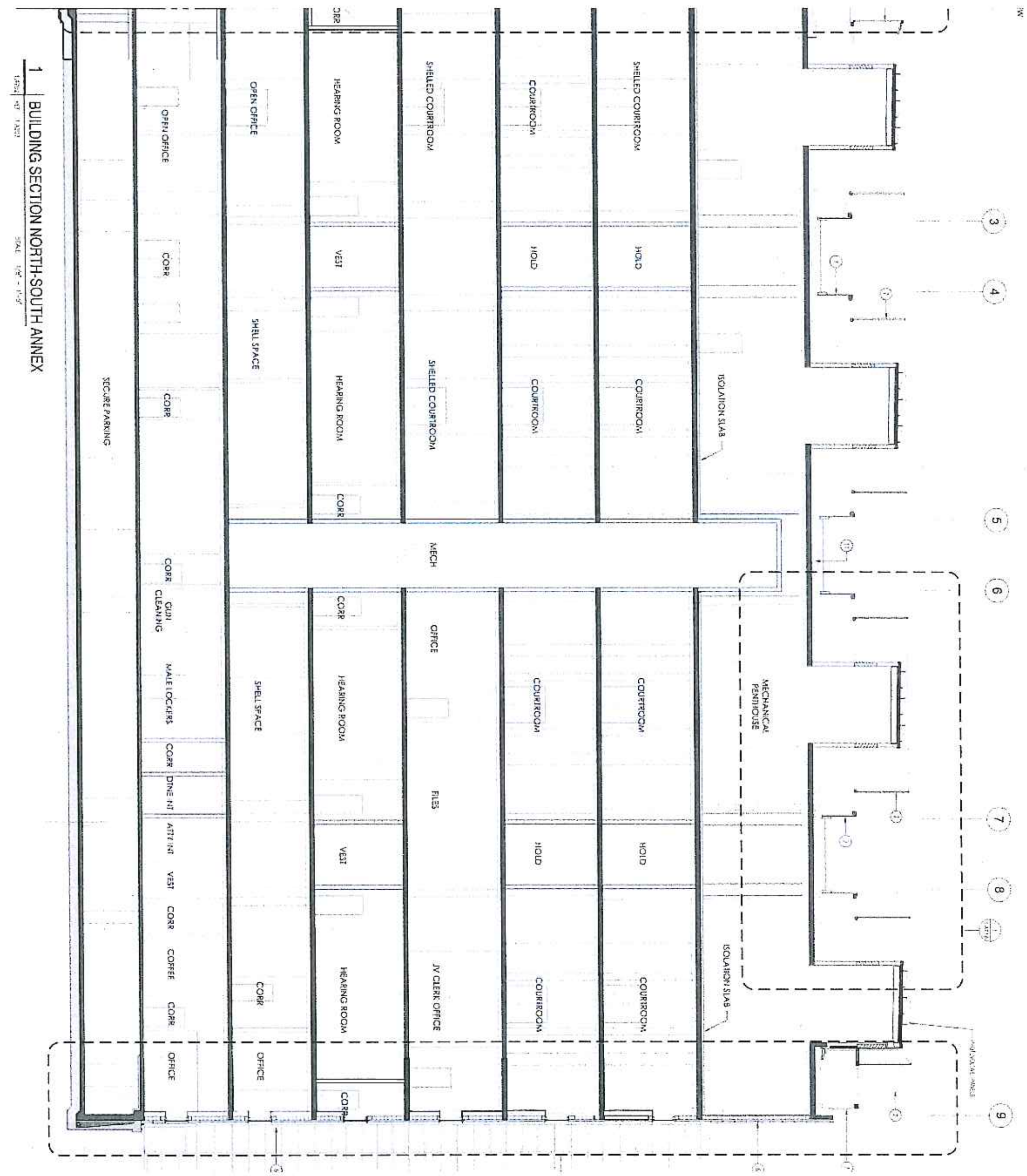
Possible areas for further investigation in this study include incorporating soil forces on the basement wall to be more representative of actual building behavior, using shell elements to model the floor systems, and to investigate the "link beam" adequacy in the shear walls.

Appendix A: Typical Plans



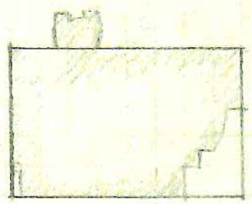



BUILDING SECTION EAST-WEST ANNEX
CHAMBER
KITCHEN
CLERK
CORR
RECYCLE
CONF
PUBLIC
JANITOR
OFFICE
MECH
ELEV
ADMIN
SHELLED SPACE
PUMP ROOM
HOLDING
FEMALE HOLDING
SECURE PARKING
VEST
ELEC
MECHANICAL PENTHOUSE


1 BUILDING SECTION NORTH-SOUTH ANNEX
SECURE PARKING
OPEN OFFICE
CORR
GUN CLEANING
MALE LOCKERS
DINE NT
ATTY NT
VEST
COFFEE
OFFICE
SHELL SPACE
HEARING ROOM
SHELLED COURTROOM
MECH
FILES
JV CLERK OFFICE
COURTROOM
HOLD
ISOLATION SLAB
MECHANICAL PENTHOUSE
3
4
5
6
7
8
9

Appendix B: Lateral Loads

WIND



150' IDEALIZED

180'

$V = 90\,mph$ F.6-1
$I = 1.15$ T.6-1
$K_z = 1.015$ T.6-3
$K_{zt} = 1.0$
$K_d = .85$ T.6-4

$q = .00256 K_z K_{zt} K_d V^2 I$
$= 20.57\ psf$

T_a (SEISMIC) = .81

$1/.81 > 1Hz$; RIGID

$G = .85$

$GC_{pi} = \pm .18$ Fully Enclosed

$P = qGC_p - q_i(GC_{pi})$

C_p

Windward = .8
SIDEWALL = -.7
LEE NORMAL 180' = -.5
NORMAL 150' = -.46

WINDWARD @ 115'

$P = 20.57(.85)(.8) - 20.57(\pm .18)$
$= 13.99 \pm 3.7\ psf$

ROOF = 0 to h/2 = -1.04 N/S
-.948 E/W
> h/2 = -.882 N/S
-.876 E/W

Wind Load Criteria		
Gcpi	0.18	ASCE 7-05 Fig. 6-5
Exposure	B	ASCE 7-05 6.5.6.3
V	90 mph	ASCE 7-05 Fig. 6-1C
I	1.15	ASCE 7-05 Tab 6-1
Kzt	1	ASCE 7-05 6.5.7.1
Kd	0.85	ASCE 7-05 Fig. 6-4

Velocity Presssure Coefficients (Kz) and Velocity Pressures (qz)			
	Height	Kz	qz
Terrace	0	0.570	11.55
1st	15	0.570	11.55
2nd	29	0.692	14.03
3rd	44.5	0.783	15.86
4th	61	0.854	17.31
5th	77.5	0.920	18.65
Penthouse	94	0.972	19.70
Roof	115	1.015	20.57

Design Wind Pressure N/S							
		Distance	Wind Pressure	Internal Pressure		Net Pressure	
				(+) Gcpi	(-) Gcpi	(+) Gcpi	(-) Gcpi
Windward	Terrace	0	7.86	3.70	-3.70	4.15	11.56
	1st	15	7.86	3.70	-3.70	4.15	11.56
	2nd	29	9.54	3.70	-3.70	5.83	13.24
	3rd	44.5	10.79	3.70	-3.70	7.08	14.49
	4th	61	11.77	3.70	-3.70	8.07	15.47
	5th	77.5	12.68	3.70	-3.70	8.98	16.38
	Penthouse	94	13.40	3.70	-3.70	9.69	17.10
	Roof	115	13.99	3.70	-3.70	10.29	17.69
Leeward	All	-	-8.74	3.70	-3.70	-12.45	-5.04
Side Walls	All	-	-12.24	3.70	-3.70	-15.94	-8.54
Roof		0 - 50	-18.19	3.70	-3.70	-21.89	-14.48
		> 50	-14.55	3.70	-3.70	-18.25	-10.85

Wind Force (NS)								
		Trib Below		Trib Above		Story Force	Story Shear	Overturning Moment
	Height	Ht	Area	Ht	Area			
Terrace	0	0	0	7.5	1125.00	18.68	343.97	0.00
1st	15	7.5	1125	7	1050.00	36.11	325.30	541.58
2nd	14	7	1050	7.75	1162.50	40.45	289.19	1173.00
3rd	15.5	7.75	1162.5	8.25	1237.50	46.87	248.75	2085.71
4th	16.5	8.25	1237.5	8.25	1237.50	50.77	201.88	3097.19
5th	16.5	8.25	1237.5	8.25	1237.50	53.03	151.10	4109.45
Penthouse	16.5	8.25	1237.5	10.5	1575.00	62.27	98.08	5853.53
Roof	21	10.5	1575	0	0.00	35.81	35.81	4117.64
Base Shear (k)								343.97
Total Overturning Moment (k-ft)								20978.10

SEISMIC

$S_{DS} = .1664$ FROM TECH1
$S_{D1} = .0816$

$T_a = C_t h_n^x = .02(139)^{.75} = .81$

$T = C_u T_a = 1.7(.81) = 1.38$

$$C_s = \frac{S_{DS}}{R/I} = \frac{.166}{5/1.25} = .0415 < \frac{S_{D1}}{T(R/I)} = \frac{.0816}{1.38(5/1.25)} = .0148$$

$$V = WC_s = 40{,}000k \cdot .0148 = 591k$$

PERIOD'S FROM E-TABS

X: .923 Y: 1.24 BOTH < 1.38, SO ADJUST

V = 884k V = 658k : HAND CALCS

V = 907k V = 681k : ETABS

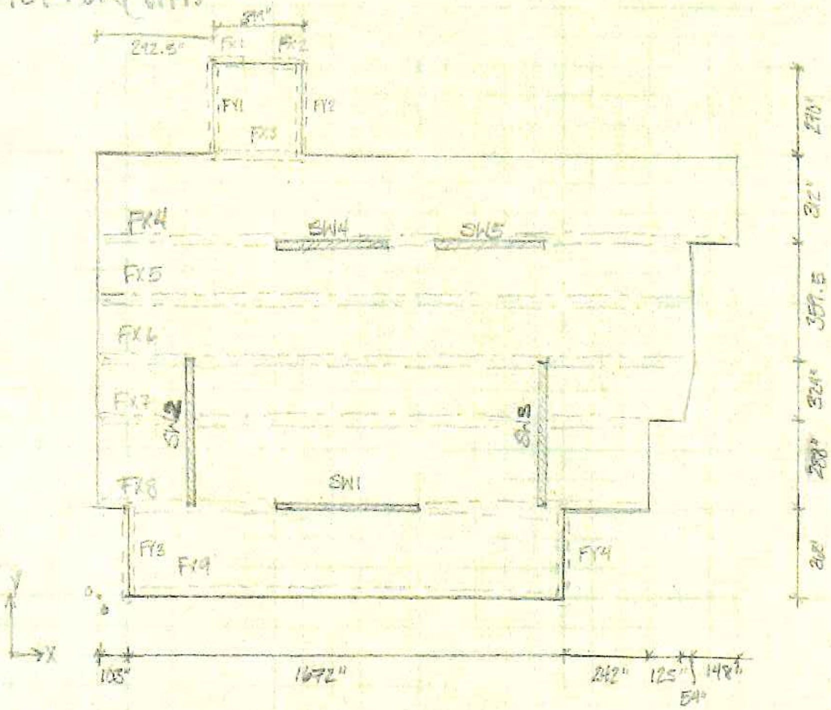
Appendix C: Center of Mass, Center of Rigidity, Torsion

CTR OF MASS & RIGIDITY CHECK

*MASS ASSUMED DISTRIBUTED EVENLY OVER AREA (AS MODELED IN E-TABS)

*GEOMETRY SIMPLIFIED SLIGHTLY FROM ACTUAL FLR. PLAN

1ST FLR (TYP.)



CENTER OF MASS

X: (1672×368)939 + (2017×288)(1008.5) + (2142×324)(1071) + (2196×359.5)(1098) + (2344×312)(1172) + (399×270)(492) / (1672×368) + (2017×288) + (2142×324) + (2196×359.5) + (2344×312) + (399×270)

X = 1047" ETABS: 1108" 5% ERROR

Y: (1672×368)(184) + (2017×288)(512) + (2142×324)(818) + (2196×357.5)(1159.75) + (2344×312)(1495.5) + (399×270)(1780.5) / (1672×368) + (2017×288) + (2142×324) + (2196×357.5) + (2344×312) + (399×270)

Y = 903.8" ETABS: 902.6" 0.1% ERROR

*RELATIVE STIFFNESS DETERMINED FROM APPLYING 1000 k LOAD IN MODEL + OBSERVING SHEAR DISTRIBUTION FOR EACH LEVEL

X-DIRECTION		Y-DIRECTION	
SW1:	.569	SW2:	.508
SW4:	.148	SW3:	.466
SW5:	.142	FY1:	.006
FX1:	.002	FY2:	.004
FX2:	.002	FY3:	.009
FX3:	.002	FY4:	.008
FX4:	.027		
FX5:	.023		
FX6:	.024		
FX7:	.019		
FX8:	.032		
FX9:	.012		

CTR OF RIGIDITY:

X: .508(482") + .466(1794") + .006(815) + .004(714") + .009(103) + .008(1776")

/ Σ RELATIVE K

X = 1099.6"   ETABS: 1142.5"   3.8% ERROR

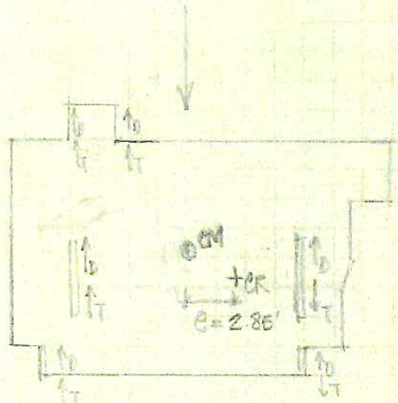
Y: .569(366) + .148(1464) + .142(1464) + .002(2)(1922) + .002(1552) + .027(1482) + .023(1200) + .024(884) + .019(686) + .032(366) + .012(0)

/ Σ RELATIVE K

Y = 758.3"   ETABS: 754.8"   0.4% ERROR

DIRECT SHEAR + TORSIONAL SHEAR

28.21k (FROM SEISMIC TABLE)



$e_a = 9'$

$J = \Sigma R_i d_i^2 = .508(55^2) + .466(54.3^2) + .006(69^2) + .004(35.7^2) + .009(81^2)$
$+ .008(58.2)^2$

$= 2123$

DIRECT SHEAR $= \dfrac{VK_i}{\Sigma K_i}$

TOTAL

$V_T = \dfrac{V(e \pm e_{acc})}{J} d_i R_i$

	DIRECT SHEAR	TOTAL
$V_{SW2} = \dfrac{28.21(2.85+9)}{2123}(55)(.508) = 4.4k$	14.3k	18.7k
$V_{SW3} = \dfrac{28.21(2.85-9)}{2123}(54.3)(.466) = 2.1k$	13.1k	15.2k
$V_{FY1} = \dfrac{28.21(2.85+9)}{2123}(69).006 = .07k$	.17k	.24k
$V_{FY2} = \dfrac{28.21(11.85)}{2123}(35.7).004 = .02k$	.11k	.13k
$V_{FY3} = \dfrac{28.21(11.85)}{2123}(81).009 = .11k$	.25k	.35k
$V_{FY4} = \dfrac{28.21(2.85\ 9)}{2123}(58.2)(.008) = .04k$	.23k	.27k

INHERENT TORSION e_A MAKES THE WORST CASE ALL ADDITIVE TORSION FOR SW3 + FY4

Appendix D: Strength Checks

Input

Material

f'c =	5.0 Ksi	- concrete strength
fy =	60 Ksi	- steel reinforcement yield strength
Es=	29000 Ksi	

wall left end — wall right end

Lw

Wall

Lw =	352 in	- wall length
tw =	12 in	- wall thickness
hw =	168 in	- wall height
cw =	0.8 in	- concrete cover @ wall

Reinforcement

Vertical — # curtains: 2

- Lw								
bar size	spacing	db	As	# bars/curtain	As total	actual spacing	Max Spacing	
#5	12 in	0.625	0.31	29	8.99	11.66	18	Meet max spacing
- wall left end (vertical)								
#5	0.8 in	0.625	0.31	1	0.31	1 in	18	Meet max spacing
- wall right end (vertical)								
#5	0.8 in	0.625	0.31	1	0.31	1 in	18	Meet max spacing
		total # bars/curtain		31				

As=	19.22	ρ_ℓ	0.46%	Meet min reinf
Ac=	4224	ρ minum ACI11.9.9.4	0.43%	

Horizontal — # curtains: 2

- wall (horizontal)				
bar size	spacing	db	As	Max Spacing
#5	12 in	0.625	0.31	18

Meet max spacing

ρ_t	0.43%	Meet min reinf
ρ minum ACI11.9.9.2	0.25%	

Loads

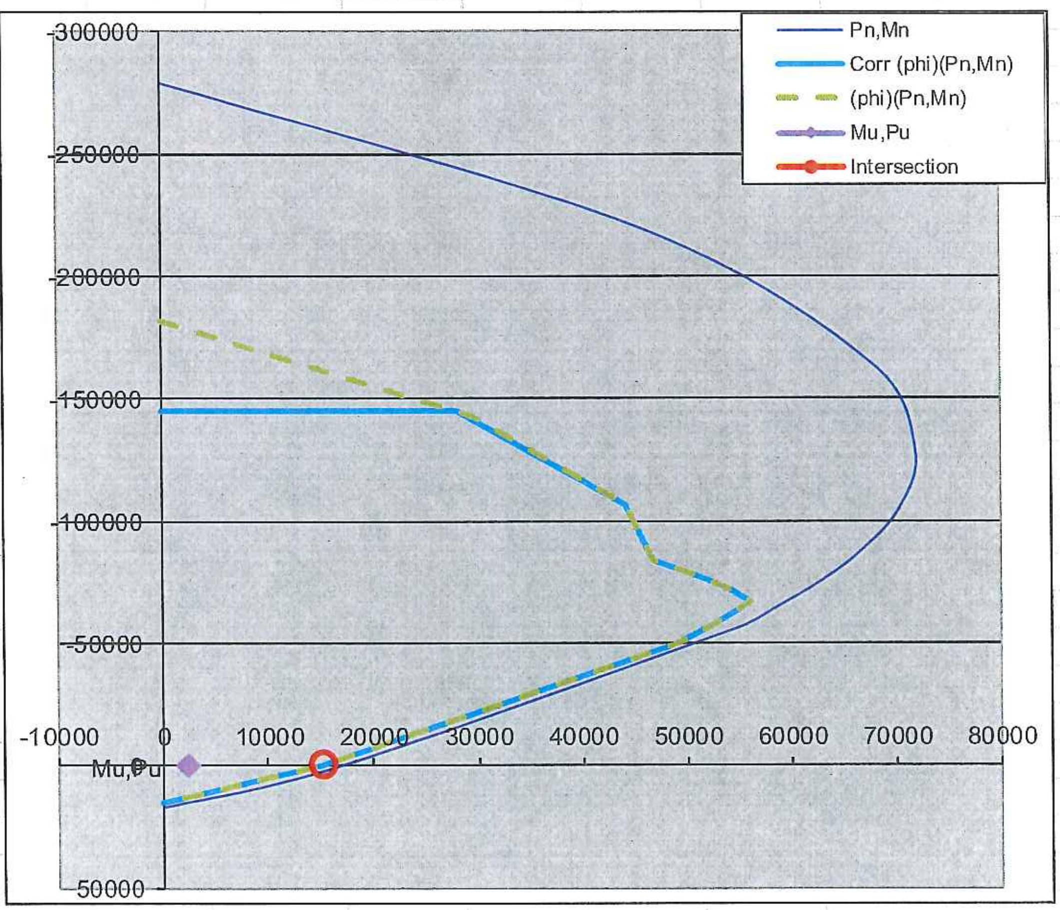
Mu=	26088720	in-lb
Vu=	155290	lb
Pu=	62	kip

Trial and error to find Cu of pure flexure

c_u	23.8	Change untill Pn=0
Pn	0	

Results

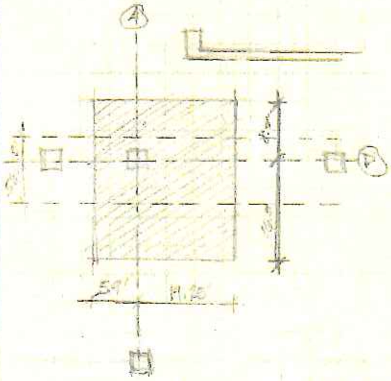
Shear	Vc=	477.89105	kips		
	ϕ0.5Vc=	179.20914	kips	>Vu	Vs not needed
	Av=	0.62	in^2		
	Vs=	0	kips		
	Vn=Vc+Vs	477.89105	<	2389.455 kips	ACI section 11.9.3
	Vn=	477.89105	kips		
	ϕVn=	358.41829	kips		
	Vu<ϕVn		PASS		



DCR= 0.1415 <1 PASS

JAKE WIEST | GRAVITY CHECK

COLUMN D-4



TRIB AREA = 626'
INFL AREA = 2504'

ASSUMED LL = 80psf + 20psf

DESIGN LL = 100psf TYP.
150psf MECH/PENTHOUSE
40psf HOLDING CELLS

LOADING FOR TERRACE - 5th
SLAB = 9" = 150 × 9/12 = 112.5psf
SHALLOW BM = 150 × 8/12 × (7.5 × 25.2) = 18.9k
LL = 100psf

5th $LL_R = .25 + \frac{15}{\sqrt{2504}} = .55$

$P_D = 112.5(626) + 18.9 = 89.3k$
$P_L = 100(626) \times .55 = 34.4k$

4th
$LL_R = .25 + \frac{15}{\sqrt{2 \times 2504}} = .46$

$P_D = 89.3k$
$P_L = 100 \times .46 \times 626 = 28.8k$

3rd
$LL_R = .25 + 15/\sqrt{3 \times 2504} = .42$

$P_D = 89.3k$
$P_L = .42 \times 100 \times 626 = 26.3k$

2nd - TERRACE
$LL_R = .40$
$P_D = 89.3k$
$P_L = .4 \times 100 \times 626 = 25k$

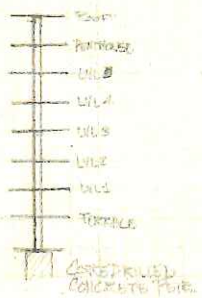
PENTHOUSE
SLAB → 11/12 × 150 = 137.5psf + 4/12 × 110 = 36.7psf (FLOATING LW SLAB)
18.9k → SHALLOW BM
LL = 150psf

$P_D = 174.2 \times 626 + 18.9 = 127.9k$
$P_L = 150 \times 626 = 93.9k$

ROOF (UPPER ROOF NEGLECTED, SLAB OPENING NEGLECTED AS WELL TO BALANCE)
ROOF SNOW LOAD USED OVER ROOF LIVE

SLAB → 10/12 × 150 = 125psf
BM → 14/12 × 150 × (14/12 × 25.2) = 5.9k

$P_D = 125 \times 626 + 5.9 = 84.2k$
$P_L = 22 \times 626 = 13.8k$



$P_u = 1.2(748) + 1.6(272) = 1333\text{k}$

ASSUMING PURE COMPRESSION

D4 → 24×24, $f'_c = 7000\text{psi}$
(8) #10's, GR. 60

$\phi P_n = .85 f'_c (bh - \sum A_{si}) + \sum A_{si} f_s$

$= .65 \times .85(7000)(24 \times 24 - 8 \times 1.27) + 1.27 \times 60{,}000$

$= 2238\text{k} > 1333\text{k}$

$A_{ST,MIN} = .01 A_g = 5.76\text{ in}^2$

$A_{ST,MAX} = .08 A_g = 46\text{ in}^2$

$A_{ST} = 10.2\text{ in}^2$ ✓ ACI 318-08 (10.9.1)

Appendix E: Overturning

$(.9-.2(.166))D = .87D = 40{,}000(.87) = 34800k$

MOMENT: $X\,(N/S) = 180/2 = 90' \times 34600k = 3{,}132{,}000$ k-ft

$Y\,(E/W) = 150/2 = 75' \times 34600k = 2{,}610{,}000$ k-ft

OVERTURNING MOMENT: $M_x \Sigma_0 = 72684.5 \times 2.5 = 181710$ k-ft

$M_y \Sigma L_0 = 54101.93 \times 2.5 = 135{,}255$ k-ft

FS: X: 17

Y: 19

Appendix F: Story Drift and Displacements

QCXE				
Story	δxe	δye	δx	δy
Roof	0.605357	0.094487	2.179285	0.340153
PentHouse	0.499639	0.079016	1.7987	0.284458
Level 5	0.396982	0.061066	1.429135	0.219838
Level 4	0.290352	0.042449	1.045267	0.152816
Level 3	0.187383	0.024896	0.674579	0.089626
Level 2	0.107652	0.013724	0.387547	0.049406
Level 1	0.043227	0.003997	0.155617	0.014389
Terrace	0.005291	-0.00002	0.019048	-7.2E-05

QCYE				
Story	δxe	δye	δx	δy
Roof	-0.02383	0.557425	-0.08578	2.00673
PentHouse	-0.02376	0.470805	-0.08554	1.694898
Level 5	-0.02074	0.383806	-0.07468	1.381702
Level 4	-0.01661	0.289659	-0.05981	1.042772
Level 3	-0.012	0.19547	-0.04319	0.703692
Level 2	-0.00771	0.112964	-0.02776	0.40667
Level 1	-0.00263	0.045682	-0.00947	0.164455
Terrace	0.000615	0.002535	0.002214	0.009126

QCX.3Y				
Story	δxe	δye	δx	δy
Roof	0.613247	0.246227	2.207689	0.886417
PentHouse	0.504422	0.204539	1.815919	0.73634
Level 5	0.399897	0.162199	1.439629	0.583916
Level 4	0.291809	0.118001	1.050512	0.424804
Level 3	0.187685	0.075155	0.675666	0.270558
Level 2	0.107401	0.042281	0.386644	0.152212
Level 1	0.043377	0.016145	0.156157	0.058122
Terrace	0.005833	0.001708	0.020999	0.006149

QC.3XY				
Story	δxe	δye	δx	δy
Roof	0.153797	0.498788	0.553669	1.795637
PentHouse	0.120267	0.412683	0.432961	1.485659
Level 5	0.092277	0.3319	0.332197	1.19484
Level 4	0.065007	0.247217	0.234025	0.889981
Level 3	0.039657	0.163685	0.142765	0.589266
Level 2	0.021461	0.092936	0.07726	0.33457
Level 1	0.009585	0.039101	0.034506	0.140764
Terrace	0.003157	0.005469	0.011365	0.019688

WC1X		
Story	δxw	δyw
Roof	0.217059	0.047819
PentHouse	0.181208	0.040207
Level 5	0.146031	0.031848
Level 4	0.108949	0.023082
Level 3	0.072182	0.014513
Level 2	0.042759	0.008368
Level 1	0.017762	0.002983
Terrace	0.002157	0.000175

WC1Y		
Story	δxw	δyw
Roof	0.008709	0.248536
PentHouse	0.005825	0.206575
Level 5	0.00386	0.168459
Level 4	0.002185	0.127997
Level 3	0.000817	0.086975
Level 2	0.000052	0.050715
Level 1	0.000182	0.022526
Terrace	0.000391	0.003592

WC2XE+		
Story	δxw	δyw
Roof	0.116947	-0.00764
PentHouse	0.096534	-0.00632
Level 5	0.078378	-0.00576
Level 4	0.05932	-0.00507
Level 3	0.040311	-0.00417
Level 2	0.02384	-0.00246
Level 1	0.0107	-0.00151
Terrace	0.00155	-0.0003

WC2XE-		
Story	δxw	δyw
Roof	0.22679	0.077664
PentHouse	0.190216	0.065228
Level 5	0.153651	0.052312
Level 4	0.114891	0.038679
Level 3	0.076296	0.025157
Level 2	0.045065	0.014561
Level 1	0.018553	0.005737
Terrace	0.001954	0.000536

WC2YE+		
Story	δxw	δyw
Roof	-0.04939	0.256462
PentHous	-0.04404	0.217848
Level 5	-0.03651	0.179733
Level 4	-0.02801	0.137841
Level 3	-0.0192	0.094843
Level 2	-0.01165	0.0562
Level 1	-0.00442	0.023316
Terrace	0.000159	0.001262

WC2YE-		
Story	δxw	δyw
Roof	0.049236	0.265784
PentHouse	0.040822	0.221636
Level 5	0.032379	0.180498
Level 4	0.023672	0.136761
Level 3	0.015178	0.092518
Level 2	0.008587	0.053911
Level 1	0.00349	0.023549
Terrace	0.000497	0.003377

WC3		
Story	δxw	δyw
Roof	0.143485	0.234039
PentHouse	0.118308	0.195095
Level 5	0.094825	0.158245
Level 4	0.070451	0.119187
Level 3	0.046495	0.079877
Level 2	0.027224	0.046537
Level 1	0.011716	0.019925
Terrace	0.001832	0.002861

WC4EX+EY+		
Story	δxw	δyw
Roof	0.048999	0.207379
PentHouse	0.038666	0.17591
Level 5	0.030542	0.145397
Level 4	0.022437	0.111862
Level 3	0.014691	0.077378
Level 2	0.008563	0.045693
Level 1	0.00412	0.019211
Terrace	0.001119	0.000982

WC4EX+EY-		
Story	δxw	δyw
Roof	0.126468	0.191036
PentHouse	0.104517	0.159371
Level 5	0.083739	0.129298
Level 4	0.062137	0.097397
Level 3	0.040885	0.065286
Level 2	0.023904	0.038033
Level 1	0.010187	0.016256
Terrace	0.001567	0.00227

WC4EX-EY+		
Story	δxw	δyw
Roof	0.102214	0.158258
PentHous	0.083879	0.131837
Level 5	0.067015	0.106953
Level 4	0.049608	0.080587
Level 3	0.032568	0.054042
Level 2	0.019029	0.031498
Level 1	0.008252	0.013518
Terrace	0.001446	0.001987

WC4EX-EY-		
Story	δxw	δyw
Roof	0.047263	0.259275
PentHouse	0.039287	0.216781
Level 5	0.031368	0.175975
Level 4	0.023295	0.132612
Level 3	0.015456	0.088939
Level 2	0.009235	0.051799
Level 1	0.003972	0.022056
Terrace	0.000554	0.002853

Appendix G: Relative Stiffness

	X-Direction			Y-Direction		
Roof	SW1	593.9	59.3%	Columns	120	12.3%
	F3X	6.5	0.6%	SW2	402.45	41.3%
	F1X	6.2	0.6%	SW3	420.85	43.2%
	F2X	4.72	0.5%	F4Y	7.84	0.8%
	F4X	63.29	6.3%	F3Y	6.59	0.7%
	F5X	87.4	8.7%	F1Y	10.5	1.1%
	F6X	81.73	8.2%	F2Y	6.31	0.6%
	F7X	67.7	6.8%		974.54	
	F8X	37.42	3.7%			
	F9X	51.85	5.2%			
		1000.71				
Penthouse	SW1	575.82	57.4%	SW2	479.69	49.0%
	SW4	70.19	7.0%	SW3	464.58	47.4%
	SW5	70.56	7.0%	F4Y	12.51	1.3%
	F3X	3.8	0.4%	F3Y	9.76	1.0%
	F1X	3.05	0.3%	F1Y	8	0.8%
	F2X	2.47	0.2%	F2Y	5.25	0.5%
	F4X	59.55	5.9%		979.79	
	F5X	41.74	4.2%			
	F6X	43.83	4.4%			
	F7X	33.76	3.4%			
	F8X	56.93	5.7%			
	F9X	42.28	4.2%			
		1003.98				
Level 5	SW1	614.43	60.7%	SW2	470.3	48.8%
	SW4	96.15	9.5%	SW3	468.4	48.6%
	SW5	96.24	9.5%	F4Y	8.84	0.9%
	F3X	2.41	0.2%	F3Y	7.32	0.8%
	F1X	2.14	0.2%	F1Y	5.94	0.6%
	F2X	1.83	0.2%	F2Y	3.61	0.4%
	F4X	41.38	4.1%		964.41	
	F5X	31.48	3.1%			
	F6X	33.87	3.3%			
	F7X	25.68	2.5%			
	F8X	52.45	5.2%			
	F9X	13.35	1.3%			
		1011.41				
Level 4	SW1	221.8	22.0%	SW2	480.41	49.8%
	SW4	250.14	24.8%	SW3	463.18	48.0%
	SW5	253.23	25.1%	F4Y	7.05	0.7%
	F3X	7	0.7%	F3Y	6.12	0.6%
	F1X	9.39	0.9%	F1Y	4.97	0.5%
	F2X	6.32	0.6%	F2Y	3	0.3%
	F4X	120.99	12.0%		964.73	
	F5X	72.82	7.2%			
	F6X	67.43	6.7%			
	F7X	24.11	2.4%			
	F8X	-10.51	-1.0%			
	F9X	-13.13	-1.3%			
		1009.59				

Level 3	SW1	586.33	57.7%	SW2	488.05	50.8%
	SW4	159.91	15.7%	SW3	454.45	47.3%
	SW5	152.55	15.0%	F4Y	5.53	0.6%
	F3X	1.21	0.1%	F3Y	5.05	0.5%
	F1X	1.14	0.1%	F1Y	4.43	0.5%
	F2X	1.05	0.1%	F2Y	2.6	0.3%
	F4X	20.93	2.1%		960.11	
	F5X	16.9	1.7%			
	F6X	19.33	1.9%			
	F7X	14.82	1.5%			
	F8X	32.58	3.2%			
	F9X	9.75	1.0%			
		1016.5				
Level 2	SW1	621.68	62.0%	SW2	512.48	53.5%
	SW4	125.35	12.5%	SW3	425.57	44.4%
	SW5	117.67	11.7%	F4Y	6.84	0.7%
	F3X	1.53	0.2%	F3Y	5.7	0.6%
	F1X	1.57	0.2%	F1Y	4.59	0.5%
	F2X	1.61	0.2%	F2Y	2.6	0.3%
	F4X	26.7	2.7%		957.78	
	F5X	21.38	2.1%			
	F6X	22.85	2.3%			
	F7X	16.14	1.6%			
	F8X	37.32	3.7%			
	F9X	9.18	0.9%			
		1002.98				
Level 1	SW1	569.47	56.9%	SW2	448.41	50.8%
	SW4	148.46	14.8%	SW3	410.66	46.5%
	SW5	141.83	14.2%	F4Y	8.04	0.9%
	F3X	1.54	0.2%	F3Y	7.06	0.8%
	F1X	1.62	0.2%	F1Y	4.92	0.6%
	F2X	1.71	0.2%	F2Y	3.12	0.4%
	F4X	27.07	2.7%		882.21	
	F5X	22.76	2.3%			
	F6X	23.61	2.4%			
	F7X	18.72	1.9%			
	F8X	31.69	3.2%			
	F9X	12.26	1.2%			
		1000.74				
Terarce	SW1	122.34	12.3%	SW2	113.06	11.9%
	SW4	80.32	8.1%	SW3	109.17	11.5%
	SW5	102.68	10.3%	F4Y	1.68	0.2%
	BASEMEN	608.56	61.1%	F3Y	3.48	0.4%
	F3X	0.9	0.1%	F1Y	1.45	0.2%
	F1X	0.89	0.1%	F2Y	0.96	0.1%
	F2X	0.94	0.1%	Basement	718.63	75.8%
	F4X	15.4	1.5%		948.43	
	F5X	15.02	1.5%			
	F6X	12.76	1.3%			
	F7X	13.55	1.4%			
	F8X	11.38	1.1%			
	F9X	11.14	1.1%			
		995.88				