IAC/InterActiveCorp Headquarters

New York, NY

Technical Report #1

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Structural Option

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TABLE OF CONTENTS

Executive Summary

The intent of this report is to analyze the structural existing conditions for IAC/InterActiveCorp Headquarters in New York City. This is an office building completed in 2007 that serves as the main office for IAC, which is a leading internet company. Serving more than four hundred of the company's personnel, this eleven story building consists of an open office plan on levels 2 through 9.

The IAC/InterActiveCorp Headquarters is very unique to the Chelsea, Manhattan landscape in which it resides. Among some of the reasons are that it is the first commercial building in New York City designed by Frank Gehry and its shape and color are very different from the buildings in its vicinity. However, the goal of this report is to focus on the unusual structural design, and to analyze its existing conditions by striving to make credible assumptions and simplifications, while studying its wind, seismic, and gravity systems.

Wind analysis for this report was performed using the ASCE 7 standard, as opposed to the New York City Building Code, which explains the lower wind pressures found in this analysis versus the designed. Another reason for this difference is that the building underwent a wind tunnel test in order to determine appropriate pressures because of its location and irregular shape. For seismic design, which was also designed using ASCE 7, the base shear determined only varied by about 3% from the initial design conducted by the firm, DeSimone Consulting Engineers. Ultimately, however, a smaller base shear was decided upon after site specific spectra analysis was conducted.

Gravity checks were performed for a 'typical' panel of the flat plate slab, based on the ACI 2008 code. This tended to produce lower required steel than what was actually used in the design. Some possible reasons for this could be due to the simplification of the column layout for hand calculation purposes. Another explanation could be serviceability factors, which were not fully studied in this report. Additionally, a typical column was analyzed for axial and bending loads. This column was found to be inadequate through use of an interaction diagram for axial and bending. This discrepancy is likely due to the same reasons as only certain aspects of the flat plate system check worked.

Other important parameters such as gravity loading, codes used, and material strength are addressed throughout this report. All of these factors contribute to gaining a better understanding of the IAC Headquarters' structural system.

Introduction

The IAC/InterActiveCorp Headquarters is located in the neighborhood of Chelsea in Manhattan. It is the first commercial office building designed by Frank Gehry in New York City and serves as the headquarters for IAC, which is a leading internet company that encompasses over 35 sites such as match.com, ask.com, gifts.com, and collegehumor.com.

There is one level below-grade and eleven above. The cellar has a parking area with cars jacked up for storage and a ramp leading up to the first floor. The first floor opens to an immense lobby space mostly unfurnished and 20 feet high. The remaining second through ninth floors provide office space for the 400+ IAC employees, designed in an open floor plan manner. Typical floor heights at these levels are 12' to 14'. Most of the columns slope in the same direction because the floors gradually set back. These sloped columns allow them to be placed in desirable locations in the floor plan. On the 6th floor, the executive floor, resides an outdoor terrace which is a byproduct of the setback at that level. The tenth and eleventh floors primarily contain the mechanical units and are known as the "mechanical penthouse". The roof is accessible and contains much of the large HVAC equipment, as well as a large window washing unit.

The IAC Headquarters is designed to accommodate the artistic vision of Frank Gehry. Completely clad with laminated, double-glazed, fritted panels, the concrete superstructure provides its function while being completely dictated by the architectural intent.

Figure 1: Model of IAC Headquarters, showing its relationship to the Hudson River & Chelsea Piers

Structural System Overview

Foundations

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

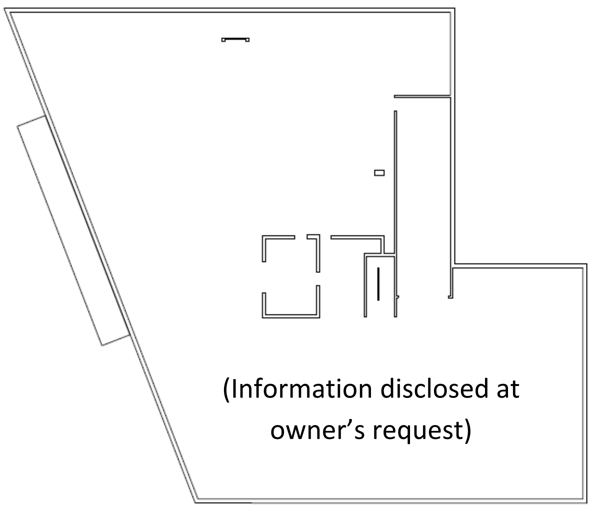


Figure 2: Foundation Plan

IAC Headquarters is actually located outboard of the original Manhattan shoreline. It was filled in the 1800s during a period of land reclamation, so the soil is very poor. The soft river sediments below the building were not suitable for the building loads, so a deep foundation system was necessary. The gravity columns are supported on concrete-filled steel pipe piles (with a conical tip, as agreed upon with NYSDEC because of environmental sensitivity). These piles have a 175 ton capacity to provide the required axial capacity. There are also twenty-three 18" diameter caissons that end bear on the bedrock. Figure 2, to the left, shows the foundation plan. Because the building is located below the 100-year flood elevation, much concern was taken with the waterproofing, as well as a hydraulic flood gate designed to seal the entrance ramp of the parking garage when needed. In addition, it was also contaminated from a previous ConEdison Manufactured Gas Plant facility, so containment was very important.

Superstructure

Floor System

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

The slab thickness for the first through fifth floors is 12" with primarily #5 @ 12" o.c. top and bottom bars in the 5000 psi strength concrete. Additional top and bottom rebar is placed at the columns and midspans respectively when necessary. This is likely because of longer spans or increased loading due to the room's function. At the sixth floor, where the building sets back (leaving space for an outdoor terrace), the slab thickness is 24". The concrete strength is 5000 psi as well, but the top and bottom reinforcing bars are typically #7 @ 12" o.c. It is at this location that the column layout changes much more drastically. This thicker slab acts as a transfer diaphragm, which, in addition to supporting vertical live, dead, and snow loads, the floor system transfers lateral forces. Forces that may be transferred through the slab are seismic and wind loading, as well as the distribution of loads from the upper columns down to the lower ones (if they are not stacked directly above one another, as is the case at the sixth floor). The seventh through roof levels have similar slab properties to the first through fifth floors, except that the upper floors have a slab thickness of 14". An unusual aspect of the slab reinforcing details is that unlike typical American Concrete Institute standard details which involves rotating rebar to match specific edge angles, the structural designers chose to design the reinforcing steel in the north-south and east-west orthogonal directions. This was done in an effort to improve the constructability of the building by eliminating the necessity to rotate rebar in various directions because of the unusual edge shape.

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

Gravity System:

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

The column strength for the building is 5950 psi. The columns in the basement are primarily 28" in diameter for the perimeter columns and 34" to 38" in diameter for the interior columns. This range of column diameters is fairly consistent throughout the ground through fifth floors, but at the sixth floor the sizes are reduced to 20" to 24". Columns are typically spaced between 25 and 30 feet apart.

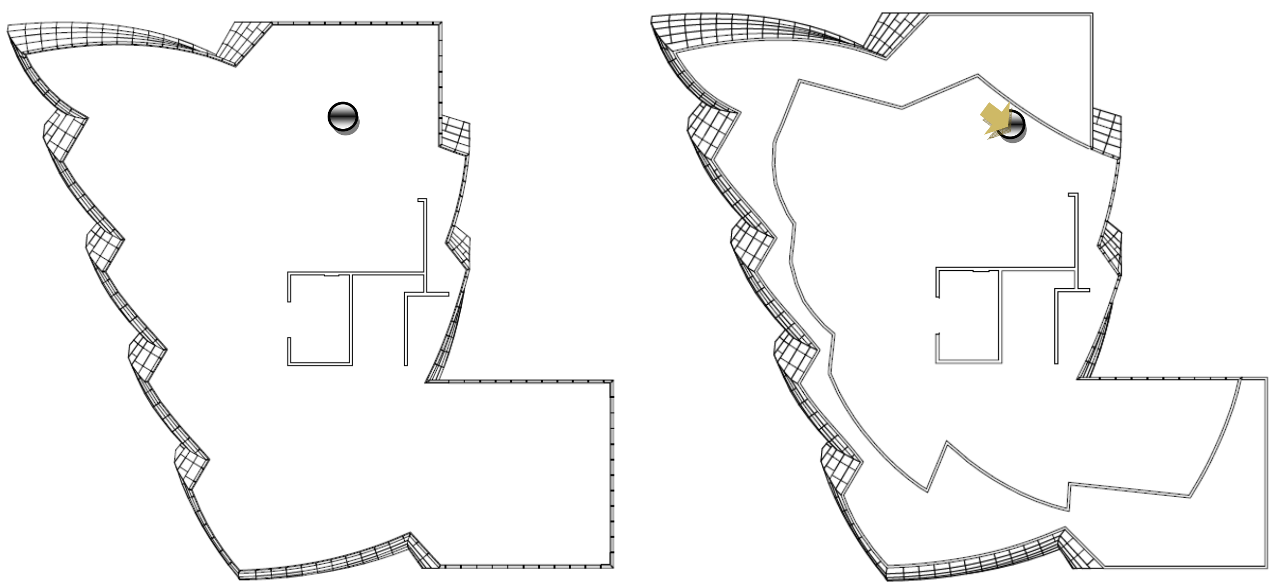


Figure 3: 5th Level vs. 6th Level Plans: illustrating the shift in column locations between floors
(Other columns not shown at the request of the owner)

Figure 3 on the previous page shows the column layout for the fifth and sixth floor plans. The black circles illustrate just one instance of the relocation of columns at the sixth floor. The distance between the two columns at this location is about 8'-0" long. At the sixth floor, the building setbacks become more distinct and, therefore, the columns begin to slope much more significantly in an effort to keep the columns along the perimeter and out of the way of the open office space.

Figure 4, shown to the right, effectively displays the coordination of the flat plate slab and the circular columns along the perimeter.

Figure 4: Photo taken during construction of superstructure

Lateral System

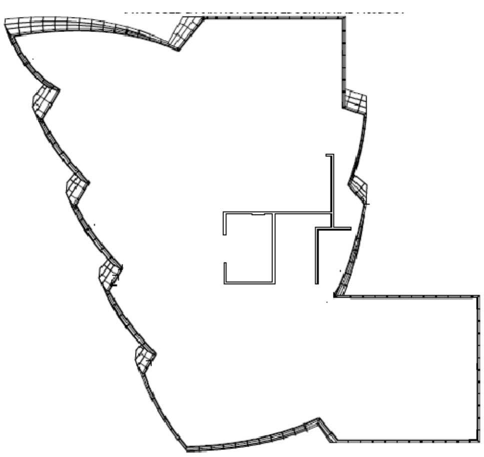


Figure 5: Typical shear wall layout (4th floor)

The columns carry the gravity loads while the shear walls, that encase the elevator and stair core, carry the lateral forces. These shear walls tend to be between 12" and 14" thick. They are reinforced by #4's at 12" in the vertical and horizontal directions. This core, with numerous shear walls acting in each direction, works together with the reinforced slab to carry wind and seismic lateral loads. The shear walls typically span from the cellar level up to the roof. Figure 5, to the left, shows the basic layout for shear walls. In addition to this shear wall core, the slab acts as a diaphragm in order to help distribute lateral loads. This is seemingly necessary because the shear wall core is so concentrated and would probably not be effective without the help of the slab to distribute loads across the entire plan.

Roof System

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

Figure 6: Photo of roof-top, showing mechanical and window-washing equipment

Codes & Design Standards

Applied to original design:

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

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

Substituted for thesis analysis:

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

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

Material Strength Requirement Summary:

Cast-in-place Concrete

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

Structural Steel

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

Masonry

- Compressive strength: 1500 psi
- Reinforcing: 60 ksi

Required Loads

Building live loads were determined by referencing ASCE 7 (to be used for this analysis) and comparing that to both the loads used as specified by the designer and the loads according to the New York City Building Code. Table 1 below outlines these findings.

LOAD DESCRIPTION	LOCATION	DESIGN LOAD	ASCE 7-05 LOAD	NYC BUILDING CODE 08
Parking	Cellar	100	40	40
Stairs	Cellar to 11^{th} floors	100	100	100
Elevator Lobby	Cellar to 9^{th} floors	100	100	100
Mechanical	Cellar, 2^{nd} to 11^{th} floors	150	125	-
Main Retail/Assembly	Ground floor	100	100	100
Ramps	Ground floor	100	250 (vehicular driveways)	--
Sidewalks	Ground floor	600	250	600
Entry	Ground floor	100	100	100
Loading Dock	Ground floor	150	--	--
Loading/Corridor	Ground floor	100	100	100
Corridor	Ground floor	100	100	100
Garden	Ground floor	100	100	100
Planter	Ground floor	20	--	--
Exterior	Ground floor	100	100	100
Office	2^{nd} to 10^{th} floors	60	50	50
Restrooms/Utility Rooms	2^{nd} to 10^{th} floors	100	100	100
Terrace	6^{th} floor	60	100	100
Elevator Machine Room	11^{th} floor	150	300 lb (concentrated load)	300 lb (concentrated load)
Roof	Roof	30+ window washing**	20	20

**Weight of window washing equipment could not be determined.

Table 1: Building Live Loads

Building dead loads were provided by the design engineers, DeSimone Consulting Engineers. These loads, shown below in Table 2, were checked and determined to be adequate for use in each of the specified load descriptions. More information about these checks can be found in the Appendix. Due to the slab thickness changes from floor to floor, there are many different resulting dead loads for which to account.

LOAD DESCRIPTION	LOCATION	DESIGN DEAD LOAD	DESIGN SUPER-IMPOSED DL
Parking	Cellar	300	20
Stairs	Cellar	300	25
Elevator Lobby	Cellar	300	25
Mechanical	Cellar	300	25
Ramp	Cellar	300	25
Main Retail/Assembly	Ground floor	150	70
Ramps	Ground floor	150	50
Sidewalks	Ground floor	150	350
Entry	Ground floor	150 150 175	160 250 300
Loading Dock	Ground floor	175	275
Loading/Corridor	Ground floor	200	20
Corridor	Ground floor	150	170
Garden	Ground floor	175 150	500 250
Planter	Ground floor	150	500
Exterior	Ground floor	150	120
Office	2nd to 5th floors	150	20
Lobby	2nd to 5th floors	125	50
Mechanical	2nd to 5th floors	150	20
Services	2nd to 5th floors	150	35
Stairs	2nd to 5th floors	150	25
Office	6th floor	300	20
Lobby	6th floor	300	50

Mechanical	6th floor	300	20
Services	6th floor	300	30
Stairs	6th floor	300	25
Terrace	6th floor	300	50
Office	7th to 10th floors	175	20
Lobby	7th to 10th floors	175	50
Mechanical	7th to 10th floors	175	20
Services	7th to 10th floors	175	35
Stairs	7th to 10th floors	150	25
Mechanical	11th floor	175	50
Elevator Machine Room	11th floor	175	20
Stairs	11th floor	175	25
Roof	Roof	175	50

Table 2: Building Dead Loads

Analysis & Conclusions

Wind Load Summary

Because the IAC Headquarters building is located in New York City, the NYC Building Code governed the structural design. During the time that this building was designed, the wind pressure was designed the same for all buildings in the city, regardless of location and surrounding buildings, etc. It consisted of only 20 psf for the first 100 feet and 25 psf for 100 to 300 feet. Effective July 2008, the New York City Building Code was changed to adopt more of the concepts from IBC, as well as the ASCE 7. For this analysis, however, ASCE-7 was used. Figure 5 illustrated the design pressure for the IAC building.

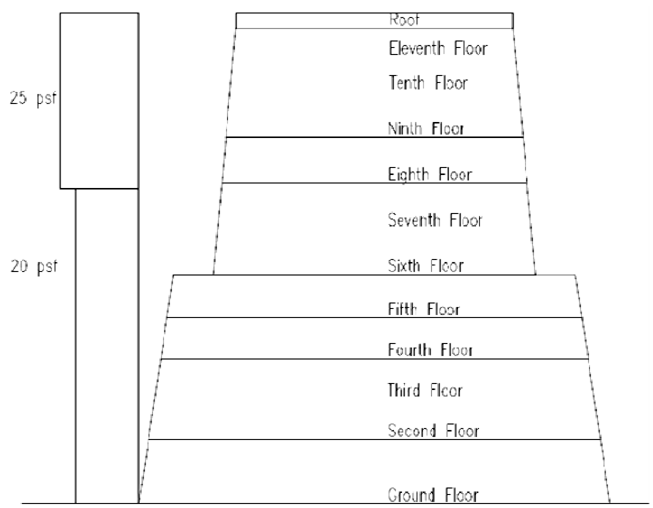


Figure 7: Wind Diagram using NYC Building Code

Comparing the above to Figures 7 and 8, which show the wind pressures and story forces using ASCE 7, it is evident that the ASCE 7 approach is much more detailed.

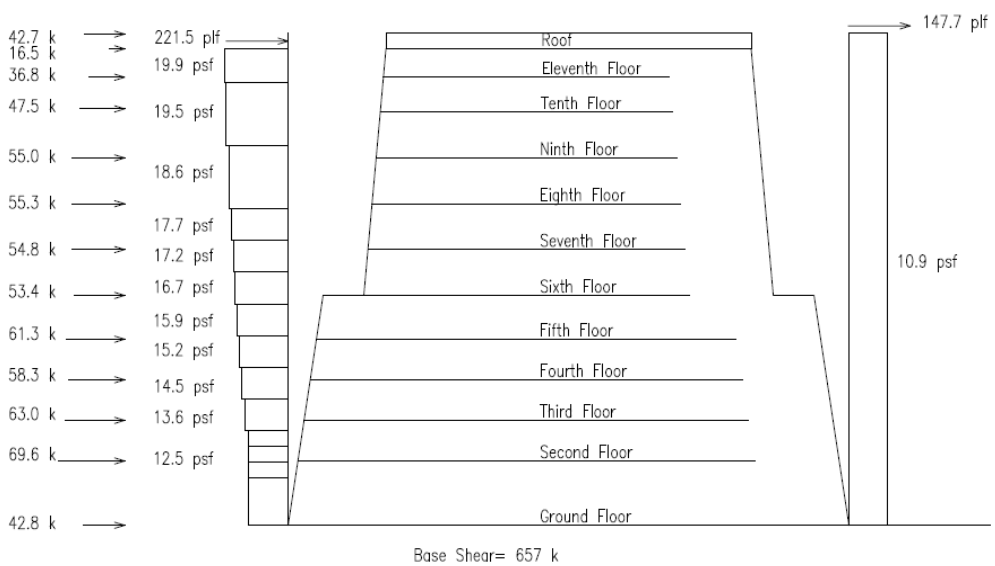


Figure 8: Wind Diagram using ASCE7- In East/West wind direction

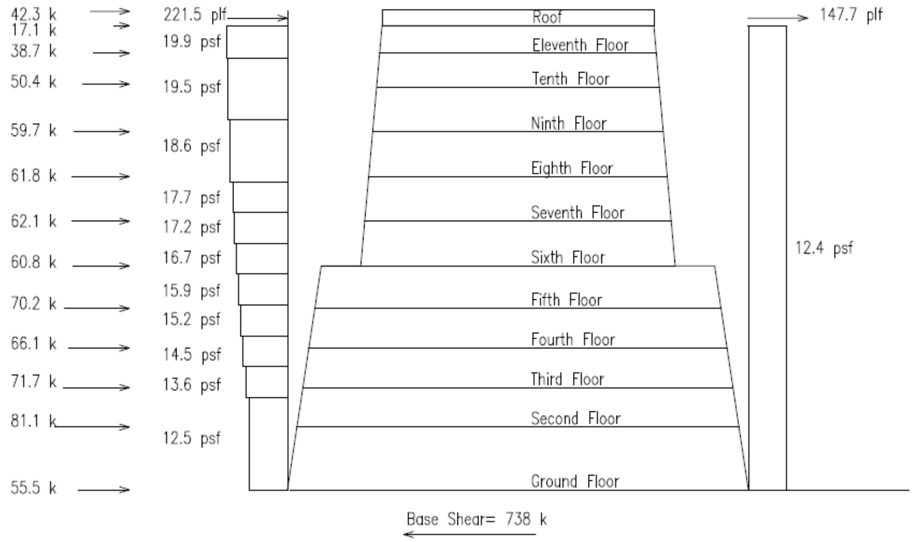


Figure 9: Wind Diagram using ASCE7- In North/South wind direction

It is not surprising that the values for wind pressures vary from the very standardized version specified in the NYC Building Code. The wind pressure at the top of the building is about 20% less than the initial design load. Because most buildings in NYC used the simplified wind analysis approach specified in the NYC Building Code, it is likely that a significant safety factor is incorporated into the design pressures; thus, the 20% lower wind pressure at the top is to be expected.

It is also important to note that the IAC building needed to conduct a wind tunnel test, due to its irregular shape and location. In order to use the Method 2 of ASCE 7 for wind design, simplifications had to be made to the façade. For instance, the procedure outlined in the Appendix for determining the wind pressures did not account for the curves of the building's façade. Instead, the footprint was essentially simplified to a rectangle with the dimensions determined by the longest width and length of the building, as shown in Figure 10 with the dashed line.

In the future, for a more in-depth analysis of the wind pressures, it may be necessary to analyze wind when it acts along the axis as shown below in Figure 10. This is because it is the longest and seemingly most critical condition. In addition, only windward and leeward conditions were considered in this analysis. Further analysis should also consider roof suction, causing possible uplift.

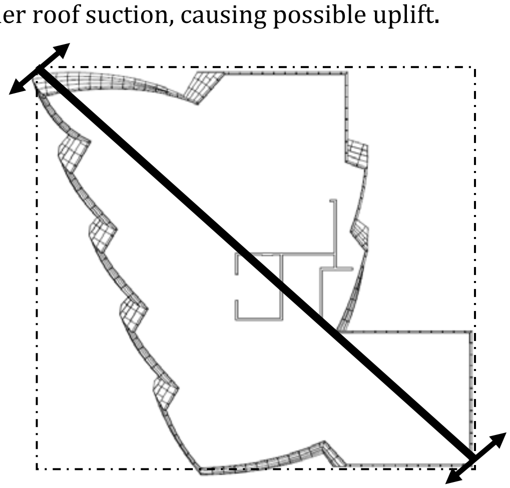


Figure 10: Plan showing location critical to wind pressures & simplified shape for design

Refer to Tables 3 through 5 for design forces, shears, and assumptions for wind using ASCE 7. For more detailed calculations, refer to the appendix.

Floor	Height Above Ground (ft)	Floor Height (ft)	Forces (k) N/S	Forces (k) E/W	Story Shear (k) N/S	Story Shear (k) E/W
1	0.00	20.33	55.45803	42.83604	737.614	656.9884
2	20.33	12.84	81.13745	69.6473934	682.1559	614.1524
3	33.17	12.83	71.6464	62.9956833	601.0185	544.505
4	46.00	12.83	66.10123	58.2564333	529.3721	481.5093
5	58.83	13.92	70.21972	61.25235	463.2709	423.2528
6	72.75	14.42	60.81644	53.3919	393.0511	362.0005
7	87.17	14.58	62.13139	54.8042063	332.2347	308.6086
8	101.75	14.42	61.78203	55.2712938	270.1033	253.8044
9	116.17	14.58	59.7399	55.0378938	208.3213	198.5331
10	130.75	11.00	50.4339	47.481	148.5814	143.4952
11	141.75	9.00	38.73673	36.776	98.1475	96.0142
Roof	150.75	5.00	17.11496	16.4934	59.41076	59.2382
Parapet	155.75	0.00	42.2958	42.7448	42.2958	42.7448
*** Effects from cellar not taken into account in wind/seismic determinations for this analysis**						

Table 3: Wind design forces and shears

Design Category	II	**G**	0.85
V	110	**q_p**	30.02
K_d	0.85	**GC_{pn}**	1.5 windward
I	1		-1 leeward
Exposure Category	B	**GC_{pi}**	0.18 windward
K_{zt}	1		-0.18 leeward
n_1	0.69		
Fully enclosed & Rigid Building (see Appendix for additional information)**			

Location	**Height above ground level, z (ft)**	**K_z**	**q (psf)**
Windward	155.75*	1.1215	29.52865
	150.75	1.11	29.22586
	140	1.09	28.69926
	120	1.04	27.38278
	100	0.99	26.0663
	90	0.96	25.27642
	80	0.93	24.48653
	70	0.89	23.43334
	60	0.85	22.38016
	50	0.81	21.32698
	40	0.76	20.0105
	30	0.7	18.43072
	25	0.7	18.43072
	20	0.7	18.43072
	15	0.7	18.43072
Leeward	All		29.23
*** Top of parapet**			

Tables 4 & 5: Wind design criteria

Seismic Summary

Latitude	40.745179
Longitude	-74.007654
S_S	0.363
S_1	0.07
Soil Class E (Soft Clay Soil)	
F_a	2.14
F_v	3.5
S_{MS}	0.77682
S_{M1}	0.245
S_{DS}	0.518
S_{D1}	0.163
Seismic Design Category	D (from S_{DS}) or C (from S_{D1})
T_s	0.315
R	5
T_a	0.86
C_t	0.02
x	0.75
hn	150.67
Importance Factor	1
T_L	6
C_s	0.03797998
k	2
$V=C_sW$	1831.368902

Tables 6: Seismic design criteria

The IAC Headquarters was designed seismically using the NYC Building Code. This code uses different variables than those outlined in Table 6 and determined from ASCE 7. For instance, the soils in NYC are classified as S0 (for buildings' foundations supported directly on hard rock) through S4 (for buildings with soft, unsuitable soil). The soil at this site is S3. This seems to correspond to soil class E in ASCE 7, because of its soft soil profile. Correlating between these two classifications is not always very clear. In order to be conservative, however, the worst soil condition was analyzed. This resulted in a base shear of 1831 kips, which is 30% over the designed shear of 1400 kips. The reason was initially thought to be due to the soil classification. However, due to further investigation, it became clear that the shear was initially designed at 1750 kips using standard analysis. It was later lowered to 1400 kips following a site-specific response spectra analysis.

The building weight was designed as 46,836 kips, which is 3% less than the weight of 48,219 kip that was determined for this analysis. However, a detailed weight was determined and is shown in the Appendix. For the building weight calculations, slabs, columns, curtainwall, and shear walls were taken into account. By incorporating the superimposed dead load of each space, it should have accounted for extra loads, such as mechanical or lighting/electrical.

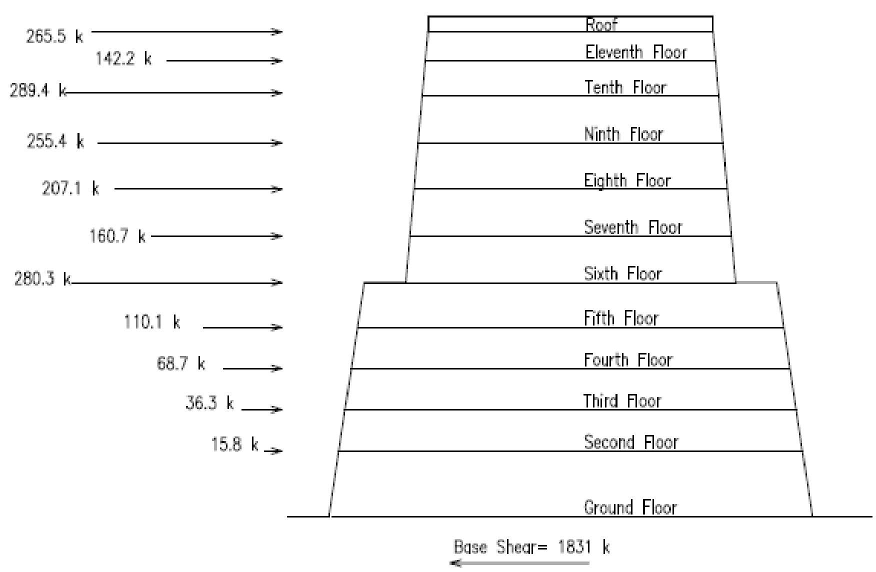


Figure 11: Seismic force diagram

Snow Loads

Snow loading was taken into consideration to determine if the roof design was sufficient to carry potential snow loads or drifting in the area. Table 7 below outlines the design criteria for snow loading on the roofing system at the IAC Headquarters. The roof and terrace are both warm roofs and are accessible. It is surprising that the main roof did not have a higher live load given its accessibility; it was designed as 30 pounds per square foot plus the weight of the window washing machine. However, its design load still outweighs the potential snow load by nearly two times and because of the flat roof, snow drifting should not be an issue.

Flat Roof		
p_g	25	lb/ft^2
C_e	1	(Terrain Category B det. in wind analysis)
		Assuming partially exposed roof
C_t	1	(Table 7-3)
I	1	(Category II)
p_f	17.5	Eqn 7-1
Warm Roof		
C_s	1	Fig 2-a (roof slope=0deg)
$p_s=C_s*p_f$	**17.5**	**psf**

Tables 7: Snow load design criteria

Other Loads:

There are a number of other important loading conditions that will need to be taken into account and will require further research. For instance, the concrete pressure slab in the basement should be checked to determine if it is sufficient to overcome the hydraulic uplift forces. Similarly, the walls of the cellar are subject to lateral soil pressure that should be considered. Analysis of the parking ramp, which goes from grade to below grade, should be taken into account in the future.

Spot Check of Typical Gravity Load Areas

The structural design of the IAC building does not follow any grid-like, uniform layout. Therefore, finding a typical concrete panel that can be modeled and spot-checked by hand requires a number of simplifications. Efforts were made using the worst case scenario, while making the layout uniform enough to be able to utilize the Direct Design Method for analyzing the flat plate slabs. This involved moving the columns in the vicinity into a more grid-like fashion to form a rectilinear panel to analyze. The area chosen for the concrete slab spot check is shown below in Figure 12. It was chosen because the columns are somewhat evenly spaced and because this floor plan is consistent on both the fourth and fifth floors.

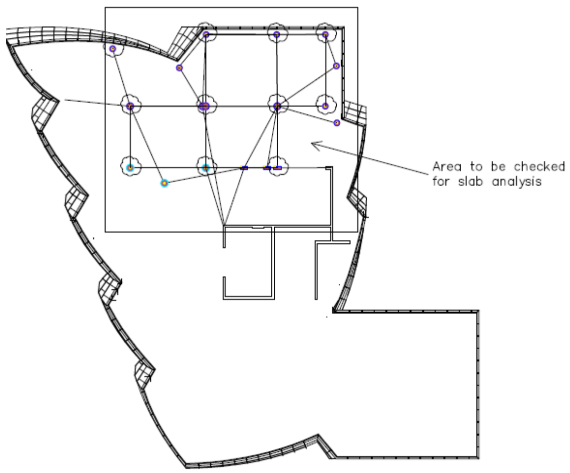


Figure 12: Spot check location on 4th floor

Upon using the Direct Design Method, the area of steel required was compared to the amount of steel specified on the drawings. The thickness of the slab was determined adequate and it was found that the amount of steel actually put into the slab was typically much higher than what was determined using this procedure. This was not the case, however for the positive moments in the middle of the spans. The results of this analysis showed that the amount of required steel calculated in the middle span was consistently more than what was actually put into the design. A possible reason for this is that the spans were adjusted to make it more regular. Certain columns were either moved or ignored entirely, so it is not surprising that the results differed.

Punching shear and wide beam action (due to the flat plate design) were then taken into account to determine if shear controlled in the slab. Both passed for shear.

The extra steel at the supports could be accounted for because of serviceability requirements. Deflection may control in the slab, causing the need for extra reinforcing. In addition, the tendency of the building to want to twist could be dealt with in part by additional slab reinforcement.

For the second spot check, an interior column on the fifth floor, that was studied as part of the Direct Design Method for slabs, was analyzed for axial loading. Simplified assumptions were made for this procedure. Because there is a transfer diaphragm immediately above the analyzed column, not all of the load from the upper column would be transferred to the column in question. In order to avoid transferring the loads through the diaphragm and accounting for the sloping columns, it was assumed conservatively that loads from the upper columns are carried to the column on the fifth floor. In addition, all columns were assumed upright. The results of this analysis showed that the design was inadequate. Potential reasons for this could be that too many conservative assumptions were made when calculating the loading that the final load on the column ended up being much higher than what it was initially designed for.

Appendix A: Calculations

Wind Loading

Table A-1: Calculated wind pressures in East/West direction

Location	Height above ground level, z (ft)	q (psf) $.00256 \cdot V^2 \cdot I \cdot K_{zt} \cdot K_z \cdot K_d$	External pressure qGC_p (psf)	Internal pressure $q_h(GC_{pi})$ (psf)	Net pressure p (psf)	
					pos GC_{pi}	neg GC_{pi}
Windward	155.75*	29.53	221.46			
	150.75	29.23	19.87	5.32	14.55	25.19
	140	28.70	19.52	5.32	14.20	24.84
	120	27.38	18.62	5.32	13.30	23.94
	100	26.07	17.73	5.32	12.41	23.05
	90	25.28	17.19	5.32	11.87	22.51
	80	24.49	16.65	5.32	11.33	21.97
	70	23.43	15.93	5.32	10.61	21.25
	60	22.38	15.22	5.32	9.90	20.54
	50	21.33	14.50	5.32	9.18	19.82
	40	20.01	13.61	5.32	8.29	18.93
	30	18.43	12.53	5.32	7.21	17.85
	25	18.43	12.53	5.32	7.21	17.85
	20	18.43	12.53	5.32	7.21	17.85
	15	18.43	12.53	5.32	7.21	17.85
Leeward	All	29.23	-10.93	5.32	-16.25	-5.61

Table A-2: Calculated wind pressures in North/South direction

Location	Height above ground level, z (ft)	q (psf)	External pressure qGC_p (psf)	Internal pressure q_h (GC_{pi}) (psf)	Net pressure p (psf)	
					pos GC_{pi}	neg GC_{pi}
Windward						
	155.75**	29.53	221.46			
	150.75	29.23	19.87	5.32	14.55	25.19
	140	28.70	19.52	5.32	14.20	24.84
	120	27.38	18.62	5.32	13.30	23.94
	100	26.07	17.73	5.32	12.41	23.05
	90	25.28	17.19	5.32	11.87	22.51
	80	24.49	16.65	5.32	11.33	21.97
	70	23.43	15.93	5.32	10.61	21.25
	60	22.38	15.22	5.32	9.90	20.54
	50	21.33	14.50	5.32	9.18	19.82
	40	20.01	13.61	5.32	8.29	18.93
	30	18.43	12.53	5.32	7.21	17.85
	25	18.43	12.53	5.32	7.21	17.85
	20	18.43	12.53	5.32	7.21	17.85
	15	18.43	12.53	5.32	7.21	17.85
Leeward	All	29.23	-12.42	5.32	-17.74	-7.10

** For parapet design, see figure below.

	Parapet			
Windward	155.75	29.53	221.48	plf (GCpn=1.5)*height of parapet*q
Leeward	155.75	29.53	-147.65	plf (GCpn=-1)*height of parapet*q

Table A-3: Coefficients used to calculate wind loading

V	**110**	**mph**			
K_d	0.85				
I	1				
Exposure Category	B	(Category II according to Table 1-1)			
K_{zt}	1				
K_z, K_h	(see table below)				
q_h	(see table below)				
n_1	3.23				
G	0.85				
q_p	30.02	Eqn 6-15 (K_h~= 1.14 @ 162'-8"			
GC_{pn}	1.5	windward			
	-1	leeward			
p_p	45.023616	windward			
	-30.015744	leeward			
GC_{pi}	0.18	(for an Enclosed Building)	(5'-0" high parapet)		
	-0.18				

Table A-4: Rigid/flexible building check

To check if flexible or rigid:	**Found to be rigid**	
Used eqn C6-16 from ASCE7		
n_1= 385(c_w)^.5/H		
c_w= 100/A_b Σ(H/hi)^2*[Ai/(1+.83(hi/Di)^2)]		
	n1	3.236393474
In N-S direction:	**cw**	1.764068
Wall1		
Area	9.5	
height	158	
cw1/100*Ab	0.041199412	Σ(H/hi)^2*[Ai/(1+.83(hi/Di)^2)]
Wall2		
Area	9.5	
height	158	
cw2/100*Ab	0.041199412	
Wall3		
Area	36.1667	
height	158	
cw3/100*Ab	1.603062881	
Wall4		
Area	22.5	
height	158	
cw4/100*Ab	0.53662614	
Wall5		
Area	6.13	
height	33	
cw5/100*Ab	20.00811381	
Wall6		
Area	24.5	
height	130	
cw6/100*Ab	1.4851251	
Wall7		
Area	6.375	
height	13	
Cw7/100*Ab	505.5051053	

**Initial assumption was that it was rigid due to the slab diaphragms that also carry lateral loads, in addition to the fact that it is a concrete building with only 11 stories and 150' tall. Calculations confirmed this assumption.

Table A-5: Kz and Cp factors

eight above ground level, z (ft)	**K_z**		**Cp**				
			Windward	Leeward			
				L/B=0-1	L/B=2	L/B>4	
155.75	1.1215		0.8	-0.5	-0.3	0.2	
150.75	1.11						
140	1.09						
120	1.04		In N-S direction		Cp		
100	0.99			L/B= .766	-0.5		
90	0.96		In E-W direction				
80	0.93			L/B= 1.3	-0.44		
70	0.89						
60	0.85						
50	0.81						
40	0.76						
30	0.7						
25	0.7						
20	0.7						
15	0.7						

Table A-6: Wind Story Forces & Shears

Floor	Height Above Ground (ft)	Floor Height (ft)	Forces (k)		Story Shear (k)		
			N/S	E/W	N/S	E/W	
1	0.00	20.33	55.458027	42.83604	1955.296	1823.745	
2	20.33	12.84	81.1374503	69.64739	1899.838	1780.909	
3	33.17	12.83	71.6464	62.99568	1818.7	1711.261	
4	46.00	12.83	66.1012333	58.25643	1747.054	1648.266	
5	58.83	13.92	70.2197188	61.25235	1680.953	1590.009	
6	72.75	14.42	60.8164375	53.3919	1610.733	1528.757	
7	87.17	14.58	62.1313875	54.80421	1549.917	1475.365	
8	101.75	14.42	61.782025	55.27129	1487.785	1420.561	
9	116.17	14.58	59.7399	55.03789	1426.003	1365.289	
10	130.75	11.00	50.4339	47.481	1366.263	1310.252	
11	141.75	9.00	38.7367333	36.776	1315.829	1262.771	
Roof	150.75	5.00	17.1149625	16.4934	1277.093	1225.995	
Parapet	155.75	0.00	42.2958	42.7448	1259.978	1209.501	
* Affects from cellar not taken into account in wind/seismic for this analysis							

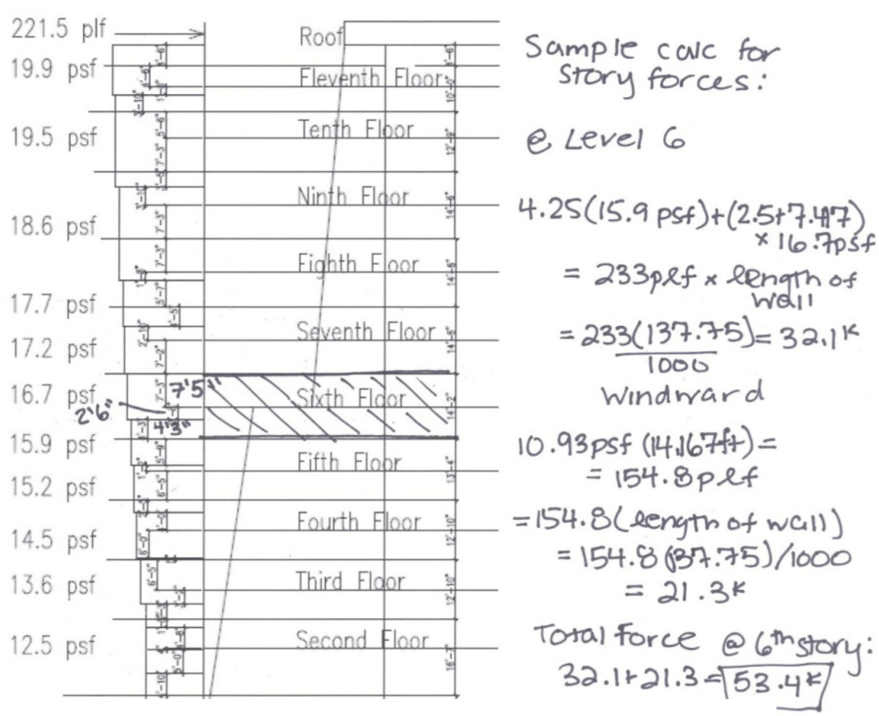


Figure A-1: Sample Story Force Calculation

Seismic Loading

Table A-7: Seismic Coefficients using Equivalent Lateral Force Procedure

Latitude	40.745179				
Longitude	-74.007654				
S_s	0.363				
S_1	0.07				
Soil Class E (Soft Clay Soil)					
F_a	2.14	(interpolated from Table 11.4-1)			
F_v	3.5	(from Table 11.4-2)			
S_{MS}	0.77682				
S_{M1}	0.245				
S_{DS}	0.518				
S_{D1}	0.163				
Seismic Design Category	D (from S_{DS}) or C (from S_{D1})	Use SDC= D			
T_s	0.315				
R	5	(assumption from table 12.2-1)			
T_a	0.86	(section 12.8.2.1)			
C_t	0.02	(in both directions)			
x	0.75				
hn	150.67				
Importance Factor	1				
T_L	6	(Figure 22-15)			
C_s	0.03797998	N-S & E-W direction	C_s=Sds/(R/I)	0.103576	
k	2	(.5<Ta<2.5)	Cs=Sd1*Tl/[T^2*R/I]	0.264945	
V=C_sW	1831.368902 k				

Table A-8: Building Weight Calculations

Floor	Floor Area	Floor Dead Load	Floor weight	h/2 above	h/2 below	Column area above	Column area below	Column weight= height*area*150pcf	Curtainwall (estimated length along perimeter)	Curtainwall weight (height*length*15psf)	Shear Wall Length (ft)	Shear Wall Thickness (in)	ShearWall Weight (L*t*h*150)	
Cellar														
1st				10.17	7.25	102.6254	102.62536	286428.1244	762.25	199135.9069	59.1	14	674033.341	Above
Retail/Assembly	17902	220.00	3938440.00			14.72622	14.7262156				3.87777	16		
Loading/Corridor&Lobby	1495	220.00	328900.00			37.83001	37.8300115				85.5	12		
Loading Dock	980	450.00	441000.00			4.25	4.25							
Entry	1570	475.00	745750.00			4.72222	4.72222222							
Ramp	200	200.00	40000.00			8.726646	8.72664626							
Planter	309	650.00	200850.00			7.068583	7.06858347				W1	1159597.37		
Exterior	550	270.00	148500.00			7.875798	7.87579825							
Garden	2023	675.00	1365525.00			Above=	Below=							
Entry	666	310.00	206460.00			187.8248	187.824837	<-- Not to be included in the building's total weight for seismic!						
Sidewalk	2800	500.00	1400000.00											
Entry	650	400.00	260000.00											
Garden	592	400.00	236800.00											
Stair	68	175.00	11900.00											
Stair	195	160.00	31200.00											
		Total	9355325.00	<-- Not to be included in the building's total weight for seismic!										
2nd				6.42	10.17	111.1775	102.6254	461485.2836	724.25	180155.1958	187	12	179987.033	Above
Office	19256	170.00	3273520.00			4.908742	14.72622						674033.34	Below
Lobby	250	175.00	43750.00			37.83001	37.83001							
Services	820	185.00	151700.00			4.25	4.25							
Stair	502	175.00	87850.00			4.72222	3.777777							
Mechanical	550	170.00	93500.00			7.875798	8.726646				W2	5145980.85		
		Total	3650320.00			6.305002	7.068583							
						6.305002	7.875798							
						Below=	Above=							
						183.3743	186.880434							
3rd				6.42	6.42	102.6254	106.901457	357903.4769	723.75	139321.5131	187	12	179987.033	Above
Office	18944	170.00	3220480.00			14.72622	9.817481						179987.03	Below
Lobby	250	175.00	43750.00			37.83001	37.83001							
Services	820	185.00	151700.00			4.25	3.77777							
Stair	502	175.00	87850.00			3.77777	3.777777							
Mechanical	550	170.00	93500.00			8.726646	8.726646				W3	4454479.05		
		Total	3597280.00			7.068583	7.068583							
						7.875798	7.068583							
						Below=	Above=							
						186.8804	184.968307							
4th				6.42	6.42	106.9015	106.901457	355513.6308	722	139057.2	187	12	179987.03	Above
Office	18505	170.00	3145850.00			9.817481	9.817481						179987.03	Below
Lobby	250	175.00	43750.00			37.83001	37.83001							
Services	820	185.00	151700.00			3.77777	3.77777							
Stair	502	175.00	87850.00			3.777777	3.777777							
Mechanical	550	170.00	93500.00			8.726646	8.726646				W4	4377194.89		
		Total	3522650.00			7.068583	6.305002							
						7.068583	7.068583							
						Below=	Above=							
						184.9683	184.204726							
5th				6.42	6.42	106.9015	89.797229	359211.2821	721	138864.6	187	12	179987.03	Above
Office	17968	170.00	3054560.00			9.817481	19.634959						179987.03	Below
Lobby	250	175.00	43750.00			37.83001	37.83001							
Services	820	185.00	151700.00			3.77777	3.77777							
Stair	502	175.00	87850.00			3.777777	3.777777							
Mechanical	550	170.00	93500.00			8.726646	8.726646							
		Total	3431360.00			6.305002	12.610004				W5	4289409.94		
						7.068583	7.068583							
							5.58505							
						Below=	Above=							
						184.2047	188.808028							
6th				7.25	6.42	89.79723	26.1799388	260482.9656	538	114265.2	171.25	12	186234.375	Above
Office	10089	320.00	3228480.00			19.63496	10.559242		below:				179987.03	Below
Lobby	250	350.00	87500.00			37.83001	9.42477796		579					
Services	820	330.00	270600.00			3.77777	4.276057							
Stair	502	325.00	163150.00			3.777777	2.66666667							
Mechanical	550	320.00	176000.00			8.726646	4.16666667				W6	7160799.57		
Terrace	7126	350.00	2494100.00			12.61								
		Total	6419830.00			7.068583								
						5.58505								
						Below=	Above=							
						188.808	57.273349							

Table A-8 (continued): Building Weight Calculations

7th				7.25	7.25	26.17994	32.72492	117153.9169	519.5	112991.25	171.25	12	186234.375	Above
Office	9332	195.00	1819740.00			10.55924	5.27964						186234.38	Below
Lobby	250	225.00	56250.00			9.424778	3.141578							
Services	820	210.00	172200.00			4.276057	3.14158							
Stair	548	175.00	95900.00			2.66667	2.66667				W7	2853953.92		
Mechanical	550	195.00	107250.00			4.166667	3.5							
		Total	2251340.00			Below=	Above=							
						57.27335	50.454388							
8th				7.25	7.25	32.72492	37.08824	108196.1558	501.5	109076.25	171.25	12	186234.375	Above
Office	8622	195.00	1681290.00			5.27964	2.63984						186234.38	Below
Lobby	250	225.00	56250.00			3.141578	0							
Services	820	210.00	172200.00			3.14158	3.14158							
Stair	548	175.00	95900.00			2.66667	2.66667				W8	2702631.16		
Mechanical	550	195.00	107250.00			3.5	3.5							
		Total	2112890.00			Below=	Above=							
						50.45439	49.03633							
9th				7.25	7.25	37.08824	34.90658	104281.4625	481	104617.5	170.25	12	185146.875	Above
Office	7907	195.00	1541865.00			2.63984	2.63984						186234.38	Below
Lobby	250	225.00	56250.00			0	0							
Services	820	210.00	172200.00			3.14158	3.14158							
Stair	548	175.00	95900.00			2.66667	2.66667				W9	2553745.22		
Mechanical	550	195.00	107250.00			3.5	3.5							
		Total	1973465.00			Below=	Above=							
						49.03633	46.85467							
10th				5.5	7.25	34.90658	30.54326	84864.44588	460.5	88070.625	154.25	12	127256.25	Above
Mechanical	7536	195.00	1469520.00			2.63984	2.63984						185146.88	Below
Services	900	210.00	189000.00			0	0							
Stair	537	175.00	93975.00			3.14158	3.14158							
Office	250	195.00	48750.00			2.66667	2.66667				W10	2286583.20		
		Total	1801245.00			3.5	3.5							
						Below=	Above=							
						46.85467	42.49135							
11th				4.5	5.5	30.54326	30.54326	58649.52225	447.5	30206.25	122.75	12	82856.25	Above
Elevator Machine	650	195.00	126750			2.63984	2.63984						127256.25	Below
Stairs	225	195.00	43875			0	0							
Mechanical	2040	225.00	459000			3.14158	3.14158							
		Total	629625			2.66667								
						3.5					W11	955644.4923		
						Below=	Above=							
						42.49135	36.32468							
Roof	6397	225.00	1439325		4.5		30.54326	24519.159	465.5	31421.25			82856.25	Below
							2.63984							
							0							
		Total	1439325				3.14158				WR	1578121.659		
							Above=							
							36.32468							
		TOTAL FROM SLABS	39555030.00	LBS			**TOTAL FROM COLUMNS**	2554170.267	**TOTAL FROM CURTAINWALL**	1387182.741	lbs	**TOTAL SHEARWALL WEIGHT**	4695887.95	
									TOTAL WEIGHT	48219.32	kips			

Snow Drift

Table A-9: Snow Drift coefficients

Flat Roof		
pg	25	lb/ft^2
Ce	1	(Terrain Category B det. in wind analysis)
		Assuming partially exposed roof
Ct	1	(Table 7-3)
I	1	(Category II)
pf=.7*CeCt*I*pg	17.5	Eqn 7-1
Warm Roof (section 7.4.1)		
Cs	1	Fig 2-a (roof slope=0deg)
ps=Cs*pf	17.5	psf
Drifts on Lower Roof of Structure		
g=.13pg+14	17.25	< 30 (snow density)
hb=ps/g	1.014493	
hc	83	Top of parapet= 155.75
		Level 6+hb= 72.75
hc/hb=	81.81429	
hd	3.95	(lu=148.75)
3/4hd=	2.9625	drift height
*** Initial checks for snow loading were considered.**		
Because roof is designed for access, the snow drift		
should be insignificant.		

Sample Dead Load Checks

1st through 5th Floors; Concrete→ 150 pcf (12”/(12”/1’)= 150 psf

6th Floor; Concrete→ 150 pcf (24”/ (12”/1’) = 300 psf

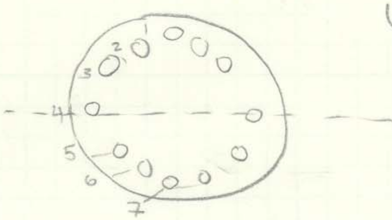
7th through Roof; Concrete → 150 pcf (14”/(12”/1’))=175 psf

Column Spot Check:

Column Check

Column- 34"ϕ , 12 #8's , #3 ties @ 16"

$\rightarrow$ .79 in^2 $\rightarrow$ 3/8" ϕ



Pure Axial Strength:

- $P_o = .85 f'_c A_c + A_s f_y = .85(5.95)\left(\left(\frac{34}{2}\right)^2 \pi - .79(12)\right) + 12(.79)(60) \Rightarrow$

$$\boxed{P_o = 5113^k}$$

BALANCED STRAIN

$\varepsilon_y = 60\text{ ksi}/29000 = \boxed{.0021}$ $\quad c = \frac{.003}{.003+.0021}(d)$

$d = 34" - 3/8" - \frac{1}{2}/2 - 1.5 = 31.9"$

$\leftarrow$ assumed clearcover

$c = \frac{.003}{.0051}(31.9) = 18.76"$

$\varepsilon_{s1} = \frac{.003}{18.76}(18.76 - 2.63) = .00258$

(see next page for cad to show d values)

$\varepsilon_{s2} = \frac{.003}{18.76}(18.76 - 4.55) = .00227$

$\varepsilon_{s3} = \frac{.003}{18.76}(18.76 - 9.8125) = .00143$

$\varepsilon_{s4} = \frac{.003}{18.76}(18.76 - 16.987) = .000284$

$\varepsilon_{s5} = \frac{.003}{18.76}(18.76 - 24.1875) = -.000868$

$\varepsilon_{s6} = \frac{.003}{18.76}(18.76 - 29.45) = -.00171$

$\varepsilon_{7} = \frac{.003}{18.76}(18.76 - 31.375) = -.00202$

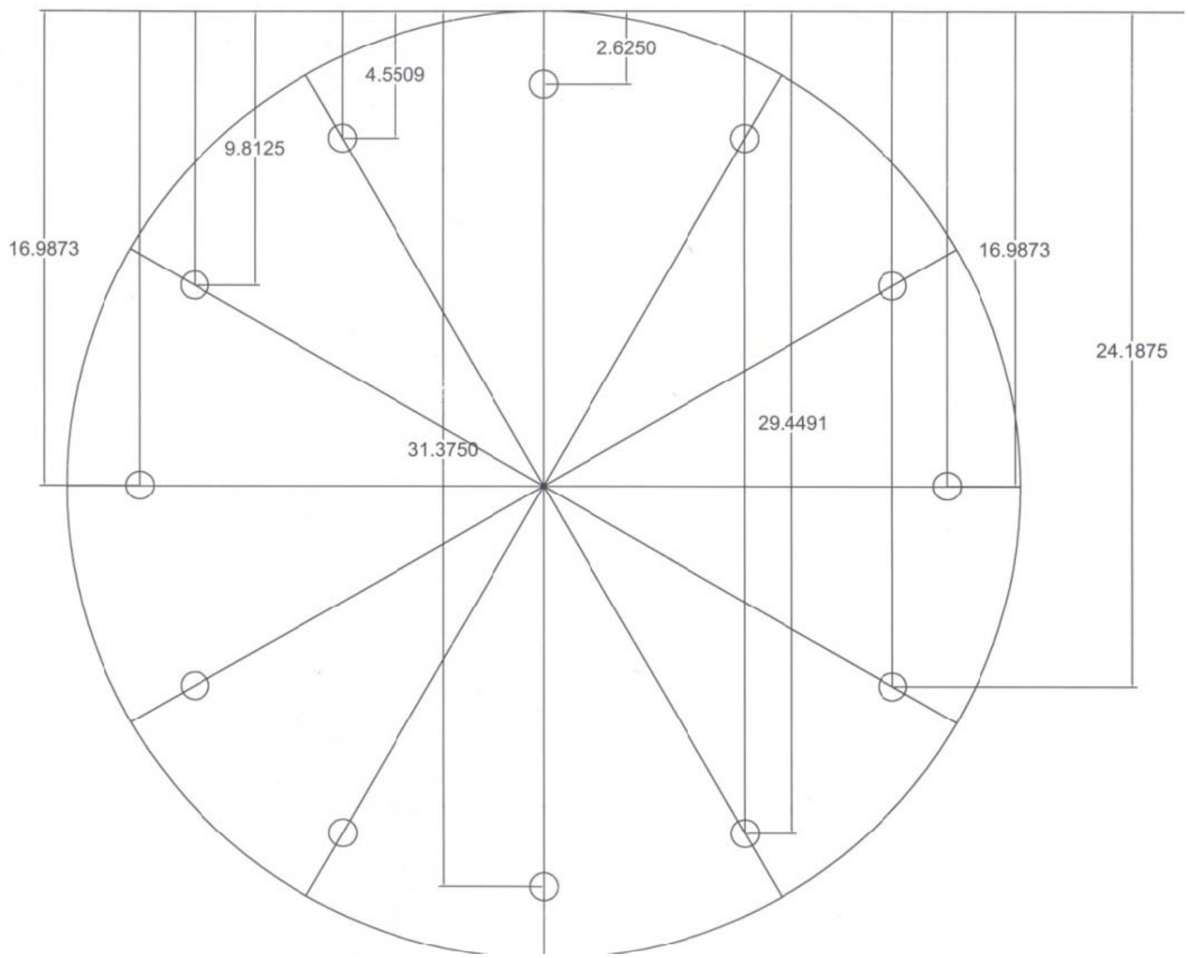


Figure A-2: 'D' values for column calculations

Level	DL	LL	1.2D+1.6L	Area (ft^2)	Length of curtainwall	Weight from curtainwall	Total Weight
6th	320	60	480	480	33.75	8608.275	239008.3
7th	195	60	330	465	33.75	8808.75	162258.8
8th	195	60	330	464	32	8352	161472
9th	195	60	330	389	32	8352	136722
10th	195	60	330	160	22	5064.84	57864.84
11th	-	-	-	-	27.75	4995	4995
Roof	225	30	318	181	26.75	2166.75	59724.75
							822 kips

Table A-10: Calculated Axial Load on Column

$f_{s1} = 60\ ksi$, $f_{s2} = 60\ ksi$, $f_{s3} = .00143(29000) = 41.5\ ksi$

$f_{s4} = 60\ ksi$, $f_{s5} = -.000868(29000) = -25.2\ ksi$

$f_{s6} = -.00171(29000) = -49.6\ ksi$

$f_{s7} = -.00202(29000) = -58.6\ ksi$

$$P_b = .85f'_c b\beta_1 c + 1f_{s1} + 2f_{s2} + 2f_{s3} + 2f_{s4} + 2f_{s5} + 2f_{s6} + 1f_{s7}$$

$$= .85(5.950)(34)(.75)(18.76) + 1(60) + 2(60) + 2(41.5) + 2(60) + 2(-25.2) + 2(-49.6) + 1(-58.6) = \boxed{2594K}$$

$$M_b = .85f'_c b\beta_1 c\left(\frac{h}{2} - \frac{\beta_1 c}{2}\right) = .85(5.95)(34)(.75) \times 18.76\left(\frac{34}{2} - \frac{.75(18.76)}{2}\right) = \boxed{2009\ ft\,K}$$

Pure Bending

$$\Sigma F = 0 \Rightarrow .85f'_c b\beta_1 c + 1f_{s1} + 2(f_{s2} + f_{s3} + f_{s4} + f_{s5} + f_{s6}) + 1f_{s7}$$

$f_{s1} = \frac{.003}{c}(c - d_1)29000$ ↳ $d_1 = 1.5 + 1/2 + 3/8 = 2.625"$

$f_{s2-7} = -60\ ksi$

$$\Sigma F \Rightarrow .85(5.95)(34)(.75)(c) + \frac{.003}{c}(c - 2.625)(29000) + 2(-60)(5) + 1(-60) \Rightarrow$$

$$660c = 129c^2 + 87c - 228.4$$

$-660c \qquad -660c$

$\boxed{c = 4.81"}$ $f_{s1} = 48\ ksi$

$$\varepsilon_{s2} = \frac{.003}{4.81}(4.81 - 4.55) = .000162 \text{ ok}$$

↳ Procedure continued using excel shown on next page

Fails → assume ε_{s1} + ε_{s2} don't yield

Table A-11: Stress/Strain Calculations for column spot check

bar	c	d	es					
2	4.81	4.55	0.000162					
3	4.81	9.8125	-0.00312					
4	4.81	16.9873	-0.00759					
5	4.81	24.1875	-0.01209					
6	4.81	29.4491	-0.01537					
7	4.81	31.375	-0.01657					
				# of bars	fs			
1	4.57	2.65	0.00126	1	36.5514223	524.51291		
2	4.57	4.55	1.31E-05	2	0.38074398	9.48052516		
3	4.57	9.8125	-0.00344	2	-60	-862.5		
4	4.57	16.9873	-0.00815	2	-60	0		
5	4.57	24.1875	-0.01288	2	-60	862.5		
6	4.57	29.4491	-0.01633	2	-60	1493.892		
7	4.57	31.375	-0.0176	1	-60	862.5		
					.85f'cbB1c(h/2-b1c/2)= 12012			
						14902.3854		
						1241.86545		
@ c=34"		d	es		Stress (ksi)	# of bars		
1	34	2.65	0.002766	80.21912	60	1	861	
2	34	4.55	0.002599	75.35735	60	2	1494	
3	34	9.8125	0.002134	61.89154	60	2	862.5	
4	34	16.9873	0.001501	43.5325	43.5	2	1.1049	
5	34	24.1875	0.000866	25.10846	25.1	2	-360.813	
6	34	29.4491	0.000402	11.64495	11.6	2	-288.819	
7	34	31.375	0.000232	6.716912	6.7	1	-96.3125	
					4851.9525	kips	18635.62	
							21108.28	1759.024
@et=.005						# of bars		
1	11.8	2.65	0.002326	67.46186	60	1	60	861
2	11.8	4.55	0.001843	53.45339	53.5	2	107	1332.15
3	11.8	9.8125	0.000505	14.6536	14.7	2	29.4	211.3125
4	11.8	16.9873	-0.00132	-38.2453	-38.2	2	-76.4	0
5	11.8	24.1875	-0.00315	-91.3316	-60	2	-120	862.5
6	11.8	29.4491	-0.00449	-130.125	-60	2	-120	1493.892
7	11.8	31.375	-0.00498	-144.324	-60	1	-60	862.5
					4204.8525		1521.802	
							7145.156	in-kips
							595.4297	ft-kips

$f_{s1} = \frac{.003}{c}(c-2.625)(29000)$, $f_{s2} = \frac{.003}{c}(c-4.55)(E)$

$f_{s3} \rightarrow f_{s7} = -60\text{ksi}$

$\Sigma F = 0 \Rightarrow \frac{.003}{c}(c-2.625)(29000) + \frac{.003}{c}(c-4.55)(29000)$
$+ 2(-60)(\underset{(4)}{f_{s3} \text{ thru } f_{s6}}) - 60 + .85 f'_c b \beta_1 c = 0$

$-228.4 + 87c - 395.9 + 87c - 480c - 60c$
$+ .85(5.95)(34)(.75)(c^2) = 0$

$128.9c^2 - 453c - 624.3 = 0$

$\boxed{c = 4.57}$

(check on excel sheet – ok✓)

$M_o = .85 f'_c b \beta_1 c\left(\frac{h}{2} - \frac{\beta_1 c}{2}\right) + f_{s1}(17-d_1) + 2f_{s2}(17-d_2)$
$+ 2f_{s3}(17-d_3) \ldots + 1f_{s7}(17-d_7)$
$=$ (done in excel✓)

$\boxed{M_o = 1241 \text{ ft-K}}$

– $c = h = 34"$

$\varepsilon_{s1} = \frac{.003}{34}(34-2.625) = .00277$, $f_{s1} = 60$

$f_{s2} = 60$ $\quad f_{s5} = 25.1$
$f_{s3} = 60$ $\quad f_{s6} = 11.6$
$f_{s4} = 43.5$ $\quad f_{s7} = 6.7$

$P_n = .85 f'_c b \beta_1 c + 1f_{s1} + 1f_{s7} + 2(f_{s2} \ldots + \ldots f_{s6})$
$= \boxed{4852\text{k}}$

$M_n = \boxed{1759'\text{K}}$

– @ $\varepsilon_t = .005 = \varepsilon_{s7}$

$c = \frac{.003}{.003 + .005}(31.375) = 11.8"$

$f_{s1} = 60$, $f_{s2} = 53.5$, $f_{s3} = 14.7$, $f_{s4} = -38.2$,
$f_{s5} = -60$, $f_{s6} = -60$, $f_{s7} = -60$

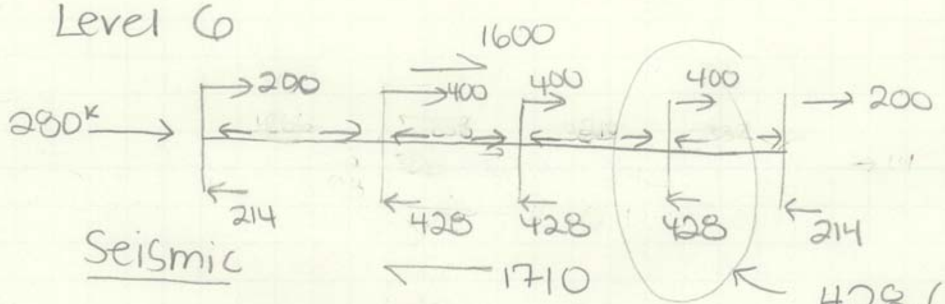
$$P_n = .85 f'_c b (\beta_1)(c) + 2(f_{s2} + \dots f_{s6}) + f_{s1} + f_{s7}$$
$$= \boxed{4205^K}$$

$$M_n = .85 f'_c \beta_1 b c \left(\frac{h}{2} - \frac{\beta_1 c}{2}\right) + f_{s,i}\left(\frac{h}{2} - d_i\right)$$
$$= \boxed{595 \text{ ft-K}}$$

P_n (K)	M_n (ft-K)	c (in)	$\varepsilon_t = \varepsilon_s$	ϕ
5113	0	∞	—	.65
4852	1759	34	.000231	.65
2594	2009	17	-.00202	.65
4521	595	11.8	-.00498	.9
0	1241	4.57	-.0176	.9

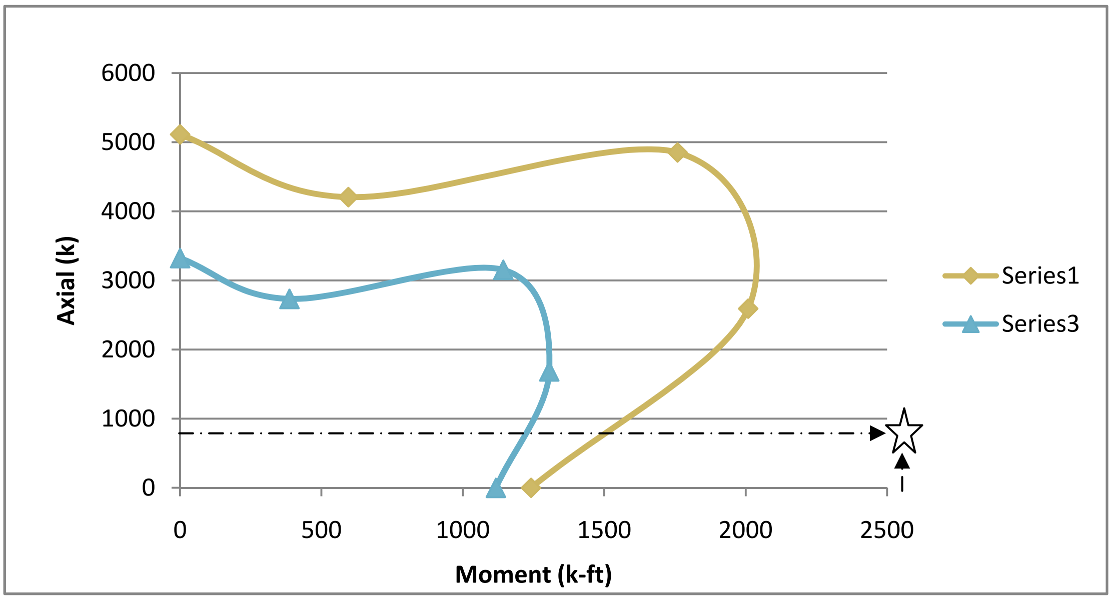
Based on loading in drawing
P_u found to equal 822^K

Level 6



$$428\left(\frac{12.83}{2}\right) =$$
$$\boxed{M = 2746 \text{ ft-K}}$$

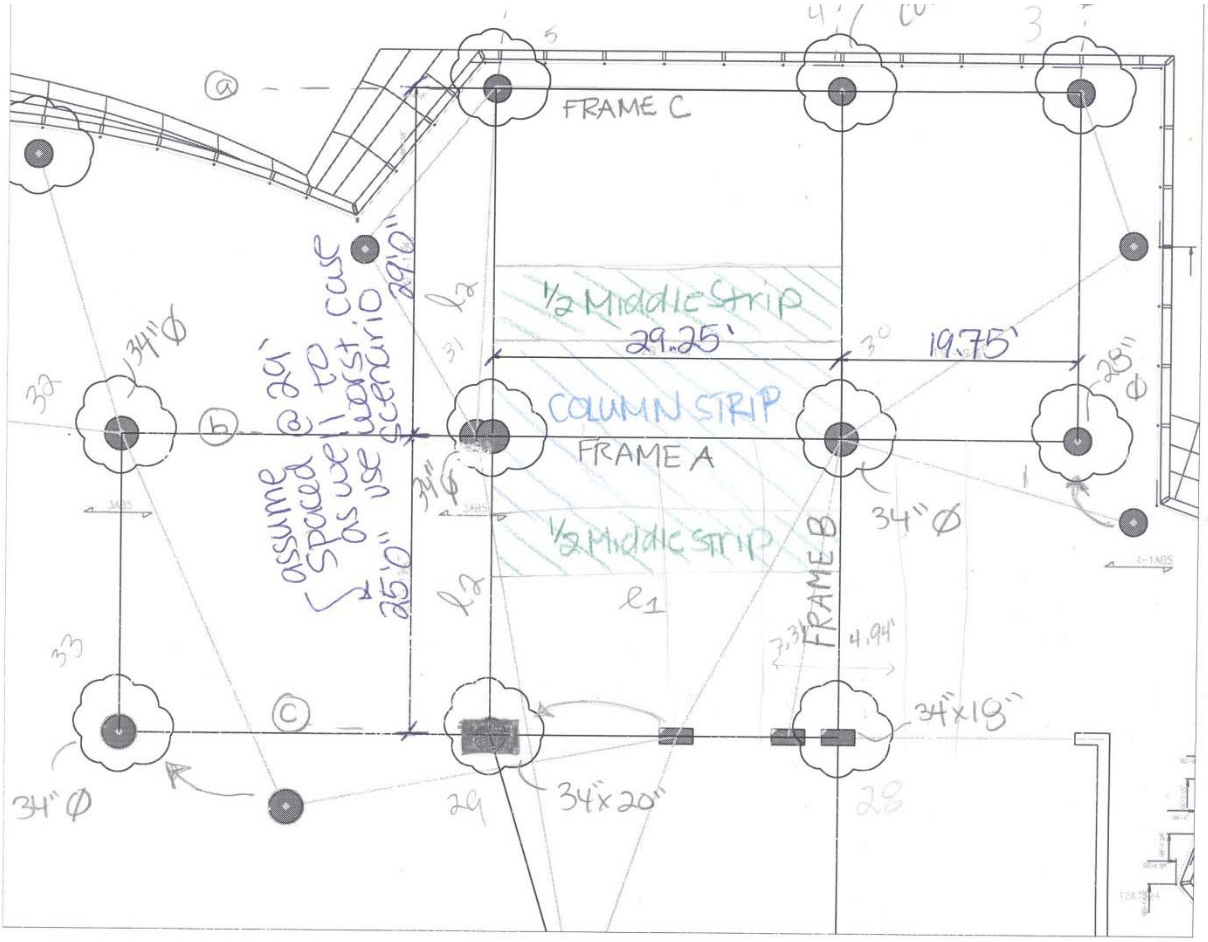
Figure A-3: Interaction Diagram for Column



Because the column's Pu & Mu do not fall within the interaction diagram, it is not adequate for the loading.

Flat Plate Slab Check:

Figure A-4: Layout for Slab Check (showing simplified assumptions)



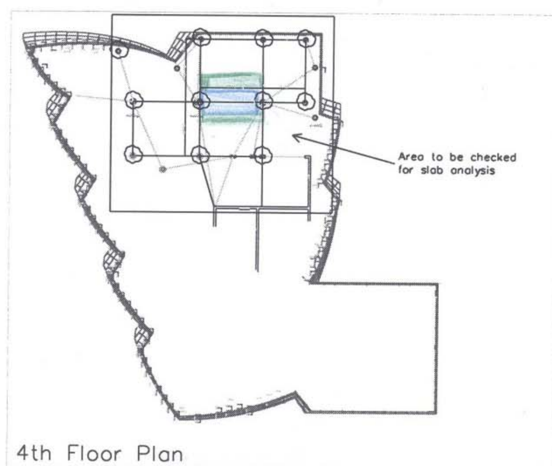


Figure A-5: Layout of steel reinforcing (as specified in drawings)

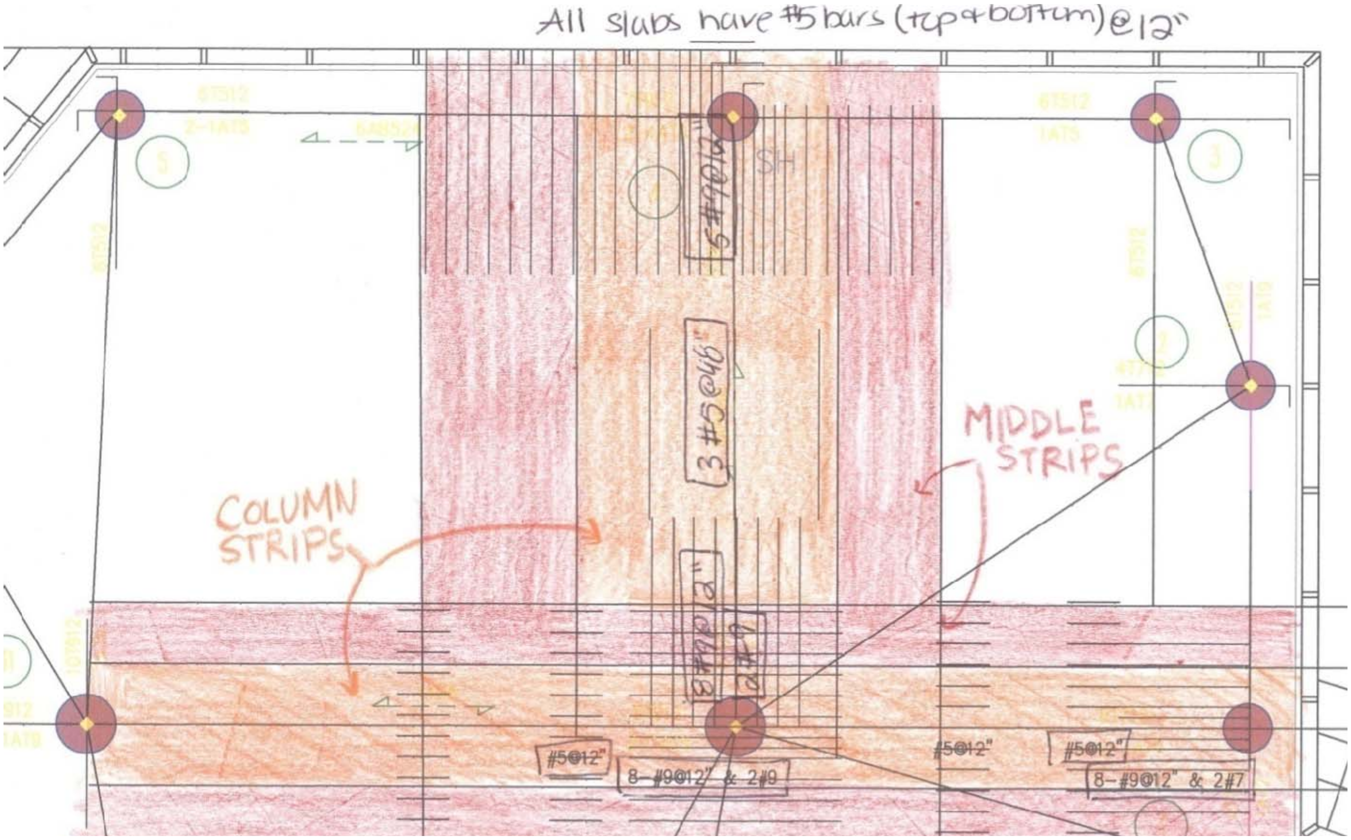


Table A-12: Steel in Column Strip & Middle Strip for Frames A & B as constructed (to be compared with Table A-15 & A-16 to check adequacy

	Ex Span Neg Ext	Ext Span Pos	Ext Span Neg Int	Interior Span Pos	Interior Span Neg		Ex Span Neg Ext	Ext Span Pos	Ext Span Neg Int	Interior Span Pos	Interior Span Neg
Area of Steel in Designed Middle Strip	5	3.1	3.93	1.86	9.86	Area of Steel in Designed Middle Strip	2.48	0.93 *	13.7	-	-
Area of Steel in Designed Column Strip	6.2	1.86	11.5	4.34	3.86	Area of Steel in Designed Column Strip	8.69	1.86	2.48	-	-

Table A-13: Flat Plate Slab Check using Direct Design Method

l1	**29.25**	**ft**	
l2 =	29	ft	
Distance of column strip	7.25	(smaller of l1/4 or l2/4)	
LL	60	psf	
DL	170	psf	
Load Comb.	300	psf	
Slab Thickness	12	in	
Column Diam	28	in	
	34	in	
Mo	wult*ln^2		
Frame A	751.024219	k-ft	
Frame B	773.333333	k-ft	
Frame C	375.512109	k-ft	
f'c	5000	psi	
fy	60,000	psi	
Min Thickness of Slab	Table 9.5c		
fy=60,000	Ext= ln/33	Int= ln/33	tmin>5"
	9.60606061	9.606060606	
	12">9.6" so ok		
d	10.625		

Table A-14: Distribution of Moments by Direct Design Method

FRAME A

Distribution of Mo	Total Moment Factor	Total Moment		CS Factor	CS Moment	MS/2 Factor	MS/2
End Span	0.26	-195.27	Ext Neg	0.26	-195.2662969	0	0
	0.52	390.53	Pos	0.31	232.8175078	0.21	157.7150859
	0.7	-525.72	Int Neg	0.53	-398.0428359	0.17	-127.6741172
Int Span	0.35	262.86	Pos	0.21	157.7150859	0.14	105.1433906
	0.65	-488.17	Neg	0.49	-368.0018672	0.16	-120.163875

FRAME B

Distribution of Mo	Total Moment Factor	Total Moment		CS Factor			MS/2
End Span	0.26	-201.07	Ext Neg	0.26	-201.07	0	0.00
	0.52	402.13	Pos	0.31	239.73	0.21	162.40
	0.7	541.33	Int Neg	0.53	-409.87	0.17	-131.47
Int Span	0.35	270.67	Pos	0.21	162.40	0.14	108.27
	0.65	502.67	Neg	0.49	-378.93	0.16	-123.73

FRAME C

Distribution of Mo	Total Moment Factor	Total Moment		CS Factor			MS/2
End Span	0.26	-97.63	Ext Neg	0.26	-97.63314844	0	0
	0.52	195.27	Pos	0.31	116.4087539	0.21	78.85754297
	0.7	-262.86	Int Neg	0.53	-199.021418	0.17	-63.83705859
Int Span	0.35	131.43	Pos	0.21	78.85754297	0.14	52.57169531
	0.65	-244.08	Neg	0.49	-184.0009336	0.16	-60.0819375

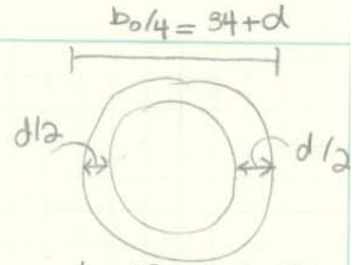
Table A-15: Moments for Column Strip

Frame A						**Frame B**					
COLUMN STRIP	Ext Span			Int Span		COLUMN STRIP	Ext Span			Int Span	
Descr.	Mext-	M+	Mint-	M+	M-	Descr.	Mext-	M+	Mint-	M+	M-
Moment Mn	-195.266	232.8175	-398.043	157.7151	-368.002	Moment Mn	-201.07	239.73	-409.87	162.40	-378.93
CS slab width (in)	174	174	174	174	174	CS slab width (in)	147	147	147	147	147
Effective Depth	10.625	10.625	10.625	10.625	10.625	Effective Depth	10.625	10.625	10.625	10.625	10.625
Mu=Mn/.9	-216.963	258.6861	-442.27	175.239	-408.891	Mu=Mn/.9	-223.407	266.3704	-455.407	180.4444	-421.037
R=Mu/bd^2	-132.544	158.0328	-270.185	107.0545	-249.794	R=Mu/bd^2	-161.549	192.6158	-329.311	130.4817	-304.457
Rho (interpolated from table A.5a in text)	0.002414	0.00269	0.004661	0.00181	0.004304	Rho (interpolated from table A.5a in text)	0.00275	0.003286	0.00572	0.002216	0.005277
As=pbd (in^2)	4.462513	4.972583	8.616469	3.346873	7.956225	As=pbd (in^2)	4.295156	5.132321	8.933925	3.460334	8.242014
As,min=.002 bt	3.6975	3.6975	3.6975	3.6975	3.6975	As,min=.002 bt	3.12375	3.12375	3.12375	3.12375	3.12375
As,req=	4.462513	4.972583	8.616469	3.6975	7.956225	As,req=	4.295156	5.132321	8.933925	3.460334	8.242014
N of #5's	14.3952	16.04059	27.79506	11.92742	25.66524	N of #5's	13.85534	16.55588	28.81911	11.16237	26.58714
N of #5's	**15**	**17**	**28**	**12**	**26**	N of #5's	**14**	**17**	**29**	**12**	**27**
Nmin#5=widthstrip/2t	7.25	7.25	7.25	7.25	7.25	Nmin#5=widthstrip/2t	6.125	6.125	6.125	6.125	6.125
	8	8	8	8	8		7	7	7	7	7

Table A-16: Moments for Middle Strip

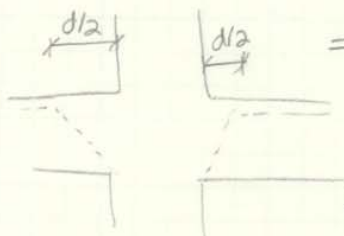
MIDDLE STRIP	Ext Span			Int Span		MIDDLE STRIP	Ext Span			Int Span	
Descr.	Mext-	M+	Mint-	M-	M+	Descr.	Mext-	M+	Mint-	M-	M+
Moment Mn	0	157.7151	-127.674	-120.164	105.1434	Moment Mn	0.00	162.40	-131.47	-123.73	108.27
MS slab width (in)	174	174	174	174	174	MS slab width (in)	147	147	147	147	147
Effective Depth	10.625	10.625	10.625	10.625	10.625	Effective Depth	10.625	10.625	10.625	10.625	10.625
Mu=Mn/.9	0	175.239	-141.86	-133.515	116.826	Mu=Mn/.9	0	180.4444	-146.074	-137.481	120.2963
R=Mu/bd^2	0	107.0545	-86.6632	-81.5653	71.36965	R=Mu/bd^2	0	130.4817	-105.628	-99.4146	86.98779
Rho (interpolated from table A.5a in text)	0	0.00181	0.00134	0.00137	0.001196	Rho (interpolated from table A.5a in text)	0	0.002216	0.001786	0.001679	0.00147
As=pbd (in^2)	0	3.346238	2.477325	2.532788	2.211105	As=pbd (in^2)	0	3.460334	2.789509	2.622388	2.2959
Asmin=.002bt	3.69	3.69	3.69	3.69	3.69	As,min=.002bt	3.12	3.12	3.12	3.12	3.12
As,req=	3.69	3.69	3.69	3.69	3.69	As,req=	3.12	3.46	3.12	3.12	3.12
N of #5s=largerAs/barsize	6.1625	6.1625	6.1625	3.6975	3.6975	N of #5s=largerAs/barsize	5.20625	5.767223	5.20625	3.12375	3.12
	7	7	7	4	4		6	6	6	4	4
Nmin#5=widthstrip/2t	7.25	7.25	7.25	7.25	7.25	Nmin#5=widthstrip/2t	6.125	6.125	6.125	6.125	6.125
	8	**8**	**8**	**8**	**8**		**7**	**7**	**7**	**7**	**7**

Punching Shear & Wide Beam Action



$$V_c = \min\begin{cases} 4\sqrt{f'_c}\, b_o d \\ (2+4/\beta_c)\sqrt{f'_c}\, b_o d \\ \left(\frac{\alpha_s d}{b_o}+2\right)\sqrt{f'_c}\, b_o d \end{cases}$$

$d = 12 - 3/4 - 5/8$
$= 10.625"$



Int. Column

$$= \begin{cases} 4(\sqrt{5000})(178.5)(10.625) = 536^k \\ (2+4/1)\sqrt{5000}(178.5)(10.625) = 805^k \\ \left(\frac{40(10.625)}{178.5}+2\right)(\sqrt{5000})(178.5)(10.625) \\ 588^k \end{cases}$$

$\therefore \boxed{536^k \text{ controls}}$

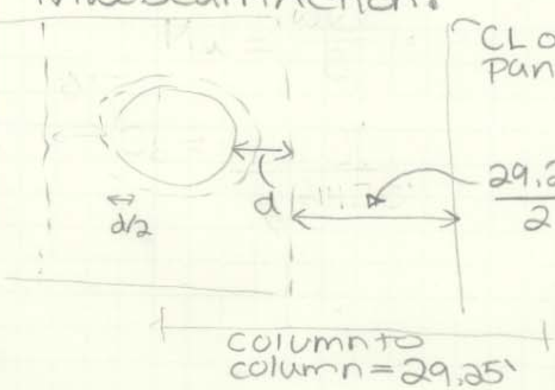
Trib Area $\Rightarrow$ $\ell = \frac{29.25+19.75}{2} = 24.5$

$w = \frac{29+25}{2} = 27$

$24.5 \times 27 =$
$662\ ft^2$

$w_u = 1.2(170) + 1.6(60) = 300\ psf$

Wide Beam Action:



$V_u = .3(13.74) =$
$\boxed{4.1\ \text{Kips}}$

$\frac{29.25}{2} - \frac{10.625}{12} = 13.74$

$V_c = 2\sqrt{f'_c}\, b_w d$
$= 2(\sqrt{5000})(12)(10.625)$
$= 18.03^k$

$\phi V_c = 18.03(.75) =$
$\boxed{13.5^k}$

$\phi V_c > V_u \therefore$ OK for wide beam action

Punching Shear (cont'd)

$V_c = 536^k$

area of panel

$V_u = .3\left(24.5 \times 27 - \left(\frac{34+10.625}{2}\right)^2 \pi / 12\right) = 159^k$

$\phi V_c = .75(536) = 402^k$

$\phi V_c > V_u \therefore OK$

This check seems to show that design is adequate for shear strength

Quick deflection check:

$h = \frac{l_n}{30} = \frac{29.25(12)}{30} = 11.7" < 12"$

$\therefore$ OK for deflection

Appendix B: Floor Plans

Figure B-1: Cellar Floor Plan

(Information disclosed at owner's request)

Figure B-2: 5th Floor Plan

(Information disclosed at owner's request)

Figure B-3: 6th Floor Plan

(Information disclosed at owner's request)

Figure B-4: 11th Floor Plan

(Information disclosed at owner's request)

Appendix C: Additional Photographs

Figure C-1: View of IAC from north-west

Figure C-2: Close-Up of building's curtain wall system

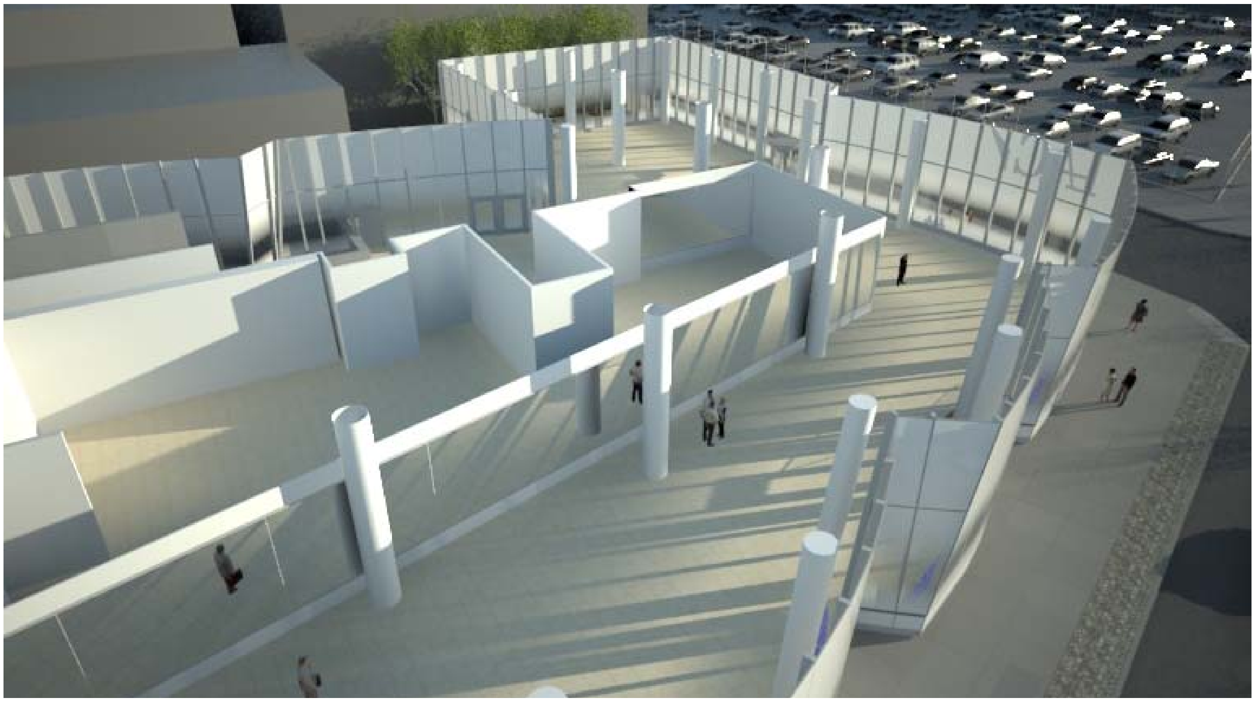


Figure C-3: Rendering of 1st floor

Figure C-4: View of an office reception area, showing the sloped columns

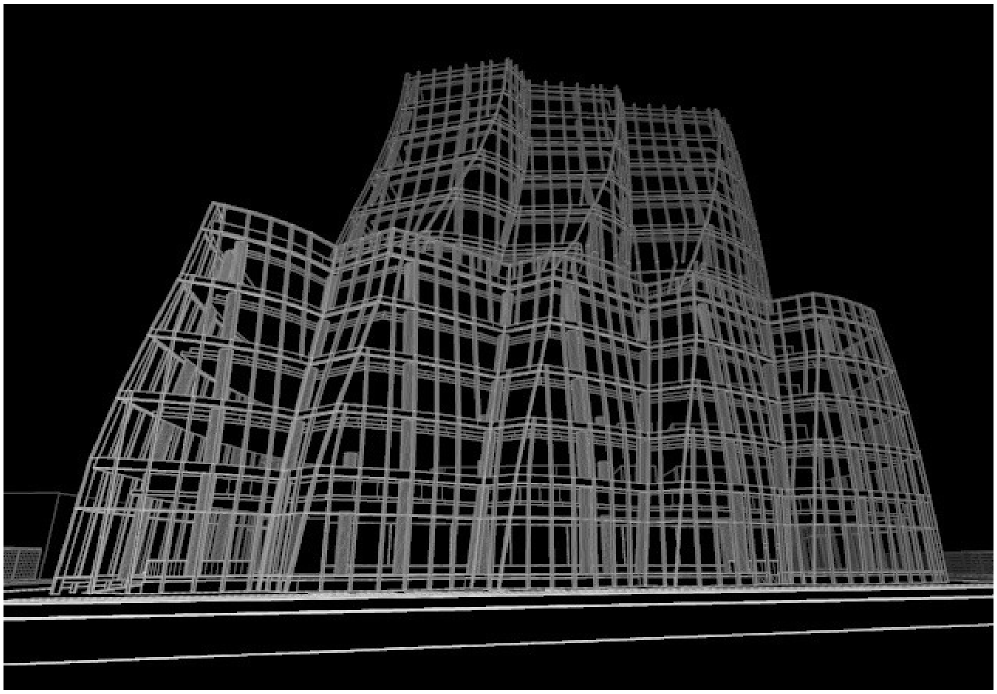


Figure C-5: 3-D Schematic of building structural system

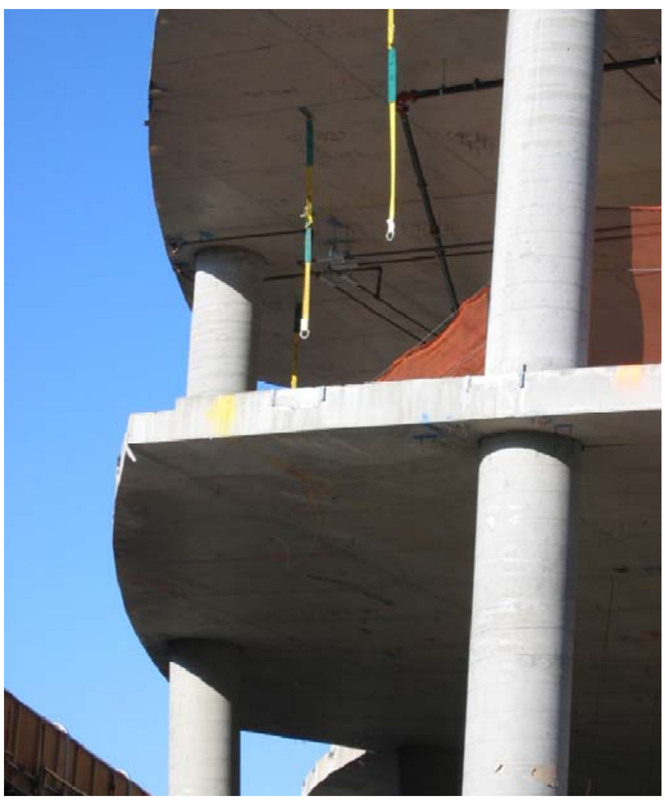


Figure C-6: Close up of flat plate system during construction at perimeter of building