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ES763545762 Scan Code

Thornton Tomasetti
Building Solutions

270 PARK AVENUE
NEW YORK, NY

STRUCTURAL PEER REVIEW REPORT
BELOW GRADE AND SUPERSTRUCTURE

NB# 121205604

August 14th, 2020

REVISED September 24, 2020

Prepared For

JP Morgan Chase
270 Park Avenue,
New York, NY 10022

Prepared By

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Thornton Tomasetti

TABLE OF CONTENTS

Attachment A
STRUCTURAL PEER REVIEW STATEMENT

This structural peer review and report is complete for the whole building, or
For phase 2 **of** 2 **phased submissions**

Structural peer reviewer name: Chris Christoforou

Structural peer reviewer address: 51 Madison
NY, NY 10010

Project address: 270 Park Ave
NY, NY 10017

Department application number for structural work: #121205604

Structural Peer Reviewer Statement

I (insert name) Chris Christoforou ***am a qualified and independent NYS licensed and registered engineer in accordance with BC Section 1617.4, and I have reviewed the structural plans, specifications, and supplemental reports for (Insert address and DOB application # for structural work)*** 270 Park Ave NY, NY 10017 ***and found that the structural design shown on the plans and specifications generally conforms to the foundation and structural requirements of Title 28 of the Administrative Code and the NYC Construction Codes. The Structural Peer Review Report is attached.***

New York State Registered Design Professional
(for Structural Peer Review only)

Name (please print) Chris Christoforou

Signature__________ Date 08/14/2020

PE/RA Seal (apply seal, then sign and date over seal)

STATE OF NEW YORK
CHRIS A. CHRISTOFOROU
065946
PROFESSIONAL ENGINEER

cc: Project Owner
Project Registered Design Professional

A. EXECUTIVE SUMMARY

The following report contains the summary of Thornton Tomasetti's (TT) peer review of the below grade documents (Phase 1) and superstructure (Phase 2) for Project Greyhound located at 270 Park Avenue, New York, NY. The peer review has been performed in accordance with the NYC 2014 Building Code Requirements. This peer review is based on the 50% Design Development documents dated 02/18/20, Structural Steel Bulletin 1 dated 04/03/20, Structural Steel Bulletin 2 dated 05/08/20, and Milestone 1 documents dated 07/24/2020. This peer review report encompasses both Phase 1 and Phase 2 reviews.

This peer review has evaluated the below grade and superstructure elements based on the loads of the tower above provided by Severud, the Engineer of Record (EOR), in addition to loads independently calculated by TT. See Appendix 1 for Severud design loads. This peer review report does not extend to elements outside the below grade design, superstructure design, or documents as listed in Section D. The foundation elements (caissons, caisson caps, and spread footings) supporting the below grade and superstructure elements were designed by Mueser Rutledge Consulting Engineers (MRCE) and were peer reviewed by Langan Engineering. See Figure 1 for peer review Phase breakdown.

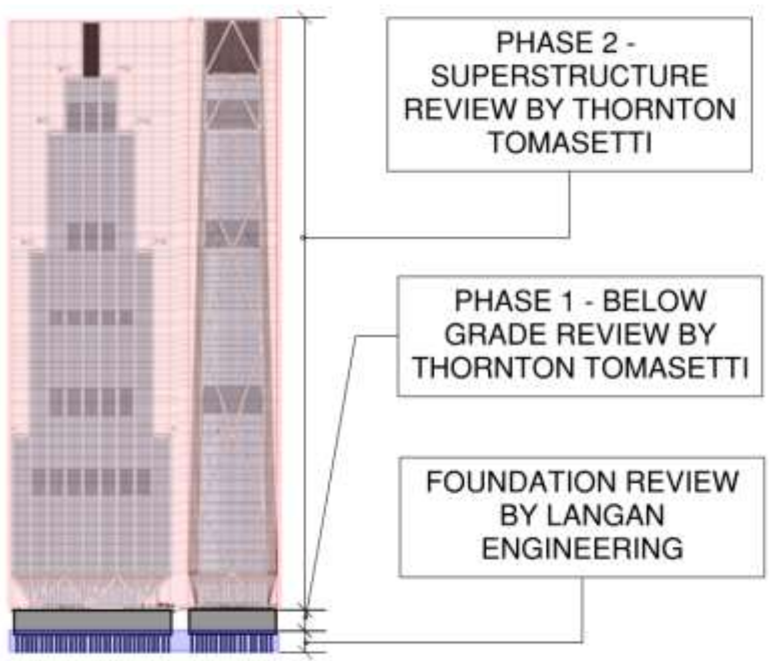


Figure 1. Peer Review Phase Breakdown

1. Confirm that the design loads conform to this code.

Thornton Tomasetti has reviewed the design loads for conformance with the NYC Building Code loading requirements. The design loads (construction dead, superimposed dead, and live) are in conformance with the NYC Building Code.

We have reviewed the wind and seismic base shear based on 2014 NYC Building Code and based on the building geometry from the 50% Design Development architectural drawing set issued on February 14, 2020. Any discrepancies have been discussed and resolved with the EOR. A building of this height and massing requires a wind tunnel test to validate the wind loads on the building's structure. A wind tunnel test has been performed by RWDI, (see RWDI Wind Test Report Dated November 15, 2019), and wind loads have been calculated using the preliminary building stiffness properties. As part of a normal design process, final building properties will be determined as the structural design of the superstructure is finalized, and a final wind tunnel report with recommendations is issued. We will peer review the finalized wind load recommendations should they vary significantly from the preliminary wind loads. We have confirmed that wind base shear and overturning moment employed by EOR are not less than ASCE-7 requirements.

2. Confirm that other structural design criteria and design assumptions conform to this code and are in accordance with general accepted engineering practice.

The structural design criteria and design assumptions are in accordance with the NYC Building Code and general engineering practice. We have resolved any discrepancies we have found with the EOR.

3. Review geotechnical and other engineering investigations that are related to the foundation and structural design and confirm that the design properly incorporates the results and recommendations of the investigations.

We have reviewed the geotechnical data report and the geotechnical interpretative report produced by Mueser Rutledge Consulting Engineers (MRCE), dated June 27th, 2019. The design of the reviewed below grade documents incorporated these recommendations. Langan Engineering has peer reviewed the foundation work performed by MRCE and issued peer review memorandums dated March 31st, 2020 and May 14th, 2020 detailing the review of MRCE's caisson, caisson cap and footing designs. See Appendix 4 for Langan's Peer Review Reports.

4. Confirm that the structure has a complete load path.

We confirm that the superstructure has a complete load path for the design loads indicated. We understand framing modifications may occur to framing above Level 11M; as such, TT will review major design changes and will modify this report if required.

5. Perform Independent calculations for a representative fraction of systems, members and details to check their adequacy. The number of representative systems, members, and details verified shall be sufficient to form a basis for the review's conclusions.

We have performed independent calculations for a representative sample of floor beams, columns, truss elements, and below grade elements to check their adequacy. Any discrepancies have been discussed with the EOR and resolved accordingly.

6. Verify that performance-specified structural components (such as certain precast concrete elements) have been appropriately specified and coordinated with the primary building structure.

No performance-specified structural components are included as part of the provided design documents.

7. Confirm that the structural integrity provisions of the code are being followed.

We have reviewed NYCBC integrity requirements for sample of key elements and determined that they are in compliance with the code provisions.

8. Review the structural and architectural plans for the building. Confirm that the structural plans are in general conformance with the architectural plans regarding loads and other conditions that may affect the structural design.

We have reviewed and confirmed that the below grade and superstructure structural documents are in general conformance with the architectural plans (based on the 50% DD plans dated Feb 14, 20200) and the below grade design loads and superstructure design loads are adequate.

9. Confirm that major mechanical items are accommodated in the structural plans.

The shear wall elements and slabs reviewed in the below grade package have been coordinated with the mechanical items. Penetrations and shaft openings are provided in the walls/floors per MEP requirements.

We have performed representative column load takedowns with general assumptions for mechanical loads as indicated on the structural documents. TT confirms mechanical equipment weights are indicated on the drawings where applicable.

10. Attest to the general completeness of the structural plans and specifications.

The below grade and superstructure documents peer reviewed in both Phase 1 and Phase 2 are generally complete.

B. INTRODUCTION AND STRUCTURAL SYSTEMS DESCRIPTION

1.0 INTRODUCTION

Thornton Tomasetti (TT) was retained by JPMorgan Chase to conduct a structural peer review for the Greyhound Project located in 270 Park Avenue, New York, NY.

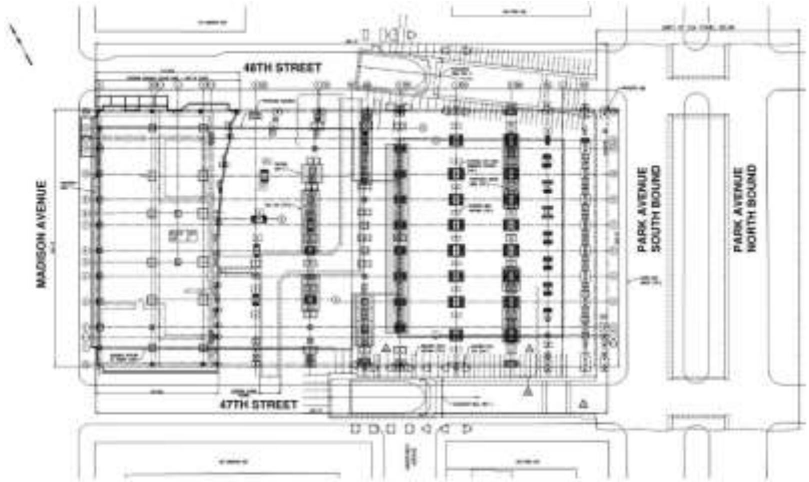


Figure 2. Site Map

The building is a 63-story high-rise office tower with an approximate height of 1388' above grade. The approximate lot size is 400'x200' and is located in the Midtown East neighborhood of Manhattan between 48th and 47th street on the north and south sides and between Madison Avenue and Park Avenue on the west and east sides of the lot. The tower tapers inward from the north and south sides and steps inward from the east and west sides at various point along the height to an approximately 45'x135' roof level footprint. The majority of the building is situated above the train shed connecting to Grand Central Terminal.

The construction of the foundations and shear walls for the new tower is to occur simultaneously with the demolition of the superstructure for the existing 52-story steel framed building currently occupying the site. Therefore, the new building foundation comprised of caisson caps and caissons will need to be cast around and on top of part of the existing building foundations to allow for concurrent demolition of the existing building superstructure.

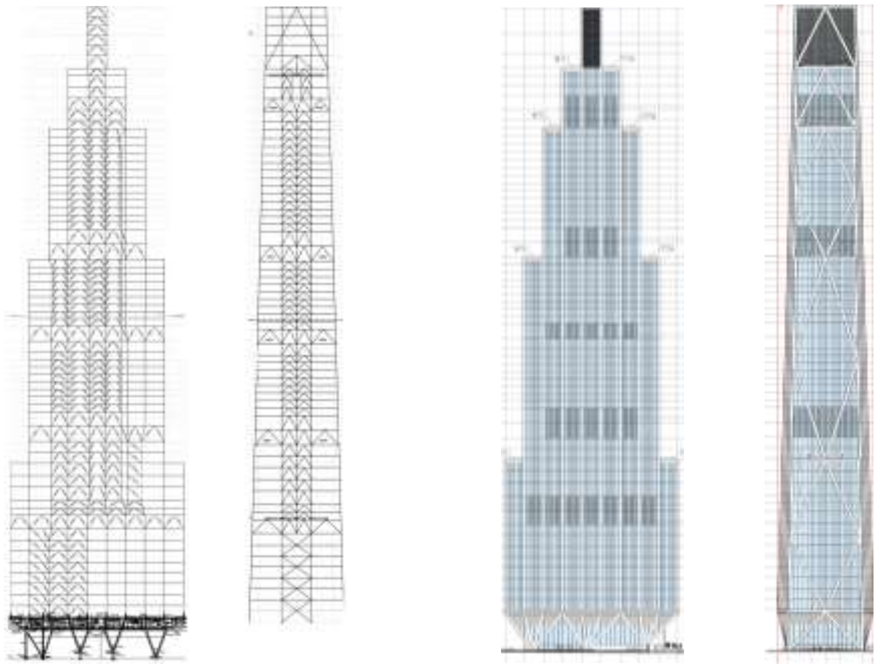


Figure 3. Building Sections

TT's role in Phase 1 was to perform a peer review of the below grade system, which includes the review of the overall building behavior. TT's review is based on the 90% MOU Foundation Set dated December 9, 2019, IFC Bulletin 6 dated April 3rd, 2020, the 50% DD Set dated February 14th, 2020, the Issued for Structural Steel Bulletin #1 (Mill Order – Seq. 1) set dated April 9th, 2020, the Issued for Structural Steel Bulletin #2 (Mill Order – Seq. 2a) dated May 8th, 2020 and IFC Bulletin 8 dated May 26, 2020. TT also studied the structural design for compliance to the recommendations in the Geotechnical reports by MRCE dated June 27, 2019 and the Wind-Induced Structural Responses report by RWDI dated November 14, 2019.

This Phase 2 peer review addresses the superstructure design while also encompassing the Phase 1 below grade review. In this review TT focused on the superstructure design based on the 50% DD Set dated February 14th, 2020, the Issued for Structural Steel Bulletin #1 (Mill Order – Seq. 1) set dated April 9th, 2020, the Issued for Structural Steel Bulletin #2 (Mill Order – Seq. 2a) dated May 8th, 2020 and Milestone 1 package dated July 24, 2020.

In general for peer reviews, the reviewers provide different, complementary services to advance the design of a building project. In this peer review report, the comments, suggestions and observations on the structural design performed to date are not intended to minimize or criticize the designer's efforts. Instead, the information in this report is intended to assist the designers by providing another perspective.

TT’s scope of work is as follows:

- Confirm that the design loads conform to the 2014 New York City Building Code.
- Confirm that other structural design criteria and design assumptions conform to the 2014 New York City Building Code and are in accordance with generally accepted engineering practice.
- Review wind tunnel reports and confirm that the design properly incorporates the results and recommendations of the investigation.
- Confirm that the structure has a complete load path.
- Independently assess the structural responses and stability of the building under actions of lateral and gravity loads.
- Perform independent calculations for a representative fraction of systems, members, and details to check their adequacy. The number of representative systems, members, and details verified shall be sufficient to form a basis for TT’s conclusions.
- Confirm that the structural integrity provisions of the 2014 New York City Building Code are being followed.
- Attest to the general completeness of the structural plans.
- Provide a written report that covers all aspects of the review performed, including conclusions reached by the reviewer.

2.0 STRUCTURAL SYSTEMS DESCRIPTION

2.1 LATERAL SYSTEM

The lateral load resisting system is composed of a steel braced core with outriggers above the 3rd floor. Below the 3rd floor is the "Tabletop" structure which consists of sloping perimeter "Fan" columns, interior "V" columns and transfer girders. The tabletop structure in conjunction with 3rd floor diaphragm completes the lateral system above grade and transfers lateral forces from the superstructure to the concrete shear walls below grade. The steel braced core transfers on two 25' deep built-up plate girders that are supported by the "V" columns on Gird Lines T3 and T7. The tabletop system transfers the lateral forces in the east-west direction down to the ground floor slab through the sloping columns. The ground floor slab serves as a diaphragm pulling the lateral forces to shear walls located at the west end of the site. The lateral forces are resolved through the shear walls into the foundation elements. In the north-south direction braces on grids D.1, SA, and K transfer lateral load to concrete shears walls aligned with the grids. The shear walls carry the lateral load down to the foundation elements. As previously indicated the steel braced core has one-story deep outriggers at levels 11, 18, 29, and 38. Supplemental diamond shaped wind bracing is also provided on the west and east sides of the tower. Columns are comprised of standard hot-rolled wide flange shapes, built-up box columns, and built up plate solid steel columns. Braces are standard hot-rolled wide flange shapes.

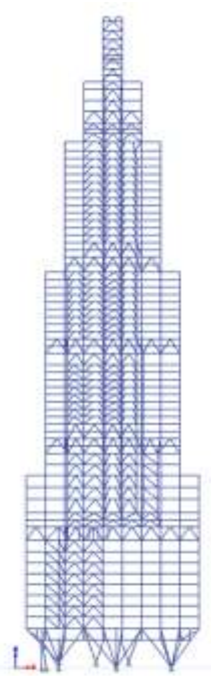


Figure 4. ETABS Image of Lateral System (TT Independent Model)

2.2 GRAVITY SYSTEM

The typical trading floor construction is 3” metal deck with 4 ½” concrete thickness for a total slab thickness of 7 ½”. The typical office floor construction is 3” metal deck with 2 ½” concrete thickness for a total slab thickness of 5 ½”. The typical mechanical floor construction is 3” metal deck with 4 ½” of concrete and an additional 3” of concrete topping. Steel framing supports the deck and spans between steel girders. Typical east-west girder span length is 40’ and typical beam span north–south rangers from 30’ to 60’.

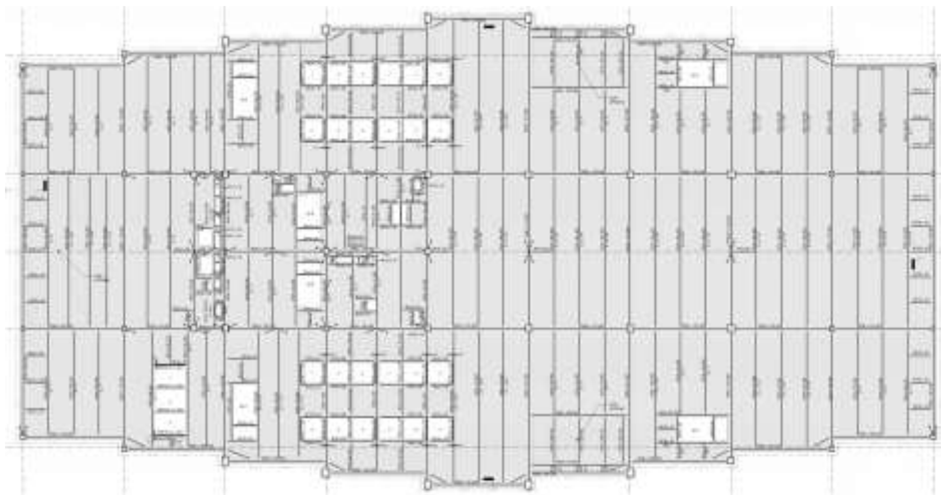
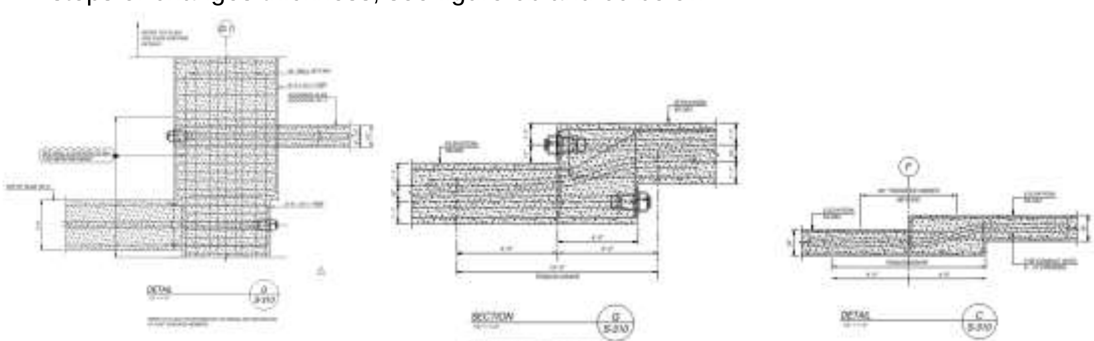


Figure 5. Typical Framing Plan

The Ground floor consists of a reinforced concrete slab that is 3’-0” thick west of gridline D-1 and a 16” thick east of gridline D-1. The slab also has four groups of PT tendons that occur at gridlines T1, T3, T7, and T9. For the tendon groups along T1 and T9, the tendons stop and start at the 6’-4” thick pier running north-south at gridline D-1, see figure 6a below). The tendons at gridlines T3 and T7 are continuous but are draped at the locations where the slab steps or changes thickness, see figure 6b and 6c below.

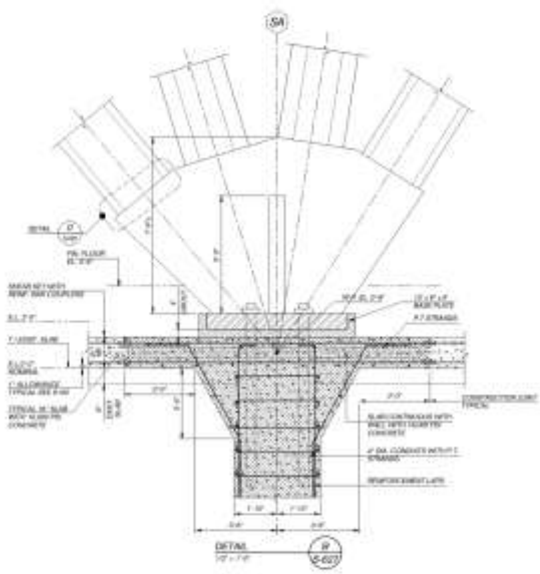


5a. **5b.** **5c.**

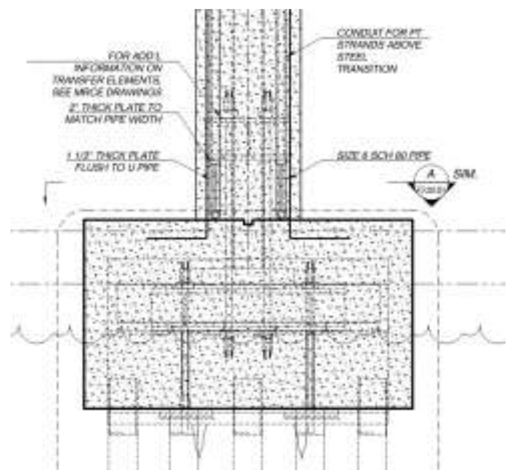
Figure 1. Typical Ground Slab Sections

2.3 FOUNDATION SYSTEM

The typical building foundation consists of caisson caps supported by caisson groups. The caisson caps receive the shear walls, which in turn receive the V and fan columns. Some of the V and fan columns along grid T1 and T9 have vertical PT that starts at the base plate, continues through the shears walls and is anchored to or around steel framing connected directly to the caissons, see Fig. 7. The vertical PT anchors the columns that see uplift forces directly to the foundation.



7a. Top of Wall / Base of Column



7b. Bottom of Wall / Top of Pile cap

Figure 7. Typical Below Grade Shear Wall Sections

C. FINDINGS AND COMMENTS

1.0 BUILDING CODES

Based on the General Notes on S-001, the structural design was conducted according to the following building codes:
2014 Edition of the New York City Building Code
ASCE-7 (2010), Minimum Design Loads for Buildings and other Structures
ASCE-7 (2005), Minimum Design Loads for Buildings and other Structures
AISC 360 (2005), Specification for Structural Steel Buildings
ACI-318 (2011), Building code requirements for Reinforced Concrete
AWS D1.1 (2004), Structural Welding Code
ASTM Standards

The building codes listed in the Structural Design Criteria is consistent with those noted above. TT finds these buildings codes acceptable and appropriate for this project.

2.0 MATERIAL PROPERTIES

The material properties noted in the General Notes on FO-800 and FO-001 for the major structural elements are noted below.

Structural Steel:	ASTM A992
Structural Steel Denoted “HW”:	ASTM A572 or A913, Grade 65
HSS Steel:	ASTM A500, Grade B
Steel Plates:	ASTM A572-50 for t ≤ 4”, ASTM A36 for t > 4”
Connection Steel	ASTM A572-50
Mini-Caissons:	10,000 psi, 80 ksi steel casing
Mini-Caisson Reinforcement:	75 ksi rebar or 150 ksi threaded rod
Caisson Caps:	12,000 psi
Shear Walls:	16,000 psi
Foundation Walls	10,000 psi
Ground Transfer Slab:	10,000 psi
Slabs and Beams:	10,000 psi
Typical Reinforcement:	ASTM A615, Grade 60 and Grade 80
Shear Wall Reinforcement:	ASTM A615, Grade 80 and Grade 97

3.0 STRUCTURAL LOADING

3.1 GRAVITY LOADS

The gravity loading consists of the member self-weight, the superimposed dead load (floor finish, partitions, ceiling & hung mechanical), and live load. The gravity design loads are shown in the loading schedule on S-701 of the Severud 50% DD Set.

Table 1. Construction Dead Loads per S-701

SLAB CONSTRUCTION	LOAD (PSF)	TT COMMENTS
10” NWC Slab	125	
12” NWC Slab	150	
16” NWC Slab	200	
36” NWC Slab	450	
Type A	50	2 ½” NWC on 3” deck (typ)
Type B, D, E, G, H, L	75	4 ½” NWC on 3” deck (typ)
Type C, F	75+40	4 ½” NWC on 3” deck (typ) + 3” topping
Type I, J	75+40 or 50	5” NWC on 3” deck (typ) + 3” or 4” topping
Type K	50+50	2 ½” NWC on 3” deck (typ) + Varied topping (4” average).

Table 2. Superimposed Dead Loads per S-701

ITEM	LOAD (PSF)	TT COMMENTS
Floor Finish - 8” Raised Floor	15	At Office Floors
Floor Finish 24” Raised Floor	35	At Trading Floors
Hung Ceiling / Mechanical	8	Trading Floor, Office Floor
Hung Ceiling / Mechanical	12	Conference, Dining, Amenity
Hung Ceiling / Mechanical	15	Mechanical/BOH
Hung Ceiling / Mechanical	20	Lobbies
Hung Ceiling / Mechanical	50	Subcellar 1, Kitchen, Tranfer Mezz., Office above mechanical, Amenity
Partitions	12	Office Floor
Miscellaneous - Trading	10	
Miscellaneous - TMD	1,000 kips	
Miscellaneous - BMU	TBD	
Miscellaneous - Mechanical	25	20” Insulation – 2ND Floor only
Miscellaneous - Planters	TBD	

Table 3. Live Loads per S-701

AREA	LIVE LOAD (PSF)	TT COMMENTS
Core	100	Treat as Lobby Space
Subcellar	300	
Cellar	100	Reasonable assumption
Ground Floor Over Cellar	300	Appropriate for Staging
Lobby	100	
Mechanical – Typical	150	
Trading	100	
Kitchen	100	Same as Dining/Restaurant per code
Dining	100	Per Code
Transfer	100	
Amenity	100	
Conference – Typical	100	Treat as Assembly Space
Conference – Roof	300	Landscaping, Reasonable Assumption
Office - Typical	50	Office Load per Code
Office - Executive	100	50 per Code, Reasonably Conservative
Client Center	100	Reasonable Assumption
Skybar	100	Same as Dining/Restaurant per code
TMD	150	Treat as Mechanical
Top Roof	100	20 psf Req'd for Roofs
BMU – Roof	TBD	

Figure 2. Gravity Design Loads Per General Notes

TT found the gravity loads to be acceptable and in conformance with the 2014 NYC Building Code.

3.2 WIND LOADS

The wind loads for the below grade and foundation design are based on the following parameters per ASCE 7-05 and the New York City Building Code:

Design Wind Speed, V	98 mph
Occupancy Category	III
Equivalent Wind Exposure	B
Importance Factor	1.15
Wind Base Shear, V (NS/EW)	16000 kips / 9400 kips (Per Drawing S-701)

Wind Base Shear, V (NS/EW)	9400 kips / 7900 kips (Per Wind Tunnel)

TT found the wind load parameters shown by the EOR are consistent with the Building Code.

The wind loads were determined for this project through wind tunnel testing conducted by RWDI. Their findings and recommendations were issued in a report for Wind Induced Structural Responses dated November 14, 2019. The wind tunnel report provides Effective Static Floor-by-Floor Wind loads for Fx, Fy and Mz. In turn, these loads are to be used with load factors given in 24 load combinations. These loads are to be applied per the ASCE7-05 load combinations.

TT compared the wind tunnel loads with 80% of the ASCE7-05 code wind load and found Severud appropriately increased the applied static loads to limit base shear to no less than 80% of the code calculated wind loads.

3.3 SEISMIC LOADS

The seismic loads are in compliance with Chapter 16 of the NYC Building Code using the following seismic parameters:

Occupancy Category	III
Importance Factor, Ie	1.25
S_S	0.281g
S_1	0.073g
Site Class	B
Sds	0.187g
S_{D1}	0.049g
Design Category	B
Seismic Force Resisting System	Steel System Not Specifically Detailed For Seismic Resistance
Response Modification Factor, R	3
Cs	0.078g
Seismic Base Shear, V	5100 kips

TT performed independent seismic calculations and found the seismic base shear to be 5300 kips based on the equivalent lateral force. TT's value is within 4% of Severud, slight differences can be attributed to the seismic weight assumptions. Considering the seismic base shear is less than the wind base shear, the latter is used for the below grade checks.

3.4 LOAD COMBINATIONS

The load combinations listed in the Design Criteria are per ASCE7-10, which is consistent with the General Notes.

A summary of the load combinations used is shown below.

Ultimate (Strength) Design
1.4D
1.2D+1.6L+0.5(Lr or S or R)
1.2D+1.6(Lr or S or R)+(f1L or 0.8W)
1.2D+1.6W+f1L+0.5(Lr or S or R)
1.2D+1.0E+f1L+f2S
0.9D+1.6W
0.9D+1.0E

The load factor on L in combinations 3, 4 and 5 is permitted to equal 0.5 for all occupancies in which Live load is less than or equal to 100 psf.

Allowable Strength (Service) Design
D
D+L
D+L+(Lr or S or R)
D+0.75L+0.75(Lr or S or R)
D+(W or 0.7E)
D+0.75L+0.75(W or 0.7E)+0.75(Lr or S or R)
0.6D+W
0.6D+0.7E

4.0 LOAD PATH REVIEW

The interior columns collect the loads from the superstructure floor by floor and transfer onto the two full story deep plate girders at the 3rd floor, spanning in an east-west direction. The load is then transferred thorough the plate girders into the interior "V-shape" columns along grids T3 and T7. Full fitted stiffeners within the plate girder help to provide a direct load path to the the "V" columns below.

The perimeter columns have a more direct load path when compared to the interior columns. At the 3rd floor, the perimeter columns transition directly into the exterior "fan-shape" columns through a solid steel node. The load is then transferred directly into the below grade shear walls. The resulting lateral (kick) forces due to the sloped columns are resisted by axial tie members in the third floor and by a PT slab at the ground floor. Finally, the column loads on the shear walls travel through the wall to the top of the caisson caps, and then to the mini-caissons and into the bedrock.

The lateral load path is similar to the "kick" forces resulting from the gravity loads. Lateral shear is transferred as axial force through the sloping columns into the thick PT slab in the ground floor. This slab distributes the in-plane forces as shear to the shear walls below. Shear in the north-south direction is resisted by the walls oriented in the same direction, which then goes into the mini-caisson caps below the walls. The east-west shear is transferred through the ground floor slab to the west side of the building (west of grid D-1) where walls oriented east-west in plan then transfer the shear from the ground floor slab to the shear walls to spread footings and mini caissons.

TT has reviewed this load path and found it to be complete.

5.0 BELOW GRADE MEMBER DESIGN CHECK

The below grade design check in this report is limited to the shear walls, slabs west of Grid D.1, and the ground floor slab. Please refer to the Langan peer review report for review of the caissons, caisson caps and spread footings.

5.1 BELOW GRADE SHEAR WALLS SUPPORTING TOWER

The tower is directly supported on the shear walls on Grids D.1, F, SA, H, and K. See Fig. 8 for plan locations of these walls highlighted in red. The walls along these grids vary in thickness from ~31" at some locations to 48" at the lowest levels. The walls are reinforced with Grade 80 and Grade 97 ksi reinforcing bars. The concrete strength of all walls is 16ksi. All vertical rebar is mechanically coupled to limit congestion, comply with code provisions, and facilitate vertical reinforcing percentages in excess of 4% at local zones. In addition to supporting the entire load of the tower above, the walls also resist the N-S lateral forces. The area highlighted in blue in Fig.8 consists of shear walls and load bearing walls supporting 3 levels of framed slabs west of the D.1 Grid. This concrete box also provides the lateral support of the tower in the E-W direction.

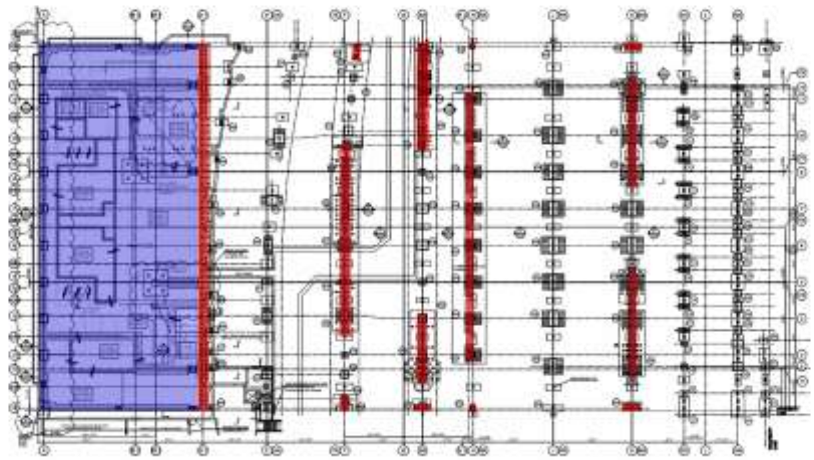


Figure 8. Shear Wall Plan

A sample of the walls were analyzed for the ultimate loads resulting from the load combinations presented in section 3.4. Column loads acting on the walls were provided by Severud, and checked against loads calculated via independent column load takedowns, in addition to column loads exported from TT's independent ETABS model. See Fig. 9 for a summary of the column loads acting on the shear walls.

270 PARK AVENUE - PROJECT GREYHOUND										
COLUMN LOADS (November 19) - 1388' Scheme including Wind Tunnel										
INTERIOR COLUMNS										
SERVICE LOADS (KIPS)										
MARK	AXIAL				SHEAR					
	DL	LL	WL-Max	WL-Min	DL		LL		WL	
					x	y	x	y	x	y
F-T3, F-T7	46880	12200	9230	-8764	100	0	100	0	600	0
H-T3, H-T7	41500	12400	7500	-6800	300	0	300	0	1100	0
K-T3, K-T7	34000	10000	10000	-9530	1000	0	400	0	1600	0
D.1-T3, D.1-T7	25000	6450	9550	-9800	400	0	100	0	1300	0
B-T3, B-T7	11200	3300	6330	-6330	0	0	0	0	0	0
LOAD COMBINATION - SERVICE LOADS										
		D+L	D+W	D+.75(L+W)	.6D+W		Max	Min		
F-T3, F-T7		59080	56110	62953	19364		63,000	-		
H-T3, H-T7		53900	49000	56425	18100		56,500	-		
K-T3, K-T7		44000	44000	49000	10870		49,000	-		
D.1-T3, D.1-T7		31450	34550	37000	5200		37,000	-		
B-T3, B-T7		14500	17530	18423	390		18,500	-	<use -2000	
EXTERIOR COLUMNS										
SERVICE LOADS (KIPS)										
MARK	AXIAL				SHEAR					
	DL	LL	WL-Max	WL-Min	DL		LL		WL	
					x	y	x	y	x	y
SA-T1, SA-T9	52200	12200	22200	-21500	900	2600	200	400	800	2700
K-T1, K-T9	31500	8000	16600	-15560	2200	800	800	200	1700	1600
D.1,T1 - D.1,T9	33850	6850	20000	-22300	3500	400	800	200	1800	3100
LOADS COMBINATION - SERVICE LOADS										
		D+L	D+W	D+.75(L+W)	.6D+W		Max	Min		
SA-T1, SA-T9		64400	74400	78000	9820		78,000	-	<use -2000	
K-T1, K-T9		39500	48100	49950	3340		50,000	-	<use -2000	
D.1,T1 - D.1,T9		40700	53850	53988	-1990		54,000	-1990	<use -3400	

Figure 9. Severud Column Loads

For review of the shear wall design, TT utilized both hand calculations and finite element analysis to confirm the shear walls would not exceed allowable stress limits under the full load of the tower. TT also reviewed the drawings for constructability. TT utilized ETABS to model sections of the shear walls to verify stresses and force distribution, see Fig. 10.

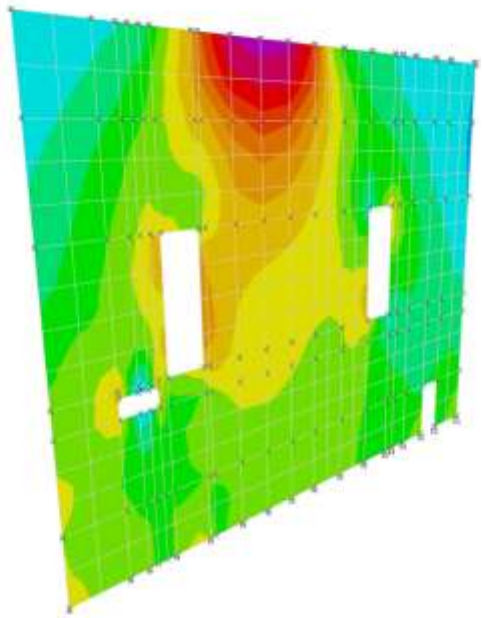


Figure 10. SA Grid Finite Element Model

TT verified a sample of the wall rebar design utilizing the finite element analysis and hand calculations. Several representative calculations have been included in Appendix 2. The sample calculations include checks at Grids SA and H. The required reinforcing per TT design checks was compared to the provided reinforcement shown in the shear wall elevations on drawings FO-300 FO-304 through in the IFC Bulletin 6 Set dated April 3rd, 2020. Overall, the horizontal and vertical reinforcement in the shear walls was determined to be acceptable.

To address uplift in the shear walls at the column locations, Severud introduced post tensioning (compression force) to anchor the columns directly to the foundation. TT has verified the number of tendons provided has sufficient capacity to resist the tensile loads.

5.2 BELOW GRADE SHEAR WALLS WEST OF GRID D.1

The below grade walls West of Grid D.1 support the framed cellar slabs in that zone in addition to carrying the E-W tower lateral loads to the foundation. TT has reviewed the wall sizes and confirmed they can support the maximum E-W lateral loads in addition to gravity loads from the cellar slabs.

The below grade walls in this area do not expierence weak axis lateral loading, as they will be built inside of an existing foundation structure.

5.3 FRAMED SLABS

TT performed hand calculations and built a finite element analysis model of the Subcellar 1 slab to confirm thickness and reinforcing. Fig. 11 provides a 3D view of the slab analysis. TT determined the slab thickness and reinforcing to be adequate for the indicated loading.

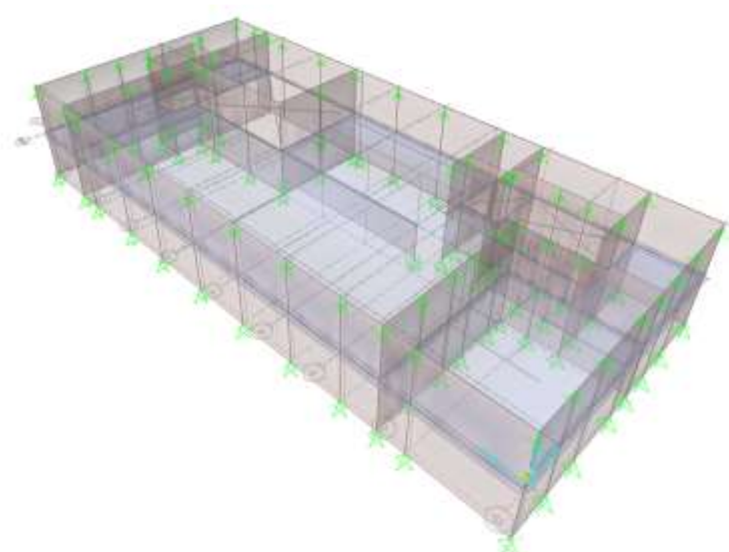


Figure 11. SAFE Model of Subcellar Slab

5.4 GROUND FLOOR SLAB

The ground floor slab provides lateral stability to the tower by transferring the east-west lateral loads from the columns to the shear walls west of Grid D.1. The ground floor slab also supports the lobby level gravity loads. West of Grid D.1 the slab is a 36” thick two-way slab supported on the below grade shear walls. East of Grid D.1 the slab is a two-way 16” thick slab supported on a mix of shear walls and existing columns that are to be left in place upon completion of the demolition of the existing tower. The typical bay in this zone is ~43’ x 20’. TT determined that the slab thickness and specified reinforcing is adequate for the indicated loading for a typical bay.

6.0 SUPERSTRUCTURE MEMBER DESIGN CHECK

6.1 FLOOR FRAMING CHECK

TT has checked floor framing for typical composite floors 20, 30, 40, 57 and 61 and has that concluded that floor framing has been designed to NYC Building Code and AISC 360 requirements. Floor framing checks where done utilizing RAM Structural Systems models. In general, the floor framing is adequate for both strength and serviceability parameters.

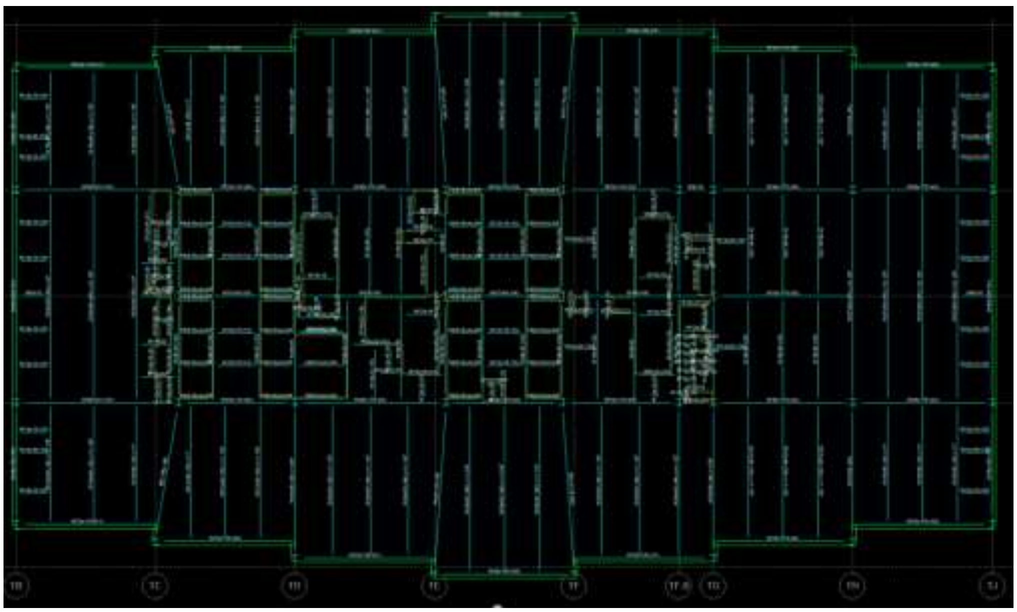


Figure 12. TT RAM Model of Level 20

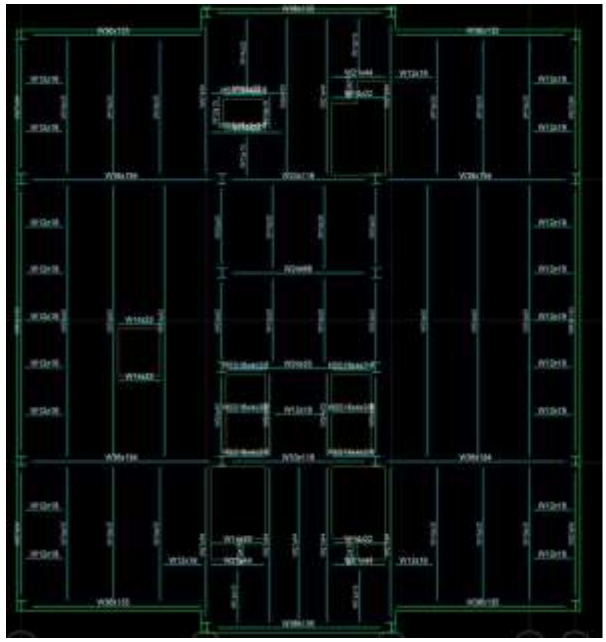


Figure 13. TT RAM Model of Level 57

6.2 VIBRATION CHECK

TT's check of the typical trading floor (Level 7) and typical office floor concluded that the floor framing vibration accelerations are within the acceptable levels described in AISC Design Guide 11 (2016). See Appendix 6 for sample calculation check.

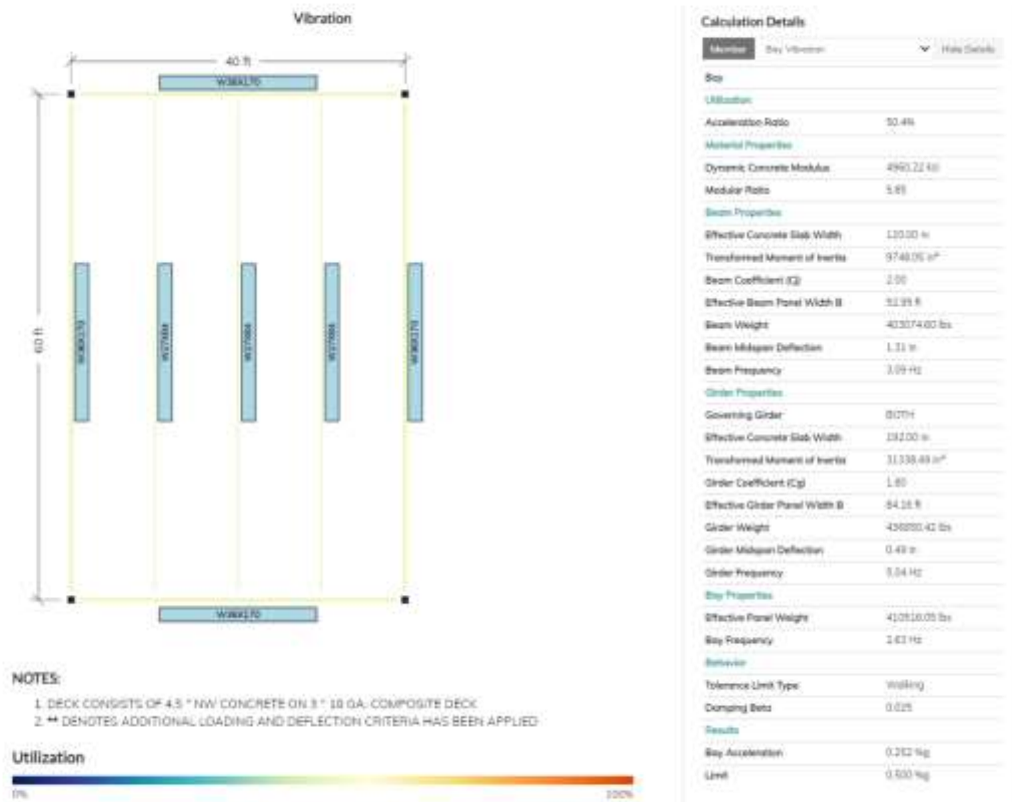


Figure 14. TT Vibration Analysis of Typ. Bay (Level 7)

6.3 COLUMN CHECK

TT performed column checks on a sample of columns and verified that they have sufficient capacity for loads and load combinations as required by NYC Building Code 2014. TT calculated column reactions at both the foundation and tabletop and concluded they match those calculated by Severud and listed in the Column Schedule on sheets S-601 through S-616.

6.4 TABLETOP CHECK

The tabletop consists of the superstructure in between the ground floor and Level 3. This structure serves to transfer the entire gravity and lateral load of the tower onto 16 points. The structure consists of two 25' deep plate girders running the entire E-W width of the tower, braced by a series of transfer trusses and the 2nd and 3rd floor diaphragms. The interior tower columns transfer on these plate girders which than span to "V" columns. The exterior tower columns transfer to fan columns along the perimeter. As previously discussed, TT created an independent model of the entire tower structure to verify load path and member capacity. This included detailed modeling and review of the tabletop structure as seen in Fig. 15.

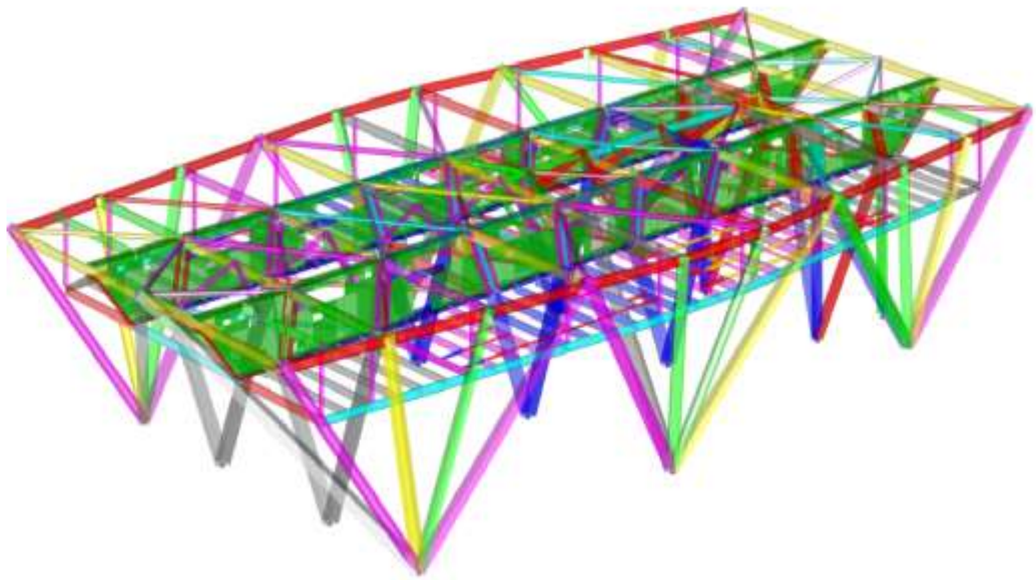


Figure 15. TT Independent Tabletop Analysis Model

TT modeled the plate girders as they appear in the 50% DD on sheet S-550. TT verified stresses in members did not exceed allowable limits for design load combinations. See Fig. 16 for a sample of the FEA output.

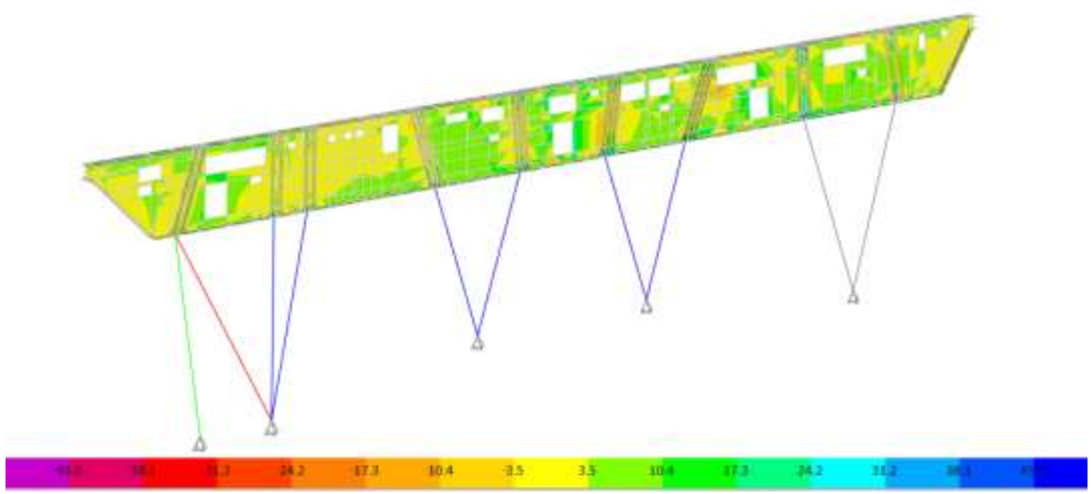


Figure 16. Plate Girder Stresses

TT also evaluated the 2^{nd} and 3^{rd} floor diaphragm framing, see Fig. 17. TT verified that the structural members selected by the EOR had sufficient capacity for both the loads listed on the structural documents in addition to the loads seen in TT's independent analysis model. TT verified that the connection forces provided in the design documents met or exceeded those in TT's independent analysis model.

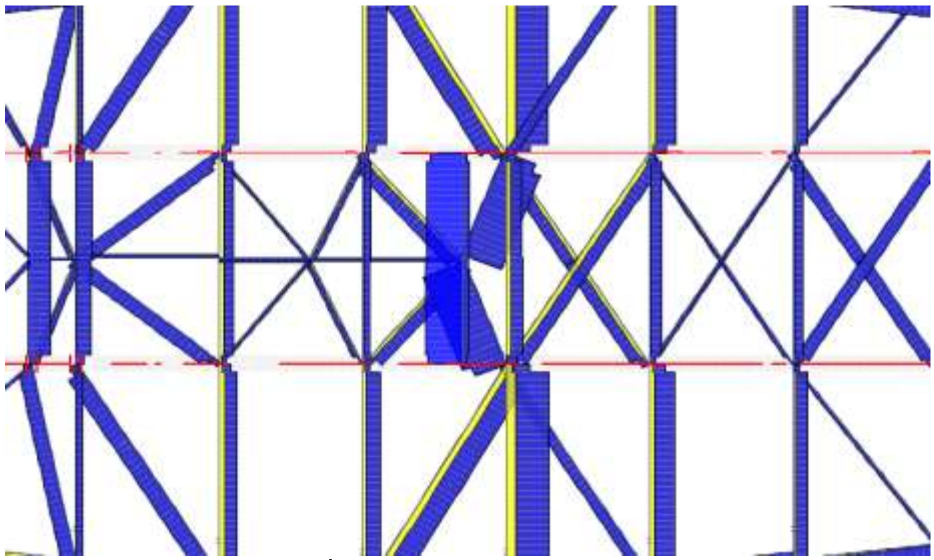


Figure 17. 3^{rd} Floor Diaphragm Axial Forces

Some minor discrepancies regarding the magnitude of connection forces were resolved in coordination with Severud Associates, see Appendix 7 for TT and SA discussion and resolution.

6.5 BRACED CORE CHECK

We have checked a limited number of braced frames based on our independent analysis model and confirmed their design. We reviewed members throughout the tower and found some members of the braced core to be undersized. We have resolved our discrepancies with the EOR and confirmed their designs will be updated as the building design progresses. We have confirmed these deign updates in Bulletin 1, Bulletin 2 and Milestone 1 packages and will continue to review future bulletins to ensure updates to the braced frame design are carried out.

6.6 OUTRIGGER TRUSS CHECK

The tower structure utilizes outriggers at several floors to stiffen the tower to meet deflection criteria. The outriggers are located on floors 11, 18, 29, 38, and 53. TT checked a sample of the outrigger trusses and confirmed they have sufficient capacities for loading criteria per the NYC building Code. In general the outrigger member sizes on the upper floors tended to be governed by stiffness requirements. See Fig. 18 for the axial force distribution in the Level 38 outrigger braces under N-S wind load. Fig. 19 provides a member capacity check of outrigger truss TR38TG. TT also reviewed representative outrigger sizes in regards to integrity loads. See Section 6.7 for additional information.

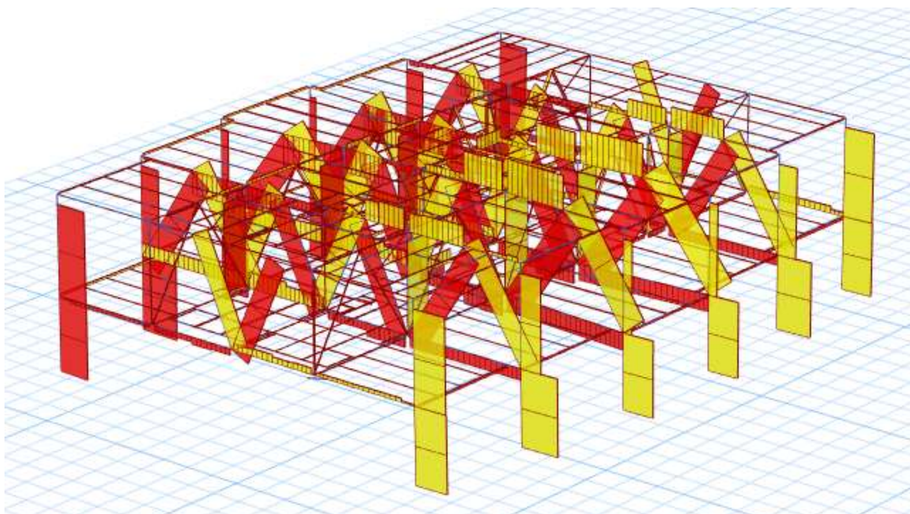


Figure 18. Level 38 Outrigger Truss Axial Forces

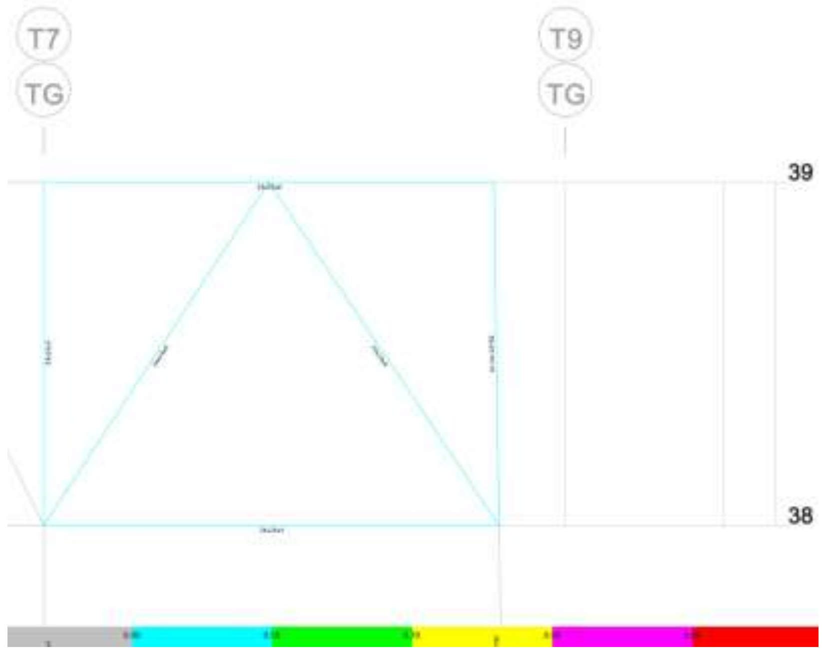


Figure 19. Level 38 TR38TG Member Capacity Check

6.7 STRUCTURAL INTEGRITY CHECK

Section 1616 of the NYC Building Code requires integrity checks to be performed on “key elements of the building”, if a structure meets certain criteria per Section 1616.1. This structure falls under this code requirement due to a building aspect ratio greater than 7 and the tower’s height exceeding 600ft.

Key elements of the structural system, including its connections, are elements which when lost result in more that local collapse or whose tributary area exceeds 3000 square feet on a single level. Additionally elements that brace a key element, which result in failure of the key element are also considered as key elements.

Per Section 1616.6, “Where key elements are present in a structure, the structure shall be designed to account for their potential loss one at a time by the alternate load path method or by the specific local resistance method.” Depending on the location of the key element Severud utilizes both these methods in this design. All key elements are identified on sheets S- 752 to S-757.

Our review for representative key elements below Level 3 indicates that they meet the integrity requirements for Key Elements via the “specific load resistance method”, where key elements shall be designed using specific local loads, as 1616.7 NYC Building Code.

1. Each compression element shall be designed for a concentrated load equal to 2 percent of its axial load but not less than 15 kips, applied at midspan in any direction, perpendicular to its longitudinal axis. This load shall be applied in combination with the full dead load and 50 percent of the live load in the compression element."

   Our sample check of the columns, beams and braces in compression below level 3 confirmed they meet this requirement.

2. Each bending element shall be designed for the combination of the principal acting moments plus an additional moment, equal to 10 percent of the principal acting moment applied in the perpendicular plane."

   TT reviewed the plate girder elements and found the structure meets this requirement as the top and bottom flanges of the girder are effectively fully braced along its length.

3. Connections of each tension element shall be designed to develop the smaller of the ultimate tension capacity of the member or three times the force in the member."

   It is understood through communication with the EOR that the connection provision 3 above is being followed for key element connection design. Severud's key element plan drawings (S-752 through S-757) in combination with their general notes (Notes S.26, S.27 and S.28 on S-001) alert the detailer to connection design requirements.

4. All structural elements shall be designed for a reversal of load. The reversed load shall be equal to 10 percent of the design load used in sizing the member."

   Our sample check of beams, columns and braces indicated the members are adequately sized for this requirement.

Above the 3rd floor Severud utilizes the outriggers and perimeter diamond bracing in combination with vierendeel moment frame action to provide an alternate load a path should a column be removed from the building structure. This alternate load path system accounts for a column element loss one at a time in order to satisfy the alternate load path method.

TT reviewed a sample of the outriggers and diamond bracing to insure it could meet the required integrity load combination (1.0D+.5L+.33W) should a key element no longer be capable of carrying load. Images of the FEA models of truss TR11TB and TR11TE are provided in Fig. 20 and Fig. 21 respectively. TT verified the representative number of elements can resist the integrity load combination.

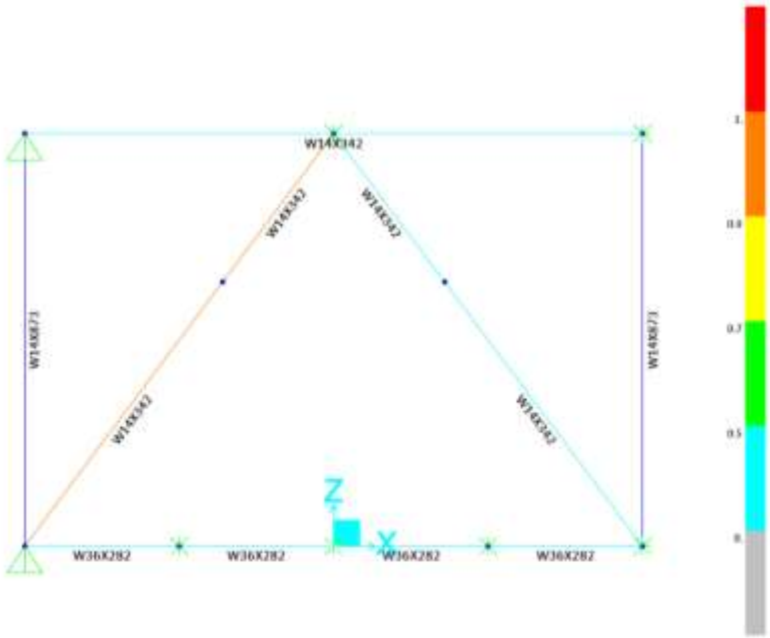


Figure 20: FEA Model of Truss TR11B (Member Capacity Check)

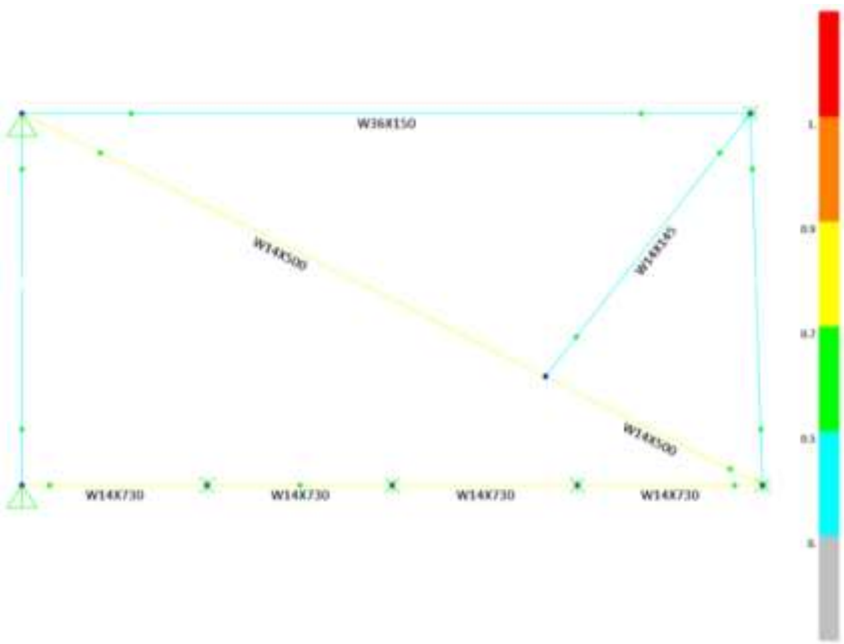


Figure 21: FEA Model of Truss TR11TE (Member Capacity Check)

7.0 DYNAMIC BEHAVIOR AND SERVICEABILITY CHECK

7.1 DYNAMIC BEHAVIOR

The building periods for the first three modes as shown in the wind tunnel test report are given in the table below. Based on TT's independent analysis model, the non-iterative P-delta eigenvalue analysis was performed. The periods and percent differences for the first three modes are shown in the table below. Slight differences can be contributed to mass and modeling assumptions. TT finds the values in general agreement.

Table 4. Building Period Comparison

Building Period (Seconds) Comparison			
Mode	**Severud**	**Thornton Tomasetti**	**Percent Difference**
1	6.15	6.10	0.90%
2	6.14	5.61	8.60%
3	4.41	4.17	5.60%

7.2 SERVICEABILIY CHECKS

As a matter of standard practice, the wind deflection limit is typically set to H/400 for a storm with a 10 year return period for standard buildings, where H is the elevation of the floor at which the deflection is measured. For taller buildings it is typical to be more stringent and use a 50 yr. return period. Per TT's independent FE model, TT found a maximum overall wind deflection at the roof of 26.9" in the North/South direction for the 50 yr. return period. The maximum allowable deflection at this height is 41.6", so the structural design, to the degree that TT was able to match the intended structural properties, meets industry standard criteria at this stage of the design.

TT also reviewed wind (10 year) and seismic inter-story drifts. For wind drift TT utilized an H/400 limit. For seismic TT reviewed the inter-story drift due to seismic ($\delta = (C_d/I_e)^* \delta_e$) for a limit of 0.02h as per the Building Code. TT did not identify any locations where drift criteria was exceeded.

TT notes that the building accelerations have also been checked by the wind tunnel consultant for appropriate acceleration criteria. The project team has recommended use of a tuned mass damper to achieve a residential acceleration criteria in lieu of standard of office criteria, thus holding the building to a stricter limit.

D. DOCUMENTS RECEIVED

TT used as a basis of this review the Architectural drawings, Structural drawings, and the calculation documents listed below.

	Document Name	Document Date
1	Severud 90% MOU Foundation Set	12/09/2019
2	MRCE Geotechnical Interpretative Report	06/27/2019
3	MRCE Geotechnical Data Report	06/27/2019
4	100% SD Structural & Architectural	08/22/2019
5	Structural Bid Set	09/26/2019
6	RWDI Preliminary Report	11/15/2019
7	IFC Bulletin 3	12/12/2019
8	IFC Bulletin 4	12/19/2019
9	IFC Bulletin 5	02/25/2020
10	IFC Bulletin 6	04/03/2020
11	50% DD Set Structural, MEP, & Architectural	02/18/2020
12	Structural Steel Bulletin #1 (Mill Order – Seq. 1)	04/03/2020
13	Structural Steel Bulletin #2 (Mill Order – Seq. 2a)	05/08/2020
14	IFC Bulletin 8	05/26/2020
15	Milestone 1	07/24/2020

Note: See Appendix 3 for below grade and Appendix 5 for the superstructure drawing list.

E. APPENDICES

Appendix 1 - Column Loads Provided By Severud

270 PARK AVENUE - PROJECT GREYHOUND

COLUMN LOADS (November 19) - 1388' Scheme including Wind Tunnel

INTERIOR COLUMNS

SERVICE LOADS (KIPS)

MARK	AXIAL				SHEAR					
	DL	LL	WL-Max	WL-Min	DL		LL		WL	
					x	y	x	y	x	y
F-T3, F-T7	46880	12200	9230	-8764	100	0	100	0	600	0
H-T3, H-T7	41500	12400	7500	-6800	300	0	300	0	1100	0
K-T3, K-T7	34000	10000	10000	-9530	1000	0	400	0	1600	0
D.1-T3, D.1-T7	25000	6450	9550	-9800	400	0	100	0	1300	0
B-T3, B-T7	11200	3300	6330	-6330	0	0	0	0	0	0

LOAD COMBINATION - SERVICE LOADS

		D+L	D+W	D+.75(L+W)	.6D+W		Max	Min		
F-T3, F-T7		59080	56110	62953	19364		63,000	-		
H-T3, H-T7		53900	49000	56425	18100		56,500	-		
K-T3, K-T7		44000	44000	49000	10870		49,000	-		
D.1-T3, D.1-T7		31450	34550	37000	5200		37,000	-		
B-T3, B-T7		14500	17530	18423	390		18,500	-	<use -2000	

EXTERIOR COLUMNS

SERVICE LOADS (KIPS)

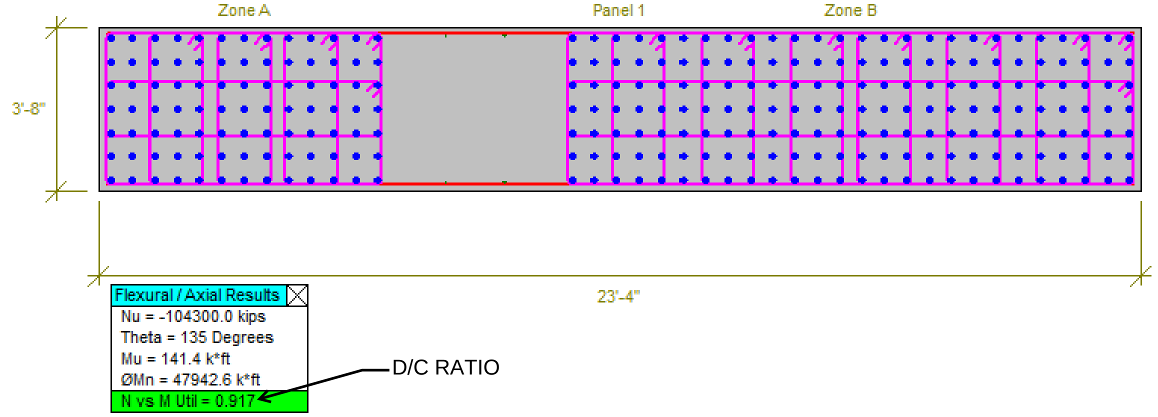
MARK	AXIAL				SHEAR					
	DL	LL	WL-Max	WL-Min	DL		LL		WL	
					x	y	x	y	x	y
SA-T1, SA-T9	52200	12200	22200	-21500	900	2600	200	400	800	2700
K-T1, K-T9	31500	8000	16600	-15560	2200	800	800	200	1700	1600
D.1,T1 - D.1,T9	33850	6850	20000	-22300	3500	400	800	200	1800	3100

LOADS COMBINATION - SERVICE LOADS

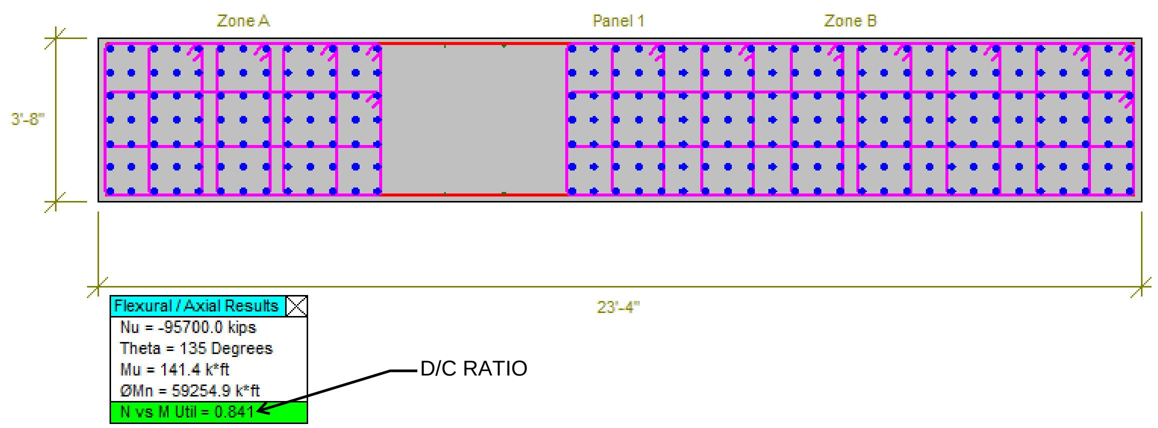
		D+L	D+W	D+.75(L+W)	.6D+W		Max	Min		
SA-T1, SA-T9		64400	74400	78000	9820		78,000	-	<use -2000	
K-T1, K-T9		39500	48100	49950	3340		50,000	-	<use -2000	
D.1,T1 - D.1,T9		40700	53850	53988	-1990		54,000	-1990	<use -3400	

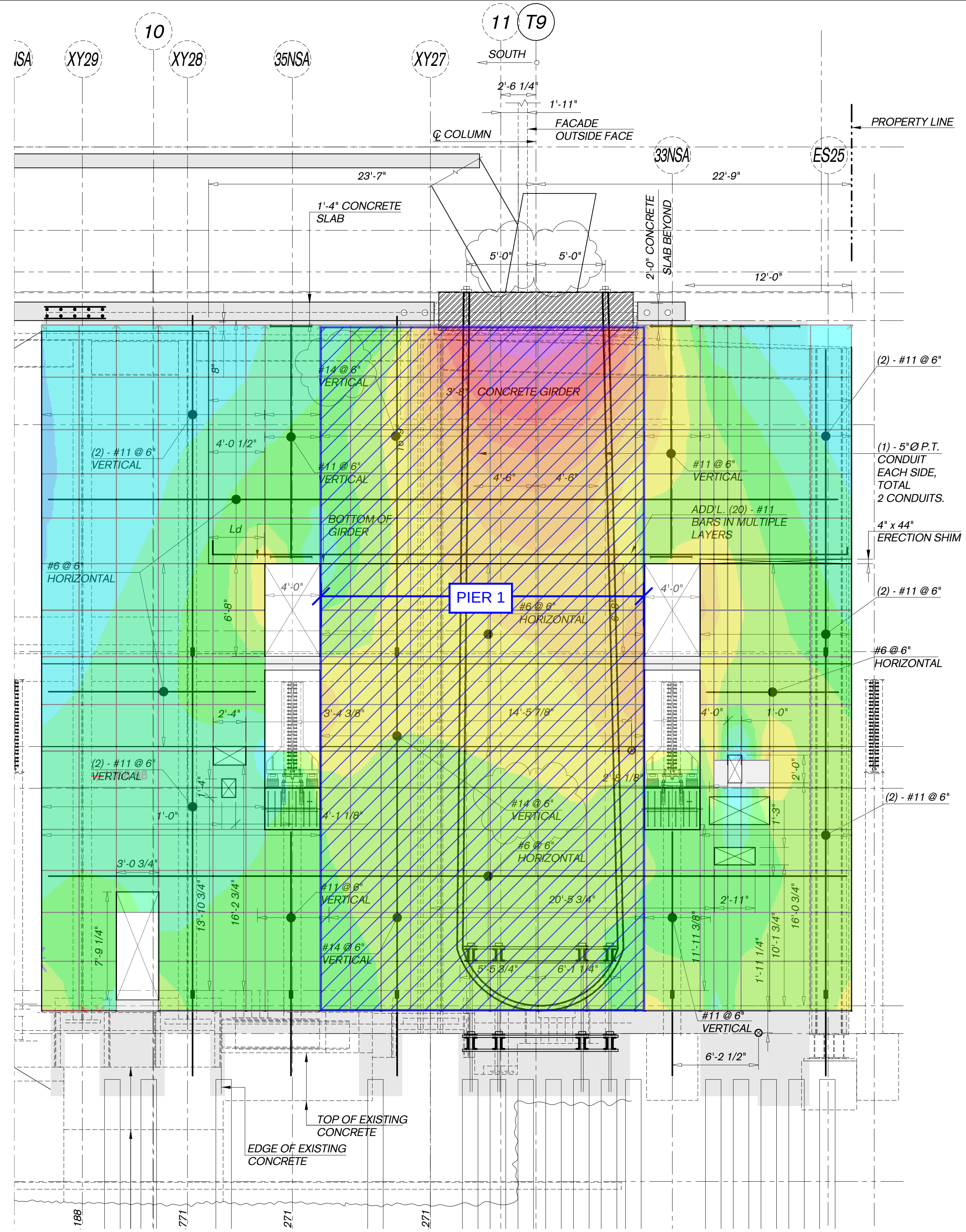
Appendix 2 - Shear Wall Calculations

CONSERVATIVELY ASSUMING ALL OF THE LOAD IS RESISTED BY PIER 1 ONLY, TT CALCULATED THE WALL CAPACITY, SEE PRINT OUT FOR FULL CALCULATION (PAGES A2-2 to A2-4)



ASSUMING LOAD DISTRIBUTION PER TT FE ANALYSIS, WALL WAS CHECKED FOR PORTION OF TOTAL LOAD SEEN BY PIER 1





SHEAR WALL ELEVATION AT COLUMN LINE SA - (R.R COLUMN LINE SA)

S-CONCRETE 2018.1.1 (c) S-FRAME Software Inc.

File Name: C:\ ... Cache\Content.Outlook\53HXPTCY\T9-SA.SCO

Summary

Status	Reviewed-OK
Maximum	1.000
V (shear) Util	0.000
N vs M Util	0.917

Section Name
Concrete Section

Consultant
Thornton Tomasetti, Inc.

American Building Standards
ACI 318-14, "Building Code Requirements for Structural Concrete"
ACI 318R-14, "Commentary for ACI 318-14"

Design Aids, Manuals, and Handbooks
The Reinforced Concrete Design Handbook, A Companion to ACI 318-14
"ACI Detailing Manual - 1994", ACI Committee 315, American Concrete Institute, 1994
"Manual of Standard Practice", Concrete Reinforcing Steel Institute, 2003

Section Dimensions
I-Shape
L1 = 280.0 in
T1 = 44.0 in

Material Properties
fc' = 16000 psi
fy (panel vert) = 97.0 ksi
fy (panel horz) = 80.0 ksi
fy (zone vert) = 97.0 ksi
fy (zone horz) = 80.0 ksi
Wc = 150 pcf
Ws = 490 pcf
Poisson's Ratio = 0.2
hagg = 0.79 in
Es = 29000 ksi
Ec = 7655 ksi
Gc = 3190 ksi
fr = 949 psi

Gross Properties
Zbar = 0.0 in
Ybar = 0.0 in
Ag = 12320 sq.in.
Ig (y-y) = 1987.6xE3 in4
Ig (z-z) = 80491xE3 in4
Ashear (Y) = 10267 sq.in.
Ashear (Z) = 10267 sq.in.
Jg = 7163.1xE3 in4

Effective Properties
Ae = 12320 sq.in.
Ie (y-y) = 1987.6xE3 in4
Ie (z-z) = 80491xE3 in4
Ase (Y) = 10267 sq.in.
Ase (Z) = 10267 sq.in.
Je = 7163.1xE3 in4

Quantities (approx.)
Concrete = 12179 lb/ft
Steel = 2464.7 lb/ft
Primary = 2093.1 lb/ft
Secondary = 371.6 lb/ft

Panel 1
4-#4 @ 18.0" V.E.F
#6 @ 6.0" H.E.F.

Zone A
91-#14 Vert
#4 Ties @ 6.0 in
Mechanical Splice
As = 204.75 sq.in.
Y: 13@6.0"
Z: 7@6.28"

Zone B
182-#14 Vert
#4 Ties @ 6.0 in
Mechanical Splice
As = 409.5 sq.in.
Y: 26@6.0"
Z: 7@6.28"

Factored Design Loads

Load Case/Combo	N (kips)	T (k*ft)	Vz (kips)	My (k*ft)	Vy (kips)	Mz (k*ft)	Mres (k*ft)	Theta
1 (W)	-104300.0	0.0	0.0	100.0	0.0	100.0	141.4	135º

N vs M Results

GLC	1
Status	Acceptable
Utilization	0.917
Maximum	1.000
Theta	135º

Axial Utilization
Nu = -104300.0 kips
ØNn (max) = -113800.5 kips
Utilization = 0.917

Moment Utilization
Mu = 141.4 k*ft
ØMn = 47747.9 k*ft
Utilization = 0.003

Mn = 125232.0 k*
Mp = 125357.3 k*

Panel 1 Reinforcing

#4 @ 18.0" V.E.F	Vert Steel Ratio	Vert Bar Spacing	Number of Curtains
	Rho = 0.00051	S = 18.00 in	Curtains Specified = 2
	Rho (min) = 0.00120	S (min) = 2.50 in	Curtains Required = 2
	Rho (max) = 0.01000	S (max) = 18.00 in	Acceptable
	Reviewed-OK Message 47	Acceptable	
#6 @ 6.0" H.E.F	Horz Steel Ratio	Horz Bar Spacing	Clear Cover
	Rho = 0.00333	S = 6.00 in	Cover Specified = 1.57 in
	Rho (min) = 0.00250	S (min) = 2.25 in	Max Cover = 14.67 in
	Acceptable	S (max) = 18.00 in	Acceptable
		Acceptable	

Zone A Reinforcing	**91-#14 Vert**	**#4 Ties @ 6.0 in**	
Vertical Bar Spacing	Vertical Bar Spacing	Tie Spacing	Splice Type
C-to-C low (y) = 6.0 in	C-to-C low (z) = 6.28 in	S = 6.0 in	Mechanical
C-to-C high (y) = 6.0 in	C-to-C high (z) = 6.28 in	S (min) = 2.5 in	Acceptable
Minimum (y) = 4.23 in	Minimum (z) = 4.23 in	S (max) = 6.0 in	
Maximum (y) = 18.0 in	Maximum (z) = 18.0 in	Acceptable	
Acceptable	Acceptable		
Area of Zone Steel		Tie Diameter	Misc Information
As = 204.75 sq.in.		d (tie) = 0.5 in	Scl (limit) = 5.0 in
As (min) = N/A		d (min) = 0.5 in	
As (max) = 274.01 sq.in.		Acceptable	
Ag (zone) = 3425.08 sq.in.			
Acceptable			

Zone B Reinforcing	**182-#14 Vert**	**#4 Ties @ 6.0 in**	
Vertical Bar Spacing	Vertical Bar Spacing	Tie Spacing	Splice Type
C-to-C low (y) = 6.0 in	C-to-C low (z) = 6.28 in	S = 6.0 in	Mechanical
C-to-C high (y) = 6.0 in	C-to-C high (z) = 6.28 in	S (min) = 2.5 in	Acceptable
Minimum (y) = 4.23 in	Minimum (z) = 4.23 in	S (max) = 6.0 in	
Maximum (y) = 18.0 in	Maximum (z) = 18.0 in	Acceptable	
Acceptable	Acceptable		
Area of Zone Steel		Tie Diameter	Misc Information
As = 409.5 sq.in.		d (tie) = 0.5 in	Scl (limit) = 5.0 in
As (min) = N/A		d (min) = 0.5 in	
As (max) = 548.57 sq.in.		Acceptable	
Ag (zone) = 6857.08 sq.in.			
Acceptable			

Panel Vertical Reinf.			**Panel Horizontal Reinf.**			**Zone Vertical Reinf.**		
fy (min)	40.0 ksi		fy (min)	40.0 ksi		fy (min)	40.0 ksi	
fy (vert)	97.0 ksi		fy (horz)	80.0 ksi		fy (vert)	97.0 ksi	
fy (max)	80.0 ksi		fy (max)	60.0 ksi		fy (max)	80.0 ksi	
Status	Reviewed-OK	Message 17	Status	Reviewed-OK	Message 18	Status	Reviewed-OK	Message 17

Concrete Strength		**Concrete Density**		**Zone Horizontal Reinf.**	
fc' (min)	2500.0 psi	Wc (min)	90.0 pcf	fy (min)	40.0 ksi
fc'	16000.0 psi	Wc	149.8 pcf	fy (horz)	80.0 ksi

fc' (max)	10000.0 psi		Wc (max)	160.0 pcf		fy (max)	100.0 ksi
Status	Reviewed-OK	Message 19	Status	Acceptable		Status	Acceptable

American Reinforcing Bars

Index	Bar Designation	Diameter (in)	Area (sq.in.)
1	#2	0.25	0.05
2	#3	0.375	0.11
3	#4	0.50	0.20
4	#5	0.625	0.31
5	#6	0.75	0.44
6	#7	0.875	0.60
7	#8	1.00	0.79
8	#9	1.128	1.00
9	#10	1.27	1.27
10	#11	1.41	1.56
11	#14	1.693	2.25
12	#18	2.257	4.00

Wall Dimensions

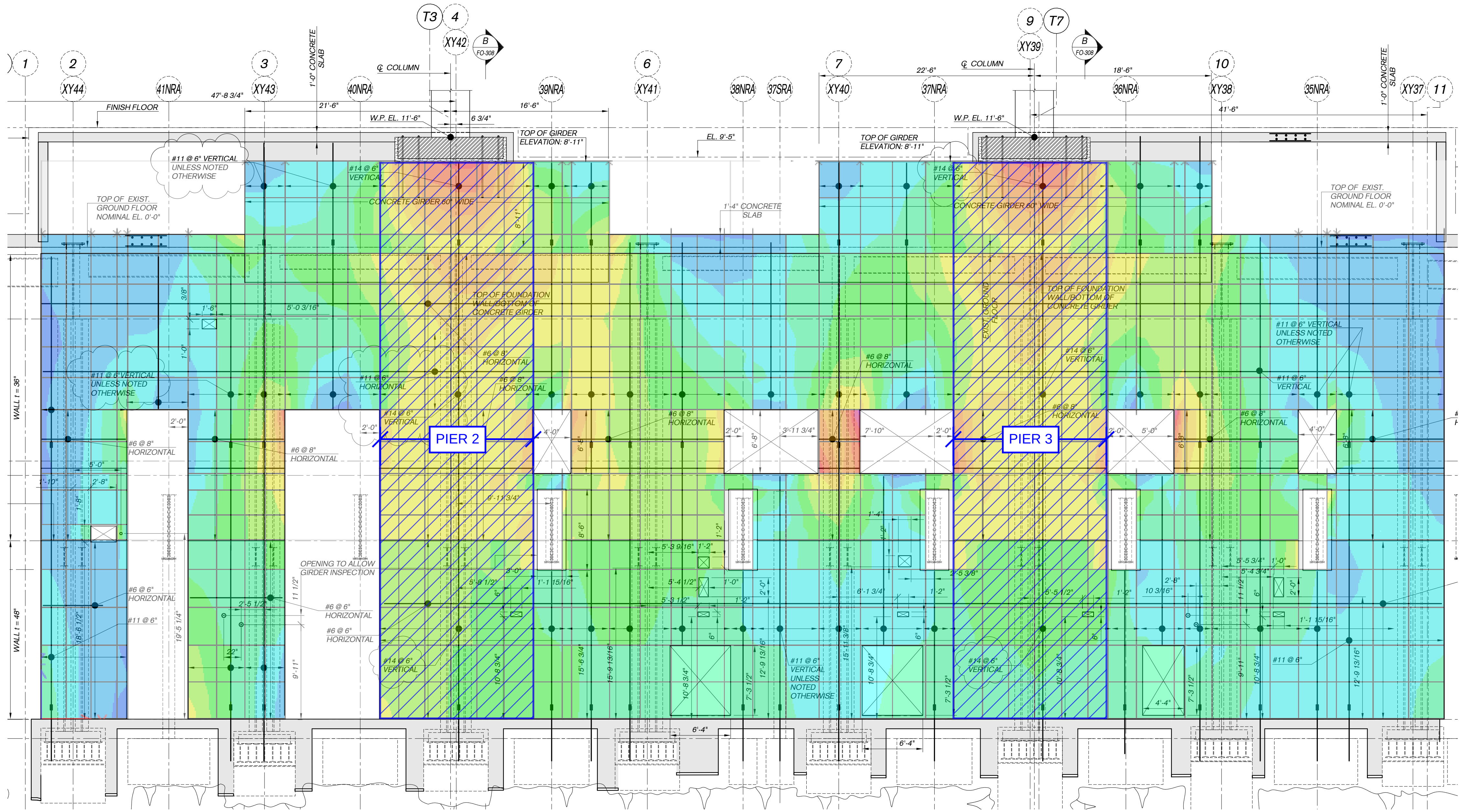
Lu (y-y) = 168.0 in, Lu (z-z) = 168.0 in, hw = 1181.1 in

Panel 1 Thickness
T = 44.0 in
T (min) = 6.72 in
Acceptable

List of Messages

Reviewed by Professional Engineer and considered OK

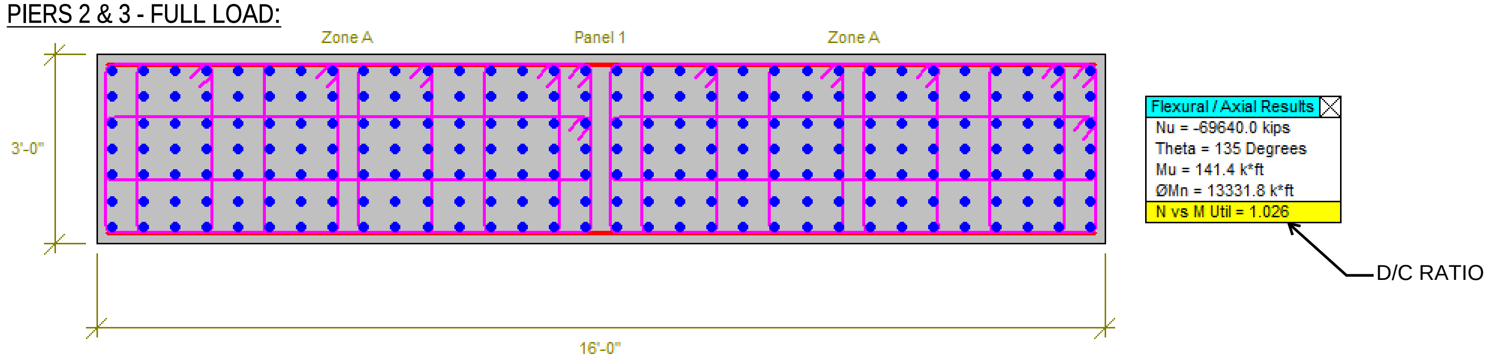
Message 17	Reviewed-OK	fy of Reinforcing is not within an Acceptable range. Clause 20.2.2.4 of ACI 318, 40 <= fy <= 80 ksi or 100 ksi
Message 18	Reviewed-OK	fy of Shear Reinforcing is not within an Acceptable range. Clause 20.2.2.4 of ACI 318, 40 <= fy <= 60 ksi
Message 19	Reviewed-OK	Strength of Concrete is not within an Acceptable range. Clauses 19.2.1.1, 22.5.3.1, or 22.7.2.1 of ACI 318, 2500 <= fc' <= 10000 psi
Message 47	Reviewed-OK	Panel Vertical Steel Ratio does not meet the minimum. Clauses 11.6.1 or 11.6.2 of ACI 318



SHEAR WALL ELEVATION AT COLUMN LINE H.1 - (R.R COLUMN LINE RA)

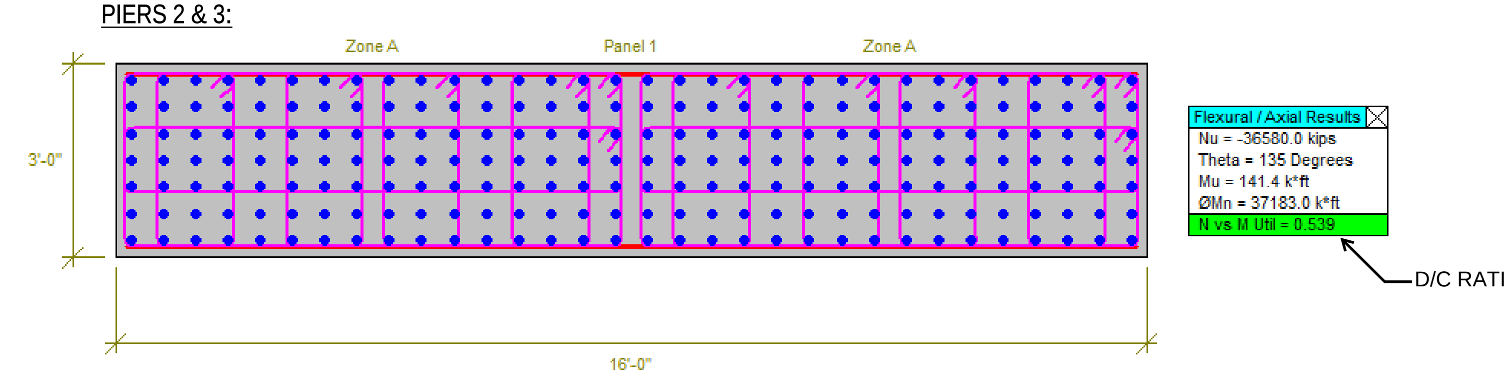
CONSERVATIVELY ASSUMING ALL OF THE LOAD IS RESISTED BY PIERS 2 AND 3 ONLY, TT CALCULATED THE WALL CAPACITY, SEE PRINT OUT FOR FULL CALCULATION (PAGES A2-6 to A2-8)

PIERS 2 & 3 - FULL LOAD:



ASSUMING LOAD DISTRIBUTION PER TT FE ANALYSIS, WALL WAS CHECKED FOR PORTION OF TOTAL LOAD SEEN BY PIERS 2 AND 3

PIERS 2 & 3:



Thornton Tomasetti	PROJECT: Project Greyhound Peer Review	SUBJECT: Shear Wall Calculations	A2-5
	REFERENCE: BY: CL/ASB DATE:		

S-CONCRETE 2018.1.1 (c) S-FRAME Software Inc.

File Name: C:\ ... ondono\Desktop\JPMC etabs\Wall T3-RA.SCO

Summary

Status	Borderline
Maximum	1.000
V (shear) Util	0.000
N vs M Util	1.026

Section Name
Concrete Section

Consultant
Thornton Tomasetti, Inc.

American Building Standards
ACI 318-14, "Building Code Requirements for Structural Concrete"
ACI 318R-14, "Commentary for ACI 318-14"

Design Aids, Manuals, and Handbooks
The Reinforced Concrete Design Handbook, A Companion to ACI 318-14
"ACI Detailing Manual - 1994", ACI Committee 315, American Concrete Institute, 1994
"Manual of Standard Practice", Concrete Reinforcing Steel Institute, 2003

Section Dimensions
I-Shape
L1 = 192.0 in
T1 = 36.0 in

Quantities (approx.)
Concrete = 6667 lb/ft
Steel = 1992.5 lb/ft
Primary = 1715.2 lb/ft
Secondary = 277.3 lb/ft

Material Properties
fc' = 15000 psi
fy (panel vert) = 97.0 ksi
fy (panel horz) = 80.0 ksi
fy (zone vert) = 97.0 ksi
fy (zone horz) = 80.0 ksi
Wc = 150 pcf
Ws = 490 pcf
Poisson's Ratio = 0.2
hagg = 0.79 in
Es = 29000 ksi
Ec = 7655 ksi
Gc = 3190 ksi
fr = 919 psi

Gross Properties
Zbar = 0.0 in
Ybar = 0.0 in
Ag = 6912.0 sq.in.
Ig (y-y) = 746496 in4
Ig (z-z) = 21234xE3 in4
Ashear (Y) = 5760.0 sq.in.
Ashear (Z) = 5760.0 sq.in.
Jg = 2633.1xE3 in4

Effective Properties
Ae = 6912.0 sq.in.
Ie (y-y) = 746496 in4
Ie (z-z) = 21234xE3 in4
Ase (Y) = 5760.0 sq.in.
Ase (Z) = 5760.0 sq.in.
Je = 2633.1xE3 in4

Panel 1
0-#4 @ 18.0" V.E.F
#6 @ 6.0" H.E.F.

Zone A
112-#14 Vert
#4 Ties @ 6.0 in
Mechanical Splice
As = 252.0 sq.in.
Y: 16@6.0"
Z: 7@4.94"

Factored Design Loads

Load Case/Combo	N (kips)	T (k*ft)	Vz (kips)	My (k*ft)	Vy (kips)	Mz (k*ft)	Mres (k*ft)	Theta
1 (W)	-69640.0	0.0	0.0	100.0	0.0	100.0	141.4	135º

N vs M Results

GLC	1	
Status	Borderline	Message 1
Utilization	1.026	
Maximum	1.000	
Theta	135º	

Axial Utilization
Nu = -69640.0 kips
ØNn (max) = -67906.8 kips
Utilization = 1.026

Moment Utilization
Mu = 141.4 k*ft
ØMn = 13331.8 k*ft
Utilization = 0.011

Mn = 52695.1 k*ft
Mp = 52754.6 k*ft

Panel 1 Reinforcing

#4 @ 18.0" V.E.F	Vert Steel Ratio	Vert Bar Spacing	Number of Curtains
	Rho = 0.00062	S = 18.00 in	Curtains Specified = 2
	Rho (min) = 0.00120	S (min) = 2.50 in	Curtains Required = 2
	Rho (max) = 0.01000	S (max) = 18.00 in	Acceptable
	Reviewed-OK Message 47	Acceptable	
#6 @ 6.0" H.E.F	Horz Steel Ratio	Horz Bar Spacing	Clear Cover
	Rho = 0.00407	S = 6.00 in	Cover Specified = 1.57 in
	Rho (min) = 0.00250	S (min) = 2.25 in	Max Cover = 12.00 in
	Acceptable	S (max) = 18.00 in	Acceptable
		Acceptable	

Zone A Reinforcing	**112-#14 Vert**	**#4 Ties @ 6.0 in**	
Vertical Bar Spacing	Vertical Bar Spacing	Tie Spacing	Splice Type
C-to-C low (y) = 6.0 in	C-to-C low (z) = 4.94 in	S = 6.0 in	Mechanical
C-to-C high (y) = 6.0 in	C-to-C high (z) = 4.94 in	S (min) = 2.5 in	Acceptable
Minimum (y) = 4.23 in	Minimum (z) = 4.23 in	S (max) = 6.0 in	
Maximum (y) = 18.0 in	Maximum (z) = 18.0 in	Acceptable	
Acceptable	Acceptable		
Area of Zone Steel		Tie Diameter	Misc Information
As = 252.0 sq.in.		d (tie) = 0.5 in	Scl (limit) = 5.0 in
As (min) = N/A		d (min) = 0.5 in	
As (max) = 276.03 sq.in.		Acceptable	
Ag (zone) = 3450.33 sq.in.			
Acceptable			

Panel Vertical Reinf.			**Panel Horizontal Reinf.**			**Zone Vertical Reinf.**		
fy (min)	40.0 ksi		fy (min)	40.0 ksi		fy (min)	40.0 ksi	
fy (vert)	97.0 ksi		fy (horz)	80.0 ksi		fy (vert)	97.0 ksi	
fy (max)	80.0 ksi		fy (max)	60.0 ksi		fy (max)	80.0 ksi	
Status	Reviewed-OK	Message 17	Status	Reviewed-OK	Message 18	Status	Reviewed-OK	Message 17

Concrete Strength			**Concrete Density**			**Zone Horizontal Reinf.**		
fc' (min)	2500.0 psi		Wc (min)	90.0 pcf		fy (min)	40.0 ksi	
fc'	15000.0 psi		Wc	149.8 pcf		fy (horz)	80.0 ksi	
fc' (max)	10000.0 psi		Wc (max)	160.0 pcf		fy (max)	100.0 ksi	
Status	Reviewed-OK	Message 19	Status	Acceptable		Status	Acceptable	

American Reinforcing Bars

Index	Bar Designation	Diameter (in)	Area (sq.in.)
1	#2	0.25	0.05
2	#3	0.375	0.11
3	#4	0.50	0.20
4	#5	0.625	0.31
5	#6	0.75	0.44
6	#7	0.875	0.60
7	#8	1.00	0.79

8	#9	1.128	1.00
9	#10	1.27	1.27
10	#11	1.41	1.56
11	#14	1.693	2.25
12	#18	2.257	4.00

Wall Dimensions

Lu (y-y) = 168.0 in, Lu (z-z) = 168.0 in, hw = 1181.1 in

Panel 1 Thickness
T = 36.0 in
T (min) = 6.72 in
Acceptable

List of Messages

Message 1	Borderline	Axial Load and Moment Utilization equals or exceeds Maximum. Clauses 22.2 and 22.4 of ACI 318
Message 17	Reviewed-OK	fy of Reinforcing is not within an Acceptable range. Clause 20.2.2.4 of ACI 318, 40 <= fy <= 80 ksi or 100 ksi
Message 18	Reviewed-OK	fy of Shear Reinforcing is not within an Acceptable range. Clause 20.2.2.4 of ACI 318, 40 <= fy <= 60 ksi
Message 19	Reviewed-OK	Strength of Concrete is not within an Acceptable range. Clauses 19.2.1.1, 22.5.3.1, or 22.7.2.1 of ACI 318, 2500 <= fc' <= 10000 psi
Message 47	Reviewed-OK	Panel Vertical Steel Ratio does not meet the minimum. Clauses 11.6.1 or 11.6.2 of ACI 318

Appendix 3 - Foundation Drawing Sheet List

STRUCTURAL DRAWINGS LIST	
DRAWING No.	DRAWING TITLE
DM-100	DEMOLITION PLAN AT SUBURBAN LEVEL
DM-101	EXISTING SUB CELLAR 2 DEMOLITION PLAN
DM-102	EXPRESS LEVEL DEMOLITION PLAN - STRUCTURAL
DM-103	EXISTING SUB CELLAR 1 DEMOLITION PLAN
DM-105	EXISTING CELLAR DEMOLTION PLAN
DM-106	HUNG CEILING DEMOLITION PLAN
DM-107	GROUND FLOOR DEMOLITION PLAN
DM-300	DEMOLITION OF BRACING AT COLUMN LINE F
DM-301	DEMOLITION OF BRACING AT COLUMN LINE SA
DM-302	DEMOLITION OF BRACING AT COLUMN LINE H
DM-303	DEMOLITION OF BRACING AT COLUMN LINE K
FO-100	FOUNDATION PLAN AT SUB-CELLAR 2 AND SUBURBAN LEVELS
FO-200	FOUNDATION DETAILS I
FO-201	FOUNDATION DETAILS II
FO-202	FOUNDATION DETAILS III
FO-203	FOUNDATION DETAILS IV
FO-205	FOUNDATION DETAILS VI
FO-230	FOUNDATION DETAILS VII
FO-231	FOUNDATION DETAILS VIII
FO-232	FOUNDATION DETAILS IX
FO-233	FOUNDATION DETAILS X
FO-234	SECTIONS AT MTA STAIR
FO-235	SECTIONS AT POPS TREE PITS
FO-250	WALL & PIER DETAILING I
FO-251	WALL & PIER DETAILING II
FO-252	WALL & PIER DETAILING III
FO-253	WALL & PIER DETAILING IV
FO-254	WALL & PIER DETAILING V
FO-255	WALL & PIER DETAILING VI
FO-256	WALL & PIER DETAILING VII
FO-300	SHEAR WALL ELEVATION AT COLUMN LINE F
FO-301	SHEAR WALL ELEVATION AT COLUMN LINE SA
FO-302	SHEAR WALL ELEVATION AT COLUMN LINE H
FO-303	SHEAR WALL ELEVATION AT COLUMN LINE K
FO-304	SHEAR WALL ELEVATION AT COLUMN LINE D.1
FO-305	SECTIONS & DETAILS I
FO-306	SECTIONS & DETAILS II
FO-307	SECTIONS & DETAILS III
FO-308	SECTIONS & DETAILS IV
FO-309	SECTIONS & DETAILS V
FO-310	SECTIONS & DETAILS VI
FO-311	SECTIONS & DETAILS VII
FO-320	EMBED PLATES AT SHEAR WALL LINE F
FO-321	EMBED PLATES AT SHEAR WALL LINE SA
FO-322	EMBED PLATES AT SHEAR WALL LINE H.1
FO-323	EMBED PLATES AT SHEAR WALL LINE K
FO-700	FOUNDATION TYPICAL DETAILS I
FO-701	FOUNDATION TYPICAL DETAILS II
FO-702	STAIR CONCRETE DETAILS
FO-800	GENERAL NOTES - FOUNDATIONS
S-001	GENERAL NOTES
S-093	INTERMEDIATE ROOF FRAMING PLAN
S-094	SUBCELLAR 1 FRAMING PLAN
S-095	EXPRESS LEVEL FRAMING PLAN - STRUCTURAL
S-096	EXPRESS LEVEL FRAMING PLAN RAILROAD
S-097	CELLAR FRAMING PLAN
S-098	HUNG CEILING FRAMING PLAN
S-099	EXISTING GROUND FLOOR FRAMING PLAN
S-100	GROUND FLOOR FRAMING PLAN - PT MAT
S-101	GROUND FLOOR UPPER FRAMING PLAN
S-301	TYPICAL MAT SCHEME DETAIL
S-302	ELEVATOR PIT DETAILS
S-310	GROUND FLOOR SECTIONS AND DETAILS

LANGAN

Memorandum

Langan Engineering, Environmental, Surveying, Landscape Architecture and Geology, D.P.C.
21 Penn Plaza, 360 West 31st Street, 8th Floor New York, NY 10001 T: 212.479.5400 F: 212.479.5444

To: Peter Crocitto – Tishman Speyer

From: Richard Lo, Marc Gallagher

Info: Roderic Ellman – MRCE

Date: 31 March 2020

Re: Peer Review of Grade 100 Reinforcing Bars in Foundations
270 Park Avenue
New York, New York
Langan Project No.: 170560101

This memorandum summarizes our peer review of the concrete foundation design by Mueser Rutledge Consulting Engineers PLLC (MRCE) for the 270 Park Avenue development. The foundations are reinforced with ASTM A1035 steel, which requires a peer review per the NYC Department of Buildings Bulletin 2018-013 (Bulletin). Our peer review is limited to the structural design of the concrete pile caps and footings, and we understand that a peer review of the superstructure design is being performed by others.

The purpose of the review is to verify that the foundation design shown on the plans generally conforms to the structural requirements of the Bulletin, and to confirm the general completeness of the plans.

Project Understanding

The site consists of the city block bound by Madison Avenue on the west, 48th Street on the north, Park Avenue on the east, and 47th Street on the south. The eastern two-thirds of the site is over existing MTA East Side Access (ESA) and Metro-North Railroad (MNR) structures. The block is completely covered by the existing 270 Park Avenue tower, which is being demolished to make way for a new tower.

The foundation of the new tower is split into two zones; the eastern MTA zone and the western non-MTA zone. Within the MTA zone, existing foundations will be retrofitted to enhance the load capacity; within the non-MTA zone completely new foundations are proposed. The retrofits are typically new pile caps or shallow footings that encase the original footings. Where piles are used, the foundation loads are shared between the piles and the pile cap bearing on rock. Compressible load transfer mats (LTMs) were selected to allow for a controlled load distribution between the piles and pile caps.

The MRCE structural analysis of the foundations was performed with 3D finite element modeling (FEM) using the commercial program Plaxis3D. Each column line was modeled to include superstructure loads via shear walls, stiffness of LTMs, and stiffness of foundation piles. The FEM analysis was used to calibrate the thickness of each LTM to achieve the desired load distribution, and to determine the design loads in the foundation elements.

Peer Review

Langan reviewed the following documents for this peer review:

- Design Calculations
  - Appendix A: Foundation Analysis and Design (RISA), 270 Park Avenue, MRCE FILE No. 13183, dated 9 December 2019
  - Foundation Analysis and Design (RISA) Addendum 1 – Response to Peer Review Comments, 270 Park Avenue, dated 20 March 2020

- Design Drawings
  - 270 Park Avenue, Foundation Design at Line D-1, IFC Bulletin 5, updated 21 February 2020
  - 270 Park Avenue, Foundation Design at Column Lines F-K, IFC Bulletin 5, updated 21 February 2020

The calculations were checked for design requirements stipulated in the Bulletin, including compliance with ACI ITG-6R-10[1] and ICC-ES AC429[2]. Requirements specific to the use of the ASTM A1035 reinforcement include specific limits on compression, tension, and shear yield stresses and a higher minimum tensile strain requirement for tension-controlled members.

The drawings were reviewed for consistency with the design calculations (i.e., reinforcement detailing, general dimensions). The drawings were also reviewed for completeness and constructability, such that sufficient information is provided to determine material specifications, material quantities, and overall geometry.

Conclusion

We have determined that the design shown on the foundation plans generally conform to the structural and foundation requirements of NYC Department of Buildings Bulletin 2018-013. The calculations are in conformance with the ACI ITG-6R-10 and ICC-ES AC429, and the drawings appropriately reflect the intent of the design engineer.

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[1] ITG-6R-10 Design Guide for the Use of ASTM A1035/A1035M Grade 100 (690) Steel Bars for Structural Concrete, ACI Committee 93, American Concrete Institute, 2010.
[2] ICC-ES AC429 Acceptance Criteria for High Strength Steel Reinforcing Bars, International Code Council Evaluation Service, 2017.

LANGAN

Memorandum

Langan Engineering, Environmental, Surveying, Landscape Architecture and Geology, D.P.C.
21 Penn Plaza, 360 West 31st Street, 8th Floor New York, NY 10001 T: 212.479.5400 F: 212.479.5444

To: Peter Crocitto, Christopher DeLuca – Tishman Speyer

From: Richard Lo, Marc Gallagher

Date: 14 May 2020

Re: Peer Review of Foundation Geotechnical Design
270 Park Avenue
New York, New York
Langan Project No.: 170560101

This memorandum summarizes our peer review of the foundation design by Mueser Rutledge Consulting Engineers PLLC (MRCE) for the 270 Park Avenue redevelopment. Our peer review is focused on the geotechnical design of the piles, pile caps, and footings. Specifically we are reviewing the foundation approach, the constructability, the technical analysis, the general conformance of the plans with the New York City Building Code, and the completeness of the plans. We understand that a peer review of the superstructure design is being performed by others.

Project Understanding

The site consists of the city block bound by Madison Avenue on the west, East 48th Street on the north, Park Avenue on the east, and East 47th Street on the south. The eastern two-thirds of the site is over existing MTA East Side Access (ESA) and Metro-North Railroad (MNR) structures. The block is completely covered by the existing 270 Park Avenue building, which is being demolished to make way for a new tower.

The foundation of the new tower is split into two zones; 1) the eastern MTA zone and 2) the western non-MTA zone. Within the MTA zone, existing foundations will be retrofitted to enhance the load capacity; within the non-MTA zone completely new foundations are proposed. The retrofits are new piles and pile caps, or shallow footings that encase original footings. Where piles are used, the foundation loads are shared between the piles and the pile cap bearing on rock. MRCE has designed compressible load transfer mats (LTMs) to control the load sharing between the piles and pile caps.

MRCE analyzed the foundations with 3D finite element modeling (FEM) using the commercial program Plaxis3D. Each column line was modeled to include superstructure loads, stiffness of LTMs, and stiffness of foundation piles. The FEM analysis was used to calibrate the thickness of each LTM to achieve the desired load sharing, and to determine the design loads in the foundation elements.

Peer Review

Langan reviewed the following MRCE documents for this peer review:

- Geotechnical Reports
    - Preliminary Geotechnical Report, 270 Park Avenue, dated June 21, 2018
    - Geotechnical Data Report, 270 Park Avenue, June 27, 2019
    - Geotechnical Interpretative Report, 270 Park Avenue, June 27, 2019
- Design Calculations
    - Appendix A: Foundation Analysis and Design (RISA), 270 Park Avenue, dated 9 December 2019
    - Foundation Analysis and Design (RISA) Addendum 1 – Response to Peer Review Comments, 270 Park Avenue, dated 20 March 2020
- Design Drawings
    - 270 Park Avenue, Foundation Design at Line D-1, IFC Bulletin 5, updated 21 February 2020
    - 270 Park Avenue, Foundation Design at Column Lines F-K, IFC Bulletin 5, updated 21 February 2020

The calculations and drawings were reviewed for the following:

Foundation Approach – The foundation system relies on a combination of deep (pile) foundations and shallow (spread footing) foundations. This atypical approach was reviewed for practicality and evaluated against other potential foundation options (i.e., larger diameter deep foundations or deeper excavations to higher quality rock).

Constructability – The constructability of the foundation system is limited by the site constraints associated within the MTA zone. Constraints include limited access headroom and limited footprint for staging, excavation, and construction. We reviewed the standard of practice construction methods for the proposed foundation systems and compared them against the reported site constraints.

Technical Analysis – The calculation package was reviewed qualitatively and quantitatively for completeness and correctness. This included an evaluation of geotechnical input parameters, geotechnical capacity analysis, and review of FEM modeling approach.

Code Compliance – The design was checked for conformance with foundation design requirements per New York City Building Code Chapter 18 – Soils and Foundations. The piles and shallow foundation elements were checked for design requirements, detailing requirements, and allowable stresses or bearing capacity.

Construction Documents – The drawings were reviewed for consistency with the design calculations (i.e., pile placement, detailing, general dimensions). The drawings were also reviewed for completeness and constructability, such that sufficient information is provided to determine material specifications, material quantities, and overall geometry.

Conclusion

We have determined that the design shown on the foundation plans generally conform to the foundation requirements of the New York City Building Code. The foundation approach is reasonable and should be constructible, and the drawings appropriately reflect the intent of the design.

Limitations

This peer review was performed in accordance with our approved contract dated 14 February 2019. As per the agreement, our review relies upon information provided by others; no information was independently verified (i.e. no confirmatory test borings or laboratory testing was performed.) Our scope of services did not include independently modeling the foundation design or performing verification calculations.

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Appendix 5 - Superstructure Drawing Sheet List

STRUCTURAL DRAWINGS LIST	
DRAWING NUMBER	DRAWING TITLE
S-000	STRUCTURAL COVER SHEET AND DRAWING LISTS
S-001	GENERAL NOTES - STRUCTURAL
S-094	SUBCELLAR 1 FRAMING PLAN
S-097	CELLAR FRAMING PLAN
S-099	EXISTING GROUND FLOOR FRAMING PLAN
S-100	GROUND FLOOR FRAMING PLAN - PT MAT
S-101	GROUND FLOOR UPPER FRAMING PLAN
S-102	LEVEL 2 FRAMING PLAN
S-103	LEVEL 3 FRAMING PLAN
S-104	LEVEL 4 FRAMING PLAN
S-105	LEVEL 5 FRAMING PLAN
S-106	LEVEL 6 FRAMING PLAN
S-107	LEVEL 7 FRAMING PLAN
S-108	LEVEL 8 FRAMING PLAN
S-109	LEVEL 9 FRAMING PLAN
S-110	LEVEL 10 FRAMING PLAN
S-111	LEVEL 11 FRAMING PLAN
S-112	LEVEL 12 FRAMING PLAN
S-113	LEVEL 13 FRAMING PLAN
S-114	LEVEL 14 FRAMING PLAN
S-115	LEVEL 15 FRAMING PLAN
S-116	LEVEL 16 FRAMING PLAN
S-117	LEVEL 17 FRAMING PLAN
S-118	LEVEL 18 FRAMING PLAN
S-119	LEVEL 19 FRAMING PLAN
S-120	LEVEL 20 FRAMING PLAN
S-121	LEVELS 21, 23, 25 AND 27 FRAMING PLANS (TYPICAL LOW RISE)
S-122	LEVELS 22, 24 AND 26 FRAMING PLANS (TYPICAL LOW RISE)
S-128	LEVEL 28 FRAMING PLAN
S-129	LEVEL 29 FRAMING PLAN
S-130	LEVEL 30 FRAMING PLAN
S-131	LEVELS 31, 33 AND 35 FRAMING PLANS (TYPICAL MID RISE)
S-132	LEVEL 32, 34 AND 36 FRAMING PLANS (TYPICAL MID RISE)
S-137	LEVEL 37 FRAMING PLAN
S-138	LEVEL 38 FRAMING PLAN
S-139	LEVEL 39 FRAMING PLAN
S-140	LEVEL 40 FRAMING PLAN
S-141	LEVELS 41 AND 43 FRAMING PLANS
S-142	LEVELS 42 AND 44 FRAMING PLANS (TYPICAL HIGH RISE OFFICE FLOORS)
S-145	LEVEL 45 FRAMING PLAN
S-146	LEVELS 46 AND 47 FRAMING PLANS
S-148	LEVEL 48 FRAMING PLAN
S-149	LEVEL 49 FRAMING PLAN
S-150	LEVEL 50 FRAMING PLAN
S-151	LEVEL 51 FRAMING PLAN
S-152	LEVEL 52 FRAMING PLAN
S-153	LEVEL 53 FRAMING PLAN
S-154	LEVEL 54 FRAMING PLAN
S-155	LEVEL 55 FRAMING PLAN
S-156	LEVEL 56 FRAMING PLAN
S-157	LEVEL 57 FRAMING PLAN
S-158	LEVEL 58 FRAMING PLAN
S-159	LEVEL 59 FRAMING PLAN
S-160	LEVEL 60 FRAMING PLAN
S-170	2ND FLOOR INTERSTITIAL FRAMING PLAN AND SECTIONS
S-171	LEVEL 11 MEZZANINE FRAMING PLAN
S-172	LEVEL 19 MEZZANINE FRAMING PLAN
S-173	LEVEL 55 MEZZANINE FRAMING PLAN
S-175	PARTIAL 19TH FLOOR RIGGING FRAMES
S-220	2ND & 3RD FLOORS SECTIONS AND DETAILS

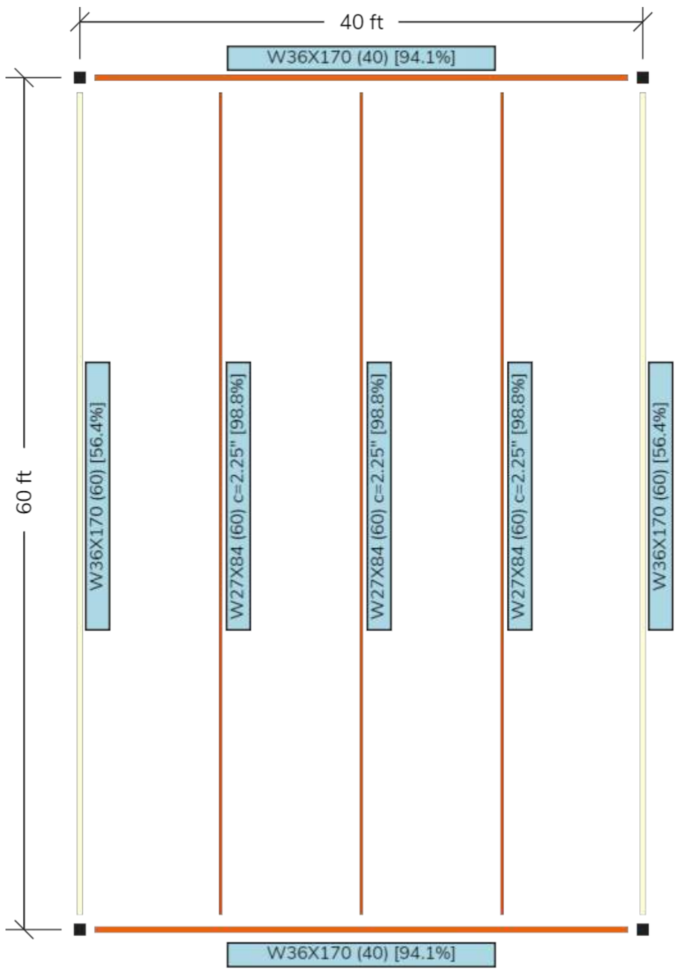
S-221	SPANDREL SECTIONS
S-222	SECTIONS
S-301	TYPICAL MAT SCHEME DETAIL
S-302	ELEVATOR PIT DETAILS
S-303	SHUTTLE (&TF) ELEVATORS FRAMING PLAN AND ELEVATIONS
S-304	SHUTTLE ELEVATOR FRAMING CONNECTION DETAILS
S-305	ELEVATOR MACHINE ROOM PLANS
S-306	INTERSTITIAL FRAMING PART PLANS I
S-307	INTERSTITIAL FRAMING PART PLANS II
S-308	INTERSTITIAL FRAMING PART PLANS III
S-309	INTERSTITIAL FRAMING PART PLANS IV
S-310	GROUND FLOOR SECTIONS AND DETAILS
S-311	ESCALATOR PIT DETAILS
S-371	ELEVATORS DETAILS
S-401	OVERALL FRAME ELEVATIONS AT GRID LINES T3 , T5 ,T7, T1 & T9
S-402	OVERALL FRAME ELEVATIONS AT GRID LINES TA, TB , D.1 ,TC & TD
S-403	OVERALL FRAME ELEVATIONS AT GRID LINES TE ,TF & TG
S-404	OVERALL FRAME ELEVATIONS AT GRID LINES TH , TJ & TK
S-411	GRID T3 FRAME ELEVATION
S-412	GRID T7 FRAME ELEVATION
S-413	GRID TA & TB FRAME ELEVATION
S-414	GRID D.1 & TC FRAME ELEVATION
S-415	GRID TD FRAME ELEVATION
S-416	GRID TE FRAME ELEVATION
S-417	GRID-TF FRAME ELEVATION
S-418	GRID-TG FRAME ELEVATION
S-419	GRID-TH FRAME ELEVATION
S-420	GRID-TJ, TK, T5, T1 & T9 FRAME ELEVATION
S-550	INTERIOR "V" COLUMNS AND PLATE GIRDER ELEVATIONS
S-551	PLATE GIRDERS CROSS SECTION AND DETAILS
S-552	PLATE GIRDER DETAILS
S-553	GROUND FLOOR PLATE GIRDER ELEVATIONS AND SECTIONS
S-560	FORGINGS MASSING
S-561	FORGINGS MASSING
S-562	TRANSFER COLUMN DETAILS AT 3RD FLOOR
S-570	TRUSS ELEVATIONS I
S-571	TRUSS ELEVATIONS II
S-572	TRUSS ELEVATIONS III
S-573	TRUSS ELEVATIONS IV
S-574	TRUSS ELEVATIONS V
S-575	TRUSS ELEVATIONS VI
S-576	TRUSS ELEVATIONS VII
S-577	TRUSS ELEVATIONS VIII
S-578	TRUSS ELEVATIONS IX
S-579	TRUSS ELEVATIONS X
S-580	TRUSS ELEVATIONS XI
S-581	TRUSS ELEVATIONS XII
S-582	TRUSS ELEVATIONS XIII
S-583	TRUSS ELEVATIONS XIV
S-584	TRUSS ELEVATIONS XV
S-585	TRUSS ELEVATIONS XVI
S-601	COLUMNS GROUND TO 3RD FLOOR
S-611	COLUMN SCHEDULE - I (LEVELS 3 TO 11)
S-612	COLUMN SCHEDULE - II (LEVELS 11 TO 18)
S-613	COLUMN SCHEDULE - III (LEVELS 19 TO 31)
S-614	COLUMN SCHEDULE - IV (LEVELS 32 TO 43)
S-615	COLUMN SCHEDULE - V (LEVELS 44 TO 54)
S-616	COLUMN SCHEDULE - VI (54th LEVEL TO TOP)
S-620	FAN COLUMN FOUNDATION DETAILS AT GRID LINE K
S-621	V COLUMN FOUNDATION DETAILS AT GROUND FLOOR AT LINE K
S-622	V COLUMN FOUNDATION DETAILS AT LOBBY LEVEL AT LINE H
S-623	FAN COLUMN FOUNDATION DETAILS AT LINE SA

S-624	V COLUMN FOUNDATION DETAILS AT LOBBY LEVEL AT LINE F
S-625	FAN COLUMN FOUNDATION DETAILS AT LINE D.1
S-626	COLUMN SUPPORT DETAILS AT LINE D.1
S-651	BUILT-UP COLUMN DETAILS
S-701	DESIGN CRITERIA AND LOADING SCHEDULE
S-702	TYPICAL FLOOR CONSTRUCTION DETAILS AND SCHEDULES
S-711	TYP FLOOR CONSTRUCTION DETAILS AND SCHEDULE
S-721	TYPICAL CONCRETE DETAILS
S-731	TYPICAL STEEL DETAILS-I COLUMNS
S-732	TYPICAL STEEL DETAILS-II
S-733	TYPICAL STEEL DETAILS-III
S-734	TYPICAL STEEL DETAILS-IV BRACING
S-735	EAST-WEST CORE WIND BRACING GEOMETRY WORKSHEET - I
S-736	EAST-WEST CORE WIND BRACING GEOMETRY WORKSHEET - II
S-737	EAST-WEST CORE WIND BRACING GEOMETRY WORKSHEET - III
S-738	EAST-WEST CORE WIND BRACING GEOMETRY WORKSHEET - IV
S-739	EAST-WEST CORE WIND BRACING GEOMETRY WORKSHEET - V
S-751	TYPICAL CONCRETE MASONRY DETAILS
S-752	KEY ELEMENTS FRAMING PLAN-I
S-753	KEY ELEMENTS FRAMING PLAN-II
S-754	KEY ELEMENTS FRAMING PLAN-III
S-755	KEY ELEMENTS FRAMING PLAN-IV
S-756	KEY ELEMENTS FRAMING PLAN-V
S-757	KEY ELEMENTS FRAMING PLAN-VI

Appendix 6 - Typical Bay Check (Strength and Service)

Bay Study: Typical Office

Project: JPMC Check
2020 Jun 02 6:10 PM



NOTES:

1. DECK CONSISTS OF 4.5" NW CONCRETE ON 3" 18 GA. COMPOSITE DECK

**2. DENOTES ADDITIONAL LOADING AND DEFLECTION CRITERIA HAS BEEN APPLIED

Result Comparisons

	Strength + Deflection	Vibration
Top Girder	W36X170	W36X170
Bottom Girder	W36X170	W36X170
Left Beam	W36X170	W36X170

	Strength + Deflection	Vibration
Right Beam	W36X170	W36X170
Infill Beam	W27X84	W27X84
Tonnage	13.4 psf	13.4 psf

Strength Analysis Results

Girder Top	
Utilization	
Post Composite Deflection Ratio	94.1%
Post Composite Flexure Ratio	89.1%
Properties	
Selected Shape	W36X170
Weight	170 lb/ft
Area	50 in2
Depth	36.2 in
Zx	668 in3
Ix	10500 in4
Composite Properties	
Ieff	in4
Studs	40
Camber	0.00 inches
% Composite	17.60 %
Strength: Capacity	
Post-Composite Factored Moment	3619.0 kip-ft
Pre-Composite Factored Moment	2445.2 kip-ft
Factored Shear	738.4 kips
Stud Strength	430.7 kips
Strength: Demands	
Mu	3225.3 kip-ft
Vu	322.5 kips
Serviceability: Demand	
Live Deflection	0.54 inches
Total Deflection	1.88 inches
Serviceability: Capacity	
Live Deflection	1.33 inches
Total Deflection	2.00 inches

Beam Interior	
Utilization	
Post Composite Deflection Ratio	98.8%
Post Composite Flexure Ratio	82.6%
Properties	
Selected Shape	W27X84
Weight	84 lb/ft
Area	24.7 in2
Depth	26.7 in
Zx	244 in3
Ix	2850 in4
Composite Properties	
Ieff	in4
Studs	60
Camber	2.25 inches
% Composite	41.43 %
Strength: Capacity	
Post-Composite Factored Moment	1500.3 kip-ft
Pre-Composite Factored Moment	915.0 kip-ft
Factored Shear	368.4 kips
Stud Strength	511.7 kips
Strength: Demands	
Mu	1239.6 kip-ft
Vu	82.6 kips
Serviceability: Demand	
Live Deflection	1.18 inches
Total Deflection	2.96 inches
Serviceability: Capacity	
Live Deflection	2.00 inches
Total Deflection	3.00 inches

Girder Bottom	
Utilization	
Post Composite Deflection Ratio	94.1%
Post Composite Flexure Ratio	89.1%
Properties	
Selected Shape	W36X170
Weight	170 lb/ft
Area	50 in2
Depth	36.2 in
Zx	668 in3
Ix	10500 in4
Composite Properties	
Ieff	in4
Studs	40
Camber	0.00 inches
% Composite	17.60 %
Strength: Capacity	
Post-Composite Factored Moment	3619.0 kip-ft
Pre-Composite Factored Moment	2445.2 kip-ft
Factored Shear	738.4 kips
Stud Strength	430.7 kips
Strength: Demands	
Mu	3225.3 kip-ft
Vu	322.5 kips
Serviceability: Demand	
Live Deflection	0.54 inches
Total Deflection	1.88 inches
Serviceability: Capacity	
Live Deflection	1.33 inches
Total Deflection	2.00 inches

Beam Left	
Utilization	
Post Composite Deflection Ratio	56.4%
Post Composite Flexure Ratio	35.2%
Properties	
Selected Shape	W36X170
Weight	170 lb/ft
Area	50 in2
Depth	36.2 in
Zx	668 in3
Ix	10500 in4
Composite Properties	
Ieff	in4
Studs	60
Camber	0.00 inches
% Composite	20.90 %
Strength: Capacity	
Post-Composite Factored Moment	3657.5 kip-ft
Pre-Composite Factored Moment	2505.0 kip-ft
Factored Shear	738.4 kips
Stud Strength	511.7 kips
Strength: Demands	
Mu	1286.0 kip-ft
Vu	85.7 kips
Serviceability: Demand	
Live Deflection	0.43 inches
Total Deflection	1.69 inches
Serviceability: Capacity	
Live Deflection	2.00 inches
Total Deflection	3.00 inches

Beam Right	
Utilization	
Post Composite Deflection Ratio	56.4%
Post Composite Flexure Ratio	35.2%
Properties	
Selected Shape	W36X170
Weight	170 lb/ft
Area	50 in2
Depth	36.2 in
Zx	668 in3
Ix	10500 in4
Composite Properties	
Ieff	in4
Studs	60
Camber	0.00 inches
% Composite	20.90 %
Strength: Capacity	
Post-Composite Factored Moment	3657.5 kip-ft
Pre-Composite Factored Moment	2505.0 kip-ft
Factored Shear	738.4 kips
Stud Strength	511.7 kips
Strength: Demands	
Mu	1286.0 kip-ft
Vu	85.7 kips
Serviceability: Demand	
Live Deflection	0.43 inches
Total Deflection	1.69 inches
Serviceability: Capacity	
Live Deflection	2.00 inches
Total Deflection	3.00 inches

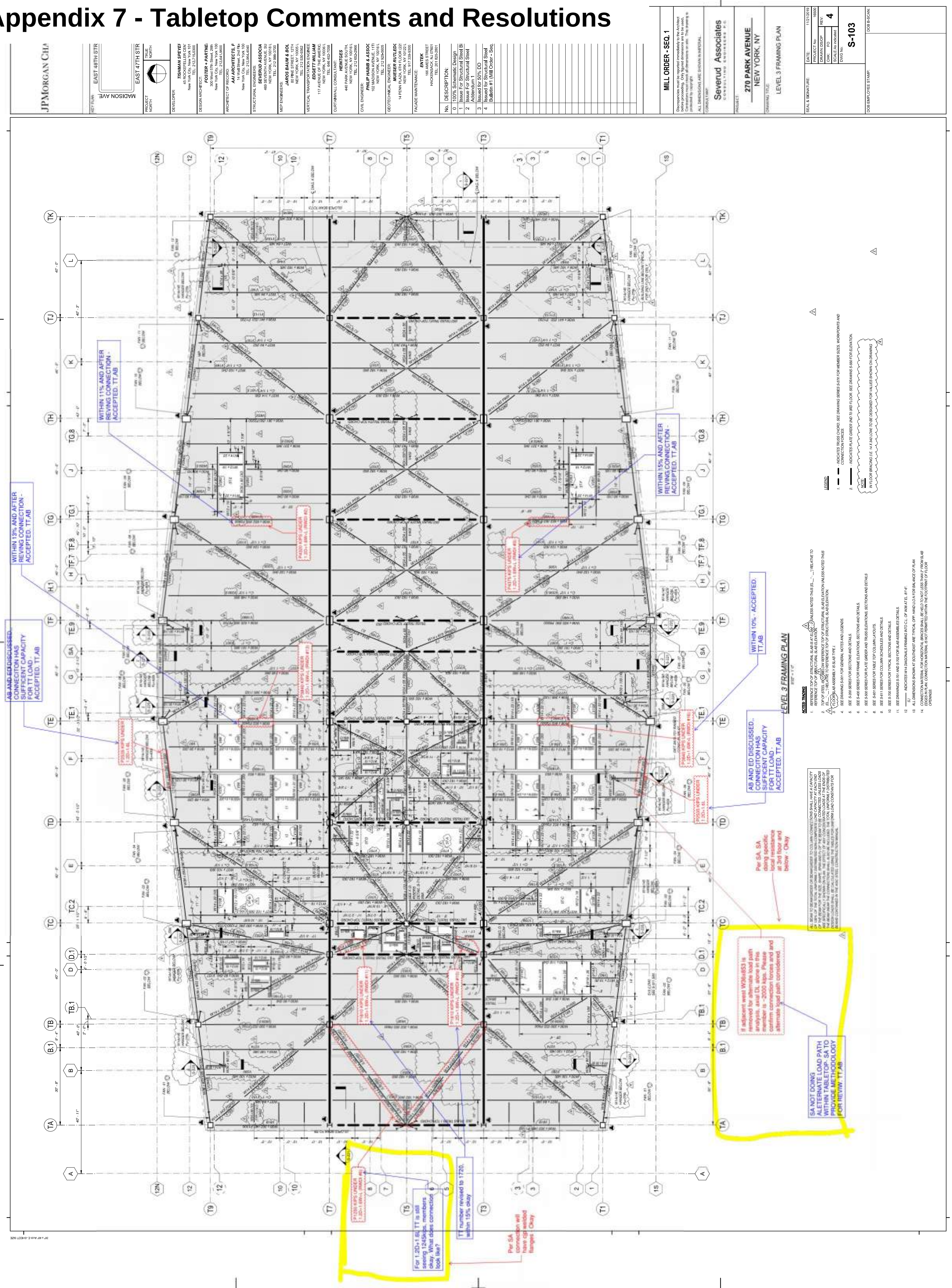
Vibrations Bay Results

Calculation Details

Member | Bay Vibration | Hide Details

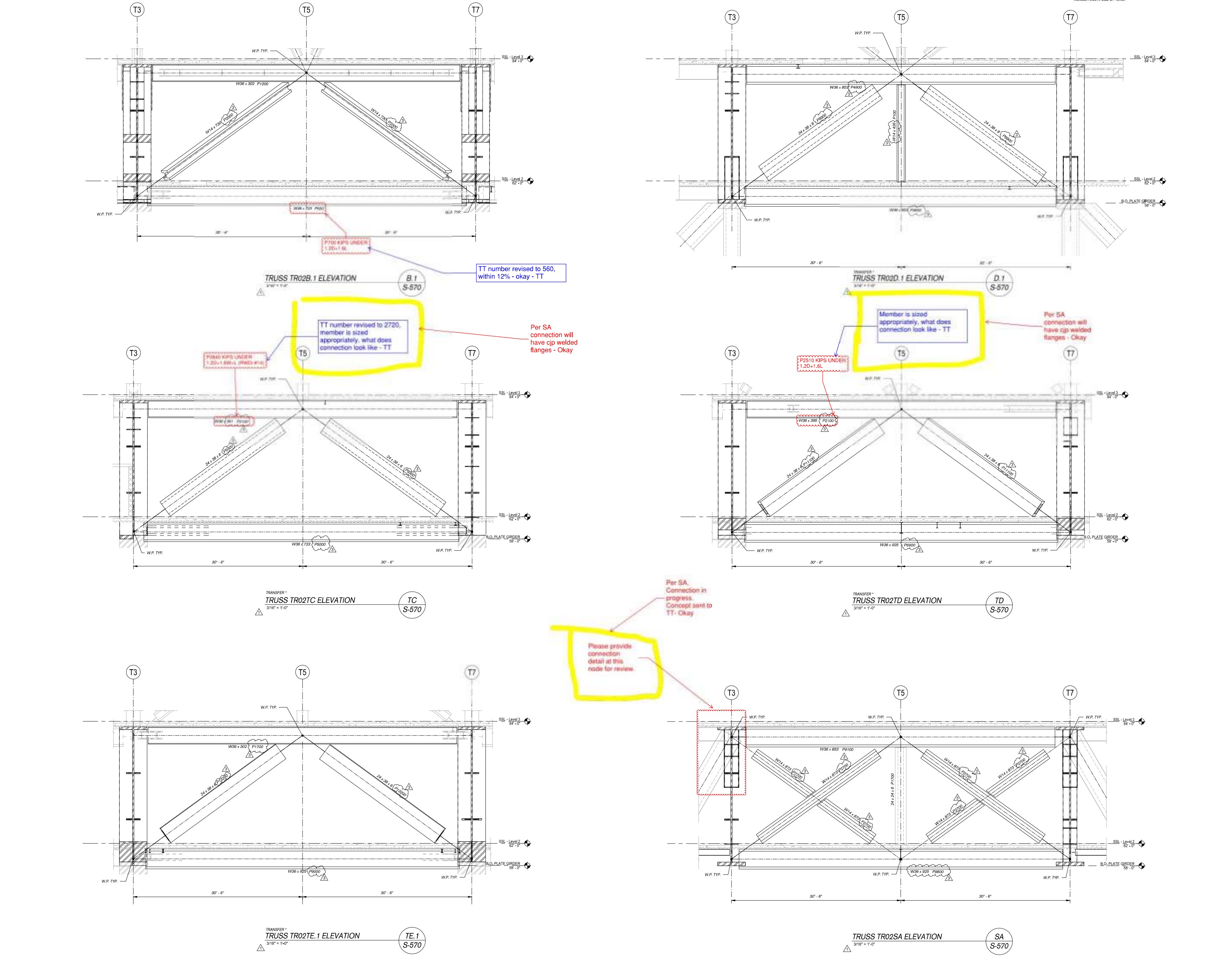
Bay	
Utilization	
Acceleration Ratio	50.4%
Material Properties	
Dynamic Concrete Modulus	4960.22 ksi
Modular Ratio	5.85
Beam Properties	
Effective Concrete Slab Width	120.00 in
Transformed Moment of Inertia	9748.05 in^4
Beam Coefficient (Cj)	2.00
Effective Beam Panel Width B	52.95 ft
Beam Weight	403074.60 lbs
Beam Midspan Deflection	1.31 in
Beam Frequency	3.09 Hz
Girder Properties	
Governing Girder	BOTH
Effective Concrete Slab Width	192.00 in
Transformed Moment of Inertia	31338.49 in^4
Girder Coefficient (Cg)	1.80
Effective Girder Panel Width B	84.16 ft
Girder Weight	436650.42 lbs
Girder Midspan Deflection	0.49 in
Girder Frequency	5.04 Hz
Bay Properties	
Effective Panel Weight	410516.05 lbs
Bay Frequency	2.63 Hz
Behavior	
Tolerance Limit Type	Walking
Damping Beta	0.025
Results	
Bay Acceleration	0.252 %g
Limit	0.500 %g

Appendix 7 - Tabletop Comments and Resolutions



NOTES:

1. SEE DRAWING S-100 FOR CONNECTION DESIGN FORCE NOTATION.
2. WHERE AXIAL FORCE PXXXXX IS NOT INDICATED ON ELEVATIONS, END CONNECTIONS OF TRUSS CHORDS, TRUSS DIAGONALS AND TRUSS VERTICALS SHALL BE DESIGNED FOR AN ULTIMATE LOAD EQUAL TO THE GROSS CROSS SECTIONAL AREA OF THE MEMBER MULTIPLIED BY 40KSI.



JPMORGAN CHASE & CO.

KEY PLAN: EAST 48TH STREET / MADISON AVE. / PARK AVE. / EAST 47TH STREET

PROJECT NORTH / TRUE NORTH

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CURTAINWALL CONSULTANT:
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TEL: 917 339-9300

FACADE MAINTENANCE:
ENTEK
166 AMES STREET
HACKENSACK, NJ 07601 USA
TEL: 201 820-2801

No.	DESCRIPTION:	DATE:
0	Issue For Structural Steel Bid	09.26.2019
1	Issue For Structural Steel Addendum 1	12.19.2019
2	Issued for 50% DD	02.14.2020
3	Issued for Structural Steel Bulletin #1 (Mill Order - Seq. 1)	04.09.2020

MILL ORDER - SEQ. 1

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ALL DIMENSIONS ARE SHOWN IN IMPERIAL.

CONSULTANT:
Severud Associates
CONSULTING ENGINEERS P.C.

PROJECT:
270 PARK AVENUE
NEW YORK, NY

DRAWING TITLE:
TRUSS ELEVATIONS I

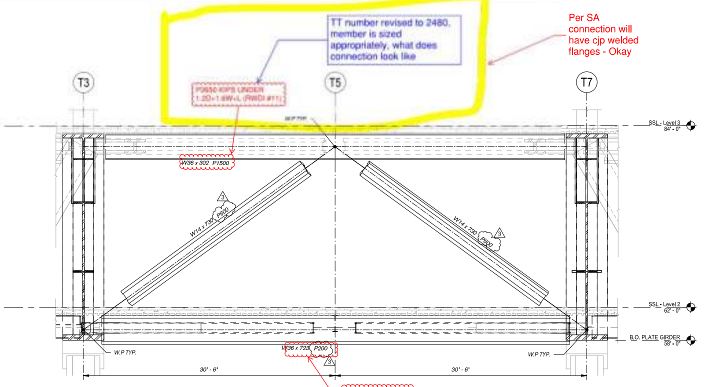
SEAL & SIGNATURE:

DATE:	11/21/2019
PROJECT No:	16500
DRAWN: DS/OP	REV: 3
CHK: FO	
SCALE: 3/16" = 1'-0"	
DWG No:	S-570

DOB EMPLOYEE STAMP:

DOB B-SCAN:

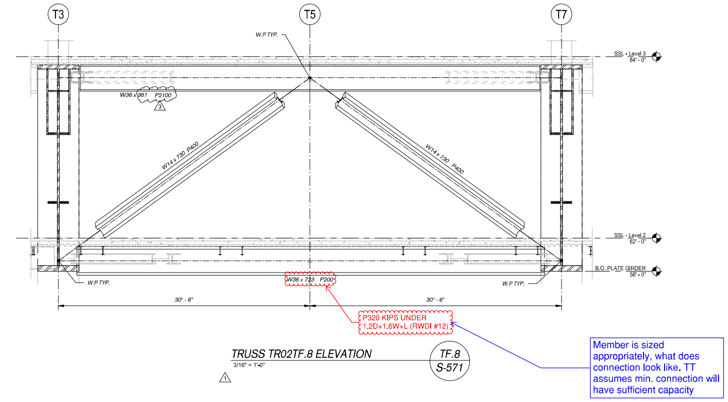
30" x 40" Arch E-SHEET SIZE



TRUSS TR02TF ELEVATION

3/16" = 1'-0"

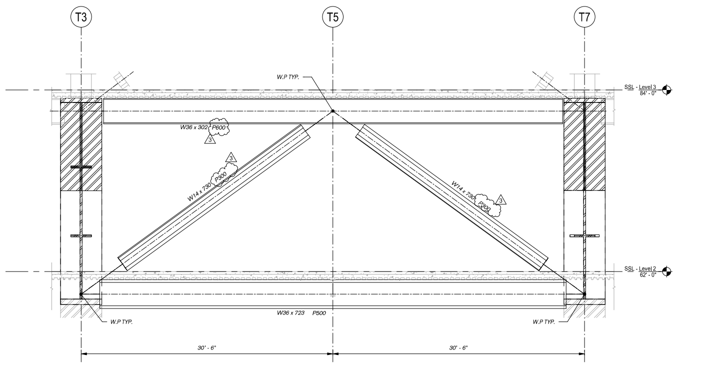
TF / S-571



TRUSS TR02TF.8 ELEVATION

3/16" = 1'-0"

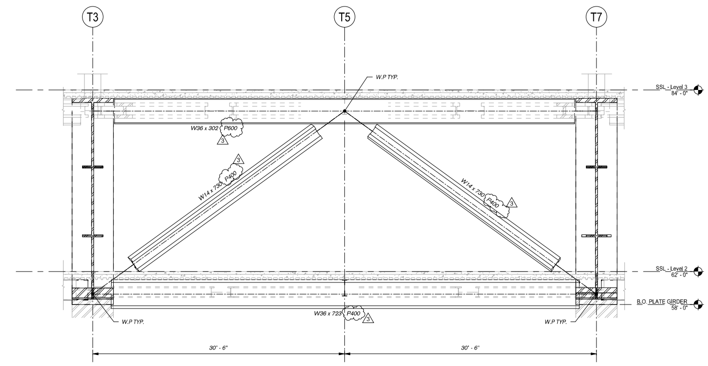
TF.8 / S-571



TRUSS TR02TH ELEVATION

3/16" = 1'-0"

TH / S-571



TRUSS TR02TJ ELEVATION

3/16" = 1'-0"

TJ / S-571

NOTES:

1. SEE DRAWING S-100 FOR CONNECTION DESIGN FORCE NOTATION.
2. WHERE AXIAL FORCE PXXXXX IS NOT INDICATED ON ELEVATIONS, END CONNECTIONS OF TRUSS CHORDS, TRUSS DIAGONALS AND TRUSS VERTICALS SHALL BE DESIGNED FOR AN ULTIMATE LOAD EQUAL TO THE GROSS CROSS SECTIONAL AREA OF THE MEMBER MULTIPLIED BY 40KSI.

NOTE: SEE S734 SIMILAR FOR CONNECTION DETAILS.
(+/-XXX) IS CONNECTION FORCE IN KIPS.

* TRANSFER COLUMN ABOVE
SEE COLUMN SCHEDULES.

JPMorgan Chase & Co.

KEY PLAN: EAST 48TH STREET / MADISON AVE. / PARK AVE. / EAST 47TH STREET

PROJECT NORTH / TRUE NORTH

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TEL: 201 820-2801

No.	DESCRIPTION:	DATE:
0	Issue For Structural Steel Bid	09.26.2019
1	Issue For Structural Steel Addendum 1	12.19.2019
2	Issued for 50% DD	02.14.2020
3	Issued for Structural Steel Bulletin #1 (Mill Order - Seq. 1)	04.09.2020

MILL ORDER - SEQ. 1

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ALL DIMENSIONS ARE SHOWN IN IMPERIAL.

CONSULTANT:
Severud Associates
CONSULTING ENGINEERS P.C.

PROJECT:
270 PARK AVENUE
NEW YORK, NY

DRAWING TITLE:
TRUSS ELEVATIONS II

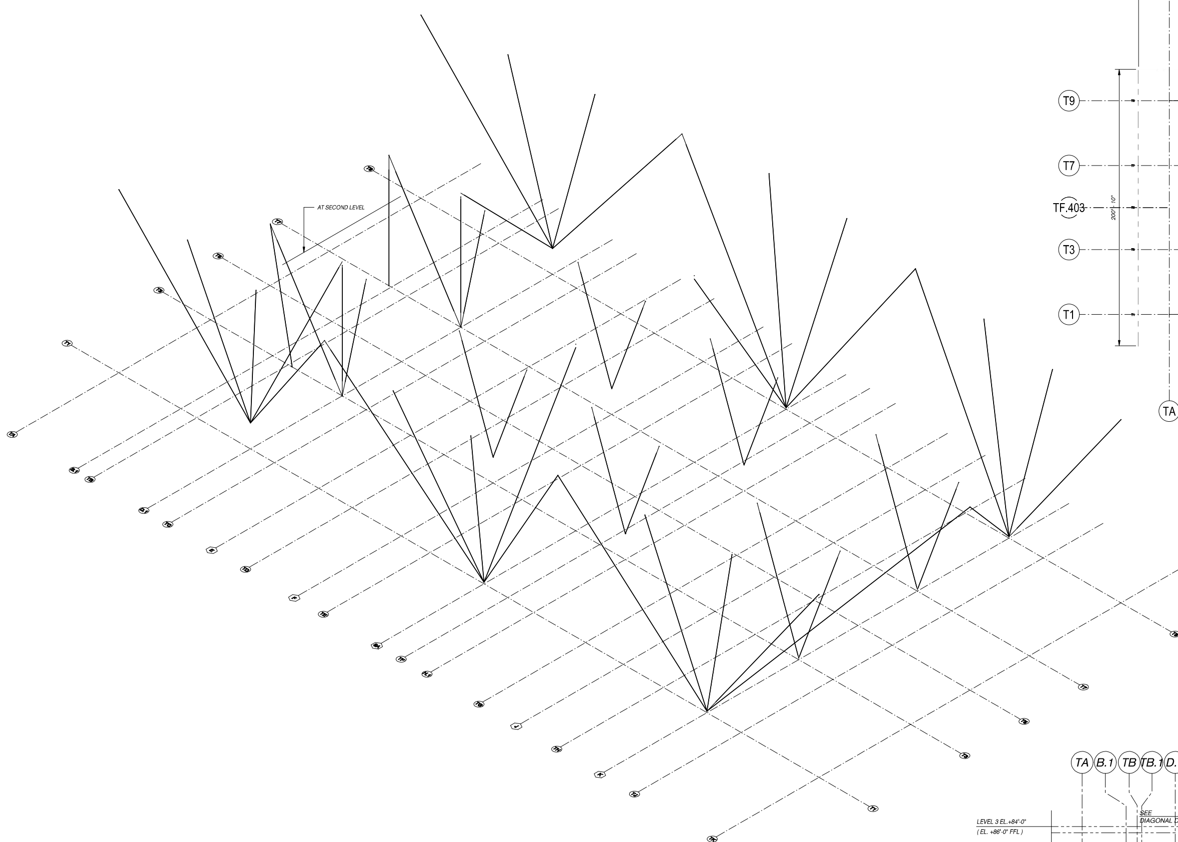
SEAL & SIGNATURE:

DATE:	11/21/2019
PROJECT No:	16500
DRAWN: DS/OP	REV: 3
CHK: FO	
SCALE: 3/16" = 1'-0"	
DWG No:	S-571

DOB EMPLOYEE STAMP:

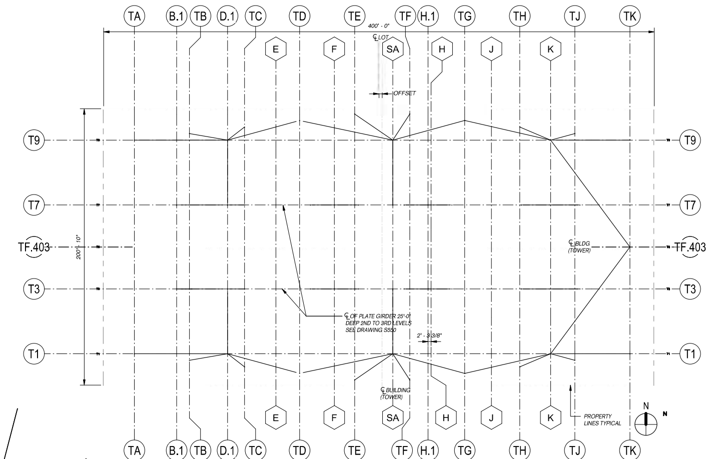
DOB B-SCAN:

30" x 42" Arch E-SHEET SIZE



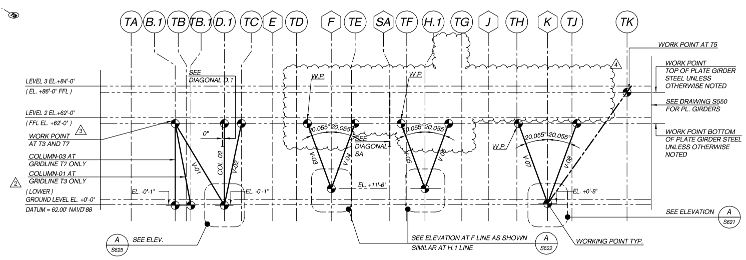
AXONOMETRIC OF TABLETOP WIREFRAME

NOTE: SEE 3RD FLOOR FOR TOP OF PERIMETER SLOPED COLUMNS AND DIAGONALS. SEE 2ND FLOOR AND S-550 FOR TOP OF COLUMNS AND DIAGONALS AT LINES T3 & T7. SEE S-550 AND S-601 FOR BOTTOM OF COLUMNS AND DIAGONALS U.O.N.



PLAN OF TABLETOP WIREFRAME

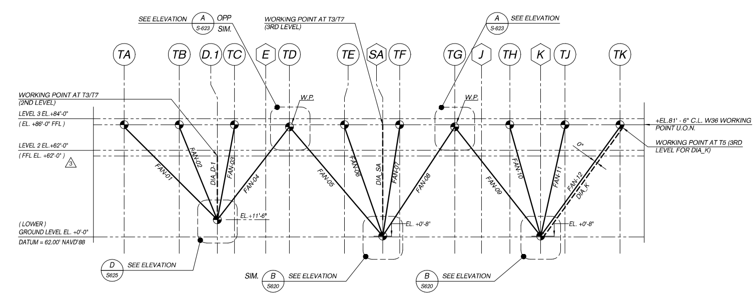
NOTE: FOR ALL DIMENSIONS, SEE ARCH DRAWINGS.



ELEVATION OF TABLETOP WIREFRAME AT INTERIOR COLUMNS LINES T7 SIMILAR AT T3

1/32" = 1'-0"

1 / S-601



ELEVATION OF TABLETOP WIREFRAME AT FAN COLUMNS LINES T1 & T9

1/32" = 1'-0"

2 / S-601

TABLE TOP COLUMN SCHEDULE

LOCATION	NAME	SIZE	COMPRESSION (k)	TENSION (k)	FABRICATION SEGMENTED CAMBER FROM THEORETICAL CENTERLINE	REMARKS
AT B.1, AT T3/T7	COLUMN_01/03	36 x 36 x 6	25,800	0	AT COLUMN_01: 360 KIPS UNDER 0.9D+1.6W (RWDI #1) AT COLUMN_03: 1,100 KIPS UNDER 0.9D+1.6W (RWDI #4)	
AT D.1, AT T3/T7	COLUMN_02/04	36 x 36 x 5	19,100	-		
AT D.1, AT T1/T9 U.O.N	FAN_01/13	34 x 48 x 6	9,300	1,800	-	-
AT D.1, AT T1/T9 U.O.N	FAN_02/14	34 x 48 x 4	13,600	-	-	-
AT D.1, AT T1/T9 U.O.N	FAN_03/15	34 x 48 x 7	26,600	1,000	-	-
AT D.1, AT T1/T9 U.O.N	FAN_04/16	34 x 48 x 7	23,600	-	-	-
AT SA	FAN_05/17	34 x 48 x 4	9,100	9,900 KIPS UNDER 1.2D+1.6W+L (RWDI #15/14)	-	-
AT SA	FAN_06/18	34 x 48 x 8 + 8" RIB	28,400		-	-
AT SA	FAN_07/19	34 x 48 x 8 + 8" RIB	29,800	-	-	-
AT SA	FAN_08/20	34 x 48 x 6	18,400	-	-	-
AT K	FAN_09/21	34 x 48 x 6	16,200	600	-	-
AT K	FAN_10/22	34 x 48 x 7	24,400	-	-	-
AT K	FAN_11/23	34 x 48 x 4	14,100	-	-	-
AT K	FAN_12/24	34 x 48 x 6	9,700	1,100	-	-
EAST END	DIA_K x 2	48 x 48 x 6 *	9,400	4,600	-	* TAPERED TO 30" x 30" FOR THE LAST 10'-0" AT EACH END, ALIGN EAST FACE WITH EAST FACADE.
AT T1 AND T9	DIA_SA x 2	48 x 48 x 8 + 8" RIB	36,300	6,400	-	-
AT T1 AND T9	DIA_D-1 x 2	48 x 48 x 8	17,200	6,900	-	-
AT D.1	V_01/09	36 x 36 x 4	6,700	-	-	-
AT D.1	V_02/10	36 x 36 x 7	30,100	-	AT V_02 ONLY: 400 KIPS UNDER 0.9D+1.6W (RWDI #2)	-
AT F	V_03/11	48 x 48 x 8	43,900	-	-	-
AT F	V_04/12	48 x 48 x 8	44,800	-	-	-
AT H.1	V_05/13	48 x 48 x 8	37,300	-	-	-
AT H.1	V_06/14	48 x 48 x 8	40,100	-	-	-
AT K	V_07/15	48 x 48 x 6	38,500	-	-	-
AT K	V_08/16	48 x 48 x 6	27,700	-	-	-

Member is sized appropriately, due to design columns have tension capacity in final condition - okay, TT

Member is sized appropriately, does not affect connection - okay, TT

Member is sized appropriately, due to design columns have tension capacity in final condition - okay, TT

NOTE:

1.ALL LOADS ARE ULTIMATE U.O.N IN KIPS (1000 LBS.).
2.FABRICATION CAMBER SHALL BE ACHIEVED VIA STRAIGHT SEGMENTS WITH END POINTS FALLING ALONG AN ARC DEFINED BY THE END POINTS OF THE MEMBER AND THE DEVIATION FROM THEORETICAL CENTERLINE NOTED IN THE SCHEDULE AT THE MIDPOINT OF THE MEMBER WHEN COMPLETELY SHORED. SPLICES MADE TO ACHIEVE FABRICATION CAMBER SHALL BE CJP AND MEET ALL REQUIREMENTS OF SPLICES NOT SPECIFICALLY NOTED ON STRUCTURAL DRAWINGS PER S-651 EXCEPT THAT THE "RIB" SHALL BE MILLED TO BEAR ONLY.

JPMorgan Chase & Co.

KEY PLAN: EAST 48TH STREET / MADISON AVE. / PARK AVE. / EAST 47TH STREET

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No.	DESCRIPTION:	DATE:
0	100% Schematic Design	08.22.2019
1	Issue For Structural Steel Bid	09.26.2019
2	Issue For Structural Steel Addendum 1	12.19.2019
3	Issued for 50% DD	02.14.2020
4	Issued for Structural Steel Bulletin #1 (Mill Order - Seq. 1)	04.09.2020

MILL ORDER - SEQ. 1

Discrepancies must be reported immediately to the Architect before proceeding. Only figured dimensions are to be used. Contractors must check all dimensions on site. This drawing is protected by copyright.

ALL DIMENSIONS ARE SHOWN IN IMPERIAL.

CONSULTANT: Severud Associates CONSULTING ENGINEERS P.C.

PROJECT: **270 PARK AVENUE** NEW YORK, NY

DRAWING TITLE: COLUMNS GROUND TO 3RD FLOOR

SEAL & SIGNATURE:

DATE: 06/06/2019
PROJECT No: 16500
DRAWN: DS/OP
CHK: FO
REV: 4
SCALE: As indicated
DWG No: **S-601**

DOB EMPLOYEE STAMP:

DOB B-SCAN:

30" x 40" Arch E SHEET SIZE

TABLES

Table 5: Predicted Peak Torsional Velocities

Worst Case Configuration

Return Period (Years)	Peak Torsional Velocities (milli-rads/sec)				
	with hurricanes		without hurricanes		Criteria
	1.00% Damping	0.75% Damping	1.00% Damping	0.75% Damping	Residential | Office
0.1	0.26	0.30	0.26	0.30	-
1	0.59	0.68	0.58	0.67	2 .0 | 2.5
10	-	-	1.3	1.5	4 .0 | 5.0

Notes

1. Periods of 6.15, 6.14, and 4.41 seconds were used along with the indicated damping ratios.
2. Torsional Velocities are predicted at Structural Level 'Level 58' (1244.8 ft above Structural Level 'Ground').
3. Tentative torsional velocity criteria are shown for the 1- and 10-year return periods based on RWDI's experience including motion simulator experiments.
4. With the inclusion of hurricanes, it is not appropriate to consider events beyond the 1-year return period when evaluating occupant comfort. Therefore, longer return period values with hurricanes are not provided.

FIGURES

Wind Tunnel Study Model

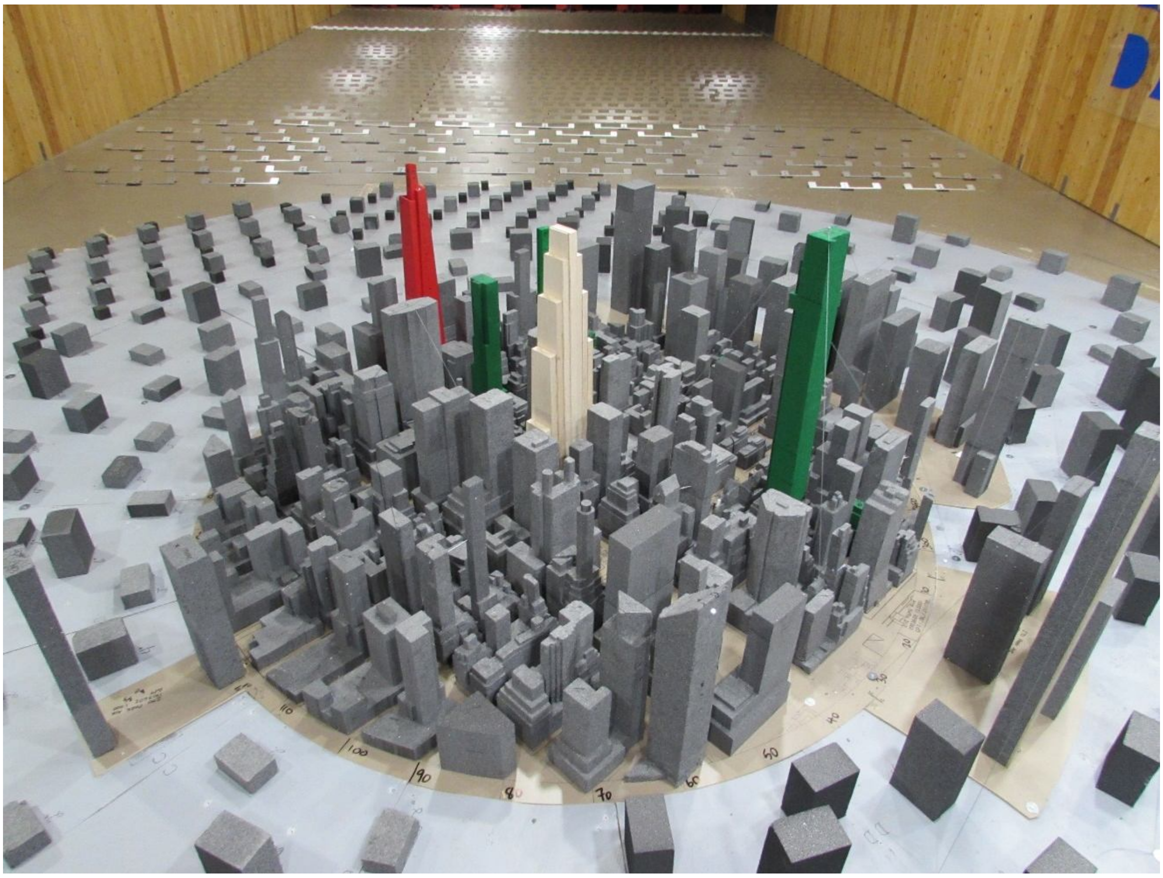
Configuration 1: Existing Configuration

270 Park Avenue – New York, NY

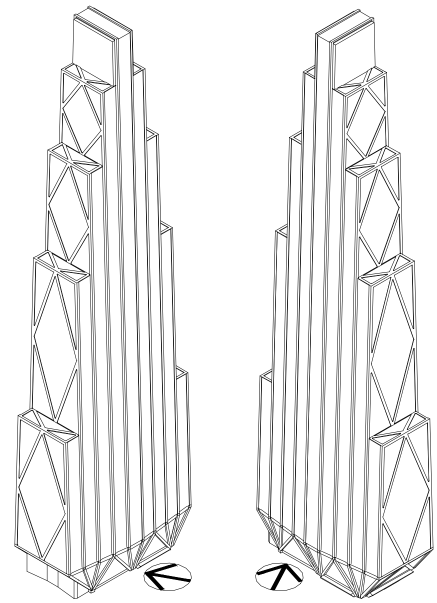
Project #1903602

Figure: 1a

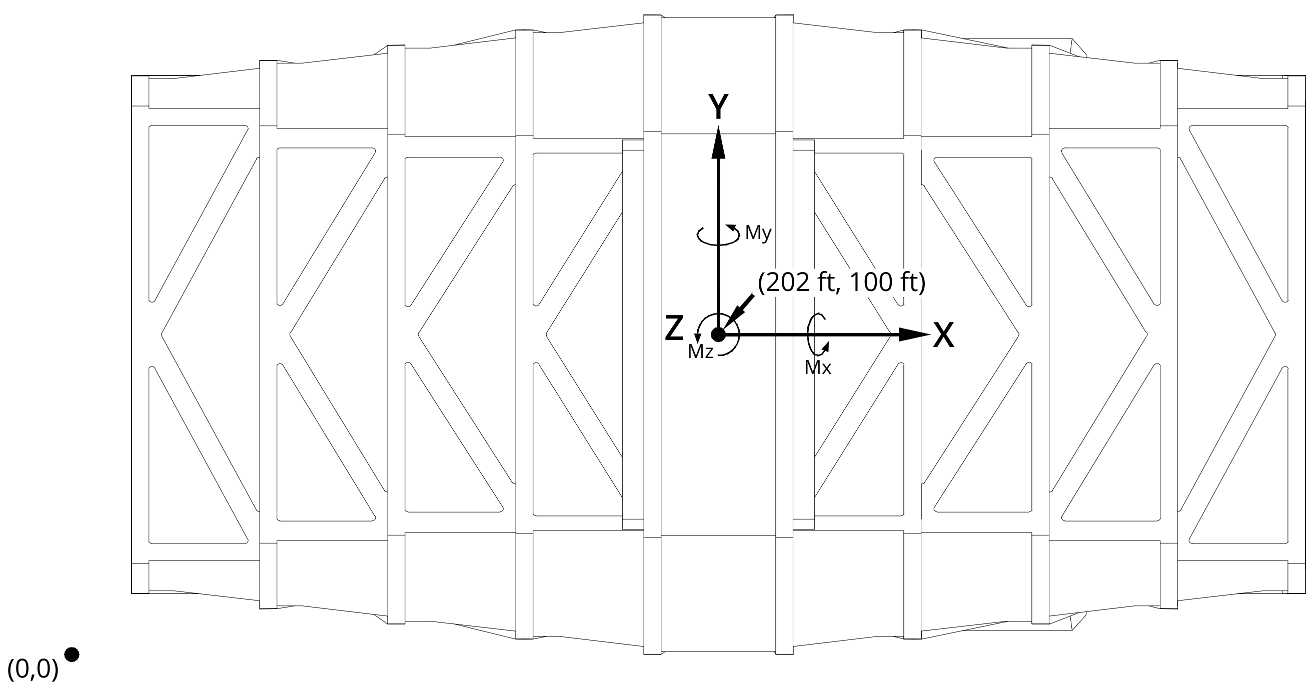
Date: November 14, 2019



Wind Tunnel Study Model Configuration 2: Future Configuration	Figure: 1b	
270 Park Avenue – New York, NY Project #1903602	Date: November 14, 2019	



Isometric View of Building



Note:
Point (0,0) indicates co-ordinate origin provided by the structural engineer.

0 30 60ft

Co-ordinate System for Structural Loading

True North

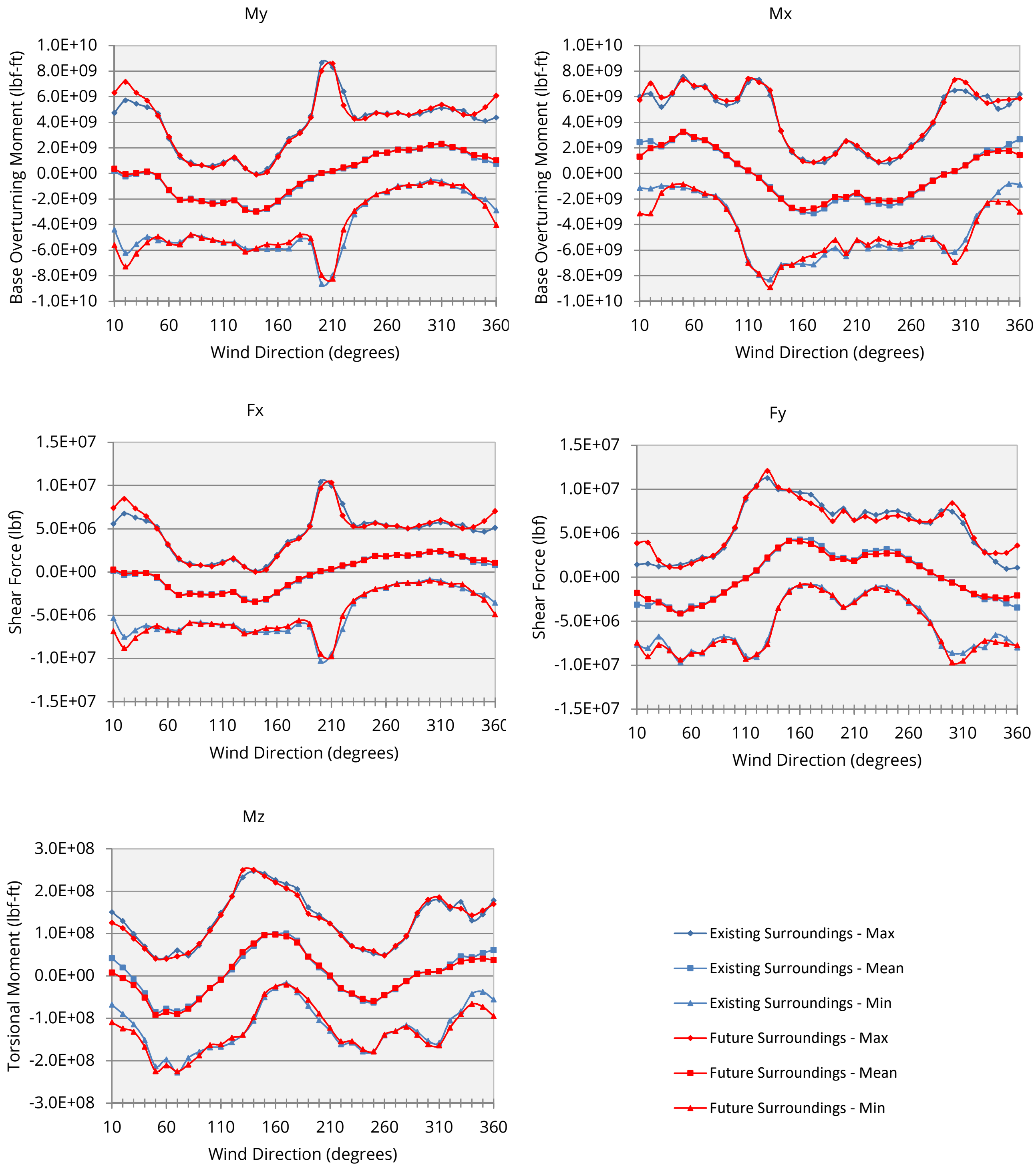
Drawn by: DPW | Figure: 4

Approx. Scale: 1"=60'

270 Park Avenue - New York, NY

Project #1903602

Date Revised: Nov. 12, 2019



Note:

1) Above loads are based on properties as provided on November 11, 2019. The natural periods were as follows:

Mode 1:	6.15 sec.
Mode 2:	6.14 sec.
Mode 3:	4.41 sec.

2) A total damping ratio of 1.5% of critical was used for structural load calculations.

3) The response plots show the maximums magnitudes for wind speeds up to and including the 50-year return period wind speed.

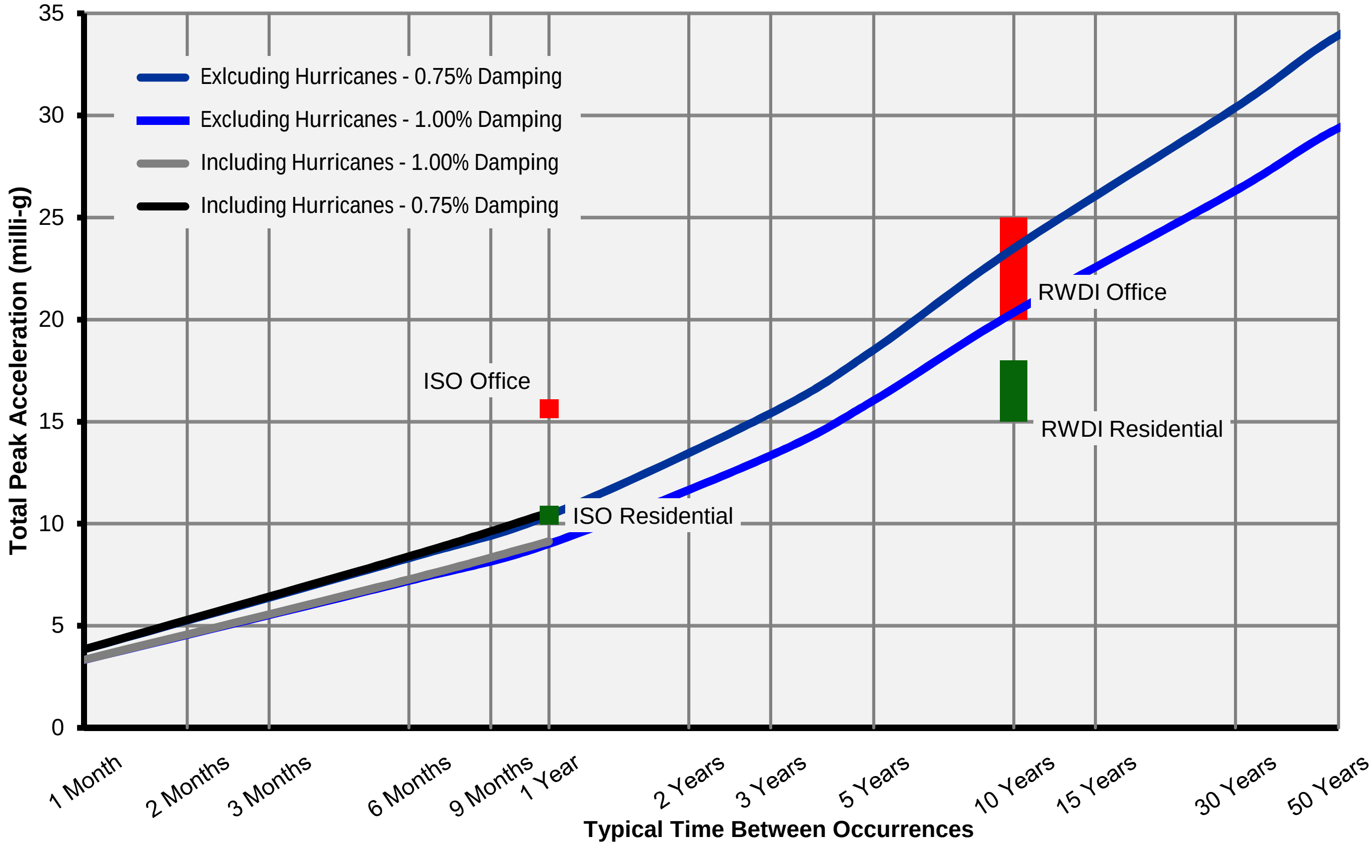
Raw Overall Base Moments, Shears and Torsion
at level 1 for 50-Year Return Period Wind Speed

Figure: 5

270 Park Avenue - New York, NY

Project #1903602

Date: Nov. 14, 2019



Return Period (Years)	Peak Accelerations(2) (milli-g) Total - [X, Y and torsional components]			
	with hurricanes (5)		without hurricanes	
	1.00% Damping	0.75% Damping	1.00% Damping	0.75% Damping
0.1	3.3 - [3.1, 3.0, 0.58]	3.9 - [3.6, 3.5, 0.66]	3.3 - [3.1, 3.0, 0.57]	3.8 - [3.6, 3.5, 0.66]
1	9.1 - [7.2, 8.0, 1.3]	11 - [8.4, 9.2, 1.5]	9.0 - [7.1, 7.9, 1.3]	10 - [8.2, 9.1, 1.5]
10	-	-	20 - [17, 18, 2.7]	23 - [20, 21, 3.2]

Notes

1. Periods of 6.15, 6.14, and 4.41 seconds were used along with the indicated damping ratios.
2. Accelerations are predicted at Structural Level 'Level 58' (1244.8 ft above Structural Level 'Ground') at a radial distance of 51 ft from the central axis of the tower (given in Figure 4).
3. ISO is the International Organization for Standardization, and the current standard (ISO 10137:2007) provides acceleration criteria for buildings at the 1-year return period. The criteria plotted on the graph have been generated based on a response-weighted interpretation of the individual modal component of the ISO criteria.
4. RWDI's criteria for residential and office buildings are based on research, experience and surveys of existing buildings, and is in agreement with general practice in North America.
5. With the inclusion of hurricanes, it is not appropriate to consider events beyond the 1-year return period when evaluating occupant comfort. Therefore, longer return period values with hurricanes are not provided.

Predicted Peak Accelerations

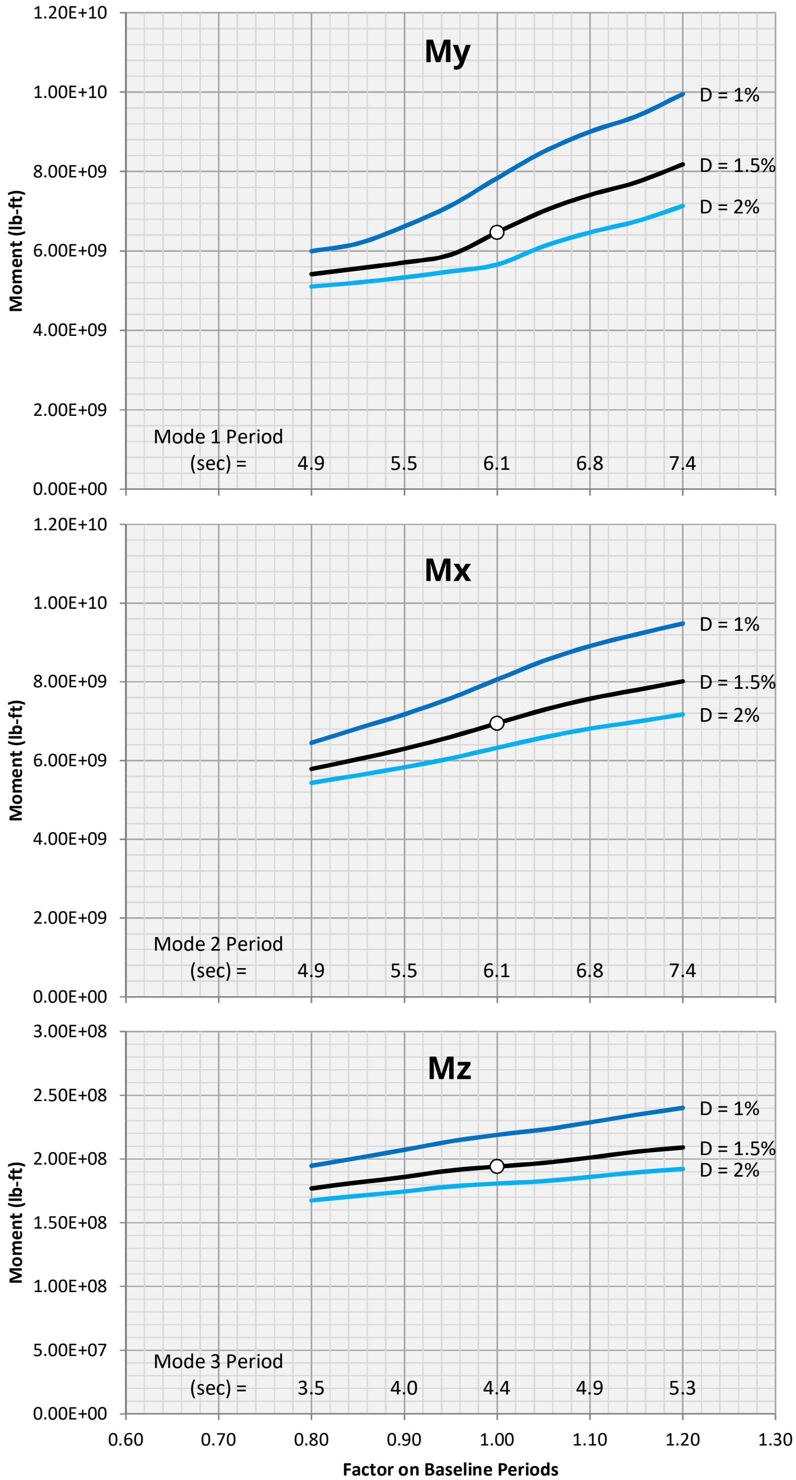
Worst Case Configuration

Figure: 6

270 Park Avenue - New York, NY

Project #1903602

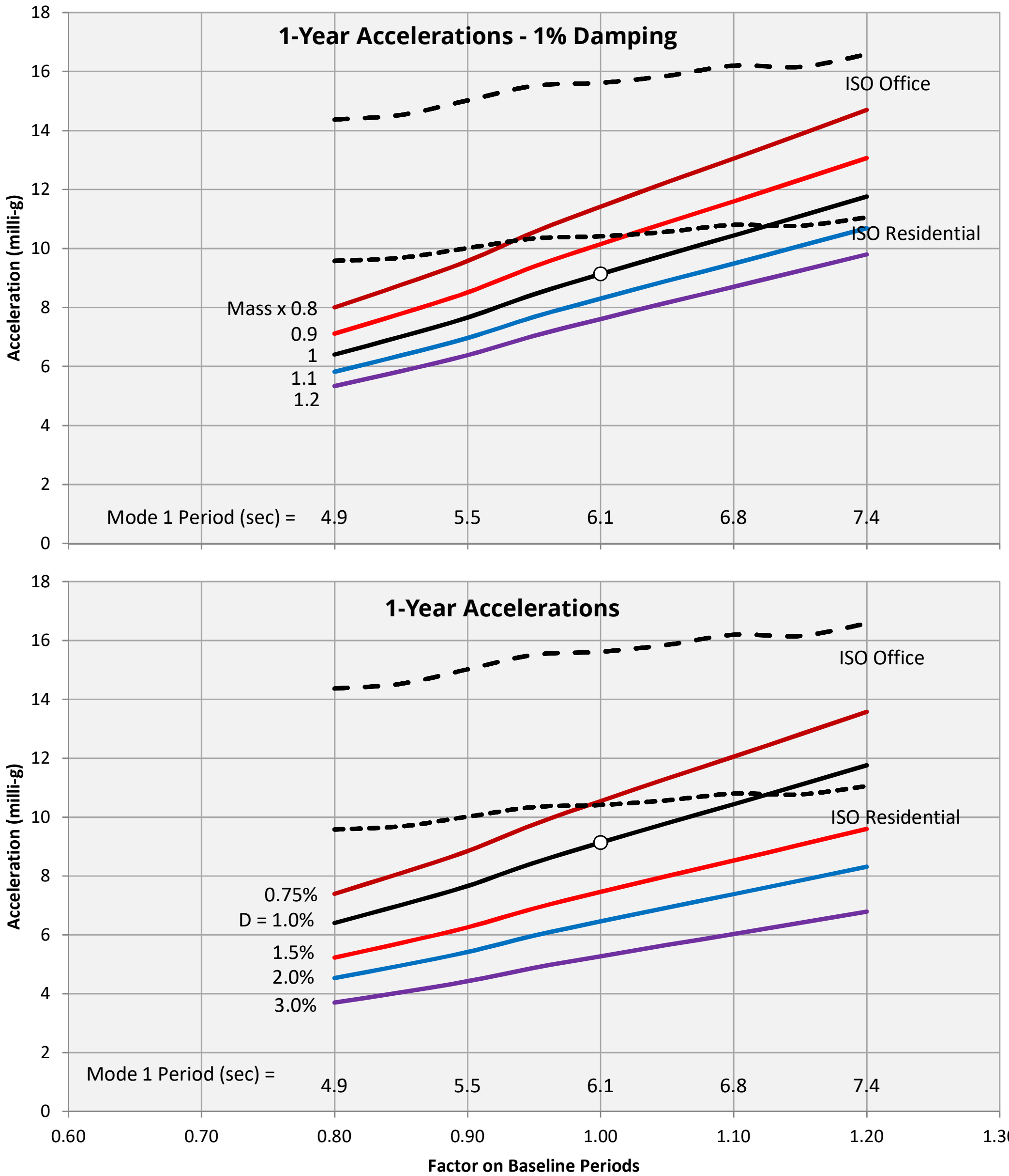
Date: November 14, 2019



Notes

1. The baseline periods for the fundamental modes are 6.15, 6.14, and 4.41 sec
2. The base loads are presented at Grade for a 50-year design wind speed (3-second gust) of 98 mph.
3. The above comparisons assume no change to the mode shapes. Some change to the curvature and coupling may be expected when mass and stiffness properties are significantly changed.

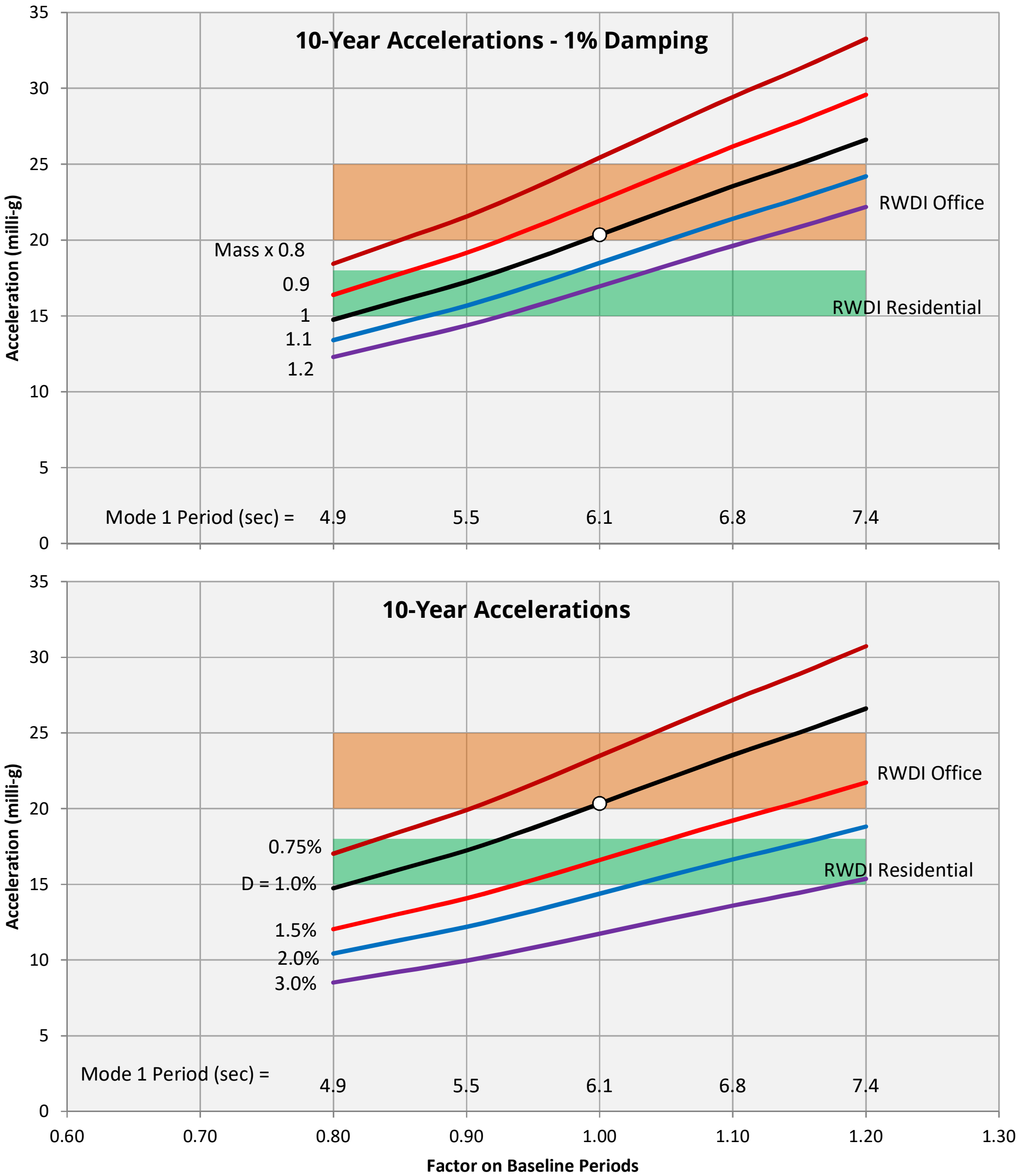
Sensitivity of Base Loads to Period and Damping **Worst Case Configuration**	Figure: 7	
270 Park Avenue - New York, NY — Project #1903602	Date: November 15, 2019	



Notes

1. The baseline periods for the fundamental modes are 6.15, 6.14, and 4.41 sec
2. Accelerations are predicted at Structural Level 'Level 58' (1244.8 ft above Structural Level 'Ground') at a radial distance of 51 ft from the central axis of the tower (given in Figure 4).
3. The above comparisons assume no change to the mode shapes. Some change to the curvature and coupling may be expected when mass and stiffness properties are significantly changed.
4. Changes in mass may be uniform changes over the entire building, but are more appropriately related to the generalized mass.

Sensitivity of 1-Year Accelerations to Mass, Period and Damping **Worst Case Configuration**	Figure: 8	
270 Park Avenue - New York, NY — Project #1903602	Date: November 15, 2019	



Notes

1. The baseline periods for the fundamental modes are 6.15, 6.14, and 4.41 sec
2. Accelerations are predicted at Structural Level 'Level 58' (1244.8 ft above Structural Level 'Ground') at a radial distance of 51 ft from the central axis of the tower (given in Figure 4).
3. The above comparisons assume no change to the mode shapes. Some change to the curvature and coupling may be expected when mass and stiffness properties are significantly changed.
4. Changes in mass may be uniform changes over the entire building, but are more appropriately related to the generalized mass.

Sensitivity of 10-Year Accelerations to Mass, Period and Damping **Worst Case Configuration**	Figure: 9	
270 Park Avenue - New York, NY — Project #1903602	Date: November 15, 2019	