Manhattan West (Southwest Residential Tower) New York, NY

Peer review Report

Rosenwasser/Grossman Consulting Engineers, P.C.

October, 2014

Prepared for Brookfield

Prepared by

Ben Pimentel, PE

Sunghwa Han, PE, SE, LEED AP

James, Myungsu Shin, PE, Phd

Ben Pimentel hereby certifies that I have performed the peer review in accordance with the New York City Building Code and requirements set forth therein.

Name: Ben Pimentel
License No.: 086645

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Table of Contents

1.1 Executive Summary

The proposed building is located at the North-East corner of the intersection of the 31st street and Dyer Avenue in New York. There is the open-cut rail corridor (track) operated by Amtrak and the Long Island Rail Road (LIRR) on the north side of the site. This cut is approximately 45 to 60 ft below the street sidewalk. This building is a part of the Ninth Avenue Development (Manhattan West Master Plan) which includes the sites encompassed by Ninth Avenue, Dyer Avenue, 31st Street, and 33rd Street, New York.

The total floor area is approximately 850,000 sq. ft and the building height to the roof of the bulkhead is 673 ft. The building is designed to be a 64 story tall tower with one cellar below grade and it will be used as a residential building equipped with amenities and retail space. The foot print of the building is L-shape at the typical floors and additional area is added below the 4th floor. At the 2nd floor, the footprint of the building glows to 227 ft x 105 ft. Below this floor, the plan dimension is reduced to accommodate the existing retaining walls confining the open track below grade.

The western property line borders with Dyer Avenue which is overbuild during the phase I of the Ninth Avenue Development (Manhattan West Master Plan). The existing open-cut rail corridor (track) is covered with precast deck and used as parking space. For this, the North face of the proposed building will overhang the existing precast concrete deck over the rail. And it will cantilever off of caisson foundations that are to be installed through the existing capping beam supporting the precast deck. The loads of the exterior columns located over the open-cut track are transferred to the interior columns at the 4th floor by sloping columns and a full height concrete transfer beam.

Rosenwasser/Grossman Consulting Engineers P.C. was retained by the owner to provide peer review based on the New York City building Code 2008 Section BC 1627. Special design requirements for the extreme loading conditions above and beyond the code-prescribed loads, which may be requested by the owner or a third party will not be included in our general code-prescribed peer review discussed herein. It shall be noted that Rosenwasser/Grossman Consulting Engineers P.C reports its own opinion and functions solely as a peer reviewer regarding the design by the engineer of record. The structural engineer of record shall retain sole responsibility for the structural design of the entire building.

For our review, two structural analysis models were created based on the referenced drawings and reports provided by the engineer of record listed below. Overall behavior of the structure was reviewed and compared with the current design. The representative structural members were checked using the results from our independent analysis. Code compliance of the design according to the New York City Building Code 2008 section 1627.6.1 is summarized in the checklist (See appendix A).

Below is the list of information Rosenwasser/Grossman Consulting Engineers P.C. received from the engineer of record.

< References >

1. Architectural drawings dated May 12, 2014
2. Structural drawings dated May 9, 2014 and August 1, 2014 (90% CD set)
3. Mechanical drawings dated May 12, 2014
4. Geotechnical report prepared by Museur Rutledge Consulting Engineers dated July 1, 2014
5. Wind Tunnel Testing Result dated January 15, 2014 (The full wind study report was not available. Only the structural wind loads along with the estimated accelerations and torsional velocities were conveyed by the engineer of record)

1.2. Design Criteria

1.2.1. Design Code and References

- New York City Building Code 2008
- ACI 318-02 Building Code Requirements for Structural Concrete

1.2.2 Design loads

1.2.2.1 Gravity loads

Typical floors for Residential units				
	Superimposed dead load	:	20	psf
	Live load	:	40	psf
Typical floors for Terrace				
	Superimposed dead load	:	40	psf
	Live load	:	100	psf

Main roof				
	Superimposed dead load	:	40	psf
	Live load	:	100	psf
Typical mechanical floors (Equip. weight is separately considered)				
	Superimposed dead load	:	20	psf
	Live load	:	150	psf
Residential Lobby				
	Superimposed dead load	:	40	psf
	Live load	:	100	psf
Residential Amenities				
	Superimposed dead load	:	40	psf
	Live load (Stack parking)	:	100	psf
Cellar Storage				
	Superimposed dead load	:	20	psf
	Live load	:	100	psf

1.2.2.2 Wind Loads

Wind tunnel testing is performed to evaluate the wind-induced structural responses by the wind tunnel testing lab (Rowan Williams Davis & Irwin Inc.) and compared with the wind loads calculated using an analytical method. Input data for computation of the wind loads for listed herein are provided by the engineer of record. It was confirmed that the wind tunnel testing results were used for the final design of the building.

- Basic Wind Speed for New York City: 98mph measured at 33 ft above ground as a 3 second gust (Based on local wind climate with annual probability with 0.02, 50 year mean recurrence interval)
- Importance Factor: I=1.0 (Structural Occupancy Category II)
- Exposure: B
- Wind Loads (Provided by the wind tunnel testing lab)
  - Total Wind Load in N-S Direction: 3,260 kips

- Total Wind Load in E-W Direction: 2,650 kips

1.2.2.3 Seismic Loads

- Site: New York City (S_S= 0.365 g, : S_1 = 0.071 g)
- Seismic Use Group I (Occupancy category II):
- Site Class: B (F_a = 1.0 & F_v= 1.0)
- Importance Factor: I=1.0 (Seismic use group I)
- Load Resisting System: "Bearing Wall System with Ordinary Reinforced Concrete Shear Wall' (Different from the load resisting system chosen by the engineer of record)
- Response Modification Factor: R=4.0
- System Over-strength Factor: Ω_o=2.5
- Deflection Amplification Factor: C_d=4.0
- Seismic Design Category: B
- Seismic Base Shear: 192,500 kips x 0.011 = 2,061 kips
  - Seismic Response Coefficient: Cs= 0.011 (As per ASCE 7-02, Eq. 9.5.5.2.1-3)
  - Building Weight = Approximately 192,500 kips (including weight of mechanical equipment)

It is our understanding that the load resisting system for the proposed building shall be "Bearing Wall System with Ordinary Reinforced Concrete Shear Wall", since the shear walls resist the entire seismic loads and also support the majority of the gravity loads. This designation is different from the load resisting system designation which the engineer of record chose for the design. Because of this discrepancy, there will be an increase in the calculated seismic base shear. However since the wind loads are significantly greater than the seismic loads, this discrepancy will have little practical effect in the final design.

1.3 Structural System

1.3.1 Gravity Load Resisting System

A typical 8 inch thick flat plate floor supported by cast-in-place concrete columns and shear walls was utilized to resist the gravity loads. Columns located at the north side of the site where the open-cut track is located along with the existing caisson caps need to be transferred at the 4th floor offset with sloping columns.

1.3.2 Lateral Load Resisting System

A. Wind loads: A combination of frames (consisting of 8 inch thick flat plate and cast-in-place concrete columns) and the shear walls are utilized to resist the wind loads. The main core walls along with the shear walls connected with tie beams located on the east side of the building provide the required stiffness to resist the wind loads and improve serviceability.

B. Seismic loads: It is confirmed by the engineer of record that only the shear walls are utilized to resist the seismic loads. Frames are not considered to be participating in resisting the seismic loads. Therefore, a separate analysis model was built to investigate the structural dynamic behaviors (frequencies) and the results from our separate analysis were taken into account through the entire peer-review process. The analysis results for two different load resisting systems are compared in section 1.5 of this peer review report.

1.4 Foundation system

Foundation system for the proposed building is comprised of three components;

1) Mat foundations supporting shear walls: The subgrade modulus for bedrock is estimated to be 40,000 kips/ft^3 by the geotechnical engineers.

2) Caissons supporting exterior columns adjacent to the open rail corridor (track) on the north side of the building

3) Spread footings supporting tower columns and plaza columns: The allowable bearing pressure is recommended to be 40 tsf for bedrock by the geotechnical engineers.

1.5 Analysis

1.5.1 Building periods: Based on our independently-built analysis model

A. For the wind loads: with a combination of frames and shear walls

- 1st Mode: 7.5 sec (Primary East-West direction)
- 2nd Mode: 6.6 sec (Primary North-South direction)
- 3rd Mode: 3.8 sec (Primary Torsion)

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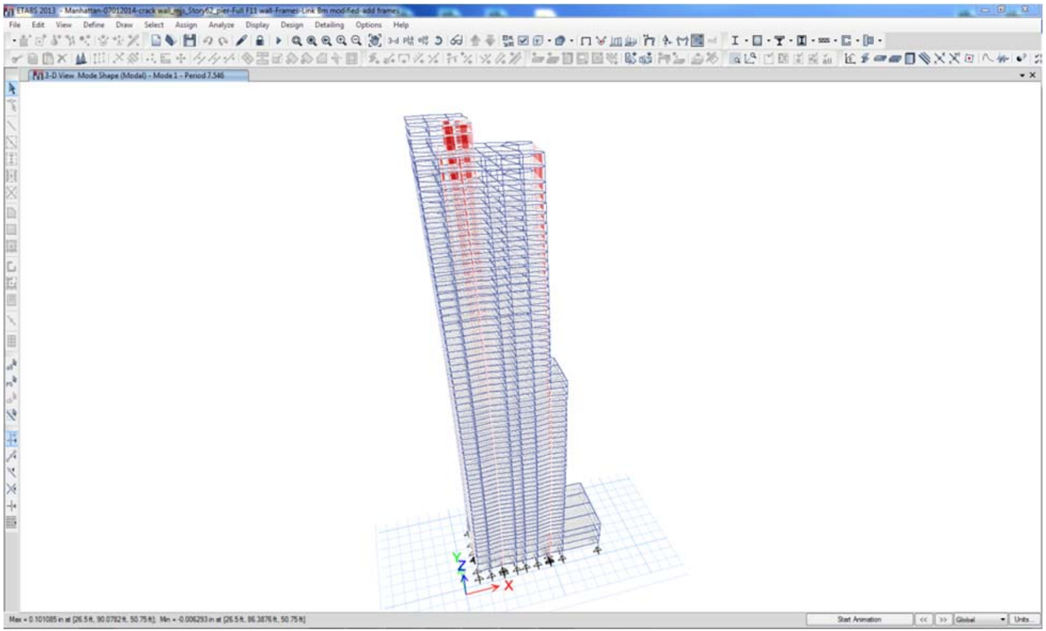

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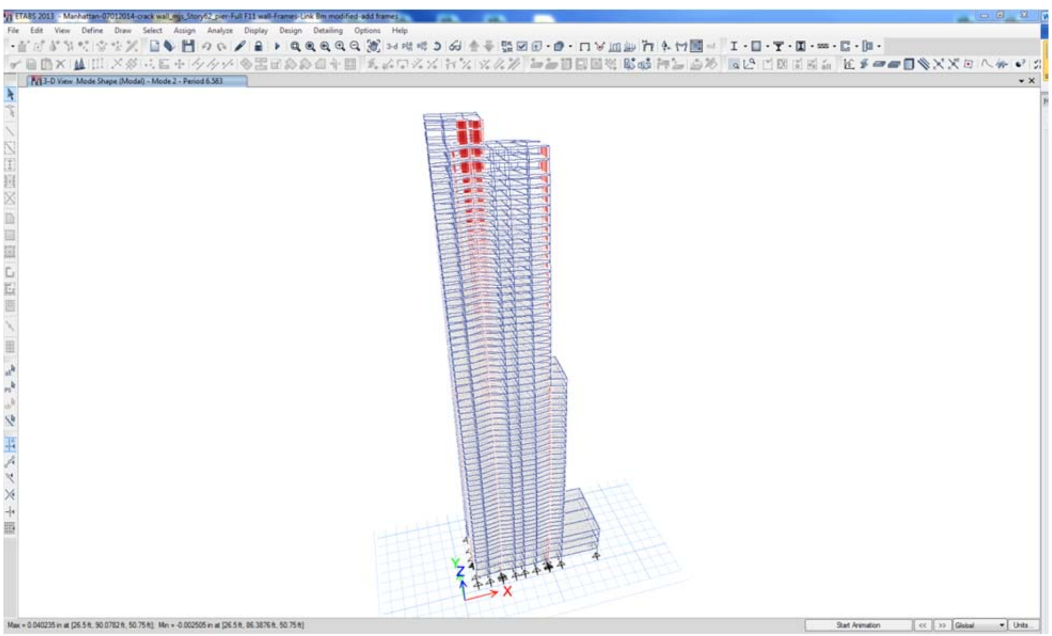

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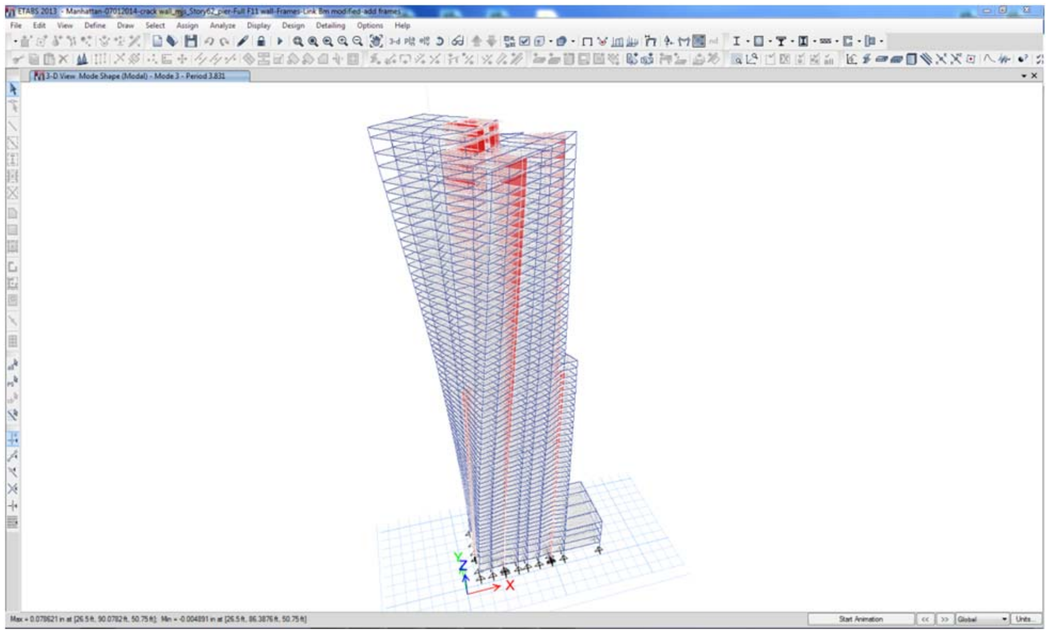

- Comparison of computed building periods

Mode		Reviewer's analysis result	**Design by engineer of record** (provided by engineer of record for calculation of the design wind loads)
1st	East-West direction	7.5 sec	7.32 sec
2nd	North-south direction	6.6 sec	6.45 sec
3rd	Torsion	3.8 sec	4.06 sec

B. For seismic design: With shear walls only (Frames are not participating in resisting the seismic loads)

- 1st Mode: 8.5 sec (Primary East-West direction)
- 2nd Mode: 7.1 sec (Primary North-South direction)

- 3rd Mode: 3.9 sec (Primary Torsion)

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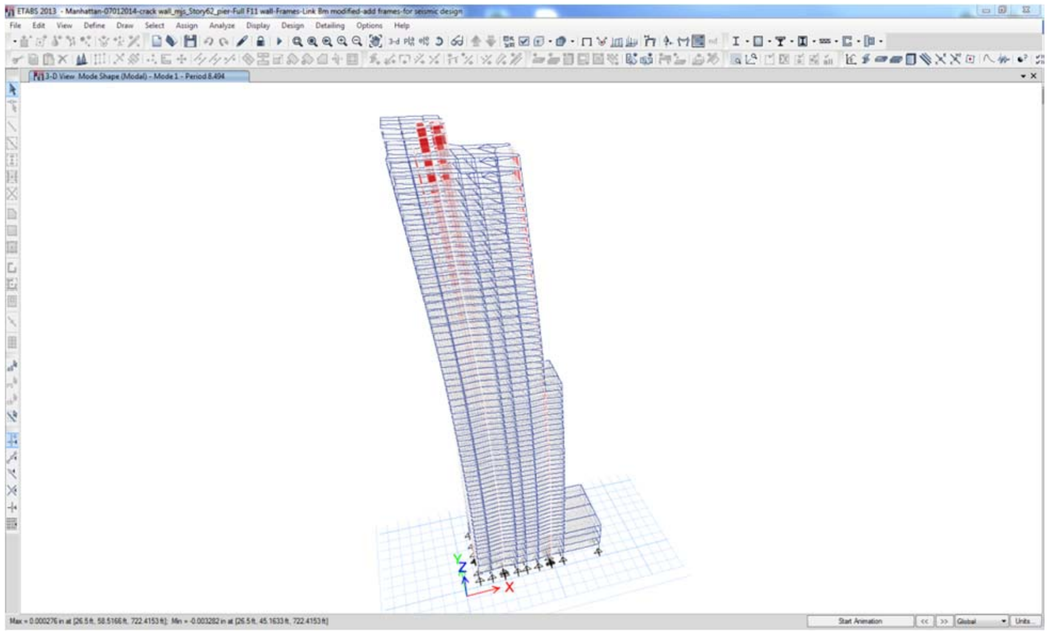

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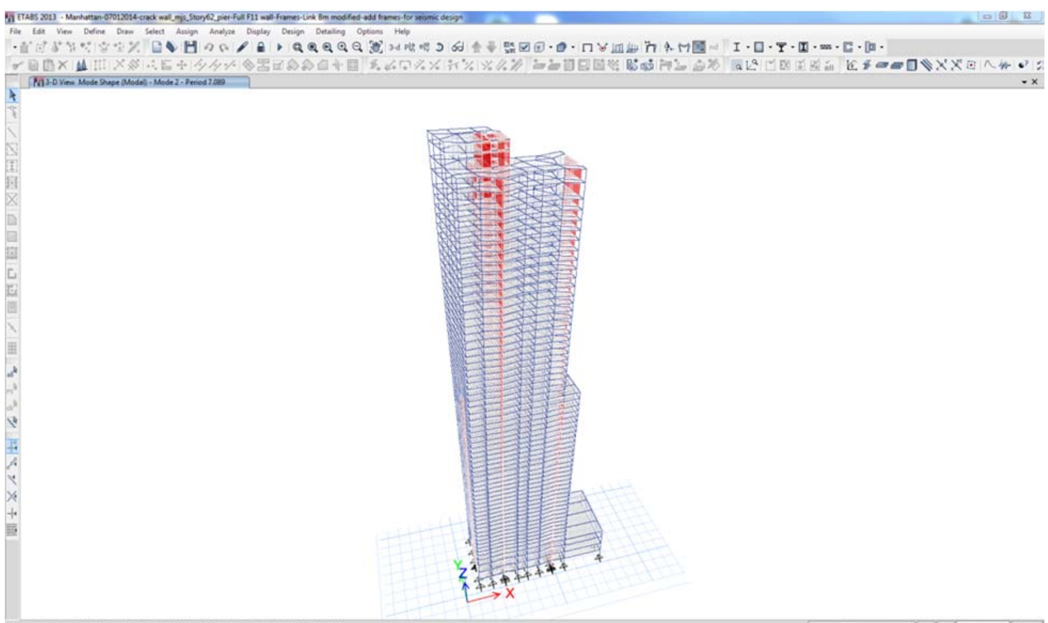

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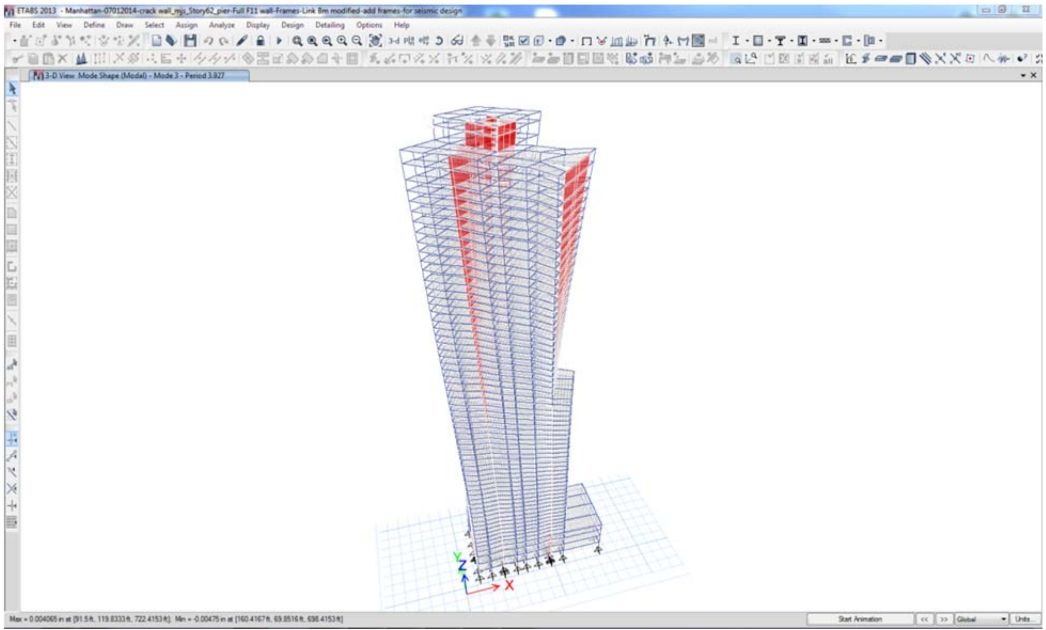
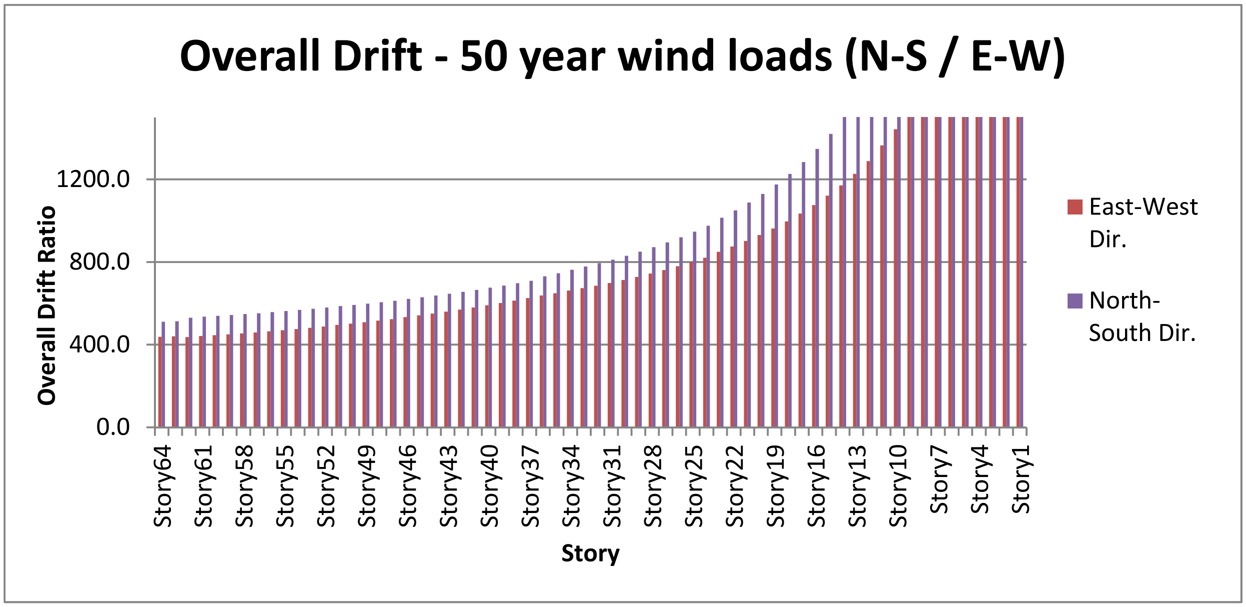


- Comparison of computed building periods

Mode		Reviewer's analysis result	**Design by engineer of record** (Seismic design criteria indicated in general notes Dwg. S-001)
1st	East-West direction	7.8 sec	
2nd	North-south direction	7.1 sec	
3rd	Torsion	3.9 sec	6.56 sec

1.5.2 Maximum Drift

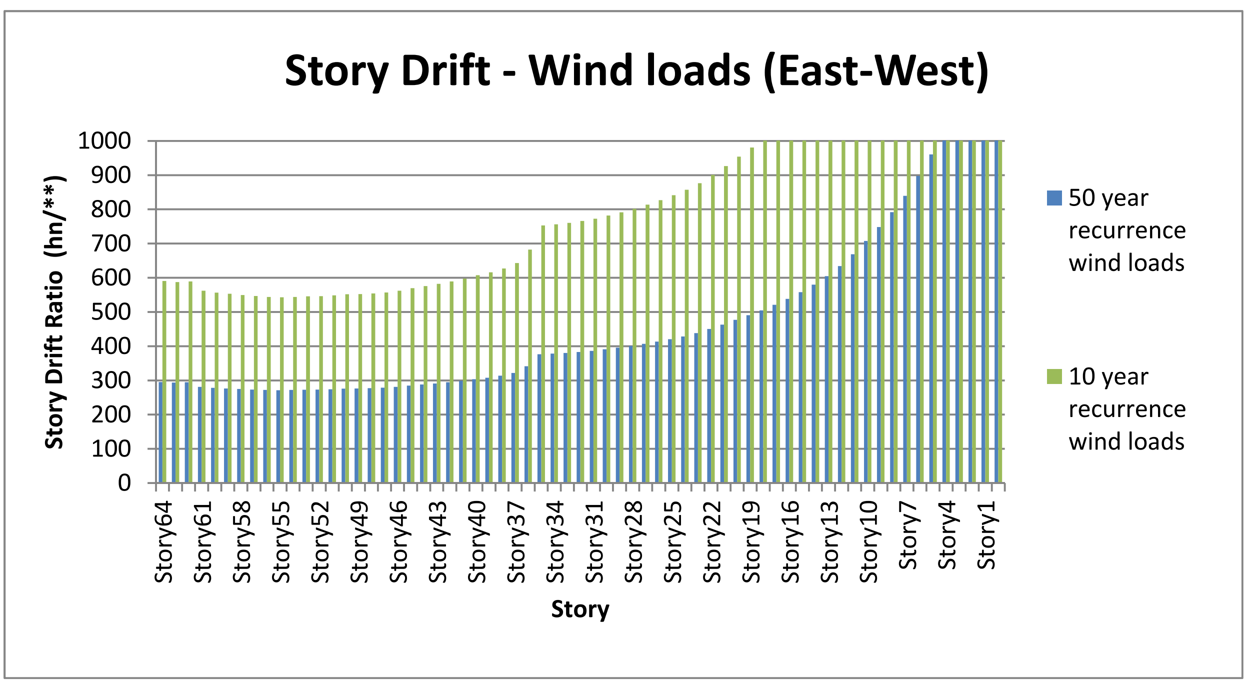
A. Wind loads: Based on 50 year recurrence wind loads



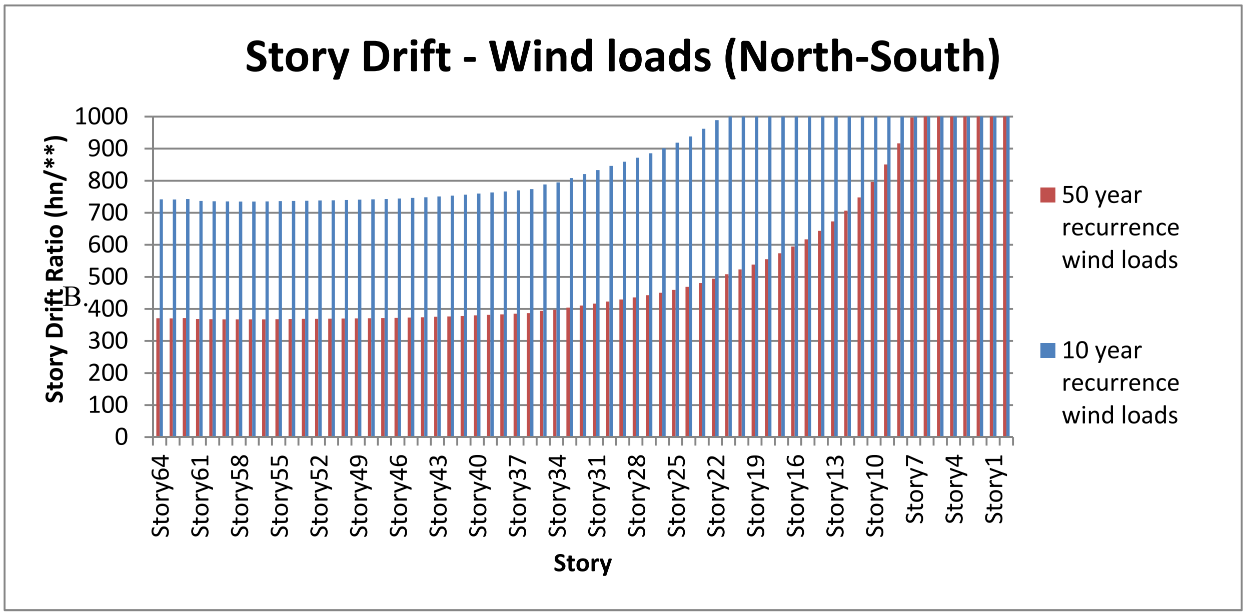
- 19.7 inch (H/438) : X-direction (East-West direction)
- 16.9 inch (H/511) : Y-direction (North-South direction)

1.5.3 Maximum Story Drift

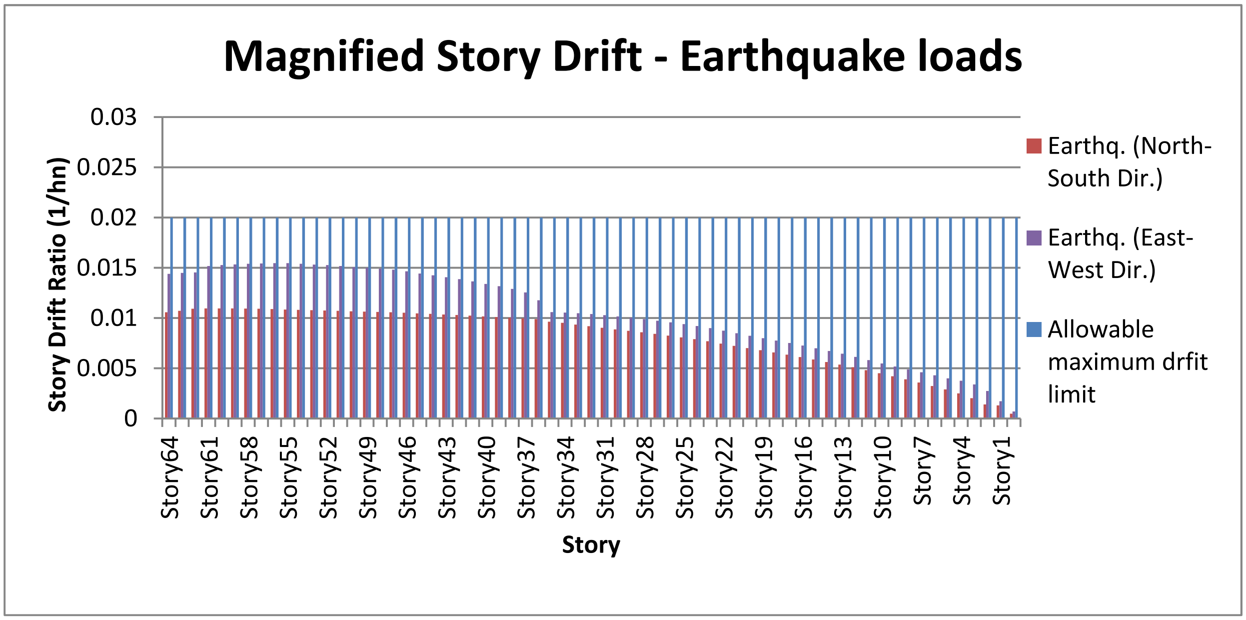
A. Wind loads: Based on 50 year /10 year recurrence wind loads



- X-direction (East-West)
    - hn/272 (50 year recurrence wind loads)
    - hn/544 (10 year recurrence wind loads) at 54F and 55F
- Y-direction (North-South)
    - hn/367 (50 year recurrence wind loads)
    - hn/735 (10 year recurrence wind loads) at 57F and 58F



B. Earthquake loads: less than 0.02 hn (Allowable maximum story drift for seismic use group)



1.5.4 Foundation

1.5.4.1 Mat foundation for shear walls

For the main core shear walls, 84 inch and 96 inch thick mat foundation supports 64 floors and the cellar floor. The shear wall located on the East side of the building is supported by 66 inch and 102 inch thick mat foundation. The subgrade modulus for vertical bearing was initially estimated to be 40,000 kips/ft^3 for bed-rock. The subgrade modulus for the final design has been decreased to 20,000 kips/ft^3. Uplift due to presence of the lateral wind loads and seismic loads is resisted by 2 ¼ inch diameter rock anchor with a capacity of 250 kips service load in tension. Hydrostatic pressure is not considered for the design of the mat, foundation slabs (slab-on-grade) or foundation walls. A permanent drainage system behind retaining walls and below slabs-on-grade will preclude buildup of water pressure.

1.5.4.2 Caissons

5 ft diameter caissons are supporting the columns located at the Northern edge of the building.

1.5.4.3 Spread footing

Spread footings resting on bedrock with 40 tsf of the bearing capacity support the tower columns and the lower level plaza columns.

1.6 Design of Structural Members

1.6.1 Link Beams

Design of link beams was reviewed. Mainly 24 inch deep cast-in-place concrete beams are utilized and wide flange steel beams are placed at the specific locations where the demand is high. All steel beams are encased with concrete for fire-resistance. It was confirmed by the engineer of record that the concrete portion of the embedded steel beams is not engaged in the strength.

See Appendix B "Sample Calculation Sheet" for detail.

1.6.2 Flat Slabs

Slab reinforcing at two typical floors (5^{th}-34^{th} floor / 37^{th}-51^{st} floor) is reviewed using the equivalent frame method and FEM. See appendix A and B for the details.

A. Two-way shear

- Two way shear under the load combination of 1.2DL+1.6LL (governing load case) is checked.
- Lateral displacement capacity of slab-column connection not to contribute to lateral resistance: Punching shear at the columns induced from deformation of the building is checked at the upper typical floors (50^{th} floor). According to our analysis, the maximum story drift due to the seismic loads is estimated to be approximately 0.015 in East-West direction at the 50^{th} floor. Therefore the maximum direct punching shear ratio (Vu/øVc) without shear reinforcing under the load combination of 1.2DL+1.0LL is allowed to be 0.4. See the direct punching shear check output for details.

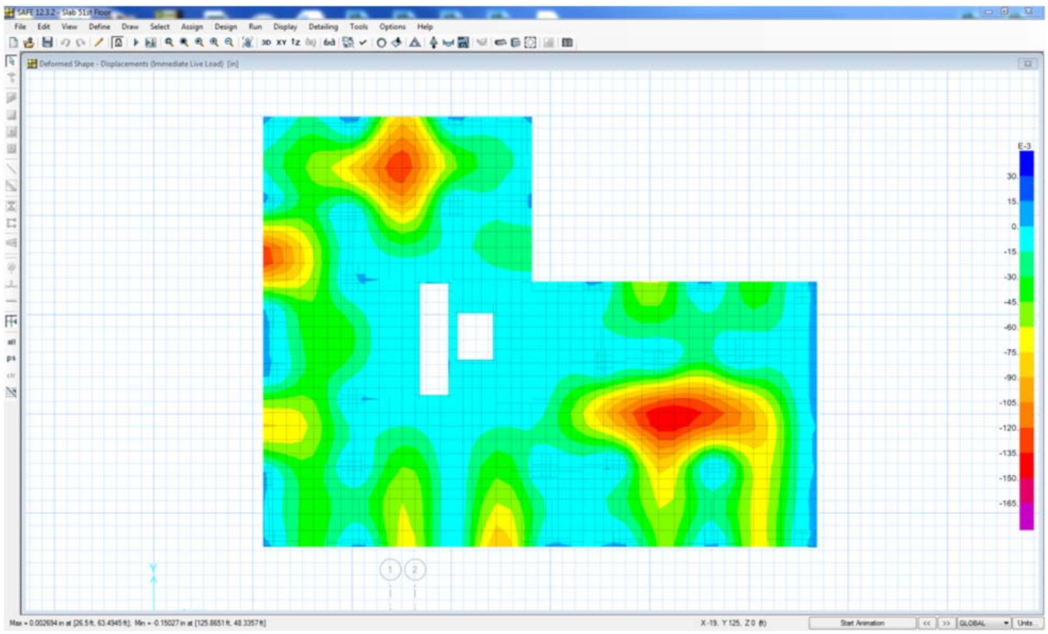
B. Structural Integrity Requirements

- Peripheral ties: Continuous reinforcing within spandrel section of flat plate slabs.
- Horizontal ties: Slab reinforcing within column cage is checked for the following load combinations.
  - 3.0 x (1.0 DL-self weight)
  - 1.5 x (larger of 1.2DL+1.6LL and 1.4DL)
- Key element analysis: Flat plate slabs are designed as bending elements.

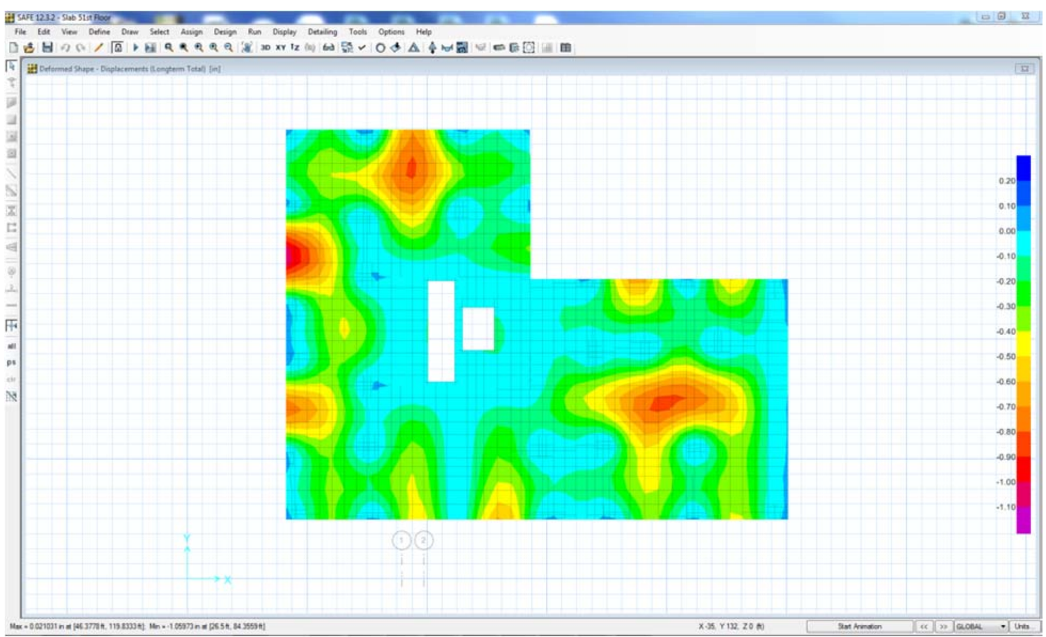
1.6.3 Serviceability

- Slab deflection: Slab deflection due to dead loads and live loads at 51st floor (Typical floor) is checked.

A. Immediate deflection due to live load: Maximum 0.15 inch



B. Total long-term deflection due to dead load and live load: Maximum 1.2 inch



1.6.4 Transfer Beams and Strap Beams

The major transfer beams at the 2nd floor and the strap beams bracing the sloping columns were checked for strength requirements and structural integrity requirements (key element analysis). According to our study, the strap beam bracing column 3, 4 and 5 shall be tied back to the shear walls. After the forces are transmitted to the sloping column connecting column 5 and 4 (Refer to Dwg. S-204 and section 5on Dwg. S-504), the horizontal component of the forces (tension forces at the strap beam) is transmitted to the adjacent vertical members through the strap beam. The shear strength of the columns is not sufficient to support this horizontal component. Therefore, the strap beam should be extended to the shear walls, which generally have sufficient shear strength capacity. In addition, as per NYCBC regarding the specific local resistance method specified in NYCBC 2008, connection of each tension element (for example, strap beams bracing sloping columns) shall be designed to develop the smaller of ultimate tension capacity of the members or three times the forces in the members. See Appendix A "Code compliance check list" and Appendix B "Sample Calculation Sheet" for detail.

1.6.5 Columns and Shear walls

Six representative columns and shear walls at the lower levels were checked for strength requirements. See Appendix B "Sample Calculation Sheet" for detail.

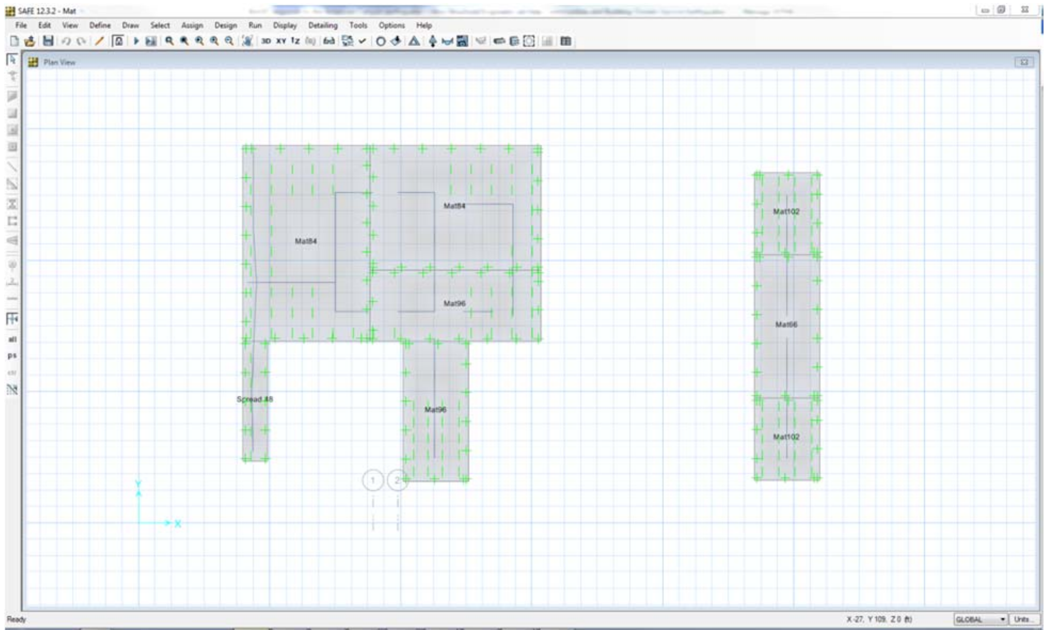
1.6.6 Mat foundation

- One-way shear: One-way shear capacity around shear walls is checked. See Appendix B "Sample Calculation Sheet" for detail.

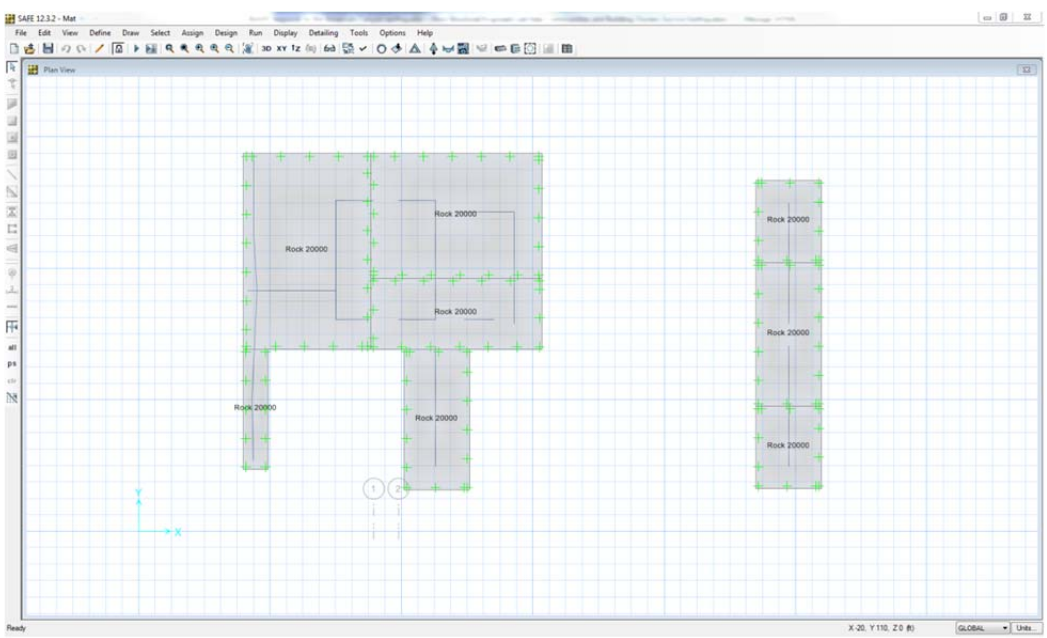
- Flexural reinforcing: Flexural reinforcing specified on the structural drawings was checked for adequacy. See Appendix B "Sample Calculation Sheet" for detail.

- Rock anchors: According to our estimate based on the rock anchor design specified on the structural drawings, rock anchor was modelled as a point support with 250 k/in spring coefficient. The reactions at these supports (where rock anchors are located) are checked.

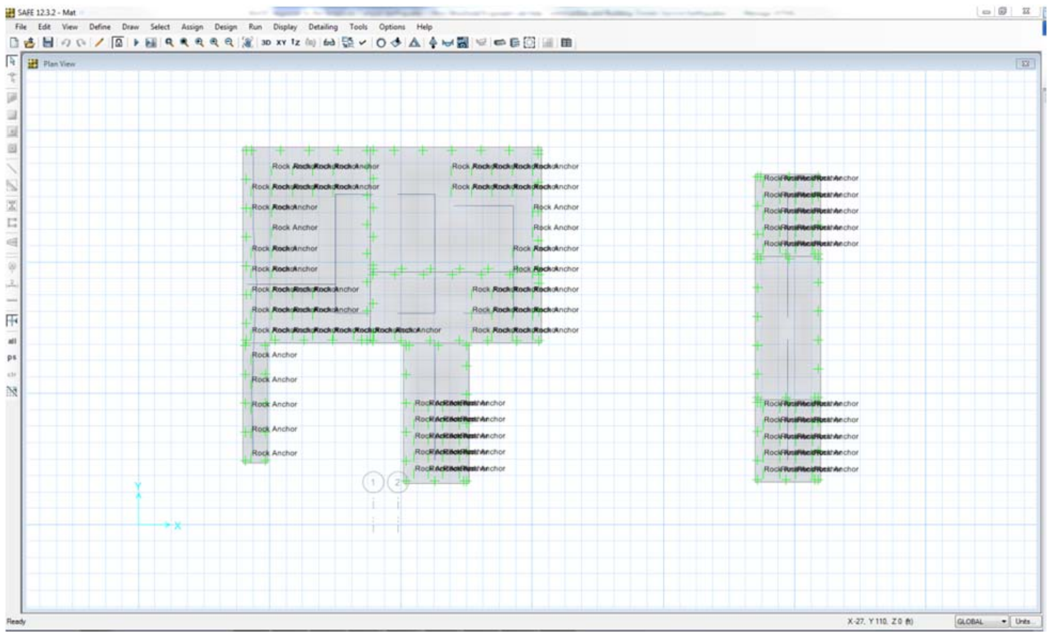
< Input Data – Thickness of mat >



< Input Data - Subgrade modulus (20,000 kips/ft^3) for mat >



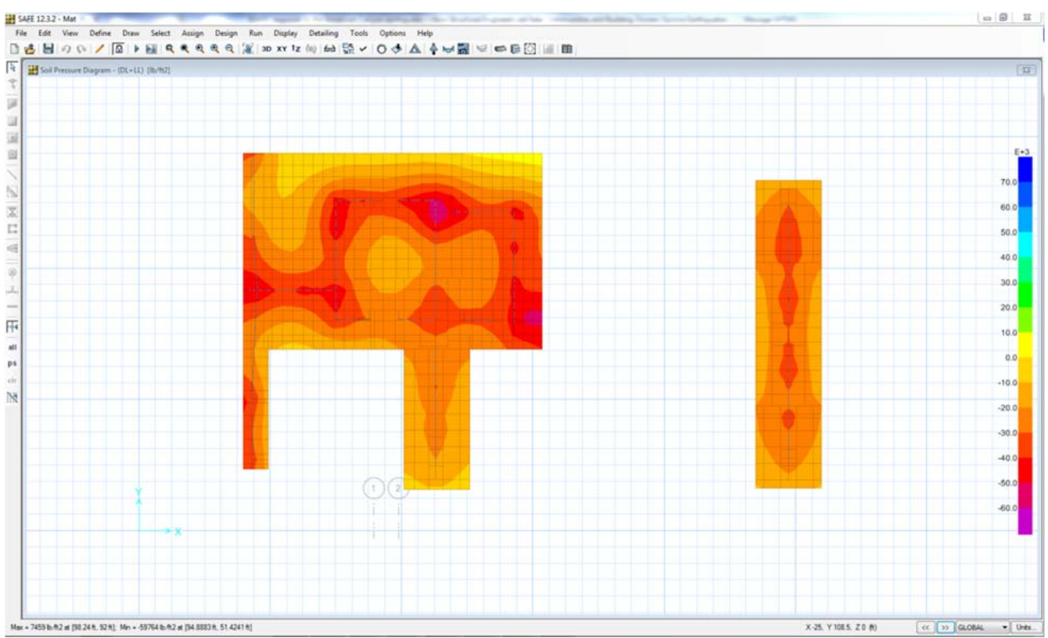
< Input Data - Location of Rock Anchors >



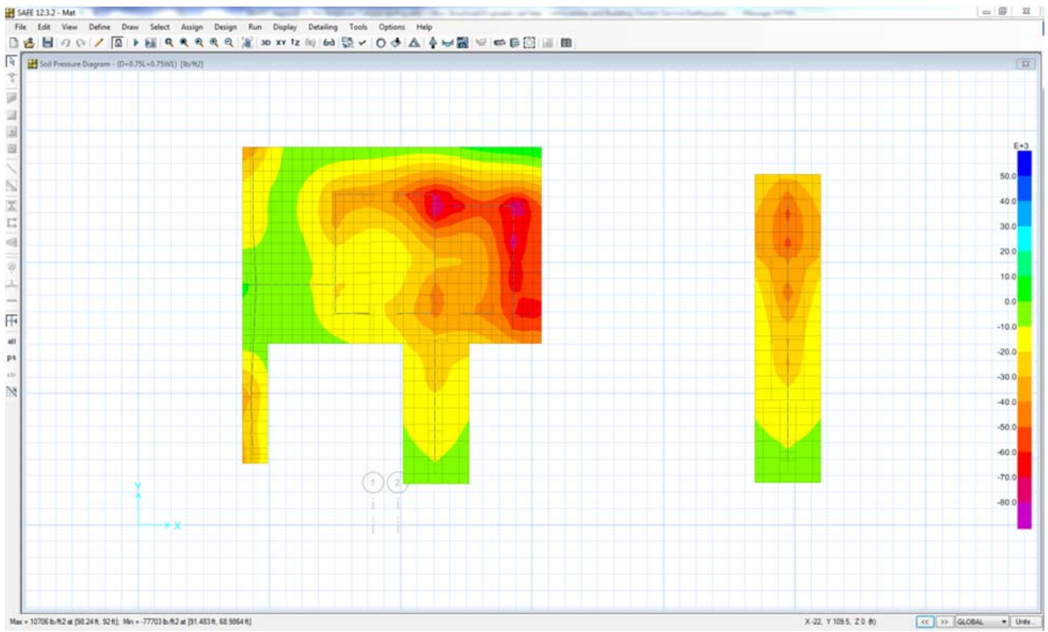
< Bearing Pressure >

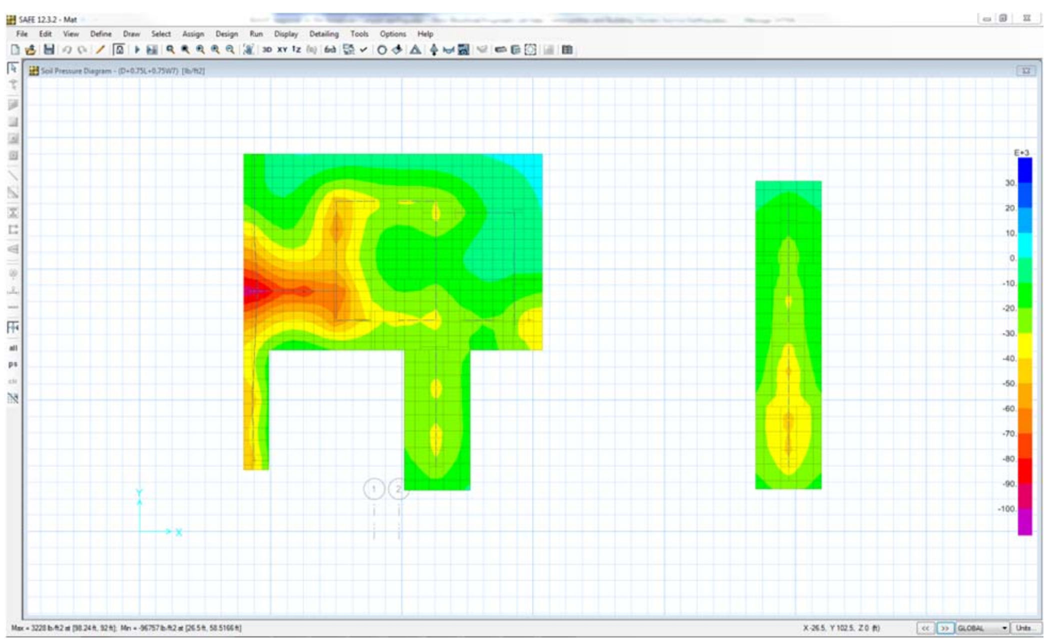
A. Gravity Loads (Dead Loads + Live Load)

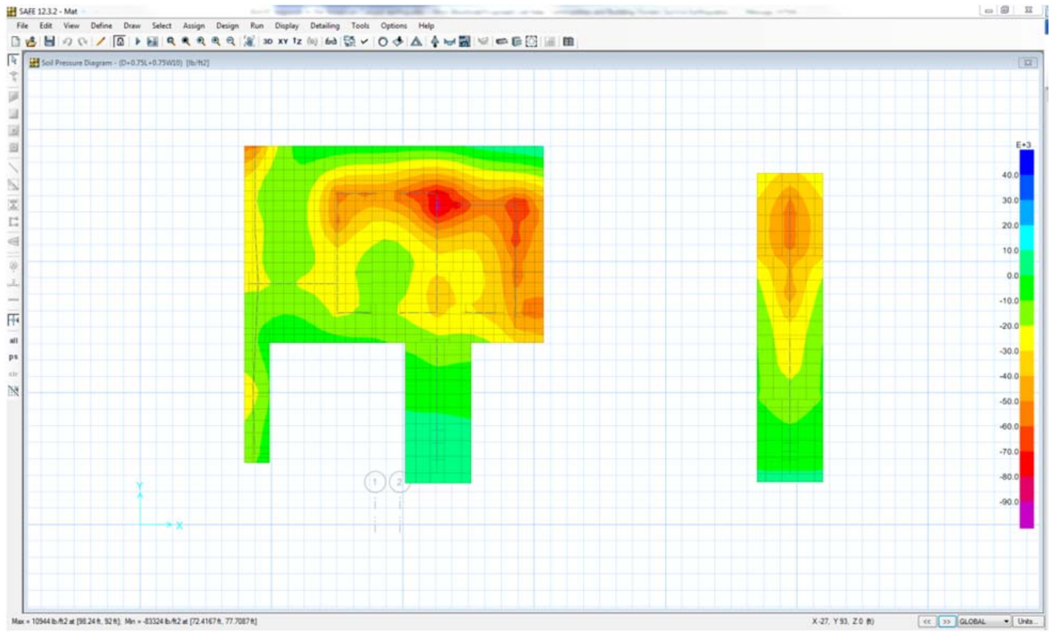
- Maximum bearing pressure: 57 ksf

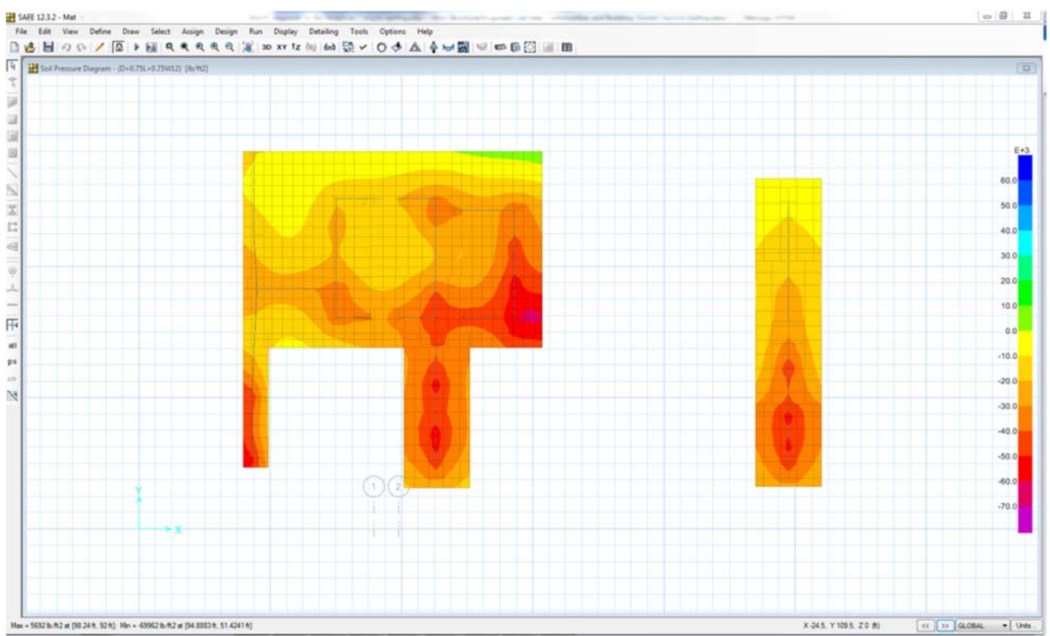


B. Combination of the Gravity Loads (Dead Loads + Live Load) and Lateral Loads (Wind loads)






SAFE 12.3.2 - Mat
File Edit View Define Draw Select Assign Design Run Display Detailing Tools Options Help
Soil Pressure Diagram - (D+0.75L+0.75W10) [lb/ft2]
E+3
40.0
30.0
20.0
10.0
0.0
-10.0
-20.0
-30.0
-40.0
-50.0
-60.0
-70.0
-80.0
-90.0
Max = 10944 lb/ft2 at (98.24 ft, 92 ft); Min = -83324 lb/ft2 at (72.4167 ft, 77.7087 ft)
X-27, Y 93, Z 0 (ft)
GLOBAL
Units...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Soil Pressure Diagram - (D+0.75L+0.75W12) [lb/ft2]
E+3
60.0
50.0
40.0
30.0
20.0
10.0
0.0
-10.0
-20.0
-30.0
-40.0
-50.0
-60.0
-70.0
Max = 5692 lb/ft2 at (98.24 ft, 92 ft); Min = -69962 lb/ft2 at (94.8883 ft, 51.4241 ft)
X-24.5, Y 109.5, Z 0 (ft)
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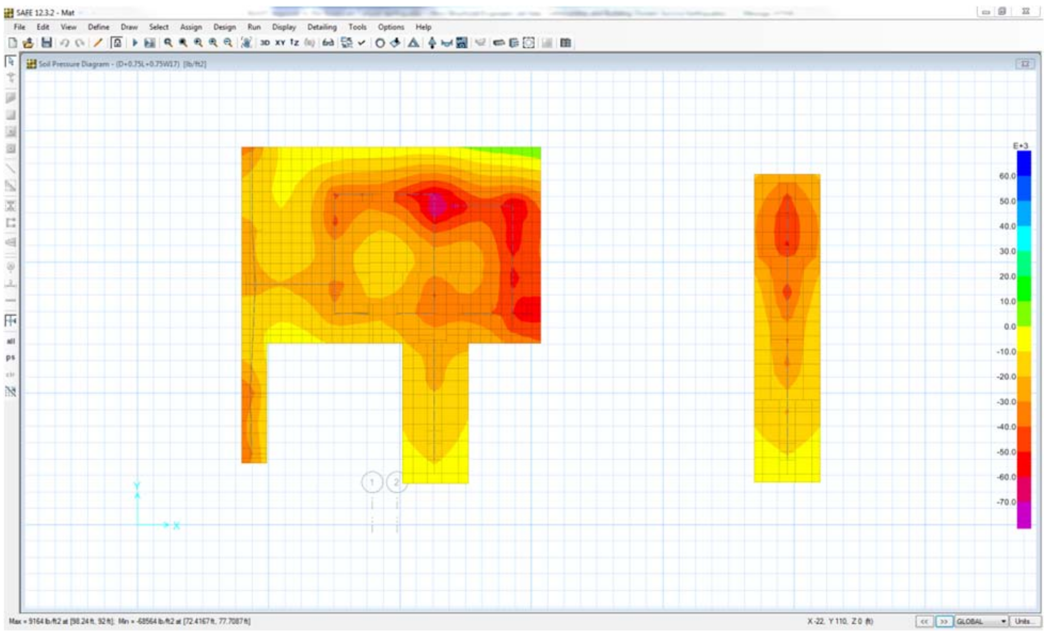
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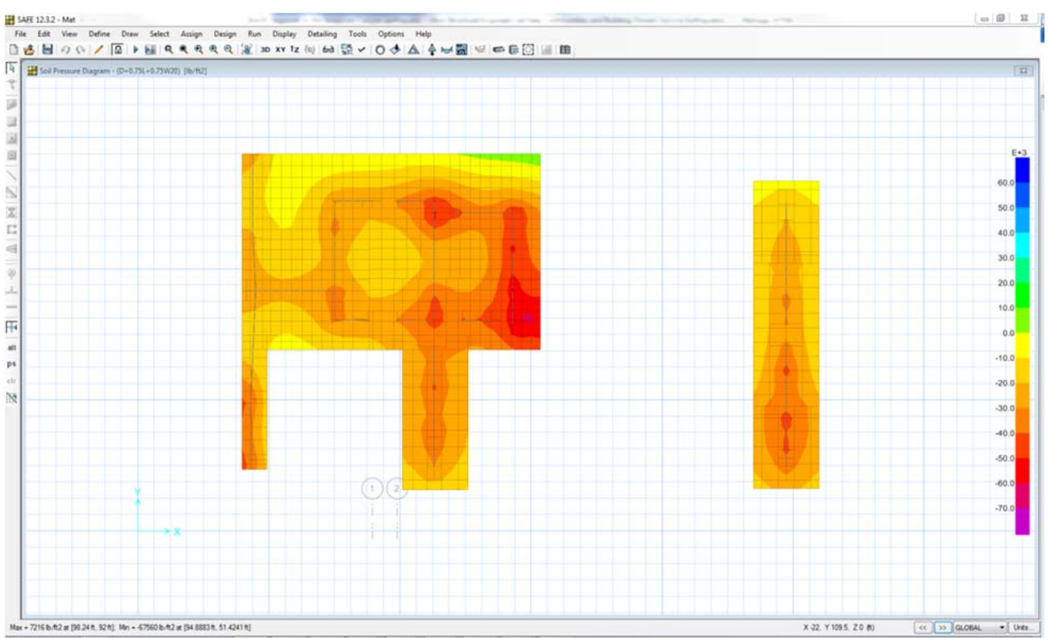
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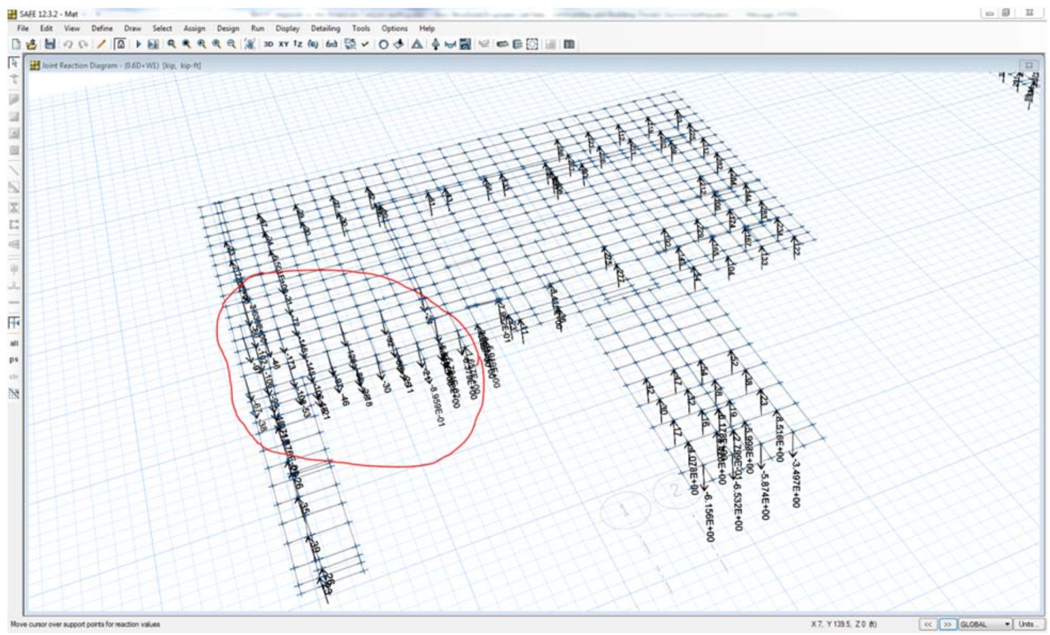
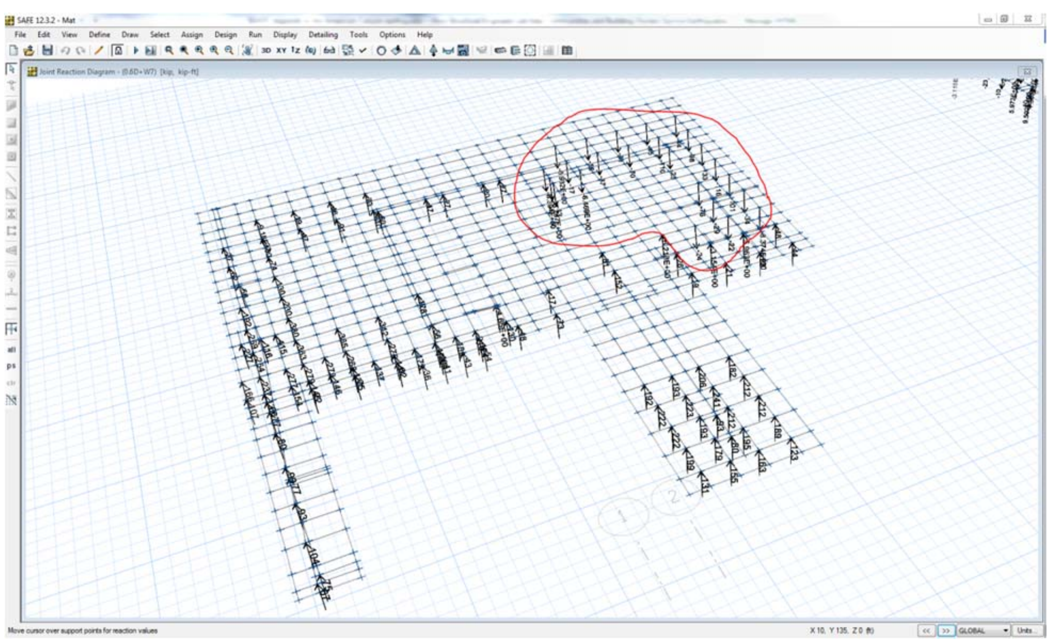
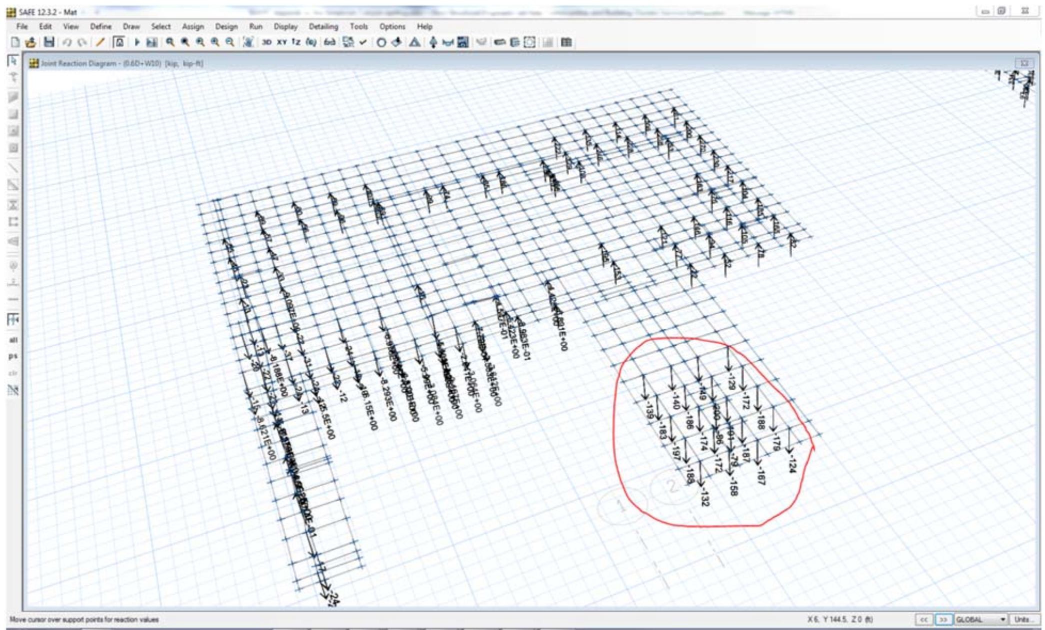
C. Reactions (uplift) at Rock Anchor

Rock anchor (250 kips of tension capacity) is modeled as a tension only point spring (250 kips/in)

- 0.6DL+Each Wind Load Case (RWDI Load Case W1, W7, W10,W12,W17 & W20)



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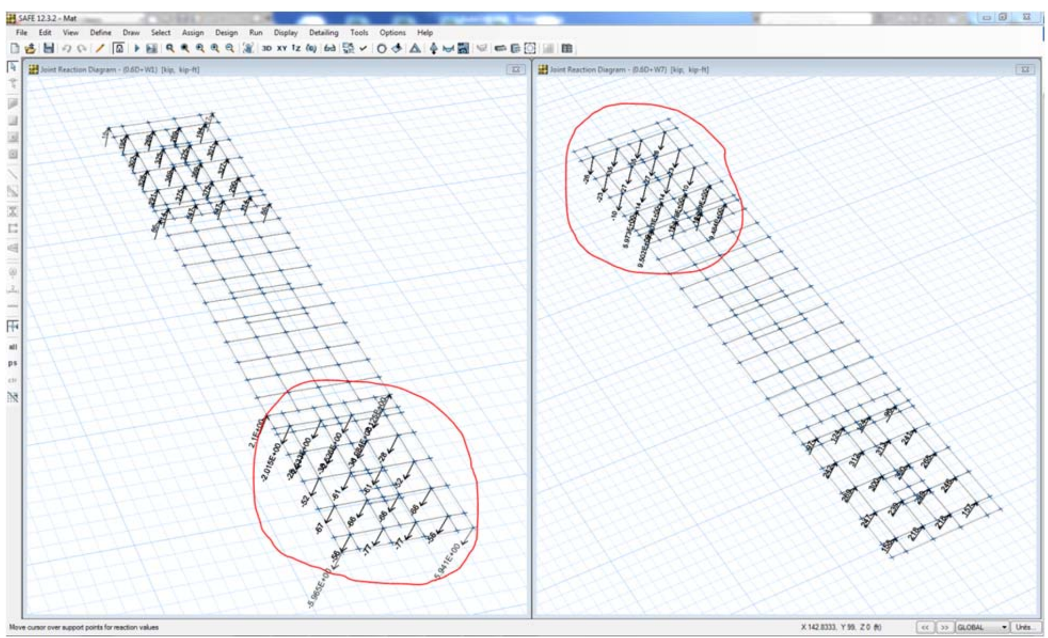
Move cursor over support points for reaction values


SAFE 12.3.2 - Mat
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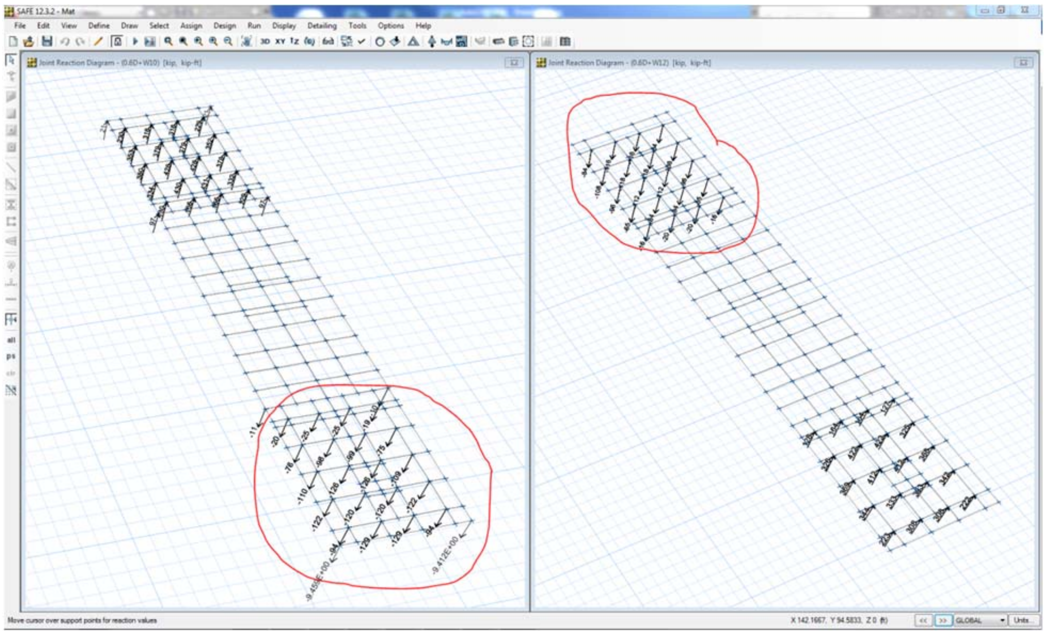
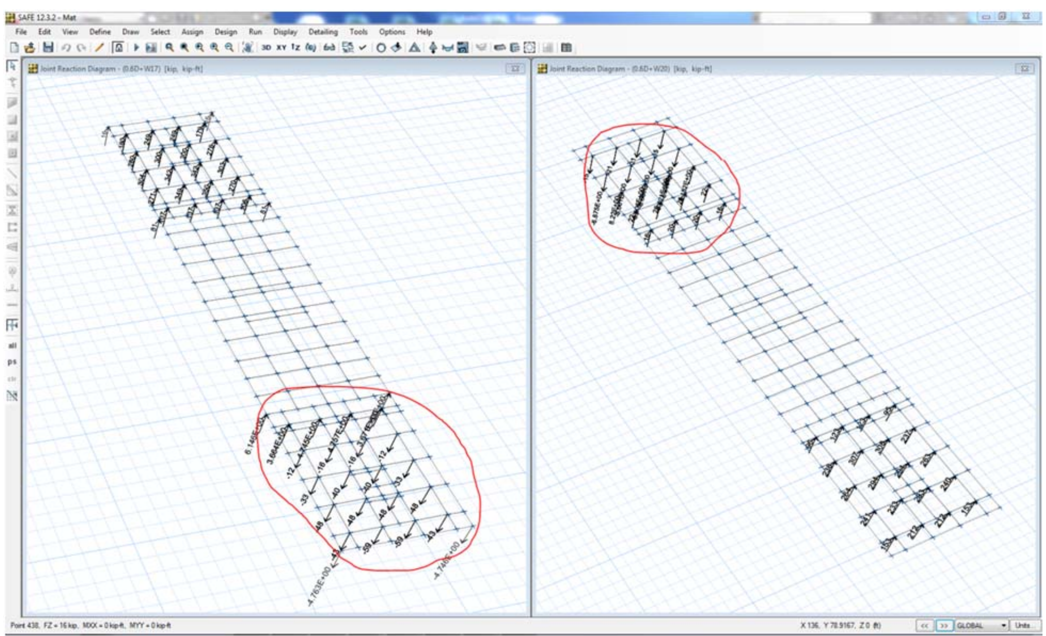
Joint Reaction Diagram - (0.6D+W7) [kip, kip-ft]
Move cursor over support points for reaction values





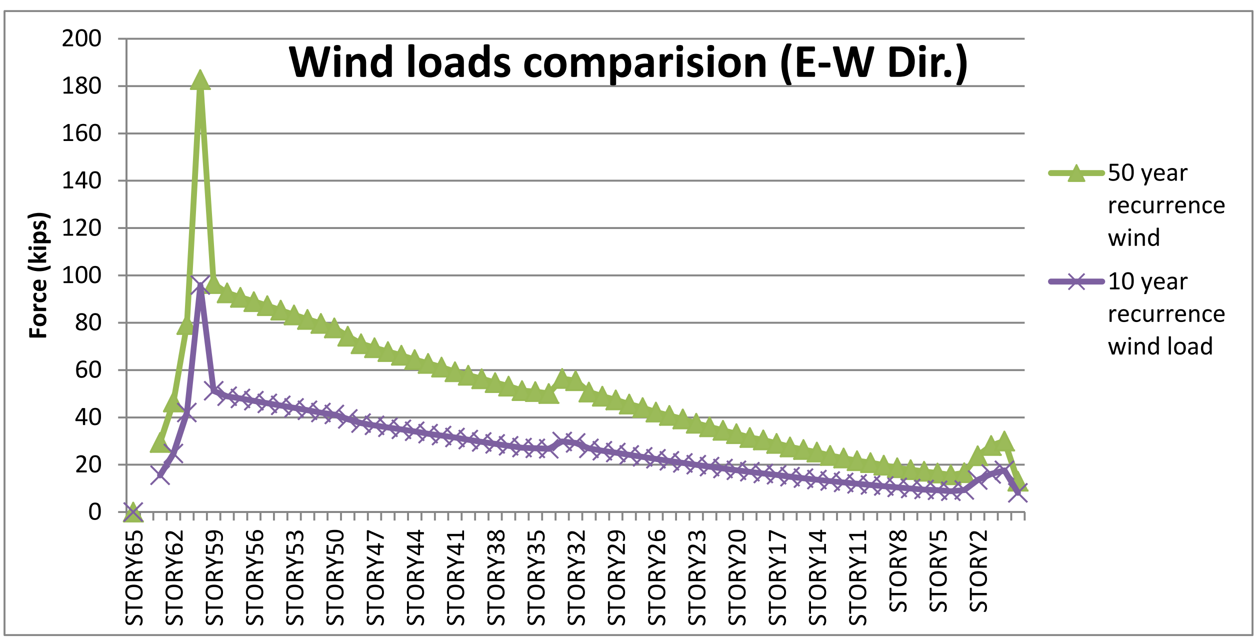
2.0 Relevant engineering investigation

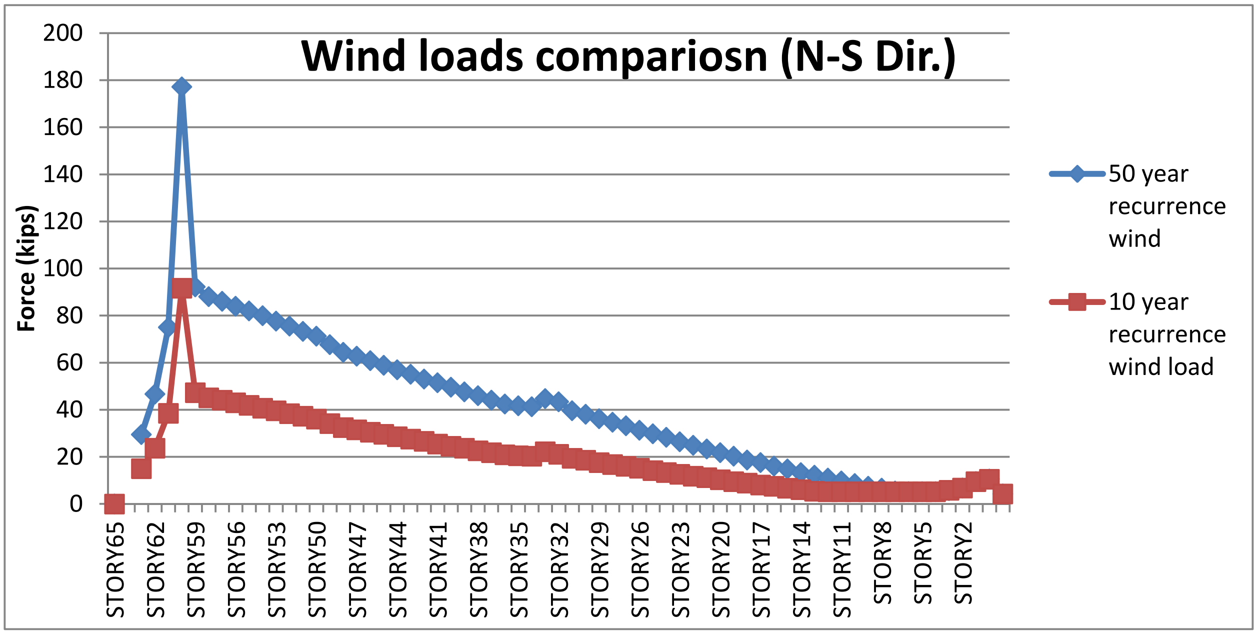
2.1 Geotechnical report

Investigation on soil condition was conducted by Museur Rutledge Consulting Engineers. In general, the current design is in compliance with the recommendations made by the geo-technical engineers.

2.2 Wind tunnel testing report

The wind induced structural loads (50 year recurrence wind loads/ 10 year recurrence wind loads) were provided (See below for comparison of two sets of the wind loads). For evaluation of serviceability, the peak accelerations and the peak torsional velocities were estimated by the wind tunnel testing lab. The final complete wind tunnel testing report was not available to us. Based on the provided information and our experience in design of the high-rise buildings, serviceability issue associated with the wind events doesn't seem critical.





Story level	Peak accelerations without hurricane (mg)		Peak torsional velocity without hurricane (milli-rad/sec)	
	10 year	1 year	10 year	1 year
61 (Topmost residential floor)	18	11	2.8	1.7
60	18	10	2.8	1.6
59	18	10	2.8	1.6
58	17	9.9	2.7	1.6
57	17	9.6	2.7	1.6

3.0 Reviewer's opinion

Rosenwasser/Grossman Consulting Engineers, P.C. has completed the peer review of the design documents prepared by the engineer of record Desimone Consulting Engineers. It is our opinion that the current design complies with the building design codes and the standard of care. The prepared drawings are reasonably completed and coordinated with other trades.

Appendix A. Code compliance check list

Appendix B. Sample calculation sheets

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
1. Design Loads					
1) Gravity loads	NYC BC 1607 Table 1607.1	Loading Schedule on Dwg. S-001		√	
2) Wind loads	NYCBC BC 1609	• Wind design data on Dwg. S-001 • Wind tunnel testing results (for structural loads and acceleration study) are conveyed by the engineer of record via email	• Wind loads are estimated using wind tunnel testing. The wind induced structural loads (wind loads for 50 year recurrence wind event and 10 year recurrence wind event) are provided for design and evaluation of serviceability.	√	
3) Seismic loads	NYCBC BC 1609	Seismic design data on Dwg. S-001			Seismic load resisting system should be defined as “Bearing wall system with ordinary reinforced concrete shear walls” in lieu of “Building frame system with ordinary reinforced concrete shear walls”. However, this discrepancy will have little impact on

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
					the final design.
4) Soil lateral loads	NYCBC BC 1610	• Geotechnical report "Recommendation for foundation design of ninth avenue development) dated June 10, 2008 • Draft Geo-technical report "Subsurface conditions on site and foundation recommendation for South-west tower" dated January 2014 • Final Geo-technical report "Subsurface conditions on site and foundation recommendation for South-West tower " dated July	• Equivalent lateral soil pressure at rest (which is used for design of the foundation walls) was estimated to be 240 lb/ft^2 (top 10 ft soil from grade) and 120 lb/ft^2 (below 10ft). • In addition, 300 lb/ft^2 of surcharge load was recommended to be used for design of the foundation walls adjacent to sidewalk.	√	

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	**Referenced Code section**	**Referenced document**	**Detail**	**Remarks (Code compliance)**	
		2014			
2. Structural Design Criteria and Assumptions					
1) Serviceability					
A. Deflection due to gravity loads	NYCBC BC 1604.3 (Table 1604.3)	Dwg. S-237 Upper typical floor (37^{th}-51st floor) Plan	• Deflection of flat plate at the typical floors was checked for the deflection limit criteria in NYCBC 2008 Table 1604.3. (L/240 for the live load only and L/180 for a combination of dead loads and live loads assuming that slab is supporting non-plaster ceiling).	√	
B. Lateral displacement		• Structural drawings • Wind tunnel testing results • Geo-technical report for seismic loads design parameters	• Story drift due to wind loads: 50 year / 10 year recurrence wind loads were used to estimate story drift. According to our study, story drifts in E-W direction at the critical floors (54F and 55F) due to 50 year wind loads and 10 year wind loads are hn/272 and hn/544 respectively. • Story drift due to earthquake	√	• Story drift criteria (more than hn/500 at 10 year recurrence wind) used for design can be acceptable, as long as non-structural elements such as cladding and components, partitions and mechanical equipment are properly designed to accommodate the

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
			loads: less than 0.02hn (maximum allowable story drift for seismic use group I/Ordinary reinforced concrete shear walls).		building displacement.
C. Perception to motion	ISO criteria (these criteria are chosen by the wind tunnel testing lab)		• Peak accelerations along with peak torsional velocities are provided for evaluating serviceability. The peak accelerations at top 6 floors (57th – main roof) for 10 year recurrence wind event are provided by the wind tunnel testing lab.	√	• The estimated accelerations at the uppermost residential floor (61st floor) seem to be within an acceptable range for the residential buildings. The peak torsional velocities seem to be higher, but this evaluation is based on the criteria tentatively set.
2) Analysis	NYCBC BC section 1604.4	• Structural drawings • Wind tunnel testing results • Geo-technical report for seismic loads design parameters	• For our review, two separate computer analysis models (for wind loads and the seismic loads separately) of the structure were created in consideration of the effects of the loads on structural properties and connections. • Assumptions on the crack	√	• It was found that there are some discrepancies on assumptions of the crack section and connections. However, the overall building behavior from our analysis model seems to be similar to the original design.

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
			section of the main structural members were discussed with the engineer of record. • Overall behavior of the structure and internal forces at representative members were reviewed and compared with the original design.		
3) Structural occupancy category and importance factors	NYCBC BC section 1604.5	MWFRS Wind design data and Seismic design data on Dwg. S-001.00	• Wind Loads: Importance factor 1.0 (Occupancy category II) • Seismic Loads: Importance factor 1.0 (Seismic use group: I (Occupancy category II)	√	
4) Anchorage	NYCBC BC section 1604.8	• Column schedule (Dwg. S-401 thru S-403) • Foundation plan Dwg. FO-101) • Foundation Mat Reinforcement Detail (Dwg. FO-121) and Typical Foundation Detail	• Design of column (Column 1, 2, 3, 21, 22 and 27) from base (foundation) to top (main roof) is reviewed. • Shear wall at the lower levels was checked for the design loads (the gravity loads and the lateral loads). • Typical connection detail of flat plate and vertical members are	√	

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
		(Dwg. FO-111)	reviewed.		
5) Lateral displacement capacity of slab-column connection not to contribute lateral resistance	NYCBC BC section 21.11.5	• Dwg. S-237 Upper Typical floor (37^{th}-51st floor) slab reinforcing	• Direct punching shear ratio was checked at 50th floor (where story drifts due to the seismic loads are the largest) since the frames are not participating in resisting the seismic loads.	√	• It was confirmed by the engineer of record that the frames (flat slabs and columns) are not participating in resisting the seismic loads. • Shear reinforcing to achieve additional ductility in slab-column connection is not required.
3. Conformity of structural design with engineering investigation					
1) Geo-technical engineering report		• Structural drawings • Geotechnical report dated June 10, 2008 • Geotechnical report dated July 1, 2014	• The geo-technical report titled “Sub-surface investigation data report, 9^{th} Avenue Development” dated May 19, 2008 was mentioned as an affiliated report in the geo-technical report dated June 10, 2008.		

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
A. Stability of the adjacent buildings or structures		Dwg. S-001 General notes	• Influence line for footing bearing surface at bedrock: As per the geo-technical report dated June 10, 2008, the incline of the influence line is recommended to be 1:1. However, for the final design, the steeper influence line (1:2) is used.	√	• It was confirmed by the engineer of record that the 1:2 influence line was determined after discussion with the geo-technical engineers
B. Soil bearing capacity		• Dwg. S-001 General notes • Geotechnical reports dated July, 2014 and June 10, 2008	• For spread footings and mat, 40tsf of the allowable bearing capacity is recommended to be used for footing design. • For columns adjacent to the open track (Amtrak and LIRR) located at the north side of the site, 58 inch diameter of drilled caisson with 500 ton of a compressive capacity is recommended to be used. A total three caissons are placed at the columns with a full height transfer beams at the 1st floor to pick up these three offset columns.	√	• It was confirmed by the engineer of record that the final mat foundation design was based on 20,000 kips/ft^3 of the subgrade modulus as per the geo-technical engineer's updated recommendation. Our review is undergone using this revised spring coefficient.

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
C. Ground water level and waterproofing			• A permanent drainage system behind retaining walls and below slabs-on-grade will be in place.	√	• Hydrostatic pressure is not considered for the design of foundation walls, slab-on-grade and mat foundation.
D. Uplift		• Dwg. FO-101 Foundation plan • Dwg. FO-121 Foundation mat reinforcement detail	• Rock tie-down anchors (2 ½ inch diameter- 150 ksi, threaded bars) with a 250 kips capacity in tension is recommended to be used to resist the potential uplift forces built up at underside of the mat foundation. • As per the engineer of record, rock anchor is modelled as a reversed point load (250 kips of upward force) at each node where rock anchor is located.	√	• Our peer-review is undergone assuming 250 k/in of tension. • Tensile reaction forces are developed in the rock anchors per allowable stress design load combinations in the NYCBC 2008. It shall be noted that tensile force demands in the rock anchors as well as associated slab forces are affected by the stiffness of the rock anchors which is assigned equal to 250 kips/in in our SAFE analysis model. • According to our review

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
					of the current mat foundation design, the current design seems sufficient.
4. Complete load path					
1) Gravity loads		Structural drawings	Gravity loads are resisted by cast-in-place flat plate (horizontal elements) and cast-in-place columns and shear walls (vertical elements). These loads are transferred to mat foundation and spared footings residing on work with a 40 tsf of bearing capacity.	√	Load path for the gravity loads is complete.
2) Wind loads		Structural drawings	• Wind loads are transferred to shear walls and concrete columns by rigid diaphragm (8 inch thick flat plate). • Lateral load resisting system consists of frames (flat plate slabs and columns) and coupled shear walls. • Cellar floor was assumed to be the base for the lateral loads and overturning moments due to	√	A load path for the wind loads is complete.

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
			wind loads are resisted by friction between footings and rock.		
3) Seismic loads		Structural drawings	• Seismic loads are transferred to shear walls by rigid diaphragm (8 inch thick flat plate). • Lateral load resisting system consists of shear walls. • Cellar floor was assumed to be the base for the lateral loads and overturning moments due to seismic loads are resisted by friction between footings and rock.	√	A load path for the seismic loads is complete.
4) Soil lateral load	NYCBC BC 1610	Cellar floor plan and ground floor plan	• Support condition of foundation walls at floors (ground floor slab and cellar floor) is reviewed.	√	A load path for the soil lateral load is complete.
5. Design of members					
	NYCBC BC 1627.6.2	Structural drawings	• Representative structural elements (flat plate at two typical floors, shear walls, columns, link beams, spread		

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
			footings, mat foundations and foundation walls) were checked based on the results from our analysis.		
1) Flat plate		• Dwg. S-237 Typical floor (37th-51st floor) slab reinforcing • Dwg. S-237 Typical floor (37th-51st floor) slab reinforcing	• Adequacy of slab thickness and reinforcing is reviewed. • Capacity of flat plate connection (Two way shear) between flat slab and columns is checked. • Slab reinforcing is reviewed using our in-house design program based on the equivalent frame analysis.	√	• In general, the current flat plate slab design seems adequate. • 4#6 top & bottom continuous additional reinforcing seems missing in the Bent (Column 4 -23-22-9) at two typical floors. "In our review, we found that the flat plates (bent for Column 4-23-22-9) are not designed for the lateral loads. We now understand, from the Engineer of Record, that this portion of the frame was not used in the lateral force resisting

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
					system. We do not see it practical to exclude this from the lateral system, but the remaining lateral load resisting system was found to have sufficient strength to resist the entire lateral load. Therefore, we take no objection strictly from a code standpoint."
2) Shear wall		Shear wall plans • Dwg. S-410.00 supporting cellar and 1st floor • Dwg.S-411.00 supporting 2nd floor	• Reinforcing at shear walls supporting ground floor (@cellar floor) and 2nd floor (@1st floor) is reviewed.	√	
3) Columns		Dwg. S-401.00 – S-402.00 Column schedule	• Axial loads at 6 representative columns and shear walls are calculated and compared with the original design. • Design of corner column (Column	√	

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
			1), Interior Column (Column 21, 22 and 27) and Exterior Columns (Column 2 and3) is reviewed.		
4) Link Beams		Dwg. S-441.00 Link beam schedule and details	Design of link beams (conventional cast-in-place concrete beams and embedded steel beams) is reviewed.	√	
5) Transfer Beams and strap beams		• Dwg. S-204 4th floor plan • Dwg. S-504.00 Concrete section	Design of transfer beams and strap beams is reviewed.	√	Our study indicated that, the strap beam bracing column 3, 4 and 5 shall be tied back to the shear walls. The engineer of record has agreed to incorporate this into the final design.
6) Mat foundation		Dwg. FO-102.00 Cellar floor rebar plan	• Adequacy of depth of mat, flexural reinforcing and shear reinforcing is reviewed. • Adequacy of rock anchor layout (location and the number of rock anchors) is reviewed.	√	
7) Spread footings		Dwg. FO-101 Foundation plan	Adequacy of dimension and reinforcing of F-9.0 for column 27 is reviewed based on 40tsf of a bearing capacity.	√	

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
8) Foundation walls		• Dwg. FO-101 Foundation Plan • Dwg. S-201 1st floor layout plan • Dwg. S-001 General notes • Dwg. FO-111 Typical foundation details • Dwg. FO-112 Foundation details	• 300 psf of surcharge load is considered for design of the foundation walls. • Adequacy of thickness and reinforcing at foundation walls (South and East) is reviewed.	√	
6. Performance-specified structural components					
1) Cladding		• Structural drawings dated May 16, 2014 and August 1, 2014 • Architectural drawing A-203.00 West elevation	Window wall system is used for cladding.	√	Cladding design, performance of cladding and their connections were excluded in our peer review.

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
7. Structural Integrity					
1) Prescriptive requirement	NYCBC BC 1625				
A. Continuity and ties	NYCBC BC 1917.2				
• Slab reinforcing	NYCBC BC 1917.2.1	• Dwg. S-205.00 Typical floor (5th–34th floor) plan • Dwg. S-237.00 Typical floor (37th–51st floor) plan • Dwg. S-503.00 Typical concrete details	• Continuous mat bottom reinforcing is specified in two orthogonal directions at flat slab. • Bottom mat reinforcement is anchored at discontinuous edges.	√	
• Peripheral ties	NYCBC BC 1917.2.2	• Dwg. S-205.00 Typical floor (5th–34th floor) plan • Dwg. S-237.00 Typical floor (37th–51st floor) plan • Dwg. S-503.00	• 4-#6 Top and Bottom continuous re-bars are shown in addition to continuous bottom mat reinforcement which can be used as peripheral ties.	√	

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
		Typical concrete details			
• Horizontal ties	NYCBC BC 1917.2.3	• Dwg. S-205.00 Typical floor (5th–34th floor) plan • Dwg. S-237.00 Typical floor (37th–51st floor) plan • Dwg. S-503.00 Typical concrete details	• Placement of slab bottom reinforcing within column cage at all columns was checked using three specific load combinations for structural integrity.	√	
• Vertical ties	NYCBC BC 1917.2.4	Dwg. S-401.00 – S-405.00 Column schedule	• Six columns (column 1, 2, 3, 21, 22, & 27) from foundation to main roof were reviewed for continuity requirements.	√	
B. Lateral bracing	NYCBC BC 1625.3	Dwg. S-205.00 Lower Typical floor (5th– 34th floor) plan Dwg. S-237.00 Upper Typical floor (37th–51st floor) plan	• Floor and roof diaphragms (flat plate slab) are tied to shear walls and columns.	√	

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
C. Vehicular impact	NYCBC BC 1625.5	Dwg. S-401.00 – S-404.00 Column schedule		√	
8. General conformance of structural plans with architectural plans					
		• Architectural drawing set dated May 12, 2014 • Structural drawing sets dated May 9, 2014 and August 1, 2014 • Mechanical drawing set dated May 12, 2014	• Column location, extent of slab, configuration of shear walls and major slab openings are reviewed for conformance with architectural drawings. • Mechanical drawings were referenced only for information of the major mechanical equipment shown on structural drawings.	√	
9. Major mechanical items					

Peer Review – Code Compliance Check List as per NYCBC BC section 1627.6.1 Scope of the structural peer review

Item	Referenced Code section	Referenced document	Detail	Remarks (Code compliance)	
1) Water tank		• Structural drawing sets dated May 9, 2014 and August 1, 2014 • Mechanical drawing set dated May 12, 2014 (Dwg. M-119)	• A total four water tanks (Two – 15'-6" diameter water tanks and two -11'-6" diameter water tanks) are located on the 65th floor. • Concrete bracing beams are shown on the 65th floor plan to support water tanks.	√	
2) Storm water detention tank		• Structural drawing set dated May 9, 2014 and August 1, 2014 • Mechanical drawing set dated May 12, 2014	Located at the cellar floor which is supported by slab on grade.	√	
10. General completeness of structural drawings					
		• Structural drawing set dated May 9, 2014 and August 1, 2014		√	Design is fairly completed and coordination regarding geometric information is in progress