100 Eleventh Avenue

New York, New York

Tyler Graybill | Structural Option
Consultant: Professor Thomas Boothby

Tuesday, December 1st 2009

©archpartners.com

Technical Report III

Table of Contents

Executive Summary

Technical Report III is a confirmation and design study of the lateral system of 100 Eleventh Avenue. Its intent was to not only confirm the building's lateral system design, but to gain an overall understanding of how it works.

100 Eleventh Avenue is a 22-story, 148,000 sf residential building located in Manhattan's West Chelsea District. The building's superstructure is cast-in-place concrete, with a two-way flat plate floor system. Lateral loads are resisted by shear walls and seven "long" columns.

The loads calculated in Technical Report I were applied to the building, with slight modifications to the seismic calculations. Because the factored wind load has both a larger base shear (direct shear) than the seismic load and a larger eccentricity (torsional shear), it was deemed the controlling load case in both directions. All succeeding manual calculations were performed with this as the assumed controlling case. Level 8 was selected as the sample floor on which to perform calculations.

Direct shear was distributed to members according to their relative stiffness, which was calculated using the equation for deflection of a cantilever. While the columns' contributions were not negligible, it was determined that the shear walls resist the majority of lateral forces. Torsional shear in each member was calculated, and it was here that the columns' contribution became important, as their large distance from the center of rigidity aids in resisting moment. With both direct and torsional shear calculated for the 8th level, these forces were checked against the shear capacity of this level's walls and columns using $V_c = 3.3\lambda\sqrt{(f'_c)}hd + (N_u d)/(4l_w)$. All members satisfied the check.

A rough overturning analysis was performed by confirming that the building's dead weight multiplied by half the building's least depth (732,410 ft-k) was sufficient to resist the overturning moment of 274,473 ft-k induced on the structure by the wind load. Again, the columns contribute significantly by increasing the depth of the building from 24' to 35', increasing the building's resistance to overturning.

An ETABS model was developed for 100 Eleventh Avenue's lateral system. This model was used to generate force distributions, centers of mass, rigidity, and pressure, and displacements. A manual calculation of the center of rigidity for the 8th level came within 98% of that calculated by ETABS. The force distribution was compared to the manually calculated values. While the distributions were similar in that the shear walls collected the majority of the load, many of the member forces varied significantly. This can be attributed to the simplified manual approach not taking into account the large variation in sizes of shear walls and columns from floor to floor.

Building and story drifts taken from the computer model were compared against a code drift limit of $0.020h_{sx}$ for seismic and $L/400$ for wind. All seismic story drifts were under the limit, while many of the building and story drifts in both directions due to wind were significantly over the $L/400$ recommended drift limit.

Introduction to 100 Eleventh Avenue

100 Eleventh Avenue is a 22-story, 170,000 sf condominium building located in Manhattan's Chelsea District, a neighborhood quickly gaining in popularity within the city and adjacent to the Hudson River. 100 Eleventh Avenue will join several other recently completed projects that have helped in revitalizing the area, such as the IAC headquarters designed by architect Frank Gehry, and the High Line, a former elevated rail line running through the area that has been converted into an elevated park.

Dubbed a "vision machine" by its Pritzker Prize-winning architect Jean Nouvel, 100 Eleventh Avenue's defining feature is its facade, a panelized curtainwall system consisting of 1650 windows, each a different size and uniquely oriented in space. Light reflecting off the randomly-oriented windows limits views into the building while still allowing occupants spectacular floor-to-ceiling views of both New York City and the Hudson River. In addition, the lower six floors are enclosed by a second facade offset 16 feet towards the street. As seen in Figure 1 below, the space between the two facades is filled with intricate steel framing and cantilevered walls, columns, and balconies. Trees are suspended in air at varying heights, creating a "hanging garden" and a unique atrium space.

The building's structural system is cast-in-place concrete – common for residential buildings in the city. The ground level contains 6000 sf of retail space, as well as an elevated garden space for the residents, which spans over a junior Olympic-sized pool. Levels 2 through 21 house the residential units, with the penthouse making up the 21st floor, containing an extensive private roof terrace.


©www.arte-factory.com

Figure 1: Space within double facade


©www.arte-factory.com

Figure 2: View from Westside Highway

Existing Structural System Summary

Foundations

100 Eleventh Avenue is located on a man-made portion of Manhattan Island. Therefore, the shallow bedrock typical of much of the island is not present, and the use of piles and drilled caissons is necessary to effectively transfer vertical and horizontal loads to the earth. 127 piles at 150 ton capacity transfer column loads to the ground. Thirteen of these are detailed to provide a 50 kip tension capacity, as several cantilevered columns may, under certain loading conditions, induce tension in the piles, as seen in Figure 4. In addition, 12 large-diameter caissons are located at the structure's shear wall core, ranging in capacity from 600-1500 ton and providing at least 50 kip in lateral capacity. At the cellar level, a 20" thick mat foundation ties the piles together, while resisting the upward soil pressure. At the building's core, this mat slab thickens to 36".

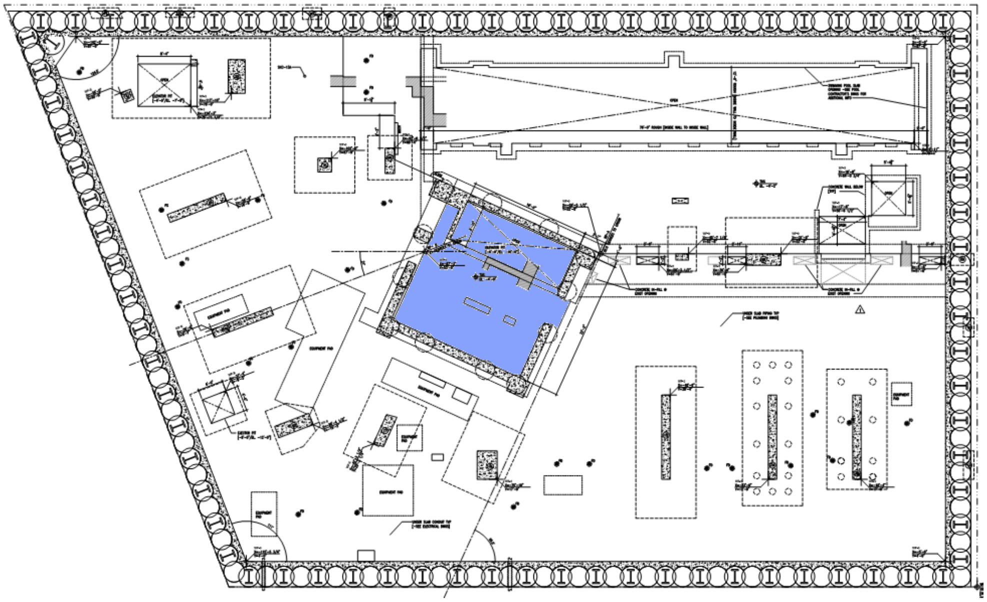


Figure 3: Cellar plan with core denoted

In order to eliminate the cost of underpinning the adjacent structures during excavation, a concrete secant wall system was used instead of traditional foundation walls. As seen in Figure 3, the secant piles are driven around the entire perimeter and resist the lateral soil pressures. The secant wall is braced at its top by the 12" ground floor slab. At all slab steps on the ground floor, torsion beams were used to resist torsion created by the lateral forces from the secant wall.

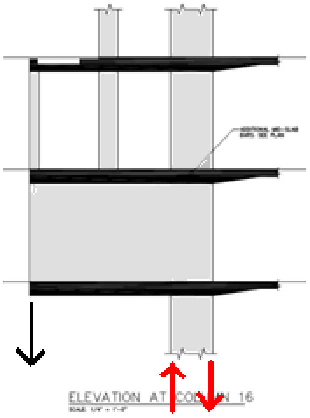


Figure 4: Cantilevered column creating tension in piles

Gravity System

Floor System

100 Eleventh Avenue has a cast-in-place two-way concrete flat-plate floor system. This type of system is common for residential buildings in New York City due to the ease of accommodation of column offsets, the minimal floor system thickness, and the sound isolation properties of concrete.

The typical floor is comprised of 9" thick, 5,950 psi concrete reinforced with a basic bottom reinforcing mat of #4 @ 12" E.W. Middle strip bars are also #4 @ 12" unless otherwise noted. Column strip bars are primarily #6 @12". Additional top and bottom bars are used where necessary, likely due to longer spans and varying loads. The slab thickness increases to 12" at the elevator core, where the bottom reinforcing steel is #5 @12" E.W. While no standard span exists, most slab spans range from 18'-23'. Due to increased loads from the curtainwall as well as spans as long as 34 feet, the slab thickens from 9" to 18.5" along the curved portion of the building. For appearances, the slab gradually increases in thickness over a distance of 5'-0", as seen in Figure 6, rather than undergoing an abrupt increase.

Figure 5: Superstructure

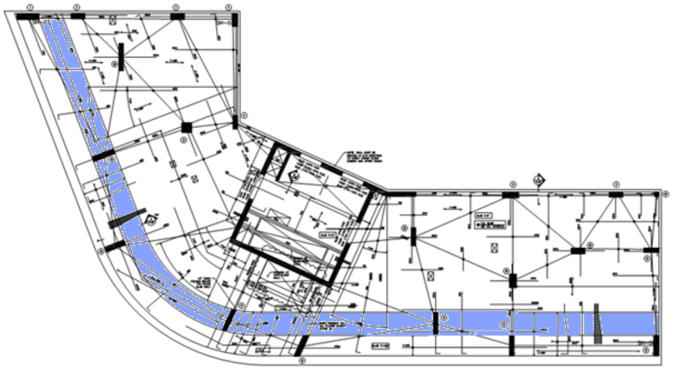


Figure 6: Typical plan with slab thickness transition area highlighted

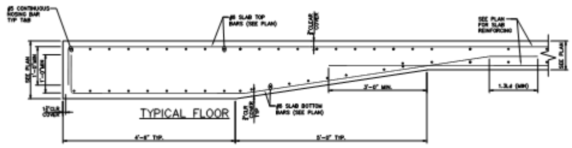


Figure 7: Detail of thickened slab at curved edge

As seen from the typical structural plan, Figure 8, floor reinforcing along the curve is detailed as straight bars with a single bend, thereby avoiding the additional costs and installation difficulties involved with curved bars. Slab reinforcing was detailed radially throughout the floor to match the building's three distinct geometric axes.

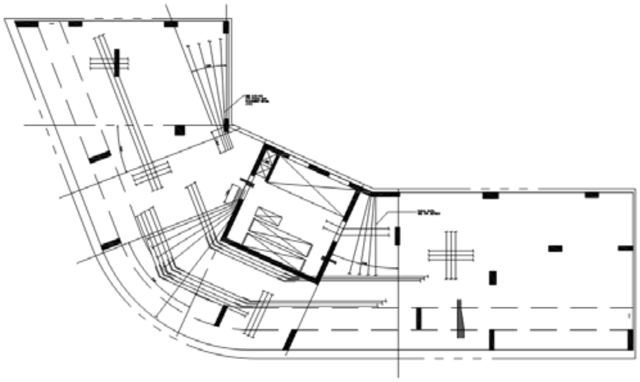


Figure 8: Slab reinforcing schematic layout

On the lower six floors, balconies begin to cantilever out towards the second street facade. An example of this is shown in Figure 9, where the balcony extends 9'-10" from the building. Notice that, due to architectural constraints, the balcony has only one corner supported by a column below. To resolve excessive deflection caused by the facade and tree loads, three post-tensioned high-strength Dywidag bars were used, highlighted in green.

Figure 9: Cantilevered balcony utilizing post-tensioning

Columns

Concrete strength for columns supporting the cellar level through the 9th level is 8 ksi; those supporting the 10th through the roof have 7 ksi concrete. As evidenced by the typical floor plan, no regular grid exists. Spans typically range from 18'-23', except on the curved edge portion, where spans of up to 34' exist. Column sizes range widely throughout a single floor, as well as from floor to floor. The majority are 12"-16" wide and 3-4 times as long, resulting in many "long" columns. This allows the columns to be placed within the walls separating individual units. Also, seven of these long columns were designed as part of the lateral system. More discussion on this can be found in the lateral system summary.

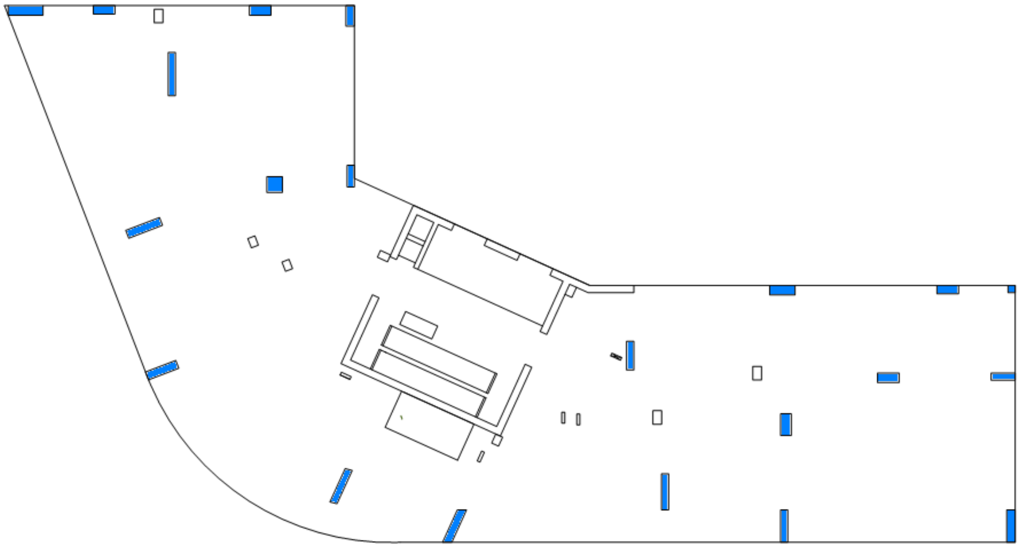


Figure 10: Typical floor column layout

On the lower six floors of the building, these seven long columns also serve as support for the complex balcony system that defines the lower floors. On these floors, intermittent boxes protrude out from the inner facade to meet the outer street facade, which is offset 16' towards the street. On the second level, several of these outstretched balconies are supported by cantilevered columns ranging in length from 18' to 28'. Figure 12 shows the columns supporting the 3rd level, with red denoting the cantilevered portion of the columns. Due to significant tensile forces at the tops of these cantilevered columns, additional reinforcement of six mid-slab #11 Grade 75 bars tie the top of the columns into the main portion of the slab.

Figure 11: Photo showing portion of cantilevered balcony system

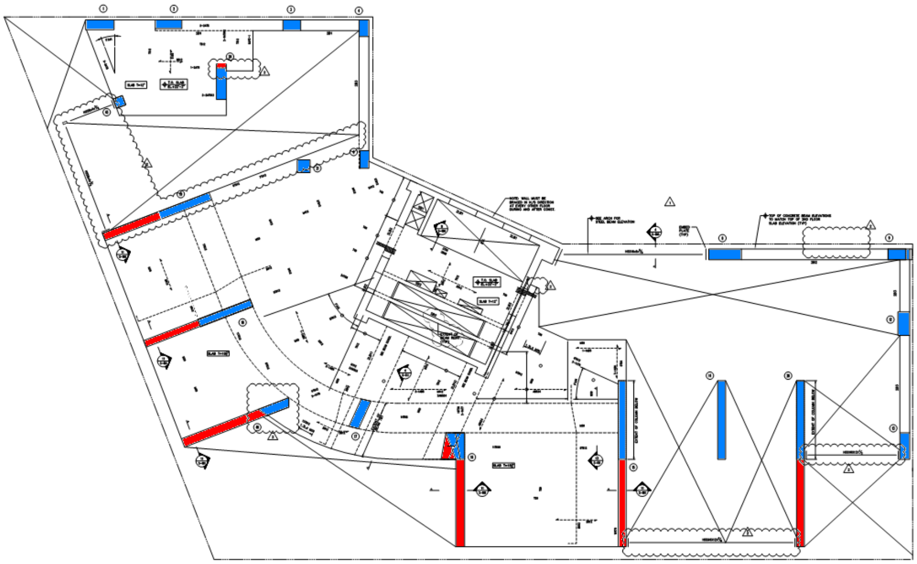


Figure 12: 2nd Floor column layout

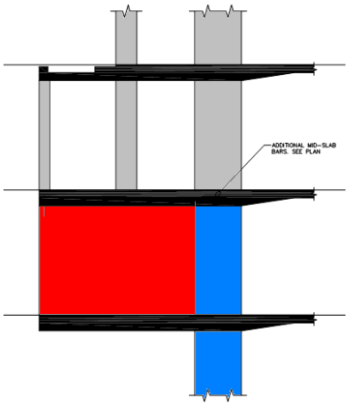


Figure 13: Cantilevered Column Elevation

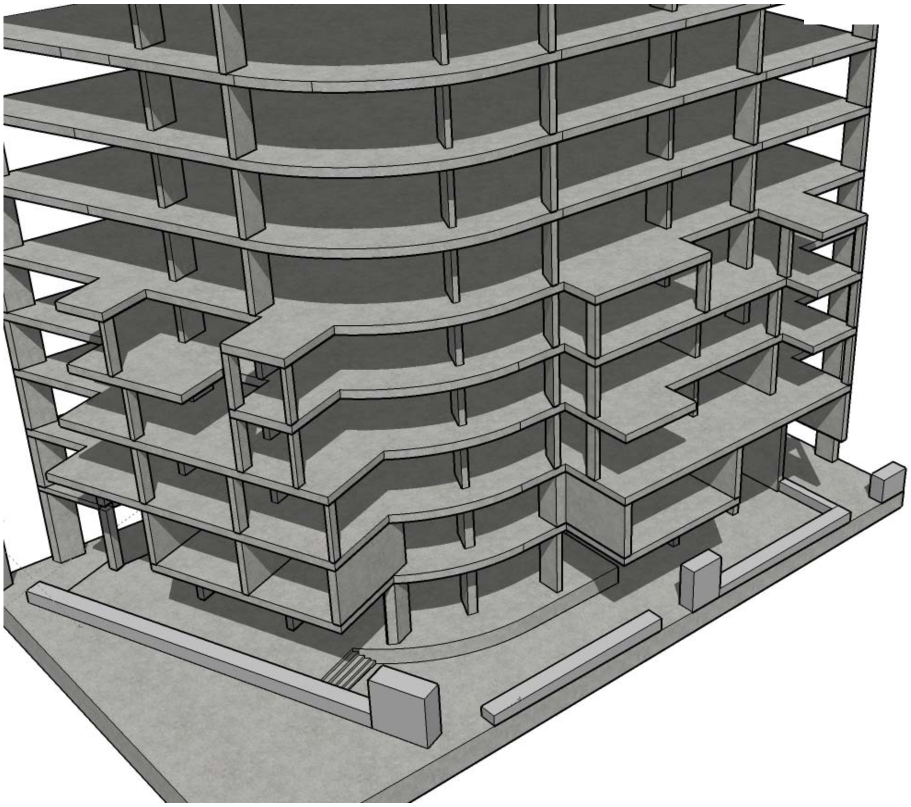


Figure 14: Model showing complicated balcony system

Lateral System

100 Eleventh Avenue's main lateral force resisting system is comprised of concrete shear walls located at the building elevator core, in combination with seven "long" columns, as shown in Figure 15 below. Because architectural constraints restricted the use of shear walls to the relatively small elevator core, the seismic loading necessitated that these seven columns also be designed to resist lateral forces. Two of these columns are connected to the main core via in-slab outrigger beams for additional stiffness. These 4' wide beams are reinforced with 11 #7 bars on both the top and bottom. The diaphragm connects the remaining columns to the building core. As lateral force is imposed on the building, the rigid floor distributes the forces to both the columns and shear walls, which in turn transfer the loads to the ground. The shear walls are typically 12" thick with #11 @12" E.F. vertically (Grade 75) and #6 @9" E.F. horizontally.

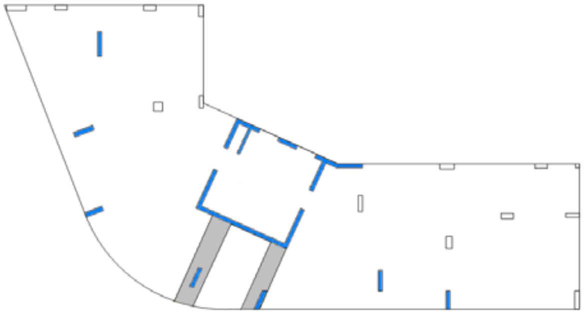


Figure 15: Lateral system with link beams denoted

An additional area of interest concerning load path is found at the cellar level. Here, a combination of large openings in the shear walls and large gravity forces induce enough shear in the link beam that traditional shear reinforcement is not sufficient. Shear forces were significant enough to require the use of a built-up member composed of 1.5" to 2" steel plates, as shown in Figures 16 and 17 below.

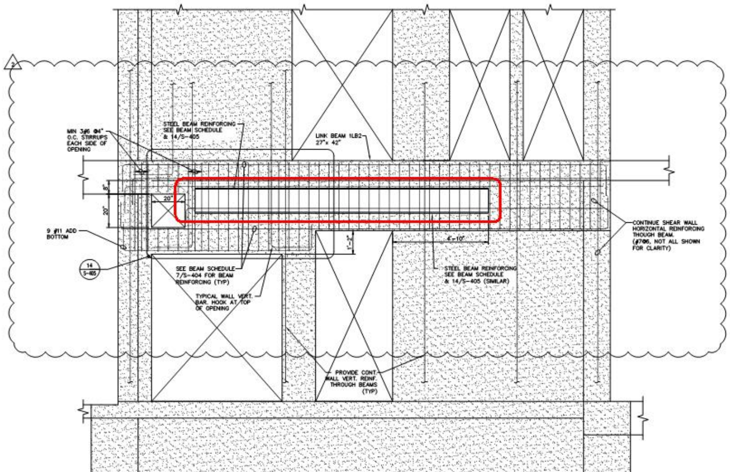


Figure 16: Elevation of Shear Wall & Link Beam

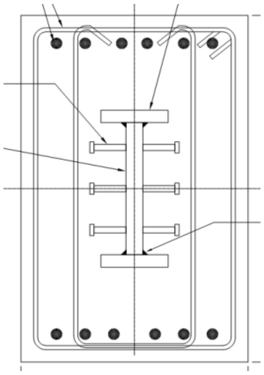


Figure17: Link Beam section showing built-up shape

Design Standards & References

Used in original design

1968 New York City Building Code

ASCE 7-05, Minimum Design Loads for Buildings and Other Structures

ACI 318-99, Building Code Requirements for Structural Concrete

Used in thesis analysis & design

ASCE 7-05 Minimum Design Loads for Buildings and Other Structures

ACI 318-08 Building Code Requirements for Structural Concrete and Commentary, 2008 Edition

Steel Construction Manual, American Institute of Steel Construction, 13th Edition

PCI Industry Handbook, 6th Edition

RS Means Assemblies Cost Data 2009

RS Means Facilities Construction Data 2009

Material Summary

Concrete	f'_c (ksi)
Foundations	5
Slabs	5.95
Columns supporting:	
- Cellar through 9th	8
- 9th through Roof	7
Shear Walls supporting:	
- Cellar through 9th	8
- 9th through Roof	7

Table 1

Reinforcement

- All #11 bars to be Grade 75 steel

- Vertical reinforcement in shear walls to be Grade 75

- Select column reinforcement to be Grade 75

- Remaining reinforcement is ASTM A615, Grade 60

Building Loads

Gravity Loads

Gravity Loads			
Description	**NYC Building Code**	**Design Load**	**ASCE 7-05 Load**
Typical Dead Load			
Normal-Weight Concrete	150 pcf		
Light-Weight Concrete	115 pcf		
Epoxy Terrazzo (3/8")	4 psf		
Superimposed Dead Load			
Partition	18 psf	18 psf	-
MEP	10 psf	10 psf	-
Live Load			
Residential	40 psf	40 psf	40 psf
Corridors	100 psf	100 psf	100 psf
Lobby	100 psf	100 psf	100 psf (1st Floor)**
Assembly	100 psf	100 psf	100 psf
Equipment Rooms	75 psf	75 psf	-
Balconies (exterior)*	60 psf	60 psf	100 psf
Additional Loads			
Planter	4.500 lb		
Curtainwall	500 plf		
* NYCBC requires exterior balconies to carry 150% of live load on adjoining occupied area, but not more than 100 psf			
** All remaining floors same as occupancy served			

Table 2

Curtainwall Load

The double facade system is connected to the concrete slab on levels 1 through 6 via Halfen channel anchors. Therefore, the weight of this complex curtainwall will need to be factored into the dead load of the structure. The structural engineers on the project assumed a 500 plf loading in their design. Once the individual facade reactions were received from the facade consultant, the initial design was checked and found to be sufficient. The 500 plf facade load will be used for initial computations.

Lateral Loads

Wind

The wind pressures used in the original design of 100 Eleventh Avenue were prescribed by New York City's building code, which applied a loading for most buildings in the city of 20 psf for the first 100 feet above grade, 25 psf for 100 to 300 feet above grade, and 30 psf up to 600 feet above grade. Therefore, it is sensible to assume that the New York City code-required loadings will be conservative, compared to that of a more detailed, building-specific calculation method. Because of this, the structural engineer DeSimone Consulting Engineers performed a more detailed wind analysis, as allowed by the city code.

Design pressures in this initial analysis were obtained using Method 2 outlined in Chapter 6 of ASCE 7-05. For the purposes of this report, several assumptions were made in order to simplify the analysis. The width and length of the building in both directions was taken as the projections of the curved facade onto a vertical plane, as shown below. The fundamental period of the building was calculated using approximate equations outlined in Chapter C6 of ASCE 7-05 and the building determined to be flexible. Also worth noting is the building's proximity to the Hudson River on the west, where unobstructed winds result in a more severe exposure category and higher pressures in that direction.

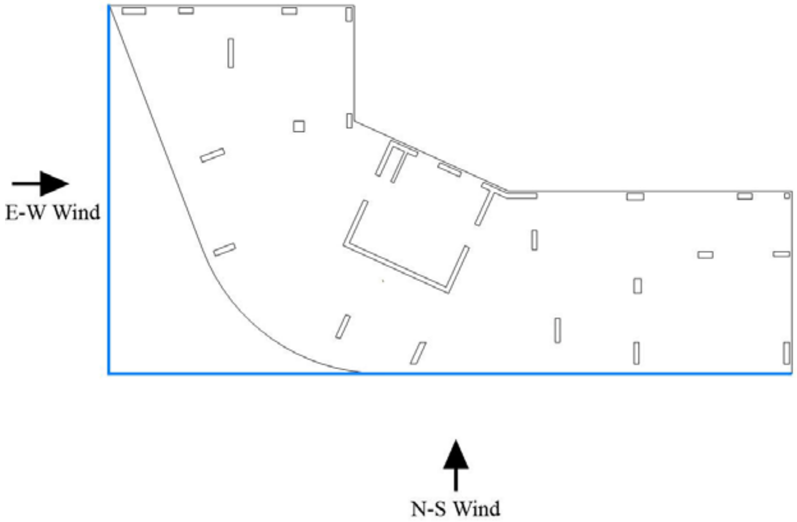


Figure 18: Wind direction axes

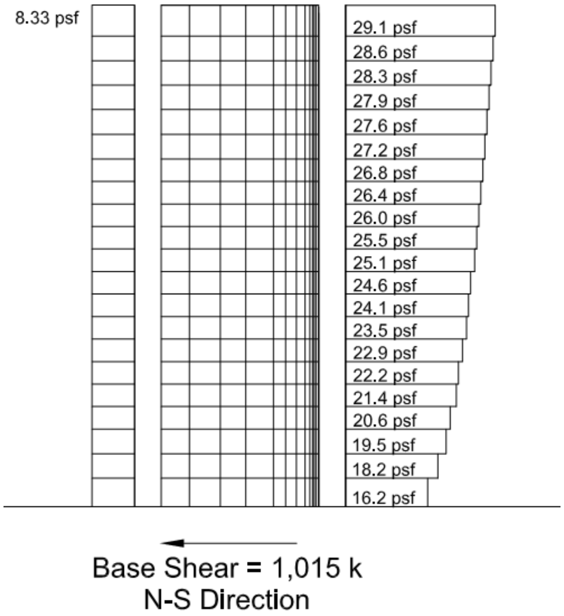


Figure 19: N-S Wind Pressure

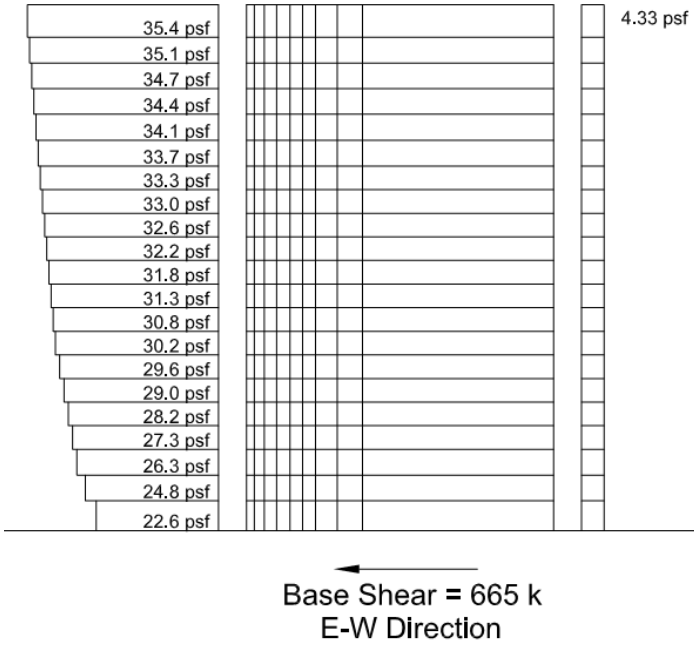


Figure 20: E-W Wind Pressure

Seismic

The equivalent lateral force method detailed in Chapter 12 of ASCE 7-05 was used to generate seismic forces for this report. Shown in Table 3 below is the vertical distribution of seismic forces. The effective seismic weight used in the calculation included structural material, facade, finishes, partitions, and MEP loads. It's important to note that due to the poor soil conditions, 100 Eleventh Avenue does not satisfy the conditions necessary to use the equivalent lateral force method. However, for the purposes of this assignment, it was assumed that the conditions were met.

It is important to keep in mind the simplifications involved with using the equivalent lateral force method to calculate seismic forces. The geotechnical report for this project states that certain portions of the site's soil "should be considered to liquefy during the design earthquake event." This statement alone eliminates the use of the equivalent lateral force method, classifying the site as Site Class F and requiring a site-response analysis. Therefore, the soil conditions are potentially much worse than for what this method accounts, and a site-specific study is likely required.

Vertical Distribution of Seismic Forces

Level	w_x	h_x	h_x^k	$w_x h_x^k$	C_{vx}	F_x (k)
EMR	366	260.9	2484	909740	0.0248	26.1
Roof	1418	244.9	2273	3223377	0.0880	92.4
21	1715	229.8	2079	3565122	0.0973	102.2
20	1687	217.8	1928	3252744	0.0888	93.2
19	1790	205.8	1780	3187036	0.0870	91.4
18	1808	193.8	1636	2958961	0.0808	84.8
17	1808	181.8	1496	2704848	0.0738	77.5
16	1784	169.8	1359	2424760	0.0662	69.5
15	1760	158.8	1237	2177287	0.0594	62.4
14	1760	147.8	1118	1968439	0.0537	56.4
13	1760	136.8	1003	1765795	0.0482	50.6
12	1760	125.8	892	1569648	0.0429	45.0
11	1760	114.8	784	1380331	0.0377	39.6
10	1760	103.8	681	1198227	0.0327	34.3
9	1760	92.8	582	1023782	0.0280	29.3
8	1760	81.8	487	857527	0.0234	24.6
7	1760	70.8	398	700101	0.0191	20.1
6	1922	59.8	314	602894	0.0165	17.3
5	2084	48.8	236	491376	0.0134	14.1
4	2182	37.8	165	359491	0.0098	10.3
3	2387	25.8	96	230076	0.0063	6.6
2	1922	13.8	40	77014	0.0021	2.2
1	3134	0.0	0	0	0.0000	0.0

$\Sigma w_i h_i^k$	36628576
V_{base}	1050 k

Table 3

ETABS Model

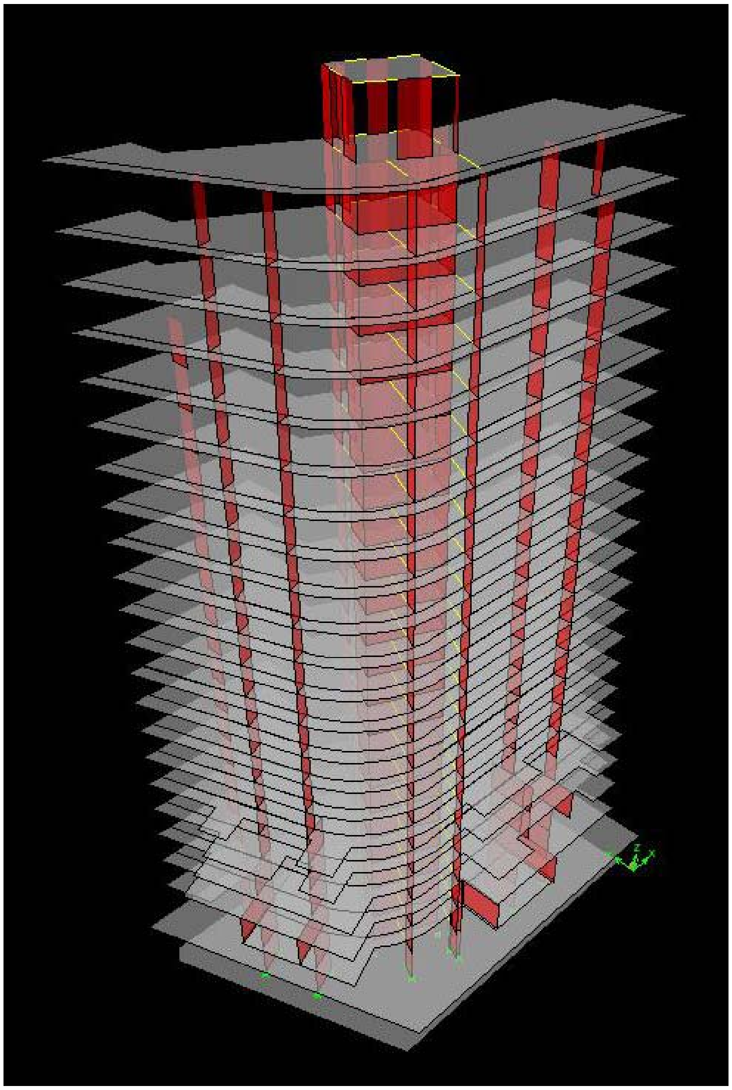


Figure 21: ETABS Model Graphic

100 Eleventh Avenue's lateral system was modeled in ETABS, a building analysis and design software developed by Computers & Structures, Inc. Only the lateral force-resisting columns and shear walls were modeled. The model will be used to verify certain hand calculations, such as center-of-rigidity and center-of-mass. It will also be relied upon to compute more intensive calculations such as story and building drift.

All shear walls and columns were modeled as plate elements with bending thickness 1/10th of the membrane thickness, to approximate membrane behavior while keeping out-of-plane bending from becoming a modeling problem. These objects were then meshed into elements of a maximum size of 24". Coupling beams and the in-slab link beams connecting two columns to the core were modeled as line elements. The concrete slab was modeled as a rigid diaphragm, with only its self-weight and superimposed dead loads applied as gravity loads.

Some important simplifications/assumptions that differentiate the model from reality are listed below:

- **Unless otherwise noted, all concrete sections modeled with full moment of inertia, I_g**
- **Lateral soil pressures acting on sub-grade levels ignored**
- **Only lateral components and loads modeled**

Load Combinations

100 Eleventh Avenue was designed using Allowable Stress Design (ASD) provisions, in which the applied loads are left unfactored. Load and Resistance Factor Design (LRFD) will be used in all thesis analysis and design. The following are the basic factored load combinations outlined in ASCE 7-05 2.3.2:

1. **1.4(D + F)**
2. **1.2(D + F + T) + 1.6(L + H) + 0.5(L_r or S or R)**
3. **1.2D + 1.6(L_r or S or R) + (L or0.8W)**
4. **1.2D + 1.6W + L + 0.5(L_r or S or R)**
5. **1.2D + 1.0E + L + 0.2S**
6. **0.9D + 1.6W + 1.6H**
7. **0.9D + 1.0E + 1.6H**

Only lateral loads were under consideration – more specifically, that of wind and seismic – which reduces the load combinations to the following:

- **1.6W**
- **1.0E**

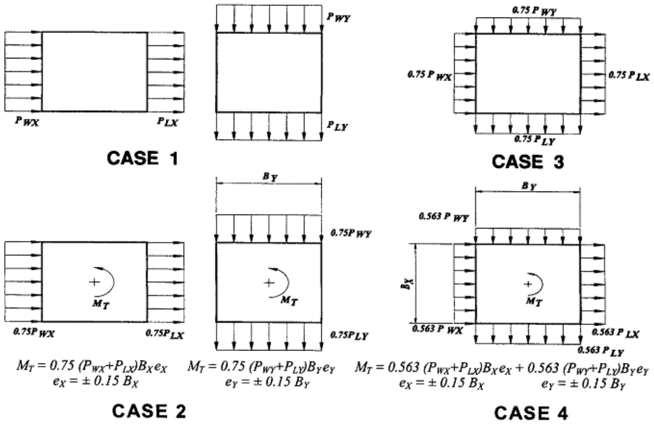


Figure 12: ASCE 7-05 Wind Cases

Wind itself has four cases that need to be considered, as outlined in ASCE 7-05 6.5.12.3 and illustrated in Figure 22. Because the lateral system is not confined to just the building's core, only Cases 1&3 were initially considered, as the additional torsional shear developed in Cases 2&4 are likely to be easily resisted by the seven columns located along the building's perimeter.

Upon inspection, wind was determined to control in both directions. The factored wind load base shears are larger in magnitude than the seismic base shears in both directions and, in general, have a greater eccentricity with respect to the center of rigidity than that of the seismic loads. Throughout the report, all manual calculations will be done with wind as the controlling load combination in both directions.

Because displacement due to wind is a serviceability requirement, building and story drift will require a separate analysis to determine the critical load case. This is addressed in a later section.

Load Path & Distribution

All lateral loads that come into contact with the building require a means of traveling down through the structure to the foundation, where they are transferred to the earth. These forces are assumed to act first on the diaphragm, which then distributes the loads to the lateral force-resisting elements on each floor. Because the diaphragm is assumed rigid, each column or shear wall goes through equal displacements, which dictates that the lateral forces are distributed according to each element's relative stiffness. Therefore, the stiffest column or shear wall will resist the largest percentage of the lateral load.

As noted in previous sections, 100 Eleventh Avenue's lateral system is composed of concrete shear walls found at the building's core and seven columns. The majority of the shear walls are 12" thick, with the exception being on the lower floors, where thicknesses vary from 41" at the sub-cellar level to 16" at the 2nd floor. The columns are expected to contribute little on the upper floors but become more of a factor on levels 1-5 as they begin to stretch to lengths of up to 28'.

In an attempt to gain an understanding of how the lateral loads are distributed in 100 Eleventh Avenue, the relative stiffness of each lateral member was determined by first assuming the walls and columns behave as cantilevers with a height equal to that of the building height. The inverse of the displacement for a cantilever was then used to calculate a member's individual stiffness, using the equations listed below:

$$k = \frac{P}{\Delta} \qquad\qquad k = \frac{1}{\frac{Ph^3}{3EI} + \frac{P \cdot 1.2h}{AG}}$$

The relative stiffness was then found using the following equation:

$$F_i = \frac{k_i}{\Sigma k_j} \cdot P$$

The relative stiffnesses in both directions were calculated for level 8 and level 3. Level 8 is meant to approximate the typical member sizes, while level 3 was chosen to analyze how the long, cantilevered columns contribute to the system. The results are tabulated in Tables 4 & 5 below, along with figures identifying individual members. Coupling beams were ignored, so that each portion of wall separated by an opening was treated as an independent shear wall.

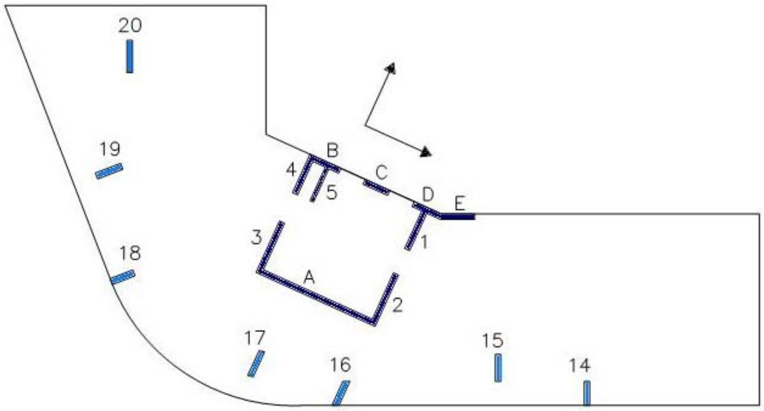


Figure 22: 8th Floor Identification Key

Relative Stiffness

Walls/Columns Supporting 8th Floor

N-S Direction

Member	f'_c (psi)	E (ksi)	h (in)	t (in)	α (deg)	b (in)	b_{eff} (in)	k (k/in)	% Lateral Load Received
SW1	8000	5098	982	12	-	104	-	18	15.0%
SW2	8000	5098	982	12	-	126	-	32	26.6%
SW3	8000	5098	982	12	-	126	-	32	26.6%
SW4	8000	5098	982	12	-	94	-	13	11.1%
SW5	8000	5098	982	8	-	82	-	6	4.9%
C14	8000	5098	982	12	24.5	54	49.1	2	1.6%
C15	8000	5098	982	12	24.5	60	54.6	3	2.2%
C16	8000	5098	982	14	0	60	60.0	4	3.4%
C17	8000	5098	982	12	0	60	60.0	3	2.9%
C18	8000	5098	982	14	46.1	54	37.4	1	0.8%
C19	8000	5098	982	14	46.1	60	41.6	1	1.1%
C20	8000	5098	982	12	24.5	72	65.5	5	3.8%

E-W Direction

Member	f'_c (psi)	E (ksi)	h (in)	t (in)	α (deg)	b (in)	b_{eff} (in)	k (k/in)	% Lateral Load Received
SWA	8000	5098	982	12	-	291	-	374	94.6%
SWB	8000	5098	982	12	-	75	-	7	1.7%
SWC	8000	5098	982	12	-	60	-	3	0.9%
SWD	8000	5098	982	12	-	67	-	5	1.2%
SWE	8000	5098	982	12	24.5	61	55.5	3	0.7%
C14	8000	5098	982	12	24.5	54	22.4	0	0.0%
C15	8000	5098	982	12	24.5	60	24.9	0	0.1%
C18	8000	5098	982	14	46.1	54	38.9	1	0.3%
C19	8000	5098	982	14	46.1	60	43.2	2	0.4%
C20	8000	5098	982	12	24.5	72	29.9	0	0.1%

Table 4

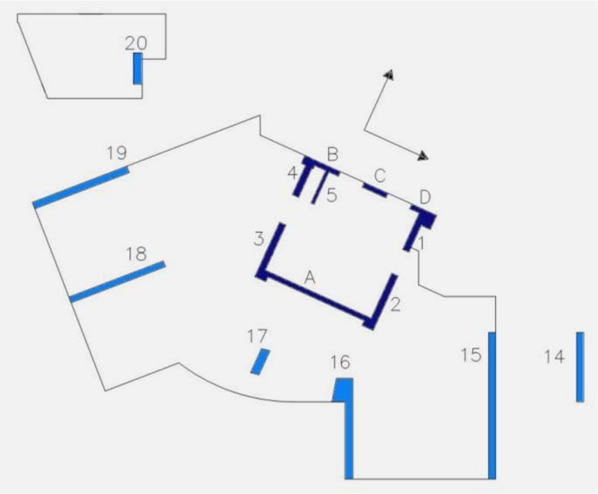


Figure 23: 3rd Floor Identification Key

Relative Stiffness

Walls/Column Supporting 3rd Floor

N-S Direction

Member	f'_c (psi)	E (ksi)	h (in)	t (in)	α (deg)	b (in)	b_{eff} (in)	k (k/in)	% Lateral Load Received
SW1	8000	5098	310	16	-	104	-	712	2.5%
SW2	8000	5098	310	16	-	138	-	1574	5.5%
SW3	8000	5098	310	16	-	138	-	1574	5.5%
SW4	8000	5098	310	16	-	94	-	533	1.9%
SW5	8000	5098	310	8	-	82	-	180	0.6%
C14	8000	5098	310	16	24.5	160	145.6	1823	6.4%
C15	8000	5098	310	16	24.5	338	307.6	11656	41.1%
C16	8000	5098	310	18	24.5	232	211.1	5432	19.1%
C17	8000	5098	310	20	0	60	60.0	180	0.6%
C18	8000	5098	310	14	46.1	232	160.9	2089	7.4%
C19	8000	5098	310	16	46.1	232	160.9	2387	8.4%
C20	8000	5098	310	20	24.5	72	65.5	233	0.8%

E-W Direction

Member	f'_c (psi)	E (ksi)	h (in)	t (in)	α (deg)	b (in)	b_{eff} (in)	k (k/in)	% Lateral Load Received
SWA	8000	5098	310	12	-	299	-	8219	51.0%
SWB	8000	5098	310	12	-	75	-	208	1.3%
SWC	8000	5098	310	12	-	60	-	108	0.7%
SWD	8000	5098	310	12	-	63	-	125	0.8%
C14	8000	5098	310	16	24.5	160	66.4	194	1.2%
C15	8000	5098	310	16	24.5	338	140.2	1643	10.2%
C16	8000	5098	310	18	24.5	232	96.2	641	4.0%
C18	8000	5098	310	14	46.1	232	167.2	2314	14.4%
C19	8000	5098	310	16	46.1	232	167.2	2644	16.4%
C20	8000	5098	310	20	24.5	72	29.9	23	0.1%

Table 5

The stiffnesses were calculated according to lengths in the direction of the core shear wall axis, denoted as two perpendicular arrows in Figures 23 and 24. Therefore, the lengths of any members not aligned with this axis were broken down into their respective components. See Figure B1 in Appendix B for an illustration of this concept.

From this analysis, several conclusions can be reached concerning the load distribution. On the upper floors, the columns' contribution is significant in the N-S direction, where they resist 21% of the lateral load. These same columns do very little in the E-W direction, where they resist less than 5% of the lateral load. On the lower levels, the columns contributions increase significantly. According to this simplified analysis, the columns supporting the 3rd floor resist over 80% of the load in the N-S direction and nearly 50% in the E-W direction.

The limitations of this method are evident in the findings at the 3rd level. It is unlikely that the columns resist such a large percentage of the lateral load. These findings are based only on the column's cross section at the level of interest and pay no attention to the fact that the columns' lengths decrease on lower floors, as seen in Figure 25 for Column 15. Thus, these "long" columns appear much stiffer than they are in reality. The true stiffness likely lies somewhere between that of the upper floors and the values found here.

Similar limitations affect the findings in members at any level, because this method ignores any influence the stories above or below play on the level of interest. Despite these inaccuracies, analyzing the stiffness based on the deflection of a cantilever provides a good approximation of the lateral system's behavior and will serve as an appropriate check on computer software solutions.

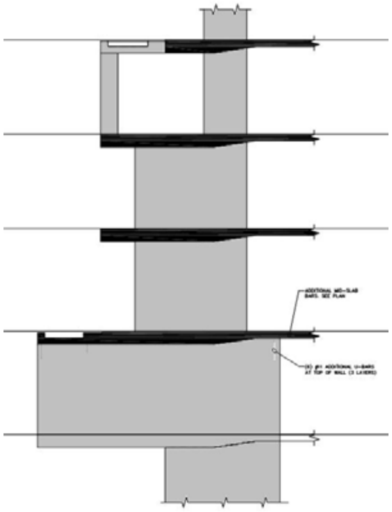


Figure 24: Column 15 Elevation

Torsion

Lateral loads applied to a structure will often induce torsion, a result of the loads being applied at an eccentricity from the building's inherent center of rigidity. Seismic loads act at the structure's center of mass, while wind loads act at the center of pressure. If either the center of mass or center of pressure do not coincide with the center of rigidity, a moment equal to the force times the eccentricity is induced.

Listed in Table 6 are the centers of rigidity, centers of mass, and centers of pressure, as calculated by ETABS. For confirmation purposes, the center of rigidity at the 8th level was also manually calculated using the following equations:

$$\bar{X} = \frac{\sum k_{iy} x_i}{\sum k_{iy}} \qquad \bar{Y} = \frac{\sum k_{ix} y_i}{\sum k_{ix}}$$

The result, also shown in Table 6, was within 98% of the computer analysis in both directions, confirming the model's output. Spreadsheets developed for this computation can be found in Appendix B.

ETABS Output

Level	Center of Rigidity (in)		Center of Mass (in)		Center of Pressure (in)	
	X	Y	X	Y	X	Y
EMR	-970	474	-935	572	-935	572
Roof	-969	458	-903	527	-871	651
21	-967	457	-920	510	-871	651
20	-967	457	-931	524	-871	651
19	-967	456	-901	539	-846	651
18	-966	455	-901	539	-846	651
17	-966	455	-901	539	-846	651
16	-965	454	-901	539	-846	651
15	-964	454	-901	539	-846	651
14	-962	454	-901	539	-846	651
13	-960	454	-901	539	-846	651
12	-958	453	-901	539	-846	651
11	-955	453	-901	539	-846	651
10	-951	453	-901	539	-846	651
9	-945	453	-901	539	-846	651
8	-939	454	-901	539	-846	651
7	-931	455	-901	539	-846	651
6	-922	457	-901	501	-849	562
5	-912	461	-907	493	-849	562
4	-897	466	-932	481	-858	562
3	-873	477	-923	467	-870	562
2	-857	517	-1123	461	-1131	562
Ground	-805	581	-864	542	-896	552

Calculated Center of Rigidity	
X	Y
-958	464

Table 6

An important observation can be taken from Table 6 if the variations of the center of mass and center of pressure from the center of rigidity are compared. In both directions, the center of pressure is approximately twice the distance from the center of rigidity as the center of mass. Therefore, we can be certain the larger wind forces will exert more torsion on the building than seismic loads, as initially stated in determining the controlling load case.

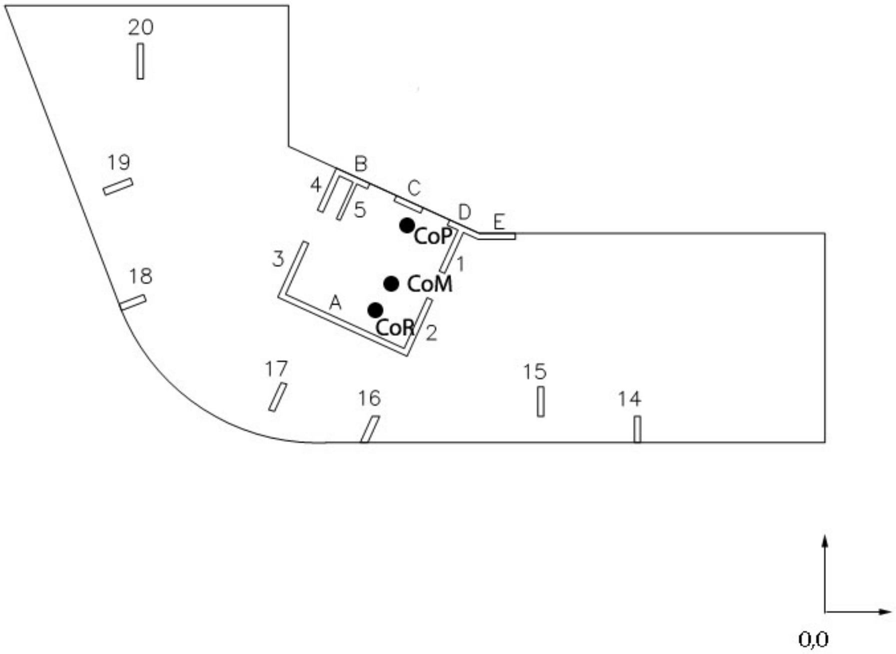
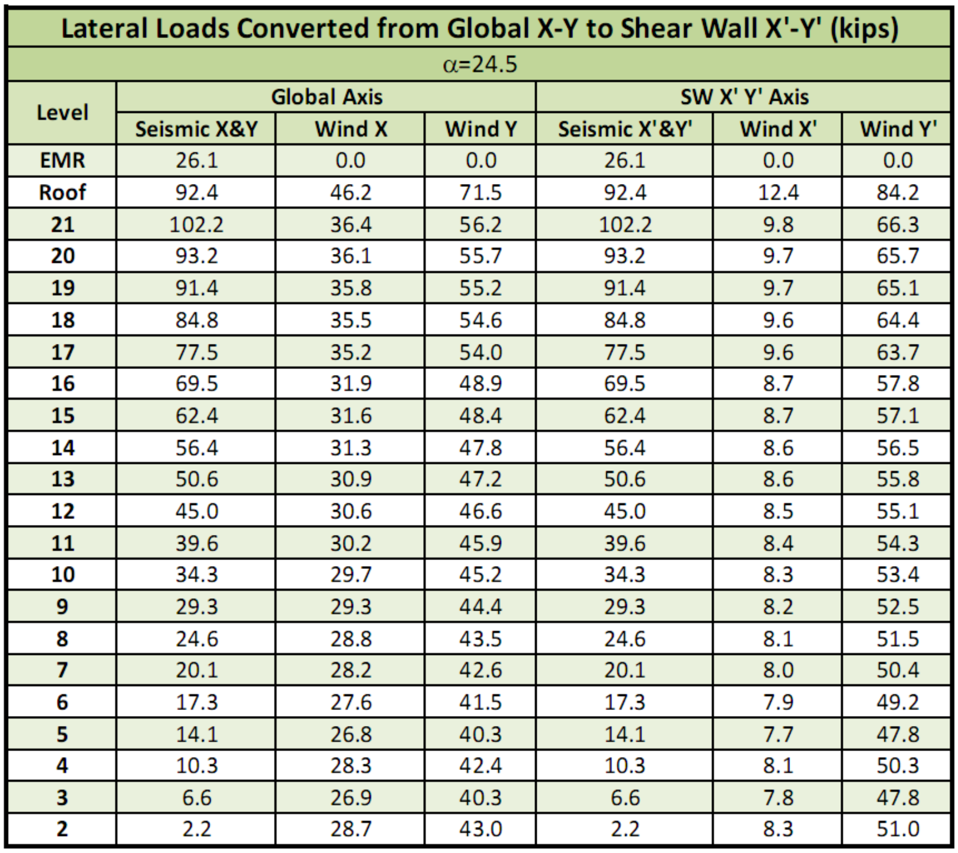


Figure 25: CoR, CoM, CoP Locations

Shear

Direct Shear

Direct shear is a direct result of the applied lateral loads. Because the 9" slab is assumed to act as a rigid diaphragm, direct shear is distributed according to the relative stiffness previously discussed. The lateral loads determined to act on the building have been calculated about a global X and Y axis that is offset 24.5° from that of the shear walls. Therefore, in order to use the relative stiffnesses calculated, the wind and seismic loads acting in the rotated shear wall axis are assumed to be composed of their respective components of the global axis loads previously calculated. This conversion is summarized in Table 7 below. See Figure B3 in Appendix B for an illustration of this concept.



Lateral Loads Converted from Global X-Y to Shear Wall X'-Y' (kips)

α=24.5

Level	Global Axis			SW X' Y' Axis		
	Seismic X&Y	Wind X	Wind Y	Seismic X'&Y'	Wind X'	Wind Y'
EMR	26.1	0.0	0.0	26.1	0.0	0.0
Roof	92.4	46.2	71.5	92.4	12.4	84.2
21	102.2	36.4	56.2	102.2	9.8	66.3
20	93.2	36.1	55.7	93.2	9.7	65.7
19	91.4	35.8	55.2	91.4	9.7	65.1
18	84.8	35.5	54.6	84.8	9.6	64.4
17	77.5	35.2	54.0	77.5	9.6	63.7
16	69.5	31.9	48.9	69.5	8.7	57.8
15	62.4	31.6	48.4	62.4	8.7	57.1
14	56.4	31.3	47.8	56.4	8.6	56.5
13	50.6	30.9	47.2	50.6	8.6	55.8
12	45.0	30.6	46.6	45.0	8.5	55.1
11	39.6	30.2	45.9	39.6	8.4	54.3
10	34.3	29.7	45.2	34.3	8.3	53.4
9	29.3	29.3	44.4	29.3	8.2	52.5
8	24.6	28.8	43.5	24.6	8.1	51.5
7	20.1	28.2	42.6	20.1	8.0	50.4
6	17.3	27.6	41.5	17.3	7.9	49.2
5	14.1	26.8	40.3	14.1	7.7	47.8
4	10.3	28.3	42.4	10.3	8.1	50.3
3	6.6	26.9	40.3	6.6	7.8	47.8
2	2.2	28.7	43.0	2.2	8.3	51.0

Table 7

With the lateral forces resolved into the same axis as the calculated relative stiffnesses, the loads can easily be distributed, as is shown for the 8th floor in Table 8.

Direct Shear - Wind (Case I)

Walls/Columns Supporting 8th Floor

Wind N-S Direction

Member	Factored Story Shear (k)	k (k/in)	% Lateral Load Received	Distributed Shear (k)
SW1	1445.0	18.0	15.0%	216.9
SW2	1445.0	31.9	26.6%	384.3
SW3	1445.0	31.9	26.6%	384.3
SW4	1445.0	13.3	11.1%	160.4
SW5	1445.0	5.9	4.9%	71.1
C14	1445.0	1.9	1.6%	23.0
C15	1445.0	2.6	2.2%	31.6
C16	1445.0	4.1	3.4%	48.9
C17	1445.0	3.5	2.9%	41.9
C18	1445.0	1.0	0.8%	11.9
C19	1445.0	1.4	1.1%	16.3
C20	1445.0	4.5	3.8%	54.5

Wind E-W Direction

Member	Factored Story Shear (k)	k (k/in)	% Lateral Load Received	Distributed Shear (k)
SWA	219	374.3	94.6%	207.6
SWB	219	6.8	1.7%	3.8
SWC	219	3.5	0.9%	1.9
SWD	219	4.8	1.2%	2.7
SWE	219	2.8	0.7%	1.5
C14	219	0.2	0.0%	0.1
C15	219	0.2	0.1%	0.1
C18	219	1.1	0.3%	0.6
C19	219	1.5	0.4%	0.8
C20	219	0.4	0.1%	0.2

Table 8

Torsional Shear

As previously discussed, in addition to direct shear, torsional shear is induced in any structure where the center of mass or pressure is not concentric with the center of rigidity, which is the case for 100 Eleventh Avenue. The torsional shear induced in each member may be determined using the following equation:

$$F_{it} = \frac{k_i d_i P_y \cdot e_x}{\sum k_j d_j^2}$$

d_i=distance from member to center of rigidity
e_x=force eccentricity
k=member stiffness

Continuing with the calculations involving level 8, torsional shear was manually calculated for wind in both directions (Case 1, ASCE 7-05 6.5.12) using eccentricities generated through ETABS, as shown in Table 6. Because columns 14, 15, 18, 19, & 20 are not aligned with the shear walls, each has stiffness in both the N-S & E-W direction that need to be accounted for in resisting the torsional shear. Table 9 below contains the results.

Torsional Shear (Wind Loading - Case I) - Supporting Level 8

Member	k_x	k_y	d_x	d_y	k_xd_y	k_yd_x	$k_xd_y^2$	$k_yd_x^2$	F_x (k)	F_y (k)
SW1	0	18	94	-	0	1701	0	160604	11	1
SW2	0	32	94	-	0	3014	0	284541	20	2
SW3	0	32	-185	-	0	-5907	0	1092808	-39	-3
SW4	0	13	-185	-	0	-2466	0	456119	-16	-1
SW5	0	6	-144	-	0	-851	0	122492	-6	0
C14	0	2	592	25	4	1132	112	670375	7	1
C15	0	3	388	-6	-1	1018	8	394845	7	1
C16	0	4	93	-203	0	379	0	35411	2	0
C17	0	3	-106	-220	0	-369	0	39094	-2	0
C18	1	1	-457	-168	-186	-452	31294	206380	-4	0
C19	2	1	-583	-36	-54	-790	1927	460615	-6	0
C20	0	5	-647	288	124	-2929	35635	1895360	-18	-2
SWA	374	0	-	-29	-10669	0	304053	0	-70	-6
SWB	7	0	-	249	1690	0	420697	0	11	1
SWC	3	0	-	249	866	0	215721	0	6	1
SWD	5	0	-	249	1206	0	300176	0	8	1
SWE	3	0	-	265	730	0	193535	0	5	0
						$\Sigma k_xd_y^2$	7321802			

*See Appendix for eccentricity calculation

Wind Y Story Shear (k)	1445
e_x* (in)	2.93
Wind X Story Shear (k)	219.36
e_y* (in)	217.8

k_i = stiffness in i direction
d_i = distance perpindicular from stiffness direction to center of rigidity
F_k = force due to Wind in K direction

Table 9

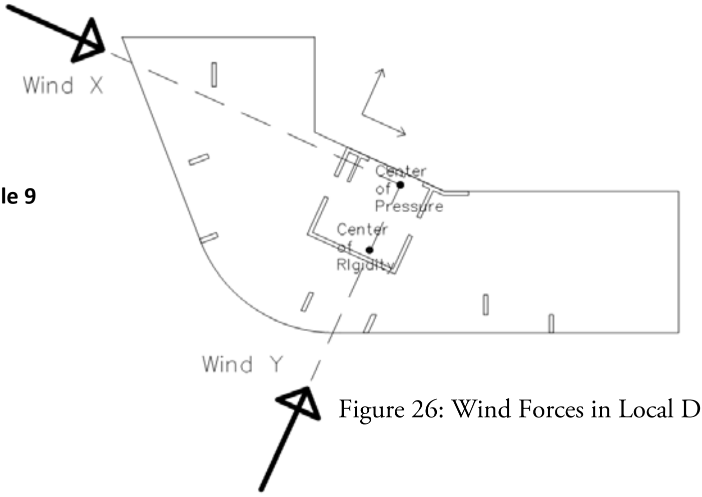
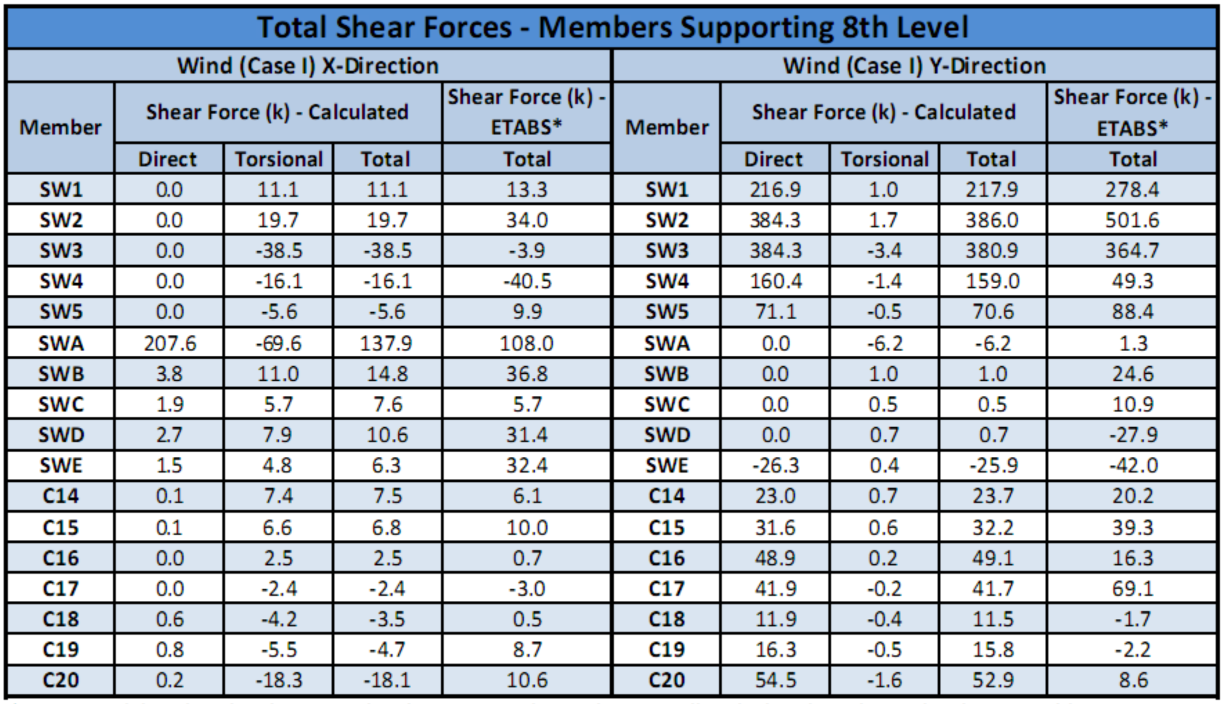


Figure 26: Wind Forces in Local Direction

Because the eccentricity in the x-direction is so small, very little torsional shear is developed by the larger wind force in the y-direction. Due to an eccentricity of 217.8" in the y-direction, the smaller x-directional wind force generates a larger moment in the structure.

It is in the torsional analysis that the value of the columns becomes more apparent. The seven lateral columns participate marginally in resisting the direct shear, which is based only on the

stiffness of a member. Torsional shear, however, is distributed according to both stiffness *and* distance from the center of rigidity. The columns - particularly 14, 19, and 20 – are able to contribute significantly in resisting the moment applied by the offset wind loading because of large distances separating them from the center of rigidity found near the core. The shear walls still resist much of the torsional moment due to their large stiffness but have much less moment arm than do the columns.



Total Shear Forces - Members Supporting 8th Level									
Wind (Case I) X-Direction					Wind (Case I) Y-Direction				
Member	Shear Force (k) - Calculated			Shear Force (k) - ETABS*	Member	Shear Force (k) - Calculated			Shear Force (k) - ETABS*
	Direct	Torsional	Total	Total		Direct	Torsional	Total	Total
SW1	0.0	11.1	11.1	13.3	SW1	216.9	1.0	217.9	278.4
SW2	0.0	19.7	19.7	34.0	SW2	384.3	1.7	386.0	501.6
SW3	0.0	-38.5	-38.5	-3.9	SW3	384.3	-3.4	380.9	364.7
SW4	0.0	-16.1	-16.1	-40.5	SW4	160.4	-1.4	159.0	49.3
SW5	0.0	-5.6	-5.6	9.9	SW5	71.1	-0.5	70.6	88.4
SWA	207.6	-69.6	137.9	108.0	SWA	0.0	-6.2	-6.2	1.3
SWB	3.8	11.0	14.8	36.8	SWB	0.0	1.0	1.0	24.6
SWC	1.9	5.7	7.6	5.7	SWC	0.0	0.5	0.5	10.9
SWD	2.7	7.9	10.6	31.4	SWD	0.0	0.7	0.7	-27.9
SWE	1.5	4.8	6.3	32.4	SWE	-26.3	0.4	-25.9	-42.0
C14	0.1	7.4	7.5	6.1	C14	23.0	0.7	23.7	20.2
C15	0.1	6.6	6.8	10.0	C15	31.6	0.6	32.2	39.3
C16	0.0	2.5	2.5	0.7	C16	48.9	0.2	49.1	16.3
C17	0.0	-2.4	-2.4	-3.0	C17	41.9	-0.2	41.7	69.1
C18	0.6	-4.2	-3.5	0.5	C18	11.9	-0.4	11.5	-1.7
C19	0.8	-5.5	-4.7	8.7	C19	16.3	-0.5	15.8	-2.2
C20	0.2	-18.3	-18.1	10.6	C20	54.5	-1.6	52.9	8.6

*ETABS model analyzed without coupling beams to replicate the manually calculated results as closely as possible

Table 10

Table 10 displays the total manually calculated results for the controlling wind load case compared to the ETABS model results. As one can see, in many members, there are vast differences between the two. One major explanation for this is the inaccuracies in treating each shear wall/column as an independent cantilevered wall. It could be argued that these members could be better modeled as fixed-fixed walls spanning from floor-to-floor. Relative stiffness would then be dominated by shear deflections (proportional to length) rather than flexural deflections (proportional to length^3). For example, SW A is significantly longer than any other wall or column in its direction; thus, it takes 94% of the load in the E-W direction when dominated by flexural deflections. When modeled as a fixed-fixed wall, this distribution lessens to just 54.1%. This contribution from SW A seems more logical, and in fact is in closer agreement with the ETABS model, which distributes just 30% of the E-W wind load to SW A.

In addition and as previously discussed, the significant increases in length of a column on the lower floors, as shown for Column 15 in Figure 28, will affect the column's stiffness on the upper floors. This is taken into account in the ETABS model and ignored in the manual computations.

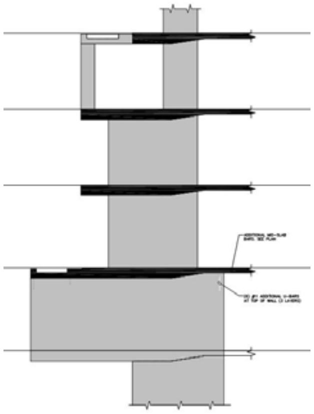


Figure 28: Column 15 Elevation

Member Shear Checks

Member shear checks were performed for Level 8, using the manually computed forces and are summarized in Table 11 below.
In addition to the full wind load being applied separately in each direction, ASCE 7-05 dictates that the case of 75% of the wind load being applied in each direction concurrently must be checked (Case III). Case I was found to control in each member, with the controlling force denoted in bold. The shear strength of concrete was determined using the following equation from ACI 318-08 for walls with horizontal in-plane shear forces:

$$V_c = 3.3\lambda\sqrt{(f'_c)}hd + (N_u d)/(4l_w) \qquad \text{Eq (11-27)}$$

The height of the wall was assumed to be the story height. Gravity loads were conservatively ignored, eliminating the second term in the above equation. It was assumed that the columns, each of which is at least 4.5' in length, will behave as shear walls in response to lateral load and that Eq (11-27) applies to them. The strength contribution of the steel reinforcing was calculated using the following equation:

$$V_s = (A_s * f_y * d)/(s)$$

As can be seen, all shear walls and columns have adequate shear capacity to resist the calculated member forces. By visual inspection, it can also be seen that the column and wall shear strength is sufficient to resist the member loads attained from the ETABS model, with the exception of SW 2, which has a capacity of 490k and a factored load of 501.6k. It is likely that this shear wall will be sufficiently strong with the inclusion of gravity loads in Eq 11-27.

Member Shear Checks - Supporting 8th Level

Member	Case I		Case III	h (in)	l_w (in)	$d=0.8l_w$	s (in)	A_s (in^2)	Conc. Strength (k)	Steel Strength (k)	ϕV_n	$(\phi V_n)_{max}$*	Check
	Wind X (k)	Wind Y (k)	0.75*(WindX+WindY) (k)										
SW1	11	**218**	172	12	104	83.2	9	0.44	295	244	404	670	OK
SW2	20	**386**	304	12	126	100.8	9	0.44	357	296	490	811	OK
SW3	-39	**381**	257	12	126	100.8	9	0.44	357	296	490	811	OK
SW4	-16	**159**	107	12	94	75.2	9	0.44	266	221	365	605	OK
SW5	-6	**71**	49	12	82	65.6	9	0.44	232	192	319	528	OK
SWA	**138**	-6	99	12	291	232.8	9	0.44	825	683	1131	1874	OK
SWB	**15**	1	12	12	75	60	9	0.44	213	176	291	483	OK
SWC	**8**	1	6	12	60	48	9	0.44	170	141	233	386	OK
SWD	**11**	1	8	12	67	53.6	9	0.44	190	157	260	431	OK
SWE	**6**	0	5	8	61	48.8	9	0.44	115	143	194	262	OK
C14	8	**24**	23	12	54	43.2	12	0.11	153	24	133	348	OK
C15	7	**32**	29	12	60	48	6	0.2	170	96	200	386	OK
C16	2	**49**	39	14	60	48	6	0.11	198	53	188	451	OK
C17	-2	**42**	29	12	60	48	6	0.11	170	53	167	386	OK
C18	-4	**12**	6	14	54	43.2	12	0.11	179	24	152	406	OK
C19	-5	**16**	8	14	60	48	12	0.11	198	26	169	451	OK
C20	-18	**53**	26	12	72	57.6	12	0.11	204	32	177	464	OK

*$(\phi V_n)_{max} = \phi 10\sqrt{f'_c}bd$

Table 11

Drift and Displacement

Due to the complexity involved with determining the story drift and building deflection of a building such as 100 Eleventh Avenue by hand, these values were taken from ETABS and then compared to acceptable values. In order to attain as accurate results as possible, concrete sections were "cracked" using modifiers of $0.85I_g$ for walls and columns and $0.50I_g$ for beams. Story drifts were taken at the center of mass of each story level, in accordance with ASCE 7-05 Section 12.8.6.

Earthquake story drift was looked at in both directions and compared to the allowable seismic story drift from Table 12.12-1 in ASCE 7-05, of $0.020h_{sx}$. Wind story drifts and overall displacement was compared to the industry standard for allowable drift due to wind of *L/400*.

As Table 12 shows, all earthquake drift requirements were met, while story drift in the E-W direction and both story drift and overall building drift in the N-S direction failed to meet industry standards on multiple levels. Levels of particular concern are 8-16, where story drifts reach twice the recommended limit. However, it's important to keep in mind that the cellar and ground levels will be restrained from displacements by lateral soil pressures, something not accounted for in the ETABS model. This would decrease displacements somewhat.

Story Data			EQ Story Drift (Strength Loads)			Wind Building & Story Drift (Service Loads)					
			E-W	N-S	Limit	E-W		N-S		Limit	
Story	Height (in)	Elevation (in)	Story Drift (in)	Story Drift (in)	Max Story Drift=$0.02h_{sx}$	Building Drift (in)	Story Drift (in)	Building Drift (in)	Story Drift (in)	Max Bldg Drift = L/400	Max Story Drift = L/400
EMR	-	3219	0.74	0.78	3.84	5.45	0.35	8.40	0.54	8.05	0.48
Roof	192	3027	0.77	0.89	3.62	4.90	0.38	7.99	0.58	7.57	0.45
21	181	2846	0.63	0.75	2.88	4.50	0.31	7.42	0.49	7.12	0.36
20	144	2702	0.65	0.72	2.88	4.30	0.33	6.96	0.48	6.76	0.36
19	144	2558	0.68	0.87	2.88	4.11	0.35	6.65	0.55	6.40	0.36
18	144	2414	0.71	0.93	2.88	3.84	0.36	6.22	0.59	6.04	0.36
17	144	2270	0.73	0.99	2.88	3.57	0.38	5.78	0.63	5.68	0.36
16	144	2126	0.69	0.96	2.64	3.29	0.37	5.33	0.61	5.32	0.33
15	132	1994	0.71	1.00	2.64	3.04	0.38	4.91	0.64	4.99	0.33
14	132	1862	0.72	1.03	2.64	2.78	0.39	4.48	0.66	4.66	0.33
13	132	1730	0.73	1.06	2.64	2.53	0.40	4.05	0.68	4.33	0.33
12	132	1598	0.73	1.07	2.64	2.27	0.41	3.62	0.70	4.00	0.33
11	132	1466	0.72	1.06	2.64	2.02	0.42	3.19	0.71	3.67	0.33
10	132	1334	0.71	1.04	2.64	1.77	0.42	2.77	0.70	3.34	0.33
9	132	1202	0.69	0.99	2.64	1.53	0.42	2.35	0.68	3.01	0.33
8	132	1070	0.66	0.93	2.64	1.29	0.41	1.95	0.65	2.68	0.33
7	132	938	0.62	0.82	2.64	1.06	0.39	1.57	0.59	2.35	0.33
6	132	806	0.57	0.65	2.64	0.81	0.36	1.22	0.48	2.02	0.33
5	132	674	0.55	0.46	2.88	0.61	0.34	0.91	0.34	1.69	0.36
4	144	530	0.47	0.35	2.88	0.43	0.29	0.65	0.26	1.33	0.36
3	144	386	0.41	0.32	3.32	0.26	0.25	0.41	0.25	0.97	0.42
2	166	220	0.24	0.20	2.96	0.13	0.16	0.21	0.16	0.55	0.37
1	148	72	0.04	0.04	1.44	0.03	0.02	0.06	0.03	0.18	0.18
Cellar	72	0	-	-	-	-	-	-	-	-	-

Black=Drift Limit Not Exceeded
Red=Drift Limit Exceeded

Table 12

Overturning

The critical overturning direction will likely be in the direction of the structure's least depth in which to resist the applied forces. Thus, for 100 Eleventh Avenue, the direction shown in Figure 29 was analyzed for overturning. In addition, wind forces are largest in this direction. The depth of 35' is the distance from columns 16 and 17 to shear walls B, C, and D. To develop a rough estimate of the possibility of overturning, the structure was simplified to a system with a depth of 35', forces equal to the total wind force on the building, and a resisting dead load equal to the weight of the building acting at its center, as shown by Figure 30. Table 13 summarizes the results, showing the wind-induced moment is well within the limits of the resisting dead load.

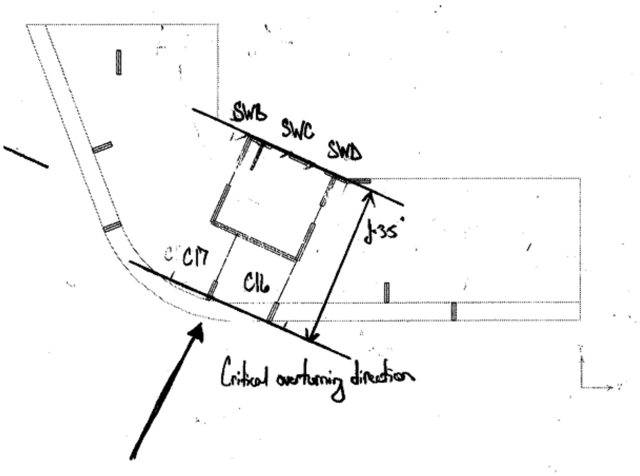


Figure 29: Critical Overturning Direction

The value of including the columns into the lateral system becomes clearly evident here. A common ratio used in working with overturning in a building is its height/depth ratio. A higher ratio corresponds to a higher tendency for overturning and deflection. In the direction analyzed, the h/d ratio was 250'/35' = 7.1. Without columns 16 and 17, h/d becomes 250'/24' = 10.4, over a 40% increase. This ratio is hardly necessary to prove this point, as it is visually clear that connecting these columns to the core shear wall via in-slab link beams, much like an outrigger system, will provide for a much more "stout" structure.

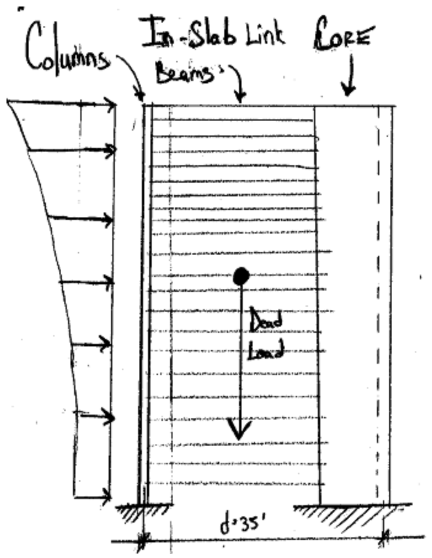


Figure30: Overturning System Elevation

Overturning Moment			
Level	Total Wind Pressure (ksf)	Elevation (in)	Moment (ft-k)
Roof	135	3027	33977
21	106	2846	25149
20	105	2702	23662
19	104	2558	22190
18	103	2414	20733
17	102	2270	19291
16	92	2126	16377
15	91	1994	15194
14	90	1862	14024
13	89	1730	12870
12	88	1598	11731
11	87	1466	10608
10	85	1334	9503
9	84	1202	8416
8	82	1070	7350
7	81	938	6306
6	79	806	5286
5	76	674	4294
4	80	530	3555
3	77	386	2461
2	82	220	1496
		ΣM	274473

Resisting Dead Load = (Building Weight) x (Moment Arm)
= (41,852k)x(35'/2) = 732,410 ft-k > 274,473 ft-k OK

Table 13

Summary & Conclusions

Technical Report III is an initial analysis of 100 Eleventh Avenue's existing lateral system. Using lateral loads calculated from Technical Report I and the load combinations listed in ASCE 7-05, wind was determined to control over seismic in both directions, due to the fact that wind loads were higher in magnitude with larger eccentricities. In the interest of being thorough, all load combinations involving wind and seismic (including wind Cases I-IV) were analyzed with the ETABS model. The displacement of the center of mass at the 22nd story in the E-W direction due to seismic load was larger than that due to the wind loads. This is unexpected, due to the initial conclusion that wind forces control in both directions. Forces in individual members where then looked, with the conclusion being that different members are controlled by different load cases. Therefore, there doesn't seem to be a single controlling load case, as the design of each structural member is dictated by the combination of forces that imparts the most critical loads on it, regardless of whether or not the combination gives the largest displacement on a given floor. The initial conclusion of wind controlling in both directions provided a solid starting point for the structure's lateral system to be analyzed, understood, and verified.

Distribution of direct shear to the members was manually calculated according to relative stiffness. It was determined that while the load resisted by the columns was not negligible, the majority of lateral load was taken by the longer, stiffer shear walls. Torsional shear was then distributed to each member according to its stiffness and distance from the center of rigidity. With the total shear on each member known, shear capacity checks were performed using ACI 318-08, with all members having sufficient strength to resists the manually calculated loads. An approximate overturning analysis was also performed, and the dead load was verified to be sufficient in resisting the overturning wind force.

Analyzing the distribution of forces provided insight into the designer's reasons for the inclusion of columns in the lateral force-resisting system. The columns perform well in resisting torsional shear because of their distance from the center of rigidity. Each member's resisting moment is a function of stiffness and distance, the latter of which benefits the columns, most of which are situated along the perimeter of the building. Their impact is further increased when overturning is considered. The addition of columns increases the depth of the lateral system, which increases the ability of a structure to resist overturning.

The building displacements and story drift output from ETABS were checked against code and industry standards. According to the model, seismic story drifts satisfy code requirements. Wind story drift and displacement is of some concern, however, as it did not meet the recommended limits for many of the stories in both directions. While this is certainly undesirable and will need to be looked into further, it is important to keep in mind that the wind limits are *recommended*, not required.

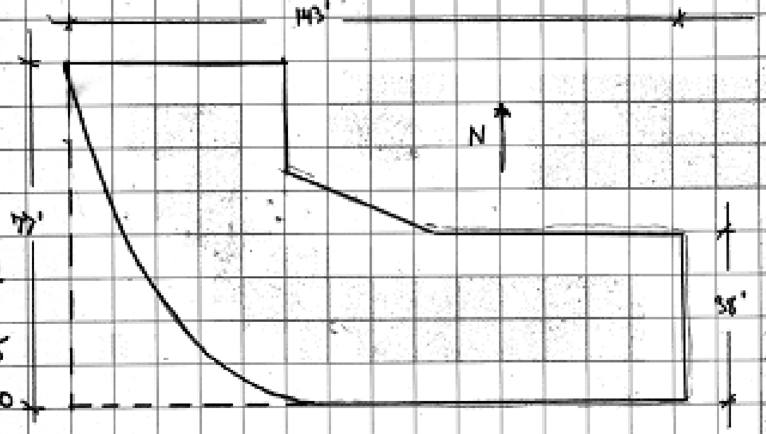
This report confirms that the lateral strength provided by the combination of shear walls and columns is sufficient. Serviceability limitations were unable to be entirely confirmed and may need to be analyzed further. In addition to these confirmations, this analysis provided a proper starting point for a better understanding of 100 Eleventh Avenue's lateral system.

APPENDIX A

LOAD CALCULATIONS

WIND

100 11th Avenue - Wind Design Pressure Calculation



Basic Wind Speed: V = 110 mph

$K_d = 0.85$

I = 1.0 (Category II)

Exposure Category:

- Surface Roughness Category B

N-S E-W

	N-S	E-W
Exposure	B	C
	$\alpha = 7.0$	$\alpha = 9.5$
	$z_g = 1200$	$z_g = 900$

$k_z = 2.01(z/z_g)^{2/\alpha}$ → or See Table 6-3

k_h: $k_h = 1.28$ $k_h = 1.53$

Topographic Effects: $k_{zt} = 1.0$

q_z:

$q_z = 0.00256\, k_z k_{zt} k_d V^2 I$

$= 0.00256 (k_z)(1.0)(0.85)(110)^2(1.0)$

$q_z = 26.33\, k_z$ (see Table for tabulated values)

$q_h = 26.33(1.28) = 33.7$ psf ← N-S

$26.33(1.53) = 40.3$ psf ← E-W

Gust Effect Factor

N-S: $B = 143'$ $h = 245'$

$L = 77'$ $\beta = 2\%$ (as recommended by ASCE 07-05 in C6.5.8)

E-W: B = 77'
L = 143'

$n_1 = \frac{150}{H} = \frac{150}{245} = 0.60$ (C6-19) < 1.0 → Building is Flexible

[Also quick check of $\frac{H}{W} = \frac{245'}{38'} = 6.44 > 4$ suggests structure is Flexible]

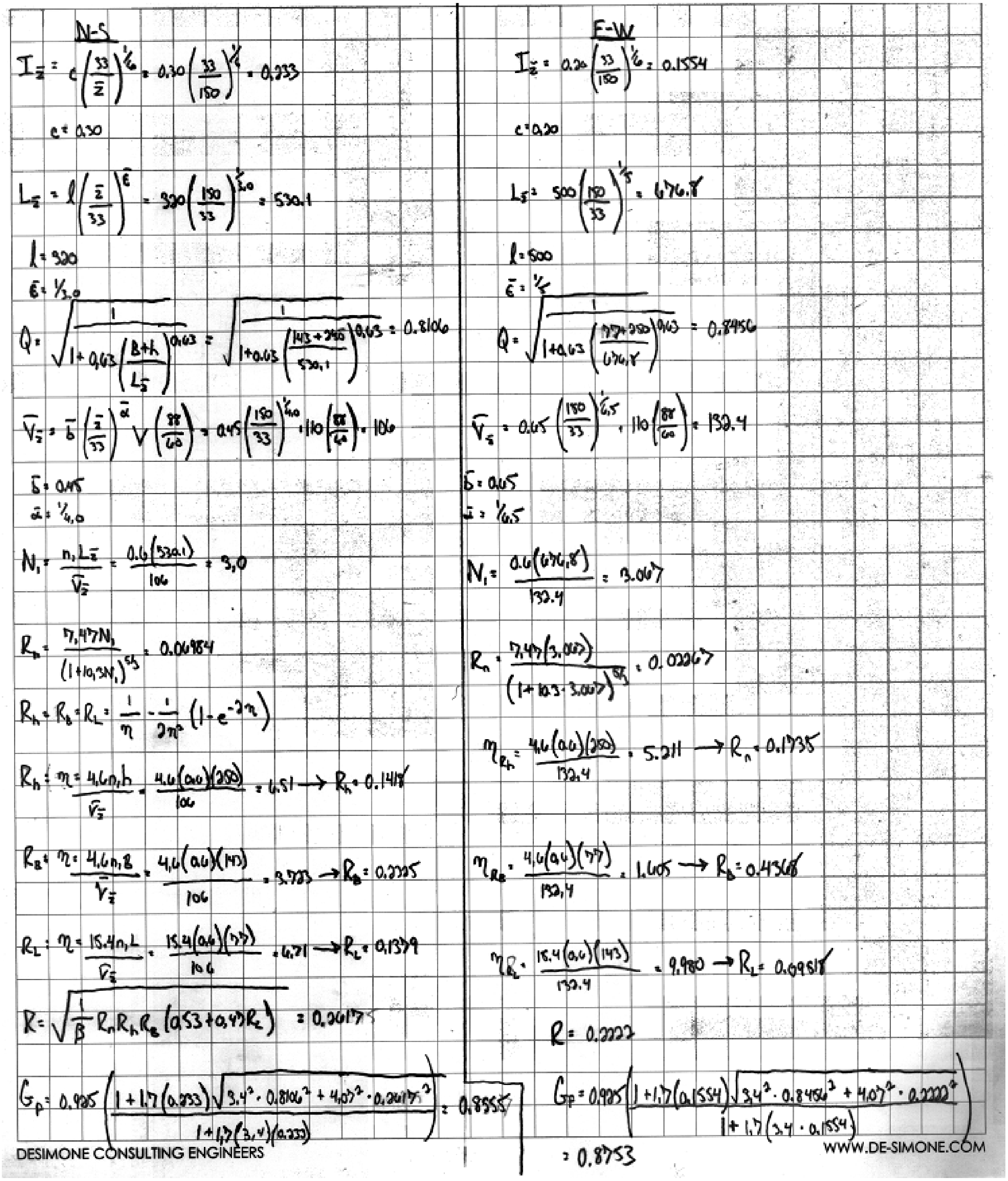
$g_Q = g_v = 3.4$

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

$= \sqrt{2\ln(3600 \cdot 0.6)} + \frac{0.577}{\sqrt{2\ln(3600 \cdot 0.6)}} = 3.92 + 0.147 = 4.07$

$\bar{z} = 0.6h = 0.6(250) = 150' > z_{min} = 30$ (N-S)

$z_{min} = 15$ (N-S)



N-S

$I_{\bar z} = c\left(\frac{33}{\bar z}\right)^{1/6} = 0.30\left(\frac{33}{150}\right)^{1/6} = 0.233$

$c = 0.30$

$L_{\bar z} = l\left(\frac{\bar z}{33}\right)^{\bar\epsilon} = 320\left(\frac{150}{33}\right)^{1/3.0} = 530.1$

$l = 320$

$\bar\epsilon = 1/3.0$

$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar z}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{143+250}{530.1}\right)^{0.63}}} = 0.8106$

$\bar V_{\bar z} = \bar b\left(\frac{\bar z}{33}\right)^{\bar\alpha} V\left(\frac{88}{60}\right) = 0.45\left(\frac{150}{33}\right)^{1/4.0} \cdot 110\left(\frac{88}{60}\right) = 106$

$\bar b = 0.45$

$\bar\alpha = 1/4.0$

$N_1 = \frac{n_1 L_{\bar z}}{\bar V_{\bar z}} = \frac{0.6(530.1)}{106} = 3.0$

$R_n = \frac{7.47N_1}{(1+10.3N_1)^{5/3}} = 0.06984$

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

$R_h: \eta = \frac{4.6n_1h}{\bar V_{\bar z}} = \frac{4.6(0.6)(250)}{106} = 6.51 \rightarrow R_h = 0.1418$

$R_B: \eta = \frac{4.6n_1B}{\bar V_{\bar z}} = \frac{4.6(0.6)(143)}{106} = 3.723 \rightarrow R_B = 0.2225$

$R_L: \eta = \frac{15.4n_1L}{\bar V_{\bar z}} = \frac{15.4(0.6)(77)}{106} = 6.71 \rightarrow R_L = 0.1379$

$R = \sqrt{\frac{1}{\beta}R_nR_hR_B(0.53+0.47R_L)} = 0.2617$

$G_p = 0.925\left(\frac{1+1.7(0.233)\sqrt{3.4^2\cdot 0.8106^2 + 4.07^2\cdot 0.2617^2}}{1+1.7(3.4)(0.233)}\right) = \boxed{0.8555}$

E-W

$I_{\bar z} = 0.20\left(\frac{33}{150}\right)^{1/6} = 0.1554$

$c = 0.20$

$L_{\bar z} = 500\left(\frac{150}{33}\right)^{1/5} = 676.8$

$l = 500$

$\bar\epsilon = 1/5$

$Q = \sqrt{\frac{1}{1+0.63\left(\frac{77+250}{676.8}\right)^{0.63}}} = 0.8456$

$\bar V_{\bar z} = 0.65\left(\frac{150}{33}\right)^{1/6.5} \cdot 110\left(\frac{88}{60}\right) = 132.4$

$\bar b = 0.65$

$\bar\alpha = 1/6.5$

$N_1 = \frac{0.6(676.8)}{132.4} = 3.067$

$R_n = \frac{7.47(3.067)}{(1+10.3\cdot 3.067)^{5/3}} = 0.02267$

$\eta_{R_h} = \frac{4.6(0.6)(250)}{132.4} = 5.211 \rightarrow R_n = 0.1735$

$\eta_{R_B} = \frac{4.6(0.6)(77)}{132.4} = 1.605 \rightarrow R_b = 0.4368$

$\eta_{R_L} = \frac{15.4(0.6)(143)}{132.4} = 9.980 \rightarrow R_L = 0.09818$

$R = 0.2222$

$G_p = 0.925\left(\frac{1+1.7(0.1554)\sqrt{3.4^2\cdot 0.8456^2 + 4.07^2\cdot 0.2222^2}}{1+1.7(3.4\cdot 0.1554)}\right)$

$= 0.8753$

DESIMONE CONSULTING ENGINEERS

WWW.DE-SIMONE.COM

Enclosure Classification → Enclosed

5' parapet at roof level

$q_p = 26.33\left(2.01\left(\frac{255}{1200}\right)^{2/7}\right) = 34.0$ (N-S)

$= 26.33\left(2.01\left(\frac{255}{900}\right)^{2/9.5}\right) = 40.58$ (E-W)

$GC_{pn} = +1.5$ ww
-1.0 lw

N-S

$p_p = q_p GC_{pn} = 34 \times 1.5 = 51$ psf ww
$= 34 \times 1.0 = 34$ psf lw

E-W

$p_p = 40.58 \times 1.5 = 60.87$ psf
$= 40.58 \times 1.0 = 40.58$ psf

} Design parapet wind pressures

MB

N-S

$\frac{L}{B} = \frac{77}{143} < 1 \rightarrow C_p = 0.8$ windward
$C_p = -0.5$ leeward

$GC_{pi} = \begin{matrix} +0.18 \\ -0.18 \end{matrix}$

Windward

$p_z = q_z G_f C_p \mp q_h (GC_{pi})$
$= q_z (0.8555)(0.8) - (33.7)(\pm 0.18)$
$= 0.6844 q_z \mp 6.066$

Leeward

$p_z = q_h (0.8555)(-0.5) \mp (33.7)(\pm 0.18) =$
$= -$[illegible]

E-W

$\frac{L}{B} = \frac{143}{77} \geq 1 > 2 \rightarrow C_p = 0.8$ ww
$C_p = -0.329$ lw (interpolated)

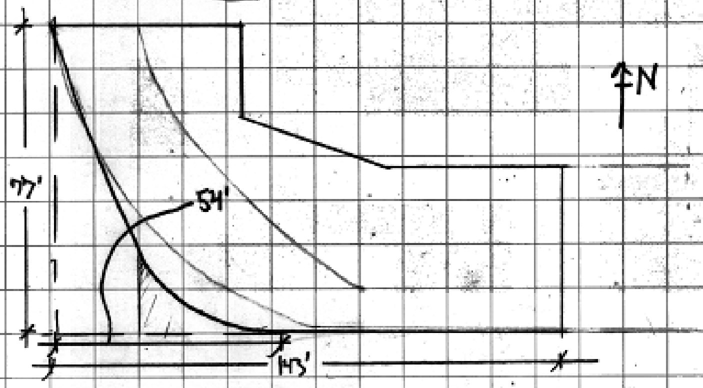
Windward

$p_z = q_z (0.8753)(0.8) \pm (40.3)(\pm 0.18)$
$= 0.70 q_z \pm 7.254$

Leeward

$p_z = q_h G_f C_p - q_h (GC_{pi})$
$= (40.3)(0.8753)(-0.329) - (40.3)(\pm 0.18)$
$= -18.86$ psf

Force Calculation



Wind pressures will be multiplied by tributary areas to obtain base shear.

Portion of building that slopes away will be multiplied by a factor (assumed here to be 0.7) to attempt in accounting for the difference in the assumed orthogonal profile behavior and the actual curved profile behavior. In E-W direction, facade does not slope away as severely and no factor will be applied.

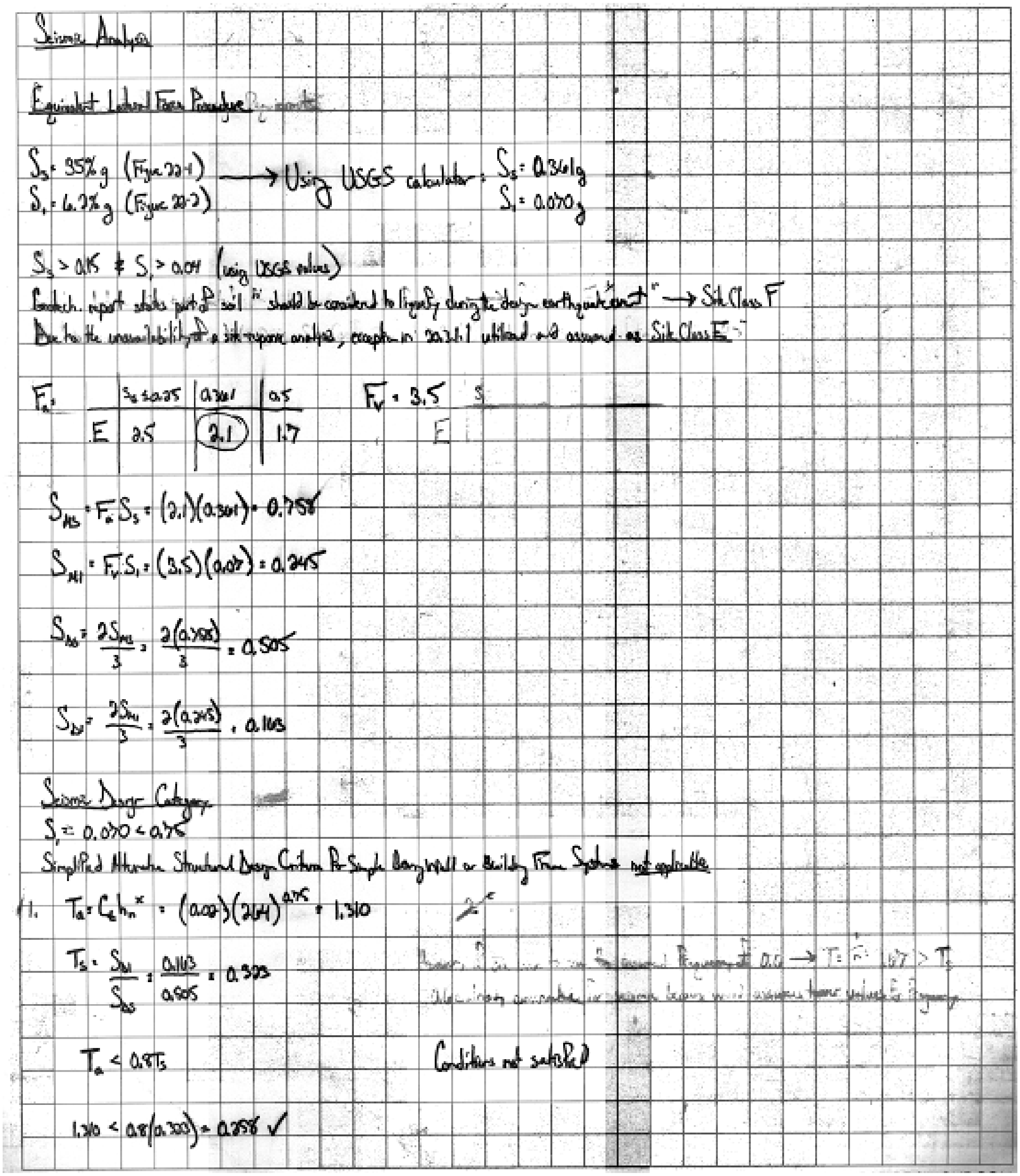
Design Wind Pressures in N-S Direction										Forces	
Location	Level	Height (ft)	Floor Height (ft)	K_z	q_z	External Pressure $q_zG_fC_p$ (psf)	Internal Pressure $q_h(GC_{pi})$ (psf)	Net pressure (psf) $+(GC_{pi})$	Net pressure (psf) $-(GC_{pi})$	Trib Area (sf)	Net Force (w/o Internal Pressure) (k)
Windward	1	13.83	13.83	0.562	14.79	10.12	±6.050	4.07	16.17	1754	43
	2	12.00	25.83	0.671	17.67	12.10	±6.050	6.05	18.15	1522	40
	3	12.00	37.83	0.749	19.71	13.49	±6.050	7.44	19.54	1522	42
	4	11.00	48.83	0.805	21.20	14.51	±6.050	8.46	20.56	1395	40
	5	11.00	59.83	0.853	22.47	15.38	±6.050	9.33	21.43	1395	42
	6	11.00	70.83	0.895	23.58	16.14	±6.050	10.09	22.19	1395	43
	7	11.00	81.83	0.933	24.57	16.82	±6.050	10.77	22.87	1395	44
	8	11.00	92.83	0.967	25.47	17.43	±6.050	11.38	23.48	1395	44
	9	11.00	103.83	0.999	26.30	18.00	±6.050	11.95	24.05	1395	45
	10	11.00	114.83	1.028	27.07	18.53	±6.050	12.48	24.58	1395	46
	11	11.00	125.83	1.055	27.79	19.02	±6.050	12.97	25.07	1395	47
	12	11.00	136.83	1.081	28.46	19.48	±6.050	13.43	25.53	1395	47
	13	11.00	147.83	1.105	29.09	19.91	±6.050	13.86	25.96	1395	48
	14	11.00	158.83	1.128	29.70	20.32	±6.050	14.27	26.37	1395	48
	15	11.00	169.83	1.150	30.27	20.72	±6.050	14.67	26.77	1395	49
	16	12.00	181.83	1.172	30.87	21.13	±6.050	15.08	27.18	1522	54
	17	12.00	193.83	1.194	31.44	21.51	±6.050	15.46	27.56	1522	55
	18	12.00	205.83	1.215	31.98	21.89	±6.050	15.84	27.94	1522	55
	19	12.00	217.83	1.234	32.50	22.24	±6.050	16.19	28.29	1522	56
	20	12.00	229.83	1.253	33.00	22.59	±6.050	16.54	28.64	1522	56
	21	15.08	244.91	1.276	33.61	23.00	±6.050	16.95	29.05	1912	71
Leeward	All	All	244.91	1.276	33.61	-14.38	±6.050	-20.43	-8.33	31055	
										ΣForce	

Table A1: N-S Direction Wind Story Forces

Design Wind Pressures in E-W Direction										Forces	
Location	Level	Height (ft)	Floor Height (ft)	K_z	q_z	External Pressure $q_zG_fC_p$ (psf)	Internal Pressure $q_h(GC_{pi})$ (psf)	Net pressure (psf) $+(GC_{pi})$	Net pressure (psf) $-(GC_{pi})$	Trib Area (sf)	Net Force (w/o Internal Pressure) (k)
Windward	1	13.83	13.83	0.834	21.97	15.39	±7.254	8.13	22.64	1065	29
	2	12.00	25.83	0.952	25.06	17.55	±7.254	10.29	24.80	924	27
	3	12.00	37.83	1.031	27.16	19.02	±7.254	11.76	26.27	924	28
	4	11.00	48.83	1.088	28.66	20.07	±7.254	12.81	27.32	847	27
	5	11.00	59.83	1.136	29.91	20.94	±7.254	13.69	28.20	847	28
	6	11.00	70.83	1.177	30.99	21.70	±7.254	14.45	28.95	847	28
	7	11.00	81.83	1.213	31.95	22.37	±7.254	15.12	29.62	847	29
	8	11.00	92.83	1.246	32.81	22.97	±7.254	15.72	30.23	847	29
	9	11.00	103.83	1.276	33.59	23.52	±7.254	16.27	30.77	847	30
	10	11.00	114.83	1.303	34.31	24.02	±7.254	16.77	31.28	847	30
	11	11.00	125.83	1.328	34.98	24.49	±7.254	17.24	31.75	847	31
	12	11.00	136.83	1.352	35.60	24.93	±7.254	17.67	32.18	847	31
	13	11.00	147.83	1.374	36.18	25.34	±7.254	18.08	32.59	847	31
	14	11.00	158.83	1.395	36.73	25.72	±7.254	18.47	32.98	847	32
	15	11.00	169.83	1.415	37.25	26.09	±7.254	18.83	33.34	847	32
	16	12.00	181.83	1.435	37.79	26.46	±7.254	19.21	33.72	924	35
	17	12.00	193.83	1.455	38.31	26.82	±7.254	19.57	34.08	924	35
	18	12.00	205.83	1.473	38.79	27.16	±7.254	19.91	34.42	924	36
	19	12.00	217.83	1.491	39.26	27.49	±7.254	20.24	34.74	924	36
	20	12.00	229.83	1.508	39.70	27.80	±7.254	20.55	35.06	924	36
	21	15.08	244.91	1.528	40.24	28.18	±7.254	20.92	35.43	1161	46
Leeward	All	All	244.91	1.528	40.24	-11.59	±7.254	-18.84	-4.33	18858	
										ΣForce	

Table A2: E-W Direction Wind Story Forces

SEISMIC



Seismic Analysis

Equivalent Lateral Force Procedure Requirements

$S_s = 35\%g$ (Figure 22-1) $\longrightarrow$ Using USGS calculator: $S_s = 0.361g$
$S_1 = 6.2\%g$ (Figure 22-2) $\qquad S_1 = 0.070g$

$S_s > 0.15$ & $S_1 > 0.04$ (using USGS values)
Geotech. report states part of soil "should be considered to liquefy during the design earthquake event" → Site Class F
Due to the unavailability of a site response analysis, exception in 20.3.1 utilized and assumed as Site Class E

F_a:

	$S_s \leq 0.25$	0.361	0.5
E	2.5	2.1	1.7

$F_v = 3.5$

$S_{MS} = F_a S_s = (2.1)(0.361) = 0.758$

$S_{M1} = F_v S_1 = (3.5)(0.07) = 0.245$

$S_{DS} = \frac{2S_{MS}}{3} = \frac{2(0.758)}{3} = 0.505$

$S_{D1} = \frac{2S_{M1}}{3} = \frac{2(0.245)}{3} = 0.163$

Seismic Design Category
$S_1 = 0.070 < 0.75$
Simplified Alternative Structural Design Criteria for Simple Bearing Wall or Building Frame Systems not applicable

1. $T_a = C_t h_n^x = (0.02)(264)^{0.75} = 1.310$

$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.163}{0.505} = 0.323$

[illegible] 0.0 → $T = $ [illegible] $1.57 > T_s$
[illegible]

$T_a < 0.8T_s$

Conditions not satisfied

$1.310 < 0.8(0.323) = 0.258$ ✓

Table 11.6.1 → SDC = D ← Governs

Table 11.7.1 → SDC = C

T_a will be used for both strength & drift limits, in lieu of T

$T_a = 1.310 > 3.5T_s = 3.5(0.323) = 1.131$ → Equivalent Lateral Force Analysis not permitted

Considering that this is an initial lateral analysis, equivalent lateral force analysis will be used rather than the Modal Response Spectrum Analysis

Response Modification Coefficient

• Ordinary Reinforced Concrete Shear Walls [illegible]

$R = 5$

Long Period Transition

$T_L = 6s$ (Figure 22-15)

$T_a = 1.310 < T_L = 6$

$$V = C_s \cdot W$$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0.505}{\left(\frac{5}{1.0}\right)} = 0.10[illegible] \quad \text{but no bigger than} \quad \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.163}{(1.310)\left(\frac{5}{1.0}\right)} = 0.02508 \leftarrow \text{Governs}$$

$$\text{or} \quad \frac{S_{D1}T_L}{T^2\left(\frac{R}{I}\right)} = \frac{(0.163)(6)}{(1.310)^2\left(\frac{5.0}{1}\right)} = 0.11[illegible]$$

W = 41,852 kip

$V = C_sW = (0.02508)(41,852) = \underline{\underline{1050^k}}$

ITEM BY CHK'D

Vertical Distribution of Shear Forces

$T_a = 1.310 > 0.5$
< 2.5 } Linear Interpolation → $k = 1.405$

				Slab									Columns			Miscellaneous	Façade		Walls	Σ
Location	Floor Height (ft)	$h_{above}/2$ (ft)	$h_{below}/2$ (ft)	Total Area (sf)	Typical Thickness (in)	Thickened area (sf)	Thickness (in)	Core area (sf)	Thickness (in)	Misc. Area (sf)	Thickness (in)	Total Slab Weight (k)	Column area above	Column area below	Total Column Weight (k)	Additional Dead Load (k)	Curtainwall =600plf*perimeter	Masonry wall =100psf*h*perimeter	Shear Walls (ft)	Total Load per floor (k)
EMR Roof		0	8	679	30							255			0	0	0	0	112	366
Roof	16	8	7.54	5206	12	1219	21	586	12			918		66.13	75	0	46	162	217	1418
21	15.08	7.54	6	5206	9	1219	18.5	586	12			752	66.13	72.38	140	267	93	275	189	1715
20	12	6	6	5419	9	1219	18.5	586	12			776	72.38	72.05	130	277	93	244	167	1687
19	12	6	6	5938	9	1219	18.5	586	12			835	72.05	92.33	148	304	93	244	167	1790
18	12	6	6	5938	9	1219	18.5	586	12			835	92.33	92.33	166	304	93	244	167	1808
17	12	6	6	5938	9	1219	18.5	586	12			835	92.33	92.33	166	304	93	244	167	1808
16	12	6	5.5	5938	9	1219	18.5	586	12			835	92.33	92.33	159	304	93	233	160	1784
15	11	5.5	5.5	5938	9	1219	18.5	586	12			835	92.33	92.33	152	304	93	223	153	1760
14	11	5.5	5.5	5938	9	1219	18.5	586	12			835	92.33	92.33	152	304	93	223	153	1760
13	11	5.5	5.5	5938	9	1219	18.5	586	12			835	92.33	92.33	152	304	93	223	153	1760
12	11	5.5	5.5	5938	9	1219	18.5	586	12			835	92.33	92.33	152	304	93	223	153	1760
11	11	5.5	5.5	5938	9	1219	18.5	586	12			835	92.33	92.33	152	304	93	223	153	1760
10	11	5.5	5.5	5938	9	1219	18.5	586	12			835	92.33	92.33	152	304	93	223	153	1760
9	11	5.5	5.5	5938	9	1219	18.5	586	12			835	92.33	92.33	152	304	93	223	153	1760
8	11	5.5	5.5	5938	9	1219	18.5	586	12			835	92.33	92.33	152	304	93	223	153	1760
7	11	5.5	5.5	5938	9	1219	18.5	586	12			835	92.33	92.33	152	304	93	223	153	1760
6	11	5.5	5.5	6541	9	1219	18.5	586	12			903	92.33	115.41	171	335	136	223	153	1922
5	11	5.5	5.5	6806	9	2087	18.5	586	12			1035	115.41	121.41	195	348	128	223	153	2084
4	11	5.5	6	6926	9	2207	18.5	586	12			1063	121.41	148.87	234	355	136	233	160	2182
3	12	6	6	7149	9	2207	18.5	586	12			1088	148.87	285.15	391	366	117	244	181	2387
2	12	6	6.915	4089	9	1538	18.5	586	12	379	12	679	285.15	176.02	439	209	136	262	196	1922
1	13.83	6.915	0	12482	12	0	18.5	586	12	2037	20	2076	176.02	0	183	639	0	140	96	3134

Table A3

Total Building Weight	41852

Superimposed Dead Load		
Item	pcf	psf
MEP	-	10
Partitions	-	18
LWC leveling slab (2")	115	19.2
Epoxy Terrazzo (3/8")	-	4
Total		51.2

Table A4

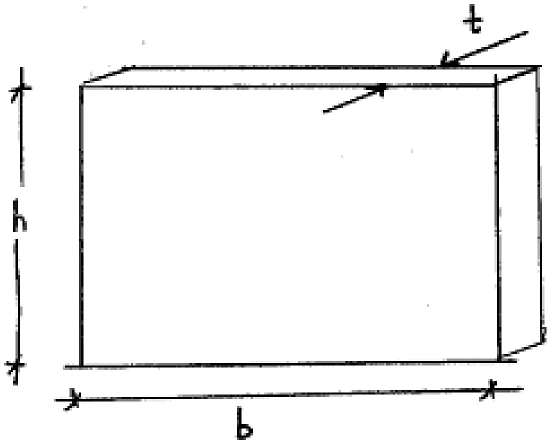
APPENDIX B

MISCELLANEOUS

TECH III

Example Shear Wall Rigidity Calculation

SW1 - supporting 7th - N-S direction



h= height from base to 7th level elevation

t = thickness

b= wall length

h= 982" b= 104"
t= 12"

$$k = \frac{1}{\frac{h^3}{3EI} + \frac{1.2h}{AG}}$$ (cantilever wall stiffness)

$G = 0.4E = 0.4(5098) = 2039$

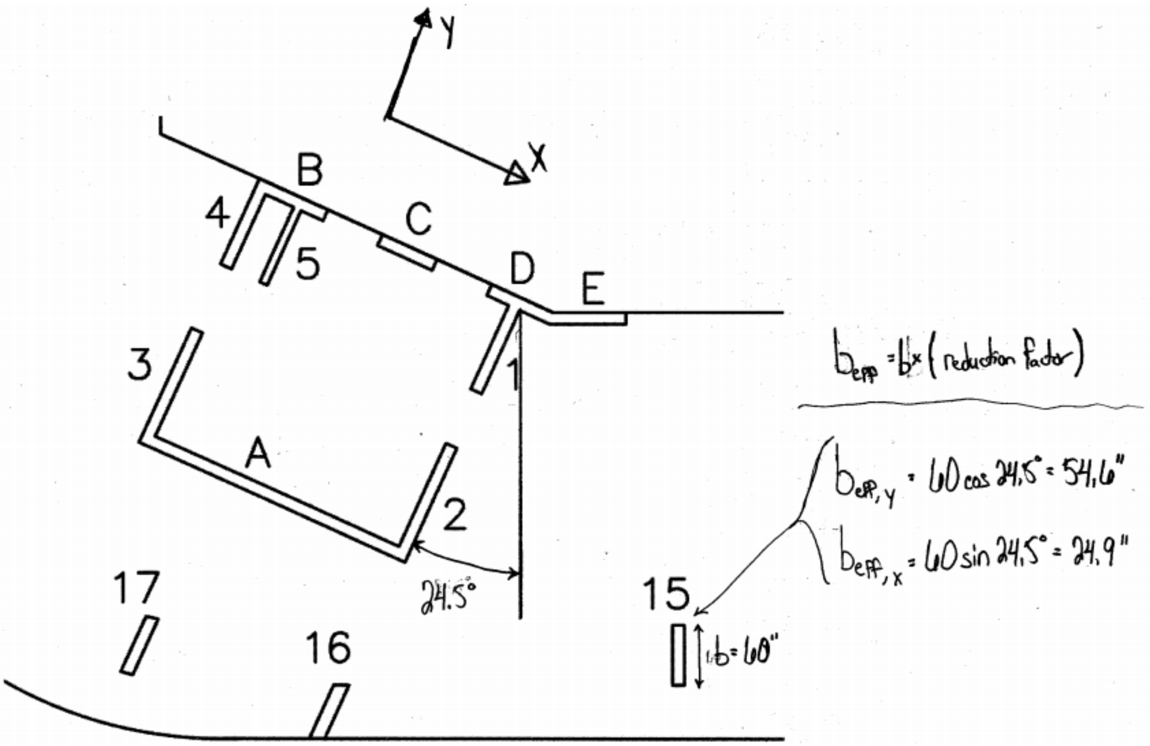
$E = 57000\sqrt{8{,}000}\,/1000$
$= 5098 \text{ ksi}$

$I = \frac{1}{12}tb^3 = \frac{1}{12}(12)(104)^3 = 1{,}124{,}864 \text{ in}^4$

$A = bt = 104 \cdot 12 = 1248 \text{ in}^2$

$$k = \frac{1}{\frac{(982)^3}{3(5098)(1{,}124{,}864)} + \frac{1.2(982)}{(1248)(2039)}}$$

$k = 18.02 \ ^{k}/_{in}$

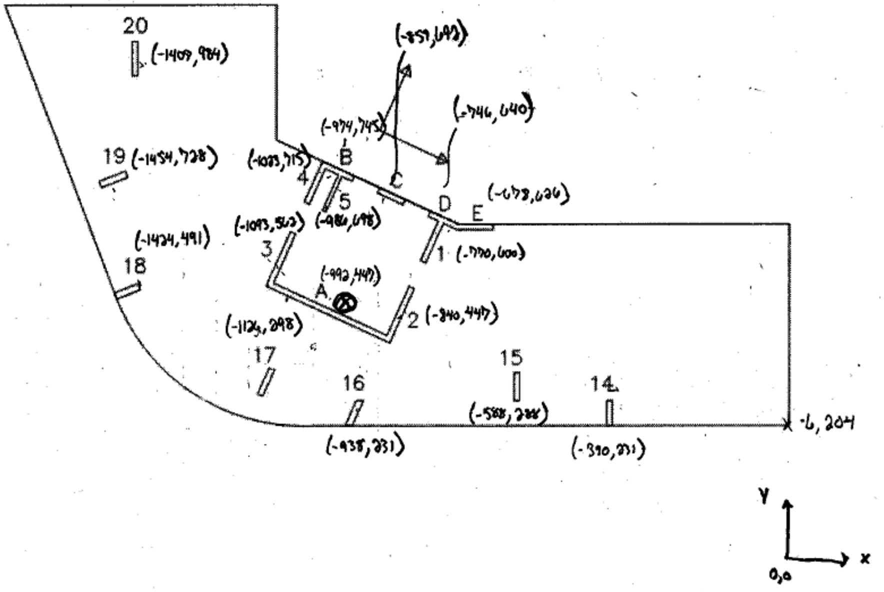


$b_{eff} = b \times (\text{reduction factor})$

$b_{eff,y} = 60 \cos 24.5° = 54.6"$

$b_{eff,x} = 60 \sin 24.5° = 24.9"$

Figure B1: Example of Effective Length Calculation



CoR → (-958, 404)

Model Output → (-938, 484)

Figure B2: Member Coordinates Used in Level 8 Center of Rigidity Calculation

Center of Rigidity

8th Story

X-Coordinate (in)

Member	f'_c (psi)	E (ksi)	h (in)	t (in)	α (deg)	b (in)	b_{eff} (in)	k_{iy} (k/in)	x_i	$k_{iy}x_i$
SW1	8000	5098	982	12	24.5	104	94.6	13.60	-770	-10471
SW2	8000	5098	982	12	24.5	126	114.7	24.11	-840	-20250
SW3	8000	5098	982	12	24.5	126	114.7	24.11	-1093	-26349
SW4	8000	5098	982	12	24.5	94	85.5	10.05	-1023	-10284
SW5	8000	5098	982	8	24.5	82	74.6	4.45	-986	-4392
C14	8000	5098	982	12	0	54	54.0	2.54	-390	-990
C15	8000	5098	982	12	0	60	60.0	3.48	-588	-2046
C16	8000	5098	982	14	24.5	60	54.6	3.06	-938	-2870
C17	8000	5098	982	12	24.5	60	54.6	2.62	-1126	-2953
C18	8000	5098	982	14	69	54	19.4	0.14	-1424	-194
C19	8000	5098	982	14	69	60	21.5	0.19	-1454	-272
C20	8000	5098	982	12	0	72	72.0	6.01	-1408	-8455
SWA	8000	5098	982	12	24.5	291	120.7	28.08	-992	-27854
SWB	8000	5098	982	12	24.5	75	31.1	0.49	-974	-473
SWC	8000	5098	982	12	24.5	60	24.9	0.25	-859	-214
SWD	8000	5098	982	12	24.5	67	27.8	0.35	-746	-258
SWE	8000	5098	982	12	90	61	0.0	0.00	-678	0
							Σk_y	124	$\Sigma k_{iy}x_i$	-118326

$(\Sigma k_{iy}x_i)/(\Sigma k_y)$ **-958**

Table B1

Center of Rigidity

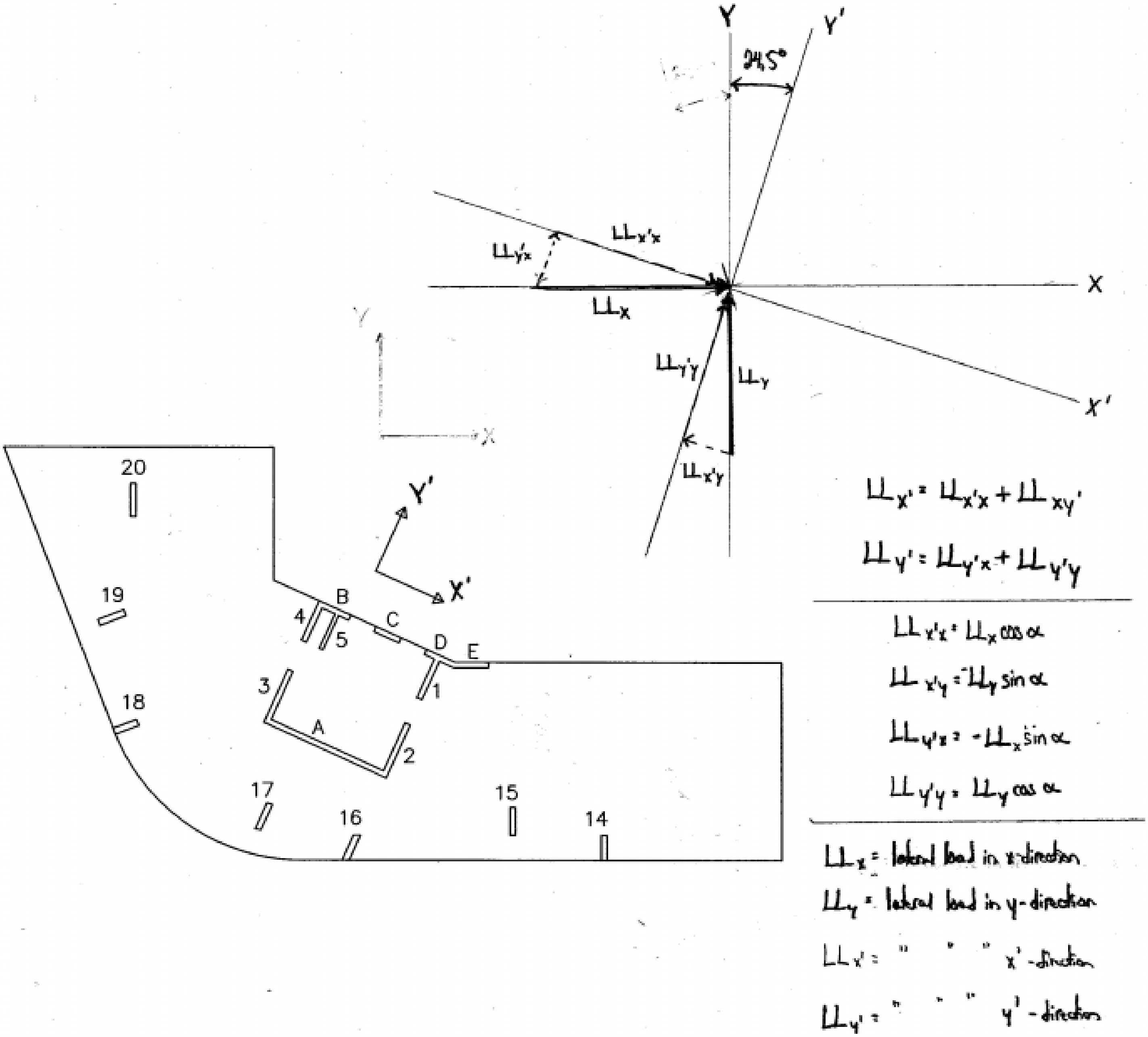
8th Story

Y-Coordinate (in)

Member	f'_c (psi)	E (ksi)	h (in)	t (in)	α (deg)	b (in)	b_{eff} (in)	k_{ix} (k/in)	y_i	$k_{ix}y_i$
SW1	8000	5098	982	12	24.5	104	43.1	1.29	600	776
SW2	8000	5098	982	12	24.5	126	52.3	2.30	447	1028
SW3	8000	5098	982	12	24.5	126	52.3	2.30	562	1292
SW4	8000	5098	982	12	24.5	94	39.0	0.96	715	683
SW5	8000	5098	982	8	24.5	82	34.0	0.42	698	295
C14	8000	5098	982	12	0	54	0.0	0.00	231	0
C15	8000	5098	982	12	0	60	0.0	0.00	288	0
C16	8000	5098	982	14	24.5	60	24.9	0.29	231	67
C17	8000	5098	982	12	24.5	60	24.9	0.25	298	74
C18	8000	5098	982	14	69	54	50.4	2.41	491	1183
C19	8000	5098	982	14	69	60	56.0	3.30	728	2405
C20	8000	5098	982	12	0	72	0.0	0.00	984	0
SWA	8000	5098	982	12	24.5	291	264.8	284.97	447	127380
SWB	8000	5098	982	12	24.5	75	68.2	5.12	745	3812
SWC	8000	5098	982	12	24.5	60	54.6	2.62	692	1815
SWD	8000	5098	982	12	24.5	67	61.0	3.65	640	2336
SWE	8000	5098	982	12	90	61	61.0	3.66	626	2289
							Σk_x	314	$\Sigma k_{ix}y_i$	145436

$(\Sigma k_{ix}y_i)/(\Sigma k_x)$ **464**

Table B2



$$LL_{x'} = LL_{x'x} + LL_{xy'}$$

$$LL_{y'} = LL_{y'x} + LL_{y'y}$$

$$LL_{x'x} = LL_x \cos\alpha$$

$$LL_{x'y} = LL_y \sin\alpha$$

$$LL_{y'x} = -LL_x \sin\alpha$$

$$LL_{y'y} = LL_y \cos\alpha$$

LL_x = lateral load in x-direction

LL_y = lateral load in y-direction

$LL_{x'}$ = " " " x'-direction

$LL_{y'}$ = " " " y'-direction

Figure B3: Converting Forces from Global Axis to Local Axis