The New York Times Building

New York, NY

IPD/BIM Thesis
Technical Report #3

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EXECUTIVE SUMMARY

In this third technical report, alternative lateral systems were investigated for The New York Times Building. The 65'-0" x 90'-0" central core of the building was analyzed for three lateral systems. The three systems were compared by story drift due to wind and seismic loads, total building drift due to wind loads, and periods of vibration due to both wind and seismic loads. The existing lateral system consists of eccentric and concentric chevron braces going up the core with outriggers at the twenty eighth and fifty first floors. The target periods of vibration of the three alternative systems are based on the existing system's periods of vibration of 6.25-6.75 seconds in the North-South direction, with the East-West direction being a little bit more flexible than the North-South direction. The three alternative lateral systems investigated include:

- concrete shear walls in the core
- concrete shear walls in the core with outriggers
- steel chevron bracing with outriggers & belt trusses

The concrete shear walls in the core exhibited periods of vibration of 6.893 seconds, 7.709 seconds, and 3.690 seconds in the North-South, West-East and torsional directions respectively for seismic. For wind, the periods of vibration were 5.926 seconds, 6.528 seconds, and 3.265 seconds in the North-South, West-East, and torsional directions respectively. There are 2'-6" thick returns in the North-South direction at 10'-0" and 20'-0" long with ten 10'-0" and two 30'-0" long coupling beams which are 3'-0" deep by 2'-6" wide. The walls in the West-East direction are 65'-0" long and range in size from 2'-6" from the basement to the twentieth floor, to 2'-0" from the twenty first to the fortieth floor and to 1'-6" from the forty first floor to the roof. The compressive strength ranges from 12,000 psi in the basement to the tenth floor, 10,000 psi from the eleventh floor to the thirtieth floor and 8,000 psi from the thirty first floor to the roof. Case 1 wind controlled total building drift with 16.76 inches in the North-South direction and 10.76 inches in the West-East direction. Member strengths met requirements for flexure and shear loading.

The concrete shear walls in the core with steel outriggers displayed periods of vibration of 6.97 seconds, 6.23 seconds, and 4.88 seconds in the North-South, West-East and torsional directions respectively for seismic. For wind, the periods of vibration were 6.44 seconds, 5.69 seconds, and 4.57 seconds in the North-South, West-East, and torsional directions respectively. There are 1'-6" thick returns in the North-South direction at 10'-0" and 20'-0" long with ten 10'-0" long coupling beams which are 4'-0" deep by 1'-6" wide. The walls in the West-East direction are 65'-0" long and range in size from 1'-4" from the basement to the thirtieth floor and to 1'-2" from the thirty first floor to the roof. The compressive strength ranges from 10,000 psi from the basement to the thirtieth floor, 8,000 psi from the thirty first floor to the fortieth floor, 6,000 psi from the forty first floor to the fiftieth floor and back to 8,000 psi from the fifty first to the roof. Columns are 2'-6" by 2'-6" with flanges 4" thick and webs 4"-4 ¾" thick. Case 1 wind controlled total building drift with 16.119 inches in the North-South direction and 16.856 inches in the West-East direction.

The steel chevron bracing with outriggers & belt truss revealed periods of vibration of 5.26 seconds, 5.17 seconds, and 3.92 seconds in the North-South, West-East and torsional directions respectively for seismic and wind. There are W14x283 braces from the first floor to the thirteenth floor, W14x176 braces from the fourteenth to the twenty seventh, HSS16x16x1/2 braces from the twenty eighth floor to the fortieth floor, and HSS12x12x3/8 braces from the forty first floor to the roof. All outrigger sizes are W36x247. All box columns are 2'-6" by 2'-6" with flanges and webs varying in size from7"-4" thick for flanges and 4"- 2½" thick for webs. All steel is grade 50 ksi. Case 1 wind controlled total building drift with 16.7 inches in the North-South direction and 19.8 inches in the West-East direction.

Overall, concrete shear walls in the core, whether with or without outriggers, seem to be a viable alternative solution that should be investigated further for the spring semester.

IPD/BIM SUMMARY

After consulting with the IPD/BIM Team 1, the option of concrete shear walls in the core will be investigated further in the upcoming semester. Though this solution allows the building to utilize the compressive strength of concrete and increase the building stiffness, there are potential issues that should be addressed. The layout of the core will most likely have to change in order to maximize the efficiency of the lateral system. Elevator openings and placement will most likely have to change as well, since the assumptions made in the following report imposed on the architectural layout of the core and neglected atypical elevator openings to help simplify the alternative lateral system. In addition mechanical systems and stairways will have to change due to lateral system impacts. However, it could be argued that if the existing lateral system had concrete shear walls, the layout of the core would be different. The construction management member of the team expressed interest in looking at lead time and potential cost benefits of this system change.

In addition to looking at an alternative lateral system, the IPD/BIM Team expressed interest in changing the façade. Not only will the façade change effect loading, but façade components will have to be investigated from a structural stand point, along with the mechanical, lighting and construction impacts of the façade.

INTRODUCTION

The New York Times Headquarters Building is home to the New York Times newsroom and twenty six floors of Times offices, as well as several law firms whose offices are leased through Forest City Ratner. Designed by architect Renzo Piano in association with FFFOWLE Architects, 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.

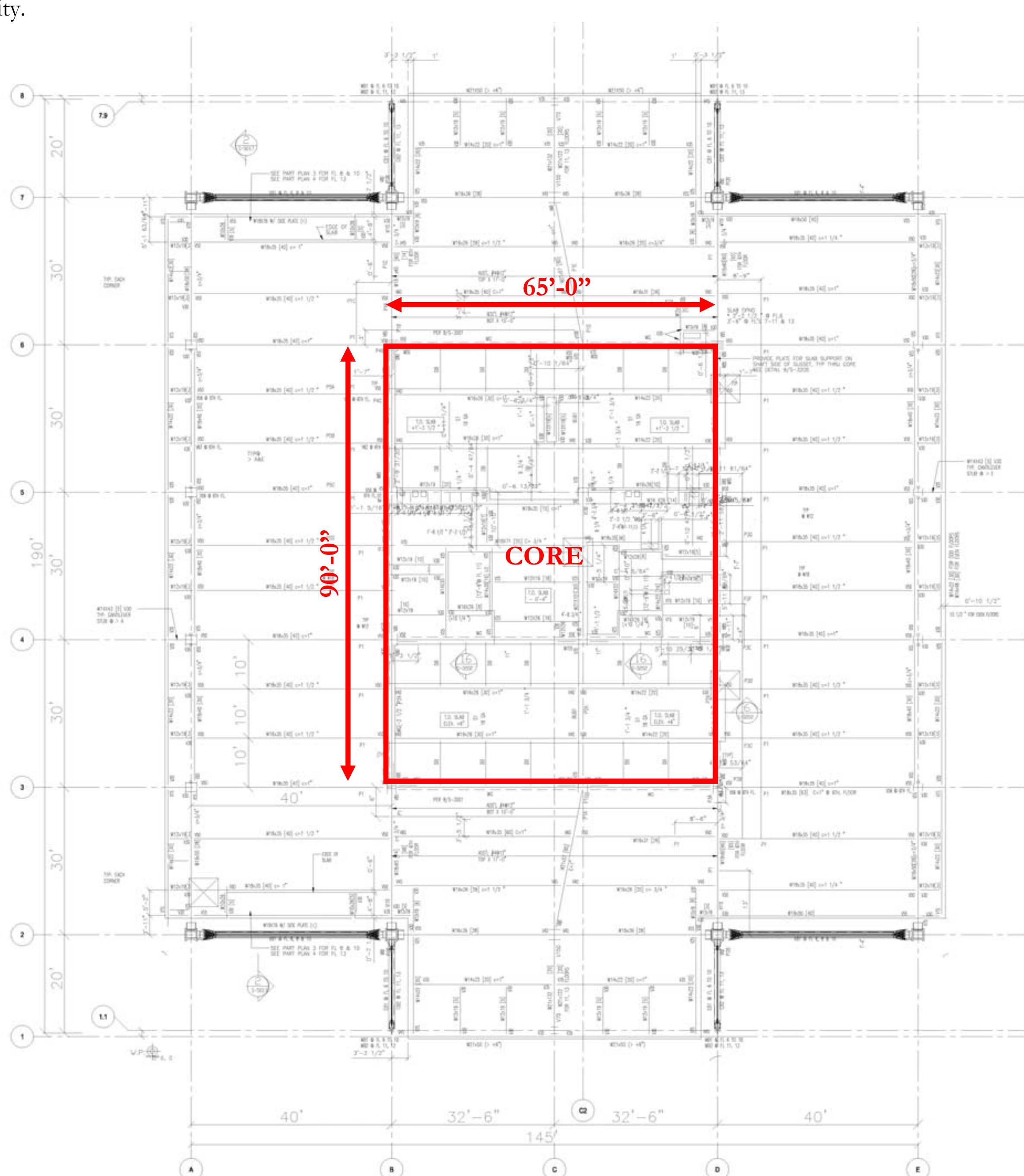


Figure 1: Typical Floor Plan

The building rises fifty two stories with a height of 744 feet to the main roof. A 300 feet mast then extends up into the sky topping out at 1048 feet above Eighth Avenue between 40th and 41st Streets. The New York Times building totals 1.5 million square feet with the New York Times Company owning 800,000 square feet and Forest City Ratner Companies owning the other 700,000 square feet. It has one 16'-0" level below grade. The ground level contains a lobby, retail space and a glass-enclosed garden. The New York Times' newsroom occupies the entire five-story podium which is east of the tower structure. The tower ascends above the podium an additional forty eight stories. Story heights average approximately 13'-9" in the tower, lending a great view to the open office plans. At the mechanical floors on levels twenty eight and fifty one though, the floor height is approximately 27'-0" 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 braces in both the East-West and North-South directions in the core. 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.

The remainder of this report evaluates three alternative lateral force-resisting system solutions. All designs are preliminary and not optimized, as the objective of this report is to study various lateral force-resisting systems that can be applied to the New York Times Building. Optimization of these systems will be investigated in more detail in the future. Some of the systems impact the architecture of the space due to the sizes and placement of shear walls. However, it can be assumed that if these alternative systems were implemented, the architecture of the core would be different. ETABS was used to perform the various lateral force-resisting system analyses. Hand calculations were used to verify certain ETABS outputs. At various locations the lateral systems were checked for strength and drift requirements.

EXISTING 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 rock; Class 1-65 and 2-65 per the New York City Building Code, with a capacity of 20 - 40 ton per square foot. However, the rock at the southeast corner of the tower only had an 8 ton per square foot capacity; Class 4-65. Of the seven columns that fall within this area (indicated in Figure 2) 24-inch diameter concrete-filled steel caissons were used. Each caisson was designed to support a load of 2,400 kips with 6,000 psi concrete.

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

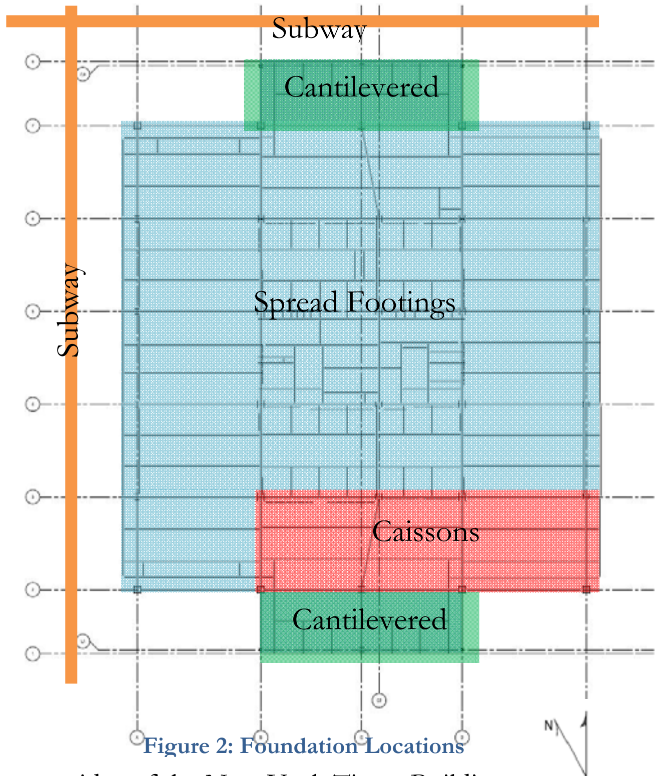


Figure 2: Foundation Locations

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. Although, 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" surrounding the 90'-0" x 65'-0" core. There are 60'-0" x 20'-0" cantilever bays on the north and south sides of the tower. The floor system is made up of 2 ½" normal weight concrete on 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.

In the New York Times spaces, the structural slab is 16" below the finish floor and the spandrel panel, due to the raised floor system for the under floor mechanical systems. For all the exterior steel of the building to maintain a centerline at the center of the spandrel panel, a crooked connection or 'dog-leg' was used. The 'dog-leg' connection allows for the end of the beam to rise 10" before it leaves the interior of the building and penetrates the building envelope. Figure 3 shows the 'dog-leg' connection penetrating the building envelope.

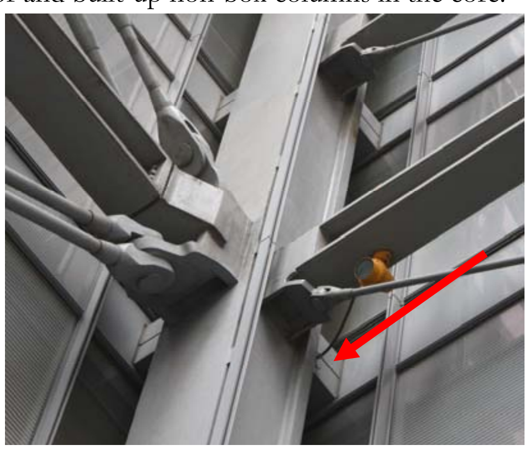


Figure 3: 'Dog-leg' penetrating building envelope

Columns

The 30"x30" box columns at the exterior notches of the tower consist of two 30" long flange plates and two web plates inset 3" from the exterior of the column on either side. The two web plates of the welded box column vary from 7" thick at the ground floor to 1" thick at the fifty second floor. This is to account for the different steel areas needed for the higher forces at the bottom of the building. To maintain consistent proportions at all floors, a hierarchy of flange plate thicknesses was developed. At the ground floor, each flange plate is 4" thick and decreases to 2" thick at the fifty second floor. See Figure 6 for box column hierarchy. 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 perimeter column is engaged in the lateral system which will be described later.

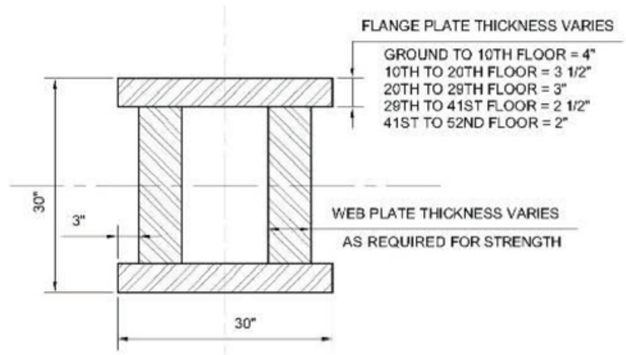


Figure 4: Box Column hierarchy, courtesy of Thornton Tomasetti

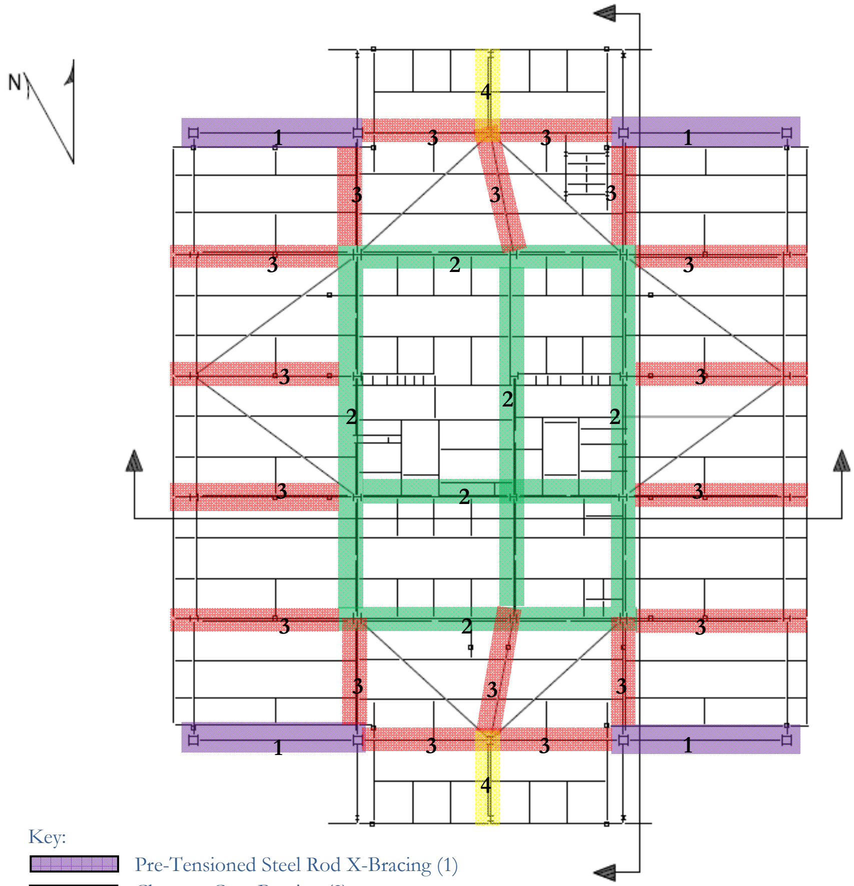
Vierendeel System

A Vierendeel system was used 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. The middle line of the cantilevered bays have beams moment connected to the columns thus creating the Vierendeel system and engaging every floor except at the outrigger levels. At the outrigger level; floor twenty eight and fifty two, large diagonal braces tie the middle line back to the core through the outrigger trusses. In extreme loading conditions, this provides a redundant load path. See Figure 5 for Vierendeel frame location. At the exterior beam lines of the cantilever, 2" diameter steel rods were connected from the columns to the ends of the beams to control deflection at every floor. This allowed the beams to be designed only for strength, thus avoiding bulky exterior members.


Figure 5: Cantilevered bays from exterior

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 twenty eight and fifty one). The structural core consists of concentric braces behind elevator shafts and eccentric braces at the elevator lobby entrances. At this time, the member sizes of these braces have yet to be disclosed, but the members were sized for strength. The core configuration remains consistent from the ground level to the twenty seventh floor as shown in Figure 6 and Figure 8 on the next page. Above the twenty eighth floor, the low rise elevators were no longer required, and the number of bracing lines in the North-South direction was reduced from two to one.



Key:
Pre-Tensioned Steel Rod X-Bracing (1)
Chevron Core Bracing (2)
Outrigger Bracing (3)
Vierendeel System at Cantilever (4)

Figure 6: Mechanical Floor Framing Plan (Floors 28 & 51)

Figure 8: Core bracing during construction, courtesy of Thornton Tomasetti

Figure 7: Outrigger bracing on mechanical floor, courtesy of Thornton Tomasetti

The outriggers on the mechanical floors engage all columns of the tower in the lateral system. The outriggers consist of single diagonal braces shown in Figure 6 and Figure 7 on previous page. The outrigger system was designed to increase the efficiency and redundancy of the tower by engaging the perimeter columns in the lateral system.

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 accommodate lateral drift and acceleration, the structural engineers utilized the double story steel rod X-braces instead of increasing the member sizes of the main lateral force resisting system. These X-braces can be seen in Figure 6 on previous page and in Figure 9. The paired rods eliminate a center node and load sharing, in addition to eliminating eccentricities at the columns. 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 prevents the members from buckling and conforms 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's 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.

Figure 9: Exposed exterior X-braced rods

Thermal differentials had to be considered due to interior steel members being maintained at room temperature and exposed steel members undergoing extreme temperature changes. Thornton Tomasetti designed the structure using a range of -10˚F to 130 ˚F. Due to the temperature deformation of the exterior columns and not the interior ones, differential deflection at upper floors exceeded L/100. To combat these thermal differentials, the outrigger trusses were utilized to even out the differential deflections. Thermal trusses were added along the east and west face at the twenty eighth and fifty first floors (Figure 10). These trusses provide bonus redundancy and limited deflection to L/300.

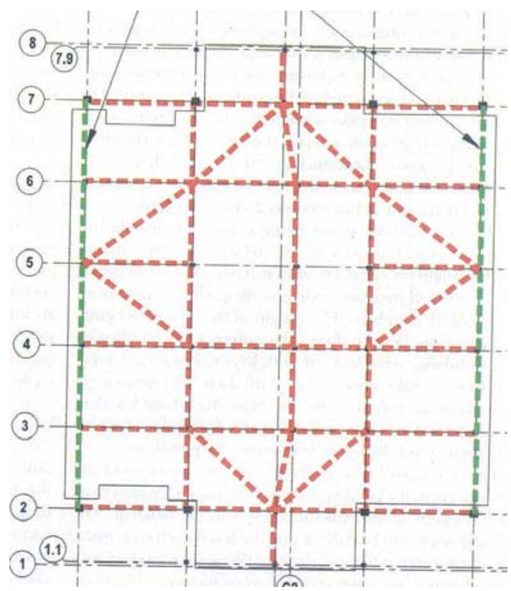


Figure 10: Thermal Truss, in green, located at the 28th and 51st floor, courtesy of Thorton Tomasetti

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 and it can therefore 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" and open plan layout of the building design. Therefore, it can be assumed that the lateral system of the podium, from the ground to sixth floor, is designed as a steel moment resisting frame.

CODES & DESIGN CRITERIA

Codes

Design Codes

National Model Code:

1968 Building Code of the City of New York with latest supplements

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
ACI 135-74 Manual of standard Practice for detailing Reinforced Concrete Structures
ACI 318-99 American Concrete Institute Building Code Requirements for Reinforced Concrete
ACI 530-95 Building Code Requirements for Masonry Structures
National Building Code of Canada, 1995
Uniform Building Code, 1997

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

Deflection Criteria

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.

Thesis Deflection Criteria

Lateral Deflections:

Total building sway deflection for ten year wind loading is limited to H/450
Allowable inter-story drift due to wind is H/400 to H/600 (ASCE 7-05 § CC.1.2)
Building story sway deflection for seismic loading is limited to $0.015h_{sx}$ (ASCE 7-05 TABLE 12.12-1)

Thermal Deflections:

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

BUILDING LOADS

Load Cases

The following LRFD load combinations are considered in this report and taken from ASCE 7-05.

1.4D
1.2D + 1.6L + 0.5L_r
1.2D + 1.6L_r + (1.0L or 0.8W)
1.2D + 1.6W + 1.0L + 0.5L_r
1.2D + 1.0E + 1.0L
0.9D + 1.6W
0.9D + 1.0E

Due to only the lateral system being modeled, combinations with wind and earthquake loads were used for this report. The controlling equations for lateral member design are:

1.6W (Equation 1)
1.0E (Equation 2)

Since the building drift due to wind is limited to H/450 and checking serviceability using factored wind load is excessively conservative due to winds short term effects, ASCE 7-05 § CC.1.2 allows the use of the following equation for drift due to wind:

D + 0.5L + 0.7W (Equation 3)

Due to the rectangular geometry of The New York Times Building, wind load cases 1 and 3 of ASCE 7 -05 in figure 6-9 were applied. Wind load cases 2 and 4 were not analyzed and will be investigated in more detail in the future when optimizing the lateral system.

Gravity Loads

The following gravity loads have been revised from the first technical report since receiving guidance from Thornton Tomasetti. ASCE 7 – 05 was used to determine both gravity and lateral loads.

Dead Loads

Typical Tower Floor Dead Load:

Load Description	Design Load	
5.5" Slab with 20 GA 3" Composite Metal Deck (50+3 for deck)	53	psf
Ceiling (Floors have ACT, Drywall, and Special Architectural Ceilings)	5	psf
Mech., Elec., Plumbing in raised floor	12	psf
Mech., Elec., Plumbing in ceiling	8	psf
Allowance for Steel Framing + Fireproofing(paint & cementitious)*	15	psf
Total Typical Floor Dead Load:	93	psf
Total Typical Floor Dead Load for Seismic:	113 psf+25 psf(on elevated area of exterior wall)	
*includes column weight therefore loading only applied to columns		

Typical Tower Mechanical Floor Dead Load:		
Load Description	**Design Load**	
6" Slab with 20 GA 3" Composite Metal Deck	57	psf
Ceiling (Floors have ACT and Special Architectural Ceilings)	5	psf
Mech., Elec., Plumbing in ceiling	8	psf
Allowance for Steel Framing + Fireproofing(paint & cementitious)*	15	psf
Total Mechanical Floor Dead Load:	110	psf
Total Typical Floor Dead Load for Seismic:	130 psf+25 psf(on elevated area of exterior wall)	
*includes column weight therefore loading only applied to columns		

Exterior Tower Wall System Dead Load (Elevation):		
Load Description	**Design Load**	
Curtain Wall with Horizontal Ceramic Rods, Aluminum and Frame	25	psf
Total Exterior Wall Dead Load:	25	psf

Tower Mechanical Area Roof Dead Load:		
Load Description	**Design Load**	
8" Composite Deck	85	psf
Allowance for Steel Framing + Fireproofing(paint & cementitious)*	15	psf
Total Mechanical Area Roof Dead Load:	100	psf
Total Typical Floor Dead Load for Seismic:	120 psf+25 psf(on elevated area of exterior wall)	
*includes column weight therefore loading only applied to columns		

Normal Tower Roof Dead Load:		
Load Description	**Design Load**	
8" Composite Deck	85	psf
Allowance for Steel Framing + Fireproofing(paint & cementitious)*	15	psf
Total Normal Roof Dead Load:	100	psf
Total Typical Floor Dead Load for Seismic:	120 psf+25 psf(on elevated area of exterior wall)	
*includes column weight therefore loading only applied to columns		

Live Loads

Live Load:		
Load Description	**ASCE 7-05 & NYC Bldg Code**	**Design Load**
Office:	50 psf	50+20 (for partitions) = 70 psf
Technology Floors:	100 psf	100 psf
Elevator Lobbies:	75 psf	75 psf
Corridors above First Floor:	80/75 psf	75 psf
All Other Lobbies & Corridors:	100 psf	100 psf
Exit Facilities:	100 psf	100 psf
Retail Areas:	100 psf	100 psf

Kitchen:	100 psf	150 psf
Cafeteria:	100 psf	100 psf
Auditorium (with fixed seats):	60 psf	100 psf
Light Storage Area:	125/100 psf	100 psf
Loading Dock:	250 psf	250 psf or actual weight whichever is greater
Mechanical Floors:	125 psf	150 psf or actual weight whichever is greater
Mechanical/Fan Rooms:	75 psf	75 psf or actual weight whichever is greater
Sidewalks	250 psf	600 psf
Roofs:	20 psf	30 psf + Drift
Roof Garden	100 psf	Not Specified

Since the weight of the mechanical equipment on the mechanical roof and the mechanical floor 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	ASCE 7-05 Design Load		New York City Building Code
p_g=	25	psf	According to § [C26-902.6] 27-561 For valleys...provide for accumulations of snow... vary from forty-five psf at the low point to fifteen psf at the ridge.
p_s =	17.5	psf	
p_d=	35.28	psf	

Since the weight of the snow on the roof plus snow drift is approximately two times smaller compared to the controlling roof live load and mechanical area roof live load, it is assumed to not control. *See Appendix A for snow load calculations.

Lateral Loads

Wind Loads

See Figure 11 and Figure 12 on the next two pages for the wind loads that were applied to The New York Times Building at the center of pressure for each level. Wind loads were determined in technical report one using ASCE 7-05 §6.5 Method 2 Analysis and have been revised with a minor change to include both screens in the wind force calculations. For detailed calculations used when determining the wind forces, see Appendix B.

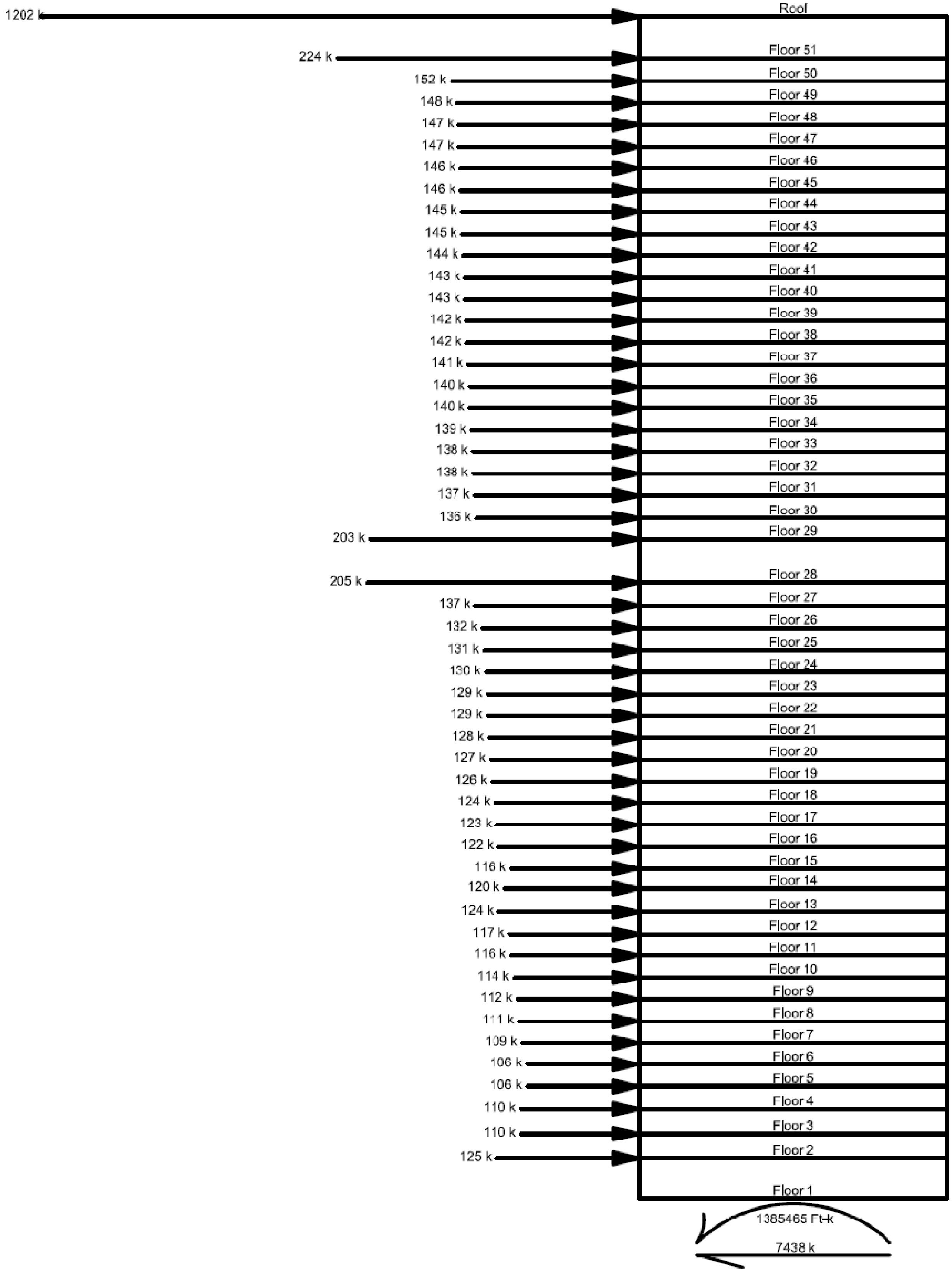


Figure 11: North/ South Wind Force Diagram

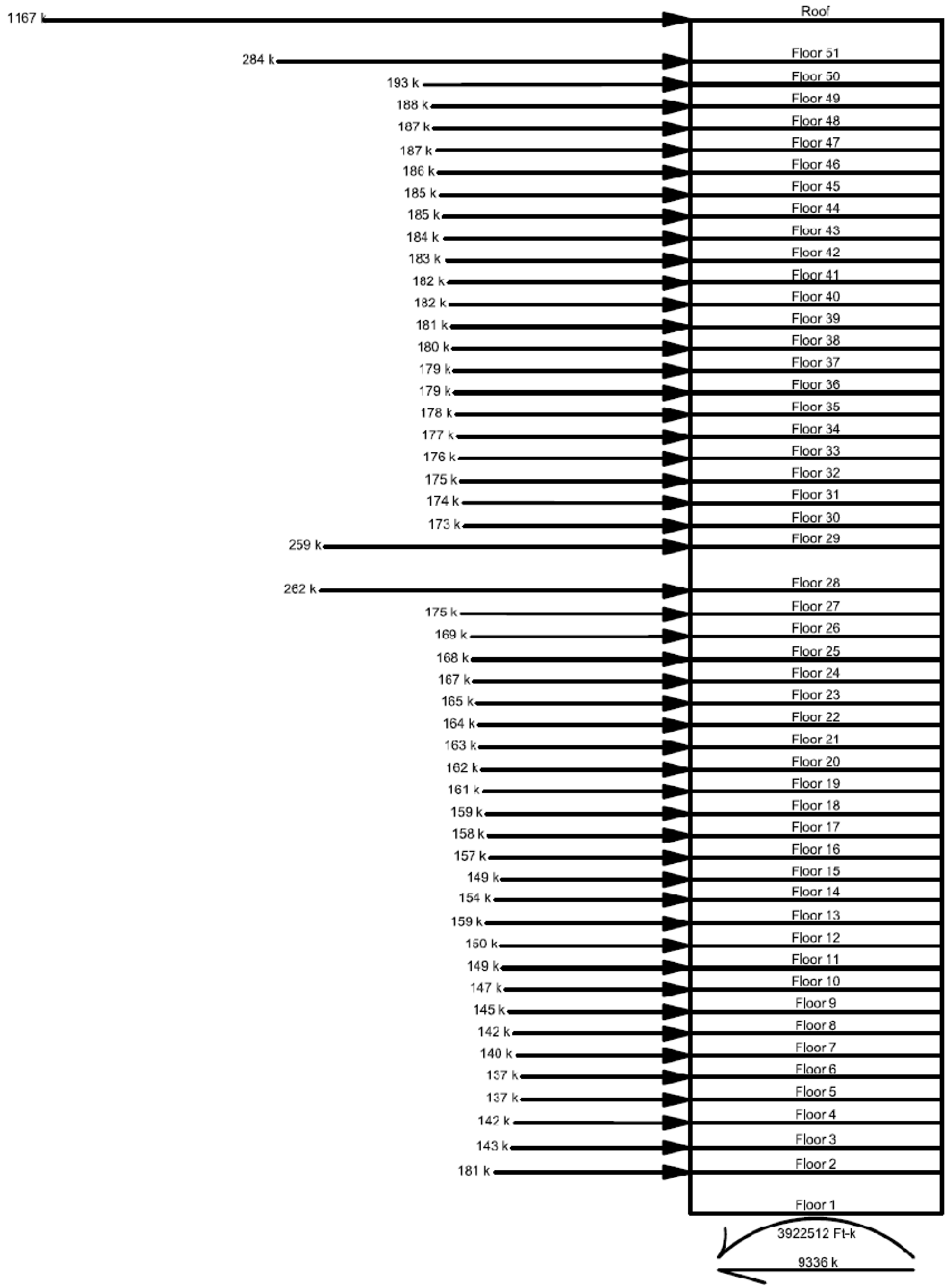


Figure 12: West/ East Wind Force Diagram

Seismic Loads

Seismic loads were determined in technical report one with the change of C_s=0.01. Using ASCE7-05 Chapters 11 and 12, the seismic forces on the building were determined. See Figure 13 below for the seismic loads that were applied to The New York Times Building in both directions at the center of mass for each level. Please see Appendix B for the calculations of the seismic loads.

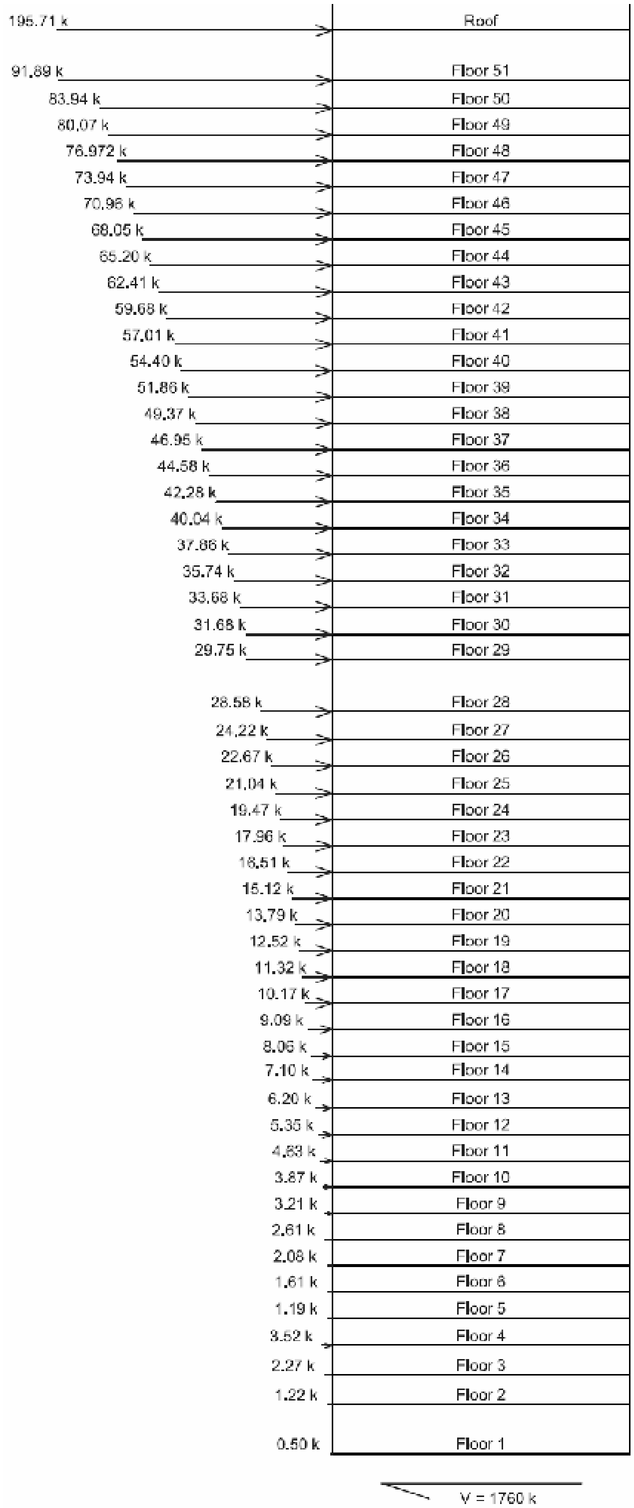


Figure 13: North/South & East West Seismic Force Diagram

ETABS MODELS

A 3D ETABS model was used to investigate preliminary alternative lateral systems for The New York Times Building. The three alternative lateral systems investigated were:

- concrete shear walls in the core
- concrete shear walls in the core with outriggers
- steel chevron bracing with outriggers & belt trusses

The following modeling assumptions pertain to the concrete shear walls in the core option. For modeling assumptions on the concrete shear walls in the core with outriggers and steel chevron bracing with outriggers & belt truss please refer to Andres Perez's and Erika Bonfanti's Technical Report 3 respectively. Each floor was treated as a rigid diaphragm. Gravity members were excluded, but the existing gravity system was incorporated for calculating the building's weight for seismic loads. When investigating the concrete shear walls, member stiffnesses were modified for both wind and seismic; 70% and 50% of the gross section properties respectively, based on ACI 318 § 8.8.1. Shear walls were modeled as shell elements, and coupling beams were modeled as frame elements with mass. The walls were meshed with a maximum size of forty eight inches. Wind loads were applied at the center of pressure of each level, and seismic loads were placed at the center of mass of each level. Due to the symmetry of the building, both the center of mass and the center of pressure are located at the same location. The New York Times Building's existing lateral system has a period of vibration of 6.25-6.75 seconds which was used as a target goal for the alternative systems. The lateral load cases that were investigated include:

- Wind loads in the East-West direction
- Wind loads in the North-South direction
- 75% of the wind loads in the North-South and East-West direction applied simultaneously
- Seismic loads in the East-West direction
- Seismic loads in the North-South direction

As mentioned above, wind load cases two and four in Figure 6-9 of ASCE 7 – 05 will be investigated in more detail in the future.

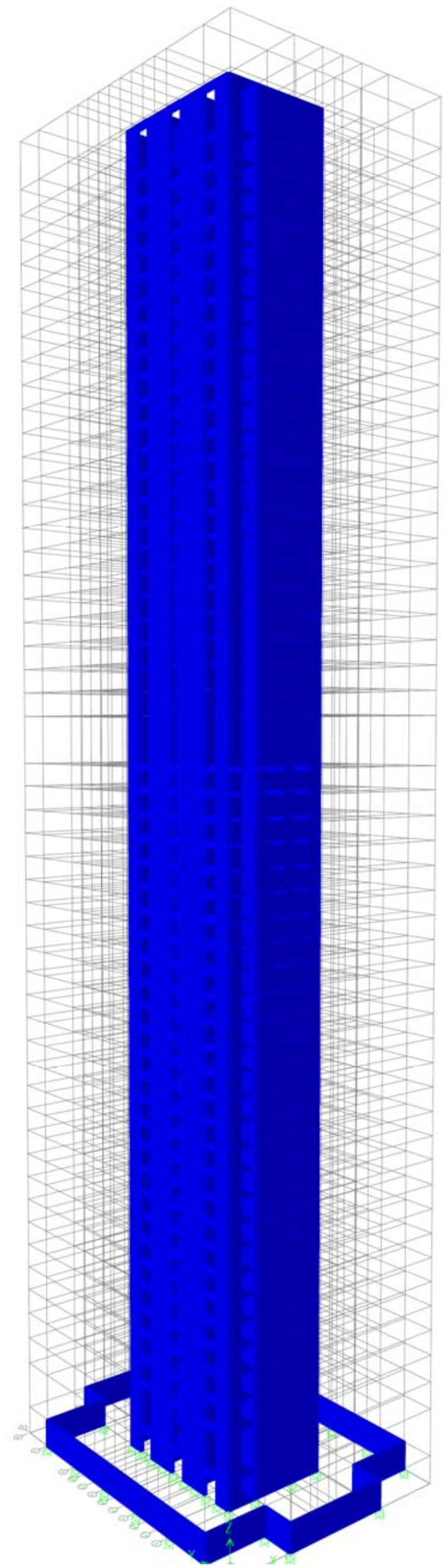


Figure 14: ETABS NYT Lateral Model

OPTION 1: CONCRETE SHEAR WALLS IN THE CORE

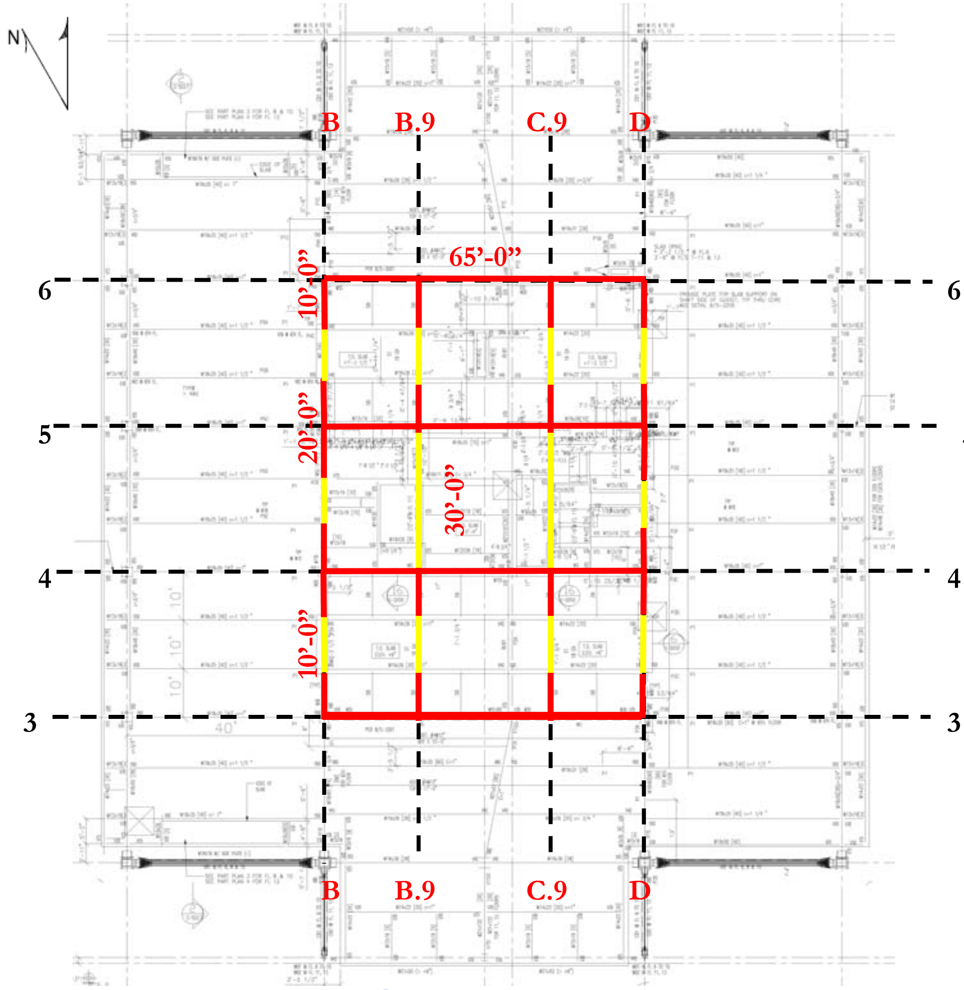


Figure 15: Concrete shear walls in the core

The following alternative lateral system was modeled in ETABS with shear walls shown in red and coupling beams shown in yellow in Figure 15. The 2'-6" thick returns in the North-South direction are 10'-0" and 20'-0" long with ten 10'-0" and two 30'-0" coupling beams which are 3'-0" deep by 2'-6" wide. This configuration and sizes of the returns and coupling beams remain constant all the way up the building. However, the compressive strength and thickness of the 65'-0" concrete shear walls change. Perimeter foundation walls are 2'-0" thick with 4,000 psi concrete. Moving in towards the core, the compressive strength changes from 12,000 psi from the basement to the tenth floor, 10,000 psi from the eleventh floor to the thirty floor, and 8,000 psi from the thirty first floor to the roof. The wall thickness changes from 2'-6" from the basement to the twentieth floor, to 2'-0" from the twenty first to the fortieth floor and finally to 1'-6" from the forty first floor to the roof. The periods of vibrations were 6.893 seconds, 7.709 seconds, and 3.690 seconds in the North-South, West-East, and torsional directions respectively for seismic. For wind, the periods of vibrations were 5.926 seconds, 6.528 seconds, and 3.265 seconds in the North-South, West-East, and torsional directions respectively. Due to seismic loading in the North-South direction, the total building displacement was 6.10 inches and 8.37inches in the West-East direction. Due to wind loading using the D + 0.5L + 0.7W drift equation, total building displacement was 16.76 inches in the North-South direction and 10.76 inches in the West-East direction for case 1 wind. Using the same drift equation, the total building displacement for case 3 wind was 8.07 inches in the North-South direction and 12.57 inches in the West-East direction. For more in depth information and calculations refer to analysis section and Appendix C.

OPTION 2: CONCRETE SHEAR WALLS IN THE CORE WITH OUTRIGGERS

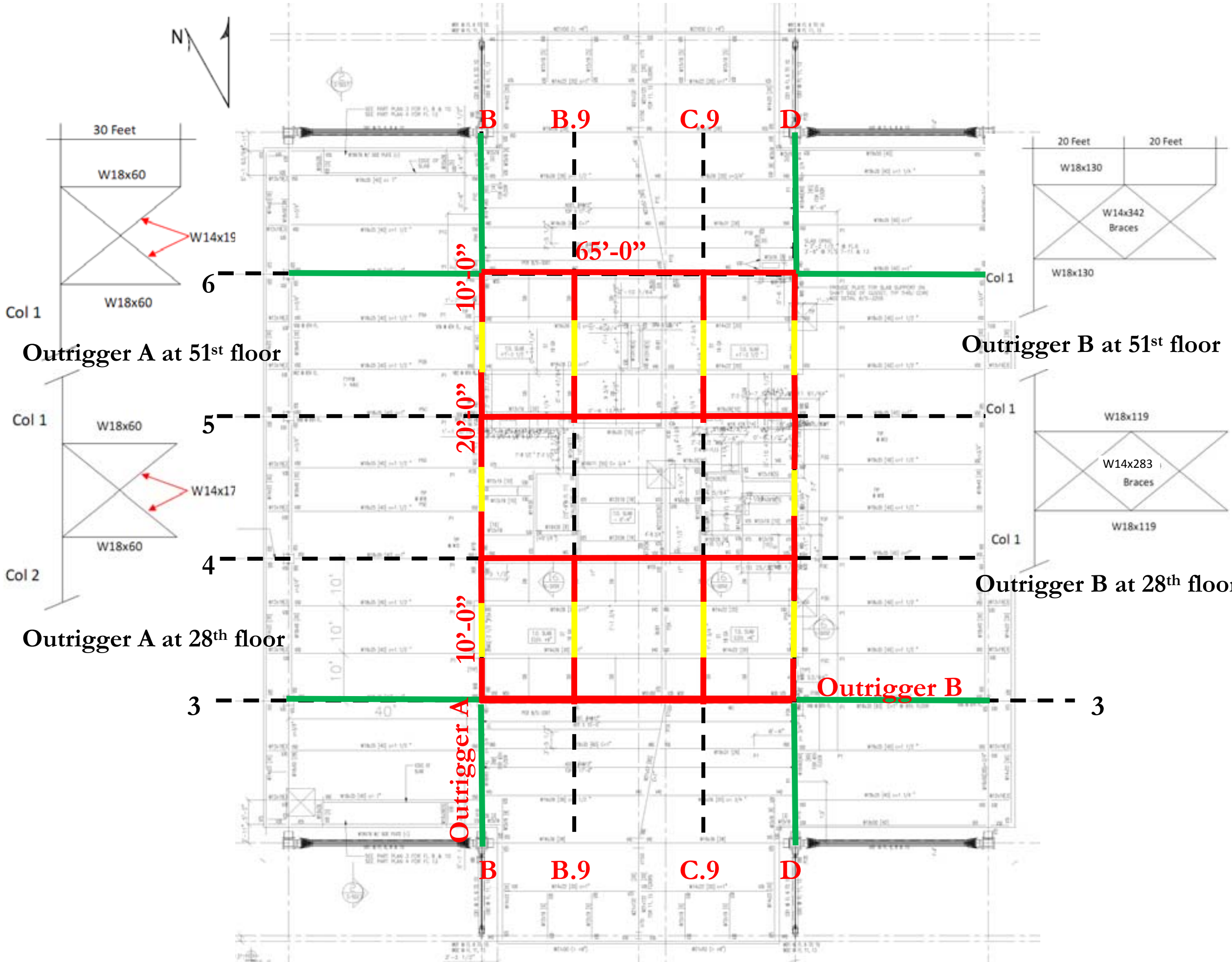


Figure 16: Concrete shear walls in the core with Steel Outriggers

The following alternative lateral system was modeled in ETABS with shear walls shown in red, coupling beams shown in yellow and steel outriggers on the twenty eighth and fifty first floors shown in green in Figure 16. The 1'-6" thick returns in the North-South direction are 10'-0" and 20'-0" long with ten 10'-0" coupling beams 4'-0" deep by 1'-6" wide. This configuration and sizes of the returns and coupling beams remain constant all the way up the building. However, the compressive strength and thickness of the 65'-0" concrete shear walls change. Moving in towards the core, the compressive strength changes 10,000 psi from the basement to the thirtieth floor, 8,000 psi from the thirty first floor to the fortieth floor, 6,000 psi from the forty first floor to the fiftieth floor, and back to 8,000 psi from the fifty first to the roof. The wall thickness changes from 1'-4" from the basement to the thirtieth floor and to 1'-2" from the thirty first to the roof. Column 1 is 2'-6" by 2'-6" with flanges and webs 4" thick, where column 2 is 2'-6" by 2'-6" with flanges 4" thick and webs 4 ¾" thick. The periods of vibration were 6.97 seconds, 6.23 seconds, and 4.88 seconds in the North-South, West-East, and torsional directions respectively for seismic. For wind, the periods of vibration were 6.44 seconds, 5.69 seconds, and 4.57 seconds in the North-South, West-East, and torsional directions respectively. Due to seismic loading in the North-South direction, the total building displacement was 8.974 inches and 8.162 inches in the West-East direction. Due to wind loading using the D + 0.5L + 0.7W drift equation, total building displacement was 16.119 inches in the North-South direction and 16.856 inches in the West-East direction for case 1 wind. For more in depth information and calculations refer to Andres Perez's Technical Report 3.

OPTION 3: STEEL CHEVRON BRACING WITH OUTRIGGERS & BELT TRUSSES

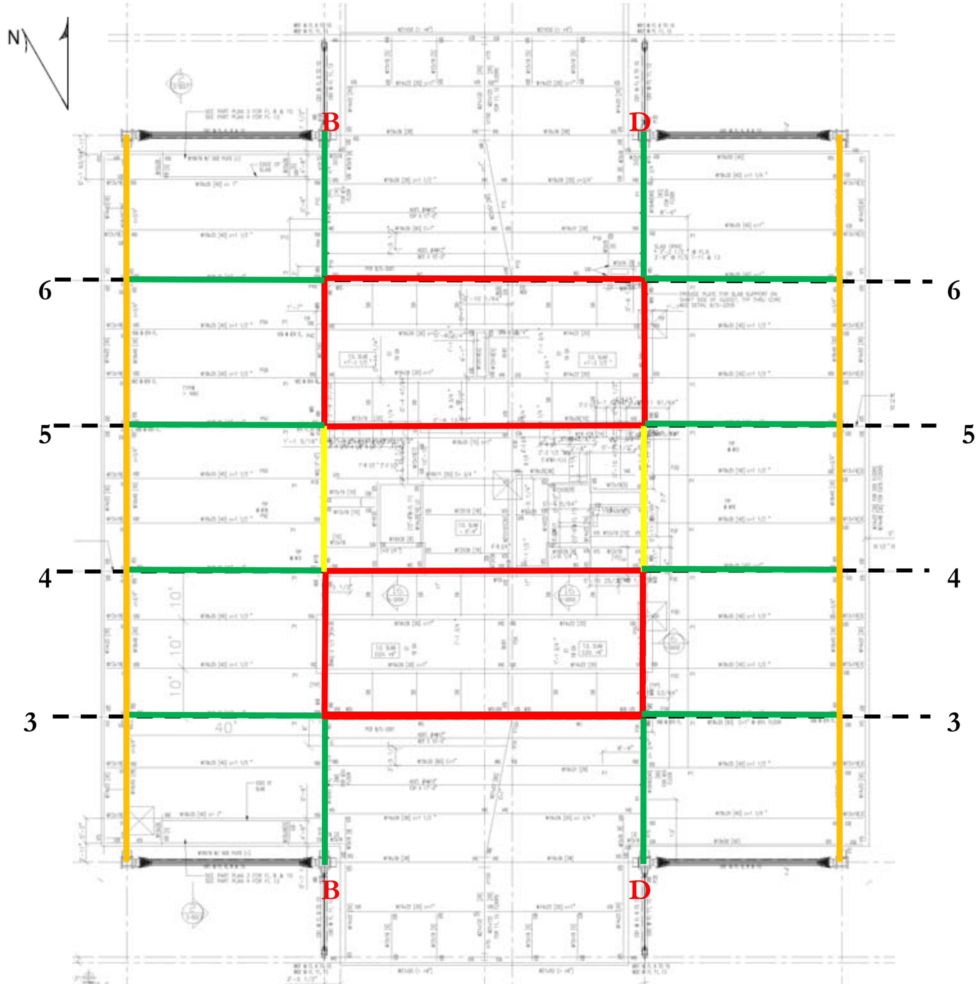


Figure 17: Steel Chevron Bracing with Outriggers & Belt Trusses

The following alternative lateral system was modeled in ETABS with chevron braces shown in red, single diagonal braces shown in yellow and steel outriggers and belt trusses on the thirty sixth floor shown in green and orange respectively in Figure 17. The bracing sizes are consistent on a single floor, however they vary in size going up the building. W14x283 were used from the first floor to the thirteenth floor, W14x176 for the fourteenth to the twenty seventh, HSS16x16x1/2 for the twenty eighth floor to the fortieth floor, and HSS12x12x3/8 for the forty first floor to the roof. All outrigger sizes are W36x247. All box columns are 2'-6" by 2'-6" with flanges and webs varying in size for required strength. Flanges are 7" thick with 4" thick webs from the first floor to the thirteenth floor. From the fourteenth floor to the twenty seventh floor flanges are 6" thick with 3½" thick webs, from the twenty eighth floor to the fortieth floor flanges are 5" thick with 3" thick webs, and from the forty first floor to the roof flanges are 4" thick with 2½" thick webs. All steel is grade 50 ksi. The periods of vibration were 5.26 seconds, 5.17 seconds, and 3.92 seconds in the North-South, West-East, and torsional directions respectively for seismic and wind. It was found that case 1 wind controlled drift which was 16.7 inches in the North-South direction and 19.8 inches in the West-East direction. For more in depth information and calculations refer to Erika Bonfanti's Technical Report 3.

ANALYSIS

The following analysis pertains to the concrete shear walls in the core option. For analysis details on the concrete shear walls in the core with outriggers and steel chevron bracing with outriggers & belt truss please refer to Andres Perez's and Erika Bonfanti's Technical Report 3 respectively. When checking the feasibility of the following alternative system, total building drift, story displacement, and the period of the vibration were primarily used to aid in sizing the alternative system, and strength of members were checked afterwards. The following analysis was performed using ETABS as mentioned above along with spreadsheets and hand calculations. The goal of the following analysis was to determine how the loads are distributed through the alternative lateral system and the feasibility of the system. The load distribution was then used to check strength and serviceability of the concrete shear walls in the core option.

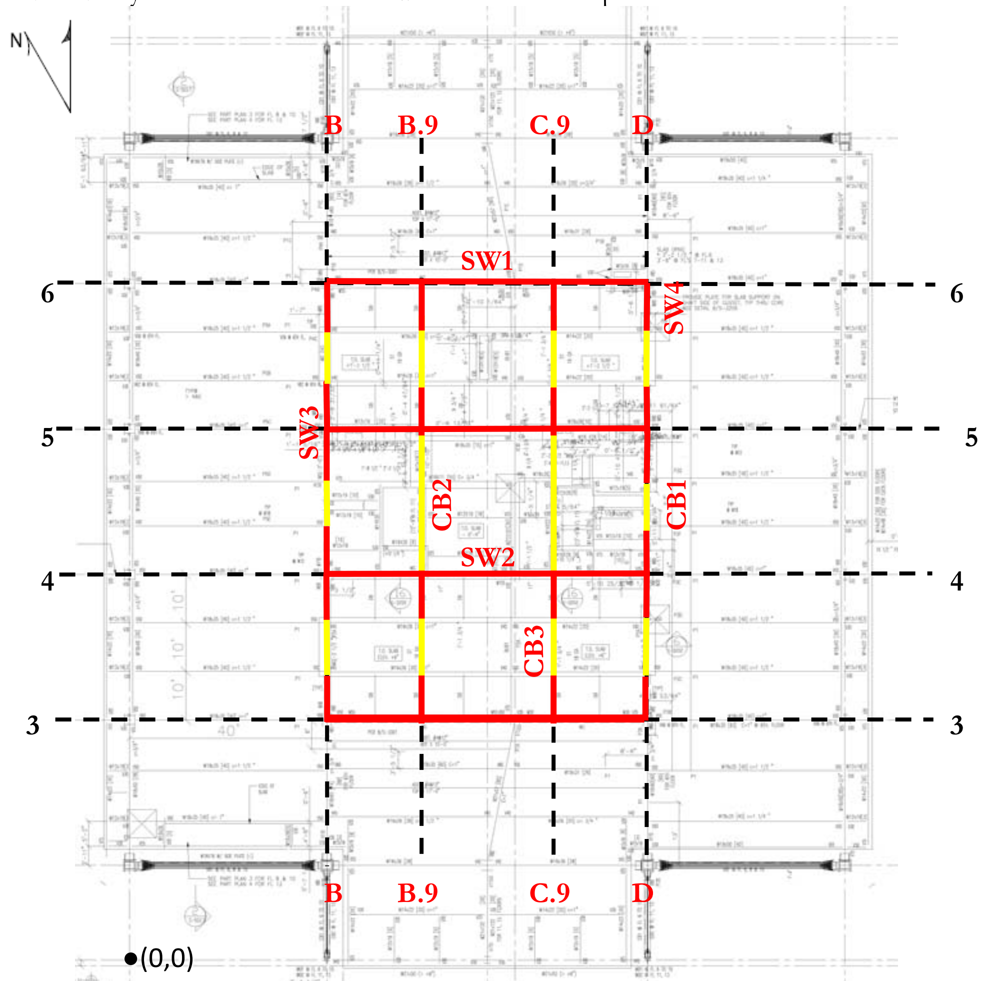


Figure 18: Concrete shear walls in the core

Relative Stiffness

The distribution of lateral forces to the concrete shear walls was determined based on each shear walls' relative stiffness. The shear walls were modeled in ETABS with fixed restraints at the base for the spread footing and cassion foundations. The floor diaphragms were assumed to be infinitely rigid which allowed lateral load distribution to each shear wall based on their stiffness. A 1000 kip horizontal load was applied in the North-South and West-East direction at the roof level. To determine the relative stiffness, section cuts were taken at the tenth floor, twenty eighth floor, and roof along grids 3, 4, 5, 6, B, B.9, C.9 and D to determine the amount of shear in the wall due to the 1000 kip load. This shear was then divided by the applied load at the roof level. The following tables summarize the relative stiffness of these three levels at the various grid lines.

10th Floor			
Frame (direction)	**Applied Load (kip)**	**Direct Shear (kip)**	**Relative Stiffness**
3 (W-E)	1000	208.44	0.208
4 (W-E)		291.4	0.291
5 (W-E)		291.4	0.291
6 (W-E)		208.44	0.208
Total		**999.68**	**1.000**
B (N-S)	1000	331.21	0.331
B.9 (N-S)		168.6	0.169
C.9 (N-S)		168.6	0.169
D (N-S)		331.21	0.331
Total		**999.62**	**1.000**

Table 1:10th Floor Relative Stiffness

28th Floor			
Frame (direction)	**Applied Load (kip)**	**Direct Shear (kip)**	**Relative Stiffness**
3 (W-E)	1000	206.9	0.207
4 (W-E)		292.97	0.293
5 (W-E)		292.97	0.293
6 (W-E)		206.9	0.207
Total		**999.74**	**1.000**
B (N-S)	1000	340.35	0.340
B.9 (N-S)		159.54	0.160
C.9 (N-S)		159.54	0.160
D (N-S)		340.35	0.340
Total		**999.78**	**1.000**

Table 2: 28th Floor Relative Stiffness

52nd Floor			
Frame (direction)	**Applied Load (kip)**	**Direct Shear (kip)**	**Relative Stiffness**
3 (W-E)	1000	209.85	0.210
4 (W-E)		290.1	0.290
5 (W-E)		290.11	0.290
6 (W-E)		209.86	0.210
Total		**999.92**	**1.000**
B (N-S)	1000	335.43	0.335
B.9 (N-S)		164.55	0.165
C.9 (N-S)		164.55	0.165
D (N-S)		335.43	0.335
Total		**999.96**	**1.000**

Table 3: 52nd Floor (Roof) Relative Stiffness

Center of Rigidity and Center of Mass

Using the relative stiffness of each shear wall in the above chart, the center of rigidity or COR was calculated. The reference origin was taken in the Southwestern corner where the grid lines of the exterior columns intersect. This location is also indicated on Figure 18 on page 22. Since the building is symmetrical in shape and the shear walls were modeled using centerline dimensions, the COR is the same on all floors. The center of mass or COM of the building is located in the middle of the building due its symmetrical shape and size. The following tables summarize the COR based on relative stiffness and ETABS output, and the COM based on building symmetry and ETABS output.

$$\text{COR: } \bar{x} = \frac{\sum k_{iy} x_i}{\sum k_{iy}} \quad \text{(Equation 4)}$$

$$\bar{y} = \frac{\sum k_{ix} y_i}{\sum k_{ix}} \quad \text{(Equation 5)}$$

COR: 10th Floor				
Frame (direction)	**Relative Stiffness**	**x_i (ft)**	**y_i (ft)**	**$\Sigma k_{iy}x_i$ / $\Sigma k_{ix}y_i$**
3 (W-E)	0.208	0	50	10.422
4 (W-E)	0.291	0	80	23.312
5 (W-E)	0.291	0	110	32.054
6 (W-E)	0.208	0	140	29.1816
Total W-E	**1.000**			**95.00**
B (N-S)	0.331	40	0	13.2484
B.9 (N-S)	0.169	59	0	9.9474
C.9 (N-S)	0.169	86	0	14.4996
D (N-S)	0.331	105	0	34.77705
Total N-S	**1.000**			**72.50**

Table 4: 10th Floor COR

COR: 28th Floor				
Frame (direction)	**Relative Stiffness**	**xi (ft)**	**y_i (ft)**	**$\Sigma k_{iy}x_i$ / $\Sigma k_{ix}y_i$**
3 (W-E)	0.207	0	50	10.345
4 (W-E)	0.293	0	80	23.4376
5 (W-E)	0.293	0	110	32.2267
6 (W-E)	0.207	0	140	28.966
Total W-E	**1.000**			**95.00**
B (N-S)	0.340	40	0	13.614
B.9 (N-S)	0.160	59	0	9.41286
C.9 (N-S)	0.160	86	0	13.72044
D (N-S)	0.340	105	0	35.73675
Total N-S	**1.000**			**72.50**

Table 5: 28th Floor COR

COR: 52nd Floor				
Frame (direction)	**Relative Stiffness**	**xi (ft)**	**y_i (ft)**	**$\Sigma k_{iy}x_i$ / $\Sigma k_{ix}y_i$**
3 (W-E)	0.210	0	50	10.4925
4 (W-E)	0.290	0	80	23.208
5 (W-E)	0.290	0	110	31.9121
6 (W-E)	0.210	0	140	29.3804
Total W-E	**1.000**			**95.00**
B (N-S)	0.335	40	0	13.4172
B.9 (N-S)	0.165	59	0	9.70845
C.9 (N-S)	0.165	86	0	14.1513
D (N-S)	0.335	105	0	35.22015
Total N-S	**1.000**			**72.50**

Table 6: 52th Floor (Roof) COR

ETABS COR OUTPUT		
STORY	**XCR (ft)**	**YCR (ft)**
1st Flr - Roof	72.50	95.00

Table 7: ETAB COR Output

ETABS COM OUTPUT		
STORY	**XCR (ft)**	**YCR (ft)**
1st Flr - Roof	72.50	95.00

Table 8: ETAB COM Output

From the tables, it is clear that the COR and the COM are in the same location due to symmetry. It also shows that the ETABS output matches the hand calculations for COR exactly. These results were expected due to the building and lateral system being symmetric. Because the COR is exactly located at the COM, moment due to torisional shear will not exist.

Strength Checks

Strength spot checks were performed on shear walls SW1-SW4 and coupling beams CB1-CB3 at the first floor, twenty sixth floor, and fifty first floor. Shear forces and moments on each element were determined from the 3D ETABS model. A summary of the forces on each shear wall and coupling beam can be produced upon request but has not been attached as part of this report. The following charts summarize the forces from the controlling load cases for the shear walls and coupling beams.

Controlling Design Forces in Shear Walls					
Story	**Pier**	**Controlling Load**	**Direction**	**V2 (k)**	**M3 (kin)**
51ST FLR	SW1	1.2D+1.6W	W-E	453.22	74,853.99
26TH FLR	SW1	1.2D+1.6W	W-E	2,053.50	2,412,330.36
1ST FLR	SW1	1.2D+1.6W	W-E	2,022.78	8,376,540.69
51ST FLR	SW2	1.2D+1.6W	W-E	707.52	84,365.90
26TH FLR	SW2	1.2D+1.6W	W-E	2,849.66	2,411,340.89
1ST FLR	SW2	1.2D+1.6W	W-E	2,430.79	8,084,650.03
51ST FLR	SW3	1.2D+1.6W	N-S	365.01	33,635.23
26TH FLR	SW3	1.2D+1.6W	N-S	1,122.34	128,832.59
1ST FLR	SW3	1.2D+1.6W	N-S	-553.26	377,070.86
51ST FLR	SW4	1.2D+1.6W	N-S	70.76	5,581.74
26TH FLR	SW4	1.2D+1.6W	N-S	292.54	25,395.06
1ST FLR	SW4	1.2D+1.6W	N-S	-297.13	42,137.44

Table 9: Controlling Design Forces in Shear Walls

Controlling Design Forces in Coupling Beams						
Story	**Spandrel**	**Controlling Load**	**Direction**	**Loc**	**V2 (k)**	**M3 (kin)**
51ST FLR	CB1	1.2D+1.6W	N-S	Left	73.95	4,816.46
51ST FLR	CB1	1.2D+1.6W	N-S	Right	90.15	-5,029.02
26TH FLR	CB1	1.2D+1.6W	N-S	Left	304.44	18,645.78
26TH FLR	CB1	1.2D+1.6W	N-S	Right	320.64	-18,859.20
1ST FLR	CB1	1.2D+1.6W	N-S	Left	210.77	12,990.81
1ST FLR	CB1	1.2D+1.6W	N-S	Right	226.96	-13,273.00
51ST FLR	CB2	1.2D+1.6W	N-S	Left	-0.35	3,017.20
51ST FLR	CB2	1.2D+1.6W	N-S	Right	48.25	-5,604.51
26TH FLR	CB2	1.2D+1.6W	N-S	Left	31.28	8,737.53

26TH FLR	CB2	1.2D+1.6W	N-S	Right	79.88	-11,272.15
1ST FLR	CB2	1.2D+1.6W	N-S	Left	-2.39	2,626.50
1ST FLR	CB2	1.2D+1.6W	N-S	Right	46.2	-5,259.27
51ST FLR	CB3	1.2D+1.6W	N-S	Left	68.57	4,499.84
51ST FLR	CB3	1.2D+1.6W	N-S	Right	84.77	-4,700.29
26TH FLR	CB3	1.2D+1.6W	N-S	Left	279.29	17,155.96
26TH FLR	CB3	1.2D+1.6W	N-S	Right	295.49	-17,330.53
1ST FLR	CB3	1.2D+1.6W	N-S	Left	204.28	12,637.58
1ST FLR	CB3	1.2D+1.6W	N-S	Right	220.48	-12,847.65

Table 10: Controlling Design Forces in Coupling Beams

It was determined that SW1-SW4 met strength requirements for shear with minimum reinforcement. The shear walls also met flexural strength capacity. The following table summarizes shear and flexural capacity as well as number and sizes of rebar for shear walls. For more in depth information and calculations refer to Appendix C.

Shear Wall Capacities and Reinforcing						
Story	**Pier**	**Direction**	**ϕV_c (k)**	**Hor. & Vert Rein.**	**ϕM_n (kin)**	**Flex. Rein.**
51ST FLR	SW1	W-E	2486	#7 @12	131,943	(4) #8
26TH FLR	SW1	W-E	3707	#8 @12	3,035,606	(58) #10
1ST FLR	SW1	W-E	5075	#9 @12	10,522,742	(205) #10
51ST FLR	SW2	W-E	2486	#7 @12	131,943	(4) #8
26TH FLR	SW2	W-E	3707	#8 @12	3,035,606	(58) #10
1ST FLR	SW2	W-E	5075	#9 @12	10,522,742	(205) #10
51ST FLR	SW3	N-S	1275	#9 @12	45,387	(6) #7
26TH FLR	SW3	N-S	1426	#9 @12	165,688	(22) #7
1ST FLR	SW3	N-S	1562	#9 @12	462,733	(62) #7
51ST FLR	SW4	N-S	257	#9 @12	7,376	(2) #7
26TH FLR	SW4	N-S	713	#9 @12	36,707	(10) #7
1ST FLR	SW4	N-S	316	#9 @12	51,337	(14) #7

Table 11: Shear Wall Capacities and Reinforcing

It was determined that the coupling beam met strength requirements for shear with minimum reinforcement at the fifty first floor for CB1 and CB3 and at all floors for CB2. These results make sense since the capacity of the concrete alone is enough to take the shear in the beam. The coupling beams also met flexural strength capacity. The following table summarizes shear and flexural capacity as well as number and sizes of rebar for the coupling beams. It should also be noted that maximum negative and positive moments occur at the ends of the beams where they frame into the shear walls. At the ends, reinforcing is provided at the top and bottom to combat wind reversal on the structure. Further investigation is required in order to understand how to optimize the coupling beam design and placement of rebar where positive moments occur at locations other than at the midspan of the beam. For more in depth information and calculations refer to Appendix C.

Coupling Beams Capacities and Reinforcing					
Story	**Spandrel**	**Direction**	**Shear Rein.**	**ϕM_n (kin)**	**Flex. Rein.**
51ST FLR	CB1	N-S	#4 @10	8,830	(5) #9
26TH FLR	CB1	N-S	#4 @10	19,093	(11) #9
1ST FLR	CB1	N-S	#5 @10	14,106	(8) #9
51ST FLR	CB2	N-S	#4 @10	8,830	(5) #9

26TH FLR	CB2	N-S	#4 @10	12,328	(7) #9
1ST FLR	CB2	N-S	#5 @10	12,380	(7) #9
51ST FLR	CB3	N-S	#4 @10	12,250	(7) #9
26TH FLR	CB3	N-S	#4 @10	19,093	(11) #9
1ST FLR	CB3	N-S	#5 @10	14,106	(8) #9

Table 12: Coupling Beam Capacities and Reinforcing

Building and Inter-story Drift

Wind and seismic drifts were computed by ETABS using equation 3, D + 0.5L + 0.7W for case 1 and case 3 wind and unfactored for seismic. These drifts were then compared to drift limitation by code. Wind drift was compared to Δ_{wind} = H/450 for the entire building and inter-story drift in the North-South and West-East directions. Seismic drift was compared to $\Delta_{seismic}$ = 0.015h_{sx} for each floor level in the North-South and West-East directions. The following charts summarize the story drift and building drift due to wind and seismic loads based on ETABS output.

West-East Case 1 Wind									
Story	**Story Height Below (ft)**	**Story Drift (in)**	**Allowable Story Drift (in)**			**Total Drift (in)**	**Allowable Total Drift (in)**		
			Δ_{wind} = H/450				**Δ_{wind} = H/450**		
2nd	26.99	0.103	<	0.720	OK	0.103	<	0.720	OK
3rd	15.47	0.080	<	0.413	OK	0.183	<	1.132	OK
4th	15.47	0.101	<	0.413	OK	0.284	<	1.545	OK
5th	14.32	0.111	<	0.382	OK	0.395	<	1.927	OK
6th	13.75	0.123	<	0.367	OK	0.518	<	2.293	OK
7th	13.75	0.137	<	0.367	OK	0.655	<	2.660	OK
8th	13.75	0.152	<	0.367	OK	0.807	<	3.027	OK
9th	13.75	0.165	<	0.367	OK	0.972	<	3.393	OK
10th	13.75	0.178	<	0.367	OK	1.150	<	3.760	OK
11th	13.75	0.191	<	0.367	OK	1.341	<	4.127	OK
12th	14.25	0.212	<	0.380	OK	1.553	<	4.507	OK
13th	13.25	0.209	<	0.353	OK	1.762	<	4.860	OK
14th	13.75	0.229	<	0.367	OK	1.992	<	5.227	OK
15th	13.75	0.241	<	0.367	OK	2.233	<	5.593	OK
16th	13.75	0.252	<	0.367	OK	2.485	<	5.960	OK
17th	13.75	0.263	<	0.367	OK	2.748	<	6.327	OK
18th	13.75	0.273	<	0.367	OK	3.021	<	6.693	OK
19th	13.75	0.283	<	0.367	OK	3.304	<	7.060	OK
20th	13.75	0.292	<	0.367	OK	3.596	<	7.427	OK
21st	13.75	0.301	<	0.367	OK	3.897	<	7.793	OK
22nd	13.75	0.311	<	0.367	OK	4.208	<	8.160	OK
23rd	13.75	0.319	<	0.367	OK	4.527	<	8.527	OK
24th	13.75	0.328	<	0.367	OK	4.855	<	8.893	OK
25th	13.75	0.335	<	0.367	OK	5.190	<	9.260	OK
26th	13.75	0.342	<	0.367	OK	5.532	<	9.627	OK
27th	13.75	0.349	<	0.367	OK	5.882	<	9.993	OK
28th	13.25	0.343	<	0.353	OK	6.224	<	10.347	OK
29th	27.50	0.729	<	0.733	OK	6.953	<	11.080	OK
30th	13.75	0.372	>	0.367	NOT GOOD	7.325	<	11.447	OK
31st	13.75	0.377	>	0.367	NOT GOOD	7.703	<	11.813	OK
32nd	13.75	0.383	>	0.367	NOT GOOD	8.085	<	12.180	OK

33rd	13.75	0.387	>	0.367	NOT GOOD	8.473	<	12.547	OK
34th	13.75	0.392	>	0.367	NOT GOOD	8.864	<	12.913	OK
35th	13.75	0.396	>	0.367	NOT GOOD	9.260	<	13.280	OK
36th	13.75	0.399	>	0.367	NOT GOOD	9.659	<	13.647	OK
37th	13.75	0.402	>	0.367	NOT GOOD	10.061	<	14.013	OK
38th	13.75	0.405	>	0.367	NOT GOOD	10.466	<	14.380	OK
39th	13.75	0.408	>	0.367	NOT GOOD	10.874	<	14.747	OK
40th	13.75	0.410	>	0.367	NOT GOOD	11.285	<	15.113	OK
41st	13.75	0.413	>	0.367	NOT GOOD	11.697	<	15.480	OK
42nd	13.75	0.415	>	0.367	NOT GOOD	12.112	<	15.847	OK
43rd	13.75	0.417	>	0.367	NOT GOOD	12.529	<	16.213	OK
44th	13.75	0.418	>	0.367	NOT GOOD	12.948	<	16.580	OK
45th	13.75	0.420	>	0.367	NOT GOOD	13.367	<	16.947	OK
46th	13.75	0.421	>	0.367	NOT GOOD	13.788	<	17.313	OK
47th	13.75	0.421	>	0.367	NOT GOOD	14.209	<	17.680	OK
48th	13.75	0.422	>	0.367	NOT GOOD	14.631	<	18.047	OK
49th	13.75	0.422	>	0.367	NOT GOOD	15.054	<	18.413	OK
50th	13.75	0.423	>	0.367	NOT GOOD	15.476	<	18.780	OK
51st	14.67	0.451	>	0.391	NOT GOOD	15.927	<	19.171	OK
Roof	27.08	0.834	>	0.722	NOT GOOD	16.761	<	19.893	OK

Table 13: Case 1 West-East Wind Drifts

North-South Case 1 Wind

Story	**Story Height Below (ft)**	**Story Drift (in)**	**Allowable Story Drift (in)**			**Total Drift (in)**	**Allowable Total Drift (in)**		
			Δ_{wind} = H/450				**Δ_{wind} = H/450**		
2nd	26.99	0.170	<	0.720	OK	0.170	<	0.720	OK
3rd	15.47	0.118	<	0.413	OK	0.288	<	1.132	OK
4th	15.47	0.133	<	0.413	OK	0.421	<	1.545	OK
5th	14.32	0.134	<	0.382	OK	0.555	<	1.927	OK
6th	13.75	0.136	<	0.367	OK	0.692	<	2.293	OK
7th	13.75	0.143	<	0.367	OK	0.835	<	2.660	OK
8th	13.75	0.150	<	0.367	OK	0.985	<	3.027	OK
9th	13.75	0.156	<	0.367	OK	1.140	<	3.393	OK
10th	13.75	0.161	<	0.367	OK	1.302	<	3.760	OK
11th	13.75	0.167	<	0.367	OK	1.468	<	4.127	OK
12th	14.25	0.180	<	0.380	OK	1.648	<	4.507	OK
13th	13.25	0.172	<	0.353	OK	1.821	<	4.860	OK
14th	13.75	0.184	<	0.367	OK	2.005	<	5.227	OK
15th	13.75	0.188	<	0.367	OK	2.193	<	5.593	OK
16th	13.75	0.192	<	0.367	OK	2.385	<	5.960	OK
17th	13.75	0.195	<	0.367	OK	2.580	<	6.327	OK
18th	13.75	0.199	<	0.367	OK	2.779	<	6.693	OK
19th	13.75	0.201	<	0.367	OK	2.980	<	7.060	OK
20th	13.75	0.204	<	0.367	OK	3.184	<	7.427	OK
21st	13.75	0.206	<	0.367	OK	3.390	<	7.793	OK
22nd	13.75	0.209	<	0.367	OK	3.599	<	8.160	OK
23rd	13.75	0.212	<	0.367	OK	3.811	<	8.527	OK
24th	13.75	0.215	<	0.367	OK	4.026	<	8.893	OK
25th	13.75	0.217	<	0.367	OK	4.243	<	9.260	OK
26th	13.75	0.220	<	0.367	OK	4.464	<	9.627	OK

27th	13.75	0.224	<	0.367	OK	4.687	<	9.993	OK
28th	13.25	0.219	<	0.353	OK	4.906	<	10.347	OK
29th	27.50	0.469	<	0.733	OK	5.376	<	11.080	OK
30th	13.75	0.231	<	0.367	OK	5.607	<	11.447	OK
31st	13.75	0.230	<	0.367	OK	5.837	<	11.813	OK
32nd	13.75	0.230	<	0.367	OK	6.067	<	12.180	OK
33rd	13.75	0.230	<	0.367	OK	6.297	<	12.547	OK
34th	13.75	0.230	<	0.367	OK	6.528	<	12.913	OK
35th	13.75	0.230	<	0.367	OK	6.758	<	13.280	OK
36th	13.75	0.230	<	0.367	OK	6.987	<	13.647	OK
37th	13.75	0.229	<	0.367	OK	7.217	<	14.013	OK
38th	13.75	0.229	<	0.367	OK	7.445	<	14.380	OK
39th	13.75	0.228	<	0.367	OK	7.673	<	14.747	OK
40th	13.75	0.227	<	0.367	OK	7.900	<	15.113	OK
41st	13.75	0.226	<	0.367	OK	8.125	<	15.480	OK
42nd	13.75	0.225	<	0.367	OK	8.350	<	15.847	OK
43rd	13.75	0.224	<	0.367	OK	8.573	<	16.213	OK
44th	13.75	0.222	<	0.367	OK	8.796	<	16.580	OK
45th	13.75	0.221	<	0.367	OK	9.017	<	16.947	OK
46th	13.75	0.220	<	0.367	OK	9.237	<	17.313	OK
47th	13.75	0.218	<	0.367	OK	9.455	<	17.680	OK
48th	13.75	0.217	<	0.367	OK	9.672	<	18.047	OK
49th	13.75	0.216	<	0.367	OK	9.888	<	18.413	OK
50th	13.75	0.215	<	0.367	OK	10.102	<	18.780	OK
51st	14.67	0.229	<	0.391	OK	10.331	<	19.171	OK
Roof	27.08	0.424	<	0.722	OK	10.755	<	19.893	OK

Table 14: Case 1 North-South Wind Drifts

West-East Case 3 Wind									
Story	**Story Height Below (ft)**	**Story Drift (in)**	**Allowable Story Drift (in)**			**Total Drift (in)**	**Allowable Total Drift (in)**		
			Δ_{wind} = H/450				**Δ_{wind} = H/450**		
2nd	26.99	0.078	<	0.720	OK	0.0775	<	0.720	OK
3rd	15.47	0.060	<	0.413	OK	0.1374	<	1.132	OK
4th	15.47	0.076	<	0.413	OK	0.213	<	1.545	OK
5th	14.32	0.084	<	0.382	OK	0.2965	<	1.927	OK
6th	13.75	0.092	<	0.367	OK	0.3884	<	2.293	OK
7th	13.75	0.103	<	0.367	OK	0.4913	<	2.660	OK
8th	13.75	0.114	<	0.367	OK	0.6049	<	3.027	OK
9th	13.75	0.124	<	0.367	OK	0.7288	<	3.393	OK
10th	13.75	0.134	<	0.367	OK	0.8626	<	3.760	OK
11th	13.75	0.143	<	0.367	OK	1.0059	<	4.127	OK
12th	14.25	0.159	<	0.380	OK	1.1648	<	4.507	OK
13th	13.25	0.157	<	0.353	OK	1.3218	<	4.860	OK
14th	13.75	0.172	<	0.367	OK	1.4937	<	5.227	OK
15th	13.75	0.181	<	0.367	OK	1.6744	<	5.593	OK
16th	13.75	0.189	<	0.367	OK	1.8636	<	5.960	OK
17th	13.75	0.197	<	0.367	OK	2.0608	<	6.327	OK
18th	13.75	0.205	<	0.367	OK	2.2656	<	6.693	OK
19th	13.75	0.212	<	0.367	OK	2.4778	<	7.060	OK
20th	13.75	0.219	<	0.367	OK	2.697	<	7.427	OK

21st	13.75	0.226	<	0.367	OK	2.9228	<	7.793	OK
22nd	13.75	0.233	<	0.367	OK	3.1559	<	8.160	OK
23rd	13.75	0.240	<	0.367	OK	3.3954	<	8.527	OK
24th	13.75	0.246	<	0.367	OK	3.641	<	8.893	OK
25th	13.75	0.251	<	0.367	OK	3.8924	<	9.260	OK
26th	13.75	0.257	<	0.367	OK	4.1493	<	9.627	OK
27th	13.75	0.262	<	0.367	OK	4.4112	<	9.993	OK
28th	13.25	0.257	<	0.353	OK	4.6682	<	10.347	OK
29th	27.50	0.547	<	0.733	OK	5.2148	<	11.080	OK
30th	13.75	0.279	<	0.367	OK	5.4941	<	11.447	OK
31st	13.75	0.283	<	0.367	OK	5.7771	<	11.813	OK
32nd	13.75	0.287	<	0.367	OK	6.064	<	12.180	OK
33rd	13.75	0.290	<	0.367	OK	6.3544	<	12.547	OK
34th	13.75	0.294	<	0.367	OK	6.6481	<	12.913	OK
35th	13.75	0.297	<	0.367	OK	6.9447	<	13.280	OK
36th	13.75	0.299	<	0.367	OK	7.244	<	13.647	OK
37th	13.75	0.302	<	0.367	OK	7.5458	<	14.013	OK
38th	13.75	0.304	<	0.367	OK	7.8497	<	14.380	OK
39th	13.75	0.306	<	0.367	OK	8.1557	<	14.747	OK
40th	13.75	0.308	<	0.367	OK	8.4635	<	15.113	OK
41st	13.75	0.309	<	0.367	OK	8.7729	<	15.480	OK
42nd	13.75	0.311	<	0.367	OK	9.0843	<	15.847	OK
43rd	13.75	0.313	<	0.367	OK	9.3969	<	16.213	OK
44th	13.75	0.314	<	0.367	OK	9.7107	<	16.580	OK
45th	13.75	0.315	<	0.367	OK	10.0254	<	16.947	OK
46th	13.75	0.315	<	0.367	OK	10.3408	<	17.313	OK
47th	13.75	0.316	<	0.367	OK	10.6568	<	17.680	OK
48th	13.75	0.317	<	0.367	OK	10.9733	<	18.047	OK
49th	13.75	0.317	<	0.367	OK	11.2901	<	18.413	OK
50th	13.75	0.317	<	0.367	OK	11.6072	<	18.780	OK
51st	14.67	0.338	<	0.391	OK	11.9455	<	19.171	OK
Roof	27.08	0.625	<	0.722	OK	12.5707	<	19.893	OK

Table 15: Case 3 West-East Wind Drifts

North-South Case 3 Wind

Story	**Story Height Below (ft)**	**Story Drift (in)**	**Allowable Story Drift (in)**			**Total Drift (in)**	**Allowable Total Drift (in)**		
			Δ_{wind} = H/450				**Δ_{wind} = H/450**		
2nd	26.99	0.127	<	0.720	OK	0.1271	<	0.720	OK
3rd	15.47	0.089	<	0.413	OK	0.2159	<	1.132	OK
4th	15.47	0.100	<	0.413	OK	0.316	<	1.545	OK
5th	14.32	0.100	<	0.382	OK	0.4163	<	1.927	OK
6th	13.75	0.102	<	0.367	OK	0.5187	<	2.293	OK
7th	13.75	0.108	<	0.367	OK	0.6262	<	2.660	OK
8th	13.75	0.112	<	0.367	OK	0.7385	<	3.027	OK
9th	13.75	0.117	<	0.367	OK	0.8552	<	3.393	OK
10th	13.75	0.121	<	0.367	OK	0.9762	<	3.760	OK
11th	13.75	0.125	<	0.367	OK	1.1013	<	4.127	OK
12th	14.25	0.135	<	0.380	OK	1.2362	<	4.507	OK
13th	13.25	0.129	<	0.353	OK	1.3655	<	4.860	OK
14th	13.75	0.138	<	0.367	OK	1.5034	<	5.227	OK

15th	13.75	0.141	<	0.367	OK	1.6445	<	5.593	OK
16th	13.75	0.144	<	0.367	OK	1.7884	<	5.960	OK
17th	13.75	0.147	<	0.367	OK	1.935	<	6.327	OK
18th	13.75	0.149	<	0.367	OK	2.0839	<	6.693	OK
19th	13.75	0.151	<	0.367	OK	2.235	<	7.060	OK
20th	13.75	0.153	<	0.367	OK	2.388	<	7.427	OK
21st	13.75	0.155	<	0.367	OK	2.5427	<	7.793	OK
22nd	13.75	0.157	<	0.367	OK	2.6994	<	8.160	OK
23rd	13.75	0.159	<	0.367	OK	2.8583	<	8.527	OK
24th	13.75	0.161	<	0.367	OK	3.0194	<	8.893	OK
25th	13.75	0.163	<	0.367	OK	3.1825	<	9.260	OK
26th	13.75	0.165	<	0.367	OK	3.3477	<	9.627	OK
27th	13.75	0.168	<	0.367	OK	3.5153	<	9.993	OK
28th	13.25	0.164	<	0.353	OK	3.6796	<	10.347	OK
29th	27.50	0.352	<	0.733	OK	4.0317	<	11.080	OK
30th	13.75	0.173	<	0.367	OK	4.205	<	11.447	OK
31st	13.75	0.172	<	0.367	OK	4.3774	<	11.813	OK
32nd	13.75	0.173	<	0.367	OK	4.5501	<	12.180	OK
33rd	13.75	0.173	<	0.367	OK	4.7229	<	12.547	OK
34th	13.75	0.173	<	0.367	OK	4.8956	<	12.913	OK
35th	13.75	0.173	<	0.367	OK	5.0682	<	13.280	OK
36th	13.75	0.172	<	0.367	OK	5.2405	<	13.647	OK
37th	13.75	0.172	<	0.367	OK	5.4124	<	14.013	OK
38th	13.75	0.171	<	0.367	OK	5.5838	<	14.380	OK
39th	13.75	0.171	<	0.367	OK	5.7546	<	14.747	OK
40th	13.75	0.170	<	0.367	OK	5.9247	<	15.113	OK
41st	13.75	0.169	<	0.367	OK	6.0939	<	15.480	OK
42nd	13.75	0.168	<	0.367	OK	6.2623	<	15.847	OK
43rd	13.75	0.168	<	0.367	OK	6.43	<	16.213	OK
44th	13.75	0.167	<	0.367	OK	6.5968	<	16.580	OK
45th	13.75	0.166	<	0.367	OK	6.7626	<	16.947	OK
46th	13.75	0.165	<	0.367	OK	6.9274	<	17.313	OK
47th	13.75	0.164	<	0.367	OK	7.0911	<	17.680	OK
48th	13.75	0.163	<	0.367	OK	7.2538	<	18.047	OK
49th	13.75	0.162	<	0.367	OK	7.4157	<	18.413	OK
50th	13.75	0.161	<	0.367	OK	7.5768	<	18.780	OK
51st	14.67	0.172	<	0.391	OK	7.7483	<	19.171	OK
Roof	27.08	0.318	<	0.722	OK	8.0661	<	19.893	OK

Table 16: Case 3 North-South Wind Drifts

North-South Seismic									
Story	**Story Height Below (ft)**	**Story Drift (in)**	**Allowable Story Drift (in)** $\Delta_{seismic} = 0.0015h_{sx}$			**Total Drift (in)**	**Allowable Total Drift (in)** $\Delta_{seismic} = 0.0015h_{sx}$		
2nd	26.99	0.074	<	0.486	OK	0.074	<	0.486	OK
3rd	15.47	0.054	<	0.278	OK	0.128	<	0.764	OK
4th	15.47	0.062	<	0.278	OK	0.190	<	1.043	OK
5th	14.32	0.063	<	0.258	OK	0.253	<	1.301	OK
6th	13.75	0.066	<	0.248	OK	0.319	<	1.548	OK
7th	13.75	0.070	<	0.248	OK	0.388	<	1.796	OK
8th	13.75	0.074	<	0.248	OK	0.462	<	2.043	OK

9th	13.75	0.078	<	0.248	OK	0.540	<	2.291	OK
10th	13.75	0.082	<	0.248	OK	0.622	<	2.538	OK
11th	13.75	0.086	<	0.248	OK	0.708	<	2.786	OK
12th	14.25	0.093	<	0.257	OK	0.801	<	3.042	OK
13th	13.25	0.090	<	0.239	OK	0.891	<	3.281	OK
14th	13.75	0.097	<	0.248	OK	0.989	<	3.528	OK
15th	13.75	0.101	<	0.248	OK	1.089	<	3.776	OK
16th	13.75	0.104	<	0.248	OK	1.193	<	4.023	OK
17th	13.75	0.106	<	0.248	OK	1.299	<	4.271	OK
18th	13.75	0.109	<	0.248	OK	1.408	<	4.518	OK
19th	13.75	0.112	<	0.248	OK	1.520	<	4.766	OK
20th	13.75	0.114	<	0.248	OK	1.634	<	5.013	OK
21st	13.75	0.116	<	0.248	OK	1.749	<	5.261	OK
22nd	13.75	0.118	<	0.248	OK	1.868	<	5.508	OK
23rd	13.75	0.121	<	0.248	OK	1.988	<	5.756	OK
24th	13.75	0.123	<	0.248	OK	2.112	<	6.003	OK
25th	13.75	0.126	<	0.248	OK	2.237	<	6.251	OK
26th	13.75	0.128	<	0.248	OK	2.365	<	6.498	OK
27th	13.75	0.131	<	0.248	OK	2.496	<	6.746	OK
28th	13.25	0.129	<	0.239	OK	2.624	<	6.984	OK
29th	27.50	0.278	<	0.495	OK	2.903	<	7.479	OK
30th	13.75	0.138	<	0.248	OK	3.040	<	7.727	OK
31st	13.75	0.137	<	0.248	OK	3.178	<	7.974	OK
32nd	13.75	0.138	<	0.248	OK	3.316	<	8.222	OK
33rd	13.75	0.138	<	0.248	OK	3.454	<	8.469	OK
34th	13.75	0.139	<	0.248	OK	3.593	<	8.717	OK
35th	13.75	0.139	<	0.248	OK	3.731	<	8.964	OK
36th	13.75	0.139	<	0.248	OK	3.870	<	9.212	OK
37th	13.75	0.138	<	0.248	OK	4.009	<	9.459	OK
38th	13.75	0.138	<	0.248	OK	4.147	<	9.707	OK
39th	13.75	0.137	<	0.248	OK	4.284	<	9.954	OK
40th	13.75	0.137	<	0.248	OK	4.421	<	10.202	OK
41st	13.75	0.136	<	0.248	OK	4.557	<	10.449	OK
42nd	13.75	0.135	<	0.248	OK	4.692	<	10.697	OK
43rd	13.75	0.134	<	0.248	OK	4.826	<	10.944	OK
44th	13.75	0.133	<	0.248	OK	4.958	<	11.192	OK
45th	13.75	0.131	<	0.248	OK	5.090	<	11.439	OK
46th	13.75	0.130	<	0.248	OK	5.220	<	11.687	OK
47th	13.75	0.129	<	0.248	OK	5.348	<	11.934	OK
48th	13.75	0.127	<	0.248	OK	5.475	<	12.182	OK
49th	13.75	0.125	<	0.248	OK	5.601	<	12.429	OK
50th	13.75	0.124	<	0.248	OK	5.725	<	12.677	OK
51st	14.67	0.131	<	0.264	OK	5.856	<	12.941	OK
Roof	27.08	0.241	<	0.488	OK	6.097	<	13.428	OK

Table 17: North-South Seismic Drifts

West-East Seismic

Story	**Story Height Below (ft)**	**Story Drift (in)**	**Allowable Story Drift (in)** $\Delta_{seismic} = 0.0015h_{sx}$			**Total Drift (in)**	**Allowable Total Drift (in)** $\Delta_{seismic} = 0.0015h_{sx}$		
2nd	26.99	0.045	<	0.486	OK	0.045	<	0.486	OK

3rd	15.47	0.036	<	0.278	OK	0.081	<	0.764	OK
4th	15.47	0.046	<	0.278	OK	0.127	<	1.043	OK
5th	14.32	0.051	<	0.258	OK	0.178	<	1.301	OK
6th	13.75	0.057	<	0.248	OK	0.235	<	1.548	OK
7th	13.75	0.064	<	0.248	OK	0.299	<	1.796	OK
8th	13.75	0.071	<	0.248	OK	0.370	<	2.043	OK
9th	13.75	0.078	<	0.248	OK	0.447	<	2.291	OK
10th	13.75	0.084	<	0.248	OK	0.532	<	2.538	OK
11th	13.75	0.091	<	0.248	OK	0.622	<	2.786	OK
12th	14.25	0.101	<	0.257	OK	0.723	<	3.042	OK
13th	13.25	0.100	<	0.239	OK	0.823	<	3.281	OK
14th	13.75	0.110	<	0.248	OK	0.933	<	3.528	OK
15th	13.75	0.116	<	0.248	OK	1.049	<	3.776	OK
16th	13.75	0.122	<	0.248	OK	1.171	<	4.023	OK
17th	13.75	0.128	<	0.248	OK	1.299	<	4.271	OK
18th	13.75	0.133	<	0.248	OK	1.432	<	4.518	OK
19th	13.75	0.138	<	0.248	OK	1.570	<	4.766	OK
20th	13.75	0.143	<	0.248	OK	1.714	<	5.013	OK
21st	13.75	0.148	<	0.248	OK	1.862	<	5.261	OK
22nd	13.75	0.153	<	0.248	OK	2.015	<	5.508	OK
23rd	13.75	0.158	<	0.248	OK	2.173	<	5.756	OK
24th	13.75	0.162	<	0.248	OK	2.335	<	6.003	OK
25th	13.75	0.167	<	0.248	OK	2.502	<	6.251	OK
26th	13.75	0.171	<	0.248	OK	2.673	<	6.498	OK
27th	13.75	0.175	<	0.248	OK	2.847	<	6.746	OK
28th	13.25	0.172	<	0.239	OK	3.019	<	6.984	OK
29th	27.50	0.366	<	0.495	OK	3.385	<	7.479	OK
30th	13.75	0.188	<	0.248	OK	3.572	<	7.727	OK
31st	13.75	0.190	<	0.248	OK	3.763	<	7.974	OK
32nd	13.75	0.193	<	0.248	OK	3.956	<	8.222	OK
33rd	13.75	0.196	<	0.248	OK	4.152	<	8.469	OK
34th	13.75	0.199	<	0.248	OK	4.351	<	8.717	OK
35th	13.75	0.201	<	0.248	OK	4.551	<	8.964	OK
36th	13.75	0.203	<	0.248	OK	4.754	<	9.212	OK
37th	13.75	0.205	<	0.248	OK	4.959	<	9.459	OK
38th	13.75	0.206	<	0.248	OK	5.165	<	9.707	OK
39th	13.75	0.208	<	0.248	OK	5.373	<	9.954	OK
40th	13.75	0.209	<	0.248	OK	5.582	<	10.202	OK
41st	13.75	0.210	<	0.248	OK	5.792	<	10.449	OK
42nd	13.75	0.212	<	0.248	OK	6.003	<	10.697	OK
43rd	13.75	0.212	<	0.248	OK	6.216	<	10.944	OK
44th	13.75	0.213	<	0.248	OK	6.429	<	11.192	OK
45th	13.75	0.214	<	0.248	OK	6.643	<	11.439	OK
46th	13.75	0.214	<	0.248	OK	6.857	<	11.687	OK
47th	13.75	0.215	<	0.248	OK	7.071	<	11.934	OK
48th	13.75	0.215	<	0.248	OK	7.286	<	12.182	OK
49th	13.75	0.215	<	0.248	OK	7.501	<	12.429	OK
50th	13.75	0.215	<	0.248	OK	7.716	<	12.677	OK
51st	14.67	0.229	<	0.264	OK	7.946	<	12.941	OK
Roof	27.08	0.424	<	0.488	OK	8.369	<	13.428	OK

Table 18: West-East Seismic Drifts

From the tables on the previous pages, the total building drifts are acceptable in the North-South and West-East direction for case 1 and case 3 wind and seismic loading. However, story drifts were acceptable in the cases stated above except for case 1 wind in the West-East direction. When looking at the seismic and wind loads, the drifts for case 1 wind is greater than seismic even though the wind loads are reduce by thirty percent. These drifts results show that wind controls drift design, which is expected in New York City. The 23 unacceptable story drift in the West-East wind case 1 was unexpected. However it should be stated when evaluating wind drifts using ASCE7-05 that values are based on common standard practice. Also, the difference between the allowable story drift and actual story drift ranges from 0.006 inches to 0.111 inches and total to 1 inch drift from the twenty ninth floor to the roof. It should also be noted that the total building drift is acceptable, therefore it can be concluded that the story drift due to wind is ok unless façade drifts or a senior engineer requires otherwise.

Overturning Moment and Foundation Impact

The overturning moment due to wind controlled the design of the shear walls that were checked in the analysis section of the report. As stated above, the shear walls are able to take the overturning moment and are not an issue with the walls. However, the foundation could be impacted by the overturning moment. The New York Times Building sits on spread footings and cassions, but due to placement and sizes of the foundations being confidential, it is unknown if the exiting foundations can carry the additional self-weight of this alternative lateral system and the changes in overturning moment due to the alternative lateral system.

CONCLUSIONS

After conducting an alternative lateral system analysis of The New York Times Building, better insight into the feasibility of an alternative lateral system was gained. ETABS was utilized in creating 3D models of the alternative lateral systems investigated in this report. Lateral elements were checked and analyzed for strength and serviceability requirements. It was determined that wind was the controlling load for The New York Times Building, which was expected. The lateral loads applied to the alternative systems are resisted by the use of concrete shear walls, outriggers, and chevron bracing. Due to the symmetry of the building, the center of mass and the center of rigidity are located at the same spot, which was verified using ETABS and hand calculations. Torsional shear is not an issue because the same location of the COR and COM. Total building drift due to wind and story drift due to seismic are not an issue either. However if the story drifts due to case 1 wind in the West-East direction are deemed unacceptable by a senior engineer or façade drift limits, these drifts will have to decrease and lateral stiffness on those floor must increase.

As stated above, the concrete shear walls in the core alternative lateral system yielded periods of vibrations of 6.893 seconds, 7.709 seconds, and 3.690 seconds in the North-South, West-East, and torsional directions respectively for seismic and 5.926 seconds, 6.528 seconds, and 3.265 seconds in the North-South, West-East, and torsional directions respectively for wind. Total building displacement was 6.10 inches in the North-South direction and 8.37inches in the West-East direction due to seismic loading. Total building displacement was 16.76 inches in the North-South direction and 10.76 inches in the West-East direction for case 1 wind and 8.07 inches in the North-South direction and 12.57 inches in the West-East direction for case 3 wind.

The concrete shear walls in the core with outriggers alternative lateral system produced periods of vibrations of 6.97 seconds, 6.23 seconds, and 4.88 seconds in the North-South, West-East and torsional directions respectively for seismic and 6.44 seconds, 5.69 seconds, and 4.57 seconds in the North-South, West-East and torsional directions respectively for wind. Total building displacement was 8.974 inches in the North-South direction and 8.162 inches in the West-East direction due to seismic loading. Total building displacement was 16.119 inches in the North-South direction and 16.856 inches in the West-East direction for case 1 wind.

The chevron bracing with outriggers and belt trusses generated periods of vibrations of 5.26 seconds, 5.17 seconds, and 3.92 seconds in the North-South, West-East and torsional directions respectively for seismic and wind. Total building displacement was 16.7 inches in the North-South direction and 19.8 inches in the West-East direction for case 1 wind.

Overall, concrete shear walls in the core whether with or without outriggers seem to be a viable alternative solution that should be investigated further for the spring semester.

APPENDIX A: GRAVITY LOADING CALCULATIONS

Building Snow Load

Load Description	ASCE 7-05 Design Load		New York City Building Code
p_g=	25	psf	According to §[C26-902.6] 27-561 For valleys...provide for accumulations of snow... vary from forty-five psf at the low point to fifteen psf at the ridge.
C_e=	1		
C_t=	1		
I=	1		
C_s=	1		
p_s=0.7CsCeCtIp$_g$	**17.5**	psf	

Snow Load

Load Description/Factor	Design Load		Comments
h =	72.84	feet	EMR height
$\gamma = 0.13p_g + 14$ =	17.25	pcf	ASCE7-05, eq. 7-3
$h_b = p_s/\gamma$ =	1.01	feet	
$h_c = h - h_b$ =	71.83	feet	
h_c / h_b =	70.80	>0.2	drift load required
controlling l_u=	66.00	feet	
$h_d = 0.43(l_u)^{1/3}(p_g+10)^{1/4}-1.5$ =	2.73	feet	Figure 7-9 and equation
$h_d = 0.75h_d$ =	2.05	feet	
$w = 4h_d$ =	8.18	feet	
$8h_c$ =	574.60	feet	> w therefore ok
$p_d = h_d\gamma$ =	**35.28**	psf	

APPENDIX B: LATERAL LOADING

Wind

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
a	7.0	---	ASCE 7-05 6.5.6.6
z_g	1200	---	ASCE 7-05 6.5.6.6

Gust Factor {Tower}						
Variable	**Equation**	**Direction**		**Unit**	**Reference (ASCE 7)**	**Comments**
		E/W	**N/S**			
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 ≥ 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	
e	---	0.3333	0.3333	---	Table 6-2	
L_z	$l(z/33)^e$	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	
a bar	---	0.25	0.25	---	Table 6-2	
V_z	$b(z/33)^aV(88/60)$	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	
h (R_h)	$4.6n_1h/V_z$	4.9533	4.9533	---	6.5.8.2	
R_h	$1/h - (1/2h^2)(1-e^{-2h})$	0.18151	0.18151	---	6.5.8.2	
h (R_B)	$4.6n_1B/V_z$	1.2890	1.0431	---	6.5.8.2	
R_B	$1/h - (1/2h^2)(1-e^{-2h})$	0.4977	0.55619	---	6.5.8.2	

		2				
h (R_L)	$15.4n_1L/V_z$	3.4923	4.3153	---	6.5.8.2	
R_L	$1/h - (1/2h^2)(1-e^{-2h})$	0.2453 9	0.20489	---	6.5.8.2	
b	---	0.01	0.01	---	C6.5.8	
R	$((1/b)(R_nR_hR_B(.53+0.47R_L)))^{1/2}$	0.8527 86	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	

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 (ft²)	**Reduction**	**Cp**
4.748	27400	0.8	-1.040
Internal Pressure			
GC_{pi}	0.18		
	-0.18		
[F 6-5, ASCE 7-05]			

N/S Wind Direction (Tower) {h/L >1.0 & q < 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 (ft²)	**Reduction**	**Cp**
3.843	27400	0.8	-1.040
Internal Pressure			
GC_{pi}	0.18		
	-0.18		
[F 6-5, ASCE 7-05]			

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})	Net Pressure p (psf) - (GC_{pi})
Windward	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
	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	44.8	9.6	35.2	54.3
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] $K_z = 2.01(15/z_g)2/a$ {z_g < 15ft} -or- $K_z = 2.01(z/z_g)2/a$ {15 ft < z < z_g} [T 6-2, ASCE 7-05]

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})
Windward	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
	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	45.8	9.6	36.2	55.3
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] $K_z = 2.01(15/z_g)2/a$ $\{z_g < 15ft\}$ -or- $K_z = 2.01(z/z_g)2/a$ $\{15\ ft < z < z_g\}$ [T 6-2, ASCE 7-05]

Seismic

Soil Classification

<u>NYCBC:</u>	2-65 (medium hard rock) 4-65 (soft rock)	recommended by geotechnical report in areas of lower bearing capacity
<u>ASCE 7-05:</u>	seismic design category C	conservative estimate
	Occ. Cat. III *T 11.5-1* Importance factor= 1.25	

Spectral Response Acceleration

(using USGS Ground Motion Parameter Calculator)

latitude: 40.756192 F_a= 1.2
longitude: -73.990130 F_v= 1.7

site class C					
T=0.2s			*T=1.0s*		
S_{MS}	0.436	g	S_{M1}	0.119	g
S_{DS}	0.291	g	S_{D1}	0.08	g

ASCE 7-05:	S_{DS} -> SDC B	*T 11.6-1*	
	S_{D1} -> SDC B	*T 11.6-2*	therefore, use site class C

Period of Building

$T_a <= 0.8T_s$			
=	0.2199		
T_s	0.2749	S_{D1}/S_{DS}	
$T_a = C_t * h_n^x$ =	2.991		
C_t	0.02	*T 12.2.1.B*	
x	0.75	*T 11.5-1*	
h	793.8		

Seismic Base Shear

$V = C_s * W$	1759.8	k	*12.8-1*			
$C_s = min\{$	0.1119		$S_{DS}/(R/I)$			
	0.0103		$S_{D1}/(T_a{*}R/I)$	Cu	1.7	0.0027
		>=	0.01	use 0.01 for C_s		
R	3.25		*T 12.2.1.B*			
I	1.25		*T 11.5-1*			

Tower Weight by Floor

		w_i (psf)						
floor	**area (sf)**	**floor**	**façade**	**wall area (sf)**	**W_i (#)**	**h_x (ft)**	**h_i (ft)**	**$w_i{*}h_i^k$**
1	96625	113	25	18893	11390943	26.9896	27.0	8.298E+09
2	96625	113	25	10828	11189329	15.4688	42.5	2.017E+10
3	96625	113	25	10828	11189329	15.4688	57.9	3.755E+10
4	96625	113	25	10026	11169276	14.3229	72.3	5.83E+10
5	21550	113	25	9625	2675775	13.75	86.0	1.979E+10
6	21550	113	25	9625	2675775	13.75	99.8	2.662E+10
7	21550	113	25	9625	2675775	13.75	113.5	3.447E+10
8	21550	113	25	9625	2675775	13.75	127.3	4.333E+10
9	21550	113	25	9625	2675775	13.75	141.0	5.32E+10
10	21550	113	25	9625	2675775	13.75	154.8	6.408E+10
11	21550	113	25	9975	2684525	14.25	169.0	7.667E+10
12	21550	113	25	9275	2667025	13.25	182.3	8.859E+10
13	21550	113	25	9625	2675775	13.75	196.0	1.028E+11
14	21550	113	25	9625	2675775	13.75	209.8	1.177E+11
15	21550	113	25	9625	2675775	13.75	223.5	1.337E+11
16	21550	113	25	9625	2675775	13.75	237.3	1.506E+11
17	21550	113	25	9625	2675775	13.75	251.0	1.686E+11
18	21550	113	25	9625	2675775	13.75	264.8	1.876E+11
19	21550	113	25	9625	2675775	13.75	278.5	2.075E+11
20	21550	113	25	9625	2675775	13.75	292.3	2.285E+11
21	21550	113	25	9625	2675775	13.75	306.0	2.505E+11
22	21550	113	25	9625	2675775	13.75	319.8	2.736E+11
23	21550	113	25	9625	2675775	13.75	333.5	2.976E+11
24	21550	113	25	9625	2675775	13.75	347.3	3.227E+11
25	21550	113	25	9625	2675775	13.75	361.0	3.487E+11
26	21550	113	25	9625	2675775	13.75	374.8	3.758E+11

27	21550	113	25	9275	2667025	13.25	388.0	4.015E+11
28	21550	105	25	19250	2744000	27.5	415.5	4.737E+11
29	21550	113	25	9625	2675775	13.75	429.3	4.93E+11
30	21550	113	25	9625	2675775	13.75	443.0	5.251E+11
31	21550	113	25	9625	2675775	13.75	456.8	5.582E+11
32	21550	113	25	9625	2675775	13.75	470.5	5.923E+11
33	21550	113	25	9625	2675775	13.75	484.3	6.275E+11
34	21550	113	25	9625	2675775	13.75	498.0	6.636E+11
35	21550	113	25	9625	2675775	13.75	511.8	7.008E+11
36	21550	113	25	9625	2675775	13.75	525.5	7.389E+11
37	21550	113	25	9625	2675775	13.75	539.3	7.781E+11
38	21550	113	25	9625	2675775	13.75	553.0	8.183E+11
39	21550	113	25	9625	2675775	13.75	566.8	8.595E+11
40	21550	113	25	9625	2675775	13.75	580.5	9.017E+11
41	21550	113	25	9625	2675775	13.75	594.3	9.449E+11
42	21550	113	25	9625	2675775	13.75	608.0	9.891E+11
43	21550	113	25	9625	2675775	13.75	621.8	1.034E+12
44	21550	113	25	9625	2675775	13.75	635.5	1.081E+12
45	21550	113	25	9625	2675775	13.75	649.3	1.128E+12
46	21550	113	25	9625	2675775	13.75	663.0	1.176E+12
47	21550	113	25	9625	2675775	13.75	676.8	1.225E+12
48	21550	113	25	9625	2675775	13.75	690.5	1.276E+12
49	21550	113	25	9625	2675775	13.75	704.3	1.327E+12
50	21550	113	25	10267	2691816.7	14.6667	718.9	1.391E+12
51	21550	105	25	18958	2736708.3	27.0833	746.0	1.523E+12
52	21550	200	25	33491	5147266.5	47.8438	793.8	3.244E+12
				ΣW	**175984.02**	**k**	**$\Sigma w_i * h_i^k$**	**2.917E+13**

Lateral Seismic Force		
k=	2.0	(T > 2.5s)
C_{vx}	*F_x*	
0.0003	0.5006	
0.0007	1.217	
0.0013	2.265	
0.0020	3.518	
0.0007	1.194	
0.0009	1.606	
0.0012	2.080	
0.0015	2.614	
0.0018	3.210	
0.0022	3.866	
0.0026	4.626	
0.0030	5.345	
0.0035	6.202	
0.0040	7.102	
0.0046	8.064	
0.0052	9.087	
0.0058	10.171	
0.0064	11.316	
0.0071	12.522	
0.0078	13.788	

0.0086	15.116	
0.0094	16.505	
0.0102	17.956	
0.0111	19.467	
0.0120	21.039	
0.0129	22.672	
0.0138	24.224	
0.0162	28.581	
0.0169	29.746	
0.0180	31.682	
0.0191	33.679	
0.0203	35.738	
0.0215	37.857	
0.0228	40.037	
0.0240	42.279	
0.0253	44.581	
0.0267	46.945	
0.0281	49.369	
0.0295	51.855	
0.0309	54.402	
0.0324	57.009	
0.0339	59.678	
0.0355	62.408	
0.0370	65.199	
0.0387	68.050	
0.0403	70.963	
0.0420	73.937	
0.0437	76.972	
0.0455	80.068	
0.0477	83.938	
0.0522	91.889	
0.1112	195.706	
$V = \Sigma F_x$	**1759.8**	**k**

APPENDIX C: CONCRETE SHEAR WALLS IN THE CORE CALCULATIONS

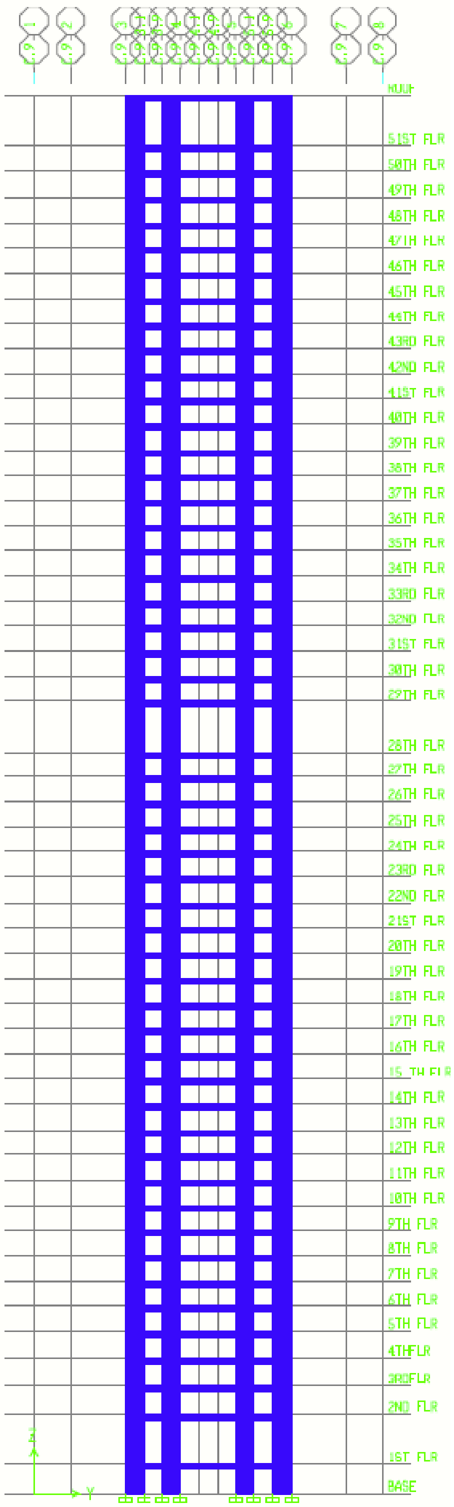


Figure 19: Wall Elevation B.9 & C.9

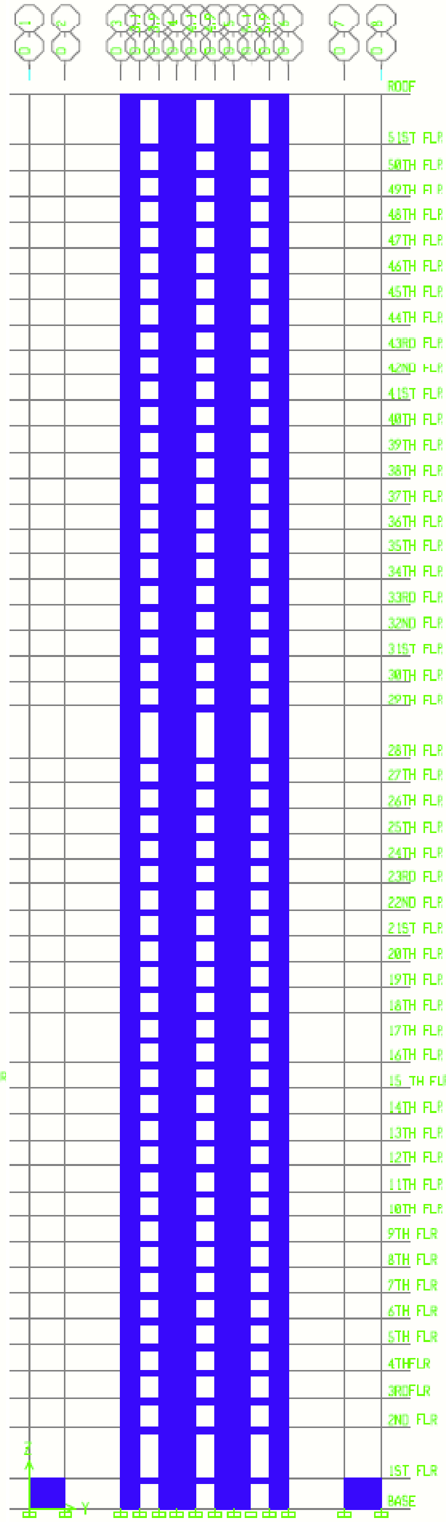


Figure 20: Wall Elevation B & D

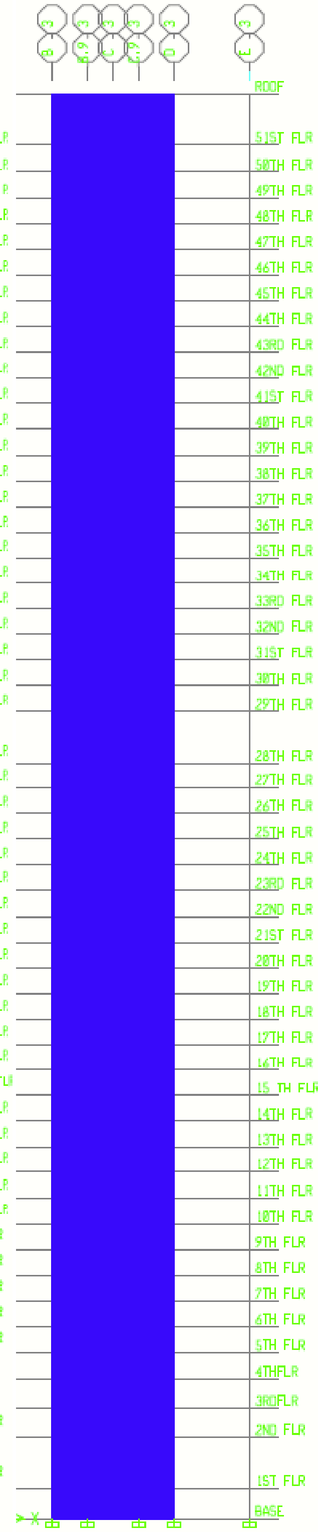


Figure 21: Wall Elevation 3, 4, 5, 6

Shear Walls					
Story	**Pier**	**Controlling Load**	**Direction**	**V2 (k)**	**M3 (kin)**
51ST FLR	SW1	1.2D+1.6W	W-E	453.22	74,853.99
26TH FLR	SW1	1.2D+1.6W	W-E	2,053.50	2,412,330.36
1ST FLR	SW1	1.2D+1.6W	W-E	2,022.78	8,376,540.69
51ST FLR	SW2	1.2D+1.6W	W-E	707.52	84,365.90
26TH FLR	SW2	1.2D+1.6W	W-E	2,849.66	2,411,340.89
1ST FLR	SW2	1.2D+1.6W	W-E	2,430.79	8,084,650.03
51ST FLR	SW3	1.2D+1.6W	N-S	365.01	33,635.23
26TH FLR	SW3	1.2D+1.6W	N-S	1,122.34	128,832.59
1ST FLR	SW3	1.2D+1.6W	N-S	-553.26	377,070.86
51ST FLR	SW4	1.2D+1.6W	N-S	70.76	5,581.74
26TH FLR	SW4	1.2D+1.6W	N-S	292.54	25,395.06
1ST FLR	SW4	1.2D+1.6W	N-S	-297.13	42,137.44

Height (ft)	f'c (psi)	L_w (ft)	t_w (in)	$\phi V_{n,max}$ (kips)	$V_u < \phi V_{n,max}$	a_{min} (ft)
27.08	8000	65	18	7,535	OK	13.54
13.75	10000	65	24	11,232	OK	6.88
26.99	12000	65	30	15,380	OK	13.49
27.08	8000	65	18	7,535	OK	13.54
13.75	10000	65	24	11,232	OK	6.88
26.99	12000	65	30	15,380	OK	13.49
27.08	8000	20	30	3,864	OK	10.00
13.75	10000	20	30	4,320	OK	6.88
26.99	12000	20	30	4,732	OK	10.00
27.08	8000	10	30	1,932	OK	5.00
13.75	10000	10	30	2,160	OK	5.00
26.99	12000	10	30	2,366	OK	5.00

V_c (kips)				
$2\sqrt{f'_c}td$ (kips)	$3.3\sqrt{f'_c}td+N_ud/l_w$ (kips)	Check $M_u/V_u-l_w/2$	$(0.6\sqrt{f'_c}+l_w(1.25\sqrt{f'_c})/(M_u/V_u-l_w/2))td$ (kip)	V_c (kips)
2009	3315	-228	Does not control	3315
2995	4942	-308	Does not control	4942
4101	6767	-228	Does not control	6767
2009	3315	-228	Does not control	3315
2995	4942	-308	Does not control	4942
4101	6767	-228	Does not control	6767
1030	1700	85	2127	1700
1152	1901	-38	Does not control	1901
1262	2082	84	2635	2082
515	850	205	343	343
576	950	45	1133	950
631	1041	204	421	421

Horizontal Reinforcing								
$.5\phi V_c$ (k)	$V_u > .5\phi V_c$	$V_{s,req}$ (kips)		Min. S (in)	Prov. S (in)	A_v	$A_{v,prov}$	ρ_t
1243	OK	-3229	MIN REINF.	18	12	0.54	0.6	0.0028
1853	REINF	-4855	MIN REINF.	18	12	0.72	0.79	0.0027
2538	OK	-6681	MIN REINF.	18	12	0.90	1.00	0.0028
1243	OK	-3229	MIN REINF.	18	12	0.54	0.6	0.0028
1853	REINF	-4855	MIN REINF.	18	12	0.72	0.79	0.0027
2538	OK	-6681	MIN REINF.	18	12	0.90	1.00	0.0028
638	OK	-1673	MIN REINF.	18	12	0.90	1.00	0.0028
713	REINF	-1874	MIN REINF.	18	12	0.90	1.00	0.0028
781	OK	-2056	MIN REINF.	18	12	0.90	1.00	0.0028
129	OK	-330	MIN REINF.	18	12	0.90	1.00	0.0028
356	OK	-937	MIN REINF.	18	12	0.90	1.00	0.0028
158	REINF	-408	MIN REINF.	18	12	0.90	1.00	0.0028

Vertical Reinforcing				
Min. Spa (in)	Prov. Spa (in)	ρ_l	A_v	$A_{v,prov}$
18	12	0.0028	0.60	0.6
18	12	0.0027	0.79	0.79
18	12	0.0028	1.00	1.00
18	12	0.0028	0.60	0.6
18	12	0.0027	0.79	0.79
18	12	0.0028	1.00	1.00
18	12	0.0027	0.96	1.00
18	12	0.0028	0.99	1.00
18	12	0.0027	0.96	1.00
18	12	0.0025	0.90	1.00
18	12	0.0027	0.96	1.00
18	12	0.0025	0.90	1.00

Flexure											
A_s (in²)	a (in)	jd (in)	A_s (in²)	$A_{s,prov}$ (in²)	d (in)	d_t (in)	a (in)	β	c (in)	ε_t (in/in)	ϕ
2.47	1.21	623.40	2.22	3.16	774	777	1.55	0.65	2.38	0.9751	0.9
79.55	23.40	612.30	72.96	73.66	774	777	21.66	0.65	33.33	0.0669	0.9
276.21	54.16	596.92	259.87	260.35	774	777	51.05	0.65	78.54	0.0267	0.9
2.78	1.36	623.32	2.51	3.16	774	777	1.55	0.65	2.38	0.9751	0.9
79.51	23.39	612.31	72.93	73.66	774	777	21.66	0.65	33.33	0.0669	0.9
266.59	52.27	597.86	250.42	260.35	774	777	51.05	0.65	78.54	0.0267	0.9
3.60	1.06	191.47	3.25	3.6	234	237	1.06	0.65	1.63	0.4335	0.9
13.81	3.25	190.38	12.53	13.2	234	237	3.11	0.65	4.78	0.1458	0.9
40.41	7.92	188.04	37.13	37.2	234	237	7.29	0.65	11.22	0.0604	0.9
1.20	0.35	95.82	1.08	1.2	114	117	0.35	0.65	0.54	0.6434	0.9
5.44	1.28	95.36	4.93	6	114	117	1.41	0.65	2.17	0.1586	0.9
9.03	1.77	95.11	8.20	8.4	114	117	1.65	0.65	2.53	0.1355	0.9

*Assumed clear cover of 3”

Flexure	
ϕM_n (ftk)	$M_u < \phi M_n$ (ftk)
131,943	OK
3,035,606	OK
10,522,742	OK
131,943	OK
3,035,606	OK
10,522,742	OK
45,387	OK
165,688	OK
462,733	OK
7,376	OK
36,707	OK
51,337	OK

Shear Wall Capacities and Reinforcing						
Story	Pier	Direction	ϕV_c (kips)	Hor. & Vert Rein.	ϕM_n (kin)	Flex. Rein.
51ST FLR	SW1	W-E	2486	#7 @12	131,943	(4) #8
26TH FLR	SW1	W-E	3707	#8 @12	3,035,606	(58) #10
1ST FLR	SW1	W-E	5075	#9 @12	10,522,742	(205) #10
51ST FLR	SW2	W-E	2486	#7 @12	131,943	(4) #8
26TH FLR	SW2	W-E	3707	#8 @12	3,035,606	(58) #10
1ST FLR	SW2	W-E	5075	#9 @12	10,522,742	(205) #10
51ST FLR	SW3	N-S	1275	#9 @12	45,387	(6) #7
26TH FLR	SW3	N-S	1426	#9 @12	165,688	(22) #7
1ST FLR	SW3	N-S	1562	#9 @12	462,733	(62) #7
51ST FLR	SW4	N-S	257	#9 @12	7,376	(2) #7
26TH FLR	SW4	N-S	713	#9 @12	36,707	(10) #7
1ST FLR	SW4	N-S	316	#9 @12	51,337	(14) #7

Coupling Beams								
Story	Spandrel	Controlling Load	Direction	Loc	V2 (k)	M3 (kin)	M3 (kft)	f'c (psi)
51ST FLR	CB1	1.2D+1.6W	N-S	Left	73.95	4,816.46	401	8000
51ST FLR	CB1	1.2D+1.6W	N-S	Right	90.15	(5,029.02)	-419	8000
26TH FLR	CB1	1.2D+1.6W	N-S	Left	304.44	18,645.78	1554	10000
26TH FLR	CB1	1.2D+1.6W	N-S	Right	320.64	(18,859.20)	-1572	10000
1ST FLR	CB1	1.2D+1.6W	N-S	Left	210.77	12,990.81	1083	12000
1ST FLR	CB1	1.2D+1.6W	N-S	Right	226.96	(13,273.00)	-1106	12000
51ST FLR	CB2	1.2D+1.6W	N-S	Left	-0.35	3,017.20	251	8000
51ST FLR	CB2	1.2D+1.6W	N-S	Right	48.25	(5,604.51)	-467	8000
26TH FLR	CB2	1.2D+1.6W	N-S	Left	31.28	8,737.53	728	10000
26TH FLR	CB2	1.2D+1.6W	N-S	Right	79.88	(11,272.15)	-939	10000
1ST FLR	CB2	1.2D+1.6W	N-S	Left	-2.39	2,626.50	219	12000
1ST FLR	CB2	1.2D+1.6W	N-S	Right	46.2	(5,259.27)	-438	12000
51ST FLR	CB3	1.2D+1.6W	N-S	Left	68.57	4,499.84	375	8000
51ST FLR	CB3	1.2D+1.6W	N-S	Right	84.77	(4,700.29)	-392	8000
26TH FLR	CB3	1.2D+1.6W	N-S	Left	279.29	17,155.96	1430	10000
26TH FLR	CB3	1.2D+1.6W	N-S	Right	295.49	(17,330.53)	-1444	10000
1ST FLR	CB3	1.2D+1.6W	N-S	Left	204.28	12,637.58	1053	12000

1ST FLR	CB3	1.2D+1.6W	N-S	Right	220.48	(12,847.65)	-1071	12000

h (in)	**b (in)**	**l_n (ft)**	**h_{min} (in)**	**Bar #**	**No. of bars**	**A_s (in^2)**	**d_t(in)**	**l_n/h**	
36	30	10	7.50	9	5	5	33.4375	3.33	R/C BEAM
36	30	10	7.50	9	5	5	33.4375	3.33	R/C BEAM
36	30	10	7.50	9	11	11	33.4375	3.33	R/C BEAM
36	30	10	7.50	9	11	11	33.4375	3.33	R/C BEAM
36	30	10	7.50	9	8	8	33.4375	3.33	R/C BEAM
36	30	10	7.50	9	8	8	33.4375	3.33	R/C BEAM
36	30	30	22.50	9	5	5	33.4375	10.00	R/C BEAM
36	30	30	22.50	9	5	5	33.4375	10.00	R/C BEAM
36	30	30	22.50	9	7	7	33.4375	10.00	R/C BEAM
36	30	30	22.50	9	7	7	33.4375	10.00	R/C BEAM
36	30	30	22.50	9	7	7	33.4375	10.00	R/C BEAM
36	30	30	22.50	9	7	7	33.4375	10.00	R/C BEAM
36	30	10	7.50	9	7	7	33.4375	3.33	R/C BEAM
36	30	10	7.50	9	7	7	33.4375	3.33	R/C BEAM
36	30	10	7.50	9	11	11	33.4375	3.33	R/C BEAM
36	30	10	7.50	9	11	11	33.4375	3.33	R/C BEAM
36	30	10	7.50	9	8	8	33.4375	3.33	R/C BEAM
36	30	10	7.50	9	8	8	33.4375	3.33	R/C BEAM

$A_{s,min}$ (in^2)	**$A_s > A_{s,min}$**	**β**	**ρ_{max}**	**$A_{s,max}$ (in^2)**	**$A_s > A_{s,max}$**	**b_{min}**	**$b > b_{min}$**
4.49	**OK**	0.65	0.0316	31.67	**OK**	20.29	**OK**
4.49	**OK**	0.65	0.0316	31.67	**OK**	20.29	**OK**
5.02	**OK**	0.65	0.0395	39.59	**OK**	27.04	**OK**
5.02	**OK**	0.65	0.0395	39.59	**OK**	27.04	**OK**
5.49	**OK**	0.65	0.0474	47.51	**OK**	23.67	**OK**
5.49	**OK**	0.65	0.0474	47.51	**OK**	23.67	**OK**
4.49	**OK**	0.65	0.0316	31.67	**OK**	20.29	**OK**
4.49	**OK**	0.65	0.0316	31.67	**OK**	20.29	**OK**
5.02	**OK**	0.65	0.0395	39.59	**OK**	22.54	**OK**
5.02	**OK**	0.65	0.0395	39.59	**OK**	22.54	**OK**
5.49	**OK**	0.65	0.0474	47.51	**OK**	22.54	**OK**
5.49	**OK**	0.65	0.0474	47.51	**OK**	22.54	**OK**
4.49	**OK**	0.65	0.0316	31.67	**OK**	22.54	**OK**
4.49	**OK**	0.65	0.0316	31.67	**OK**	22.54	**OK**
5.02	**OK**	0.65	0.0395	39.59	**OK**	27.04	**OK**
5.02	**OK**	0.65	0.0395	39.59	**OK**	27.04	**OK**
5.49	**OK**	0.65	0.0474	47.51	**OK**	23.67	**OK**
5.49	**OK**	0.65	0.0474	47.51	**OK**	23.67	**OK**

a (in)	**c (in)**	**εt (in/in)**	**φ**	**ϕM_n (kin)**	**$M_u < \phi M_n$ (ftk)**
1.47	2.26	0.0413381	0.9	8,830	**OK**
1.47	2.26	0.0413381	0.9	8,830	**OK**
2.59	3.98	0.0221921	0.9	19,093	**OK**
2.59	3.98	0.0221921	0.9	19,093	**OK**
1.57	2.41	0.038567	0.9	14,106	**OK**
1.57	2.41	0.038567	0.9	14,106	**OK**
1.47	2.26	0.0413381	0.9	8,830	**OK**

1.47	2.26	0.0413381	0.9	8,830	**OK**
1.65	2.53	0.0365876	0.9	12,328	**OK**
1.65	2.53	0.0365876	0.9	12,328	**OK**
1.37	2.11	0.0445051	0.9	12,380	**OK**
1.37	2.11	0.0445051	0.9	12,380	**OK**
2.06	3.17	0.0286701	0.9	12,250	**OK**
2.06	3.17	0.0286701	0.9	12,250	**OK**
2.59	3.98	0.0221921	0.9	19,093	**OK**
2.59	3.98	0.0221921	0.9	19,093	**OK**
1.57	2.41	0.038567	0.9	14,106	**OK**
1.57	2.41	0.038567	0.9	14,106	**OK**

V_c (k)	$.5\phi V_c$ (k)	V_u @ d	V_s	$8\sqrt{f'_c}bd$	$V_s < 8\sqrt{f'_c}bd$	$4\sqrt{f'_c}bd$	$V_s < 8\sqrt{f'_c}bd$
179.4	67.29	93.59	-54.66	717.78	**OK**	358.89	**OK**
179.4	67.29	100.76	-45.10	717.78	**OK**	358.89	**OK**
200.6	75.23	308.96	211.32	802.50	**OK**	401.25	**OK**
200.6	75.23	316.13	220.88	802.50	**OK**	401.25	**OK**
219.8	82.42	215.28	67.27	879.09	**OK**	439.55	**OK**
219.8	82.42	222.45	76.83	879.09	**OK**	439.55	**OK**
179.4	67.29	4.16	-173.90	717.78	**OK**	358.89	**OK**
179.4	67.29	43.73	-121.14	717.78	**OK**	358.89	**OK**
200.6	75.23	35.80	-152.89	802.50	**OK**	401.25	**OK**
200.6	75.23	75.37	-100.13	802.50	**OK**	401.25	**OK**
219.8	82.42	2.12	-216.95	879.09	**OK**	439.55	**OK**
219.8	82.42	41.69	-164.19	879.09	**OK**	439.55	**OK**
179.4	67.29	73.08	-82.00	717.78	**OK**	358.89	**OK**
179.4	67.29	80.25	-72.44	717.78	**OK**	358.89	**OK**
200.6	75.23	283.80	177.78	802.50	**OK**	401.25	**OK**
200.6	75.23	290.97	187.34	802.50	**OK**	401.25	**OK**
219.8	82.42	208.79	58.61	879.09	**OK**	439.55	**OK**
219.8	82.42	215.96	68.17	879.09	**OK**	439.55	**OK**

S_{max}	Use S=	$A_{v,min}$ (in^2)	A_v (in^2)	$A_v > A_{v,min}$
16.71875	10	0.335	0.40	**OK**
16.71875	10	0.335	0.40	**OK**
16.71875	10	0.375	0.40	**OK**
16.71875	10	0.375	0.40	**OK**
16.71875	10	0.411	0.62	**OK**
16.71875	10	0.411	0.62	**OK**
16.71875	10	0.335	0.40	**OK**
16.71875	10	0.335	0.40	**OK**
16.71875	10	0.375	0.40	**OK**
16.71875	10	0.375	0.40	**OK**
16.71875	10	0.411	0.62	**OK**
16.71875	10	0.411	0.62	**OK**
16.71875	10	0.335	0.40	**OK**
16.71875	10	0.335	0.40	**OK**
16.71875	10	0.375	0.40	**OK**
16.71875	10	0.375	0.40	**OK**
16.71875	10	0.411	0.62	**OK**
16.71875	10	0.411	0.62	**OK**

Coupling Beams Capacities and Reinforcing					
Story	**Spandrel**	**Direction**	**Shear Rein.**	**ϕM_n (kin)**	**Flex. Rein.**
51ST FLR	CB1	N-S	#4 @10	8,830	(5) #9
26TH FLR	CB1	N-S	#4 @10	19,093	(11) #9
1ST FLR	CB1	N-S	#5 @10	14,106	(8) #9
51ST FLR	CB2	N-S	#4 @10	8,830	(5) #9
26TH FLR	CB2	N-S	#4 @10	12,328	(7) #9
1ST FLR	CB2	N-S	#5 @10	12,380	(7) #9
51ST FLR	CB3	N-S	#4 @10	12,250	(7) #9
26TH FLR	CB3	N-S	#4 @10	19,093	(11) #9
1ST FLR	CB3	N-S	#5 @10	14,106	(8) #9