The New York Times Building

New York, NY

IPD/BIM Thesis
Technical Report #1

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Structural Option

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EXECUTIVE SUMMARY

The purpose of the first technical report is to analyze and compile the existing structural conditions for the New York Times Headquarters in New York City. The building houses the New York Times newsroom, retail spaces along its base, as well as New York Times and rentable corporate offices in the tower. As a result of an architectural competition, Renzo Piano's design intends to exemplify transparency and lightness through every detail, as well as become a signature building in the New York City skyline. Exposed structural elements and connections were designed with great attention to the overall appearance of the building.

Gravity, wind, and seismic systems were studied in detail to yield a basis of design for the structure as produced by Thornton Tomasetti. Codes and methods applied to the analyses are outlined within the report, as well as a more comprehensive discussion and depiction of each system and other elements requiring future consideration. Calculations are also provided in the appendices for reference.

Gravity loads were compiled and analyzed using ASCE 7-05 and IBC 2006; both codes are more recent than the Building Code of the City of New York used for the original design. A typical bay was investigated to compare beam, girder, and column sizes for accuracy using assumed dead and live loads. Values obtained from analysis were slightly lower than those used in the original design; this could be due to a difference in live load reductions or an increase in member sizes due to lateral forces. A wind analysis was completed by referencing ASCE 7-05; however, Thornton Tomasetti performed wind tunnel tests on the structure, possibly leading to different final lateral values. Seismic forces were obtained from Chapters 11 and 12 of ASCE 7, but did not control laterally over wind.

In addition to the structural investigation of the gravity and lateral loads, parameters such as thermal loading, building drift, and cantilevers must be considered to fully understand the structure of the New York Times Headquarters. Although these factors and elements are not within the scope of this report, they are presented as essential future considerations.

INTRODUCTION

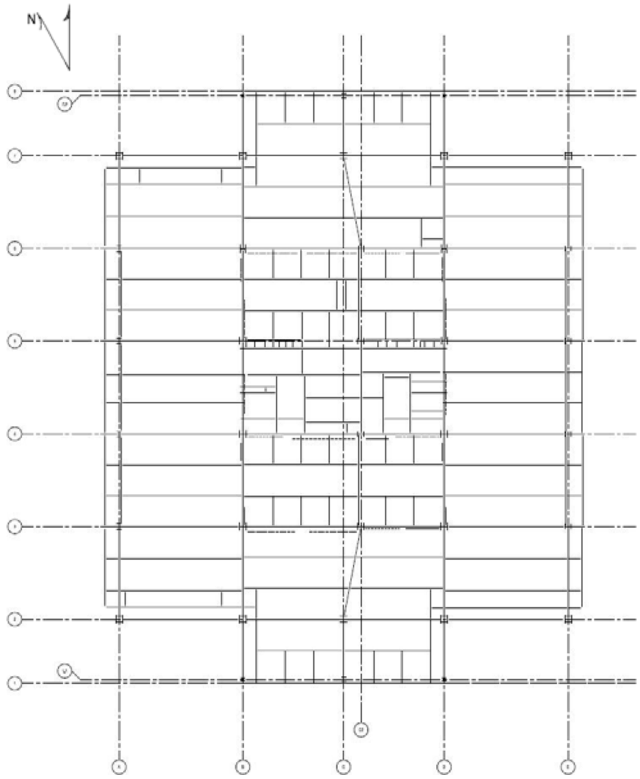


Figure 1: Typical Tower Framing Plan

The New York Times Headquarters Building is home to the New York Times newsroom and 26 floors of Times offices, as well as several law firms, their offices leased through Forest City Ratner. It was intended to be a flagship building promoting sustainability, lightness, and transparency. The architectural façade reflects the ever-changing environment surrounding the building, an appropriate acknowledgement of the heart of New York City.

The building rises 1046 feet above Eighth Avenue between 41st and 42nd Streets. It has one level below grade. The ground level contains a lobby, retail space, and a glass-enclosed garden. New York Times' newsroom occupies the entire five-story podium of the structure, and the tower ascends above the podium an additional 48 stories. Story heights average approximately 13 feet 9 inches in the tower, lending a great view to the open office plans. At the mechanical floors on levels 28 and 51, though, the floor height is approximately 27 feet to accommodate equipment and two-story outriggers.

The steel structural system is comprised of composite floor beams and columns configured as shown in Figure 1, with lateral chevron and K braces in both the East-West and North-South directions. Foundations are a combination of concrete spread footings and caissons to develop the required capacity. Many structural elements are also architectural details, including the exposed X bracing on the exterior of the structure and the built-up columns at the corner notches. Overall, the building exhibits ingenuity in design and construction, with close attention paid to every detail.

STRUCTURAL SYSTEM

Foundation

The foundation of the New York Times Headquarters combines typical spread footings with caissons to achieve its maximum axial capacity. Below the building's 16-foot cellar, the tower and podium mostly bear on 20 to 40 ton rock, Class 1-65 and 2-65 per the New York City Building Code. However, a core sample taken just before finalizing the site investigation report indicated that rock at the southeast corner of the tower only had an 8 ton per square foot capacity, Class 4-65. At the seven columns that fall within this area, indicated in red on Figure 2, 24-inch diameter concrete-filled steel caissons were used to replace the original foundation designs. Each caisson was designed to support a load of 2,400 kips with 6,000 psi concrete.

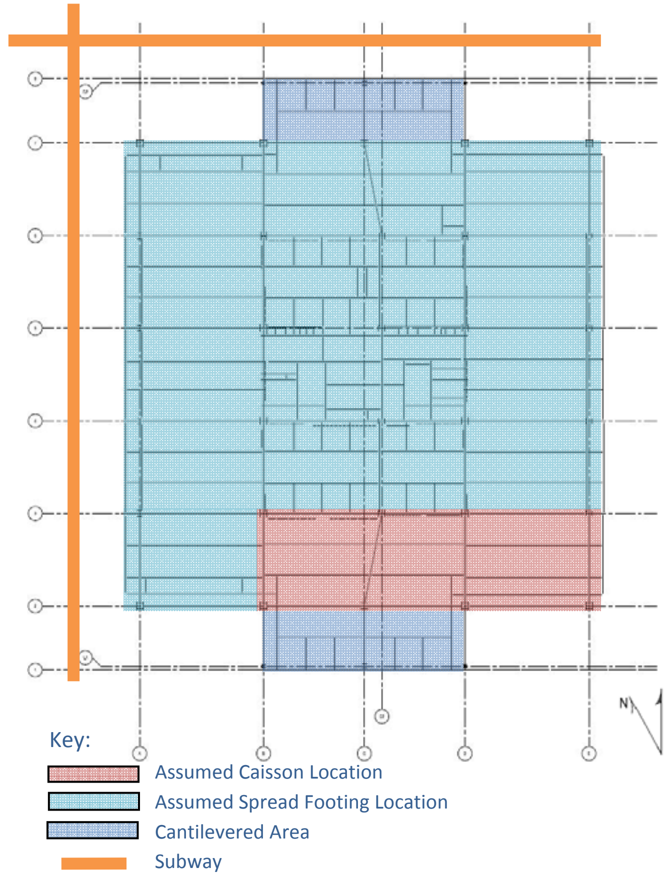


Figure 2: Foundation Locations

Under the other 21 columns (indicated on Figure 2 in teal), spread footings with a compressive strength of 6,000 psi are used to support the loads. The areas depicted in blue represent the two cantilevered sections of the tower. The columns which fall in these areas do not directly transfer load to the ground which removes the need for footings at these locations.

The New York City Subway does pass the north and eastern sides of the New York Times Building. However, this is not a major site restriction since the transit system passes below Eighth Avenue and 41st Street and not directly beneath the structure. But, vibration effects on the foundation and building structure may have had an impact on the design.

Floor System

The floor system is a composite system with a typical bay size of 30'-0"x 40'-0" with 2 ½" normal weight concrete and 3" metal deck, typically spanning 10'-0" from W12x19 to W18x35 infill beams. The W12x19 and W18x35 beams span into W18x40 girders. The girders frame into the various built-up columns, box columns along the exterior and built-up non-box columns in the core. Framing of the core consists of W12 and HSS shapes framing into W14 and W16 shapes which frame into W33 girders that frame into the core columns.


Figure 3: 'Dog-leg' beam connection

In the New York Times spaces, the structural slab is 16 inches below the finish floor and the spandrel panel, due to the raised floor system for the under floor mechanical systems. The solution to this issue was to create a 'dog leg' at the end connection of the beam, depicted in Figure 3 on the previous page.

Columns

The 30" by 30" box columns (Figure 4) at the exterior notches of the tower consist of two 30 inch long flange plates and two web plates inset 3 inches from the exterior of the column on either side. The web plates decrease in thickness from 7 inches at the bottom of the building to adjust to the loads at each level. The flange plates decrease in thickness to relate to the architectural vision of the tower. Although the yield strength of the plates also varies with tower height, the strength was assumed to be a uniform 50 ksi for calculations. Interior columns are a combination of built-up sections and rolled shapes. Column locations stay consistent throughout the height of the building, and every column is engaged in the lateral system.

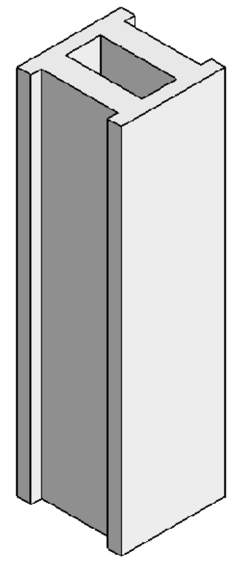


Figure 4: Box Column Modeled in Revit Structure

Vierendeel Frame

A Vierendeel frame was used by Thornton Tomasetti as a combined solution at the 20 foot cantilever sections of the tower. Renzo Piano did not want columns obstructing the glass storefronts at the ground level, so these sections were cantilevered from the main structure. A unique way to control deflections in the middle beams of the cantilevered section, the ladder-like frame ties together the beams at each level. It connects to 28^{th} and 52^{nd} floor outriggers; these outriggers effectively transfer loads from the frame to the core of the tower. The beams are also moment connected to each vertical member. See Figures 17 and 19 in Appendix A for exact locations.

At the exterior beam lines of the cantilever, steel rods were connected from the columns to the ends of the beams to control deflection. This allowed the beams to be designed only for strength, avoiding bulky exterior members.

Lateral System

The main lateral load resisting system for the tower of the New York Times Building consists of a centralized steel braced frame core with outriggers on the two mechanical floors (Levels 28 and 51). The structural core consists of chevron, knee, and single diagonal bracing which surrounds elevator shafts, MEP shafts, and stair wells. At this time, the member sizes of these braces have yet to be disclosed. The core configuration remains consistent from the ground level to the 27th floor as shown in Figure 5. But above the 28th floor, the low rise elevators were no longer required. In order to optimized the rentable space on the upper levels of the tower, the number of bracing lines in the North-South direction were reduced from two to one (Figure 6). Please refer to Figures 7 and 8 to view the typical core bracing configurations.

The outriggers on the mechanical floors consist of K-braces (Figure 19 in Appendix A) and single diagonal braces. The outrigger system was designed to increase the efficiency and redundancy of the tower by engaging the perimeter columns into the lateral system. Please refer to Appendix A to view the framing plans and bracing elevations of the outrigger system.

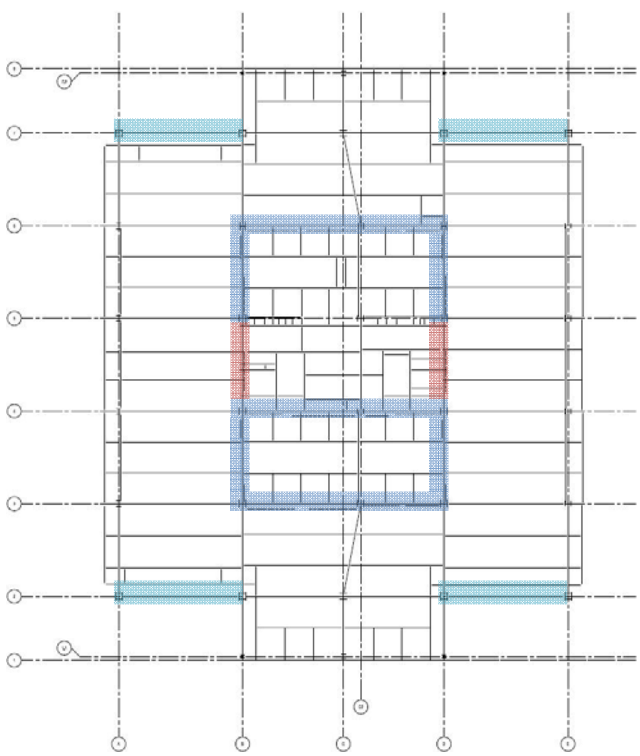


Figure 5: Typical Lateral System (Floors 1-27)

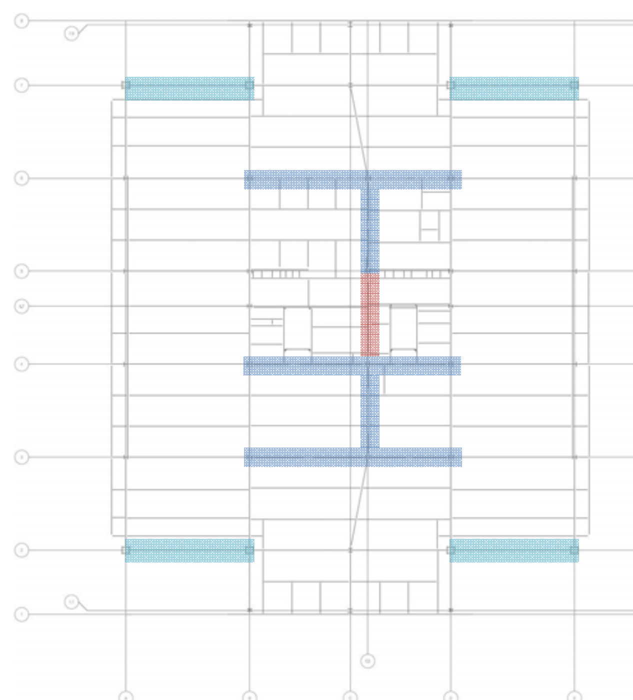


Figure 6: Typical Lateral System (Floors 29-50)

Key:

- Single Diagonal Bracing
- Pre-Tensioned Steel Rod X-Bracing
- Chevron & Open Knee Bracing

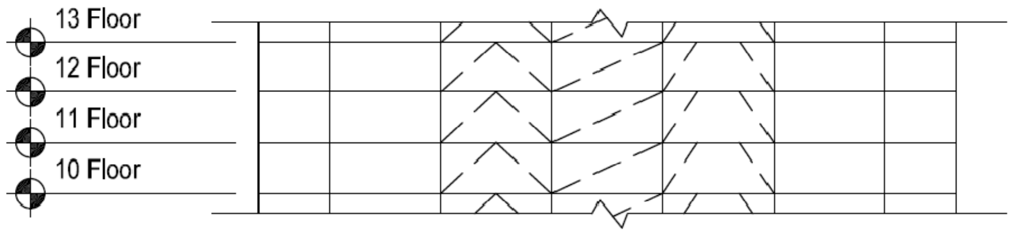


Figure 7: Typical Core N/S Core Bracing Elevation

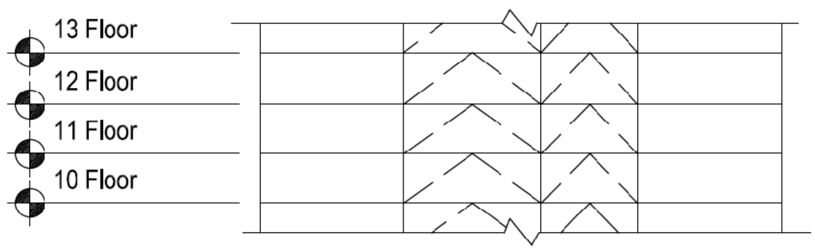


Figure 8: Typical Core E/W Core Bracing Elevation

During the design of the tower, the engineers at Thornton Tomasetti sized the members of the main lateral force resisting system merely for strength. In order to increase stiffness and meet deflection criterion, the structural engineers utilized the double story steel rod X-braces (original to Renzo Piano's exterior design) instead of increasing the member sizes of the main lateral force resisting system. These X-braces can be seen in Figure 29 of Appendix F and in Figures 5 and 6 above. The high strength steel rods transition from 2.5" to 4" in diameter and were prestressed to 210 kips. This induced tensile load prevents the need for large compression members which would not conform to the architectural vision of the exterior.

Although the X-braces did reduce the need for an overall member size increase, the lateral system still did not completely conform to the deflection criterion. Therefore, some of the 30” by 30” base columns were designed as built-up solid sections which reduced the building drift caused by the building overturning moment. After combining these solid base columns and the X-braces with the main lateral force resisting system, the calculated deflection of the tower due to wind was L/450 with a 10 year return period and a building acceleration of less than 0.025g for non-hurricane winds.

According to information obtained from the structural engineer, the podium of the New York Times Building was designed with a separate lateral system. Though information about the podium was not disclosed by the owner, an educated guess can be made about its lateral system. The podium contains the New York Times Newsroom; therefore it can be assumed that steel bracing, which would cut down on the usable floor space, would not be used. Also, the use of concrete shear walls would go against the architect’s “transparent” building design. Therefore, it can be assumed that the lateral system of the podium is designed as a steel moment resisting frame.

CODES AND REFERENCES

Design Codes:

National Model Code:

1968 Building Code of the City of New York

Structural Standards:

ASCE 7-98, Minimum Design Loads for Buildings and other Structures

Structural Design Codes:

AISC – LRFD, Steel Construction Manual 2nd edition, American Institute of Steel Construction, 1998

National Building Code of Canada, 1995

Uniform Building Code, 1997

Design Deflection Criteria:

Lateral Deflections:

Total building sway deflection for ten year wind loading is limited to H/450

Thermal Deflections:

The shortening and elongating effects due to thermal fluctuations is designed to L/300.

At this point in time additional gravity and lateral deflections were not disclosed.

Thesis Codes:

National Model Code:

2006 International Building Code

Structural Standards:

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

Design Codes:

AISC – LRFD, Steel Construction Manual 13th edition, American Institute of Steel Construction

MATERIAL STRENGTHS

Structural Steel:

Wide Flanges Shapes……………….…….…………..…………….ASTM A572 or A992, Grade 50
Built-Up Sections……………………………...…………..ASTM A572, Grade 50 & Grade 42
HSS Shapes…………………………………………….…..……………ASTM A500 Grade B
Diagonal & X-Braced Rods……….…….…………..…..…………………...ASTM A572, Grade 65
Connection Plates………………………………………………………………….....ASTM A36

Concrete:

Caissons…………………………………………………………...…………...f'_c = 6000 psi
Spread Footings……………………………………….……………..…………...f'_c = 6000 psi
Slabs on Deck (normal weight concrete)…………………..…………………………..f'_c = 4000 psi

Metal Decking:

3" Composite Deck………………………………..……….…………………………..F_y = 40 ksi

At this point in time shear studs, welds, bolts, and reinforcement strengths were not disclosed by the designer.

LOADINGS

ASCE 7-05 and Thornton Tomasetti provided guidance to determine loading for both gravity and lateral loads.

Gravity Loads

Dead Loads

Typical Tower Floor Dead Load			
Load Description	Design Load		Reference
5.5" Slab with 20 GA 3" Composite Metal Deck Vulcraft Deck Type: 3VL20, Max unshored clear span is 11'-9" with a capacity of 127 psf (50−3 for deck)	53	psf	Thornton Tomasetti, Kyle Krall & Vulcraft Catalog page 48
Ceiling (Floors have ACT, Drywall, and Special Architectural Ceilings)	5	psf	Thornton Tomasetti, Kyle Krall
Mechanical, Electrical, Plumbing in raised floor system	12	psf	Thornton Tomasetti, Kyle Krall
Mechanical, Electrical, Plumbing in ceiling	8	psf	Thornton Tomasetti, Kyle Krall
Allowance for Self Weight of Steel Framing + Fireproofing(intumescent paint & cementitious)*	15	psf	
Total Typical Floor Dead Load:	93	psf	

*includes column weight therefore loading only applied to columns

Table 1: Typical Tower Floor Dead Load

Typical Tower Mechanical Floor Dead Load			
Load Description	Design Load		Reference
6" Slab with 20 GA 3" Composite Metal Deck, Vulcraft Deck Type: 3VL20, Max unshored clear span is 10'-11" with a capacity of 143 psf	57	psf	Thornton Tomasetti, Kyle Krall & Vulcraft Catalog page 48
Ceiling (Floors have ACT and Special Architectural Ceilings)	5	psf	Thornton Tomasetti, Kyle Krall
Mechanical, Electrical, Plumbing in ceiling	8	psf	Thornton Tomasetti, Kyle Krall
Allowance for Self Weight of Steel Framing + Fireproofing(intumescent paint & cementitious)*	15	psf	
Total Mechanical Floor Dead Load:	93	psf	

*includes column weight therefore loading only applied to columns

Table 2: Typical Tower Mechanical Floor Dead Load

Exterior Tower Wall System Dead Load (Elevation)			
Load Description	Design Load		Reference
Windows, glass, frame, and sash	8	psf	ASCE7-05, Table C3-1
Horizontal ceramic rods, Aluminum, and frame	5	psf	
Total Exterior Wall Dead Load:	13	psf	

Table 3: Exterior Tower Wall System Dead Load

Since the weight of the curtain wall system is unknown, ASCE7-05 was used to determine the self weight of the glass and the ceramic tube system self weight was conservatively assumed to be lighter than the self weight of the windows. In the spot checks below, it is assumed that the system self weight of the wall creates a uniform load up the building.

Tower Mechanical Area Roof Dead Load			
Load Description	**Design Load**		**Reference**
8" Composite Deck	85	psf	
Allowance for Self Weight of Steel Framing + Fireproofing(intumescent paint & cementitious)*	15	psf	
Total Mechanical Area Roof Dead Load:	100	psf	

*includes column weight therefore loading only applied to columns

Table 4: Tower Mechanical Area Roof Dead Load

Normal Tower Roof Dead Load			
Load Description	**Design Load**		**Reference**
8" Composite Deck	85	psf	
Allowance for Self Weight of Steel Framing + Fireproofing(intumescent paint & cementitious)*	15	psf	
Total Normal Roof Dead Load:	100	psf	

*includes column weight therefore loading only applied to columns

Table 5: Normal Tower Roof Dead Load

Live Loads

Tower Roof Live Load			
Load Description	**Design Load**		**Reference**
Ordinary flat roof:	20	psf	ASCE7-05, Table 4-1
Roof gardens:	100	psf	ASCE7-05, Table 4-1
Controlling Roof Live Load:	100	psf	

Table 6: Tower Roof Live Load

Tower Mechanical Area Roof Live Load			
Load Description	**Design Load**		**Reference**
Mechanical, Electrical, Plumbing:	125	psf	ASCE7-05, Table 4-1
Walkways and elevated platforms:	60	psf	ASCE7-05, Table 4-1
Controlling Roof Live Load:	125	psf	

Table 7: Tower Mechanical Area Roof Live Load

Since the weight of the mechanical equipment on the roof is unknown, and ASCE7-05 and the Building Code of the City of New York provides no minimum live load, the self weight of the equipment was conservatively assumed to be equivalent to light manufacturing therefore at a minimum the live load should be 125 psf.

Tower Typical Office Area Live Load			
Load Description	**Design Load**		**Reference**
Office:	50	psf	ASCE7-05, Table 4-1
Partitions:	20	psf	ASCE7-05, Table 4-1
Total Typical Office Area Live Load:	70	psf	

Table 8: Tower Typical Office Area Live Load

Tower Cafeteria Floor Live Load			
Load Description	**Design Load**		**Reference**
Cafeteria:	100	psf	ASCE7-05, Table 4-1
Total Cafeteria Floor Live Load:	100	psf	

Table 9: Tower Cafeteria Floor Live Load

Tower Core Floor Live Load			
Load Description	Design Load		Reference
Lobbies:	100	psf	ASCE7-05, Table 4-1
Total Core Floor Live Load:	**100**	psf	

Table 10: Tower Core Floor Live Load

Tower Mechanical Floor Live Load			
Load Description	Design Load		Reference
Mechanical Room:	125	psf	ASCE7-05, Table 4-1
Total Mechanical Floor Live Load:	**125**	psf	

Table 11: Tower Mechanical Live Load

Since the weight of the mechanical equipment on the mechanical floors is unknown, and ASCE7-05 and the Building Code of the City of New York provides no minimum live load, the self weight of the equipment was conservatively assumed to be equivalent to light manufacturing therefore at a minimum the live load should be 125 psf.

Snow Loads

Snow Load			
Load Description/Factor	Design Load		Reference
p_g=	25	psf	ASCE7-05, Table 7-1
C_e=	1		ASCE7-05, Table 7-2
C_t=	1		ASCE7-05, Table 7-3
I=	1		ASCE7-05, Table 7-4
C_s=	1		ASCE7-05, Fig. 7-2a
p_s=0.7CsCeCtIp_g	17.5	psf	ASCE7-05, eq. 7-1 & 7-2

Table 12: Snow Load Calculation

Since the weight of the snow on the roof is approximately five times smaller compared to the controlling roof live load and mechanical area roof live load, drift was not calculated for this report since it is assumed to not control.

Lateral Loads

Wind Loads

As mentioned, the 1968 Building Code of the City of New York was the governing code for the design of the New York Times Building. During the time of the building's design, this code permitted the use of a simplified approach for calculating the wind loads of all buildings not more that 300 ft within the Borough of Manhattan. Although, for structures which exceeded this height, the code required that wind load be determined using ASCE 7-98. Thornton Tomasetti opted to use a wind tunnel analysis (Method 3) within ASCE 7-98 to determine the wind design loads. However, for the analysis in this report, Method 2 of ASCE 7-05 was used. Unfortunately, the engineers have yet to divulge the results from wind tunnel analysis meaning a true comparison cannot be made to the actual wind design loadings. Also when comparing the Method 2 provisions from ASCE 7-98 to ASCE 7-05, it was found that few changes had been made between the two issues. This means that the results between the two versions would have minimal differences.

A few simplifying assumptions had to be made in order use Method 2 of ASCE 7-05. First of all, the tower was analyzed with a rectangular foot print instead of a cruciform shape. Essentially, area was added at the corners of the façade to simplify the corner notches. Secondly, the screens around each face of the roof top allow air flow through them. To consider the wind load transferred to the lateral system, the screens were first treated as if they were a solid face of the building. After the windward pressure was calculated on this "solid face", a multiplier of 0.5 was implemented to account for the permeability of the screen. The resulting pressure was then transferred to the building. It was also assumed that due to the permeability of the screens, no leeward pressure would develop.

The calculations for the wind pressures, loads, story shears, and overturning moments of the tower are shown in Tables 13 to 15. The pressure and loading diagrams can also be viewed in Figures 9 through 12. The analysis shows that the controlling wind loads are in the East/West direction with a base shear of 9336 kips and overturning moment of 3.7 million ft-kips. This direction was expected to control due to its wider façade face. Please note that the base shears and overturning moments calculated in this report only consider the direct loading from windward and leeward pressures. In the future, a more detailed analysis will have to be performed to consider the building response due to roof suction and side wall suction. Ideally, loading should be obtained from a wind tunnel analysis. For additional calculations as well as the wind analysis of the podium, please refer to Appendix D.

Calculated Wind Pressures in East/West Direction of Tower {Using Method 2, ASCE 7-05}							
	Height (z)	K_z [a]	q_z & q_h (psf) {$.00256K_zK_{zt}K_dV^2I$}	External Pressure (psf) {qGC_p}	Internal Pressure (psf) {q_hGC_{pi}}	Net Pressure p (psf)	
						+ (GC_{pi})	- (GC_{pi})
	15.0	0.57	17.40	14.4	9.6	4.8	23.9
	33.4	0.72	21.87	18.1	9.6	8.5	27.6
	48.9	0.81	24.39	20.1	9.6	10.6	29.7
	63.8	0.87	26.31	21.7	9.6	12.2	31.3
	77.8	0.92	27.85	23.0	9.6	13.4	32.6
	86.0*	0.95	28.66	23.7	9.6	14.1	33.2
	91.5	0.96	29.18	24.1	9.6	14.5	33.6
	105.3	1.00	30.37	25.1	9.6	15.5	34.6
	119.0	1.04	31.45	26.0	9.6	16.4	35.5
	132.8	1.07	32.45	26.8	9.6	17.2	36.3
	146.5	1.10	33.37	27.6	9.6	18.0	37.1
	160.3	1.13	34.24	28.3	9.6	18.7	37.8
	174.0	1.16	35.06	28.9	9.6	19.4	38.5
	188.4	1.18	35.86	29.6	9.6	20.0	39.2
	202.1	1.21	36.59	30.2	9.6	20.6	39.8
	215.3	1.23	37.25	30.8	9.6	21.2	40.3
	229.0	1.25	37.92	31.3	9.6	21.7	40.9
	242.8	1.27	38.55	31.8	9.6	22.3	41.4
	256.5	1.29	39.17	32.3	9.6	22.8	41.9
	270.3	1.31	39.75	32.8	9.6	23.3	42.4
	284.0	1.33	40.32	33.3	9.6	23.7	42.8
	297.8	1.35	40.87	33.7	9.6	24.2	43.3
	311.5	1.37	41.40	34.2	9.6	24.6	43.7
	325.3	1.38	41.91	34.6	9.6	25.0	44.2
	339.0	1.40	42.41	35.0	9.6	25.5	44.6
	352.8	1.42	42.90	35.4	9.6	25.9	45.0
Windward	366.5	1.43	43.37	35.8	9.6	26.2	45.4
	380.7	1.45	43.84	36.2	9.6	26.6	45.8
	401.8	1.47	44.52	36.8	9.6	27.2	46.3
	422.4	1.49	45.16	37.3	9.6	27.7	46.8
	436.1	1.51	45.58	37.6	9.6	28.1	47.2
	449.9	1.52	45.98	38.0	9.6	28.4	47.5
	463.6	1.53	46.38	38.3	9.6	28.7	47.9
	477.4	1.54	46.77	38.6	9.6	29.0	48.2
	491.1	1.56	47.15	38.9	9.6	29.4	48.5
	504.9	1.57	47.52	39.2	9.6	29.7	48.8
	518.6	1.58	47.89	39.5	9.6	30.0	49.1
	532.4	1.59	48.25	39.8	9.6	30.3	49.4
	546.1	1.61	48.60	40.1	9.6	30.6	49.7
	559.9	1.62	48.95	40.4	9.6	30.8	50.0
	573.6	1.63	49.29	40.7	9.6	31.1	50.3
	587.4	1.64	49.62	41.0	9.6	31.4	50.5
	601.1	1.65	49.95	41.2	9.6	31.7	50.8
	614.9	1.66	50.28	41.5	9.6	31.9	51.1
	628.6	1.67	50.60	41.8	9.6	32.2	51.3
	642.4	1.68	50.91	42.0	9.6	32.5	51.6
	656.1	1.69	51.22	42.3	9.6	32.7	51.8
	669.9	1.70	51.52	42.5	9.6	33.0	52.1
	683.6	1.71	51.82	42.8	9.6	33.2	52.3
	697.4	1.72	52.12	43.0	9.6	33.5	52.6
	711.5	1.73	52.42	43.3	9.6	33.7	52.8
	732.1	1.75	52.85	43.6	9.6	34.1	53.2
	745.5**	1.75	53.12	43.9	9.6	34.3	53.4
	802***	1.79	54.24	22.4	9.6	12.8	32.0
Leeward	All	---	53.12	-27.4	9.6	-37.0	-17.8
Side	All	---	53.12	-38.4	9.6	-47.9	-28.8
Roof	745.5	---	53.12	-57.0	9.6	-66.6	-47.4

* Top of Podium

** Finish Floor Elevation of Roof

*** Top of Screen Elevation (0.5 multiplier is applied to account for the ability for wind to pass through the screen.)

[a] $K_z = 2.01(15/z_g)2/\alpha$ {$z_g < 15$ft} -or- $K_z = 2.01(z/z_g)2/\alpha$ {15 ft $< z < z_g$} [T 6-2, ASCE 7-05]

Table 13: Calculated Wind Pressure

Calculated Wind Pressures in North/South Direction of Tower (Using Method 2, ASCE 7-05)							
	Height (z)	K_z[a]	q_z & q_h (psf) $\{.00256K_zK_{zt}K_dV^2I\}$	External Pressure (psf) $\{qGC_p\}$	Internal Pressure (psf) $\{q_hGC_{pi}\}$	Net Pressure p (psf)	
						+ (GC_{pi})	- (GC_{pi})
	15.0	0.57	17.40	14.6	9.6	5.0	24.2
	33.4	0.72	21.87	18.3	9.6	8.8	27.9
	48.9	0.81	24.39	20.4	9.6	10.9	30.0
	63.8	0.87	26.31	22.1	9.6	12.5	31.6
	77.8	0.92	27.85	23.4	9.6	13.8	32.9
	86.0*	0.95	28.66	24.0	9.6	14.5	33.6
	91.5	0.96	29.18	24.5	9.6	14.9	34.0
	105.3	1.00	30.37	25.5	9.6	15.9	35.0
	119.0	1.04	31.45	26.4	9.6	16.8	35.9
	132.8	1.07	32.45	27.2	9.6	17.6	36.8
	146.5	1.10	33.37	28.0	9.6	18.4	37.5
	160.3	1.13	34.24	28.7	9.6	19.2	38.3
	174.0	1.16	35.06	29.4	9.6	19.8	39.0
	188.4	1.18	35.86	30.1	9.6	20.5	39.6
	202.1	1.21	36.59	30.7	9.6	21.1	40.2
	215.3	1.23	37.25	31.2	9.6	21.7	40.8
	229.0	1.25	37.92	31.8	9.6	22.2	41.4
	242.8	1.27	38.55	32.3	9.6	22.8	41.9
	256.5	1.29	39.17	32.8	9.6	23.3	42.4
	270.3	1.31	39.75	33.3	9.6	23.8	42.9
	284.0	1.33	40.32	33.8	9.6	24.3	43.4
	297.8	1.35	40.87	34.3	9.6	24.7	43.8
	311.5	1.37	41.40	34.7	9.6	25.2	44.3
	325.3	1.38	41.91	35.1	9.6	25.6	44.7
	339.0	1.40	42.41	35.6	9.6	26.0	45.1
	352.8	1.42	42.90	36.0	9.6	26.4	45.5
Windward	366.5	1.43	43.37	36.4	9.6	26.8	45.9
	380.7	1.45	43.84	36.8	9.6	27.2	46.3
	401.8	1.47	44.52	37.3	9.6	27.8	46.9
	422.4	1.49	45.16	37.9	9.6	28.3	47.4
	436.1	1.51	45.58	38.2	9.6	28.7	47.8
	449.9	1.52	45.98	38.6	9.6	29.0	48.1
	463.6	1.53	46.38	38.9	9.6	29.3	48.5
	477.4	1.54	46.77	39.2	9.6	29.7	48.8
	491.1	1.56	47.15	39.5	9.6	30.0	49.1
	504.9	1.57	47.52	39.9	9.6	30.3	49.4
	518.6	1.58	47.89	40.2	9.6	30.6	49.7
	532.4	1.59	48.25	40.5	9.6	30.9	50.0
	546.1	1.61	48.60	40.8	9.6	31.2	50.3
	559.9	1.62	48.95	41.0	9.6	31.5	50.6
	573.6	1.63	49.29	41.3	9.6	31.8	50.9
	587.4	1.64	49.62	41.6	9.6	32.1	51.2
	601.1	1.65	49.95	41.9	9.6	32.3	51.5
	614.9	1.66	50.28	42.2	9.6	32.6	51.7
	628.6	1.67	50.60	42.4	9.6	32.9	52.0
	642.4	1.68	50.91	42.7	9.6	33.1	52.3
	656.1	1.69	51.22	43.0	9.6	33.4	52.5
	669.9	1.70	51.52	43.2	9.6	33.6	52.8
	683.6	1.71	51.82	43.5	9.6	33.9	53.0
	697.4	1.72	52.12	43.7	9.6	34.1	53.3
	711.5	1.73	52.42	44.0	9.6	34.4	53.5
	732.1	1.75	52.85	44.3	9.6	34.8	53.9
	745.5**	1.75	53.12	44.5	9.6	35.0	54.1
	819***	1.80	54.57	22.9	9.6	13.3	32.4
Leeward	All	---	53.12	-24.8	9.6	-34.4	-15.3
Side	All	---	53.12	-38.4	9.6	-47.9	-28.8
Roof	745.5	---	53.12	-57.9	9.6	-67.5	-48.3

* Top of Podium

** Finish Floor Elevation of Roof

*** Top of Screen Elevation (0.5 multiplier is applied to account for the ability for wind to pass through the screen.)

[a] $K_z = 2.01(15/z_g)2/\alpha$ $\{z_g < 15\text{ft}\}$ -or- $K_z = 2.01(z/z_g)2/\alpha$ $\{15\text{ ft} < z < z_g\}$ [T 6-2, ASCE 7-05]

Table 14: Calculated Wind Pressure

Calculated Wind Forces on Tower {Using Method 2, ASCE 7-05}							
Level	Height Above Ground (ft)	Load (kips)		Shear (kips)		Moment (ft-kips)	
		E/W	N/S	E/W	N/S	E/W	N/S
2	25.66	181	125	9154	7313	4653	22602
3	41.13	143	110	9012	7203	5867	15686
4	56.59	142	110	8870	7094	8035	15568
5	70.92	137	106	8732	6987	9733	14572
6	86.00	137	106	8595	6881	11813	14616
7	98.42	140	109	8455	6772	13777	15197
8	112.17	142	111	8313	6662	15969	15735
9	125.92	145	112	8168	6550	18203	16239
10	139.67	147	114	8022	6436	20476	16714
11	153.42	149	116	7873	6320	22784	17165
12	167.17	150	117	7723	6203	25126	17594
13	180.92	159	124	7564	6079	28680	19583
14	195.83	154	120	7411	5960	30095	18414
15	208.42	149	116	7262	5844	30963	17217
16	222.17	157	122	7105	5721	34793	19142
17	235.92	158	123	6947	5598	37277	19496
18	249.67	159	124	6788	5474	39786	19839
19	263.42	161	126	6627	5348	42319	20171
20	277.17	162	127	6465	5221	44874	20495
21	290.92	163	128	6302	5094	47452	20809
22	304.67	164	129	6138	4965	50050	21116
23	318.42	165	129	5973	4836	52670	21416
24	332.17	167	130	5806	4705	55309	21708
25	345.92	168	131	5639	4574	57968	21994
26	359.67	169	132	5470	4442	60645	22274
27	373.42	175	137	5295	4305	65272	23944
28	388.00	262	205	5033	4100	101622	53782
29	415.50	259	203	4774	3897	107549	52550
30	429.25	173	136	4601	3761	74465	23610
31	443.00	174	137	4427	3624	77246	23860
32	456.75	175	138	4251	3486	80043	24106
33	470.50	176	138	4075	3348	82856	24347
34	484.25	177	139	3898	3209	85684	24585
35	498.00	178	140	3721	3069	88526	24820
36	511.75	179	140	3542	2929	91383	25051
37	525.50	179	141	3363	2788	94254	25278
38	539.25	180	142	3182	2647	97139	25503
39	553.00	181	142	3002	2504	100038	25725
40	566.75	182	143	2820	2362	102951	25943
41	580.50	182	143	2637	2218	105876	26159
42	594.25	183	144	2454	2074	108815	26372
43	608.00	184	145	2271	1930	111766	26582
44	621.75	185	145	2086	1784	114730	26790
45	635.50	185	146	1901	1639	117707	26996
46	649.25	186	146	1715	1492	120695	27199
47	663.00	187	147	1528	1345	123696	27400
48	676.75	187	147	1341	1198	126708	27599
49	690.50	188	148	1153	1050	129732	27795
50	704.25	193	152	960	898	135997	29368
51	718.67	284	224	676	674	204265	63635
Roof	745.50	431	410	245	264	321228	176730
Screen *	802 & 819	245	264	---	---	---	---
Total	802 &819	9336	7438	9336	7438	3739561	1381094

* Loads from the screens are superimposed on to the Roof level.

Table 15: Wind Loads, Shears & Moments

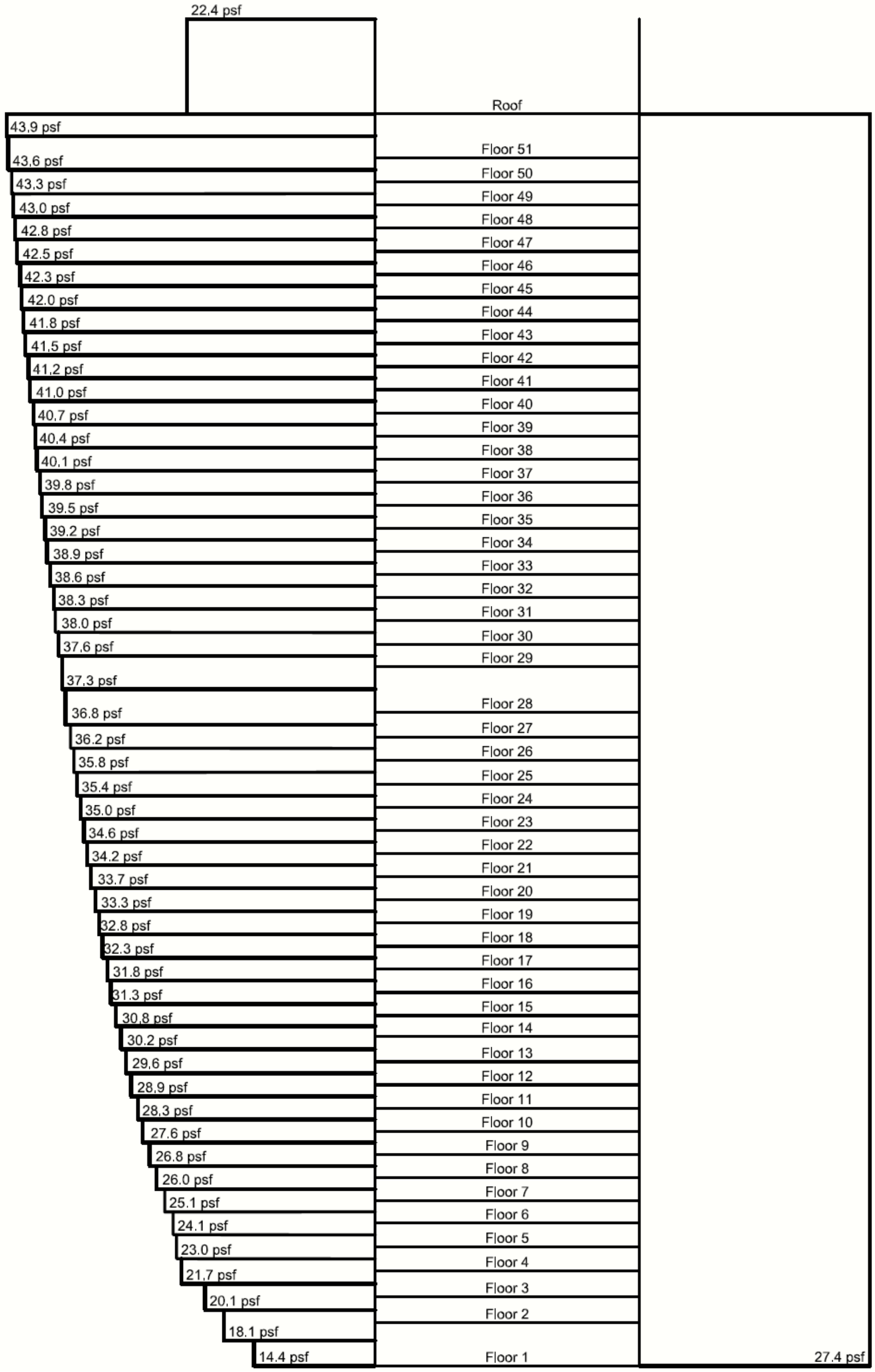


Figure 9: East/West Wind Pressure Diagram

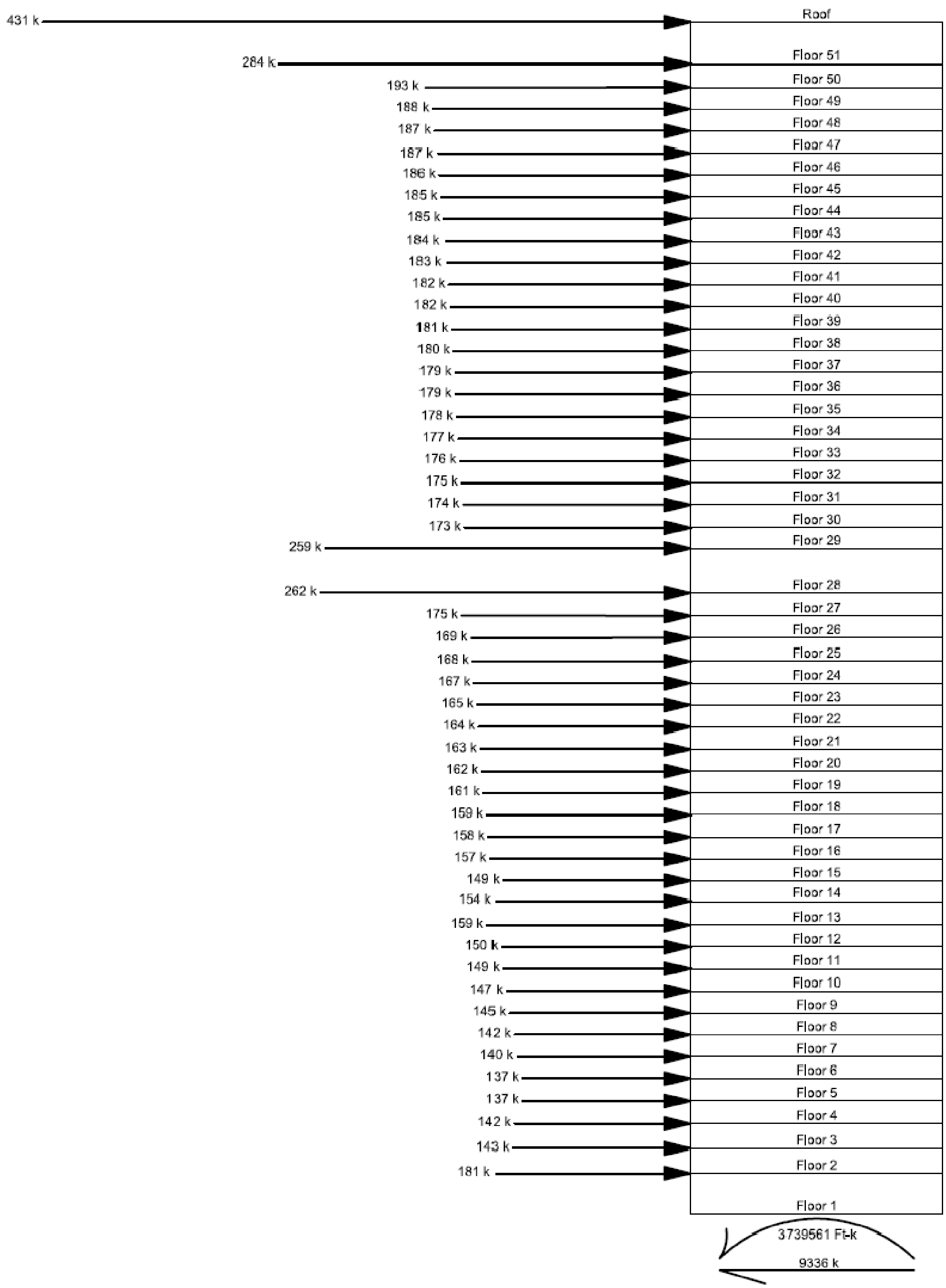


Figure 10: East/West Wind Force Diagram

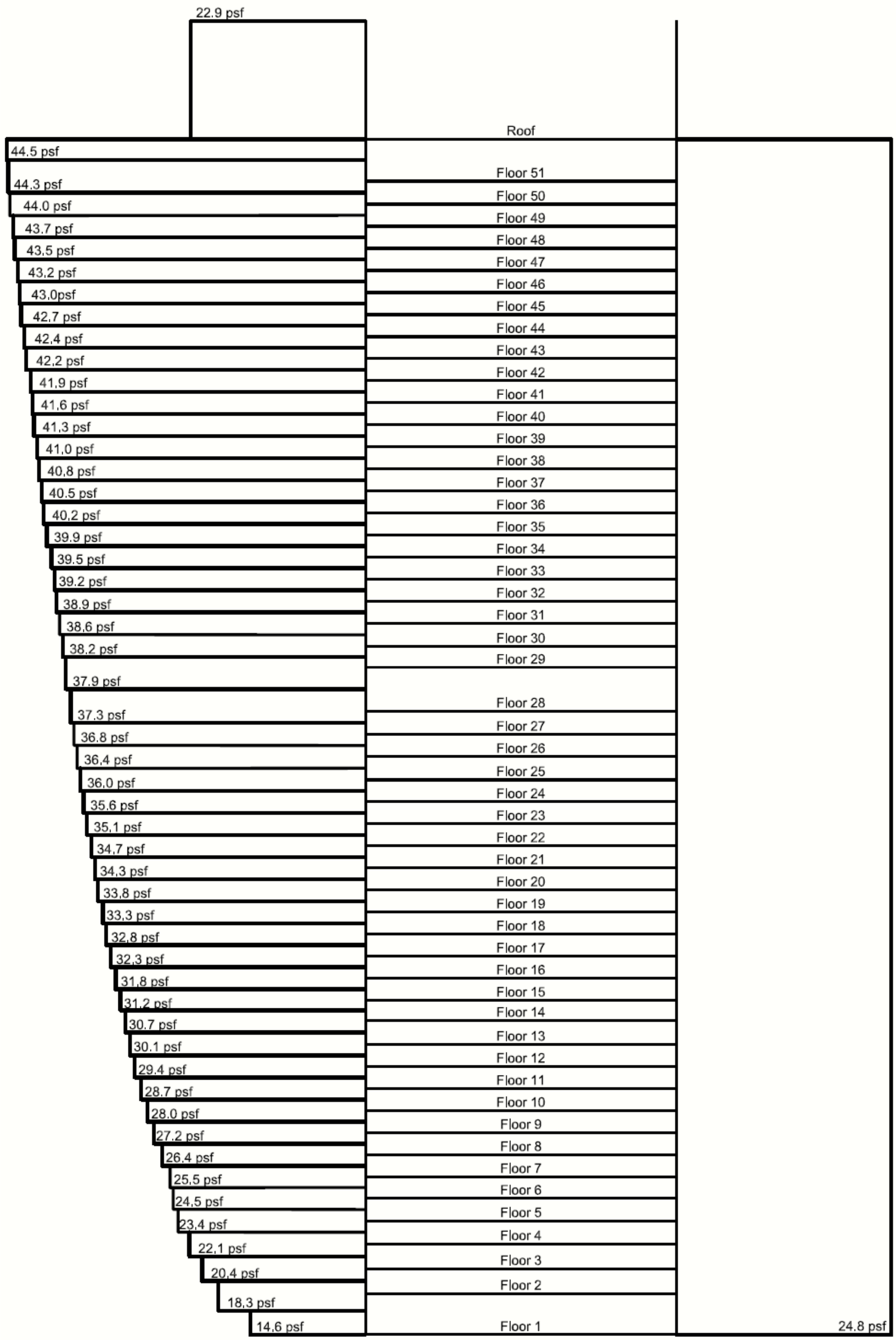


Figure 11: North/South Wind Pressure Diagram

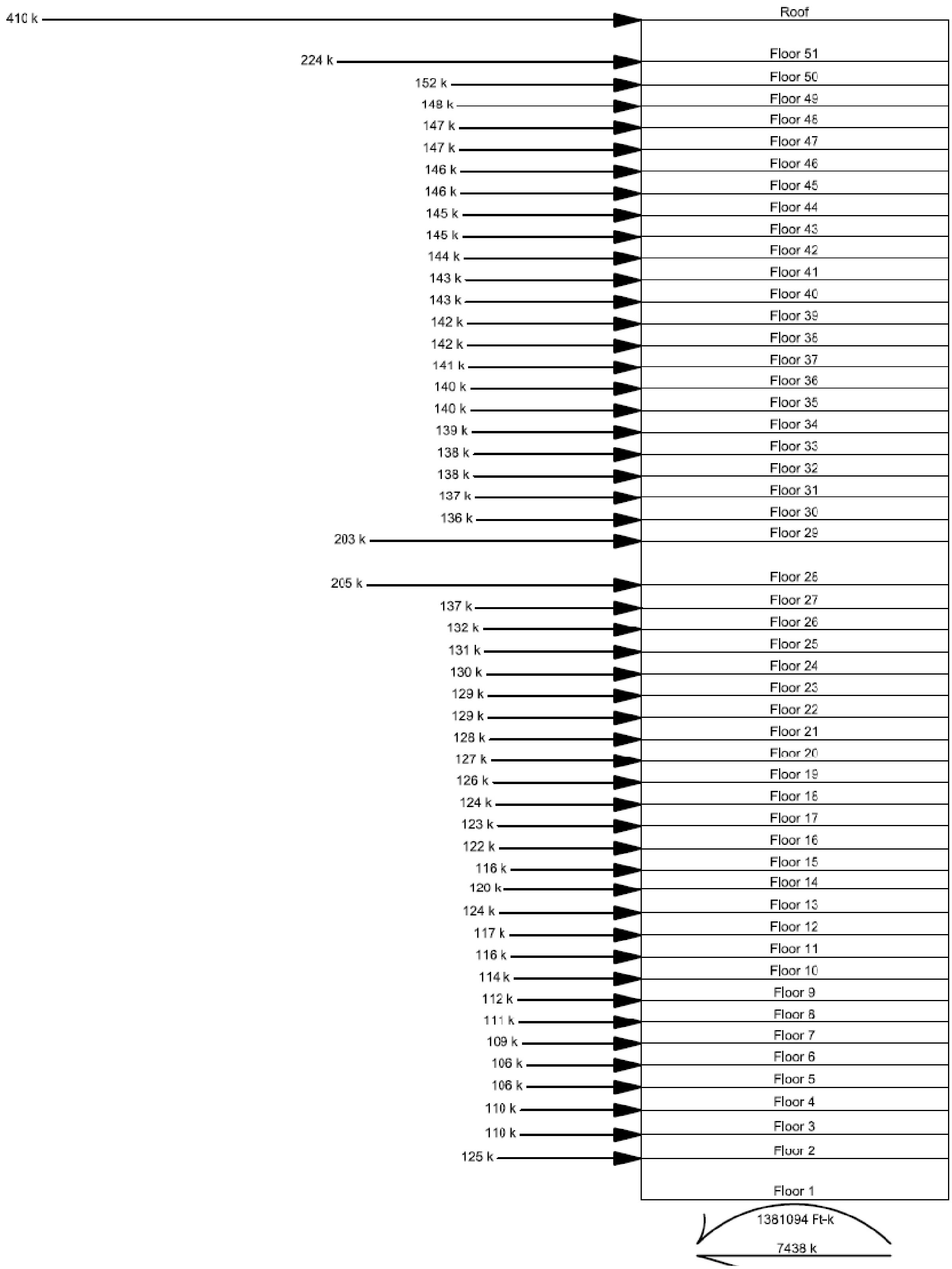


Figure 12: North/South Wind Force Diagram

Seismic Loads

To design for seismic loading conditions on the New York Times Headquarters, Thorton Tomasetti used the New York City Building Code as a basis for calculation. To convert the classification to that used in ASCE 7-05, the assumed bearing capacities and N values were compared to ASCE values. For example, the site had 40 ton per square foot rock, which is classified as Class 2-65 Medium Hard Rock in the NYC Building Code. In ASCE 7-05, Site Class A is designated as Hard Rock and Site Class B is designated as Rock. To be conservative, Class 2-65 rock was equated with Site Class B in ASCE. However, in one corner of the site the rock has a bearing capacity of only 8 tons per square foot, Class 4-65. This lower bearing capacity better equated with Site Class C in ASCE 7-05. Therefore, Site Class C was used in the analysis to be conservative.

Calculations of the design spectral response acceleration, using the USGS Ground Motion Parameter Tool and ASCE 7-05, yielded S_{DS} and S_{D1} values that corresponded to Site Class B using Tables 11.6-1 and 11.6-2, which are less conservative than those assumed from Site Class C. Therefore, the remaining seismic values were calculated using Site Class C. The base shear was determined to be 1834 kips, calculated from the effective seismic weight, including the assumed dead loads and partition loads from Tables 1, 3, and 8. The lateral seismic forces at each level increase with elevation, and range from 1.1 kips to 94 kips, as shown in Figure 13 below. The period of the building due to seismic loads was determined to be 2.902 seconds. The Response Modification Coefficient (R) used in calculations was assumed as 3.25, based on ordinary steel concentrically braced frames. This number is a bit conservative, as there is a distribution of different braced frames throughout the tower. In addition, the height of the building was increased slightly to include seismic effects above the roof level, as a contribution of the extended façade. Refer to Tables 26-29 and Figures 27 and 28 of Appendix E for calculation details.

Due to the height and location of the New York Times building, it was expected that the lateral loading due to wind pressure would control over seismic loadings. After comparing the results of the two loading conditions, it was clearly evident that this was the case.

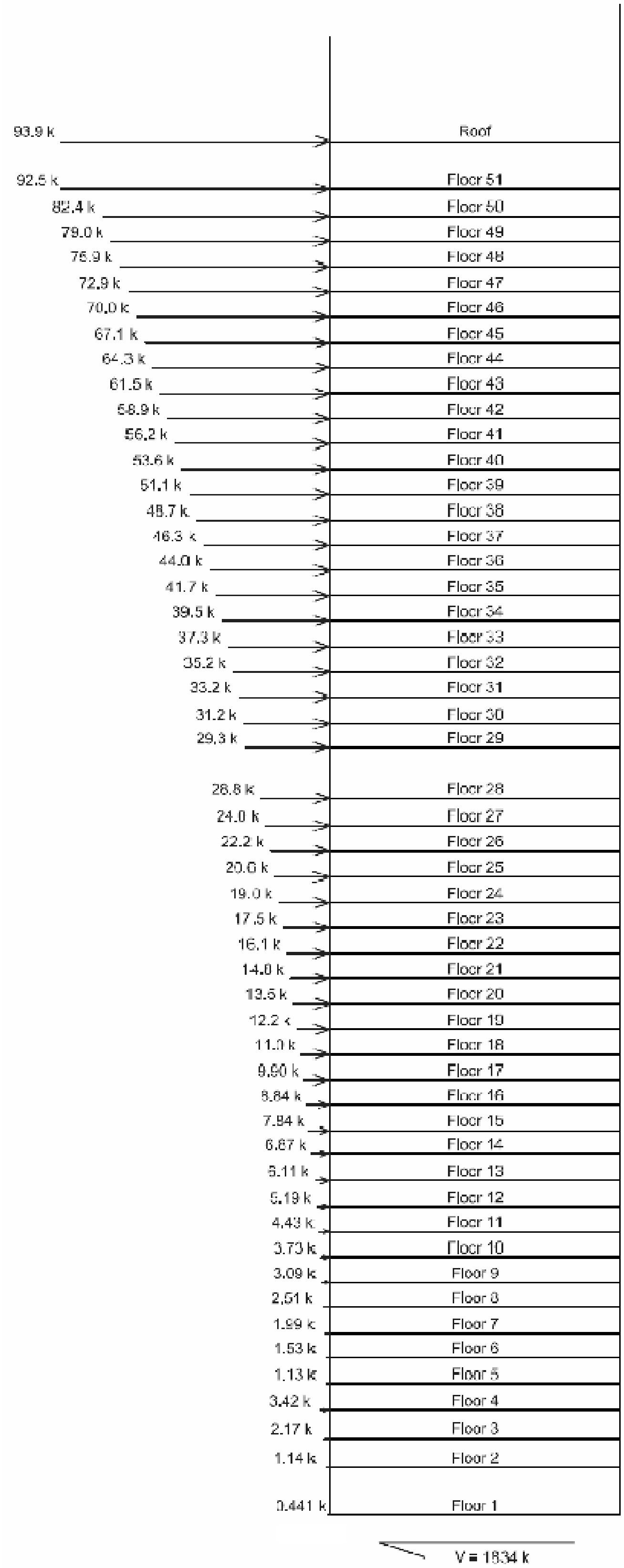


Figure 13: Lateral Seismic Forces, N/S and E/W

Miscellaneous Loads

Other miscellaneous loads were considered for the existing design of the New York Times Building and will need to be addressed in the future for this fifth year capstone project. The first condition which needs to be addressed is the thermal loading on the structure of the building, which causes deflections throughout the structure. Thornton Tomasetti designed the structure using a ΔT of -10 to 130 ˚F after consulting historical temperature data for New York City and the National Building Code of Canada. The Canadian Code was used because it provides descriptive guidelines for thermal design. In order to counter the deflections due to thermal fluctuations, thermal trusses were added to the east and west faces of the mechanical floors where outriggers where not required for lateral load resistance. These trusses improved thermal deflections to L/300. The location of these thermal trusses is highlighted in teal in Figure 14 below. In addition to thermal loadings, the design of the New York Times Buildings considered loadings due to impact and blasts. This information is confidential and will not be disclosed by the owner or the design team.

Please note that these loadings are merely mentioned in this report and were not analyzed. However, these loadings, especially those due to thermal fluctuations, must be considered and will have to be analyzed in the future.

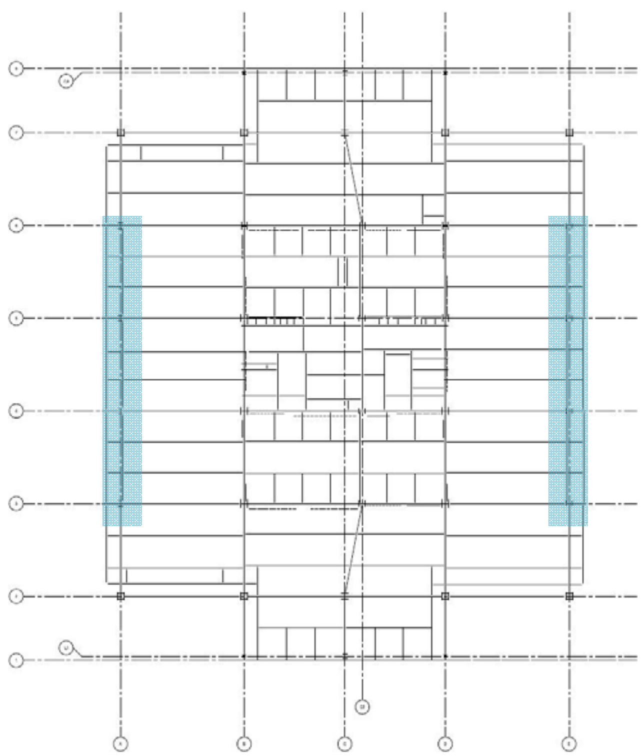


Figure 14: Thermal Truss Locations

TYPICAL FLOOR FRAMING SPOT CHECKS

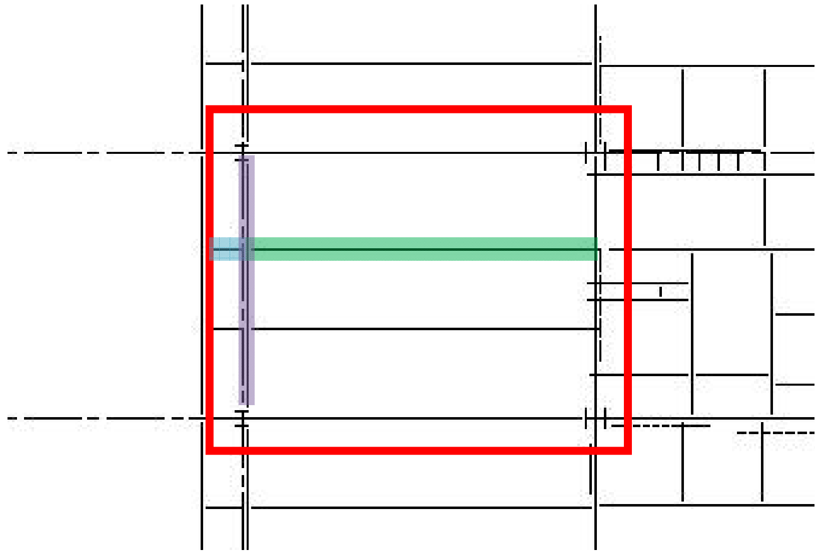


Figure 15: Typical 30'-0" by 40'-0" bay

Figure 15 shows the typical bay that was analyzed. Typical interior beams in green, W18x35 [40] c=1.5", and typical edge beams in blue, W12x19 [3], frame into the typical girder in purple, W18x40 [30] c=3/4", which in turn frames into built-up edge box columns or built-up core columns.

Metal Decking

It was determined from Thornton Tomasetti's guidance and the architectural plans that the typical office bay metal decking chosen was a 20 gage, 3 inch deep deck with yield strength of 40 ksi, with 2.5 inch of concrete topping. The following table was taken from Vulcraft page 48 for a 3 inch deep deck:

(N=9) NORMAL WEIGHT CONCRETE (145 PCF)

Total Slab Depth	Deck Type	SDI Max. Unshored Clear Span			Superimposed Live Load, PSF Clear Span (ft.-in.)														
		1 Span	2 Span	3 Span	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6	13'-0	13'-6	14'-0
	3VLI22	7'-8	9'-7	9'-7	216	195	149	133	120	109	99	90	83	76	70	64	59	54	50
5"	3VLI21	8'-11	11'-3	11'-4	230	206	187	170	128	116	106	96	88	81	74	68	63	58	54
	3VLI20	9'-6	11'-11	12'-4	241	216	196	178	163	150	111	101	93	85	78	72	66	61	57
(t=2")	3VLI19	10'-8	13'-2	13'-7	265	237	214	194	178	163	151	140	102	94	86	79	73	67	62
	3VLI18	11'-8	14'-1	14'-6	289	261	238	218	201	186	173	161	151	142	106	98	92	86	80
44 PSF	3VLI17	12'-7	14'-11	15'-5	309	278	253	231	212	196	182	170	159	150	141	133	97	91	85
	3VLI16	13'-4	15'-8	15'-11	327	294	267	243	223	206	191	178	167	156	147	139	132	96	89
	3VLI22	7'-0	8'-9	8'-9	247	190	170	152	137	124	113	103	94	87	80	73	67	62	57
5 1/2"	3VLI21	8'-4	10'-4	10'-4	262	235	213	162	146	133	120	110	101	92	85	78	72	66	61
	3VLI20	9'-0	11'-5	11'-9	275	247	223	203	186	140	127	116	106	97	89	82	76	70	65
(t=2 1/2")	3VLI19	10'-1	12'-7	13'-0	302	270	244	222	203	186	172	128	117	107	98	90	83	77	71
	3VLI18	11'-1	13'-5	13'-11	330	298	271	248	229	212	197	184	173	130	121	112	105	98	92
50 PSF	3VLI17	11'-11	14'-3	14'-9	352	317	288	263	242	224	208	194	182	171	128	119	111	104	97
	3VLI16	12'-8	15'-0	15'-5	373	335	304	277	255	235	218	203	190	178	168	159	117	109	102

Figure 16: 3" Vulcraft Metal Deck Loading Table

In Figure 16 in red, the maximum un-shored clear span for three spans is 11 feet and 9 inches. For a typical bay between beams the clear span is 9 feet, therefore the deck meets the clear span criteria. In addition to the span, the superimposed live load is 70 psf live load for office and 40 psf, dead load for office minus the self weight of the composite deck system (see Table 1: Typical Tower Floor Dead Load for loading). With the superimposed live load of 110 psf being less than 186 psf, the capacity of the deck in yellow, the deck meets all criteria and has the necessary strength needed.

Typical Composite Beam

Typical composite beam sizes are W18x35 [40] c=1.5" and W12x19 [3]. Figure 15 displays these beam locations; beams are spaced 10 feet on center and span 40 feet for the W18 and 5 feet and 4 inches for the W12. These members were checked for flexure strength, shear strength, total live load deflection, and construction dead load. The design calculations are included at the end of this report in Appendix B.

After analyzing the typical composite beams, it was found that the W18 and W12 meet all strength and serviceability requirements. It was also found the calculated shear and flexural forces in the beams were fifteen percent less than designed values. This is due to the fifteen percent increase Thornton Tomasetti added in for changes of office space and expansion of light MEP systems. For the W18 beams, the minimum partial composite strength for a neutral axis of one inch meets the requirements, but the number of shear studs is less than the design number of shear studs. Similarly, the minimum partial composite strength of the W12 beams for a neutral axis of half an inch meets the requirements, but the number of shear studs is greater than the design number of shear studs. In the case of the W18, the reason to increase shear studs could be to allow for more flexural strength and ease of constructability by placing one shear stud every foot as oppose to uneven shear stud spacing. In the case of the W12, the location of neutral axis is smaller than the assumed calculated neutral axis, which causes the number of shear studs to decrease, therefore verifying Thornton Tomasetti's results.

Typical Composite Girder

Typical composite girder size is W18x40 [30] c=3/4". Figure 15 displays the location of the girder, which spans 30 feet. This girder was checked for flexure strength, shear strength, total live load deflection, and construction dead load. The design calculations are included at the end of this report in Appendix B.

After analyzing the typical composite girder, it was found that the W18 meet all strength and serviceability requirements. As with the typical composite beams, the calculated shear and flexural forces in the girder were thirteen percent less than designed values. This could be due to the fifteen percent increase Thornton Tomasetti added in for changes of office space and expansion of light MEP systems for the composite beams. For the W18 girder the minimum partial composite strength for a neutral axis of one and a half inches meets the requirements, but the number of shear studs is more than the design number of shear studs. As with the W12, the location of neutral axis is smaller than the assumed calculated neutral axis, which causes the number of shear studs to decrease therefore verifying Thornton Tomasetti's results.

Typical Column

Typical built-up box columns used in the analysis are 30" by 30" with 4 inch flange plates and 7 inch web plates. Column load takedowns are included at the end of this report in Appendix C. In Table 16 in Appendix C, the column load takedowns include live load reduction and in Table 17 in Appendix C, the column load takedowns include unreduced live loads. The unbraced lengths of the column were determined by floor to floor heights and were assumed to be pinned at the top and bottom. At this time it is unknown if office space live load are unreduced or partially reduced; further investigation is required. The design calculations for the built-up box columns are included at the end of this report in Appendix C.

After analyzing the typical built-up box column at level 6, it was found that it meets all strength and serviceability requirements. The flexural buckling of the built-up box column controls over flexural-torsional buckling of the column, therefore only elastic flexural buckling was checked. In addition to the column meeting the requirements, it was found the column's capacity is four times greater than a factored applied load with reduced live load and is two times greater than a factored applied load with unreduced live load. This large capacity is due to the column's large cross-sectional area which could be a result of blast design in addition to the columns contributing to the tower's lateral system. As stated before, live load reduction can affect the size of the columns.

ANALYSIS & CONCLUSIONS

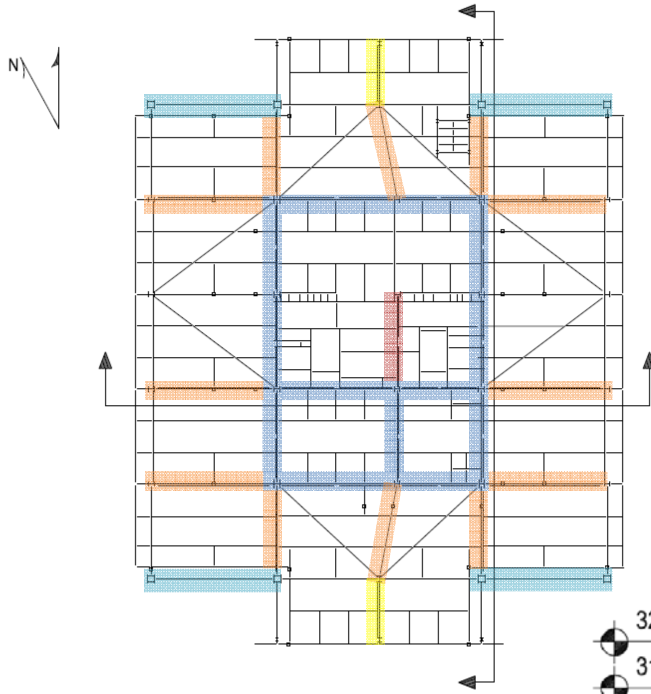
The gravity system was analyzed for dead and live loads as a confirmation of the loads used in design. The check on the beams yielded a different number of shear studs, possibly because the designers wanted to use even stud spacing or preferred a different level of composite action. The difference in results could also be due to the assumed stud strengths. In addition, inclusion of blast and progressive collapse design could influence these results. Gravity checks done for the columns showed that the sizes were larger than necessary, mostly likely because the columns were used in the lateral system to counteract the overturning moment.

Unfactored seismic and wind forces, as shown by the diagrams in Figures 10, 12, and 13, were analyzed to determine the controlling lateral loading condition. Wind base shear is approximately five times larger than seismic base shear, and wind point loads at each floor are much greater than those induced by an earthquake. This clearly indicates that wind loads control as the design lateral loading condition. In future technical reports, the lateral system will be analyzed in more detail as a check of the bracing and member sizes.

There are several other unique structural challenges that arose during design, but were outside the scope of this report. First, thermal loads were factored into the design due to the exposed structural elements and the large amount of glass in the façade. The building has the potential to expand and contract in extreme temperatures, and Thornton Tomasetti designed members to resist forces induced by these movements. The team utilized the Canadian National Building Code, which has more specific directions for temperature loads, to include thermal effects in their design. This undoubtedly had an impact on design loads, and must be considered in further detail.

In addition, there are large 20 foot cantilevers that create the cruciform shape of the tower, which were not analyzed for loads and deflections in this technical report. However, they presented a unique challenge to the designers and must also be analyzed in the future. The effects of the mast and roof screen walls were also not included in full detail in this report. Finally, the connections and subway system adjacent to the building should be studied to examine how it influenced the design of the structure and foundations.

APPENDIX A: LATERAL SYSTEMS



Key:
- Single Diagonal Bracing
- Pre-Tensioned Steel Rod X-Bracing
- Chevron & Open Knee Bracing
- Outrigger Bracing
- Single Diagonal Brace at Cantilever

Figure 17: Mechanical Floor Framing Plan (Floors 28 & 51)

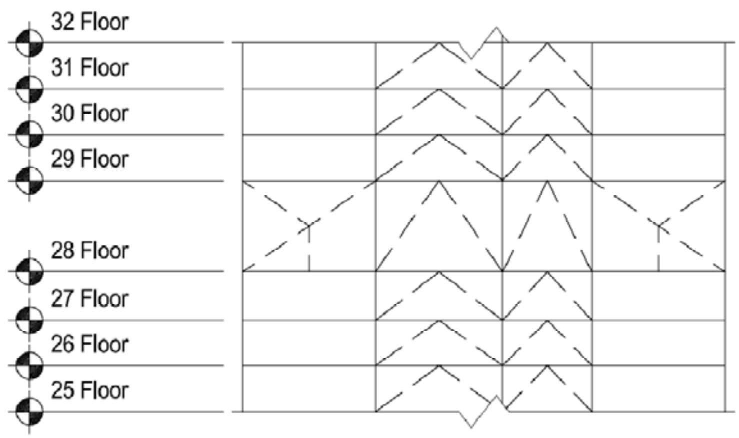


Figure 18: Typical E/W Outrigger Section (28th Floor)

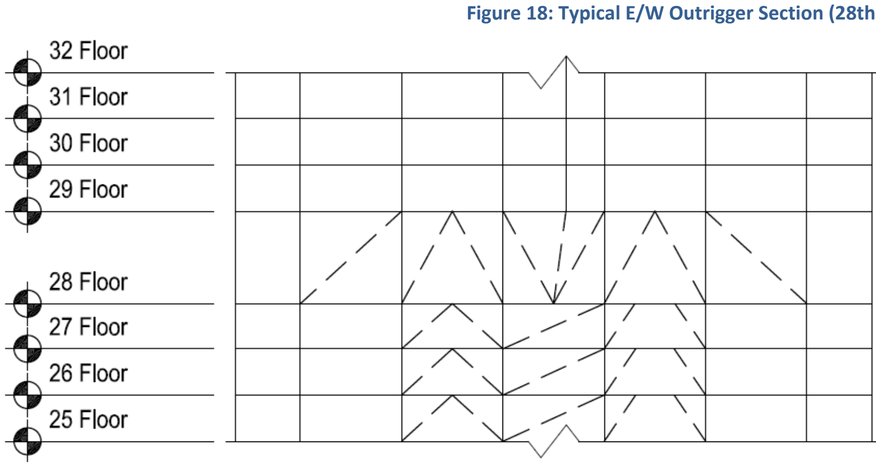


Figure 19: Typical N/S Outrigger Section (28th Floor)

APPENDIX B: TYPICAL BAY SPOT CHECKS

Typical Beam (W18x35 [40] c=1.5")				
Material Properties:				
Concrete	f'_c =	4 ksi		
Beam	F_y =	50 ksi		
	F_u =	65 ksi		
Spacing:	10.000 ft			
Span:	40.000 ft			
Loads:				
Dead Loads:				
Slab:	0.053 ksf			
Beam Weight:	0.004 ksf			
MEP/Ceiling:	0.025 ksf			
Live Loads:				
Non-Reduced:	0.070 ksf			
Total dead load:	0.815 klf			
Total live load:	0.700 klf			
Const. dead load (unshored):	0.565 klf			
Const. live load (unshored):	0.200 klf			
w_u=1.2D+1.6L=	2.098 klf			
$V_u=w_u l/2$	41.960 k			
$M_u=w_u l^2/8$	419.600 ftk			
b_{eff}=	120.000 in			
Assume a=	1.000 in			
$Y_2=t_{slab}-a/2$=	5.000 in			
Check I_{req}:				
Δ=l/240+camber=	3.500 in			
$I_{req}=5w_{CDL}l^4/(384E\Delta)$=	320.631 in^4	<	510.000 in^4	OK
Check member strength as un-shored:				
$w_{u(unshored)}$=1.2D+1.6L=	0.998 klf			
$M_{u(unshored)}=w_u l^2/8$=	199.600 ftk	<	249.000 ftk	OK
ΣQ_n=	260.000 k			
Check member strength:				
ϕM_n=	435.000 ftk	>	419.600 ftk	OK
ϕV_n=	159.000 k	>	41.960 k	OK
Check a:				
$a=\Sigma Qn/0.85f'_c b_{eff}$=	0.637 in	<	1.000 in	OK
Check Δ_{LL}:				
Δ_{LL}=l/360=	1.333 in			
$\Delta_{LL}=5w_{LL}l^4/(384EI_{LB})$=	0.119 in	<	1.333 in	OK
Check studs:				
Q_n=	17.200 kips/stud		Table 3-21	
# of studs=$\Sigma Q_n/Q_n$=	15.116 therefore use		16.000 studs/side	

Figure 20: Typical Composite W18 Analysis

Typical Beam (W12x19 [3] c=0")					
Material Properties:					
Concrete	$f'_c =$	4 ksi			
Beam	$F_y =$	50 ksi			
	$F_u =$	65 ksi			
Spacing:	10.000 ft				
Span:	5.333 ft				
Loads:					
Dead Loads:					
Slab:	0.053 ksf				
Beam Weight:	0.002 ksf				
MEP/Ceiling:	0.025 ksf				
Live Loads:					
Non-Reduced:	0.070 ksf				
Total dead load:	0.799 klf				
Total live load:	0.700 klf				
Const. dead load (unshored):	0.549 klf				
Const. live load (unshored):	0.200 klf				
w_u=1.2D+1.6L=	2.079 klf				
$V_u=w_ul/2$	5.543 k				
$M_u=w_ul^2/8$	7.391 ftk				
b_{eff}=	16.000 in				
Assume a=	0.500 in				
$Y_2=t_{slab}-a/2$=	5.250 in				
Check I_{req}:					
$\Delta=l/240$+camber=	0.267 in				
$I_{req}=5w_{CDL}l^4/(384E\Delta)$=	1.292 in^4	<	96.300 in^4	OK	
Check member strength as un-shored:					
$w_{u(unshored)}$=1.2D+1.6L=	0.979 klf				
$M_{u(unshored)}=w_ul^2/8$=	3.480 ftk	<	249.000 ftk	OK	
ΣQ_n=	69.700 k				
Check member strength:					
ϕM_n=	144.000 ftk	>	7.391 ftk	OK	
ϕV_n=	85.700 k	>	5.543 k	OK	
Check a:					
$a=\Sigma Qn/0.85f'_cb_{eff}$=	0.107 in	<	0.500 in	OK	
Check Δ_{LL}:					
$\Delta_{LL}=l/360$=	0.178 in				
$\Delta_{LL}=5w_{LL}l^4/(384EI_{LB})$=	0.0002 in	<	0.178 in	OK	
Check studs:					
Q_n=	17.200 kips/stud		Table 3-21		
# of studs=$\Sigma Q_n/Q_n$=	4.052 therefore use		5.000 studs/side		

Figure 21: Typical Composite W12 Analysis

Typical Girder (W18x40 [30] c=3/4")					
Material Properties:					
Concrete	f'_c =	4 ksi			
Beam	F_y =	50 ksi			
	F_u =	65 ksi			
Span:	30.000 ft				
Loads:					
Dead Loads:					
P_{W18x35}:	16.300 k				
P_{W12x19}:	2.131 k				
Beam Weight:	0.040 klf				
Live Loads:					
P_{W18x35}:	14.000 k				
P_{W12x19}:	1.867 k				
Total dead load (P_u):	18.431 k				
Total dead load (w_u):	0.040 klf				
Total live load(P_u):	15.867 k				
Const. dead load (unshored):	12.764 k				
Const. dead load (unshored):	0.040 klf				
Const. live load (unshored):	0.533 k				
P_u=1.2D+1.6L=	47.503 k				
w_u=1.2D+1.6L=	0.048 klf				
V_u=w_ul/2+P_u=	48.223 k				
M_u=$w_u l^2$/8+P_ul/3	480.435 ftk				
b_{eff}=	90.000 in				
Assume a=	1.500 in				
Y_2=t_{slab}-a/2=	4.750 in				
Check I_{req}:					
Δ=l/240+camber=	2.250 in				
I_{req}=5$w_{DL}l^4$/(384EΔ)+$P_{DL}l^3$/(28EΔ)=	337.126 in^4	<	612.000 in^4	OK	
Check member strength as un-shored:					
$P_{u(unshored)}$=1.2D+1.6L=	16.170 k				
$w_{u(unshored)}$=1.2D+1.6L=	0.048 klf				
$M_{u(unshored)}$=$w_u l^2$/8+P_ul/3=	167.101 ftk	<	294.000 ftk	OK	
ΣQ_n=	351.000 k				
Check member strength:					
φM_n=	516.500 ftk	>	480.435 ftk	OK	
φV_n=	159.000 k	>	48.223 k	OK	
Check a:					
a=ΣQn/0.85$f'_c b_{eff}$=	1.147 in	<	1.500 in	OK	
Check Δ_{LL}:					
Δ_{LL}=l/360=	1.000 in				
Δ_{LL}=$P_{LL}l^3$/(28EI_{LB})=	0.680 in	<	1.000 in	OK	
Check studs:					
Q_n=	17.200 kips/stud		Table 3-21		
# of studs=ΣQ_n/Q_n=	20.407 therefore use		21.000 studs/side		

Figure 22: Typical Composite Girder

APPENDIX C: TYPICAL COLUMN CHECKS

COLUMN A4 (Located in Office Area) LL Reduction															
Story level	Column Below Level	Tributary Area (ft^2)	Live Load Influence Area (ft^2)	Live Load Reduction Factor	Dead Load (kips)	Wall Load (kips)	Roof Live Load (kips)	Snow Load (kips)	Floor Live Load (kips)	Column Load 1.4D (kips)	Column Load 1.2D+1.6L+.5(Lr or S) (kips)	Column Load 1.2D+1.6(Lr or S)+L (kips)	Column Load 1.2D+L+0.5(Lr or S) (kips)	Column Load 1.2D+L+0.2S (kips)	Column Design Load
52 Roof	1	765	0	1.00	76.48	28.41	76.48	13.38		146.85	164.11	248.25	164.11	128.55	248.25
51 Mech	2	1530	2730	1.00	147.61	38.97			95.61	261.22	415.11	441.88	357.75	322.19	441.88
50	3	2295	5459	0.45	218.75	44.69			119.86	368.81	546.14	558.36	474.23	438.66	558.36
49	4	3059	8189	0.42	289.88	50.05			142.12	475.90	673.55	672.41	588.28	552.71	673.55
48	5	3824	10919	0.40	361.01	55.42			163.53	582.99	799.60	785.62	701.48	665.92	799.60
47	6	4589	13648	0.40	432.14	60.78			184.95	690.08	925.66	898.82	814.69	779.13	925.66
46	7	5354	16378	0.40	503.27	66.14			206.37	797.17	1051.72	1012.03	927.90	892.33	1051.72
45	8	6119	19108	0.40	574.40	71.50			227.78	904.26	1177.77	1125.24	1041.11	1005.54	1177.77
44	9	6884	21838	0.40	645.53	76.87			249.20	1011.35	1303.83	1238.45	1154.31	1118.75	1303.83
43	10	7648	24567	0.40	716.66	82.23			270.61	1118.44	1429.89	1351.65	1267.52	1231.95	1429.89
42	11	8413	27297	0.40	787.79	87.59			292.03	1225.53	1555.94	1464.86	1380.73	1345.16	1555.94
41	12	9178	30027	0.40	858.92	92.95			313.44	1332.62	1682.00	1578.07	1493.93	1458.37	1682.00
40	13	9943	32756	0.40	930.05	98.32			334.86	1439.71	1808.06	1691.27	1607.14	1571.58	1808.06
39	14	10708	35486	0.40	1001.18	103.68			356.27	1546.80	1934.11	1804.48	1720.35	1684.78	1934.11
38	15	11473	38216	0.40	1072.31	109.04			377.69	1653.89	2060.17	1917.69	1833.56	1797.99	2060.17
37	16	12238	40945	0.40	1143.44	114.40			399.11	1760.98	2186.23	2030.90	1946.76	1911.20	2186.23
36	17	13002	43675	0.40	1214.57	119.77			420.52	1868.07	2312.28	2144.10	2059.97	2024.40	2312.28
35	18	13767	46405	0.40	1285.70	125.13			441.94	1975.16	2438.34	2257.31	2173.18	2137.61	2438.34
34	19	14532	49134	0.40	1356.83	130.49			463.35	2082.25	2564.40	2370.52	2286.38	2250.82	2564.40
33	20	15297	51864	0.40	1427.96	135.85			484.77	2189.34	2690.45	2483.72	2399.59	2364.03	2690.45
32	21	16062	54594	0.40	1499.09	141.22			506.18	2296.43	2816.51	2596.93	2512.80	2477.23	2816.51
31	22	16827	57323	0.40	1570.22	146.58			527.60	2403.52	2942.57	2710.14	2626.01	2590.44	2942.57
30	23	17591	60053	0.40	1641.35	151.94			549.02	2510.61	3068.62	2823.35	2739.21	2703.65	3068.62
29	24	18356	62783	0.40	1712.49	157.30			570.43	2617.70	3194.68	2936.55	2852.42	2816.86	3194.68
28 Mech	25	19121	65513	1.00	1783.62	168.03			666.04	2732.30	3445.87	3130.39	3046.25	3010.69	3445.87
27	26	19886	68242	0.40	1854.75	173.20			687.45	2839.12	3571.70	3243.36	3159.23	3123.66	3571.70
26	27	20651	70972	0.40	1925.88	178.56			708.87	2946.21	3697.75	3356.57	3272.43	3236.87	3697.75
25	28	21416	73702	0.40	1997.01	183.92			730.28	3053.30	3823.81	3469.77	3385.64	3350.07	3823.81
24	29	22180	76431	0.40	2068.14	189.28			751.70	3160.39	3949.87	3582.98	3498.85	3463.28	3949.87
23	30	22945	79161	0.40	2139.27	194.65			773.11	3267.48	4075.92	3696.19	3612.05	3576.49	4075.92
22	31	23710	81891	0.40	2210.40	200.01			794.53	3374.57	4201.98	3809.39	3725.26	3689.70	4201.98
21	32	24475	84620	0.40	2281.53	205.37			815.95	3481.66	4328.04	3922.60	3838.47	3802.90	4328.04
20	33	25240	87350	0.40	2352.66	210.73			837.36	3588.75	4454.09	4035.81	3951.68	3916.11	4454.09
19	34	26005	90080	0.40	2423.79	216.10			858.78	3695.84	4580.15	4149.02	4064.88	4029.32	4580.15
18	35	26770	92809	0.40	2494.92	221.46			880.19	3802.93	4706.21	4262.22	4178.09	4142.52	4706.21
17	36	27534	95539	0.40	2566.05	226.82			901.61	3910.02	4832.26	4375.43	4291.30	4255.73	4832.26
16	37	28299	98269	0.40	2637.18	232.18			923.02	4017.11	4958.32	4488.64	4404.50	4368.94	4958.32
15	38	29064	100998	0.40	2708.31	237.55			944.44	4124.20	5084.38	4601.84	4517.71	4482.15	5084.38
14 Cafeteria	39	29829	103728	1.00	2779.44	242.91			1020.92	4231.29	5298.54	4770.12	4685.99	4650.42	5298.54
13	40	30594	106458	0.40	2850.57	248.27			1042.34	4338.38	5424.60	4883.33	4799.19	4763.63	5424.60
12	41	31359	109188	0.40	2921.70	253.44			1063.76	4445.20	5550.42	4996.30	4912.17	4876.60	5550.42
11	42	32123	111917	0.40	2992.83	259.00			1085.17	4552.56	5676.71	5109.74	5025.61	4990.04	5676.71
10	43	32888	114647	0.40	3063.96	264.36			1106.59	4659.65	5802.77	5222.95	5138.82	5103.25	5802.77
9	44	33653	117377	0.40	3135.09	269.72			1128.00	4766.74	5928.82	5336.16	5252.02	5216.46	5928.82
8	45	34418	120106	0.40	3206.23	275.08			1149.42	4873.83	6054.88	5449.36	5365.23	5329.67	6054.88
7	46	35183	122836	0.40	3277.36	280.45			1170.83	4980.92	6180.94	5562.57	5478.44	5442.87	6180.94
6	47	35948	125566	0.40	3348.49	285.81			1192.25	5088.01	6306.99	5675.78	5591.65	5556.08	6306.99
5	48	36713	128295	0.40	3419.62	291.17			1213.66	5195.10	6433.05	5788.99	5704.85	5669.29	6433.05
4	49	37477	131025	0.40	3490.75	296.76			1235.08	5302.51	6559.38	5902.46	5818.33	5782.76	6559.38
3	50	38242	133755	0.40	3561.88	302.79			1256.50	5410.53	6686.24	6016.47	5932.34	5896.77	6686.24
2	51	39007	136484	0.40	3633.01	313.32			1277.91	5524.85	6818.49	6135.88	6051.74	6016.18	6818.49
1	52	39772	139214	0.40	3704.14	319.56			1299.33	5633.17	6945.60	6250.14	6166.00	6130.44	6945.60

Table 16: Column A4 load takedowns with LL reduction

COLUMN A4 (Located in Office Area) No LL Reduction															
Story level	Column Below Level	Tributary Area (ft^2)	Live Load Influence Area (ft^2)	Live Load Reduction Factor	Dead Load (kips)	Wall Load (kips)	Roof Live Load (kips)	Snow Load (kips)	Floor Live Load (kips)	Column Load 1.4D (kips)	Column Load 1.2D+1.6L+.5(Lr or S) (kips)	Column Load 1.2D+1.6(Lr or S)+L (kips)	Column Load 1.2D+L+0.5(Lr or S) (kips)	Column Load 1.2D+L+0.2S (kips)	Column Design Load
52 Roof	1	765	0	1.00	76.48	28.41	95.61	13.38		146.85	164.11	248.25	164.11	128.55	**248.25**
51 Mech	2	1530	2730	1.00	147.61	38.97			76.48	261.22	384.52	422.76	338.63	303.07	**422.76**
50	3	2295	5459	1.00	218.75	44.69			152.97	368.81	599.12	591.47	507.34	471.77	**599.12**
49	4	3059	8189	1.00	289.88	50.05			229.45	475.90	813.28	759.74	675.61	640.05	**813.28**
48	5	3824	10919	1.00	361.01	55.42			305.94	582.99	1027.45	928.02	843.89	808.32	**1027.45**
47	6	4589	13648	1.00	432.14	60.78			382.42	690.08	1241.62	1096.30	1012.16	976.60	**1241.62**
46	7	5354	16378	1.00	503.27	66.14			458.91	797.17	1455.78	1264.57	1180.44	1144.87	**1455.78**
45	8	6119	19108	1.00	574.40	71.50			535.39	904.26	1669.95	1432.85	1348.71	1313.15	**1669.95**
44	9	6884	21838	1.00	645.53	76.87			611.88	1011.35	1884.12	1601.12	1516.99	1481.43	**1884.12**
43	10	7648	24567	1.00	716.66	82.23			688.36	1118.44	2098.28	1769.40	1685.27	1649.70	**2098.28**
42	11	8413	27297	1.00	787.79	87.59			764.84	1225.53	2312.45	1937.68	1853.54	1817.98	**2312.45**
41	12	9178	30027	1.00	858.92	92.95			841.33	1332.62	2526.62	2105.95	2021.82	1986.25	**2526.62**
40	13	9943	32756	1.00	930.05	98.32			917.81	1439.71	2740.78	2274.23	2190.09	2154.53	**2740.78**
39	14	10708	35486	1.00	1001.18	103.68			994.30	1546.80	2954.95	2442.50	2358.37	2322.81	**2954.95**
38	15	11473	38216	1.00	1072.31	109.04			1070.78	1653.89	3169.12	2610.78	2526.65	2491.08	**3169.12**
37	16	12238	40945	1.00	1143.44	114.40			1147.27	1760.98	3383.28	2779.06	2694.92	2659.36	**3383.28**
36	17	13002	43675	1.00	1214.57	119.77			1223.75	1868.07	3597.45	2947.33	2863.20	2827.63	**3597.45**
35	18	13767	46405	1.00	1285.70	125.13			1300.23	1975.16	3811.61	3115.61	3031.47	2995.91	**3811.61**
34	19	14532	49134	1.00	1356.83	130.49			1376.72	2082.25	4025.78	3283.88	3199.75	3164.18	**4025.78**
33	20	15297	51864	1.00	1427.96	135.85			1453.20	2189.34	4239.95	3452.16	3368.03	3332.46	**4239.95**
32	21	16062	54594	1.00	1499.09	141.22			1529.69	2296.43	4454.11	3620.43	3536.30	3500.74	**4454.11**
31	22	16827	57323	1.00	1570.22	146.58			1606.17	2403.52	4668.28	3788.71	3704.58	3669.01	**4668.28**
30	23	17591	60053	1.00	1641.35	151.94			1682.66	2510.61	4882.45	3956.99	3872.85	3837.29	**4882.45**
29	24	18356	62783	1.00	1712.49	157.30			1759.14	2617.70	5096.61	4125.26	4041.13	4005.56	**5096.61**
28 Mech	25	19121	65513	1.00	1783.62	168.03			1835.63	2732.30	5317.22	4299.97	4215.84	4180.28	**5317.22**
27	26	19886	68242	1.00	1854.75	173.20			1912.11	2839.12	5531.15	4468.02	4383.88	4348.32	**5531.15**
26	27	20651	70972	1.00	1925.88	178.56			1988.59	2946.21	5745.31	4636.29	4552.16	4516.59	**5745.31**
25	28	21416	73702	1.00	1997.01	183.92			2065.08	3053.30	5959.48	4804.57	4720.43	4684.87	**5959.48**
24	29	22180	76431	1.00	2068.14	189.28			2141.56	3160.39	6173.65	4972.84	4888.71	4853.15	**6173.65**
23	30	22945	79161	1.00	2139.27	194.65			2218.05	3267.48	6387.81	5141.12	5056.99	5021.42	**6387.81**
22	31	23710	81891	1.00	2210.40	200.01			2294.53	3374.57	6601.98	5309.40	5225.26	5189.70	**6601.98**
21	32	24475	84620	1.00	2281.53	205.37			2371.02	3481.66	6816.15	5477.67	5393.54	5357.97	**6816.15**
20	33	25240	87350	1.00	2352.66	210.73			2447.50	3588.75	7030.31	5645.95	5561.81	5526.25	**7030.31**
19	34	26005	90080	1.00	2423.79	216.10			2523.98	3695.84	7244.48	5814.22	5730.09	5694.53	**7244.48**
18	35	26770	92809	1.00	2494.92	221.46			2600.47	3802.93	7458.65	5982.50	5898.37	5862.80	**7458.65**
17	36	27534	95539	1.00	2566.05	226.82			2676.95	3910.02	7672.81	6150.77	6066.64	6031.08	**7672.81**
16	37	28299	98269	1.00	2637.18	232.18			2753.44	4017.11	7886.98	6319.05	6234.92	6199.35	**7886.98**
15	38	29064	100998	1.00	2708.31	237.55			2829.92	4124.20	8101.15	6487.33	6403.19	6367.63	**8101.15**
14 Cafeteria	39	29829	103728	1.00	2779.44	242.91			2906.41	4231.29	8315.31	6655.60	6571.47	6535.90	**8315.31**
13	40	30594	106458	1.00	2850.57	248.27			2982.89	4338.38	8529.48	6823.88	6739.75	6704.18	**8529.48**
12	41	31359	109188	1.00	2921.70	253.44			3059.38	4445.20	8743.41	6991.92	6907.79	6872.22	**8743.41**
11	42	32123	111917	1.00	2992.83	259.00			3135.86	4552.56	8957.81	7160.43	7076.30	7040.73	**8957.81**
10	43	32888	114647	1.00	3063.96	264.36			3212.34	4659.65	9171.98	7328.71	7244.57	7209.01	**9171.98**
9	44	33653	117377	1.00	3135.09	269.72			3288.83	4766.74	9386.15	7496.98	7412.85	7377.28	**9386.15**
8	45	34418	120106	1.00	3206.23	275.08			3365.31	4873.83	9600.31	7665.26	7581.13	7545.56	**9600.31**
7	46	35183	122836	1.00	3277.36	280.45			3441.80	4980.92	9814.48	7833.53	7749.40	7713.84	**9814.48**
6	47	35948	125566	1.00	3348.49	285.81			3518.28	5088.01	10028.65	8001.81	7917.68	7882.11	**10028.65**
5	48	36713	128295	1.00	3419.62	291.17			3594.77	5195.10	10242.81	8170.09	8085.95	8050.39	**10242.81**
4	49	37477	131025	1.00	3490.75	296.76			3671.25	5302.51	10457.25	8338.63	8254.50	8218.93	**10457.25**
3	50	38242	133755	1.00	3561.88	302.79			3747.73	5410.53	10672.22	8507.71	8423.58	8388.01	**10672.22**
2	51	39007	136484	1.00	3633.01	313.32			3824.22	5524.85	10892.58	8682.18	8598.05	8562.48	**10892.58**
1	52	39772	139214	1.00	3704.14	319.56			3900.70	5633.17	11107.80	8851.51	8767.38	8731.81	**11107.80**

Table 17: Column A4 load takedowns without LL reduction

Built-up Exterior Box Column

Material Properties:

Beam	F_y =	50	ksi
	F_u =	65	ksi

Geometric Properties:

d=	30	in
b_f=	30	in
t_f=	4	in
h=	22	in
t_w=	7	in

Calculate built-up section properties (ignoring fillet welds):

A_1=	120	in^2	I_{xx1}=	20440	in^4
A_2=	154	in^2	I_{xx2}=	6211	in^4
A_3=	154	in^2	I_{xx3}=	6211	in^4
A_4=	120	in^2	I_{xx4}=	20440	in^4
A_g=	548	in^2	I_{yy1}=	9000	in^4
NA_{xx}=	15	in	I_{yy2}=	11755	in^4
I_{xxT}=	53,302.67	in^4	I_{yy3}=	11755	in^4
NA_{yy}=	15	in	I_{yy4}=	9000	in^4
I_{yyT}=	41,510.67	in^4			
r_x=	9.862	in			
r_y=	8.703	in			

Check Slenderness ratio:

K_x=K_y=	1 Column is pinned in x-direction			
L_x=L_y=	165 in			
$KL/r \leq 200$				
K_xL_x/r_x=	16.73	<	200	**OK**
K_yL_y/r_y=	18.96	<	200	**OK**

Calculate the elastic flexural buckling stress:

Since the unbraced length is the same for both axes, the y-y axis will govern by inspection.

K_yL_y/r_y= 18.96 < $4.71(E/F_y)^{1/2}$ = 133.68 **Fcr=[0.658^(Fy/Fe)]Fy**

$F_e=\pi^2E/(KL/r)^2$= 796.36 ksi

$F_{cr}=[0.658^{Fy/Fe}]F_y$= 48.70 ksi

Torsional buckling will not govern since $KL_y > KL_z$,therefore no need to check elastic critical torsional buckling stress

$\phi P_n=0.9F_{cr}A_g$=	24,020.40	k	>	6,433.05	k	**OK**
(Neglecting LL Reduction)						
$\phi P_n=0.9F_{cr}A_g$=	24,020.40	k	>	10,242.81	k	**OK**

Figure 23: Built-up Exterior Box Column Analysis

APPENDIX D: WIND ANALYSIS

Method 2 Wind Load Design Variables			
Variable	Value	Unit	Reference
V	110	miles/hr	ASCE 7-05 6.5.4
K_d	0.85	---	ASCE 7-05 6.5.4.4
Occupancy Cat.	III	---	IBC Table 1604.5
I	1.15	---	ASCE 7-05 6.5.5
Surf. Rough. Cat.	B	---	ASCE 7-05 6.5.2
Exp. Cat.	B	---	ASCE 7-05 6.5.6
K_{zt}	1	---	ASCE 7-05 6.5.7
α	7.0	---	ASCE 7-05 6.5.6.6
z_g	1200	---	ASCE 7-05 6.5.6.6

Table 18: Wind Load Design Variables

Gust Factor {Tower}						
Variable	Equation	Direction E/W	Direction N/S	Unit	Reference (ASCE 7)	Comments
$n_1 (f_{n1})$	150/h	0.20121	0.20121	---	C6.5.8	Flexible Structure
gQ = gv	---	3.4	3.4	---	6.5.8.2	
g_r	$(2LN(3600n_1))^{1/2}+(0.577/(2LN(3600n_1))^{1/2}$	3.7881	3.7881	---	6.5.8.2	
h	---	745.5	745.5	ft		
z bar	.6h	447.3	447.3	ft		
z_{min}	---	30	30	ft	Table 6-2	z bar $\geq z_{min}$ (ok)
c	---	0.3	0.3	---	Table 6-2	
I_z	$c(33/z)^{1/6}$	0.1943	0.1943	---	6.5.8.1	
ℓ	---	320	320	ft	Table 6-2	
ε	---	0.3333	0.3333	---	Table 6-2	
L_z	$\ell(z/33)^{\varepsilon}$	762.98	762.98	ft	6.5.8.1	
B	---	194.00	157.00	ft		
L	---	157.00	194.00	ft		
Q	$(1/(1+0.63((B+h)/L_z)^{0.63})^{1/2}$	0.76288	0.76690	---	6.5.8.1	
V	---	110	110	miles/hr	6.5.4	
b bar	---	0.45	0.45	---	Table 6-2	
α bar	---	0.25	0.25	---	Table 6-2	
V_z	$b(z/33)^{\alpha}V(88/60)$	139.3022	139.3022	ft/s	6.5.8.2	
N_1	n_1L_z/V_z	1.1020	1.1020	---	6.5.8.2	
R_n	$7.47N_1/(1+10.3N_1)^{5/3}$	0.12474	0.12474	---	6.5.8.2	
η (R_h)	$4.6n_1h/V_z$	4.9533	4.9533	---	6.5.8.2	
R_h	$1/\eta - (1/2\eta^2)(1-e^{-2\eta})$	0.18151	0.18151	---	6.5.8.2	
η (R_B)	$4.6n_1B/V_z$	1.2890	1.0431	---	6.5.8.2	
R_B	$1/\eta - (1/2\eta^2)(1-e^{-2\eta})$	0.49772	0.55619	---	6.5.8.2	
η (R_L)	$15.4n_1L/V_z$	3.4923	4.3153	---	6.5.8.2	
R_L	$1/\eta - (1/2\eta^2)(1-e^{-2\eta})$	0.24539	0.20489	---	6.5.8.2	
β	---	0.01	0.01	---	C6.5.8	
R	$((1/\beta)(R_nR_hR_B(.53+0.47R_L)))^{1/2}$	0.852786	0.888092	---	6.5.8.2	
G_f	$0.925(1+1.7I_z(g_Q{}^2Q^2+g_R{}^2R^2)^{1/2})/(1+1.7g_vI_z)$	1.032	1.048	---	6.5.8.2	

Table 19: Tower Gust Factor

E/W Wind Direction (Tower) {h/L >1.0 & q < 10}			
L/B	Wall Pressure Coeff. (Cp)		
	Windward	Leeward	Side
0.809	0.8	-0.5	-0.7
h/L	Roof Pressure Coeff. (Cp)		
	Roof Area (ft^2)	Reduction	Cp
4.748	27400	0.8	-1.040

Internal Pressure	
GC_{pi}	0.18
	-0.18

[F 6-5, ASCE 7-05]

Table 20: Tower E/W Wind Pressure Coefficients

N/S Wind Direction (Tower) {h/L >1.0 & θ < 10}			
L/B	Wall Pressure Coeff. (Cp)		
	Windward	Leeward	Side
1.236	0.8	-0.453	-0.7
h/L	Roof Pressure Coeff. (Cp)		
	Roof Area (ft^2)	Reduction	Cp
3.843	27400	0.8	-1.040

Internal Pressure	
GC_{pi}	0.18
	-0.18

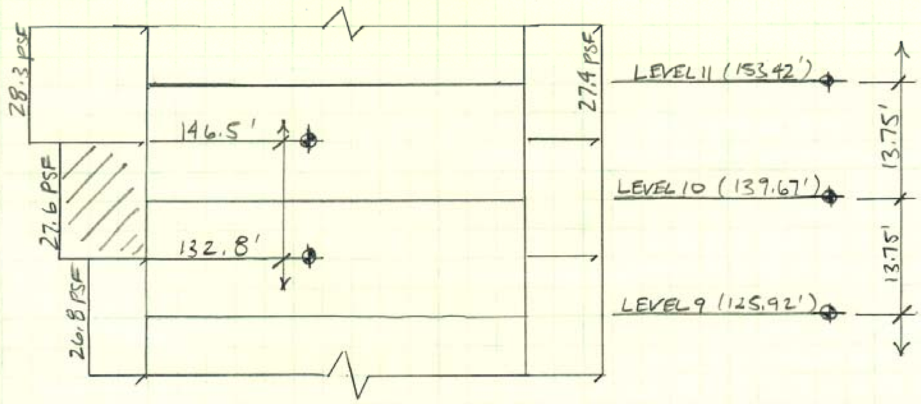
[F 6-5, ASCE 7-05]

Table 21: Tower N/S Wind Pressure Coefficients

Typical Wind Force Calculation

Level 10 w/ E-W Wind Direction

Partial Pressure Diagram:



Building Face (B) = 194'

Calculation:

Windward:

$(27.6 \text{ psf})(194')(146.5' - 132.8') = 73355.3 \text{ lb}$

Leeward:

$(27.4 \text{ psf})(194')(146.5' - 132.8') = 72823.7 \text{ lb}$

Total:

$\dfrac{(73355.3 + 72823.7) \text{ lb}}{1000 \text{ lb/k}} = \boxed{146 \text{ k}}$

Note: 0.6% Difference to Excel Due to Rounding.

Figure 24: Typical Wind Force Calculation

Gust Factor {Podium}					
Variable	Equation	N/S	Unit	Reference (ASCE 7)	Comments
n_1	100/h	1.16279	---	C6.5.8	Rigid Structure
$g_Q = g_v$	---	3.4	---	6.5.8.2	
h	---	86	ft		
z bar	.6h	51.6	ft		
z_{min}	---	30	ft	Table 6-2	z bar $\geq z_{min}$ (ok)
c	---	0.3	---	Table 6-2	
I_z	$c(33/z)^{1/6}$	0.2785	---	6.5.8.1	
ℓ	---	320	ft	Table 6-2	
ε	---	0.3333	---	Table 6-2	
L_z	$\ell(z/33)^{\varepsilon}$	371.42	ft	6.5.8.1	
B	---	245.00	ft		
Q	$(1/(1+0.63((B+h)/L_z)^{0.63})^{1/2}$	0.79408	---	6.5.8.1	
G	$.925((1+1.7g_Q I_z Q)/(1+1.7g_v I_z))$	0.80752	---	6.5.8.1	

Table 22: Podium Gust Factor

N/S Wind Direction Cp (Podium) {h/L ≤ 0.5 & θ < 10}				
L/B	Wall Pressure Coeff. (Cp)			
	Windward	Leeward	Side	
0.759	0.8	-0.5	-0.7	
h/L	Roof Pressure Coeff. (Cp)			
	0' - 34'	34' - 68'	68' - 136'	> 136'
0.462	-0.238	-0.238	-0.206	-0.190

Internal Pressure	
GC_{pi}	0.18
	-0.18

[F 6-5, ASCE 7-05]

Table 23: Podium N/S Wind Pressure Coefficients

Calculated Wind Pressures in North/South Direction of Podium {Using Method 2, ASCE 7-05}							
	Height (z)	K_z[a]	q_z & q_h (psf) $\{.00256K_zK_{zt}K_dV^2I\}$	External Pressure (psf) $\{qGC_p\}$	Internal Pressure (psf) $\{q_hGC_{pi}\}$	Net Pressure p (psf)	
						+ (GC_{pi})	- (GC_{pi})
Windward	15.0	0.57	17.40	11.2	5.2	6.1	16.4
	33.4	0.72	21.87	14.1	5.2	9.0	19.3
	48.9	0.81	24.39	15.8	5.2	10.6	20.9
	63.8	0.87	26.31	17.0	5.2	11.8	22.2
	77.8	0.92	27.85	18.0	5.2	12.8	23.2
	86.0*	0.95	28.66	18.5	5.2	13.4	23.7
Leeward	All	---	28.66	-11.6	5.2	-16.7	-6.4
Roof	86.0[b]	---	28.66	-5.5	5.2	-10.7	-0.3
	86.0[c]	---	28.66	-5.5	5.2	-10.7	-0.3
	86.0[d]	---	28.66	-4.8	5.2	-9.9	0.4
	86.0[e]	---	28.66	-4.4	5.2	-9.6	0.8

* Top of Podium
a Kz = 2.01(15/zg)2/a {zg < 15ft} -or- Kz = 2.01(z/zg)2/a {15 ft < z < zg} [T 6-2, ASCE 7-05]
[b] Windward edge to 34'
[c] 34' to 68'
[d] 68' to 136'
[e] 136' to 186'
Note: Wind pressures on East/West direction of podium were not calculated because East & West faces are not exposed.

Table 24: North/ West Wind Pressure on Podium

Calculated Wind Forces in North/South Direction of Podium{Using Method 2, ASCE 7-05}				
Level	Height Above Ground (ft)	Load (kips)	Shear (kips)	Moment (ft-kips)
2	25.66	129	370	3322
3	41.13	104	266	4259
4	56.59	104	162	5901
5	70.92	102	61	7210
6	86.00	61	0	5204
Total	86.00	499	499	25895

Table 25: Wind Loads, Shears & Moment on Podium

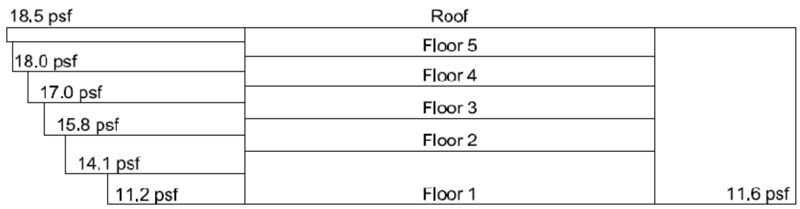


Figure 25: Podium Wind Pressure Diagram

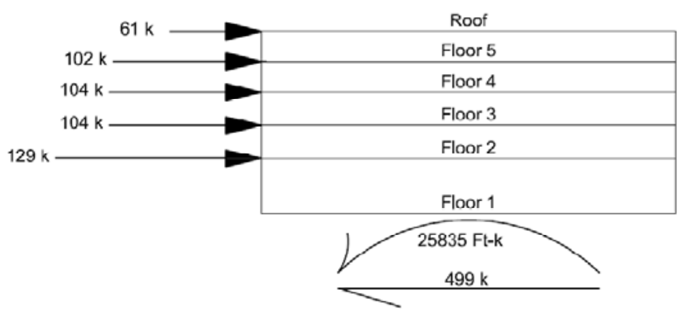


Figure 26: Podium Wind Force Diagram

APPENDIX E: SEISMIC ANALYSIS

Seismic Weight by Floor								
level	area (sf)		w_i (psf)		W_i (#)	h_x (ft)	h_i (ft)	$w_i{*}h_i^k$
	floor	façade	floor	façade				
1	96625	17639	113	13	11147926.04	25.20	25.2	7.08E+09
2	96625	10828	113	13	11059390.63	15.47	40.7	1.83E+10
3	96625	10828	113	13	11059390.63	15.47	56.1	3.49E+10
4	96625	10026	113	13	11048963.54	14.32	70.5	5.49E+10
5	21550	9625	113	13	2560275	13.75	84.2	1.82E+10
6	21550	9625	113	13	2560275	13.75	98.0	2.46E+10
7	21550	9625	113	13	2560275	13.75	111.7	3.19E+10
8	21550	9625	113	13	2560275	13.75	125.5	4.03E+10
9	21550	9625	113	13	2560275	13.75	139.2	4.96E+10
10	21550	9625	113	13	2560275	13.75	153.0	5.99E+10
11	21550	9625	113	13	2560275	13.75	166.7	7.12E+10
12	21550	9625	113	13	2560275	13.75	180.5	8.34E+10
13	21550	10442	113	13	2570891.667	14.92	195.4	9.81E+10
14	21550	8808	113	13	2549658.333	12.58	208.0	1.10E+11
15	21550	9625	113	13	2560275	13.75	221.7	1.26E+11
16	21550	9625	113	13	2560275	13.75	235.5	1.42E+11
17	21550	9625	113	13	2560275	13.75	249.2	1.59E+11
18	21550	9625	113	13	2560275	13.75	263.0	1.77E+11
19	21550	9625	113	13	2560275	13.75	276.7	1.96E+11
20	21550	9625	113	13	2560275	13.75	290.5	2.16E+11
21	21550	9625	113	13	2560275	13.75	304.2	2.37E+11
22	21550	9625	113	13	2560275	13.75	318.0	2.59E+11
23	21550	9625	113	13	2560275	13.75	331.7	2.82E+11
24	21550	9625	113	13	2560275	13.75	345.5	3.06E+11
25	21550	9625	113	13	2560275	13.75	359.2	3.30E+11
26	21550	9625	113	13	2560275	13.75	373.0	3.56E+11
27	21550	10179	113	13	2567479.167	14.54	387.5	3.86E+11
28	21550	19279	113	13	2685779.167	27.54	415.0	4.63E+11
29	21550	9625	113	13	2560275	13.75	428.8	4.71E+11
30	21550	9625	113	13	2560275	13.75	442.5	5.01E+11
31	21550	9625	113	13	2560275	13.75	456.3	5.33E+11
32	21550	9625	113	13	2560275	13.75	470.0	5.66E+11
33	21550	9625	113	13	2560275	13.75	483.8	5.99E+11
34	21550	9625	113	13	2560275	13.75	497.5	6.34E+11
35	21550	9625	113	13	2560275	13.75	511.3	6.69E+11
36	21550	9625	113	13	2560275	13.75	525.0	7.06E+11
37	21550	9625	113	13	2560275	13.75	538.8	7.43E+11
38	21550	9625	113	13	2560275	13.75	552.5	7.82E+11
39	21550	9625	113	13	2560275	13.75	566.3	8.21E+11
40	21550	9625	113	13	2560275	13.75	580.0	8.61E+11
41	21550	9625	113	13	2560275	13.75	593.8	9.03E+11
42	21550	9625	113	13	2560275	13.75	607.5	9.45E+11
43	21550	9625	113	13	2560275	13.75	621.3	9.88E+11
44	21550	9625	113	13	2560275	13.75	635.0	1.03E+12
45	21550	9625	113	13	2560275	13.75	648.8	1.08E+12
46	21550	9625	113	13	2560275	13.75	662.5	1.12E+12
47	21550	9625	113	13	2560275	13.75	676.3	1.17E+12
48	21550	9625	113	13	2560275	13.75	690.0	1.22E+12
49	21550	9625	113	13	2560275	13.75	703.8	1.27E+12
50	21550	10063	113	13	2565962.5	14.38	718.2	1.32E+12
51	21550	18664	113	13	2677786.333	26.66	744.8	1.49E+12
52	21550	12306	113	13	2595128	17.58	762.4	1.51E+12
ROOF	27400	0	200	13	5480000	0.00	762.4	3.19E+12
				ΣW	172979.631	k	$\Sigma w_i{*}h_i^k$	2.95E+13

Table 26: Seismic Weight by Floor

Lateral Seismic Force		
level	C_{vx}	F_x
1	0.0002	0.441
2	0.0006	1.139
3	0.0012	2.170
4	0.0019	3.416
5	0.0006	1.131
6	0.0008	1.530
7	0.0011	1.990
8	0.0014	2.510
9	0.0017	3.090
10	0.0020	3.730
11	0.0024	4.431
12	0.0028	5.192
13	0.0033	6.111
14	0.0037	6.867
15	0.0043	7.837
16	0.0048	8.839
17	0.0054	9.902
18	0.0060	11.025
19	0.0067	12.208
20	0.0073	13.451
21	0.0080	14.755
22	0.0088	16.119
23	0.0096	17.543
24	0.0104	19.027
25	0.0112	20.572
26	0.0121	22.177
27	0.0131	24.008
28	0.0157	28.811
29	0.0160	29.314
30	0.0170	31.225
31	0.0181	33.195
32	0.0192	35.226
33	0.0203	37.317
34	0.0215	39.468
35	0.0227	41.680
36	0.0240	43.952
37	0.0252	46.284
38	0.0265	48.677
39	0.0279	51.129
40	0.0292	53.642
41	0.0306	56.216
42	0.0321	58.849
43	0.0336	61.543
44	0.0351	64.298
45	0.0366	67.112
46	0.0382	69.987
47	0.0398	72.922
48	0.0414	75.917
49	0.0431	78.973
50	0.0449	82.415
51	0.0504	92.511
52	0.0512	93.938
ROOF	0.1081	198.36
	V= ΣF_x (k)	1834.2

Table 29: Lateral Seismic Forces by Floor

Soil Classification			
code	site class	reference	comments
NYCBC:	2-65 (medium hard rock)	T 11-2	recommended by geotechnical report
	4-65 (soft rock)	T 11-2	in areas of lower bearing capacity
ASCE 7-05:	seismic design category C	T 20.3-1	conservative estimate

Table 27: Soil Classification

Spectral Response Acceleration			
T=0.2s		T=1.0s	
S_{MS}	0.436	S_{M1}	0.119
S_{DS}	0.291	S_{D1}	0.08

Table 28: Spectral Response Acceleration

SEISMIC FORCE CALCULATIONS:

+ SOIL CLASSIFICATION: 4-65-NYCBC
SDC C - ASCE7-05

+ SPECTRAL RESPONSE ACCELERATION PARAMETERS:

- USING USGS GROUND MOTION PARAMETER TOOL:

LAT: 40.756192
LONG: -73.990130 → FOR NY TIMES ADDRESS (ITOUCHMAP.COM/LATLONG.HTML)

- SDC C: $F_a = 1.2$
$F_v = 1.7$

$T = 0.2s$ $S_{MS} = 0.436g$ $S_{DS} = 0.291g$

$T = 1.0s$ $S_{M1} = 0.119g$ $S_{D1} = 0.080g$

(ASCE 7-05 11.4.3 & 11.4.4)

- OCCUPANCY CATEGORY III

T 11.5-1 (ASCE): $I = 1.25$

T 11.6-1: $S_{DS} = 0.291g > 0.167$, < 0.33 → SDC B

T 11.6-2: $S_{D1} = 0.080g > 0.067$, < 0.133 → SDC B

∴ USE SDC C IN CALCULATIONS

- CH. 11.6: $S_1 < 0.75$ → CHECK REQUIREMENTS:

11.4.5 $T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.080}{0.291} = 0.275s$

$0.8T_s = 0.220s$

12.8.2.1 $T_a = C_t h_n^x$ T 12.8-2 $C_t = 0.02$, $x = 0.75$

$= (0.02)(762.4)^{0.75}$

$= 2.902s$

#1: $T_a \leq 0.8T_s$

$2.902s \not\leq 0.220s$ → CANNOT USE T 11.6-1 ONLY

Figure 27: Seismic Calculations and Variables

EQUIVALENT LATERAL FORCE
PROCEDURE:

+ RESPONSE MODIFICATION COEFFICIENT

T 12.2-1 $R = 3\frac{1}{4} \rightarrow$ B.4 ORDINARY STL. CONCENTRIC BRACED FRAMES

FIG. 22-15 $T_L = 6$ sec.

+ 12.8.1 SEISMIC BASE SHEAR

12.8.1.1 $C_s = \dfrac{S_{DS}}{\left(\frac{R}{I}\right)} = \dfrac{0.291}{\left(\frac{3.25}{1.25}\right)} = 0.1119$

• $T \leq T_L$ $\quad C_s = \dfrac{S_{D1}}{T\left(\frac{R}{I}\right)} = \dfrac{0.080}{2.902\left(\frac{3.25}{1.25}\right)} = 0.0106$

2.9s < 6s

$C_s \geq 0.01 \rightarrow C_s = 0.0106$ FOR CALCS.

- $V = C_s \cdot W$ *

$= 0.0106(172980\text{ k}) = 1834\text{ k}$

+ 12.8.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES

$F_x = C_{vx} \cdot V$

$C_{vx} = \dfrac{w_x \cdot h_x^k}{\sum_{i=1}^{n} w_i \cdot h_i^k}$ * $k = 2.0$ $(T_a > 2.5\text{ s})$

* DETAILED CALCULATIONS IN EXCEL.

Figure 28: Seismic Equivalent Lateral Force Calculations

APPENDIX F: SITE PHOTOS

Figure 29: Exterior X-bracing

Figure 30: Exterior view of NY Times HQ

Figure 31: Box Column

Figure 32: Outrigger on 28th Floor