IAC/InterActiveCorp Headquarters

New York, NY

Technical Report #3

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EXECUTIVE SUMMARY

The primary lateral system for the IAC Headquarters is shear walls located around the stairs and elevator core. These walls provide resistance to lateral wind and seismic loads and are interconnected to form a core that can effectively resist these loads.

For the purposes of this report, the lateral system was modeled using ETABS. Each floor was created within the program so that the loads would act at the center of mass. Shear walls were modeled independently of the gravity system in order to determine the capabilities of the main lateral force resisting system. Wind and seismic forces were added to the building based on the forces computed in Technical Report #1. In order to simplify the model, only the shear walls and floor plates were modeled.

A number of hand calculations were also performed in order to evaluate the effectiveness of the model. Using basic equations for shear walls, relative stiffnesses and deflections were determined. Additionally, torsional shear was very important to consider in this analysis because of the building's tendency to twist counter-clockwise. In order for these calculations to be possible, the effect of coupling beams were neglected and the shear walls were treated as individual walls independent of one another. Also, because the shear walls were the same at every level except the first and eleventh stories, the typical layout, as shown below, was evaluated for the entire height of the building, neglecting the changes at these two floors. This created a simplified, conservative approach to checking the shear wall behavior.

As a result of these analyses, torsion was confirmed to play a major role in the behavior of the building. Additionally, all of the shear walls passed shear capacity checks and drift checks. Further discussion of the lateral system of the IAC Headquarters will follow in this report.

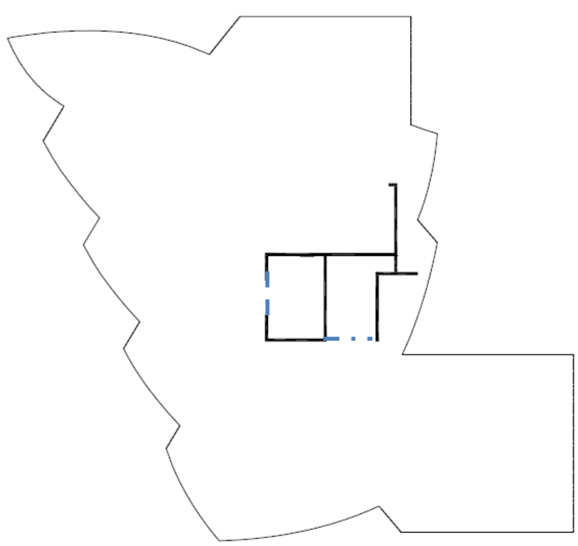

Figure 1: Typical shear wall layout used for all hand calculations

INTRODUCTION

The IAC Headquarters is an 11-story office building that serves InterActiveCorp, an internet and media conglomerate. It is located on West 18th Street in the Chelsea neighborhood of Manhattan and is positioned along the Hudson River. At approximately 130,000 sq ft in size and 150 feet high, the IAC/InterActiveCorp Headquarters stands out along the New York City skyline because of its unique sculptural shape.

The existing structure for the IAC Headquarters is a reinforced concrete flat-plate floor system supported by circular columns. Many of these columns are slanted to achieve a superior interior space. Shear walls provide the bulk of the lateral resistance and are positioned in a core around the elevators and stairs.

Included in this report is a description of the existing conditions, the codes and design standards used, loads and load cases used, as well as an in-depth analysis of the shear wall lateral resisting system in the IAC/InterActiveCorp Headquarters. This analysis will focus on deflection, torsion, overturning, and strength checks. This report incorporates both an ETABS computer model and hand calculations.

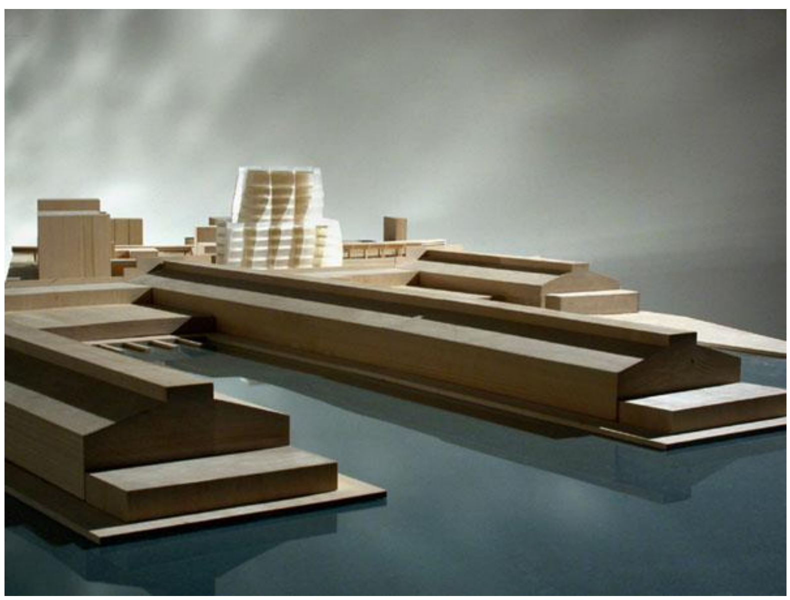

Figure 2: Model of the location of IAC Headquarters in relation to Chelsea Piers

EXISTING STRUCTURAL SYSTEM

Floor System

The structure of IAC/InterActiveCorp Headquarters is a cast-in-place two-way concrete flat plate system. This type of system is primarily used in residential construction because it allows for ease of coordination between trades. More importantly, however, it allows the designer to place columns with relative ease in locations that would optimize the interior space. Despite the advantages of a flat plate system, it is, nevertheless, fairly unusual that this commercial building was designed by this method.

The slab thickness for the first through fifth floors is 12" with primarily #5 @ 12" o.c. top and bottom bars in the 5000 psi strength concrete. Additional top and bottom rebar is placed at the columns and midspans of the room where necessary. At the sixth floor, where the building is set back (leaving space for an outdoor terrace), the slab thickness is 24". The concrete strength is 5000 psi as well, but the top and bottom reinforcing bars are typically #7 @ 12" o.c. It is at this location that the column layout changes much more radically. This thicker slab acts as a transfer diaphragm, which, in addition to supporting vertical live, dead, and snow loads, transfers lateral forces. Lateral forces, such as wind and seismic, may be transferred through the slab. Additionally, where columns are no longer stacked on top of each other, the slab must act as a transfer to carry loads from the upper columns to the lower ones. The seventh through roof levels have similar slab properties to the first through fifth floors, except that the upper floors have a slab thickness of 14". An unusual aspect of the slab reinforcing details is that, unlike typical American Concrete Institute standard details which involves rotating rebar to match specific edge angles, the structural designers chose to design the reinforcing steel in the north-south and east-west orthogonal directions. This was done in an effort to improve the constructability of the building by eliminating the necessity to rotate rebar in various directions because of the unusual edge shape. Because of the use of additional top and bottom bars in necessary locations and the overall uniformity of the bar layout, it seems that orienting the bars in the orthogonal directions is a plausible solution.

Though the building is primarily concrete, some steel shapes are used throughout the building to add additional stability. Steel hollow structural sections (HSS 12x4x1/2) act as elevator rail support posts on the ground floor and S8x18.4 shapes are used for the same purpose on the upper levels. Hollow structural sections are also used on the 11th floor as bracing.

Gravity System:

While the IAC building has a fairly uniform design among floors, each of the structural floor plans differ slightly because of the gradual building setback, including a more noticeable setback at the sixth floor. In order to accommodate this setback and allow for columns to be placed in desirable locations, most of the columns in the building's superstructure are sloped, making the building tend to twist counter-clockwise under its own weight. This causes significant torsional rotation, which needed to be taken into consideration during the initial design process. In fact, a number of short-term and long-term studies were made through three-dimensional computer simulations to design the lateral system and predict curtain wall displacements. Torsion will be analyzed later in this report using basic hand calculation methods.

The column strength for the building is 5950 psi. The reason for this seemingly random column strength is because buildings constructed in New York City with strengths greater than 6000 psi need to undergo more frequent test cylinders; therefore, by specifying a strength just under 6000 psi, fewer tests would need to be conducted. The columns in the basement are primarily 28" in diameter for the perimeter columns and 34" to 38" in diameter for the interior columns. This range of column diameters is fairly consistent throughout the ground through fifth floors, but at the sixth floor the sizes are reduced to 20" to 24". Columns are typically spaced between 25 and 30 feet apart.

Figure 3: Flat-plate system during construction of the IAC Headquarters

At the sixth floor, the building setbacks become more distinct and, therefore, the columns begin to slope much more significantly in an effort to keep the columns along the perimeter and out of the way of the open office space. In addition a number of columns are displaced at the 6th floor level, resulting in column offsets up to 8'-0" long.

Figure 3, shown above, effectively displays the coordination of the flat plate slab and the circular columns along the perimeter.

Lateral System

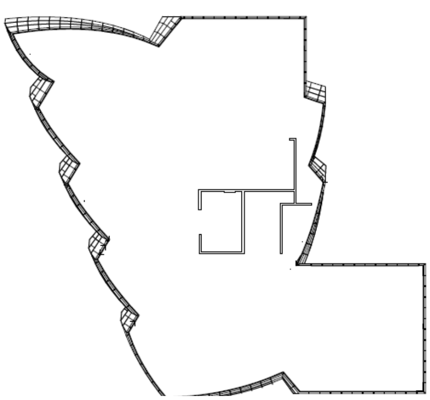

Figure 4: Typical shear wall layout (4th floor)

The columns carry the gravity loads while the shear walls that encase the elevator and stair core carry the lateral forces. These shear walls tend to be between 12" and 14" thick. This core, with numerous shear walls acting in each direction, works together with the reinforced slab to carry wind and seismic lateral loads. The shear walls typically span from the cellar level up to the roof. Figure 4, to the left, shows the basic layout for shear walls. In addition to this shear wall core, the slab acts as a diaphragm in order to help distribute lateral loads. This seems necessary because the shear wall core is so concentrated and would likely be ineffective without the contribution of the slab to distribute loads across the entire plan. In the case of the IAC Headquarters, the shear walls were most likely placed around the elevator and stair cores for aesthetic reasons. In other words, the architect did not want to interrupt the open office layout with intermittent shear walls.

Roof System

The roof is composed of 14" thick, 5000 psi concrete. Twenty-inch diameter columns support the roof along the perimeter, along with 14x14 inch posts intermittently positioned to support mechanical equipment. To provide additional reinforcement for the roof level, HSS 10X10X1/2" square tubes were used on the eleventh floor (mechanical mezzanine level) along the perimeter of the building. A fairly large window washing unit to service the entire building facade is located on the roof; however, information has not yet been found providing the unit's weight. A CMU wall and steel W-shapes are also used on the eleventh floor mechanical mezzanine level to support the mechanical equipment.

Foundation System

There is one below-grade basement level in the IAC building with a slab thickness of 24 inches. It was designed as a pressure slab in order to resist hydraulic uplift forces. A 48" thick structural mat supports the building core. This core mat is primarily reinforced at the top and bottom by #9's and #11's at 6" on center. In order to oppose lateral forces from the soil, the foundation wall is 18" thick with #4 bars primarily as reinforcement. All of the concrete in the foundation is 5000 psi concrete.

The gravity columns are supported on concrete-filled steel pipe piles (with a conical tip, as agreed upon with NYSDEC because of environmental sensitivity). These piles have a 175-ton capacity to provide the required axial capacity. There are also twenty-three 18" diameter caissons that end bear on the bedrock. Because the building is located below the 100-year flood elevation, waterproofing was a major concern. In addition, a hydraulic flood gate was designed to seal the entrance ramp of the parking garage if necessary. The site was also contaminated from a previous ConEdison Manufactured Gas Plant facility, so containment was very important.

Codes & Design Standards

Applied to original design:

New York City Building Code with amendments (2003), Chapter 16- Structural Design

American Concrete Institute (ACI 318-99), Building Code Requirements for Structural Concrete

Substituted for thesis analysis:

American Society for Civil Engineers (ASCE-7-05), Minimum Design Loads for Buildings and Other Structures, 2005

American Concrete Institute (ACI 318-08), Building Code Requirements for Structural Concrete

Material Strength Requirement Summary:

Cast-in-place Concrete

- Foundations: 5000 psi
- Formed Slabs: 5000 psi
- Columns & Walls: 5950 psi
- Reinforcement: 60 ksi

Structural Steel

- Rolled Shapes: 50 ksi
- Connection Material: 36 ksi

Masonry

- Compressive strength: 1500 psi
- Reinforcing: 60 ksi

LOADS & LOAD CASES

The loads used for this analysis were taken from the wind and seismic forces determined in Technical Report #1. The following three figures summarize these results. See Appendix A for the detailed calculations of lateral loads.

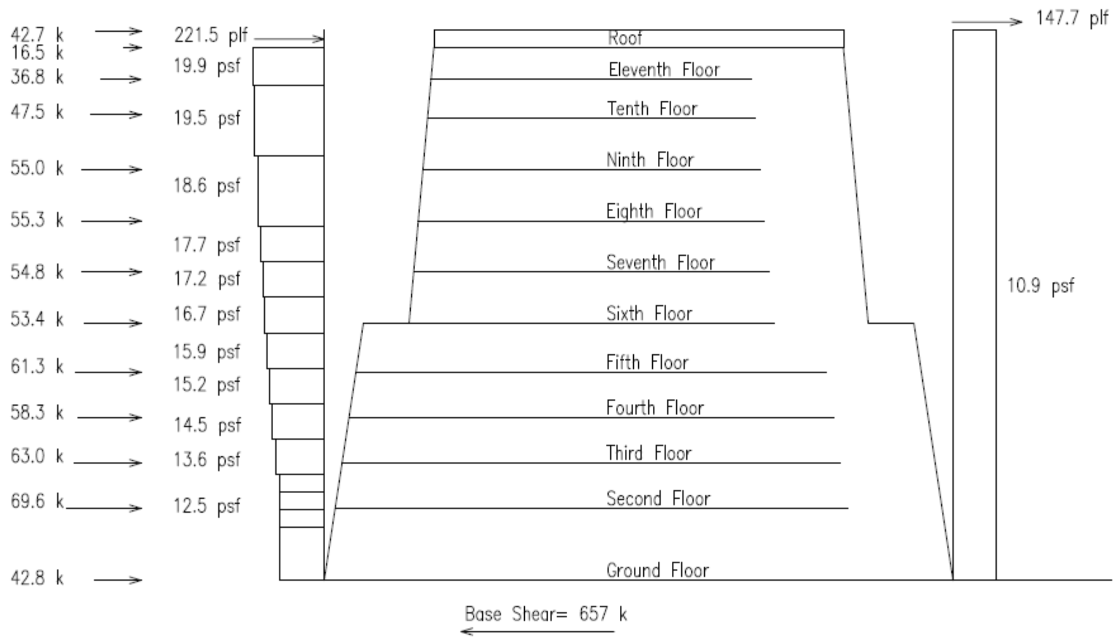

Figure 5: Wind Diagram using ASCE7-05 - In East/West wind direction

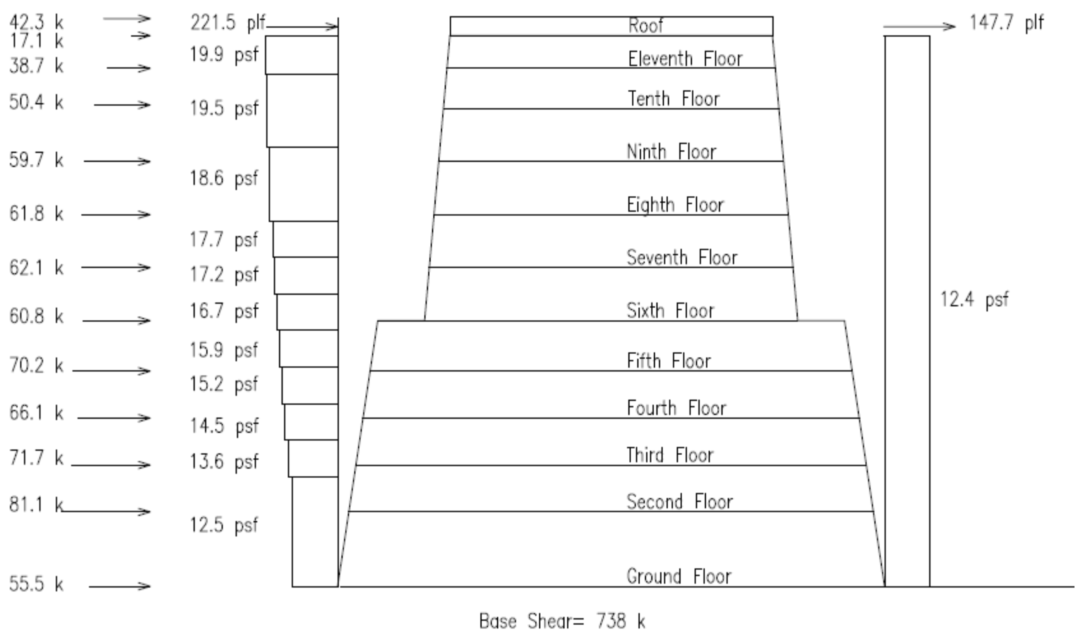

Figure 6: Wind Diagram using ASCE7-05 - In North/South wind

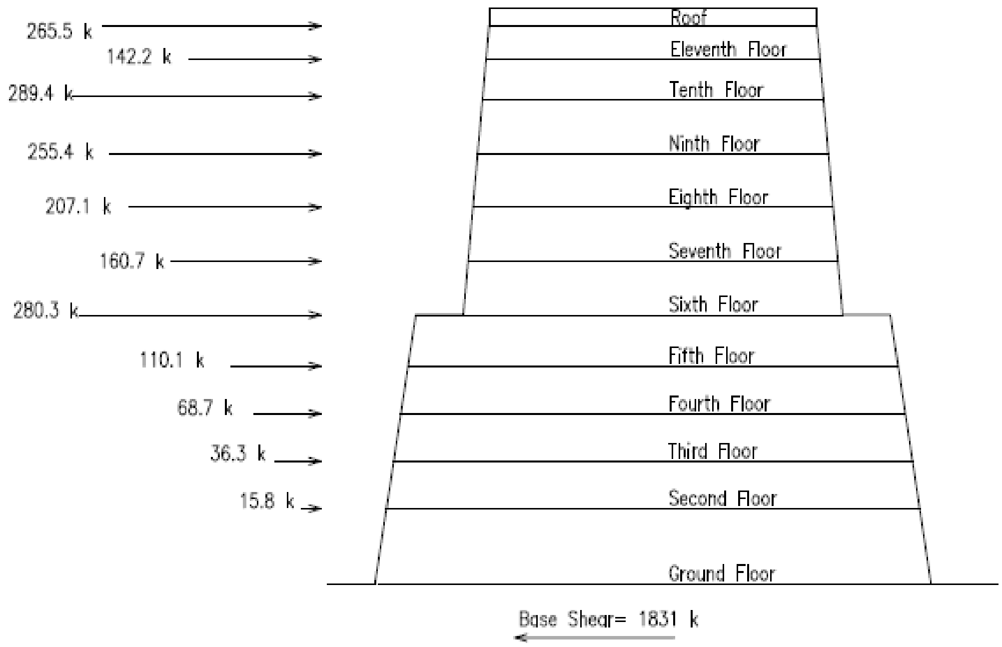

Figure 7: Seismic force diagram using ASCE-7-05

The load combinations considered for this report were taken from ASCE 7-05. These LRFD load combinations include:

1.4 (D+F)
1.2 (D+F+T) + 1.6 (L+H) + 0.5 (Lr or S or R)
1.2 D + 1.6 (Lr or S or R) + (L or .8W)
1.2 D + 1.6 W + L + .5 (Lr or S or R)
1.2 D + 1.0 E + L + .2S
.9 D + 1.6 W +1.6 H
.9 D + 1.0 E + 1.6 H

where the coefficients stand for:

D= dead load	Lr= roof live load	W= wind load
E= earthquake load	L= live load	T= self-straining force
R= rain load	S= snow load	

F= load due to fluids with well-defined pressures and maximum heights
H= load due to lateral earth pressure, ground water pressure, or pressure of bulk materials

Because many of the factors do not apply to lateral design, only combinations that involve dead, wind and earthquake loads were used for this report. The controlling equation for deflection was: 1.2D + 1.0E+L+.2S (or 1.2D+1.0E in this case).

Load Path

Shear walls resist the horizontal forces acting on a building, primarily through wind and seismic loading, while the floor essentially acts as a beam to transfer these lateral loads to the shear walls. These walls act as cantilevered beams fixed at their base, by carrying loads down to the foundation. Shear walls are subject to variable shear, bending and compression forces.

Relative Stiffnesses

The shear that each shear wall carries is dependent on the relative stiffness of each element. This theory of relative stiffness distributes the shear force acting on the diaphragm to the walls based on their stiffness and, thus, their overall dimensions.

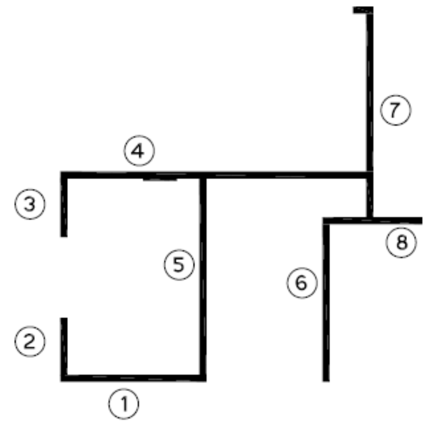

Figure 8: Typical shear walls labeled for calculations

The following two tables illustrate the relative stiffnesses for the walls on the second floor and the figure to the right shows the corresponding labels for the shear walls. It is important to note that because the actual shear walls are all connected and acting together, the hand method shown below is not entirely accurate. Instead, it provides an adequate estimation of the stiffnesses.

2nd Floor										
		Arbitrary Force	h	d	t	delta flexural	delta shear	Deflection	Relative Stiffness	
	Wall2	1000000	309	105	12	1.932	0.167	2.099	0.017	
	Wall3	1000000	309	108	12	1.776	0.163	1.938	0.019	
	Wall5	1000000	309	351	12	0.052	0.050	0.102	0.354	
	Wall6	1000000	309	276	12	0.106	0.064	0.170	0.212	
	Wall7	1000000	309	372	12	0.043	0.047	0.091	0.398	
								Sum of Ri:		
Ec	4.40E+06									
Er	1.76E+06									

Figure 9: Relative stiffnesses of walls acting in N-S direction on the 2nd Story

	Arbitrary Force	h	d	t	delta flexural	delta shear	Deflection	Relative Stiffness
Wall1	1000000	309	240	12	0.162	0.073	0.235	0.160
Wall4	1000000	309	528	12	0.015	0.033	0.048	0.775
Wall8	1000000	309	168	12	0.472	0.105	0.576	0.065
							Sum of Ri:	

Figure 10: Relative stiffnesses of walls acting in E-W direction on the 2nd Story

These relative stiffness values were determined by applying an arbitrary force to the diaphragm. Using Equation (1) below for the deflection of a cantilever, the stiffness of each wall was found by taking the reciprocal of the delta cantilever solution. Once the wall stiffnesses were found, relative stiffnesses were calculated using the second equation below. The relative stiffnesses for other floors are available in Appendix A.

$$\Delta_{cant} = \frac{Ph^3}{3EI} + \frac{1.2Ph}{E_r A} \qquad \text{Eqn (1)}$$

Relative Stiffness for Element i = $\frac{R_i}{\sum R}$ Eqn (2)

ETABS Model

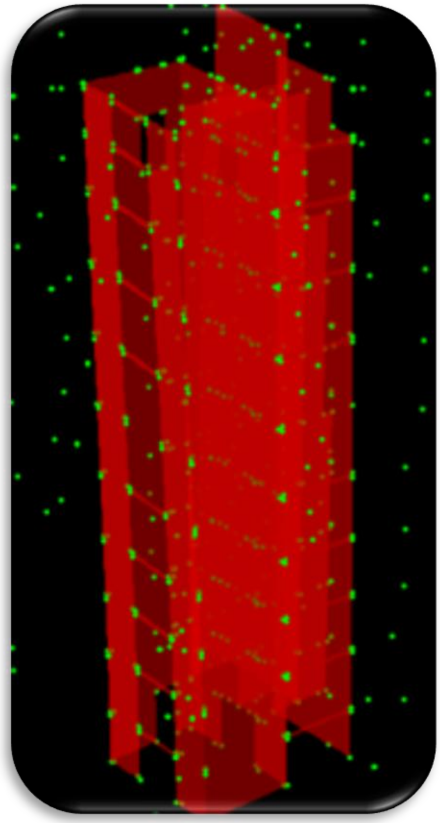

Figure 11: 3-D shear wall system for the IAC Headquarters

As previously mentioned, the lateral system for the IAC Headquarters was modeled using ETABS, a three-dimensional building analysis and design software program.

The concrete shear wall core and floor diaphragms were modeled independent of the remainder of the structural system in order to simply analyze the lateral system.

For ease, the floors' edges were drawn without curves; however, the mass is very comparable to that of the actual design so that the center of mass is accurate. Additionally, major floor openings such as elevator and stair openings were modeled.

The floors in the model were designed without properties; thus, each floor essentially behaved as a place holder in order to determine the center of mass and pressure. This caused potential problems when there were openings between two shear walls. In a real case the shear walls would have been braced somewhat at each floor location by the slab. However, these dummy floors did not provide the additional bracing that is present in the actual design.

The earthquake loads were applied to the center of mass at each level, while the wind loads were applied to the center of pressure. Sample calculations of these center points are available in Appendix A.

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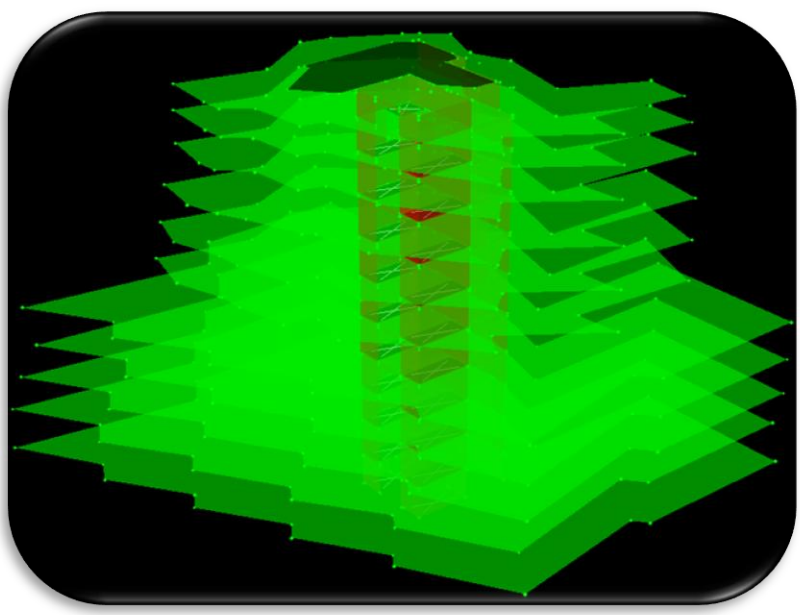


Figure 12: Overall ETABS model with floors shown in green and shear walls in red

Analysis

Torsion

According to the ETABS model that was created, torsion is the controlling mode for the building. This is not surprising because of the concentrated core and the large floor diaphragms. In addition, the center of pressure acts outside of the core on each floor, causing an eccentricity and, thus, a torsional tendency in the building. For this reason, it is especially important to account for the effects of torsional shear in addition to direct shear. An analysis was performed in order to determine the torsional shear for the second floor and also the overall building torsion caused by seismic forces. An inherent torsional moment was determined by multiplying the distance between the center of rigidity and the center of mass, and was added to an accidental torsional moment as determined from section 12.8 of ASCE 7-05 in order to find the overall building torsion.

	$M_{tot\text{- }N/S}$
	431.5961
	1055.717
	2047.578
	3322.474
	8643.249
	4596.368
	5895.395
	7181.529
	8034.142
	3918.232
	7294.292
Sum:	52420.57

	$M_{tot\text{- }E/W}$
	195.8226
	415.9284
	824.2397
	1690.755
	4471.252
	2258.853
	3294.176
	4486.527
	5224.321
	2540.806
	4675.322
Sum:	30078

As expected, the torsional moment for the building in the North/South direction is greater than that of the East/West direction. This is because the difference between the center of mass and center of rigidity is much greater for the N/S direction. This can be seen in the figure below which shows the rough locations of these centers in relation to the shear walls.

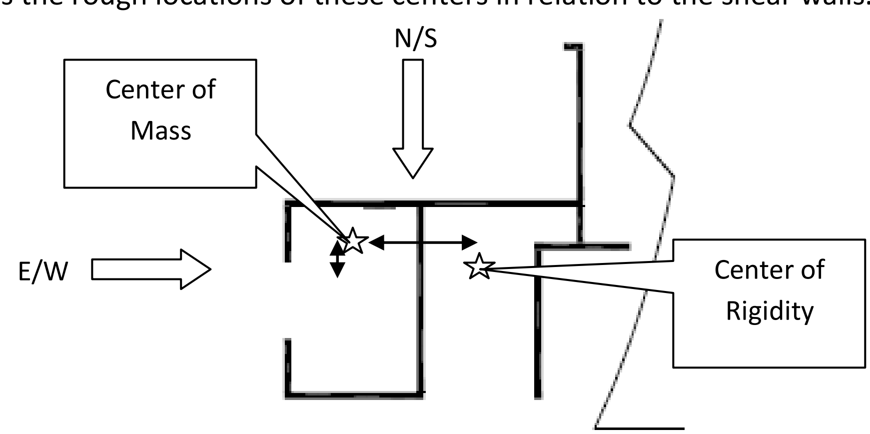


Figure 1: Floor plan showing general layout of the center of mass and center of rigidity

Using the equation shown below for torsional shear, the total shear of the walls was determined by adding the direct shear to the torsional shear. It is important to consider torsional shear because the eccentricity between center of mass and center of rigidity causes an increase in the shear wall that must be taken into consideration. The table below shows the total shear found in each of the walls on the second story. The torsional shears for other floors can be found in Appendix A.

$$V_i = \frac{V_{tot}\, e d_i R_i}{J} \qquad \text{Eqn (3)}$$

V_i= torsional shear of element i
V_{tot}= story shear
e= distances from center of mass to center of rigidity
d_i= distance from element i to center of rigidity
R_i= relative stiffness of element i
J= torsional moment of inertia ($\sum R_i * d_i^2$)

N/S DIRECTION											
Wall	Rel. Stiffness	Story Shear	Location of Wall in ETABS	Dist. fr. Center of Rigidity (da)		Dist fr COM to COR (e)	Torsional Shear	Direct Shear	Total Shear		Ridi^2
1	E-W			252.739							10220.32
2	0.017	1831.37	2856.36	392.656		266.235	77.899603	31.13329	109.03289		2621.038
3	0.019	1831.37	2856.36	392.656		266.235	87.064262	34.79603	121.86029		2929.396
4	E-W			109.501							9292.613
5	0.354	1831.37	3111.43	137.586		266.235	568.39676	648.305	1216.7017		6701.187
6	0.212	1831.37	3317.6	68.584		266.235	169.68082	388.2504	557.93126		997.1982
7	0.398	1831.37	3399	149.984		266.235	696.62987	728.8853	1425.5151		8953.09
8	E-W			31.631							65.03381
									3431.0413	J	41779.88

E/W DIRECTION											
Wall	Rel. Stiffness	Story Shear	Location of Wall in ETABS	Dist. fr. Center of Rigidity		Dist fr COM to COR	Torsional Shear	Direct Shear	Total Shear		Ridi^2
1	0.16	1831.37	3277.03	252.739		32.577	57.744719	293.0192	350.76392		10220.32
2	N/S			392.656							2621.038
3	N/S			392.656							2929.396
4	0.775	1831.37	3639.27	109.501		32.577	121.18247	1419.312	1540.4942		9292.613
5	N/S			137.586							6701.187
6	N/S			68.584							997.1982
7	N/S			149.984							8953.09
8	0.065	1831.37	3561.4	31.631		32.577	2.9359341	119.0391	121.97498		65.03381
										J	41779.88

Figures 14&15: Calculated total shear in each of the walls in the N/S and E/W directions for the 2nd story

Deflection & Story Drift

The deflection values for the IAC Headquarters were taken from the ETABS model at the center of mass of each floor, in accordance with section 12.8.6 of ASCE7-05. These story drifts were compared to Table 12.12-1 of ASCE7-05 for seismic. Deflections due to wind loads were compared to the L/400 requirement that is a design standard based on the 1968 Structural Engineering Handbook by Gaylord & Gaylord.

The shear walls for the IAC Headquarters passed both the story drift and overall deflection checks for both wind and seismic loading according to the ETABS model; however, it failed with the deflection determined from hand calculations. This can be attributed to the simplified approach. This method involved using the Δ_{cant} equation and treating the walls separately. An example of this method for the story displacement of the second story can be seen in the figures to the right.

1st Floor - (2nd Story)										
		Shear (k)	h	d	t	delta flexural (in)	delta shear (in)	Deflection (in)	Relative Stiffness	
	Wall2	64.08201	143	105	12	0.012	0.005	0.017	0.035	
	Wall3	68.58856	143	108	12	0.012	0.005	0.017	0.037	
	Wall5	609.1713	143	351	12	0.003	0.014	0.017	0.333	
	Wall6	430.8147	143	276	12	0.005	0.013	0.017	0.235	
	Wall7	658.7134	143	372	12	0.003	0.014	0.017	0.360	
Ec	4.40E+06		Story Shear	1831.37						
Er	1.76E+06									

	Shear	h	d	t	delta flexural	delta shear	Deflection	Relative Stiffness
Wall1	23.22113	1719	240	12	0.65	0.01	0.66	0.087
Wall4	234.2951	1719	528	12	0.61	0.04	0.66	0.882
Wall8	8.02381	1719	168	12	0.65	0.00	0.66	0.030
		Story Shear	265.54					

Figures 16&17: Calculated story drifts in each of the walls in the N/S and E/W directions for the 2nd story

Drift does not tend to control for shear walls, so it is not surprising that the deflections tended to be very minor. In addition, because the main mode of motion for the building is torsion, the system would be more likely to twist than to translate and cause substantial deflections. More information regarding the calculations and checks can be found in Appendix A.

OVERTURNING

The overturning moment plays a crucial role in the design of the foundations. It also would affect the columns. Figures 18 and 19 below show the overturning moments determined based on 1.6*wind and 1.0*earthquake factors. The combination of poor soil and probable uplift led the geotechnical engineers to design a deep foundation system consisting of both piles and mini-caissons. This was because piles have a relatively low uplift capacity. In order to accommodate hydrostatic uplift and uplift associated with an overturning, tie-down anchors were installed within each mini-caisson.

Force Calcs		N/S	Force (k)		E/W	Force (k)	Overturning Moment (k-ft)	
N/S	(psf*height)	width	N/S	(psf*height)	width	E/W	N/S	E/W
1st	253.233	219.000	88.733	237.978	180.000	68.538	0.000	0.000
2nd	413.967	196.000	129.820	389.092	179.000	111.436	2639.239	2265.490
3rd	373.158	192.000	114.634	353.908	178.000	100.793	3802.418	3343.307
4th	346.533	190.750	105.762	327.283	178.000	93.210	4865.051	4287.673
5th	372.025	188.750	112.352	352.025	174.000	98.004	6609.642	5765.561
6th	408.850	148.750	97.306	387.600	137.750	85.427	7079.033	6214.817
7th	429.975	144.500	99.410	408.225	134.250	87.687	8665.589	7643.652
8th	444.475	139.000	98.851	422.725	130.750	88.434	10058.114	8998.167
9th	452.575	132.000	95.584	430.825	127.750	88.061	11103.975	10230.003
10th	406.725	124.000	80.694	387.600	122.500	75.970	10550.772	9933.025
11th	321.467	120.500	61.979	306.467	120.000	58.842	8785.491	8340.797
Roof	145.350	117.750	27.384	138.600	119.000	26.389	4128.129	3978.208
Parapet	359.200	117.750	67.673	359.200	119.000	68.392	10370.930	10652.004
		Sforces	1180.182		Sum:	1051.181	88658.382	81652.706

Figures 18: Calculated overturning moment due to 1.6Wind

Level	Story Weight wx (kips)	Height hx (ft)	wxhx^k	Lateral Force, Fx (kips)	Story Shear, Vx (kips)	Overturning Moment (k-ft)
R	1578121.659	150.667	35824064631.840	265.536	265.536	40007.411
11	955644.492	141.667	19179254948.404	142.161	407.697	20139.460
10	2286583.201	130.667	39040615434.008	289.378	697.074	37812.020
9	2553745.218	116.167	34462007443.656	255.440	952.515	29673.616
8	2702631.161	101.667	27934697802.422	207.058	1159.573	21050.915
7	2853953.922	87.167	21684420843.582	160.730	1320.302	14010.275
6	7160799.571	72.667	37812197372.251	280.272	1600.575	20366.459
5	4289409.942	58.833	14847195080.803	110.051	1710.626	6474.649
4	4377194.891	46.000	9262144388.888	68.653	1779.279	3158.041
3	4454479.052	33.167	4900051678.190	36.320	1815.599	1204.622
2	5145980.852	20.333	2127570218.785	15.770	1831.369	320.657
1	1159597.372	0.000	0.000	0.000	1831.369	0.000
	39518141.333	959.000	247074219842.828	1831.369		194218.126

Figures 19: Calculated overturning moment due to 1.0Earthquake

Shear Capacity Checks

The capacity of the shear walls were checked using equation 21-7 from ACI318-08 shown below:

$$\Phi V_n = \emptyset A_{cv}\left[\propto_c \sqrt{(f'_c)} + \rho_t f_y\right] \qquad \text{Eqn (4)}$$

	Vu(k)	Vert. Reinf	l(in)	hw(in)	Av	Acv (in^2)	αc	ρt	φVn (k)	Check:
SW1	350.764	(2)#11@6"	240	166	3.12	2880	2	0.043333	5949.229	ok
SW2	109.033	(2)#11@6"	108	166	3.12	1296	2.07	0.043333	2682.401	ok
SW3	121.86	(2)#8@12"	106	166	1.58	1272	2.13	0.010972	784.7924	ok
SW4	1540.494	(2)#9@12" & (2)#7@12"	528	166	3.14	6336	2	0.021806	6950.303	ok
SW5	1216.702	(2)#11@8"	354	166	3.12	4248	2	0.0325	6704.212	ok
SW6	557.931	(2)#7@12"	284	166	1.2	3408	2	0.008333	1672.32	ok
SW7	1425.515	(2)#7@12"	360	166	1.2	4320	2	0.008333	2119.843	ok
SW8	121.975	(2)#9@12" & (2)#7@12"	166	166	1.58	1992	2	0.010972	1214.033	ok

Figures 20: Calculated shear wall capacity checks for the second story

The Vu force was determined from the hand-calculated method of determining the total shear in each of the walls at each story. This shear took into account both the direct shear and the torsional shear. Each of the shear walls at the second story meet the qualifications based upon the check.

CONCLUSIONS

The decision to model only the concrete shear walls was an effective approach for analyzing the lateral force resisting system for the IAC Headquarters. The shear walls were found to be capable of resisting the lateral loads and conforming to the various performance requirements of ASCE7 and ACI.

Nevertheless, in order to most accurately model the lateral capabilities of the IAC Headquarters, it would have been necessary to model the entire building with all of the columns and gravity loading included. This is because, while the shear wall core is the predominant lateral resisting system, many other features of the building contribute to the lateral stiffness. For instance, the slab helps to transfer the lateral loads. Additionally, the sloped columns along the perimeter provide lateral stiffness, as would the curtain wall. While it is important to acknowledge the contribution of all of these factors and others in determining the lateral stiffness of the building, it can often save time and be considered conservative to only analyze the shear walls. In the future when conducting a more in-depth analysis, the entire building should be modeled when determining its stiffness.

Design of the lateral system in the IAC Headquarters is dependent on all of the factors studied in this report, in addition to others. Furthermore, the effectiveness of the lateral system in resisting lateral loads will directly affect the foundations and design of the remainder of the structural system.

Appendix A: Calculations

Figure A-1 : Center of Pressure Checks:

Calculated Center of Pressure							Computer Output	
Story	X direction		Average	Y direction		Average	x	y
1	1774.71	4120.1	2947.405	4608.14	2449.96	3529.05	2947.405	3529.050
2	1795.63	4119.98	2957.805	4631.25	2532.85	3582.05	2957.805	3582.050
3	1834.99	4119.98	2977.485	4594.39	2452.51	3523.45	2977.485	3523.450
4	1888.15	4119.98	3004.065	4608	2452.51	3530.255	3004.065	3530.255
5	1861.57	4119.98	2990.775	4608	2452.5	3530.25	2990.775	3530.250
6	2267.25	3969.45	3118.35	4315.19	2638.71	3476.95	3118.350	3476.950
7	2267.25	3919.83	3093.54	4297.96	2638.71	3468.335	3093.540	3468.335
8	2289.41	3864.95	3077.18	4264.75	2638.71	3451.73	3077.180	3451.730
9	2324.74	3815.19	3069.965	4218.75	2658.71	3438.73	3069.965	3438.730
10	2347.3	3784.39	3065.845	4218.75	2658.71	3438.73	3065.845	3438.730
11	2343.61	3763.44	3053.525	4205.13	2679.51	3442.32	3053.525	3442.320

Figure A-2: Center of Rigidity Check (2nd Story Shear Walls)

Calculated Center of Rigidity (2nd Story check)										
N/S Direction		xi	h	d	t	Δflexural	Δshear	Deflection	Ri	Relative Stiffness
	Wall2	0	309	105.000	12.000	1.932	0.167	2.099	0.476	0.017
	Wall3	0	309	108.000	12.000	1.776	0.163	1.938	0.516	0.019
	Wall5	240	309	351.000	12.000	0.052	0.050	0.102	9.826	0.354
	Wall6	453	309	276.000	12.000	0.106	0.064	0.170	5.881	0.212
	Wall7	528	309	372.000	12.000	0.043	0.047	0.091	11.028	0.398
								Sum of Ri:	27.727	
Ec	4.40E+06									
Er	1.76E+06			xr=	391.1396	+2856.36=	3247.49961			
						vs				
						3249.016	Difference=~2in			
				Origin:	2856.36	3277.03				

E/W Direction	yi	h	d	t	delta flexural	delta shear	Deflection	Ri	Relative Stiffness
Wall1	0.000	309.000	240.000	12.000	0.162	0.073	0.235	4.255	0.160
Wall4	351.000	309.000	528.000	12.000	0.015	0.033	0.048	20.632	0.775
Wall8	276.000	309.000	168.000	12.000	0.472	0.105	0.576	1.735	0.065
							Sum of Ri:	26.622	
yr	290.007	+3277.03=	3567.037						
			vs						
	3529.77	Difference= ~37in							

STORY2	D1	11.7121	11.7121	2982.781	3562.346	98.3823	98.3823	3016.236	3512.968	3249.016	3529.769

Wind Loading

Figure A-3: Calculated wind pressures in East/West direction

Location	Height above ground level, z (ft)	q (psf)	External pressure qGC_p (psf)	Internal pressure q_h (GC_{pi}) (psf)
Windward	155.75*	29.53	221.46	
	150.75	29.23	19.87	5.32
	140	28.70	19.52	5.32
	120	27.38	18.62	5.32
	100	26.07	17.73	5.32
	90	25.28	17.19	5.32
	80	24.49	16.65	5.32
	70	23.43	15.93	5.32
	60	22.38	15.22	5.32
	50	21.33	14.50	5.32
	40	20.01	13.61	5.32
	30	18.43	12.53	5.32
	25	18.43	12.53	5.32
	20	18.43	12.53	5.32
	15	18.43	12.53	5.32
Leeward	All	29.23	-10.93	5.32

Figure A-4: Calculated wind pressures in North/South direction

Location	Height above ground level, z (ft)	q (psf)	External pressure qGC_p (psf)	Internal pressure q_h (GC_{pi}) (psf)
Windward	155.75**	29.53	221.46	
	150.75	29.23	19.87	5.32
	140	28.70	19.52	5.32
	120	27.38	18.62	5.32
	100	26.07	17.73	5.32
	90	25.28	17.19	5.32
	80	24.49	16.65	5.32
	70	23.43	15.93	5.32
	60	22.38	15.22	5.32
	50	21.33	14.50	5.32
	40	20.01	13.61	5.32
	30	18.43	12.53	5.32
	25	18.43	12.53	5.32
	20	18.43	12.53	5.32
	15	18.43	12.53	5.32
Leeward	All	29.23	-12.42	5.32

Figure A-5,6,&7: Coeffiecients for wind calculations, Kz & Cp factors

V	110	mph
K_d	0.85	
I	1	
Exposure Category	B	(Category II according to Table 1-1)
K_{zt}	1	
K_z, K_h	(see table below)	
q_h	(see table below)	
n_1	0.69	
G	0.85	
q_p	30.02	Eqn 6-15 (K_h~= 1.14 @ 162'-8"
GC_{pn}	1.5	windward
	-1	leeward
p_p	45.023616	windward
	-30.015744	leeward
GC_{pi}	0.18	(for an Enclosed Building)
	-0.18	

Cp

Windward	Leeward		
	L/B=0-1	L/B=2	L/B>4
0.8	-0.5	-0.3	0.2

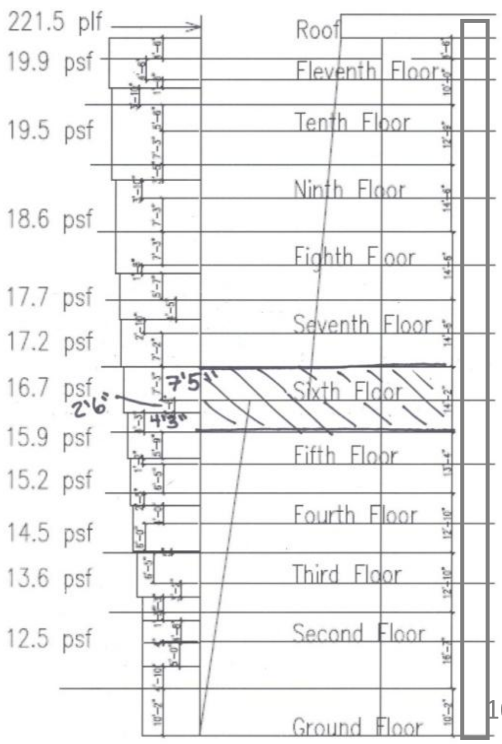
Height above ground level, z (ft)	K_z
155.75	1.1215
150.75	1.11
140	1.09
120	1.04
100	0.99
90	0.96
80	0.93
70	0.89
60	0.85
50	0.81
40	0.76
30	0.7
25	0.7
20	0.7
15	0.7

Figure A-8: Wind Story Forces & Shears

Floor	Height Above Ground (ft)	Floor Height (ft)	Forces (k)		Story Shear (k)	
			N/S	E/W	N/S	E/W
1	0.00	20.33	88.73284	68.537664	1180.182	1051.181
2	20.33	12.84	129.8199	111.435829	1091.45	982.6438
3	33.17	12.83	114.6342	100.793093	961.6296	871.2079
4	46.00	12.83	105.762	93.2102933	846.9954	770.4148
5	58.83	13.92	112.3516	98.00376	741.2334	677.2046
6	72.75	14.42	97.3063	85.42704	628.8818	579.2008
7	87.17	14.58	99.41022	87.68673	531.5755	493.7738
8	101.75	14.42	98.85124	88.43407	432.1653	406.087
9	116.17	14.58	95.58384	88.06063	333.3141	317.653
10	130.75	11.00	80.69424	75.9696	237.7302	229.5923
11	141.75	9.00	61.97877	58.8416	157.036	153.6227
Roof	150.75	5.00	27.38394	26.38944	95.05722	94.78112
Parapet	155.75	0.00	67.67328	68.39168	67.67328	68.39168

* Affects from cellar not taken into account in wind/seismic for this analysis

Figure A-9: Sample Story Force Calculation



Sample calc for story forces:

@ Level 6

4.25(15.9 psf)+(2.5+7.417) × 16.7psf

= 233plf × length of wall

= 233(137.75)/1000 = 32.1k

Windward

10.93psf (14.167ft) =

= 154.8plf

=154.8(length of wall)

= 154.8(137.75)/1000

= 21.3k

Total force @ 6th story:

10.93 psf 32.1+21.3 = $\boxed{53.4k}$

10.93 psf

Seismic Loading

Figure A-10: Seismic Coefficients using Equivalent Lateral Force Procedure

Latitude	40.745179	
Longitude	-74.007654	
S_s	0.363	
S_1	0.07	
Soil Class E (Soft Clay Soil)		
F_a	2.14	(interpolated from Table 11.4-1)
F_v	3.5	(from Table 11.4-2)
S_{MS}	0.77682	
S_{M1}	0.245	
S_{DS}	0.518	
S_{D1}	0.163	
Seismic Design Category	D (from S_{DS}) or C (from S_{D1})	Use SDC= D

T_s	0.315	
R	5	(assumption from table 12.2-1)
T_a	0.86	(section 12.8.2.1)
C_t	0.02	(in both directions)
x	0.75	
hn	150.67	
Importance Factor	1	
T_L	6	(Figure 22-15)
C_s	0.03798	N-S & E-W
k	2	(.5<Ta<2.5)
$V=C_sW$	1831.4	k

Figure A-11: Building Weight

Floor	Floor Area	Floor Dead Load	Floor weight	h/2 above	h/2 below	Column area above	Column area below	Column weight= height*area*150pcf	Curtainwall (estimated length along perimeter)	Curtainwall weight (height*length*15psf)	Shear Wall Length (ft)	Shear Wall Thickness (in)	ShearWall Weight (L*t*h*150)	
Cellar														
1st				10.17	7.25	102.6254	102.62536	286428.1244	762.25	199135.9069	59.1	14	674033.341	Above
Retail/Assembly	17902	220.00	3938440.00			14.72622	14.7262156				3.87777	16		
Loading/Corridor&Lobby	1495	220.00	328900.00			37.83001	37.8300115				85.5	12		
Loading Dock	980	450.00	441000.00			4.25	4.25							
Entry	1570	475.00	745750.00			4.72222	4.72222222							
Ramp	200	200.00	40000.00			8.726646	8.72664626							
Planter	309	650.00	200850.00			7.068583	7.06858347				W1	1159597.37		
Exterior	550	270.00	148500.00			7.875798	7.87579825							
Garden	2023	675.00	1365525.00			Above=	Below=							
Entry	666	310.00	206460.00			187.8248	187.824837	<-- Not to be included in the building's total weight for seismic!						
Sidewalk	2800	500.00	1400000.00											
Entry	650	400.00	260000.00											
Garden	592	400.00	236800.00											
Stair	68	175.00	11900.00											
Stair	195	160.00	31200.00											
		Total	9355325.00	<-- Not to be included in the building's total weight for seismic!										
2nd				6.42	10.17	111.1775	102.6254	461485.2836	724.25	180155.1958	187	12	179987.033	Above
Office	19256	170.00	3273520.00			4.908742	14.72622						674033.34	Below
Lobby	250	175.00	43750.00			37.83001	37.83001							
Services	820	185.00	151700.00			4.25	4.25							
Stair	502	175.00	87850.00			4.72222	3.777777							
Mechanical	550	170.00	93500.00			7.875798	8.726646				W2	5145980.85		
		Total	3650320.00			6.305002	7.068583							
						6.305002	7.875798							
						Below=	Above=							
						183.3743	186.880434							
3rd				6.42	6.42	102.6254	106.901457	357903.4769	723.75	139321.5131	187	12	179987.033	Above
Office	18944	170.00	3220480.00			14.72622	9.817481						179987.03	Below
Lobby	250	175.00	43750.00			37.83001	37.83001							
Services	820	185.00	151700.00			4.25	3.77777							
Stair	502	175.00	87850.00			3.77777	3.777777							
Mechanical	550	170.00	93500.00			8.726646	8.726646				W3	4454479.05		
		Total	3597280.00			7.068583	7.068583							
						7.875798	7.068583							
						Below=	Above=							
						186.8804	184.968307							
4th				6.42	6.42	106.9015	106.901457	355513.6308	722	139057.2	187	12	179987.03	Above
Office	18505	170.00	3145850.00			9.817481	9.817481						179987.03	Below
Lobby	250	175.00	43750.00			37.83001	37.83001							
Services	820	185.00	151700.00			3.77777	3.77777							
Stair	502	175.00	87850.00			3.777777	3.777777							
Mechanical	550	170.00	93500.00			8.726646	8.726646				W4	4377194.89		
		Total	3522650.00			7.068583	6.305002							
						7.068583	7.068583							
						Below=	Above=							
						184.9683	184.204726							
5th				6.42	6.42	106.9015	89.797229	359211.2821	721	138864.6	187	12	179987.03	Above
Office	17968	170.00	3054560.00			9.817481	19.634959						179987.03	Below
Lobby	250	175.00	43750.00			37.83001	37.83001							
Services	820	185.00	151700.00			3.77777	3.77777							
Stair	502	175.00	87850.00			3.777777	3.777777							
Mechanical	550	170.00	93500.00			8.726646	8.726646							
		Total	3431360.00			6.305002	12.610004				W5	4289409.94		
						7.068583	7.068583							
							5.58505							
						Below=	Above=							
						184.2047	188.808028							

Figure A-11 (continued): Building Weight Calculations

6th				7.25	6.42	89.79723	26.1799388	260482.9656	538	114265.2	171.25	12	186234.375	Above
Office	10089	320.00	3228480.00			19.63496	10.559242		below:				179987.03	Below
Lobby	250	350.00	87500.00			37.83001	9.42477796		579					
Services	820	330.00	270600.00			3.77777	4.276057							
Stair	502	325.00	163150.00			3.777777	2.66666667							
Mechanical	550	320.00	176000.00			8.726646	4.16666667				W6	7160799.57		
Terrace	7126	350.00	2494100.00			12.61								
		Total	6419830.00			7.068583								
						5.58505								
						Below=	Above=							
						188.808	57.273349							
7th				7.25	7.25	26.17994	32.72492	117153.9169	519.5	112991.25	171.25	12	186234.375	Above
Office	9332	195.00	1819740.00			10.55924	5.27964						186234.38	Below
Lobby	250	225.00	56250.00			9.424778	3.141578							
Services	820	210.00	172200.00			4.276057	3.14158							
Stair	548	175.00	95900.00			2.66667	2.66667				W7	2853953.92		
Mechanical	550	195.00	107250.00			4.166667	3.5							
		Total	2251340.00			Below=	Above=							
						57.27335	50.454388							
8th				7.25	7.25	32.72492	37.08824	108196.1558	501.5	109076.25	171.25	12	186234.375	Above
Office	8622	195.00	1681290.00			5.27964	2.63984						186234.38	Below
Lobby	250	225.00	56250.00			3.141578	0							
Services	820	210.00	172200.00			3.14158	3.14158							
Stair	548	175.00	95900.00			2.66667	2.66667				W8	2702631.16		
Mechanical	550	195.00	107250.00			3.5	3.5							
		Total	2112890.00			Below=	Above=							
						50.45439	49.03633							
9th				7.25	7.25	37.08824	34.90658	104281.4625	481	104617.5	170.25	12	185146.875	Above
Office	7907	195.00	1541865.00			2.63984	2.63984						186234.38	Below
Lobby	250	225.00	56250.00			0	0							
Services	820	210.00	172200.00			3.14158	3.14158							
Stair	548	175.00	95900.00			2.66667	2.66667				W9	2553745.22		
Mechanical	550	195.00	107250.00			3.5	3.5							
		Total	1973465.00			Below=	Above=							
						49.03633	46.85467							
10th				5.5	7.25	34.90658	30.54326	84864.44588	460.5	88070.625	154.25	12	127256.25	Above
Mechanical	7536	195.00	1469520.00			2.63984	2.63984						185146.88	Below
Services	900	210.00	189000.00			0	0							
Stair	537	175.00	93975.00			3.14158	3.14158							
Office	250	195.00	48750.00			2.66667	2.66667				W10	2286583.20		
		Total	1801245.00			3.5	3.5							
						Below=	Above=							
						46.85467	42.49135							
11th				4.5	5.5	30.54326	30.54326	58649.52225	447.5	30206.25	122.75	12	82856.25	Above
Elevator Machine	650	195.00	126750			2.63984	2.63984						127256.25	Below
Stairs	225	195.00	43875			0	0							
Mechanical	2040	225.00	459000			3.14158	3.14158							
		Total	629625			2.66667								
						3.5					W11	955644.4923		
						Below=	Above=							
						42.49135	36.32468							
Roof	6397	225.00	1439325		4.5		30.54326	24519.159	465.5	31421.25			82856.25	Below
							2.63984							
							0							
		Total	1439325				3.14158				WR	1578121.659		
							Above=							
							36.32468							
		TOTAL FROM SLABS	39555030.00	LBS			**TOTAL FROM COLUMNS**	2554170.267	**TOTAL FROM CURTAINWALL**	1387182.741	lbs	**TOTAL SHEARWALL WEIGHT**	4695887.95	
									TOTAL WEIGHT	48219.32	kips			

ETABS Model:

The table below shows the Mass/Area's assigned to each of the floor levels in ETABS. This was determined from the weight per floor found in Technical Report 1 by accounting for the weight of slabs, shear walls, curtain walls, and columns.

Calculations:

Weight of floor in lbs/ft^2 X (1/32.2) X (1/1728) X (1/1000)= Mass/Area in k/in^2

Floor	Mass/Area
1	4.33E-06
2	3.80E-06
3	3.81E-06
4	3.84E-06
5	6.66E-06
6	6.66E-06
7	4.50E-06
8	4.56E-06
9	4.46E-06
10	5.89E-06
11	4.43E-06

Figure A-12: Mass/Area input into ETABS

Figure A-13: Computer Output of Centers of Mass & Centers of Rigidity

Story	Diaphragm	XCM	YCM	XCR	YCR
ROOF	D1	3030.926	3493.011	3289.62	3628.044
STORY10	D1	3032.57	3490.804	3290.679	3627.216
STORY9	D1	3034.815	3486.942	3291.401	3625.567
STORY8	D1	3029.482	3491.137	3290.587	3620.635
STORY7	D1	3023.125	3506.499	3289.016	3614.411
STORY6	D1	3022.093	3521.387	3286.195	3606.239
STORY5	D1	3015.038	3511.34	3281.193	3594.985
STORY4	D1	3021.249	3503.203	3274.193	3579.707
STORY3	D1	3001.688	3522.31	3264.163	3559.188
STORY2	D1	2982.781	3562.346	3249.016	3529.769
STORY1	D1	2977.705	3527.694	3222.121	3486.877

These values for centers were used to determine relative stiffnesses, torsion, and deflection. It was assumed in this report that the ETABS model correctly determined the centers of mass and rigidity per floor.

Figure A-14: Accidental Torsional Moment in the N-S direction

N-S Torsional Moment, M_{ta}				
Story	Structural Width (in)	5% Width (ft)	Story Force (k)	Torsion (ft-k)
1	2345.39	9.772458	15.8	154.4048
2	2324.35	9.684792	36.3	351.5579
3	2284.99	9.520792	68.7	654.0784
4	2231.83	9.299292	110.1	1023.852
5	2258.41	9.410042	280.3	2637.635
6	1702.2	7.0925	160.7	1139.765
7	1652.58	6.88575	207.1	1426.039
8	1575.54	6.56475	255.4	1676.637
9	1490.45	6.210208	289.4	1797.234
10	1437.09	5.987875	142.2	851.4758
Roof	1419.83	5.915958	265.5	1570.687

Figure A-15: Inherent Torsional Moment in the N-S direction

N-S Torsional Moment, M_t						
Story	Center of Mass	Center of Rigidity	Difference (in)	Story Force (k)		Torsion (ft-k)
1	3011.596	3222.121	210.525	15.8		277.1913
2	3016.236	3249.016	232.78	36.3		704.1595
3	3020.757	3264.163	243.406	68.7		1393.499
4	3023.662	3274.193	250.531	110.1		2298.622
5	3024.085	3281.193	257.108	280.3		6005.614
6	3028.079	3286.195	258.116	160.7		3456.603
7	3030.048	3289.016	258.968	207.1		4469.356
8	3031.939	3290.587	258.648	255.4		5504.892
9	3032.787	3291.401	258.614	289.4		6236.908
10	3031.881	3290.679	258.798	142.2		3066.756
Roof	3030.926	3289.62	258.694	265.5		5723.605

Figure A-16: Accidental Torsional Moment in the E-W direction

E-W Torsional Moment, M_{ta}				
Story	Structural Width (in)	5% Width (ft)	Story Force (k)	Torsion (ft-k)
1	2158.18	8.992417	15.8	142.0802
2	2098.4	8.743333	36.3	317.383
3	2141.88	8.9245	68.7	613.1132
4	2155.49	8.981208	110.1	988.831
5	2155.5	8.98125	280.3	2517.444
6	1676.48	6.985333	160.7	1122.543
7	1659.25	6.913542	207.1	1431.794
8	1626.04	6.775167	255.4	1730.378
9	1560.04	6.500167	289.4	1881.148
10	1560.04	6.500167	142.2	924.3237
Roof	1525.62	6.35675	265.5	1687.717

Figure A-17: Inherent Torsional Moment in the E-W direction

E-W Torsional Moment, M_t						
Story	Center of Mass	Center of Rigidity	Difference (in)	Story Force (k)		Torsion (ft-k)
1	3527.694	3486.877	40.817	15.8		53.74238
2	3562.346	3529.769	32.577	36.3		98.54543
3	3522.31	3559.188	36.878	68.7		211.1266
4	3503.203	3579.707	76.504	110.1		701.9242
5	3511.34	3594.985	83.645	280.3		1953.808
6	3521.387	3606.239	84.852	160.7		1136.31
7	3506.499	3614.411	107.912	207.1		1862.381
8	3491.137	3620.635	129.498	255.4		2756.149
9	3486.942	3625.567	138.625	289.4		3343.173
10	3490.804	3627.216	136.412	142.2		1616.482
Roof	3493.011	3628.044	135.033	265.5		2987.605

Figure A-18: Table from ASCE7-05 for seismic drift checks

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	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.
[b] For seismic force–resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.

Story Drift @ top level: $.02h_{sx}$= .02 (108) = 2.16 > (2.2253-2.065)= .1603" * 5 = .8015" ok (From ETABS)

> (1.131-0.891)= .200 in *5= 1" ok (From calc. methods)

Story Drift @ story two: $.02h_{sx}$= .02 (166)=3.21 > .1406 in * 5 = .703" ok (From ETABS)

> (.065-.017) = .048 in *5 = .24" ok (From calc. methods)

Overall Deflection: L/400 = 1719*(12)/400 = 4.30 in > 2.2253" ok (From ETABS)

< 8.05" **not ok** (From calc. methods)

The multiplication factor of 5 applied to story displacements is from Figure 12.8.2 of ASCE7-05 and is an amplification factor for seismic buildings.*

		Hand-Calculated				ETABS Output			
Story	Story Height	N/S	E/W	N/S	E/W	N/S	E/W	N/S	E/W
		Story Displacement	Story Displacement	Story Drift	Story Drift	Story Displacement	Story Displacement	Story Drift	Story Drift
1	143	0.017	0.02	0.000119	0.000143	0.0634	0.0382	0.000288	0.000546
2	309	0.065	0.07	0.000291	0.000288	0.204	0.095	0.000655	0.001068
3	463	0.160	0.15	0.000612	0.000519	0.3728	0.1029	0.00096	0.0015
4	617	0.318	0.27	0.001026	0.000805	0.5613	0.2808	0.003562	0.002326
5	783	0.564	0.46	0.001486	0.001118	0.8191	0.3925	0.004	0.0043
6	957	0.815	0.64	0.001438	0.001032	1.0846	0.5116	0.00145	0.00217
7	1131	1.151	0.88	0.001935	0.001394	1.3623	0.6643	0.00364	0.00418
8	1305	1.429	1.08	0.001598	0.001123	1.6174	0.8283	0.00357	0.00389
9	1479	1.506	1.12	0.00044	0.00026	1.8623	0.9783	0.00291	0.00375
10	1611	1.131	0.84	0.002838	0.002151	2.065	1.0813	0.00319	0.00342
Roof	1719	0.891	0.66	0.00222	0.007743	2.2253	1.1626	0.00328	0.0016
	Overal Defl.	8.048	6.17						

Figure A-19: Summary of Drifts and Displacements by hand and from ETABS

Figure A-20: Calculated deflection of walls at 11th story in N-S direction

11th Floor										
		Shear	h	d	t	delta flexural	delta shear	Deflection	Relative Stiffness	
	Wall2	2.672505	1719	105	12	0.889	0.002	0.891	0.010	
	Wall3	2.907714	1719	108	12	0.889	0.003	0.891	0.011	
									0.000	
	Wall5	97.07632	1719	351	12	0.864	0.027	0.891	0.366	
	Wall6	47.75016	1719	276	12	0.875	0.017	0.891	0.180	
	Wall7	115.1333	1719	372	12	0.861	0.030	0.891	0.434	
Ec	4.40E+06		Story Shear	265.54						
Er	1.76E+06									

Figure A-21: Calculated relative stiffness for members at 11th story in N-S direction

11th Floor										
		Arbitrary Force	h	d	t	delta flexural	delta shear	Deflection	Relative Stiffness	
	Wall2	1000000	1719	105	12	332.646	0.931	333.577	0.010	
	Wall3	1000000	1719	108	12	305.688	0.905	306.593	0.011	
	Wall5	1000000	1719	351	12	8.905	0.278	9.183	0.366	
	Wall6	1000000	1719	276	12	18.316	0.354	18.670	0.180	
	Wall7	1000000	1719	372	12	7.480	0.263	7.743	0.434	
								Sum of Ri:		
Ec	4.40E+06									
Er	1.76E+06									

	Arbitrary Force	h	d	t	delta flexural	delta shear	Deflection	Relative Stiffness
Wall1	1000000	1719	240	12	27.856	0.407	28.263	0.087
Wall4	1000000	1719	528	12	2.616	0.185	2.801	0.882
Wall8	1000000	1719	168	12	81.212	0.582	81.794	0.030
							Sum of Ri:	

Figure A-22: Calculated relative stiffness for members at 11th story in E-W direction

Figures A-23 & 24: Calculated torsional shear of members at 11th story in N-S & E-W directions

TORSIONAL SHEAR FORCES ON ELEVENTH STORY LATERAL FRAME ELEMENTS:

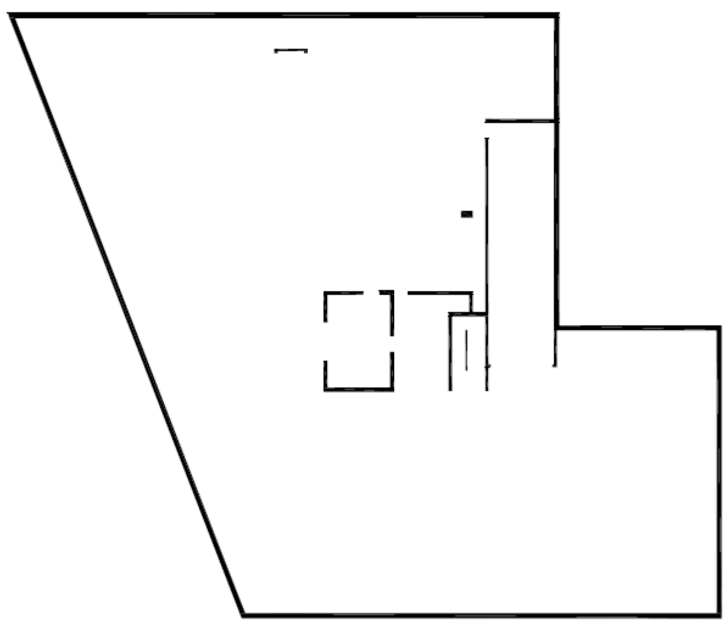
N/S DIRECTION											
Wall	Rel. Stiffness	Story Shear	Location of Wall in ETABS	Dist. fr. Center of Rigidity (da)		Dist fr COM to COR (e)	Torsional Shear	Direct Shear	Total Shear		Ridi^2
1	E-W			252.739							10791.46
2	0.01	1831.37	2856.36	434.319		258.109	48.50087	18.3137	66.81457		1886.33
3	0.011	1831.37	2856.36	434.319		258.109	53.35096	20.14507	73.49603		2074.963
4	E-W			109.501							10575.59
5	0.366	1831.37	3111.43	179.249		258.109	732.6196	670.2814	1402.901		11759.65
6	0.18	1831.37	3317.6	26.921		258.109	54.11335	329.6466	383.7599		130.4532
7	0.433	1831.37	3399	108.321		258.109	523.7708	792.9832	1316.754		5080.579
8	E-W			31.631							30.0156
									3243.726	J	42329.05

y 3627.216
x 3290.679

E/W DIRECTION											
Wall	Rel. Stiffness	Story Shear	Location of Wall in ETABS	Dist. fr. Center of Rigidity		Dist fr COM to COR	Torsional Shear	Direct Shear	Total Shear		Ridi^2
1	0.088	1831.37	3277.03	350.186		136.412	181.8744	161.1606	343.035		10791.46
2	N/S			392.656							1886.33
3	N/S			392.656							2074.963
4	0.882	1831.37	3639.27	109.501		136.412	570.0026	1615.268	2185.271		10575.59
5	N/S			137.586							11759.65
6	N/S			68.584							130.4532
7	N/S			149.984							5080.579
8	0.03	1831.37	3561.4	31.631		136.412	5.600468	54.9411	60.54157		30.0156
										J	42329.05

Appendix B: Shear Wall Plans

Figure B-1 : Cellar Floor Plan



LINK BEAMS (w x h)

LB-1: 72x12
LB-2: 72x14
LB-3: 12x18

Figure B-2 : First Floor Plan

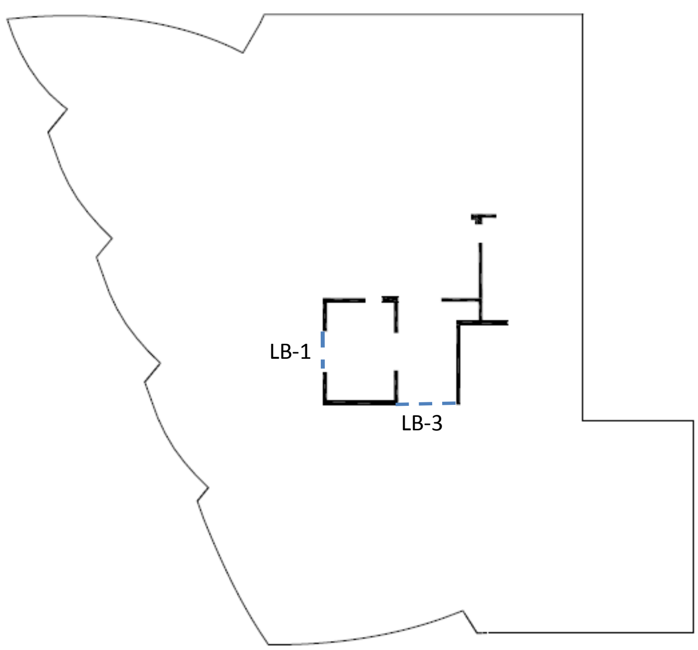


Figure B-3 : Second Through Fifth Floors (typ)

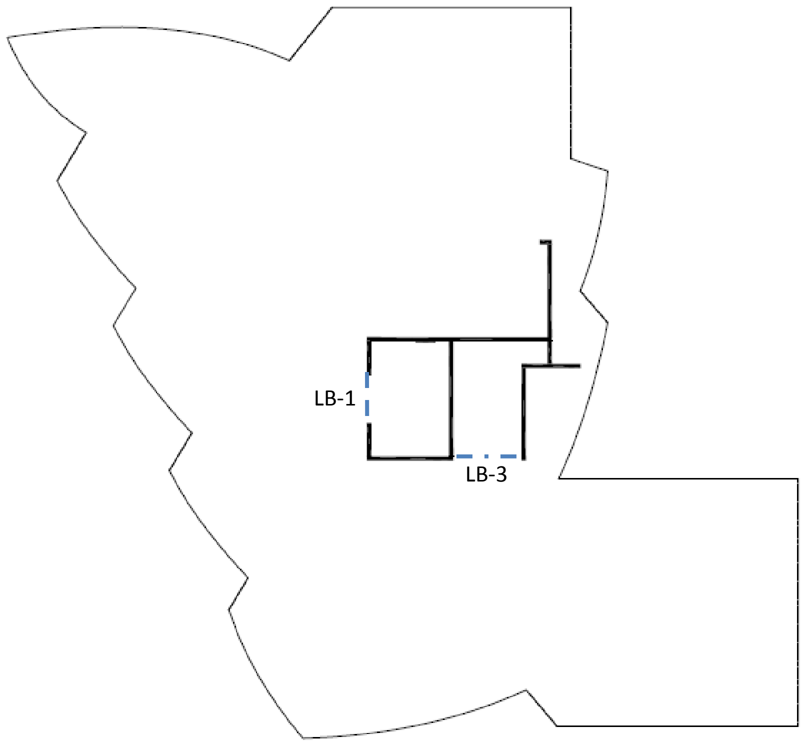


Figure B-4 : Sixth through Eighth Floors (typ)

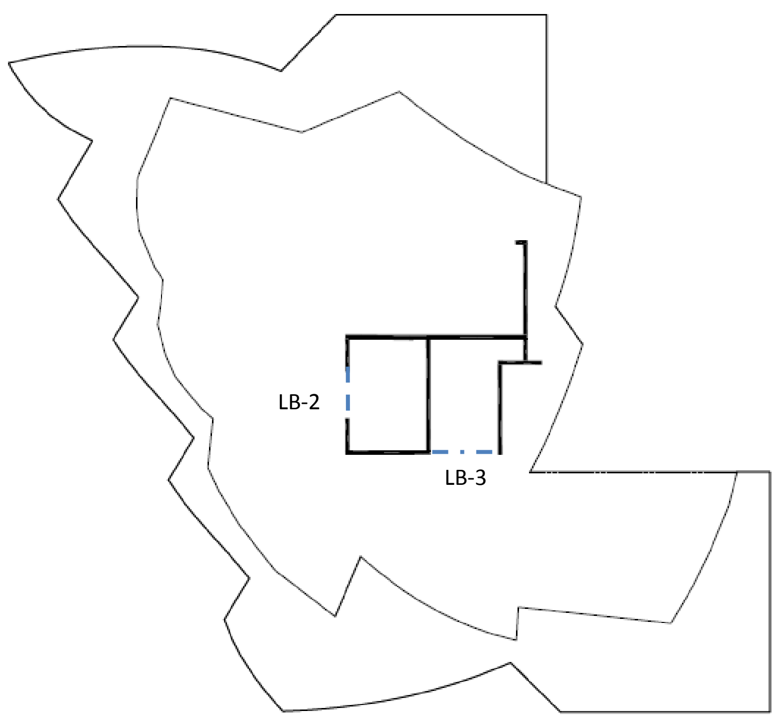


Figure B-5 : Ninth Floor Plan

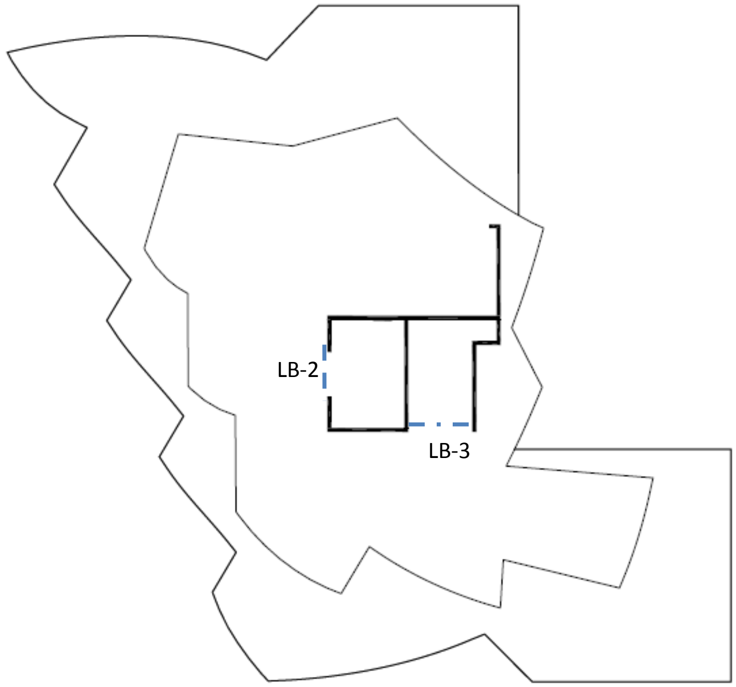


Figure B-6 : Tenth Floor Plan

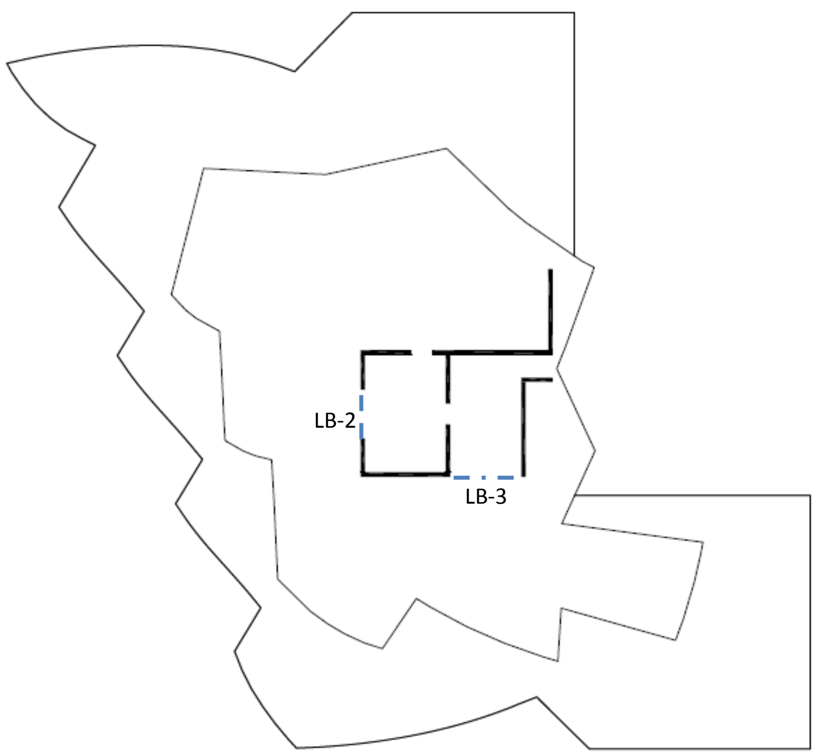


Figure B-7 : Eleventh Floor Plan