100 Eleventh Avenue

New York, New York

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Technical Report I

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Executive Summary

The overarching goal of Technical Report No.1 is to attain a preliminary understanding of 100 Eleventh Avenue's existing structural system. The foundation system was determined to be comprised of piles and caissons as well as a secant wall system to resist lateral soil loads. The lateral system was identified as 12" thick concrete shear walls at the building's elevator core in combination with seven columns designed to resist lateral forces.

Along with determining live loads and dead loads, a snow load of 20 psf was calculated. A wind analysis was carried out using ASCE 7-05's Method 2, resulting in a base shear of 1,015 k controlling in the east-west direction. This direction will control due to winds coming off of the Hudson River. Seismic loads were calculated using ASCE 7-05's Equivalent Lateral Force Method, and a base shear of 868 k was determined. The seismic base shear as calculated using the original design's values from the 1968 New York City Building Code proved to be 2.5 times as large. This large difference is likely due to the assumptions made in order to use ASCE 7-05 due to the site's extremely poor soil.

Two spot checks were made on the structure's gravity framing system. The first employed ACI 318-08's Direct Design Method to analyze the two-way flat-plate floor system. Despite significant simplifications being made due to the irregular column layout, the results proved to be very similar to the design. From ACI Table 9.5(c), the conclusion was made that deflections control the slab design, mandating the 9" slab thickness. A column on the 7th floor was also analyzed for the interaction of axial loads and moment. The column was found to be overdesigned for the loads acting on it. This is likely due to the desire, for constructability purposes, to keep the column size and reinforcement the same from the 4th floor, where the loads are largest, through the 21st floor, where axial loads are at a minimum.

Introduction

100 Eleventh Avenue is a 22-story, 170,000 sf ultra-luxury condominium building located in Manhattan's Chelsea District, a neighborhood next to the Hudson River that is quickly gaining in popularity within the city. 100 Eleventh Avenue will join several other recently completed projects that have helped in revitalizing the area, such as IAC's headquarters designed by architect Frank Gehry, and the High Line, an 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 façade, 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 bottom six floors are enclosed by a second façade 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, boasting an extensive private roof terrace.

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Figure 1: Space within double façade

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Figure 2: View from Westside Highway

Structural System Overview

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".

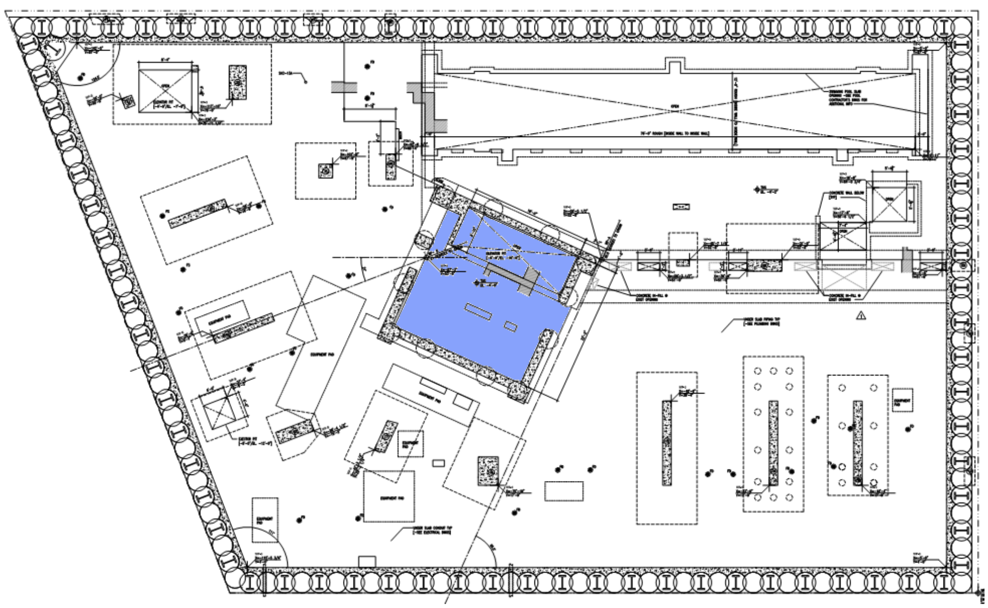


Figure 3: Cellar plan with core denoted

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.

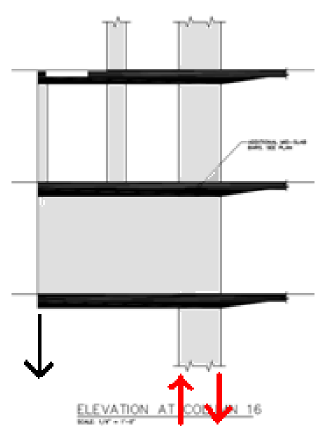


Figure 4: Cantilevered column creating tension in piles

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 relative ease in which columns can transfer, the minimal floor system thickness, and the sound isolation properties of concrete.

Figure 5: Superstructure

The typical floor is comprised of 9" thick, 5,950 psi concrete reinforced with a basic bottom reinforcing mat of #4 @ 12" E.W. Mid-strip bars are also #4 @ 12" unless otherwise noted. Column strip bars are primarily #6 @12". Additional top and bottom bars are added 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. Due to aesthetics, the slab gradually increases in thickness over a distance of 5'-0", as seen in Figure 6, rather than an abrupt increase.

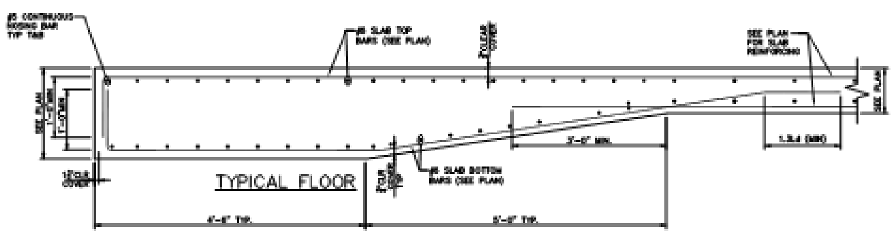


Figure 6: Detail of thickened slab at curved edge

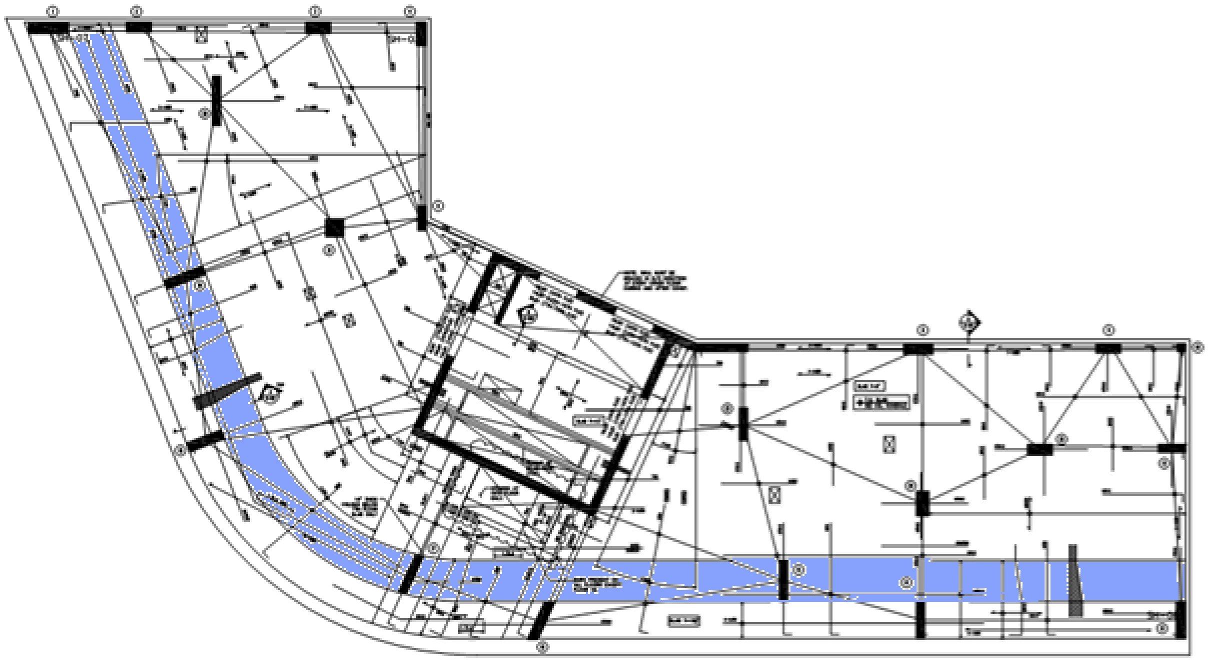


Figure 7: Typical plan with slab thickness transition area highlighted

As seen from the typical structural plan, Figure 7, 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 axis.

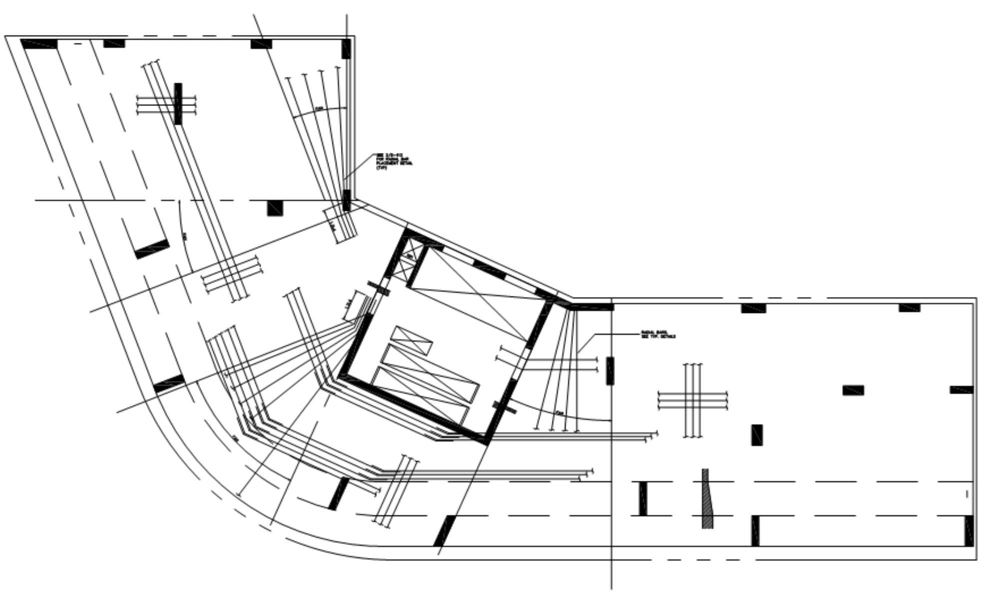


Figure 8: Slab reinforcing schematic layout

The ground floor is comprised of a variety of slab thicknesses and elevations. The majority is 12" thick with a basic bottom reinforcing mat of #5 @12" E.W. and #5 strip bars, but varies from 17" thick to 20" thick and up to #6 @12". Also throughout the ground floor, bars are placed at mid-height of slab to transfer the ground floor's lateral forces around openings in the slab, as seen in Figure 9.

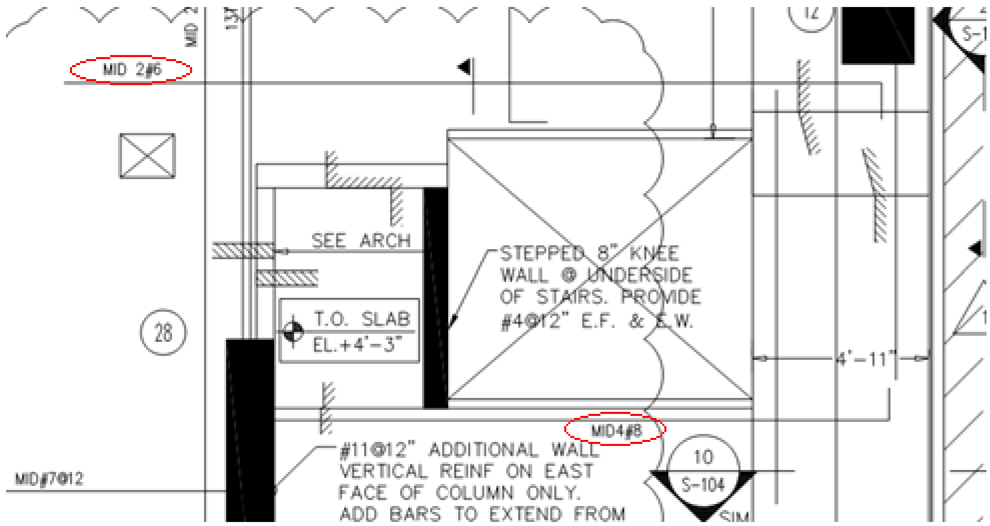


Figure 9: Mid-height bars adjacent to opening

On the third floor, several columns transfer as they make way for a large, two-story, column-free space on the 1st floor. Six large transfer beams carry the forces, the largest of which is 84" wide x 60" deep and reinforced with 38 #ll bars and 38 #9 bars on the bottom and top, respectively. On the 19th floor, three columns transfer as the building sets back 13 feet on the east side. The gravity forces are transferred via the slab, which is 18.5" thick with #10 @6" E.W. on both top and bottom of slab.

Figure 10: 3rd Floor transfer beams

On the lower six floors, balconies begin to cantilever out towards the second street façade. An example of this is shown in Figure 11, where the balcony extends 9'-10" from the building. Notice that,

due to architectural restraints, the balcony has only one corner supported by a column below. To resolve excessive deflection caused by the façade and tree loads, three post-tensioned high-strength Dywidag bars were used, highlighted in green.

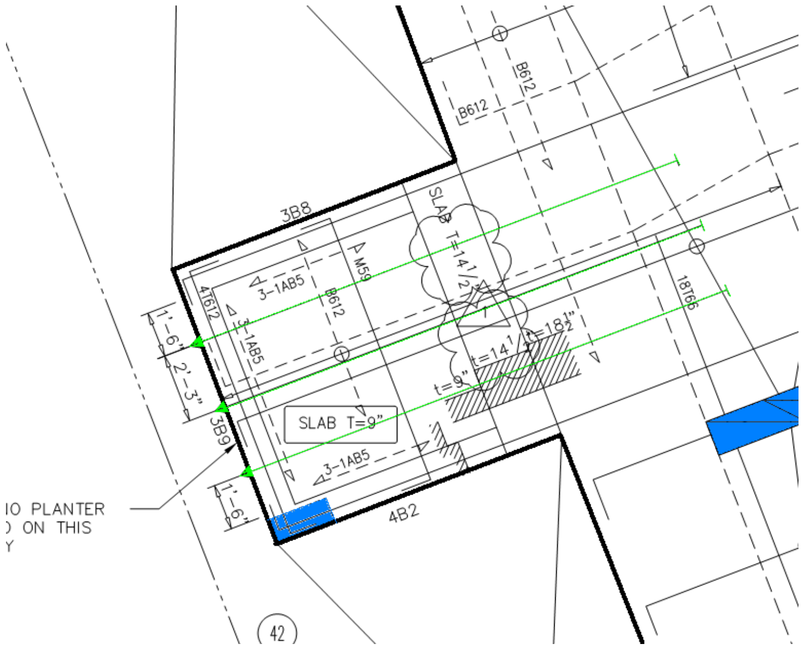


Figure 11: Cantilevered balcony utilizing post-tensioning

Columns

Column strength for columns supporting the cellar level through the 9th level are 8 ksi; those supporting the 10th through the roof are 7 ksi. 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

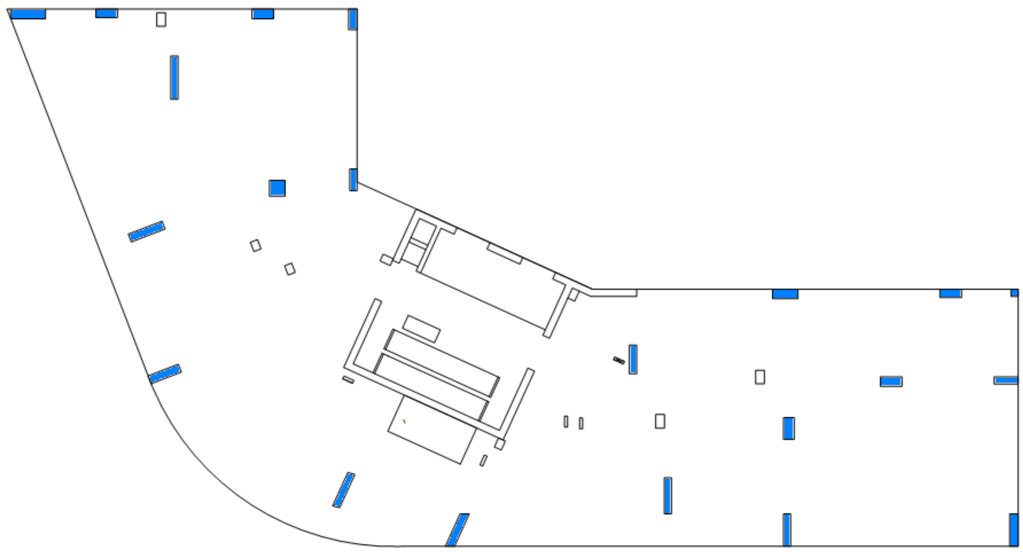


Figure 12: Typical floor column layout

sizes range widely throughout a single floor, as well as from floor to floor. The vast 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.

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 "poke" out from the inner façade to meet the outer street façade, 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 14 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

Figure 13: Photo showing portion of cantilevered balcony system

of the slab.

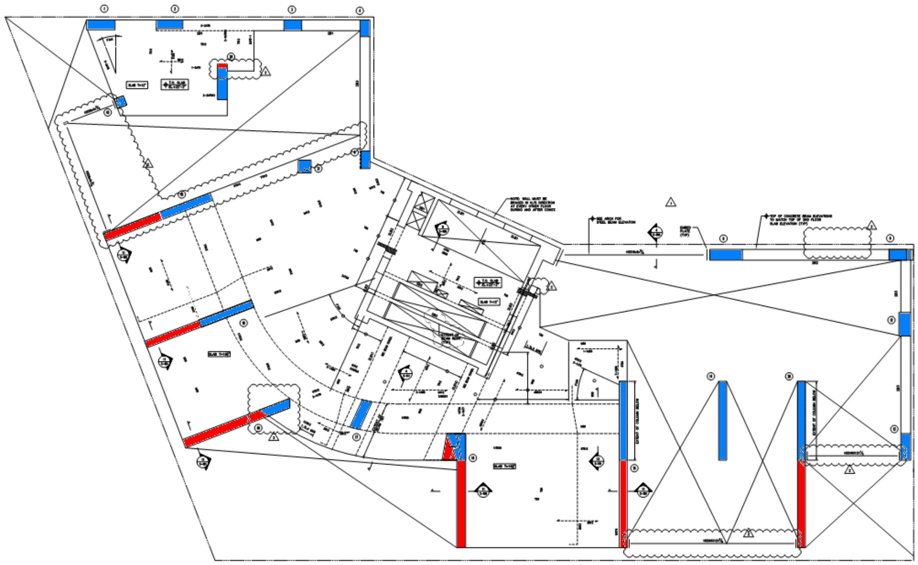


Figure 14: 2nd Floor column layout

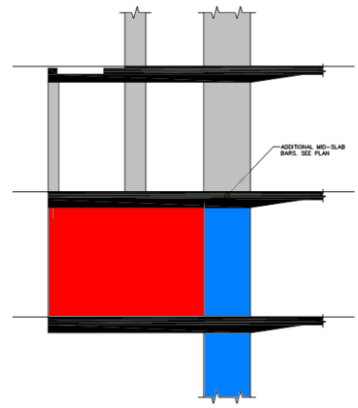


Figure 15: Cantilevered Column Elevation

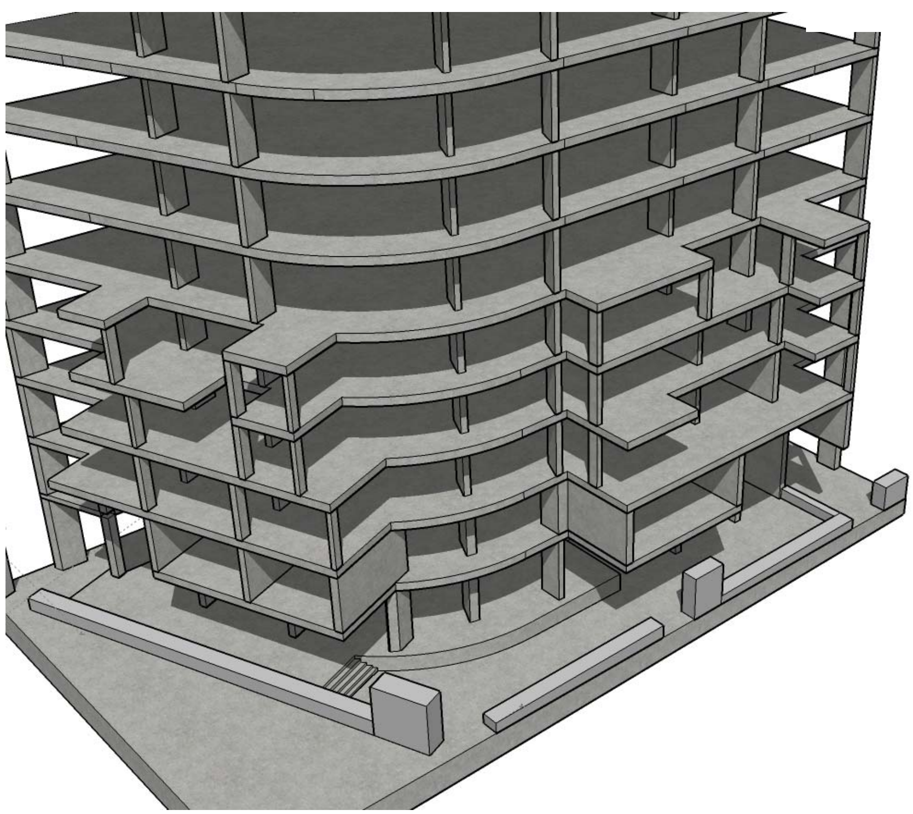


Figure 16: Model showing complicated balcony system

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 17 below. Because architectural restraints constricted the use of shear walls to the relatively small elevator core, the seismically poor soil 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.

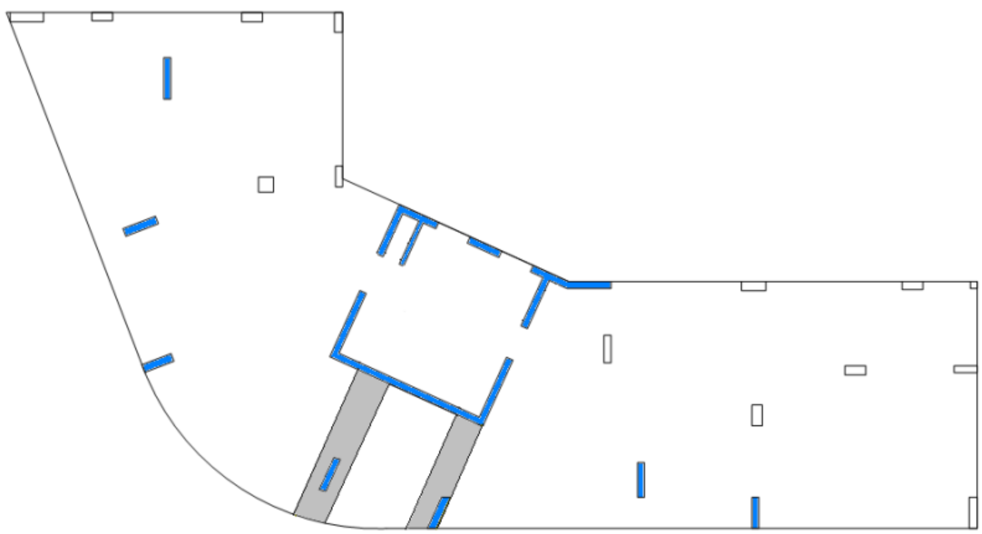


Figure 17: Lateral system with link beams denoted

Code & Design Standards

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

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 façade 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 façade reactions were received from the façade consultant, the initial design was checked and found to be sufficient. The 500 plf façade load will be used for the initial analysis.

Snow

New York City lies in a 25 psf ground snow load region. The flat roof snow load falls below the minimum of $p_f = (I)*p_g = 20$ psf; therefore, 20 psf will be used as the design snow load.

Wind Analysis	
Variable	**Value**
p_g	25 psf
C_e	1.0*
C_t	1.0
I	1.0
p_f	17.5 psf
$p_{min,f}$	20.0

*Assuming partially exposed roof and Exposure C as calculated in wind analysis

Table 3

Lateral Loads

Wind

The wind pressures for the original design of 100 Eleventh Avenue was governed by New York City's building code, which applies 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 attained 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 façade onto a vertical plane, as shown below. The fundamental period

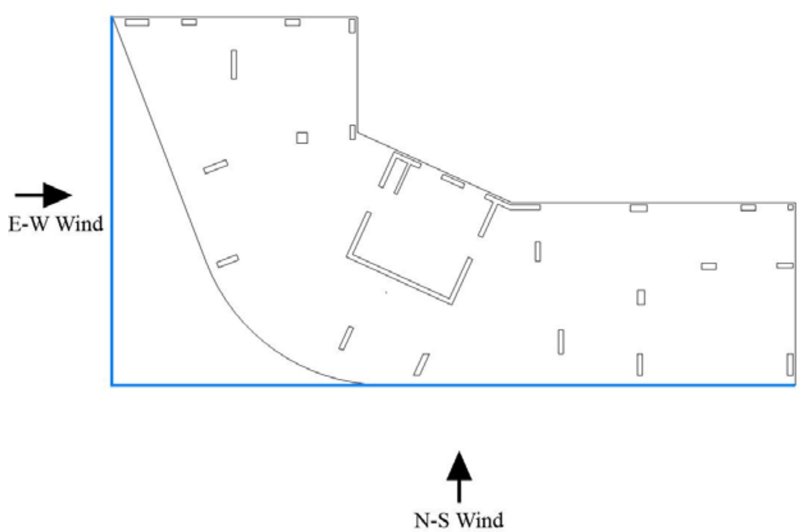


Figure 18: Wind direction axes

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 that due to the building's proximity to the westward Hudson River, the exposure category is more severe in the E-W direction, resulting in higher pressures.

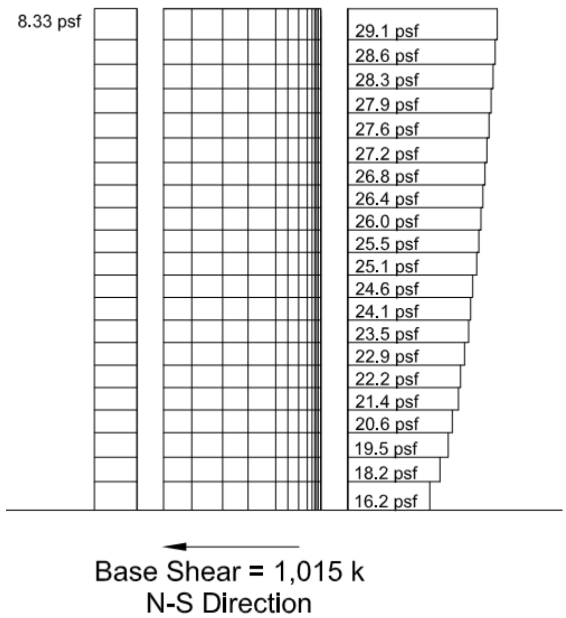


Figure 19: N-S Wind Pressure

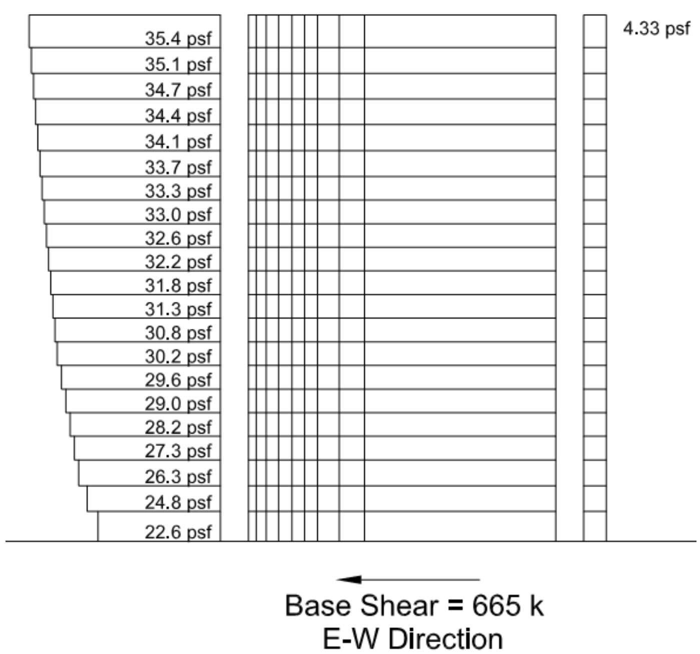


Figure 20: E-W Wind Pressure

Surprisingly, the more detailed method outlined in ASCE 7-05 produces higher wind pressures. This may be a case in which the New York City building code would not be sufficient in defining the wind load on the building. Wind acts differently on each individual building and an umbrella loading such as that defined in the city's code, though usually conservative, cannot *always* define every building's wind load. It is worth noting, however, that the New York City building code does not include leeward wind pressures. When the leeward pressures are subtracted from the ASCE 7-05 calculated values, it is easier to see similarities between the two. For instance, at an elevation of roughly 100 feet (referring to Appendix Tables A1 and A2 for corresponding floor heights), the ASCE and NYCBC values are 23.52 psf and 25 psf, respectively. As the building's height approaches 300 feet, the ASCE-calculated values appear to be approaching, in a parabolic fashion, the 30 psf specified in the city code.

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 4 below is the vertical distribution of seismic forces. The effective seismic weight used in the calculation included structural material, façade, 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.

The original design's seismic forces were calculated under the New York City Building Code. This method is summarized below for base shear with comparisons made.

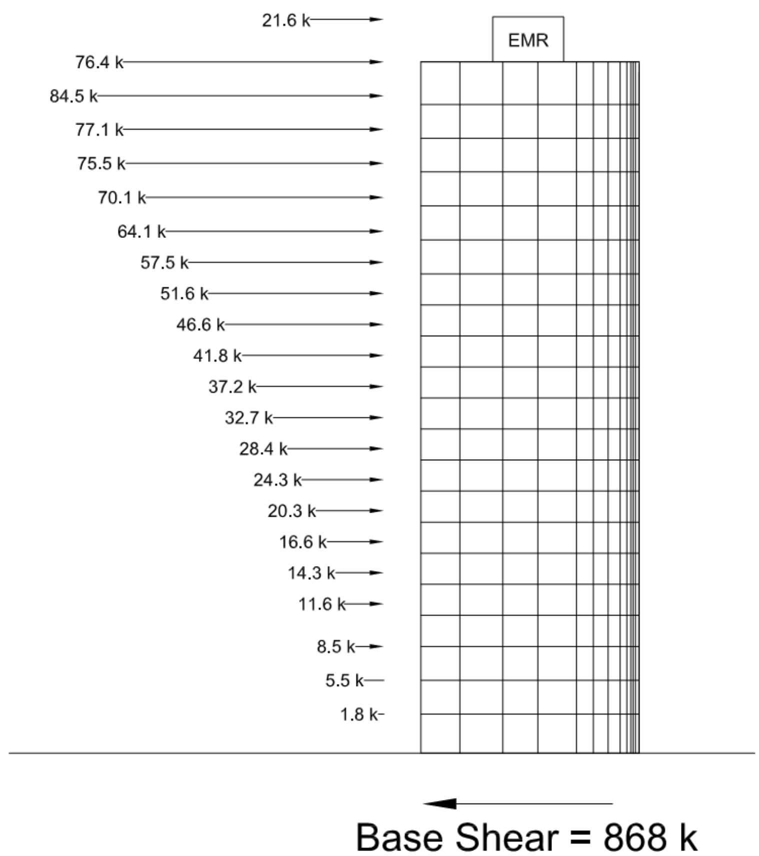


Figure 21: Seismic Loads

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	21.6
Roof	1418	244.9	2273	3223377	0.0880	76.4
21	1715	229.8	2079	3565122	0.0973	84.5
20	1687	217.8	1928	3252744	0.0888	77.1
19	1790	205.8	1780	3187036	0.0870	75.5
18	1808	193.8	1636	2958961	0.0808	70.1
17	1808	181.8	1496	2704848	0.0738	64.1
16	1784	169.8	1359	2424760	0.0662	57.5
15	1760	158.8	1237	2177287	0.0594	51.6
14	1760	147.8	1118	1968439	0.0537	46.6
13	1760	136.8	1003	1765795	0.0482	41.8
12	1760	125.8	892	1569648	0.0429	37.2
11	1760	114.8	784	1380331	0.0377	32.7
10	1760	103.8	681	1198227	0.0327	28.4
9	1760	92.8	582	1023782	0.0280	24.3
8	1760	81.8	487	857527	0.0234	20.3
7	1760	70.8	398	700101	0.0191	16.6
6	1922	59.8	314	602894	0.0165	14.3
5	2084	48.8	236	491376	0.0134	11.6
4	2182	37.8	165	359491	0.0098	8.5
3	2387	25.8	96	230076	0.0063	5.5
2	1922	13.8	40	77014	0.0021	1.8
1	3134	0.0	0	0	0.0000	0.0

$\Sigma w_i h_i^k$	36628576
V_{base}	868.0 k

Table 4

Original Seismic Design Criteria	
Seismic Zone Factor, Z	0.15
Importance Factor, I	1
R_w (shear walls)	8
Coefficient, C	2.75
Building Weight, W	41,852 k*
Base Shear, $V=(ZIC/R_w)W$	2158 k

*Building weight calculated by hand, as actual building weight used in design is unknown

Table 5

The NYCBC seismic base shear is approximately 2.5 times as large, a significant difference. This is almost surely due to the assumptions made in order to use ASCE 7-05's equivalent lateral force method. 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. The soil is actually much worse than the values used in ASCE 7-05, which would explain the higher base shear values used in design.

Additional Loads

There are a number of other loads that will need to be taken into account in future analysis. These include lateral pressure from the soil acting on the ground slab and pressure due to the high water table acting upwards on the pressure slabs.

Gravity System Spot Checks

Floor System

The first spot check performed on this structure was of the two-way flat plate floor system. The floor system does not follow any regular layout or grid. Therefore, in order to utilize the Direct Design Method, a number of simplifications were needed in order to meet the method's requirements. A northeast portion of the slab on the typical floor plan was chosen. Two additional requirements that needed to be overlooked in order to use this method was the need for at least two spans in both directions and for these spans to be fairly uniform.

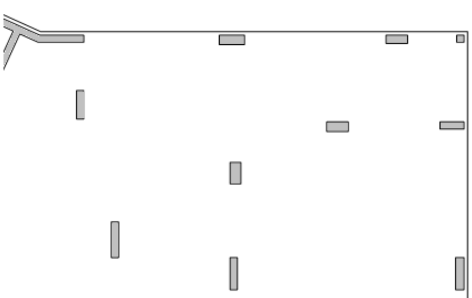


Figure 22: Actual Layout

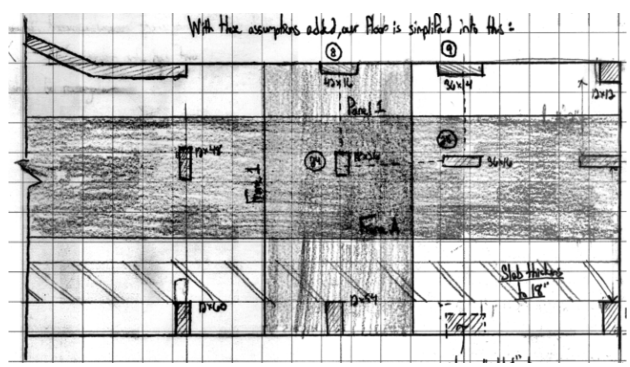


Figure 23: Simplified Layout

Shown in Tables 6 and 7 are the comparisons to the original design. The 9" slab has a basic bottom reinforcing mat of #4@12" E.W. The positive moment (bottom reinforcing) values from the Direct Design Method were, for the most part, controlled by minimum steel requirements. These correspond closely with the original design. The negative moment reinforcement calculated with the Direct Design Method also tended to mirror the values used in the original design, with the exception of the column strip being more heavily reinforced in the actual design.

Frame A (E-W)						
	Column Strip			Mid Strip		
	Negative	Positive	Negative	Negative	Positive	Negative
Calculated Reinf	4 #5	#4's@12"*	4 #5	4 #5*	#4@12"*	4 #5*
Design Reinf	6#6	#4@12"	5#6	8 #4	#4@12"	5 #4

*Reinforcement governed by $A_{s,min}$

Frame 1 (N-S)						
	Column Strip			Mid Strip		
	Ext. Negative	Positive	Int. Negative	Ext. Negative	Positive	Int. Negative
Calculated Reinf	4 #5*	#4@9"*	5 #5	#4@13"*	#4@13"*	#4@13"*
Design Reinf	6 #6	#4@12"	6 #6	#4@12"	#4@12"	#4@12"

*Reinforcement governed by $A_{s,min}$

Tables 6 & 7

The slab thickness was also checked against the Minimum Thickness of Slabs ithout Interior Beams table (ACI Table 9.5c). With clear spans up to 24 feet, the minimum slab thickness to control deflection is 8'-9", which corresponds nicely with the design thickness of 9". This, in combination with the fact that much of the bottom reinforcing was likely governed by minimum steel requirements, makes it likely that the floor design was controlled by deflection requirements. The differences in the column strip negative moments are likely due to the simplifications made in order to use the Direct Design Method.

Additionally, a column was selected to check the two-way punching shear of the slab. The slab's shear resistance was sufficient.

Columns

Column 24 supporting the 7th level was chosen to be checked for strength capacity. Axial load in the column from the floors supported were added using tributary areas. Live loads were not reduced, as it is believed the structural engineer left live loads unreduced for the design. Moment distributed from the slab was found using ACI (Eq. 13.7), and the interaction diagram was drawn by solving for critical points along the curve. Slenderness effects were ignored. Only

the weak axis is analyzed, as this is where the maximum moment acts, making it the critical section.

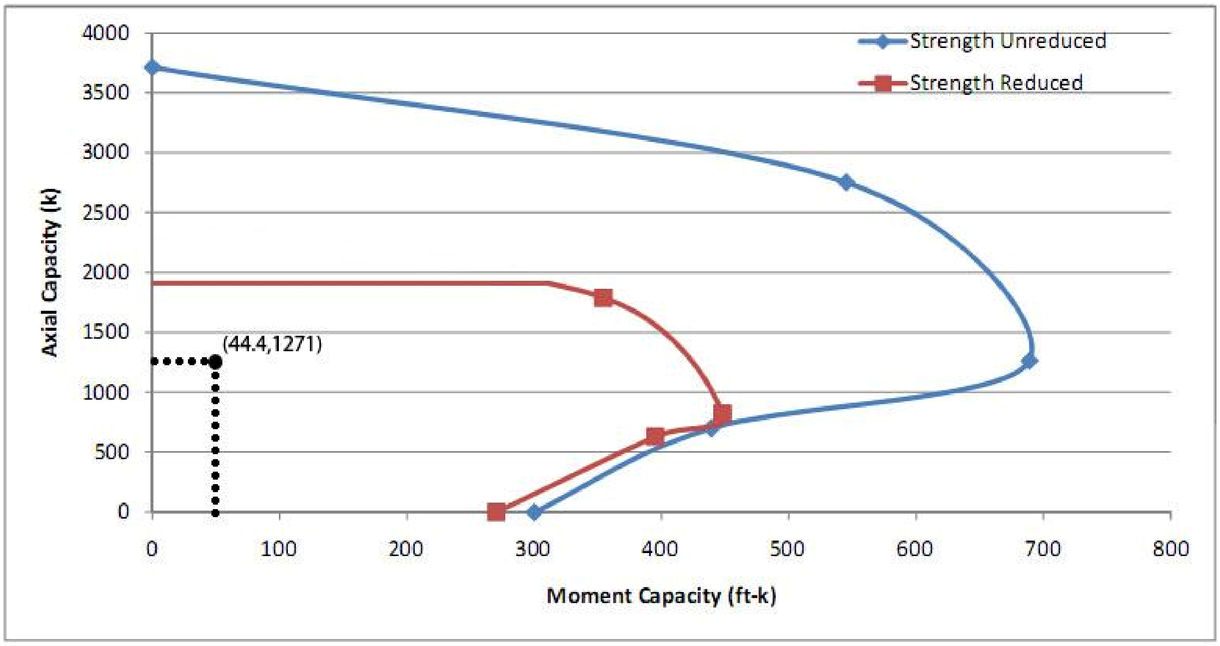


Figure 24: Column 24 Interaction Diagram

As can be seen from the interaction diagram, Column 24's capacity is adequate. It is at approximately 65% of its axial capacity and 10% of its moment capacity. It would appear that the column is, in fact, oversized. One probable reason for this is the desire to keep the column dimensions and reinforcing the same from floor to floor, for constructability purposes. Column 24 remains unchanged from the 21st floor, where it has very little loading, through the 4th floor, where it must resist loads from all the levels above. At the 4th floor, with the accumulation of axial load from the 5th and 6th floors, it is likely the column will reach its capacity.

P_n (k)	M_n (ft-k)	c	ε_t	ϕ	ϕP_n (k)	ϕM_n (ft-k)
3711	0	Infinity	0.003	0.65	2412	0
2753	545	18	0.000395	0.65	1789	354
1263	689	9.25	-0.00207	0.65	821	448
700	439	5.86	-0.005	0.9	630	395
0	300	2.22	-0.01389	0.9	0	270

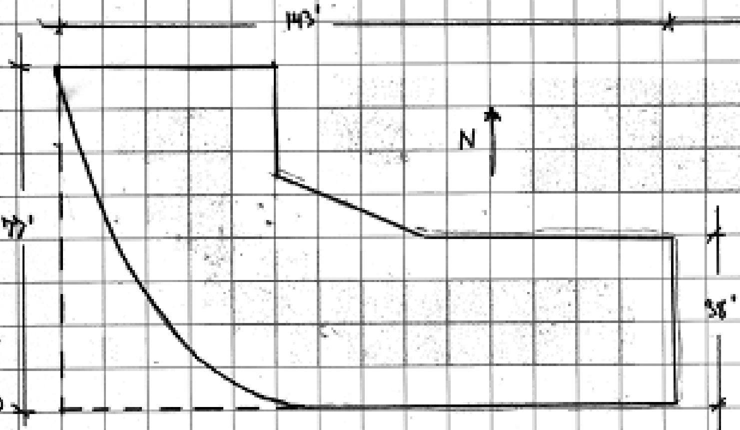
Table 8

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
Exposure	B	C
	$\alpha = 7.0$	$\alpha = 9.5$
	$z_g = 1200$	$z_g = 900$

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

K_h: $K_h = 1.28$ $\quad K_h = 1.53$

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

$B = 143'$ $\quad h = 245'$

$L = 77'$ $\quad \beta = 2\%$ (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]

$g_Q = g_v = 3.4$

$\bar{z} = 0.6h = 0.6(250) = 150' > z_{min} = 30$ (N-S)
$z_{min} = 15$ (N-S)

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

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

N-S

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

$c = 0.30$

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

$l = 320$

$\bar{\epsilon} = 1/3.0$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{143+280}{530.1}\right)^{0.63}}} = 0.8106$$

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

$\bar{b} = 0.45$

$\bar{\alpha} = 1/4.0$

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

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

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

$$R_h: \eta = \frac{4.6n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.6)(280)}{106} = 6.51 \rightarrow R_h = 0.1418$$

$$R_B: \eta = \frac{4.6n_1 B}{\bar{V}_{\bar{z}}} = \frac{4.6(0.6)(143)}{106} = 3.723 \rightarrow R_B = 0.2325$$

$$R_L: \eta = \frac{15.4n_1 L}{\bar{V}_{\bar{z}}} = \frac{15.4(0.6)(77)}{106} = 6.71 \rightarrow R_L = 0.1379$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B \left(0.53+0.47R_L\right)} = 0.26175$$

$$G_f = 0.925\left(\frac{1+1.7(0.233)\sqrt{3.4^2 \cdot 0.8106^2 + 4.07^2 \cdot 0.26175^2}}{1+1.7(3.4)(0.233)}\right) = 0.8555$$

E-W

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

$c = 0.20$

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

$l = 500$

$\bar{\epsilon} = 1/5$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{77+280}{676.8}\right)^{0.63}}} = 0.8456$$

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

$\bar{b} = 0.65$

$\bar{\alpha} = 1/6.5$

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

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

$$\eta_{R_h} = \frac{4.6(0.6)(280)}{132.4} = 5.211 \rightarrow R_h = 0.1735$$

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

$$\eta_{R_L} = \frac{15.4(0.6)(143)}{132.4} = 9.980 \rightarrow R_L = 0.09518$$

$R = 0.2222$

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

$= 0.8753$

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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

MWFRS

N-S

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

$C_p = -0.5$ leeward

$GC_{pi} = {+0.18 \atop -0.18}$

Windward

$p_z = q_z G_f C_p \pm q_h (GC_{pi})$

$= q_z(0.8555)(0.8) - (33.7)(\pm 0.18)$

$= 0.6844 q_z \mp 6.066$

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

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

$= 0.70 q_z \pm 7.254$

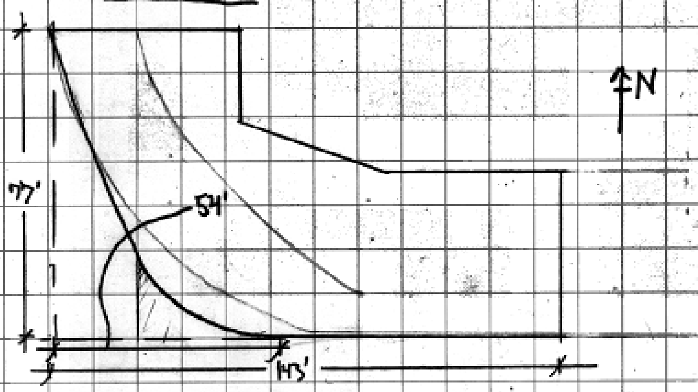
Leeward

$p_z = q_h G_f C_p - q_h (GC_{pi})$

$= (40.3)(0.8753)(-0.329) - (40.3)(\pm 0.18)$

$= -18.86$ 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)	Force $+(GC_{pi})$ (lb)	Force $-(GC_{pi})$ (lb)
Windward	1	13.83	13.83	0.562	14.79	10.12	±6.050	4.07	16.17	1754	7136	28355
	2	12.00	25.83	0.671	17.67	12.10	±6.050	6.05	18.15	1522	9200	27612
	3	12.00	37.83	0.749	19.71	13.49	±6.050	7.44	19.54	1522	11320	29732
	4	11.00	48.83	0.805	21.20	14.51	±6.050	8.46	20.56	1395	11800	28677
	5	11.00	59.83	0.853	22.47	15.38	±6.050	9.33	21.43	1395	13010	29887
	6	11.00	70.83	0.895	23.58	16.14	±6.050	10.09	22.19	1395	14069	30947
	7	11.00	81.83	0.933	24.57	16.82	±6.050	10.77	22.87	1395	15017	31894
	8	11.00	92.83	0.967	25.47	17.43	±6.050	11.38	23.48	1395	15878	32755
	9	11.00	103.83	0.999	26.30	18.00	±6.050	11.95	24.05	1395	16668	33546
	10	11.00	114.83	1.028	27.07	18.53	±6.050	12.48	24.58	1395	17401	34278
	11	11.00	125.83	1.055	27.79	19.02	±6.050	12.97	25.07	1395	18086	34963
	12	11.00	136.83	1.081	28.46	19.48	±6.050	13.43	25.53	1395	18728	35605
	13	11.00	147.83	1.105	29.09	19.91	±6.050	13.86	25.96	1395	19335	36212
	14	11.00	158.83	1.128	29.70	20.32	±6.050	14.27	26.37	1395	19911	36788
	15	11.00	169.83	1.150	30.27	20.72	±6.050	14.67	26.77	1395	20458	37335
	16	12.00	181.83	1.172	30.87	21.13	±6.050	15.08	27.18	1522	22939	41350
	17	12.00	193.83	1.194	31.44	21.51	±6.050	15.46	27.56	1522	23531	41943
	18	12.00	205.83	1.215	31.98	21.89	±6.050	15.84	27.94	1522	24098	42509
	19	12.00	217.83	1.234	32.50	22.24	±6.050	16.19	28.29	1522	24642	43053
	20	12.00	229.83	1.253	33.00	22.59	±6.050	16.54	28.64	1522	25164	43576
	21	15.08	244.91	1.276	33.61	23.00	±6.050	16.95	29.05	1912	32415	55551
Leeward	All	All	244.91	1.276	33.61	-14.38	±6.050	-20.43	-8.33	31055	634342	258582
										ΣForce	1015150	1015150

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)	Force $+(GC_{pi})$ (lb)	Force $-(GC_{pi})$ (lb)
Windward	1	13.83	13.83	0.834	21.97	15.39	±7.254	8.13	22.64	1065	8660	24109
	2	12.00	25.83	0.952	25.06	17.55	±7.254	10.29	24.80	924	9512	22917
	3	12.00	37.83	1.031	27.16	19.02	±7.254	11.76	26.27	924	10868	24274
	4	11.00	48.83	1.088	28.66	20.07	±7.254	12.81	27.32	847	10852	23140
	5	11.00	59.83	1.136	29.91	20.94	±7.254	13.69	28.20	847	11594	23883
	6	11.00	70.83	1.177	30.99	21.70	±7.254	14.45	28.95	847	12236	24524
	7	11.00	81.83	1.213	31.95	22.37	±7.254	15.12	29.62	847	12803	25092
	8	11.00	92.83	1.246	32.81	22.97	±7.254	15.72	30.23	847	13313	25601
	9	11.00	103.83	1.276	33.59	23.52	±7.254	16.27	30.77	847	13777	26066
	10	11.00	114.83	1.303	34.31	24.02	±7.254	16.77	31.28	847	14204	26492
	11	11.00	125.83	1.328	34.98	24.49	±7.254	17.24	31.75	847	14600	26888
	12	11.00	136.83	1.352	35.60	24.93	±7.254	17.67	32.18	847	14969	27257
	13	11.00	147.83	1.374	36.18	25.34	±7.254	18.08	32.59	847	15316	27604
	14	11.00	158.83	1.395	36.73	25.72	±7.254	18.47	32.98	847	15642	27931
	15	11.00	169.83	1.415	37.25	26.09	±7.254	18.83	33.34	847	15952	28240
	16	12.00	181.83	1.435	37.79	26.46	±7.254	19.21	33.72	924	17751	31156
	17	12.00	193.83	1.455	38.31	26.82	±7.254	19.57	34.08	924	18082	31487
	18	12.00	205.83	1.473	38.79	27.16	±7.254	19.91	34.42	924	18397	31803
	19	12.00	217.83	1.491	39.26	27.49	±7.254	20.24	34.74	924	18699	32104
	20	12.00	229.83	1.508	39.70	27.80	±7.254	20.55	35.06	924	18987	32392
	21	15.08	244.91	1.528	40.24	28.18	±7.254	20.92	35.43	1161	24295	41141
Leeward	All	All	244.91	1.528	40.24	-11.59	±7.254	-18.84	-4.33	18858	355325	81732
										ΣForce	665834	665834

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) $\qquad\qquad S_1 = 0.070g$

$S_s > 0.15$ & $S_1 > 0.04$ (using USGS values)
Geotech. report states part of soil "should be considered to liquefy during the design earthquake event" $\longrightarrow$ 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:

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

$F_v = 3.5$
E

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

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

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

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

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

$T_S = \dfrac{S_{D1}}{S_{DS}} = \dfrac{0.163}{0.505} = 0.323$

[illegible] $\rightarrow T = [illegible] > T_s$
[illegible]

$T_a < 0.8 T_S$ $\qquad$ 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
• per 21.2.1.4, Seismic Design Category D, E, or F uses special structural walls

R = 6

Long Period Transition
$T_L = 6s$ (Figure 22-15)

$T_a = 1.310 < T_L = 6$

$V = C_s \cdot W$

$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0.505}{\left(\frac{6}{1.0}\right)} = 0.08417$ but no bigger than $\frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.163}{(1.310)\left(\frac{6}{1.0}\right)} = 0.02074$ ← Governs

or $\frac{S_{D1} T_L}{T^2\left(\frac{R}{I}\right)} = \frac{(0.163)(6)}{(1.310)^2\left(\frac{6.0}{1}\right)} = 0.09498$

W = 41,852 kip

$V = C_s W = (0.02074)(41,852) = \underline{868.0\text{ k}}$

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

Total Building Weight	41852

Table A3

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

SNOW

Snow Loads

Ground Snowload

$p_g = 25\text{psf}$

Flat roof $\rightarrow p_s = p_f$

$p_f = 0.7 C_e C_t I p_g \geq 20 I$

Thermal Factor

$C_t = 1.0$

Importance Factor

$I = 1.0$

Exposure Category

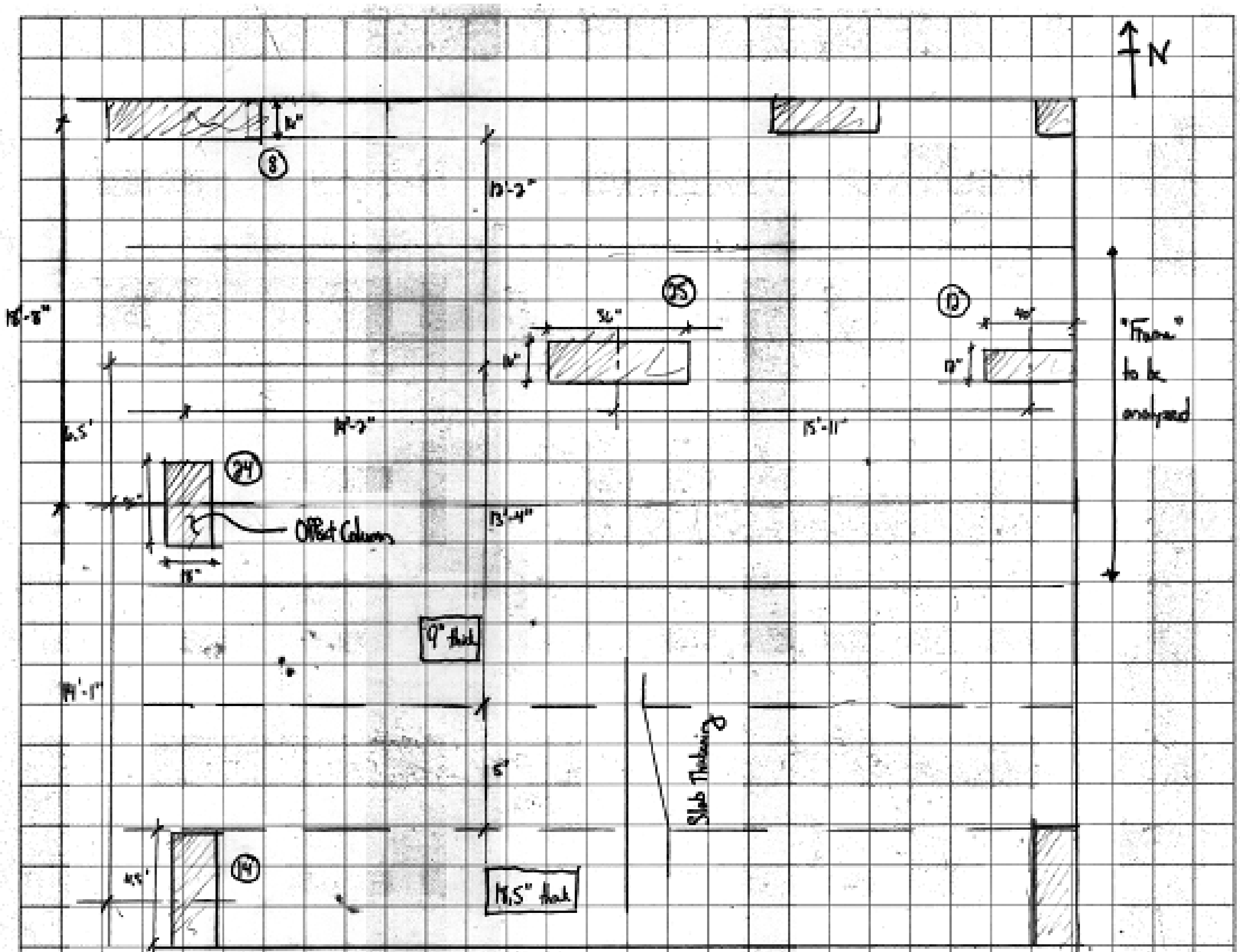
From wind, most case of Exposure C, assuming partially exposed roof $\rightarrow 1.0 = C_e$

$p_f = 0.7(1.0)(1.0)(1.0)(25) = 17.5\text{ psf} \rightarrow$ use minimum of $20\text{psf}(I) = \boxed{20\text{psf} = p_f}$

APPENDIX B

SPOT CHECK CALCULATIONS

SPOT CHECK – FLOOR SYSTEM



- Slab spanning along column line 13-25-24 assumed to be frame to be analyzed
- Slab also spanning along column line 14-24-8 to be used as a frame to check slab in other direction

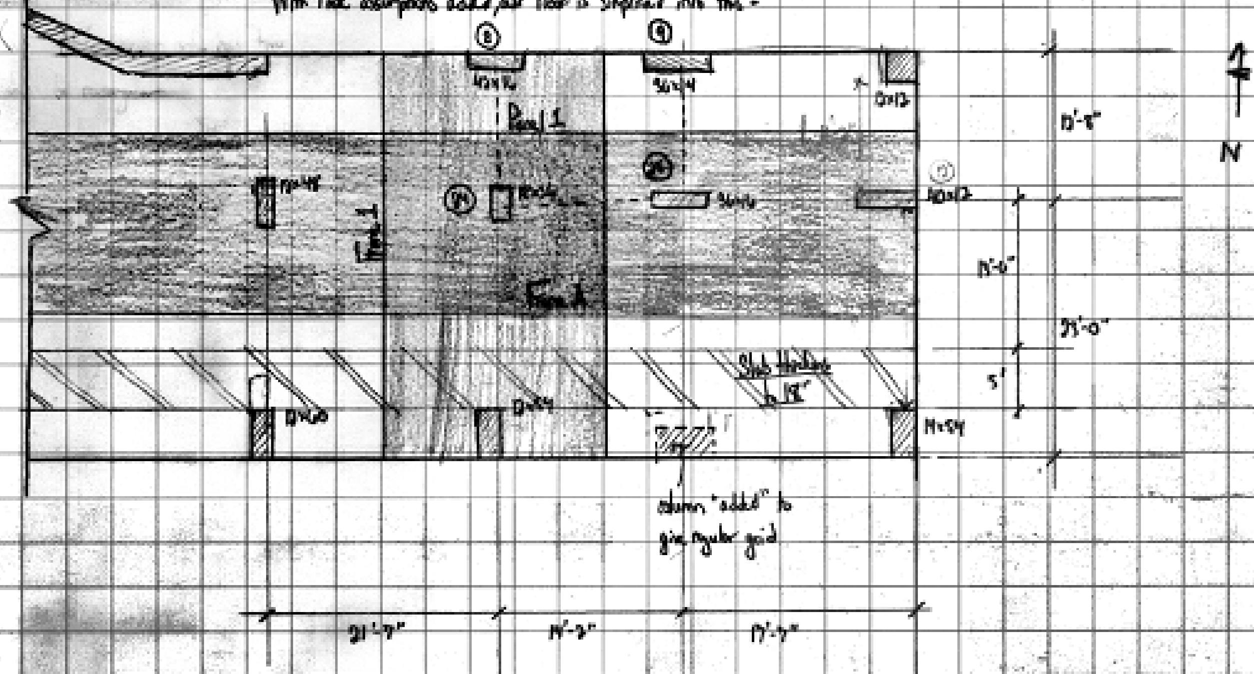
Quick check of ACI Table 9.5(c) for minimum thickness of slab w/o interior beams:

$$t_{min} = \frac{l_n}{33} = \frac{(16.5\cdot 12)}{33} = 6" \longrightarrow 9" > 6" \therefore \underline{OK}$$

[$l_{n\,max}$ was found to be btw. columns 24 & 8]

Moments will be calculated using Direct Design Method. Due to the irregularity of the column layout, this is not a valid method. However, for the purposes of this report, it is assumed that the layout is sufficiently "regular" and that any columns that do not align are simply "offset" an allowable distance, as the DDM does allow offsets.

With these assumptions added, our floor is simplified into this:



So, ignoring the span differences, DDM will now be implemented.

Factored Loads: (gravity)

Dead Load → $\underbrace{\left(150\text{ psf}\cdot\frac{9}{12}\right)+\left(115\text{ psf}\cdot\frac{2}{12}\right)+\left(4\text{ psf}\right)}_{\text{floor system}}+\underbrace{\left(18\text{ psf}+10\text{ psf}\right)}_{\text{superimposed DL}} = 136\text{ psf}+28\text{ psf} = 164\text{ psf}$

Live Load → 40 psf

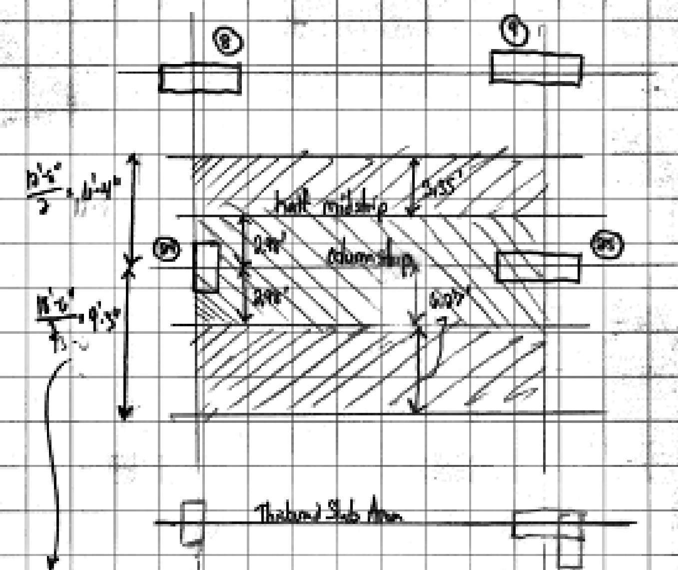
Factored → $1.2(164\text{ psf})+1.6(40) = 260.8\text{ psf}$

Panel 1 - Frame A

$l_n = 14.17' - \frac{9"}{12} - \frac{18"}{12} = 11.92'$

$l_2 = 18'\text{-}8" = 18.67'$ (actual distance used to design for worst case)

Column strip width $= \frac{11.92}{4} = 2.98'$ on each side



$M_o = \frac{q_u l_2 l_n^2}{8} = \frac{(0.2608)(6.33+9.25)(11.92)^2}{8}$

$= 72.17$ ft-k

Note: thickened slab area treated as the boundary of a new panel, because it is so much more stiff, much like a row of columns

Divide into positive & negative moments:

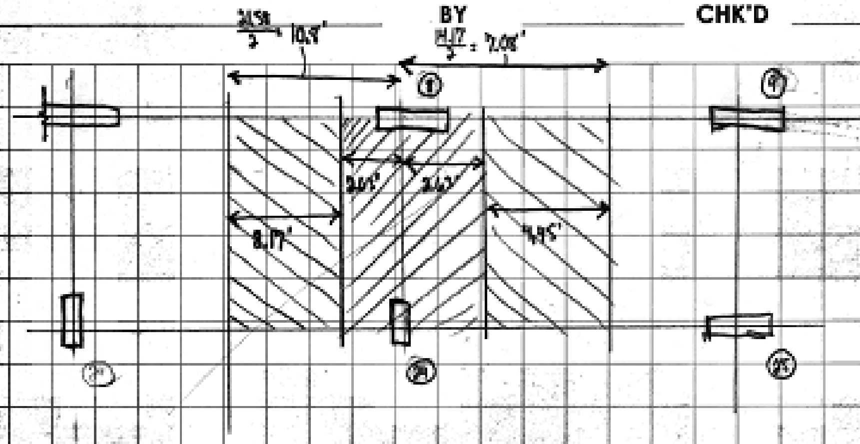
Neg. $M = -0.65(72.7) = -47.3^k$
Pos. $M = 0.35(72.7) = 25.4^k$

Negative moment
Column strip $= 0.75 \times -47.3^k = -35.5^k$
Mid strip $= 0.25 \times -47.3^k = -11.8^k$

Positive moment
Column strip $= 0.60 \times 25.4 = 15.2^k$
Mid strip $= 0.40 \times 25.4 = 10.2^k$

ITEM BY CHK'D

Panel 1 - Frame 1



$l_n = 12.07' - \frac{8}{12} - \frac{18}{12} = 10.5'$

$l_{\text{transverse}} = 14.17'$

Column strip width $= \frac{10.5}{4} = 2.63'$

$M_o = \frac{(0.2408)(10.8+7.08)(10.5)^2}{8} = 64.3$ ft-k

Divide into positive & negative moments:

$\text{Neg } M = -0.70(64.3) = -45.0$ ft-k

$\text{Positive } M = 0.52(64.3) = 33.4$ ft-k

$\text{Neg } M = -0.26(64.3) = -16.7$ ft-k

Negative Interior Moment

Column strip $= 0.75(-45) = -33.8$ ft-k

$M.S. = 0.25(-45) = -11.25$ ft-k

Positive M

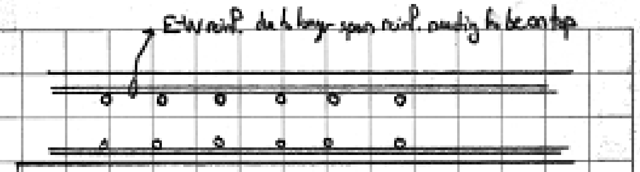
$C.S. = 0.60(33.4) = 20.0$ ft-k

$M.S. = 0.40(33.4) = 13.4$ ft-k

Negative Ext. Moment

$C.S. = 1.0(-16.7) = -16.7$ ft-k

$M.S. = 0$



Required Steel Area

$A_s = \dfrac{M_u}{\phi f_y jd}$ *assume $jd = 0.95d$ to start

From A

Column Strip - (25) & (24) $M_u = -35.5^{k}$

Stripwidth = 5.96'

*assume $d = h - 1.1 = 9" - 1.1 = 7.9"$

(-) moment $M_u = -35.5$

$A_s = \dfrac{(35.5 \cdot 12000)}{0.9 \times 60\,000 \times 0.95(7.9)} = 1.05 in^2$

$a = \dfrac{A_s f_y}{0.85 f'_c b} = \dfrac{(1.05)(60000)}{0.85(5950)(5.96 \cdot 12)} = 0.1742$

$\beta_1 = 0.75$ $c = \dfrac{a}{\beta_1} = \dfrac{0.1742}{0.75} = 0.2322 \rightarrow$ clearly tension controlled

4 #5's $\rightarrow A_s = 1.24 in^2$
6 #4's $\rightarrow A_s = 1.2 in^2$

$A_{s,min} = 0.0018bh = 0.0018(5.96)(9)(12) = 1.16$
Therefore, both may require 4 #5's or 6 #4's

(+) moment $M_u = 15.2^{k}$

$A_s = \dfrac{(15.2 \times 12000)}{0.9 \times 60000 \times 0.95(7.9)} = 0.450 in^2$

3 #4's $\rightarrow A_s = 0.60 in^2 < A_{s,min}$
Use $A_{s,min}$

So need to fit 4 #5's in ≈ 6' mid portion $\rightarrow$ space at max spacing of 18"

or (6) #4's @ 12" o.c.

Mid Strip - (25) & (24)
Stripwidth = 3.35' & 6.27'

(-) $M_u = -11.8$

$A_s = \dfrac{(\frac{11.8}{2} \cdot 12000)}{0.9 \cdot 60000 \cdot 0.95 \cdot 7.9} = 0.195 in^2$

(+) $M_u = \dfrac{10.2}{2}$
By visual inspection, this is governed by $A_{s,min} \rightarrow$ 4 #5's
or 7 #4's @ 12" o.c.

Check $A_{s,max} = 0.0018(6.27)(9)(12) = 1.22 in^2 \rightarrow$ must use $A_{s,min} =$ 4 #5's

Bar spacing, min = 2(h) = 2(9) = 18"

Frame 1 - Area steel calculation

Column strip over (8); $M_u = -16.7^{k}$; strip width = 5.25'

$d = \underbrace{h - 1.7}_{\text{assumed}} = 9 - 1.7 = 7.3"$

$A_s = \dfrac{M_u \cdot 12}{0.9 \cdot 60 \cdot 0.95 \cdot 7.3} = 0.03204 M_u$

$0.03204(-16.7) = +0.5351 \text{ in}^2$

$a = \dfrac{A_s f_y}{0.85 \cdot f'_c \cdot b} = \dfrac{(0.5351)(60)}{0.85(5.45)(5.25 \times 12)} = 0.1008$

$c = \dfrac{a}{\beta_1} = \dfrac{0.1008}{0.75} = 0.1344$ → clearly tension controlled

$A_{s,min} = 0.0018\,bh = 0.0018(5.25)(9) = 1.02 \text{ in}^2$ 35

Use $A_{s,min}$ → 4(#5's) $A_s = 1.24 \text{ in}^2$

Column strip over (24) ; $M_u = -45$

$A_s = 0.03204(45) = 1.44 \text{ in}^2$

5(#5's) = 1.55 in²

Column strip, (+) Moment = 20^{1k}

$A_s = 0.03204(20) = 0.6408 \text{ in}^2 < A_{s,min}$ → Use $A_{s,min}$

Both Spacing → 4(#5's) @ 18" o.c or (7) #4's @ 9" o.c.

Because both positive & negative mid-strip moments are less than 20^{1k}, $A_{s,min}$ will control. Use

$A_{s,min} = 0.0018(9)(8.125') = 1.58 \text{ in}^2$ → 6(#5's) means #5 @ 24" o.c. but must use minimum spacing of 18" or (8) #4's @ 13" o.c.

Summary

Frame A (E-W)

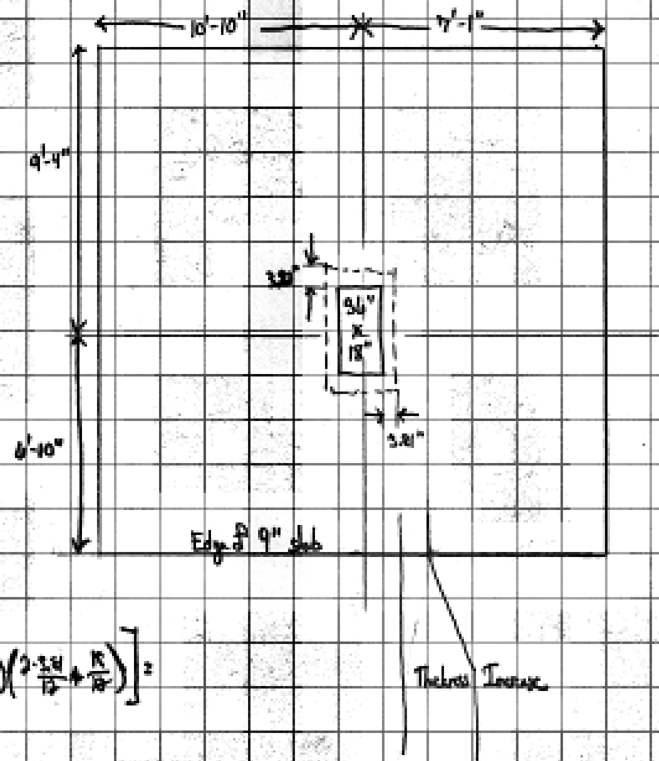
- Column Strip
  - Positive: 4#5's @ 18"*
  - Negative: 4 #5's
- ~~Midstrip~~
- Midstrip
  - Positive: 4#5's @ 18"*
  - Negative: #5's @ 18"*

Frame 1 (N-S)

- Column Strip
  - Ext. Neg.: 4 #5's*
  - Positive: 4#5's* @ 18"
  - Int. Neg.: 5 #5's
- Midstrip
  - Ext. Neg.: #5 @ 18"* →
  - Pos.
  - Int. Neg.

* Governed by $A_{s,min}$

Punching Shear Check



• moment transfer assumed to be negligible

Column 24 to be checked using actual floor layout

No. 5 bars typically used for flexural reinf.

$d_{avg} = 9" - \underbrace{\tfrac{3}{4}"}_{\text{clear cover}} - 0.625" = 7.625"$

$d/2 = 3.81"$

$q_u = 260.8$ psf (as calculated in slab flexural check)

$$V_u = 0.2608 \text{ ksf}\left[(10.83+7.08)(9.33+6.83) - \left(\frac{2\cdot 3.81}{12}+\frac{36}{12}\right)\left(\frac{2\cdot 3.81}{12}+\frac{18}{12}\right)\right] =$$

$$= 0.2608\,[289.4 - 7.76] = 73.5^k$$

$$V_n = V_c = \left(2+\frac{4}{\beta}\right)\lambda\sqrt{f'_c}\,b_o d = \left(2+\frac{4}{2}\right)(1.0)\sqrt{5{,}950}\,(138.5)\frac{(7.625)}{1000} = \underline{326\text{ k}} \quad \text{(ACI Eq. 11-31)}$$

$$\beta = \frac{36}{18} = 2;\; b_o = (2\cdot 3.81+18)\cdot 2 + (2\cdot 3.81+36)\cdot 2 = 138.5 \text{ in.}$$

$$V_c = \left(\frac{\alpha_s \cdot d}{b_o}+2\right)\lambda\sqrt{f'_c}\cdot b_o d = \left(\frac{40\cdot 7.625}{138.5}+2\right)\sqrt{5950}\cdot(138.5)(7.625) = \underline{342\text{ k}} \quad \text{(ACI Eq. 11-32)}$$

$\alpha_s = 40$

$$V_c = 4\lambda\sqrt{f'_c}\,b_o d = 4(1.0)\sqrt{5950}\,(138.5)(7.625) = \underline{325\text{ k}} \quad \text{(ACI Eq. 11-33)}$$

↑ governs

$$\phi V_c = 0.75(325) = 244\text{ k} > V_u = 73.5 \therefore \text{OK}$$

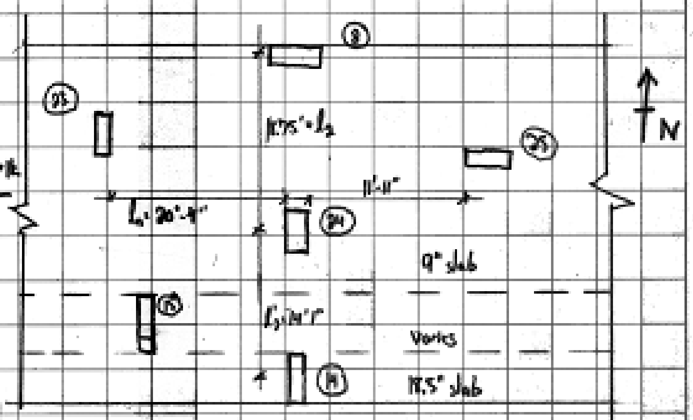
SPOT CHECK - COLUMN

COLUMN SPOT CHECK

Column 24, supporting the 7th Floor, will be checked.

ACI Eq. (13-7) $\longrightarrow M_u = 0.07\left[\left(q_{Du} + 0.5\,q_{Lu}\right) l_2\, l_n^2 - q'_{Du}\, l'_2\,(l'_n)^2\right]$

M_u is the design moment in column from slab. Despite being part of the Direct Design Method, span lengths will be taken from actual layout, rather than the one simplified for use in DDM. It is believed that the actual spans will give a closer approximation to the moments imposed.



E-W Direction

$M_u = 0.07\left[(196 + 0.5 \cdot 64) \cdot 18.75 \cdot 20.33^2 - 249 \cdot 14.08 \cdot 11.92^2\right] \approx \underline{88.8^{\text{ft-k}}}$

$q_{Du} = 1.2\left[51.2\,\text{psf} + \left(150 \times \frac{9''}{12}\right)\right] = 196\,\text{psf} = q'_{Du}$

(51.2 psf: superimposed; $150 \times \frac{9''}{12}$: structural slab)

$q'_{Du} = 1.2\left[51.2\,\text{psf} + \left(150 \times \frac{12.5}{12}\right)\right] = 249\,\text{psf}$ (12.5" is avg thickness, taking into account the thickened slab portion)

$q_{Lu} = 1.6(40\,\text{psf}) = 64\,\text{psf} = q'_{Lu}$

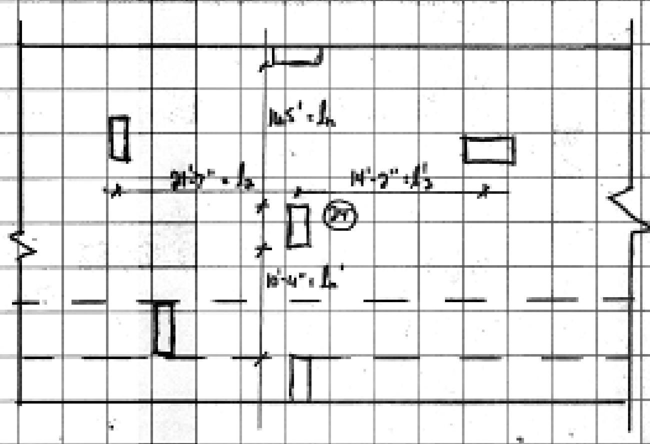
$l_2 = 18.75'$

$l_n = 20.33'$

$l'_2 = 14.08'$

$l'_n = 11.92'$

$M_u = 96.2^{\text{ft-k}}$



N-S Direction

$l_2 = 21.58'$

$l_n = 16.5'$

$l'_2 = 14.17'$

$l'_n = 10.33'$

$M_u = 0.07\left[(196 + 0.5 \cdot 64) \cdot 21.58 \cdot 16.5^2 - 249 \cdot 14.17 \cdot 10.33^2\right]$

$= 67.4^{\text{ft-k}}$ $\longrightarrow$ *Column identical below and above; equal moment goes to both $\rightarrow$ $M_{uy} = \frac{88.8}{2} = 44.4^{\text{ft-k}}$

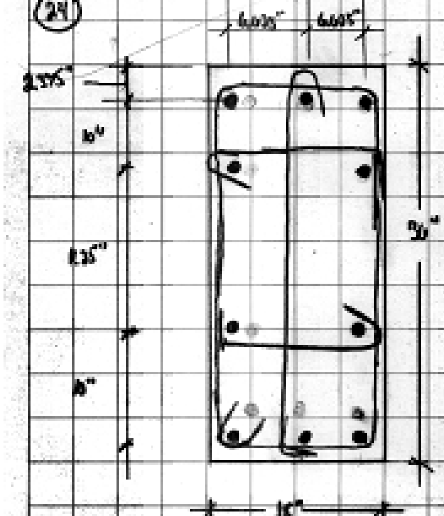
$M_{ux} = \frac{67.4}{2} = 33.7^{\text{ft-k}}$

Column 24 Load Takedown								
Statistics				Loads				Load Combination
Column Below Level	Floor Height	A_t	Column Area	Self-Weight (k)	Dead (psf)	Live (psf)	Snow (psf)	$1.2D+1.6L+0.5(L_r \text{ or } S)$
Roof	16	217	2	5	195	40	20	67
21	15.08	297	4	7	164	40	0	144
20	12	297	4	7	164	40	0	222
19	12	297	4	7	282	40	0	341
18	12	297	4	7	164	40	0	419
17	12	297	4	7	164	40	0	496
16	12	297	4	7	164	40	0	574
15	11	297	4	7	164	40	0	651
14	11	297	4	7	164	40	0	729
13	11	297	4	7	164	40	0	806
12	11	297	4	7	164	40	0	883
11	11	297	4	7	164	40	0	961
10	11	297	4	7	164	40	0	1038
9	11	297	4	7	164	40	0	1116
8	11	297	4	7	164	40	0	1193
7	11	297	4	7	164	40	0	1271

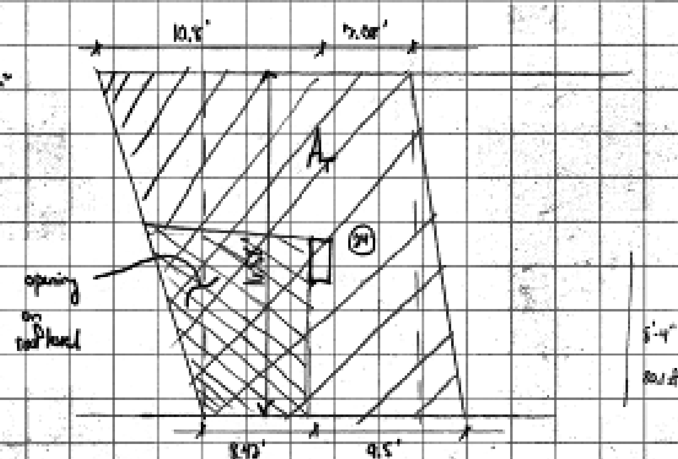
Table B1

For simplicity, the moments in each direction are assumed to have no interaction, so that biaxial loading can be broken down into P_u/M_{uy} & P_u/M_{ux}. Column only checked about weak axis, as this is where the largest moment is acting, making this the critical section. Slenderness effects were ignored.

24



10 #8's → $A_s = 0.79 in^2$
#3 @12"
$f'_c = 8$ ksi
$f_y = 60$ ksi
$P_u = 1271^k$
$M_{uy} = 44.4^{'k}$



$A_T = (8.42+7.08)(16.58) + (10.8-8.42)(16.58)\frac{1}{2} + (9.5-7.08)(16.58)\frac{1}{2}$

$= 297 sf$

$w_u = 260.8$ psf (from slab check)

$P_u = (297sf)(260.8psf) = 77.5^k$

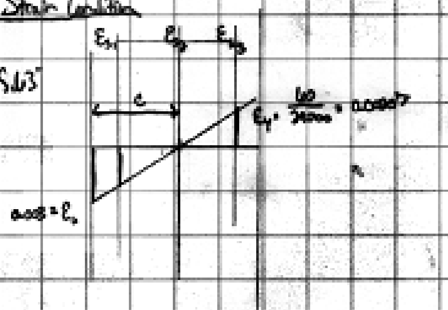
$1\frac{1}{2}$" clear cover

Pure Compression

$P_o = 0.85(5.95ksi)(30\cdot18 - 10\cdot0.79) + 10(0.79)(60) = 3711^k$

Balanced Strain Condition

$d = 18 - 2.375 = 15.63"$



$c = \frac{0.003}{0.003+0.00207}\cdot(18-2.375)$

$= 9.25"$

$\varepsilon_{s2} = \frac{0.003}{9.25}(9.25-9) = 0.0000811 \approx 0$

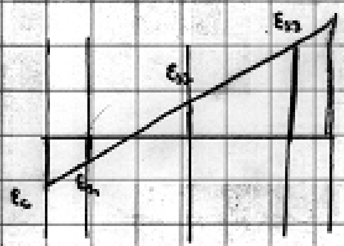
$\varepsilon_{s1} = \frac{0.003}{9.25}(9.25-2.375) = 0.00223 \rightarrow$ use 0.00207

$P_n = 0.85(5.95)(30)(0.75\cdot9.25) + \cancel{4\cdot0.79\cdot60} - \cancel{4\cdot0.79\cdot60} = 1263^k$

$M_n = 0.85(5.95)(30)(0.75)(9.25)\left(9 - \frac{0.75\cdot9.25}{2}\right) + 4\cdot0.79\cdot60(9-2.375) - 4\cdot0.79(9-15.63)$

$= 689^{'k}$

Pure bending



assume ε_{s1} & ε_{s2} do not yield

$$f_{s_1} = \frac{0.003}{c}(c-2.375)(29000) = 87 - \frac{206.6}{c}$$

$$f_{s_2} = \frac{0.003}{c}(c-9)(29000) = 87 - \frac{783}{c}$$

$$f_{s_3} = -60$$

$$\Sigma F = 0 = 0.85(5.95)(36)(0.75)(c) + 4 \cdot 0.79 \cdot \left(87 - \frac{206.6}{c}\right) + 2\left(87 - \frac{783}{c}\right) \cdot 0.79 + 4 \cdot 0.79 \cdot -60 = 0$$

$$136.6c + 274.9 - \frac{652.9}{c} + 137.5 - \frac{1237}{c} - 189.6 = 0$$

$$136.6c^2 + 274.9c - 652.9 + 137.5c - 1237 - 189.6c = 0$$

$$136.6c^2 + 222.8c - 1889.9 = 0$$

$$c = \frac{-222.8 \pm \sqrt{222.8^2 - 4 \cdot 136.6 \cdot -1889.9}}{2 \cdot 136.6} = \frac{-222.8 \pm 1040.77}{273.2} = 2.99 \text{ in.}$$

$$f_{s_1} = 17.9 \text{ ksi}$$

$$f_{s_2} = -175 \quad \therefore \text{ No Good}$$

assume only ε_{s1} does not yield

$$\Sigma F = 0 = 136.6c + 4 \cdot 0.79\left(87 - \frac{206.6}{c}\right) + 2 \cdot 0.79 \cdot -60 + 4 \cdot 0.79 \cdot -60 = 0$$

$$136.6c + 274.9 - \frac{652.9}{c} + -94.8 + -189.6 = 0$$

$$136.6c^2 - 9.5c - 652.9 = 0$$

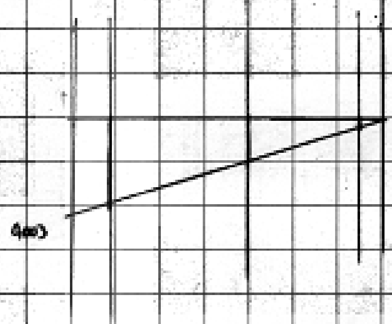
$$c = \frac{9.5 \pm \sqrt{9.5^2 - 4 \cdot 136.6 \cdot -652.9}}{2 \cdot 136.6} = \frac{9.5 \pm 597.4}{273.2} = 2.22 \text{ in.}$$

$$f_{s_1} = -6.06 \text{ ksi}$$

$$M_n = 0.85(5.95)(36)(0.75)(2.22)\left(9 - \frac{0.75 \cdot 2.22}{2}\right) - 4 \cdot 0.79 \cdot 6.06\left(9 - 2.375\right) + 2 \cdot 0.79 \cdot 60(0) + 4 \cdot 0.79 \cdot 60(15.63 - 9)$$

$$= 2476 + 126.9 + 1257 = 3606^{\text{ik}} = 300^{\text{'k}}$$

c=h



$c = 18"$

$\varepsilon_{s1} = \frac{0.003}{18}(18-2.375) = 0.00260 > 0.00207 \rightarrow$ use $f_{s1} = 60$

$\varepsilon_{s2} = \frac{0.003}{18}(18-9) = 0.0015 \qquad f_{s2} = 43.5\text{ ksi}$

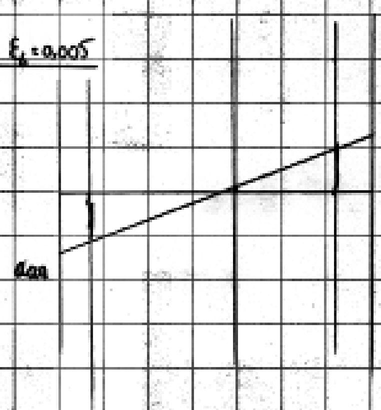
$\varepsilon_{s3} = \frac{0.003}{18}(18-15.63) = 0.000395 \qquad f_{s3} = 11.5\text{ ksi}$

$P_o = 0.85(5.95)(36)(0.75)(18) + 4\cdot0.79\cdot60 + 2\cdot0.79\cdot43.5 + 4\cdot0.79\cdot11.5 = 2{,}753\text{ k}$

$M_o = 0.85(5.95)(36)(0.75)(18)\left(9 - \frac{0.75\cdot18}{2}\right) + 4\cdot0.79\cdot60\left(9-2.375\right) + 2\cdot0.79\cdot43.5\left(9-9\right) + 4\cdot0.79\cdot11.5\left(9-15.63\right)$

$= 545^{\text{ft-k}}$

$\varepsilon_t = 0.005$



$c = \frac{0.003}{0.003+0.005}(15.63) = 5.86"$

$\varepsilon_{s1} = \frac{0.003}{5.86}(5.86-2.375) = 0.00178 \qquad f_{s1} = 51.7\text{ ksi}$

$\varepsilon_{s2} = \frac{0.003}{5.86}(5.86-9) = -0.00161 \qquad f_{s2} = -46.6\text{ ksi}$

$\varepsilon_{s3} = \frac{0.003}{5.86}(5.86-15.63) = -0.00499 \qquad f_{s3} = -60\text{ ksi}$

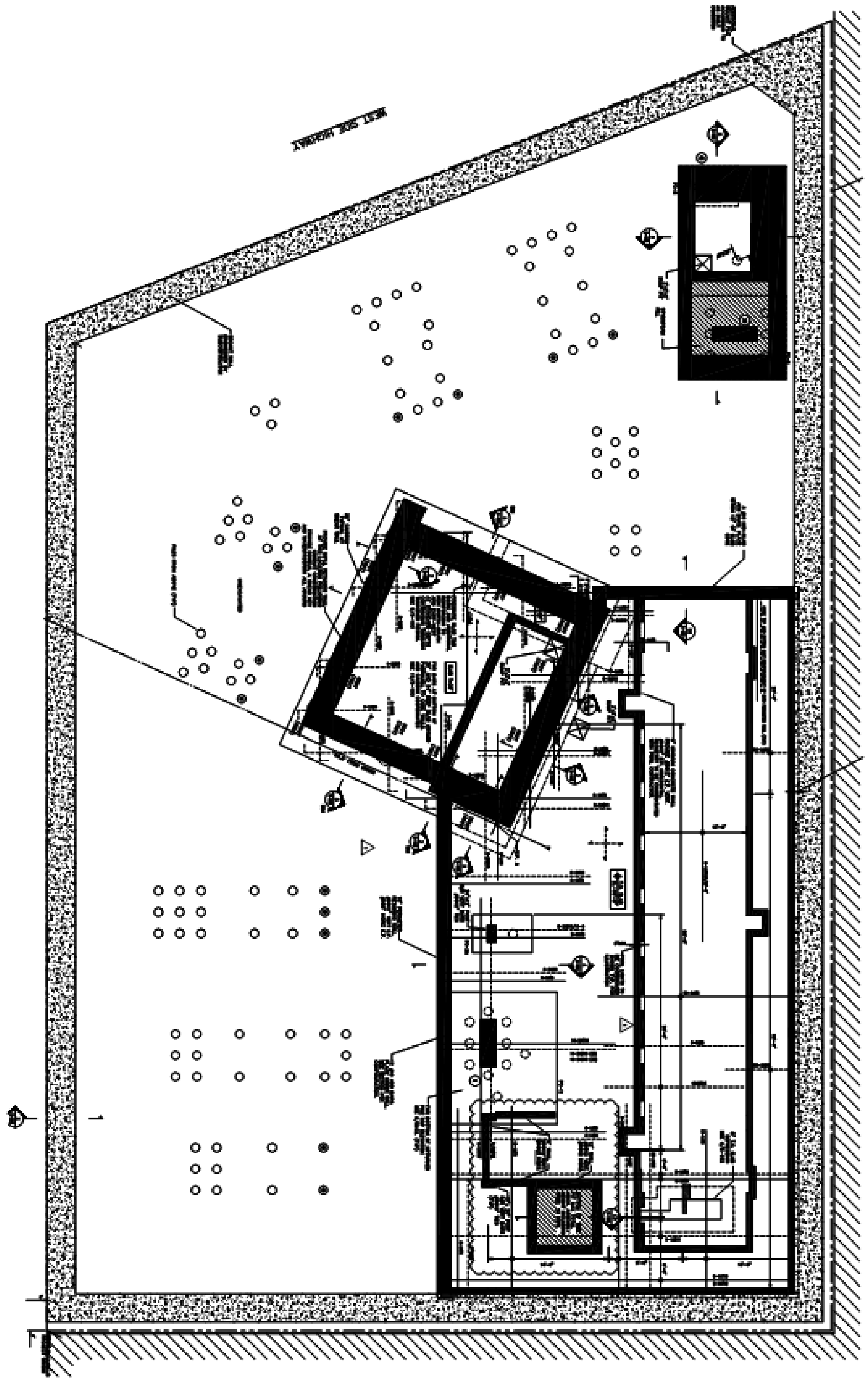
$P_o = 0.85(5.95)(36)(0.75)(5.86) + 4\cdot0.79\cdot51.7 - 2\cdot0.79\cdot46.6 - 4\cdot0.79\cdot60 = 700\text{ k}$

$M_o = 0.85(5.95)(36)(0.75)(5.86)\left(9 - \frac{0.75\cdot5.86}{2}\right) + 4\cdot0.79\cdot51.7\left(9-2.375\right) - 4\cdot0.79\cdot60\left(15.63-9\right)$

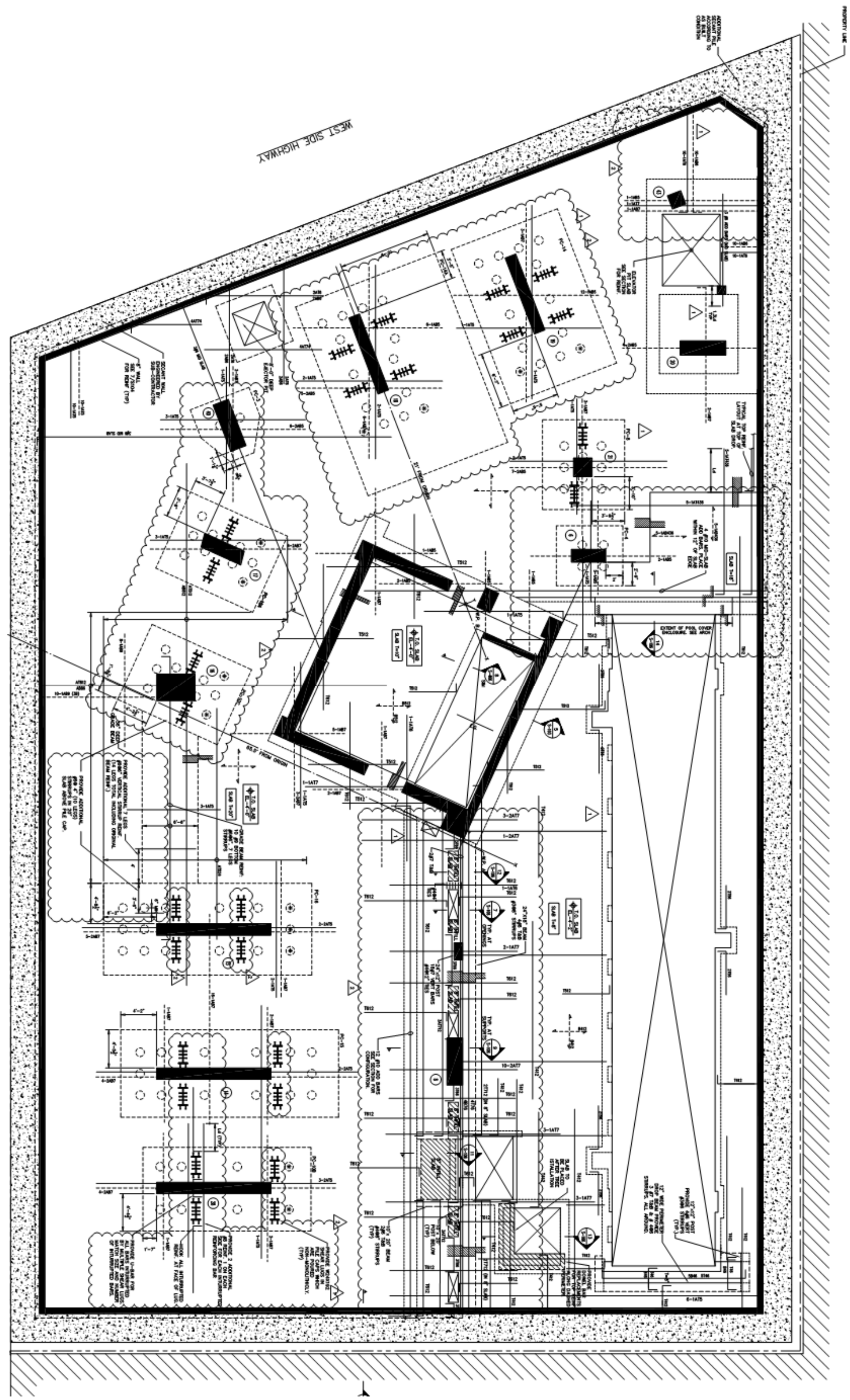
$= 439\text{ ft-k}$

APPENDIX C

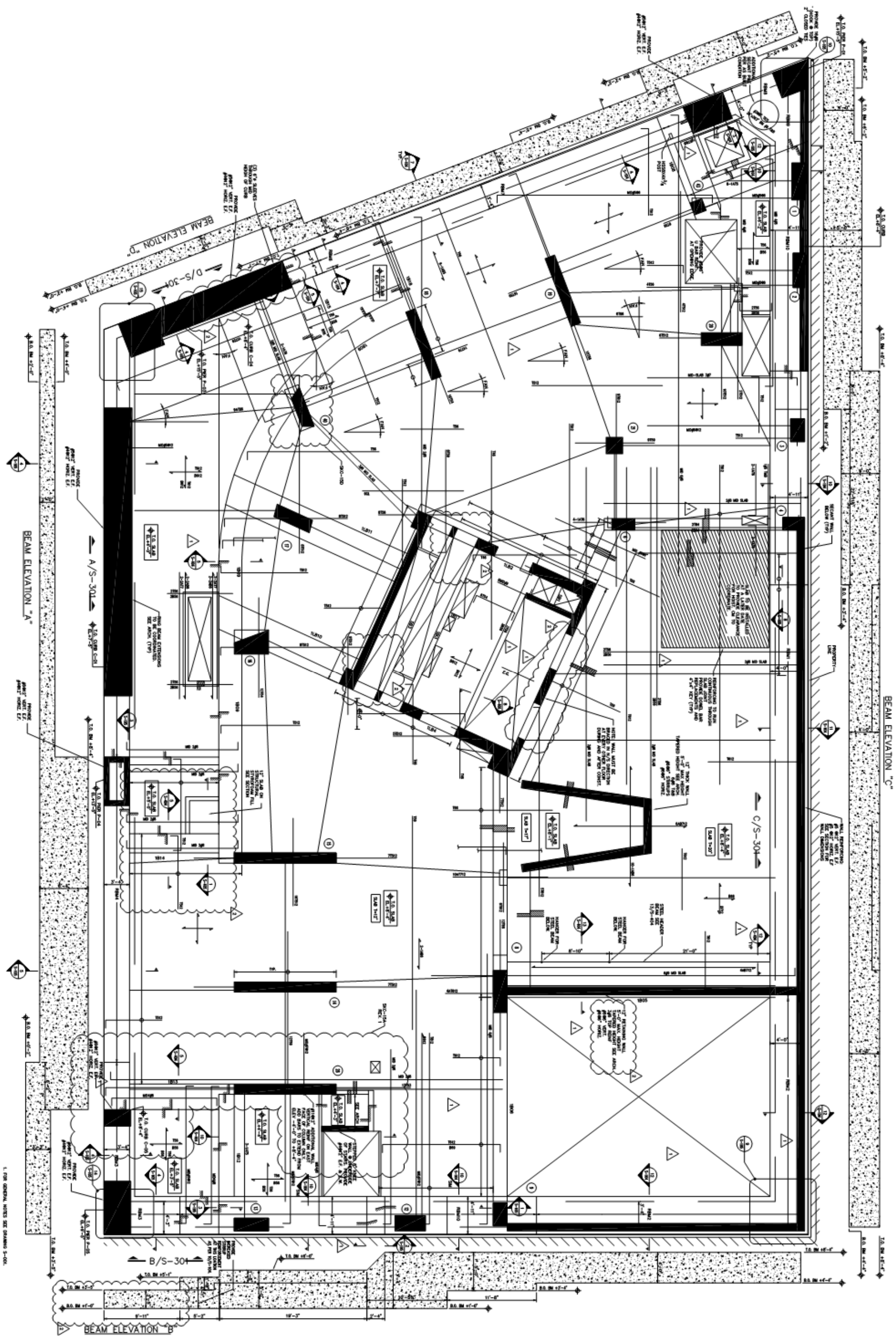
PLANS & ELEVATIONS



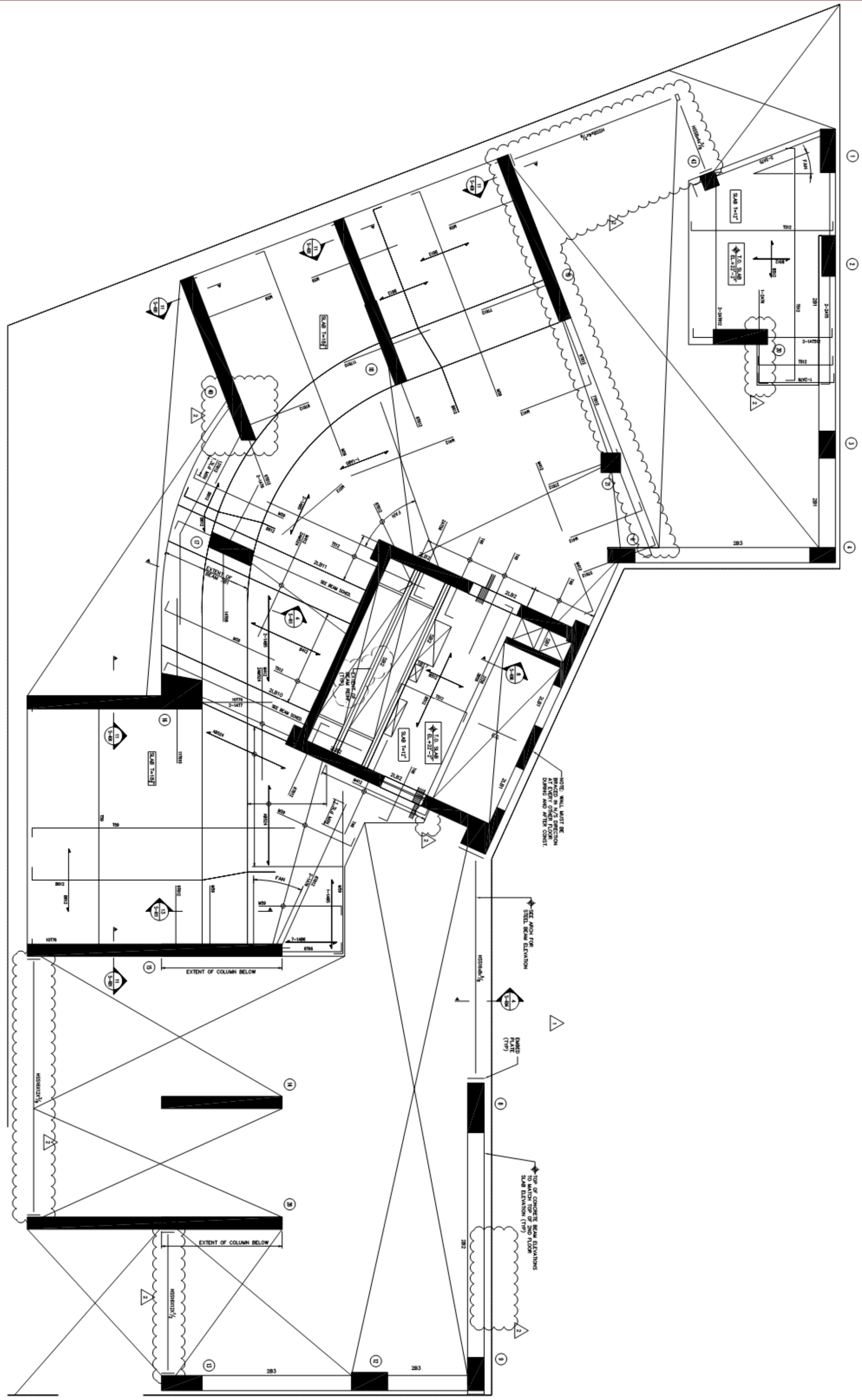
Sub-Cellar Floor Plan



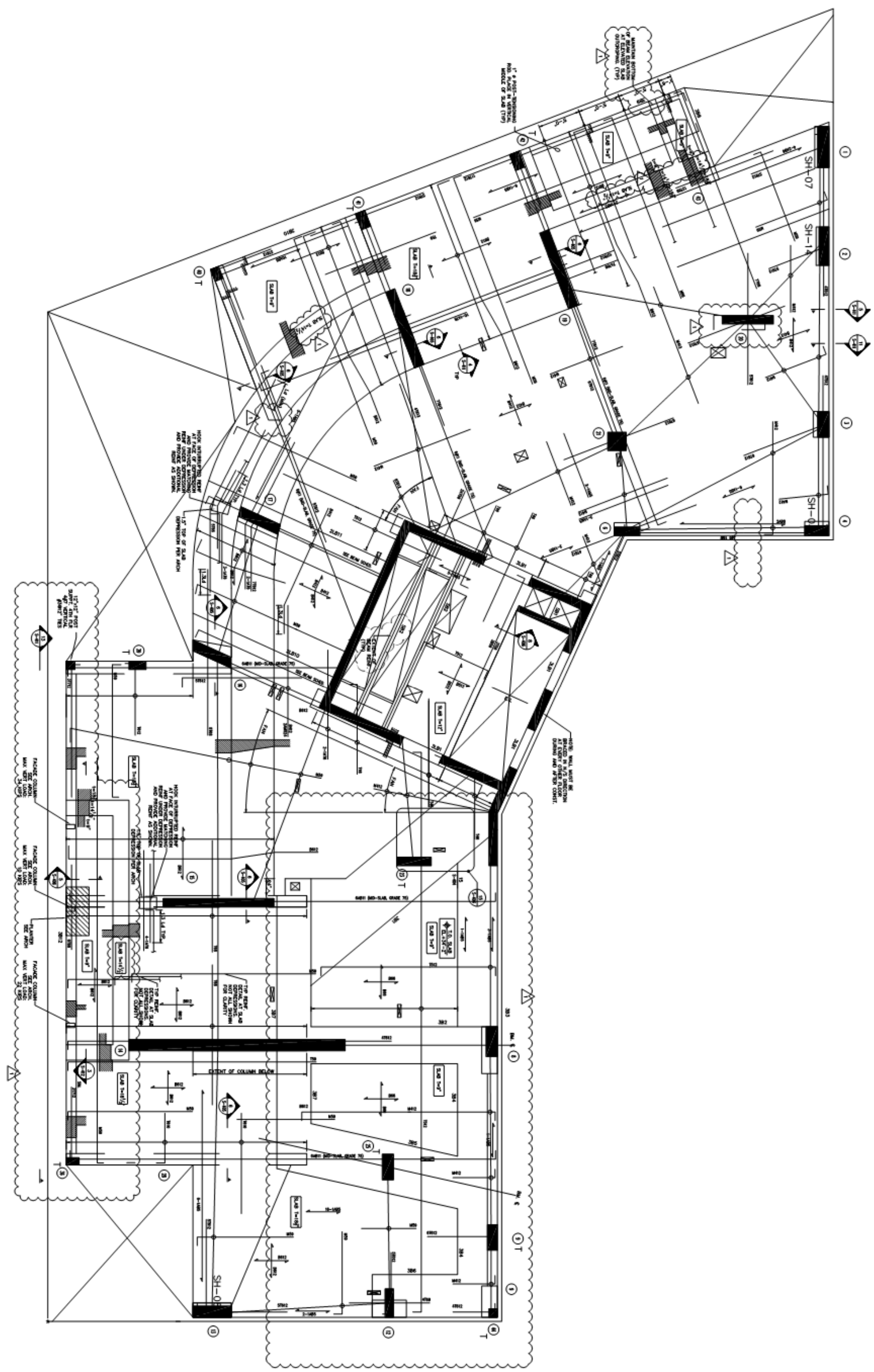
Cellar Floor Plan



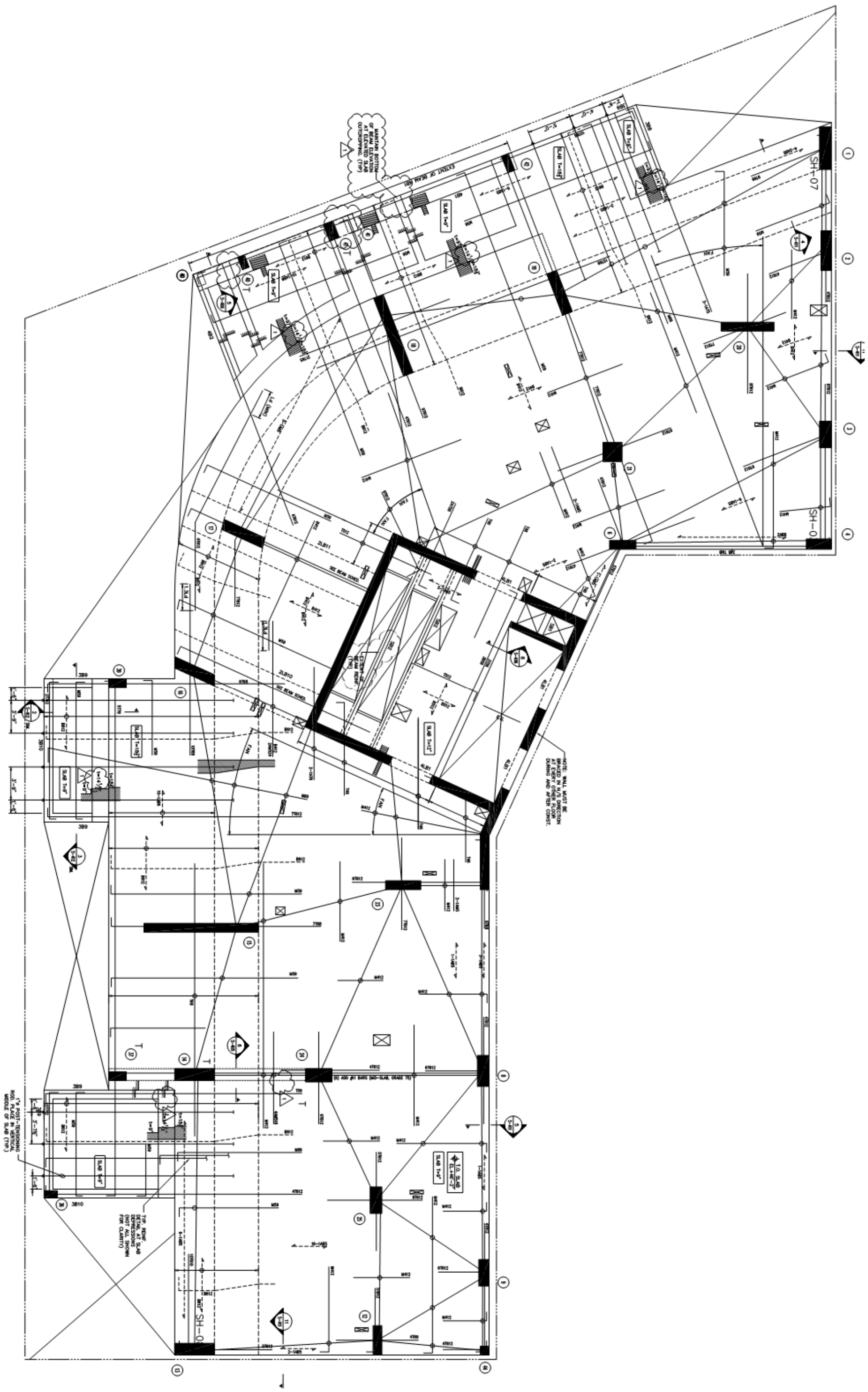
Ground Floor Plan



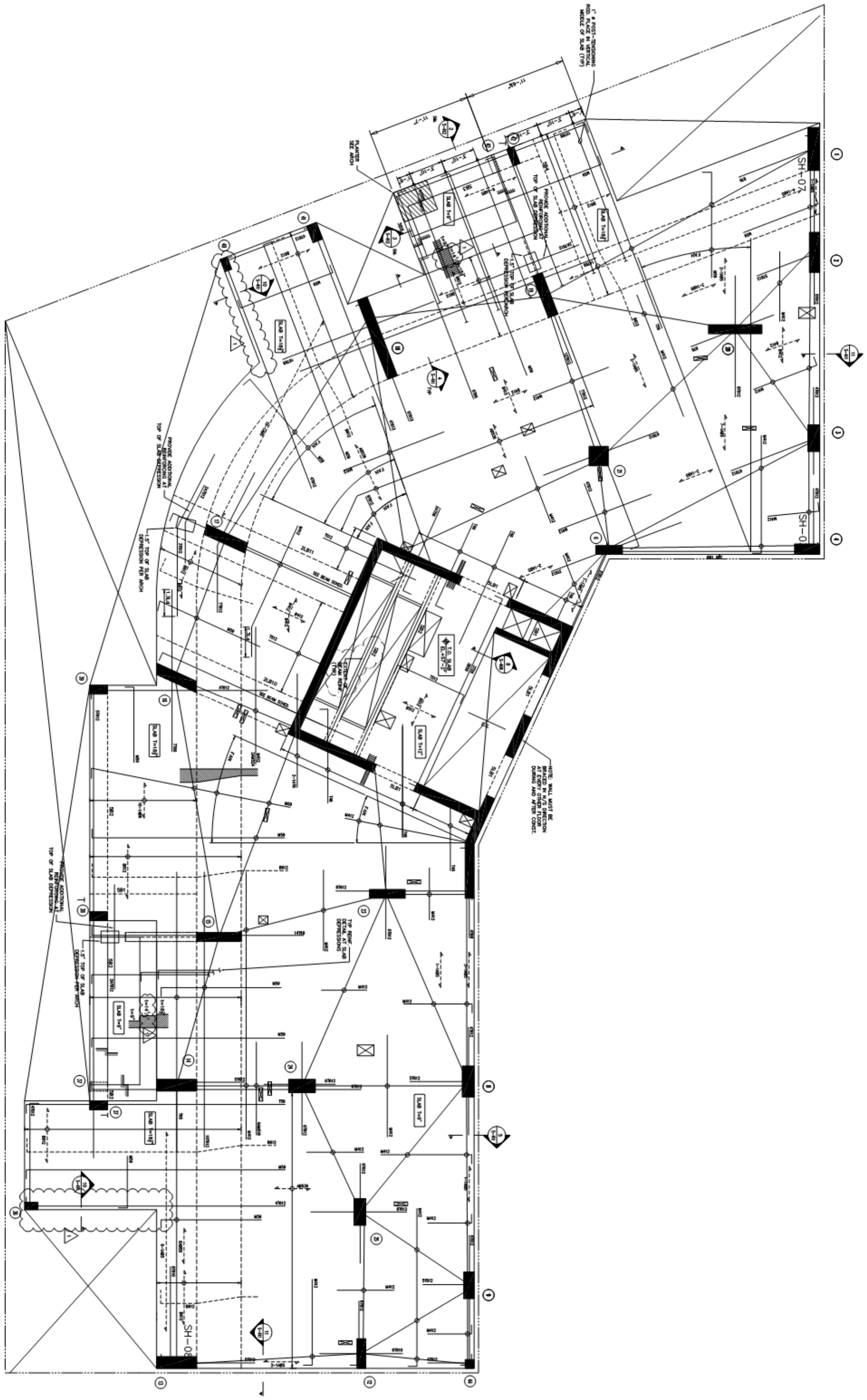
2nd Floor Plan



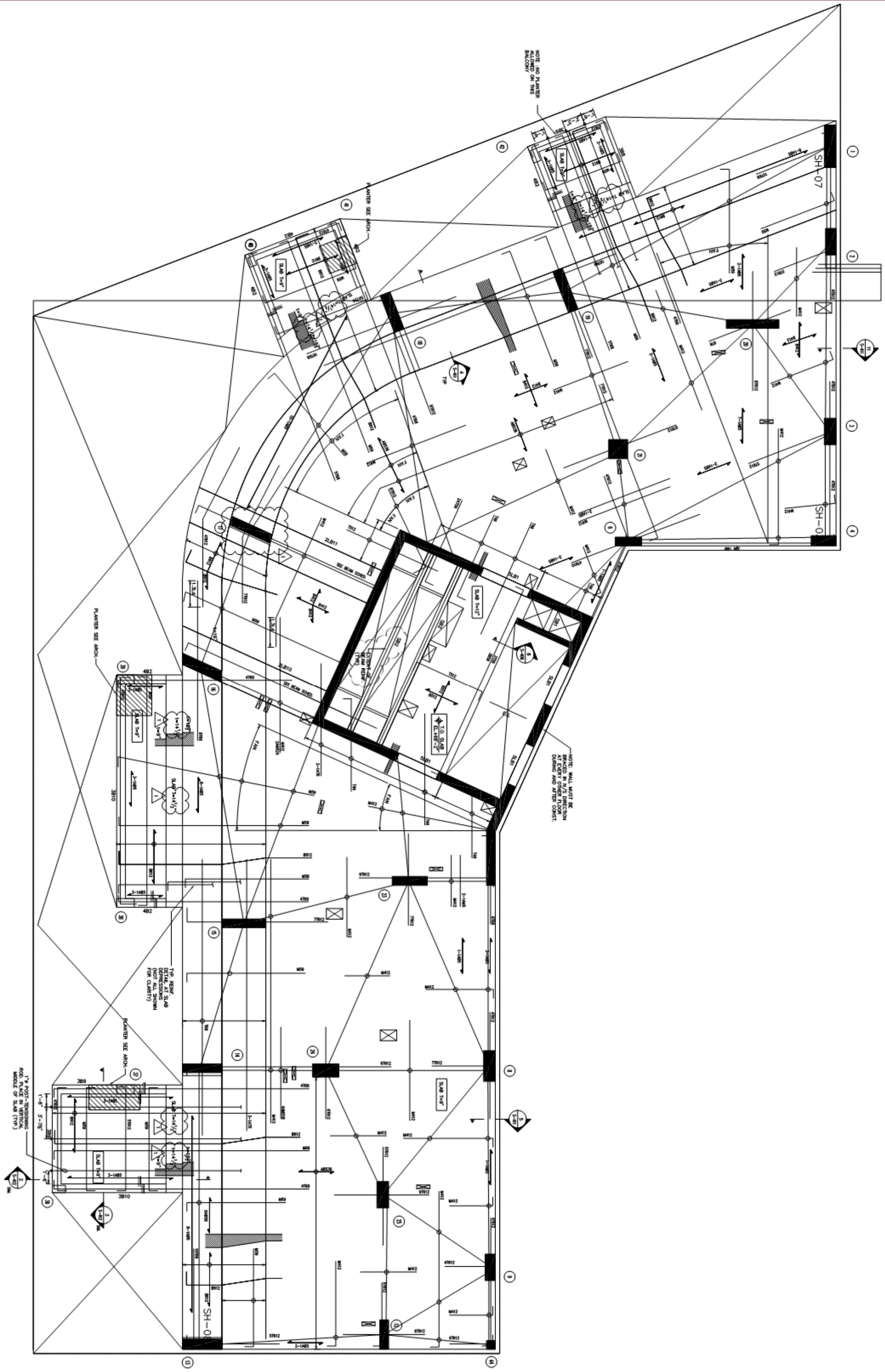
3rd Floor Plan



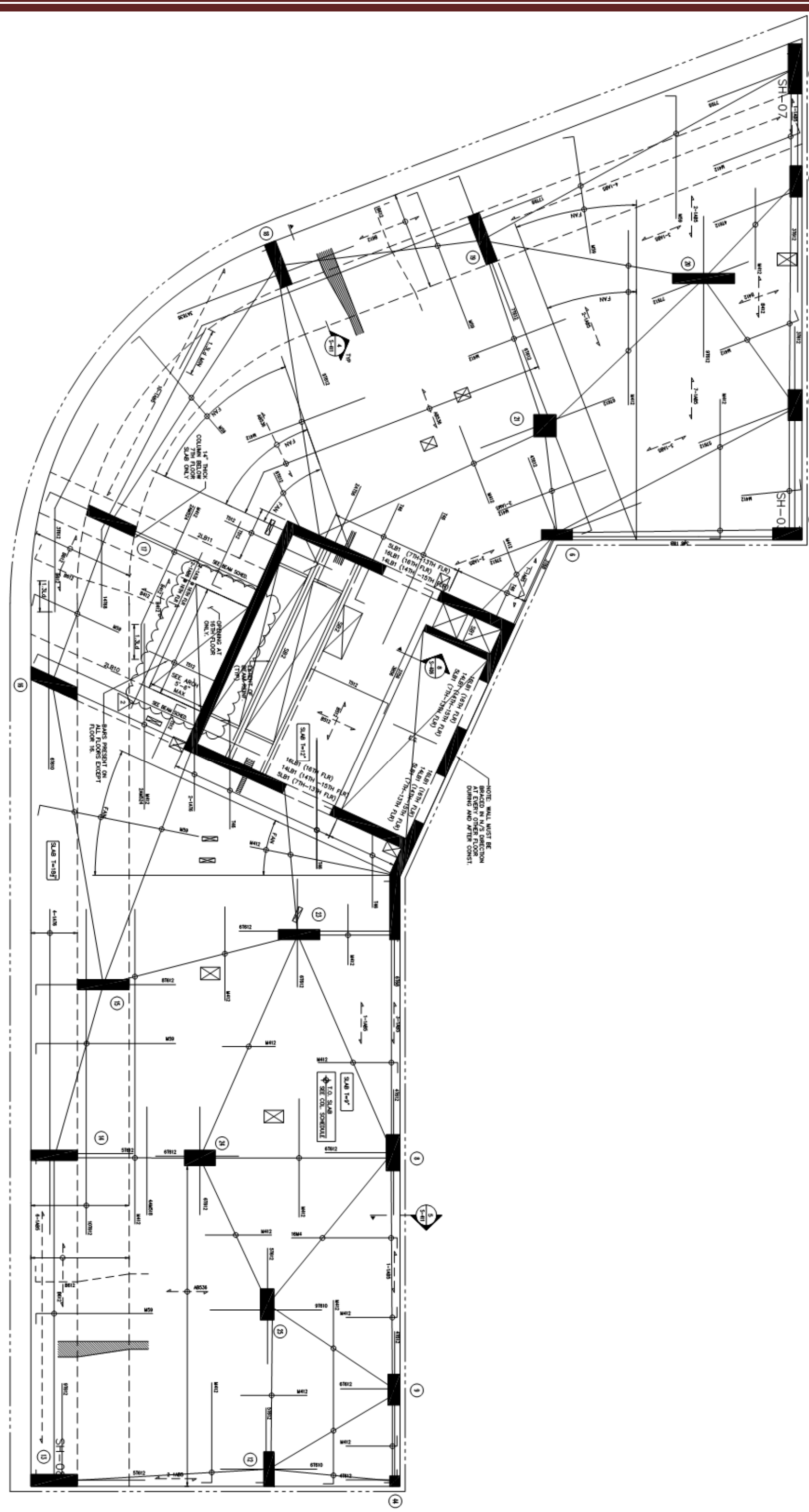
4th Floor Plan



5th Floor Plan

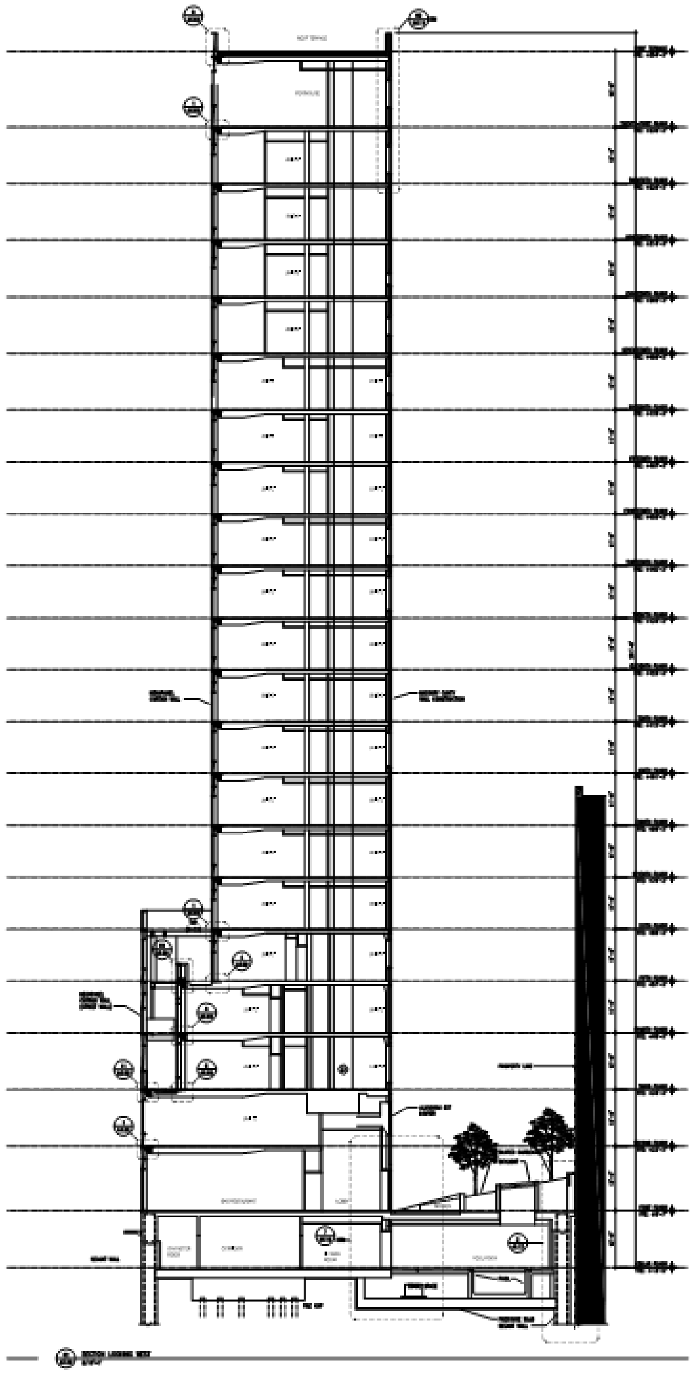


6th Floor Plan



7th-16th Floor Plan

17th-Roof Plans differ from typical plan only slightly



Section through east portion of building looking west

APPENDIX D

IMAGES

Figure D1: View looking west of the dark gray brick facade

Figure D2: View of thickened slab

Figure D3: View from Westside Highway