STRUCTURAL PEER REVIEW STATEMENT

This structural peer review and report, dated 11 December 2017, is complete for the foundation and superstructure submission.

Structural Peer Reviewer Name: **William J. Faschan**
Leslie E. Robertson Associates

Structural Peer Reviewer Address: 40 Wall Street, FL 23
New York, NY 10005

Project Address: 11 Hoyt Street, Brooklyn, NY, Block# 157, Lot# 1

Department Application Number for Structural Work: 321197058

Structural Peer Reviewer Statement:

I, William J. Faschan, 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 11 Hoyt Street, Brooklyn, NY, Block# 157, Lot# 1, Application # 321197058 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 2008 NYC Construction Codes. The Structural Peer Review Report is attached.

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

Name William J. Faschan

STATE OF NEW YORK WILLIAM JAMES FASCHAN 058672 LICENSED PROFESSIONAL ENGINEER

Signature /Seal ______________ Date: 11 December 2017

Cc: Project Owner: Mr. Chris McCartin, 11 Hoyt Property Owner, LP
Project Registered Design Professional: Mr. Bart Sullivan, McNamara Salvia

11 Hoyt Street, Brooklyn, NY
Structural Peer Review Report

P1120

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11 December 2017

1. Introduction

At the request of Tishman Speyer Properties, Leslie E. Robertson Associates, R.L.L.P. has conducted a Structural Peer Review of the foundation and superstructure design of 11 Hoyt Street, as required by the New York City Building Code section 1617. This report presents our findings and conclusions.

The building is located at 11 Hoyt Street in Brooklyn, NY and the structural design was prepared by McNamara Salvia Structural engineers.

1.1 Documents Reviewed

We have reviewed the following:

- Structural and Architectural Drawings, listed in Appendix A.
- Structural Design Criteria by McNamara Salvia, dated 1 Mar 2017, attached to this report as Appendix B.
- The geotechnical report, *Geotechnical Engineering Study for 11 Hoyt Street, Brooklyn, Kings County, New York*, dated 23 December 2016, by Langan, attached to this report as Appendix C.
- *Preliminary Results – Wind-Induced Structural Responses, 11 Hoyt – Brooklyn, NY* dated 30 August 2017 and supporting material, by RWDI, attached to this report as Appendix D.

2. Design Criteria

We reviewed drawing S-001.01 *General Notes,* as well as the structural design criteria and geotechnical report. Our observations are discussed below.

2.1 Geotechnical Report

In reviewing drawing S-001.01 and the geotechnical report from Langan, we found that recommendations made in the report were incorporated in the drawings and design criteria.

2.2 Structural Design Criteria

For the Foundation Peer Review, we reviewed loading schedule information provided in drawing S-001.01. We find that the drawing includes a loading schedule appropriate for the types of occupancies defined by the architectural drawings and includes other information pertinent to the structural design. We recommend that the EOR review the following list for items that should also be included and addressed in the design criteria:

- Clearly identify the equivalent uniform partition loads used in the design (NYCBC 1603.1.1).
- Base shear for wind loads (NYCBC 1603.1.5).
- Maximum soil bearing capacity of soil under the mat foundation (NYCBC 1603.1.7).
- Design criteria loading of foundation walls due to static and seismic earth pressures, surcharge, and hydrostatic pressures (NYCBC 1603.1.9)

3. Superstructure Review

3.1 Architectural and Structural Drawings

We reviewed and compared the architectural drawings with structural drawings prepared by McNamara Salvia and found that the structural drawings were in general conformance with the architectural drawings.

3.2 Midas Gen Model

We independently developed a global building model using Midas Gen. The latter was generated from the structural CAD drawings provided by McNamara Salvia. Elements including the foundation mat were modeled per the information found in the structural drawings. The spring stiffness of GIEs (Ground Improvement Elements) and soil under the mat which were provided by Langan were also incorporated into the mat. Gravity and earthquake loads were defined using the information provided in the structural drawings. Finally, wind loads were defined following recommendations found in the wind tunnel report by RWDI. The model was used to review the global behavior of the building, as well as to obtain loads for the design checks of some foundation and structural elements. Figure 1, below, shows different views of the Midas Gen model.

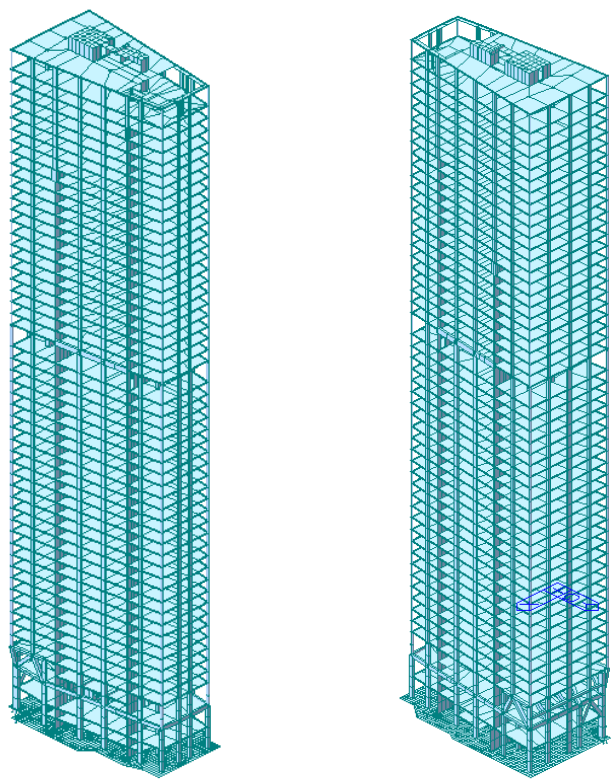
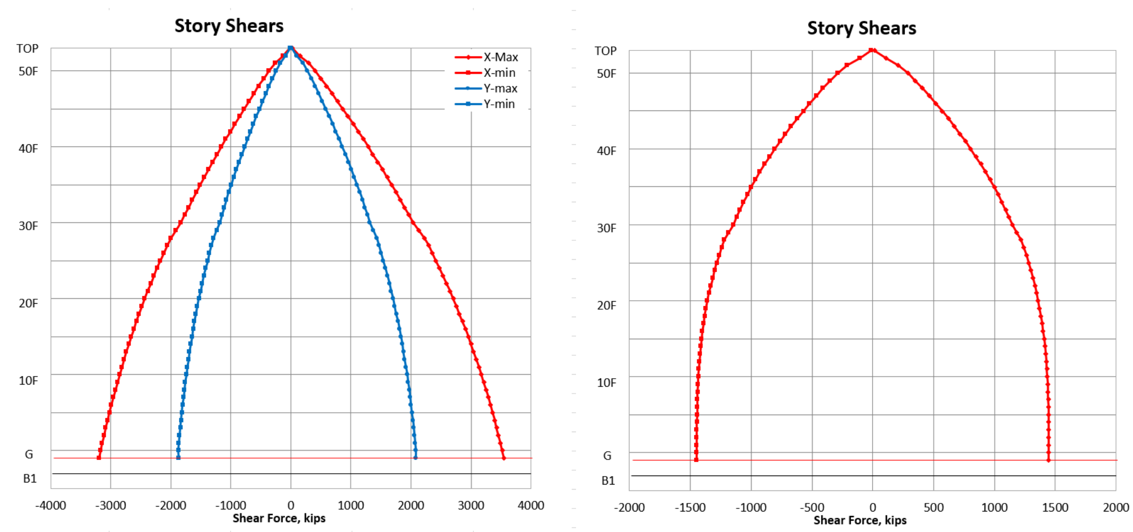


Figure 1 – Global Midas Gen Model View

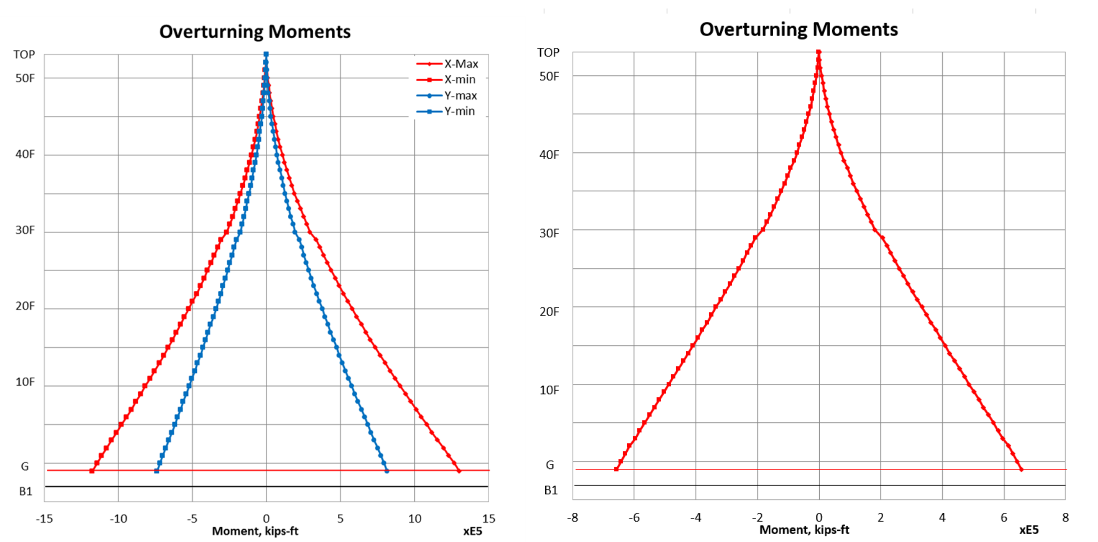
3.2.1 Base Shear Check

The Midas Gen model was also used to compare the building base shear from earthquake and wind loads shown on drawing S-001.01 and in the wind tunnel report respectively. We found the base shear from wind loads to be similar, but found an 8% reduction in the base shear from earthquake loads compared to the design base shear shown on drawing S-001.01. Figure 2 and 3 present the global shears and overturning moments taken from the Midas Gen model for wind and earthquake loads.



(a) (b)

Figure 2 – Global Model Shears (a) Wind Loads (Envelope), (b) Earthquake Loads (Envelope)



(a) (b)

Figure 3 – Global Model Overturning Moments (a) Wind Loads (Envelope), (b) Earthquake Loads (Enveloped)

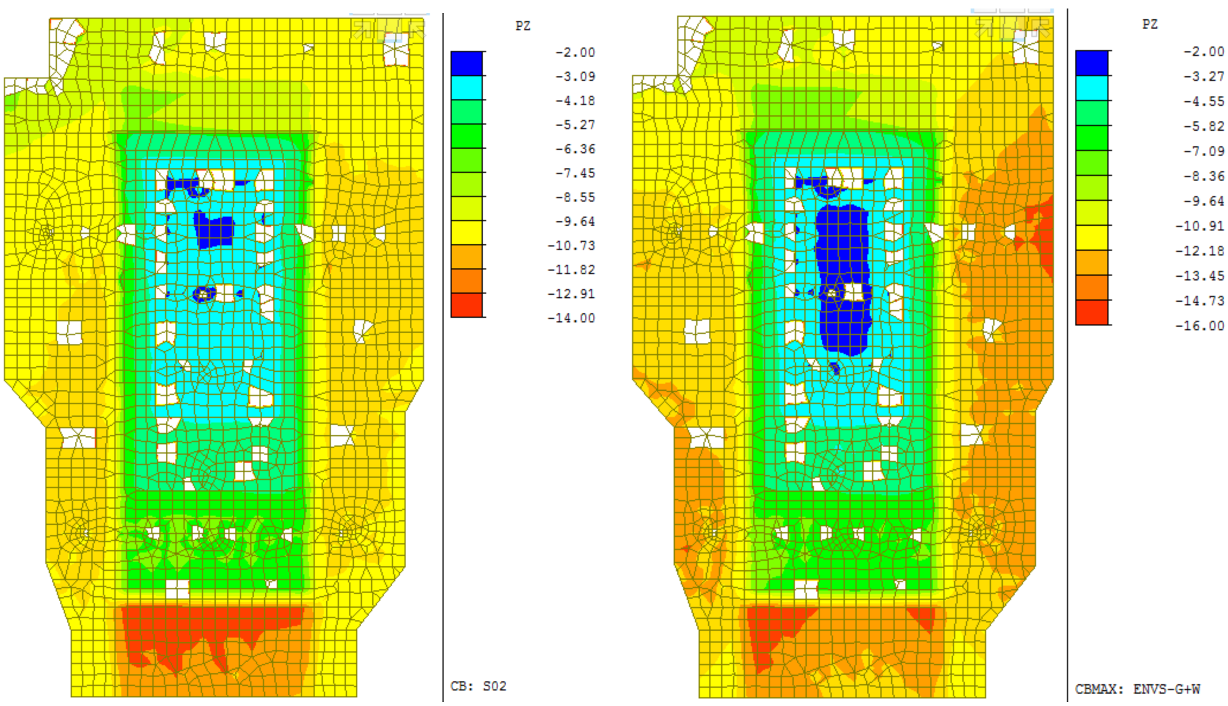
3.3 Load Path

We reviewed typical floors, walls and columns as well as the mat foundation and GIE and found they generally seemed to be well proportioned for the size and type of building. The superstructure seems to have a continuous load path.

4. Mat Foundation

4.1 Soil Pressures

The soil pressures due to gravity loads and gravity+wind loads are shown in Figure 4. The peak pressure is 14 KSF for the gravity load case and 16 KSF for wind+gravity load case.



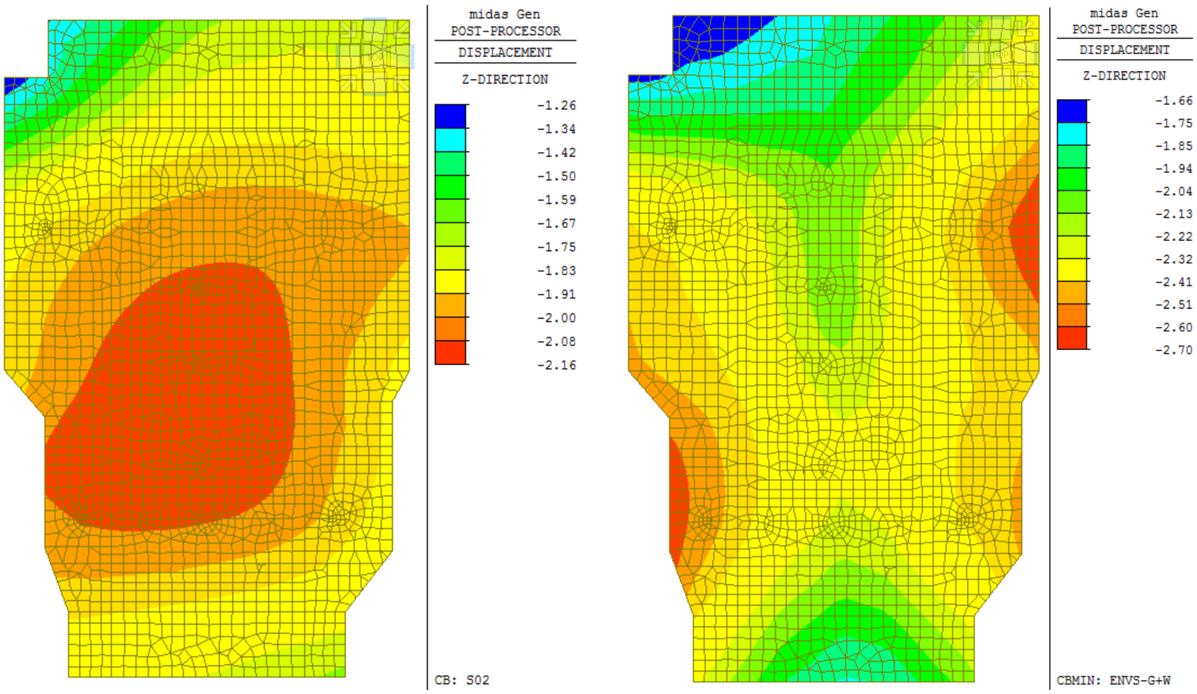
(a) (b)

Figure 4 – Soil Pressures due to (a) Gravity Loads (b) Gravity+Wind Loads(Envelope) in KSF

These 14 KSF and 16 KSF of pressure compare to a maximum expected pressure, as noted in the Langan report of 20 KSF.

4.2 Settlements

The settlements due to gravity loads and gravity+wind loads are as shown in Figure 5, with a peak settlement of 2.16 inches for gravity case and 2.70 inches for gravity+wind case.



(a) (b)

Figure 5 – Settlements due to (a) Gravity Loads (b) Gravity+Wind Loads (Envelope) in IN

4.3 Flexural Capacity

The flexural capacity of the mat foundation was reviewed using the Midas Gen model according to the information shown in drawings FO-100.00 and FO-201.00. Table 1 summarizes the DCRs of typical reinforcement in different portions of the mat foundation. For the additional reinforcement presented in drawing FO-201.00, we made some spot checks for locations indicated in Figure 6, with findings summarized in Table 2.

Table 1 – Summary of Mat Foundation Flexural Capacity Check (Typical reinforcement)

Mat Thickness	Direction	Bottom Reinforcement (DCR)	Top Reinforcement (DCR)
6'-6" NORTH	N-S	#11@12 (0.54)	#11@8 (-)
	E-W	#11@6 (0.74)	#10@12 (-)
6'-6" SOUTH	N-S	#11@12 (0.95)	#11@8 (0.35)
	E-W	#11@8 (0.90)	#11@12 (0.54)
6'-6" EAST&WEST	N-S	#11@6 (0.30)	#11@8 (0.35)
	E-W	#11@6 (0.30)	#11@12 (0.54)
10'-0"	N-S	#11@6 +#11@6 (0.56)	#11@8 (0.56)
	E-W	#11@6 +#11@6 (0.76)	#11@12 (0.84)

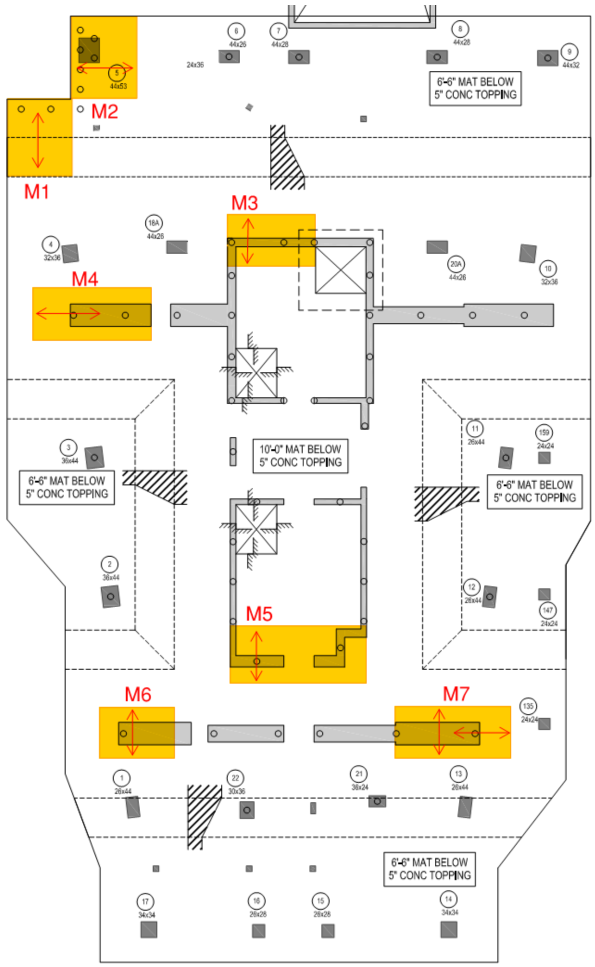


Figure 6 – Location of Mat Foundation Reinforcement Checks (from FO-100.00)

Table 2 –Summary of Mat Foundation Flexural Capacity Check (Additional reinforcement)

Location	Direction	Reinforcement	DCR
1. 6'-6" North- NW Corner	Bottom, N-S	#11@6 + #11@6 (per Note7 of FO-201.00)	0.42
2. 6'-6" North- C5	Bottom, E-W	#11@6 + #11@6	0.79
3. 10'-0" - Core Wall-N	Bottom, N-S	#11@6 + #11@6 +#11@6 + #9@12	0.90
4. 10'-0" – Core Wall-NW	Top, E-W	#11@12 + #10@12	0.87
5. 10'-0" – Core Wall-S	Bottom, N-S	#11@6 + #11@6 + #11@6 + #11@12	0.94
6. 10'-0" – Core Wall-SW	Bottom, N-S	#11@6 + #11@6 + #11@6 + #9@12	0.92
7. 10'-0" – Core Wall-SE	Bottom, E-W	11@6 + #11@6 +11@6 + #11@6	0.96
	Bottom, N-S	11@6 + #11@6 11@6 + #9@12	0.90

From the sample calculations, we believe that the flexural design of the mat foundation is adequate.

4.4 One-Way Shear Capacity

The one-way shear capacity of the mat foundation was reviewed using the information shown in drawings FO-100.00 and FO-202.00. The locations where we reviewed the one-way shear design are shown in Figure 7. Table 3 summarizes our findings.

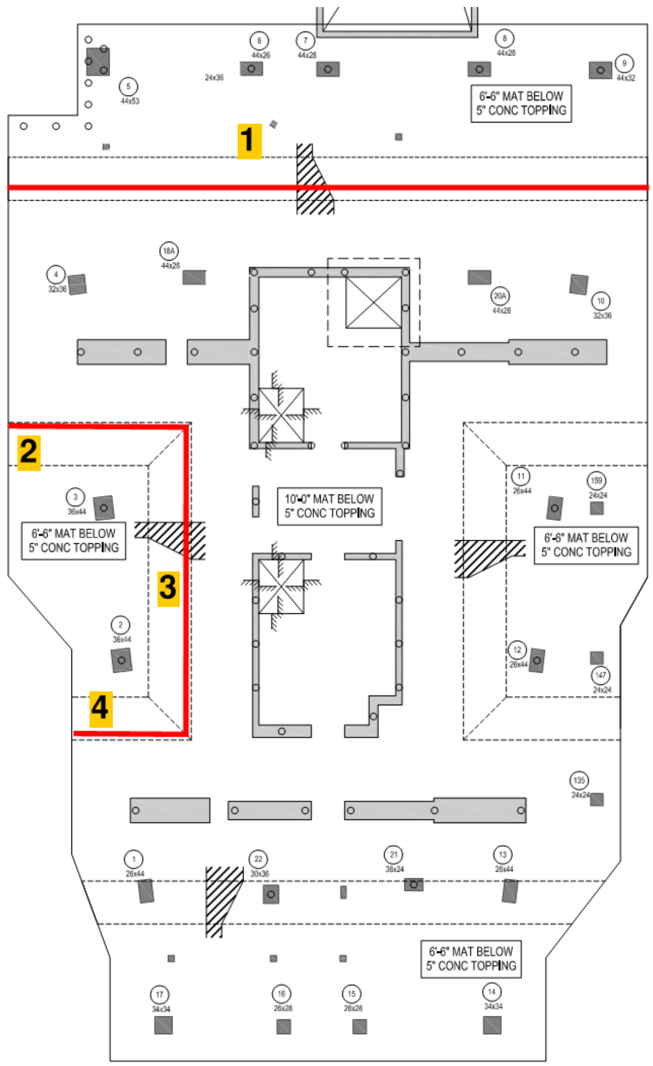


Figure 7 – Location of Mat Foundation One-Way Shear Checks (from FO-100.00)

Table 3– Summary of Mat Foundation Shear Capacity Check

Location	Reinforcement	DCR
1. N-S Direction	(2) #8 @ 12 (b=24")	0.27
2. N-S Direction	(2) #8 @ 12 (b=24")	0.29
3. E-W Direction	-	0.29
4. N-S Direction	-	0.72

The drawings do not clearly indicate the layout of shear reinforcement. We recommend additional shear reinforcement detail to further explain the arrangement. However, from the sample calculations, we believe the shear design of the mat foundation is adequate in one-way shear.

4.5 Two-Way Shear Capacity around Columns and GIEs

The two-way shear capacity of the mat foundation around the columns and GIEs was reviewed using the information shown in drawing FO-202.00. Unbalanced bending moment is considered for C5 and GIEs at the NW corner of the mat foundation, with critical perimeters shown in Figure 8. Table 4 summarizes our findings.

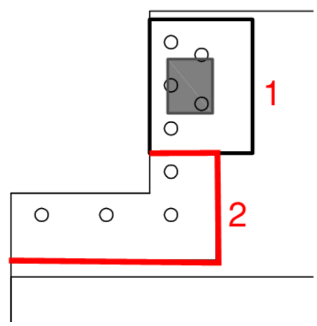


Figure 8 – Critical Perimeters for Two-Way Shear Check of C5 and GIEs at NW Corner

Table 4– Summary of Mat Foundation Punching Shear Capacity Check

Location	Reinforcement	DCR	Slab thickness
1. C5, N-S Direction	(2) #8 @ 12 (b=24”)	1.45	NG
2. Piles, N-S Direction	(2) #8 @ 12 (b=24”)	0.74	OK
3. C9	-	0.51	OK
4. C8	-	0.37	OK
5. C3	-	0.35	OK
6. C16	-	0.24	OK

We find that the slab around C5 is overstressed under the combination of unbalanced moment and shear force. The shear stress, calculated considering the moment in N-S direction, exceeds the maximum shear capacity of the 6’-6” slab. We recommend that the EOR increase the slab thickness for this local area around NW corner and revise the design of shear reinforcement considering the unbalanced moment. For other reviewed locations, we believe the punching shear capacity of slab around the columns and GIEs is adequate.

4.6 Dowels between columns and the mat foundation

Assuming the dowels between the columns and the foundation mat will be provided to match with the vertical reinforcement of the column at the lowest level, we reviewed the axial tensions in the columns to confirm whether the assumed vertical reinforcement can resist these tension forces. We found that two additional bars and one additional bar are required for C9 and C13, respectively.

5. Columns

We spot checked the axial and flexural designs of the columns using the information shown in the column schedule in Drawings S-910.00 and S-911.00 along with typical details given in Drawing S-915.00. We used the loads we obtained from our Midas Gen model and checked the designs using the Midas Gen design module. Our design check results are summarized in Table 5 to Table 11.

From the sample calculations, we find that Column C3 is modestly overstressed under combined axial and flexural loads. To account for slenderness effects, magnified second-order moment is applied in the check of slender columns. We note that Column C15 at B1 and Column C22 at 30F are overstressed. We recommend that the EOR review the design of double-height columns at B1-1F and the design of transferred columns at 30F.

We also note, reviewing the information provided on the plan drawings, that Column C21 is shown to be 29X30 at Floors 3 to 19 (S-030.01 to S-130.00), but seems to be shown as 21X38 in the column schedule on S-910.00. We recommend that the EOR check the drawings.

Table 5 - Summary of Column Mark C1 Axial and Flexural Capacity Check

Mark	Story	Dimensions		fc' (psi)	Reinforcement	DCR	
		Width(in)	Depth (in)			Moment	Compression
C1	B1	26	44	12000	10#9	0.09	0.58
	3F	28	25	12000	8#9	0.60	0.75
	5F	28	25	12000	8#8	0.08	0.74
	17F	28	25	12000	8#7	0.16	0.55
	20F	28	25	10000	8#7	0.18	0.60
	30F	14	24	10000	4#9	0.66	0.78
	38F	14	24	8000	4#9	0.43	0.62

Table 6- Summary of Column Mark C3 Axial and Flexural Capacity Check

Mark	Story	Dimensions		fc' (psi)	Reinforcement	DCR	
		Width(in)	Depth (in)			Moment	Compression
C3	B1	36	44	12000	14#11	0.05	0.61
	3F	20	44	12000	20#11	0.42	0.81
	5F	20	44	12000	16#11	0.05	0.80
	15F	20	44	12000	8#9	0.07	0.70
	20F	20	44	10000	8#9	0.09	0.73
	30F	18	36	10000	6#9	1.10	0.63
	38F	18	36	8000	6#9	0.09	0.53

Table 7 - Summary of Column Mark C5 Axial and Flexural Capacity Check

Mark	Story	Dimensions		fc' (psi)	Reinforcement	DCR	
		Width(in)	Depth (in)			Moment	Compression
C5	B1	44	53	12000	48#11	0.11	0.47
	3F	D30		12000	11#11	0.16	0.60
	5F	D26		12000	9#11	0.05	0.77
	14F	D26		12000	5#9	0.06	0.68
	20F	D26		10000	5#9	0.07	0.69
	30F	D20		10000	5#8	0.33	0.77
	38F	D20		8000	5#8	0.20	0.64

Table 8 - Summary of Column Mark C15 Axial and Flexural Capacity Check

Mark	Story	Dimensions		fc' (psi)	Reinforcement	DCR	
		Width(in)	Depth (in)			Moment	Compression
C15	B1	26	28	12000	10#11	1.20	0.72
	3F	22	28	12000	8#11	0.18	0.74
	5F	22	28	12000	6#11	0.06	0.74
	9F	22	28	12000	6#9	0.06	0.74
	20F	22	28	10000	6#9	0.08	0.60
	30F	22	16	10000	4#9	0.37	0.68
	38F	22	16	8000	4#9	0.23	0.55

Table 9 - Summary of Column Mark C18 Axial and Flexural Capacity Check

Mark	Story	Dimensions		fc' (psi)	Reinforcement	DCR	
		Width(in)	Depth (in)			Moment	Compression
C18	28F	14	44	10000	8#8	0.42	0.56
	30F	21	28	10000	6#8	0.17	0.61

Table 10 - Summary of Column Mark C19 Axial and Flexural Capacity Check

Mark	Story	Dimensions		fc' (psi)	Reinforcement	DCR	
		Width(in)	Depth (in)			Moment	Compression
C19	28F	24	30	10000	6#9	0.42	0.52
	30F	21	28	10000	6#9	0.33	0.59

Table 11 - Summary of Column Mark C22 Axial and Flexural Capacity Check

Mark	Story	Dimensions		fc' (psi)	Reinforcement	DCR	
		Width(in)	Depth (in)			Moment	Compression
C22	B1	30	36	12000	8#10	0.08	0.57
	3F	24	36	12000	8#9	0.08	0.63
	17F	24	36	12000	6#9	0.09	0.46
	20F	24	36	10000	6#9	0.10	0.50
	30F	24	36	10000	6#8	2.08	0.59
	35F	30	16	10000	6#7	0.44	0.47
	38F	30	16	8000	6#7	0.50	0.50

6. Shear Walls

We spot checked the shear wall designs of the walls identified in Figure 9 to Figure 12. The axial and flexural, and shear capacities were checked for the reinforcement shown in Drawings S-922.00 to S-929.00 and in the typical details in Drawing S-920.00. Our findings are summarized in Table 12.

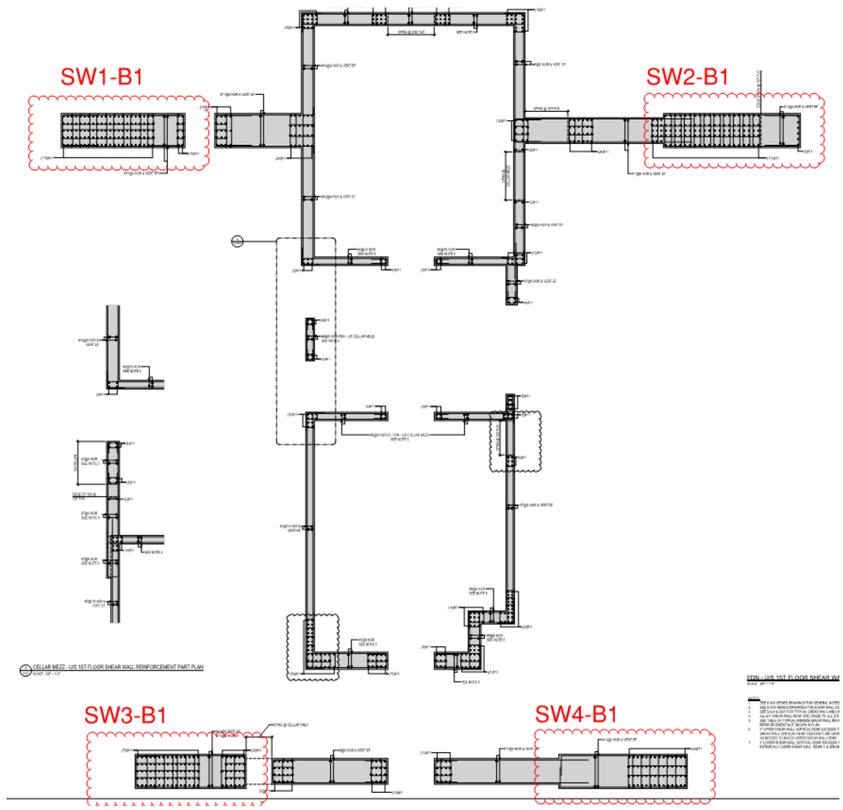


Figure 9 - Shear Walls Checked for Axial and Flexural Capacity, B1 (from S-922.00)

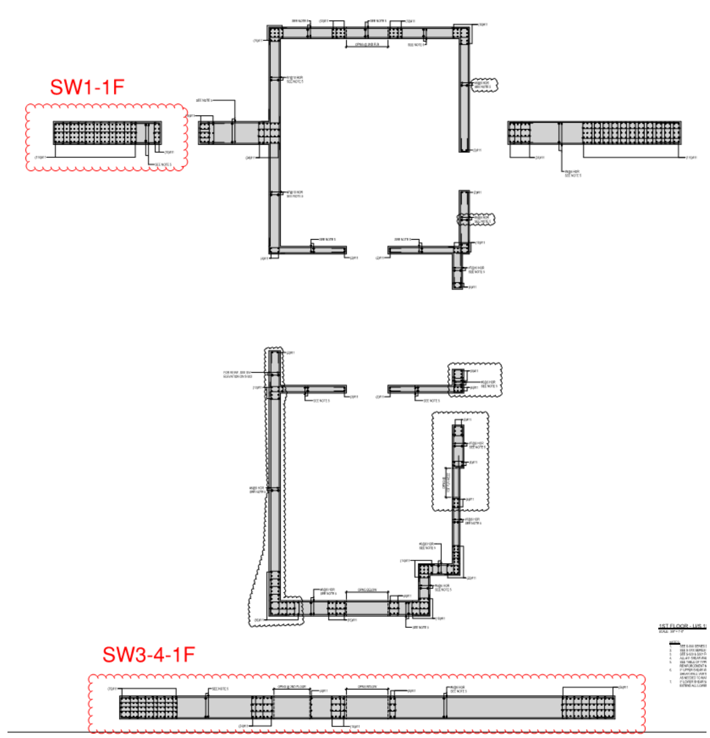


Figure 10 - Shear Walls Checked for Axial and Flexural Capacity, 1F (from S-923.00)

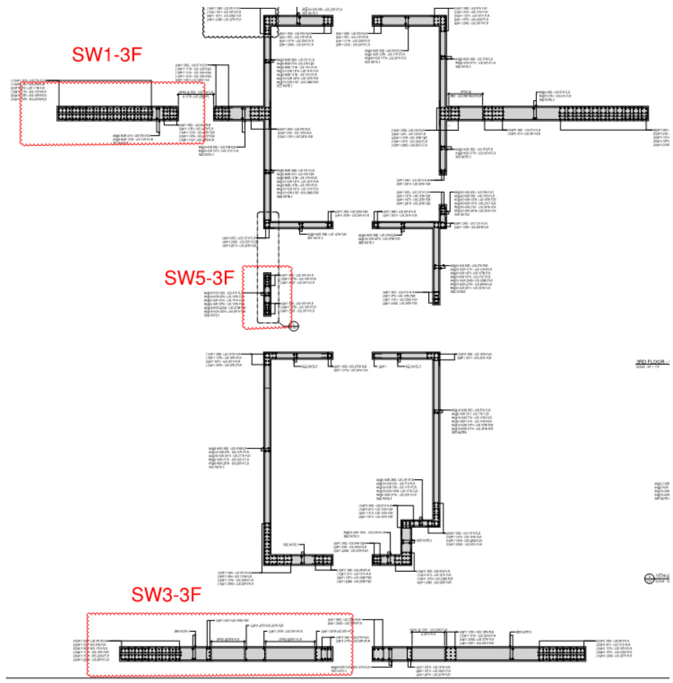


Figure 11 - Shear Walls Checked for Axial and Flexural Capacity, 3F (from S-927.00)

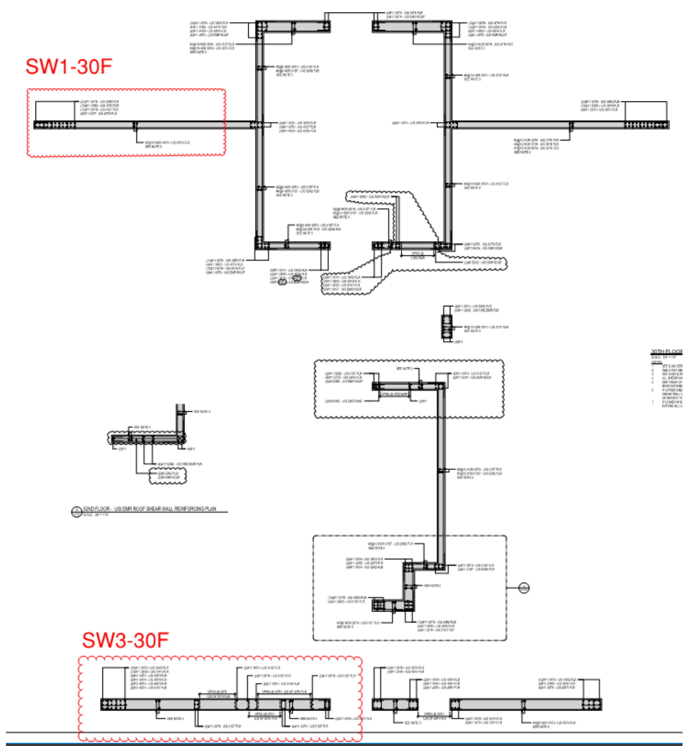


Figure 12 - Shear Walls Checked for Axial and Flexural Capacity, 30F (from S-929.00)

Table 12 - Summary of Shear Wall Axial and Flexural, and Shear Capacity Check

Story	Shear Wall Mark	Dimensions		Reinforcement	DCR			
		Width (in)	Length (in)		Compression	Tension	P-M	Shear
B1	SW1	48	173	See S-922.00	0.47	0.52	0.59	0.28
	SW2	48	192		0.38	0.42	0.48	0.53
	SW3	48	156		0.42	0.60	0.90	0.39
	SW4	48	180		0.41	0.57	0.58	0.34
1F	SW1	36	168	See S-923.00	0.56	0.59	0.60	0.43
	SW3-4	36	774		0.18	0.00	0.44	0.24
3F	SW1	24	201	See S-927.00	0.72	0.56	0.72	0.49
	SW3	24	354		0.48	0.70	0.97	0.78
	SW5	12	72		0.42	0.04	0.42	0.48
30F	SW1	12	343	See S-929.00	0.39	0.24	0.44	0.45
	SW3	18	354		0.25	0.03	0.43	0.30

Based on the spot checks, we believe that the flexural design and shear design of the shear walls are adequate.

7. Typical Floor

The flexural design of the typical floor slab was checked using our Midas Gen model and a design spreadsheet. Column punching shear was also checked at 6 locations at the 32nd floor.

7.1 Slab Reinforcement

The slab reinforcement was checked at 10 locations on the 32nd floor using information shown on Drawing S-320.00. Figure 13 shows where the capacity was checked and
Table 13 summarizes our findings.

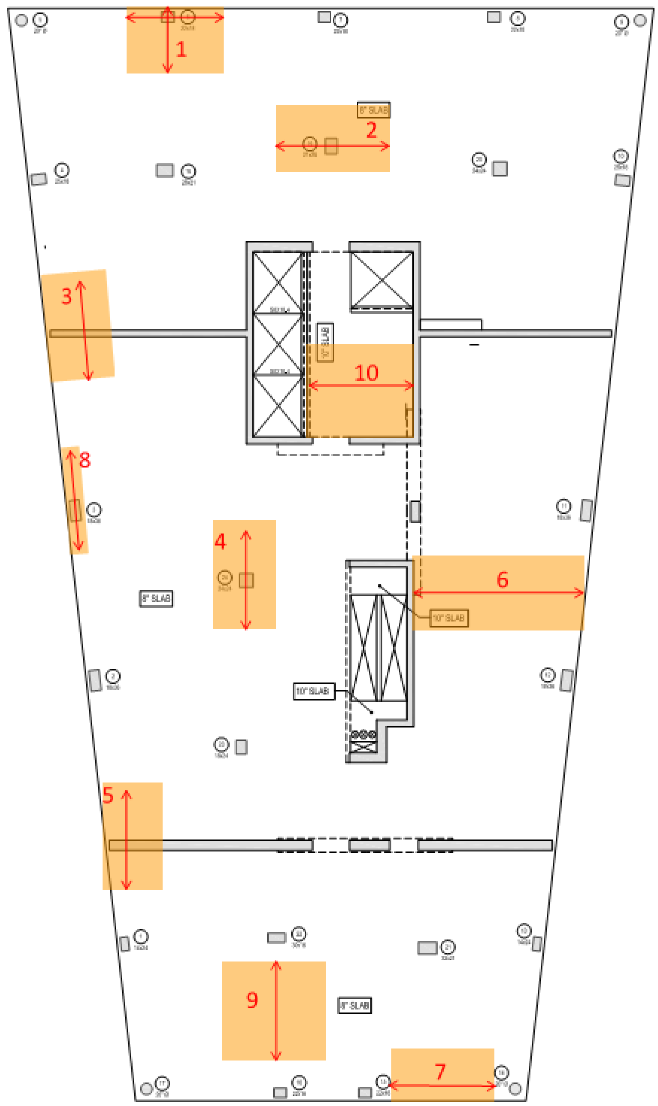


Figure 13 - Location of Slab Reinforcement Checks (from S-320.00)

Table 13 - Summary of Slab Flexural Capacity Check

Location	Direction	Reinforcement	(DCR)
1. 8’-0” C6	Top, E-W	#5@6	0.79
	Bottom, N-S	#4@12	0.98
2. 8’-0” C19	Top, E-W	#5@6	0.65
3. 8’-0” North horizontal Wall	Top, N-S	#5@6	0.92
4. 8’-0” C24	Top, N-S	#5@6	0.78
5. 8’-0” South horizontal wall	Top, N-S	#5@12	1.27
6. 8’-0” C11 to C12	Bottom, E-W	#4@12	0.97
7. 8’-0” C14 to C15	Bottom, E-W	#4@12	0.98
8. 8’-0” C3	Bottom, N-S	#4@12+#5@12	0.84
9. 8’-0” C16 to C22	Bottom, N-S	#4@12	0.98
10. 10’-0” Core	Top, E-W	#5@9	0.75
	Bottom, E-W	#4@10	0.74

From the sample calculation, we find that the flexural design of slab is adequate at most locations. However, a portion of the slab at the southern east-west wall is somewhat locally overstressed under negative bending moment in the N-S direction. We did not apply the code provisions for load re-distribution to determine whether the column bay is adequate overall; we suggest that the EOR review the flexural capacity in this area of the slab.

7.2 Punching Shear

Punching shear was checked for 6 locations at the 32nd floor using information shown on Drawing S-320.00 along with the typical studrail details given in Drawing S-940.00. Table 14 summarizes our findings.

Table 14 - Summary of Typical Floor Punching Shear Capacity Check

Location	Studrail Mark	(DCR)
1. C3	-	0.66
2. C12	-	0.57
3. C20	-	0.58
4. C23	-	0.58
5. C5	SR15	0.80
6. C24	SR1	< 0.67

From the sample calculations, we believe that the punching shear capacity of the typical slab is adequate.

8. Link Beams

The shear design of link beams was checked using a design spreadsheet and the information shown in the link beam schedule on Drawing S921.00 along with the typical link beams details provided on Drawing S-920.00. Our findings are summarized in Tables 15 and 16.

Table

Table 15 - Summary of Link Beam Shear Capacity Check

Type	Link Beam Mark - Story	Stirrups			DCR
		Legs	Size	s (in)	
Shear capacity adequate	LB1-5F	4	#5	4	0.76
	LB3-29F	3	#5	4	0.89
	LB6-38F	4	#5	4	0.97
	LB7-50F	2	#5	4	0.99
	LB8-16F	3	#5	4	0.96
	LB9-21F	5	#5	4	0.95
	LB10-1F	2	#4	8	0.56
	LB11-7F	3	#5	4	0.97
Shear capacity inadequate	LB2-20F	3	#5	4	1.11
	LB2-19F	3	#5	4	1.19
	LB2-16F	3	#5	4	1.21
	LB2-15F	3	#5	4	1.11
	LB2-4F	2	#5	8	1.20
	LB6-5F	4	#5	5	1.03
	LB6-4F	4	#5	8	1.02
	LB8-20F	2	#5	4	1.15
	LB8-18F	3	#5	4	1.01
	LB8-17F	3	#5	4	1.01
	LB9-29F	4	#5	5	1.22
	LB10-3F	2	#5	6	1.07
	LB11-31F	2	#5	6	1.16
	LB11-8F	3	#5	4	1.03
	LB11-5F	2	#5	4	1.03

Table 16 - Summary of Link Beam Maximum Shear Capacity Check

Type	Link Beam Mark - Story	Dimensions		DCR
		Width (in)	Depth (in)	
Section size inadequate	LB2-18F	24	22	1.00
	LB2-17F	24	22	1.01
	LB5-29F	36	22	1.12
	LB12-25F	36	22	1.01
	LB12-24F	36	22	1.05
	LB12-23F	36	22	1.09
	LB12-22F	42	22	1.05
	LB12-21F	42	22	1.06

According to the summary in Table 16, we find that the maximum shear capacity of LB5 at 29F and LB12 at 23F is inadequate. We recommend that the EOR enlarger the cross-section size of these members or embed the steel shapes. Where the DCRs are close to 1.0, we assume that the small exceedances are likely due to differences in our computer model and that of the EOR and are we conclude that these exceedances are not meaningful.

In Table 15, we note that some link beams are overstressed in shear and need more shear reinforcement. Again, where the overstresses are small, we attribute these to computer modeling assumptions and we do not consider the overstresses meaningful. Where the overstress appears to be more significant, we recommend that the EOR review the shear design of these overstressed members.

9. Struts and Ties

We checked the capacity of some of the strut and tie members, and associated node, shown in Drawing 2/S-935.00. Figure 14 shows the members and node that we have checked and

Table 15 summarizes our findings.

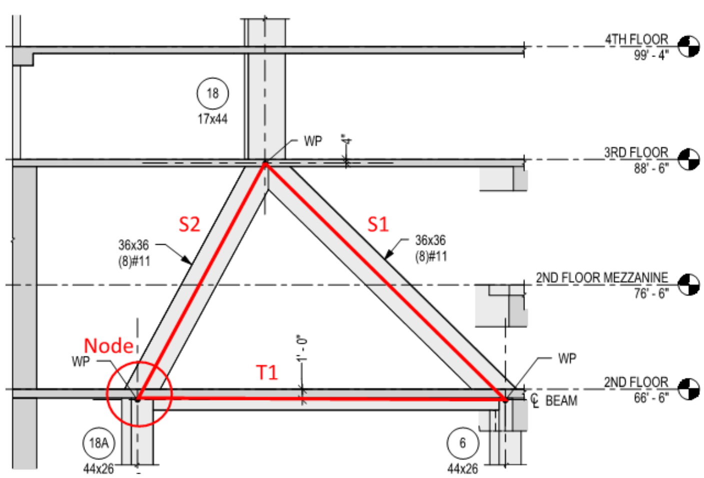


Figure 14 - Location of Strut and Tie Members and Node Check (from S-935.00)

Table 15 - Summary of Strut and Tie Members and Node Capacity Check

Mark	Reinforcement	DCR
S1	8#11	0.44
S2	8#11	0.42
T1	3#18 Top & Bottom	0.84
Node	-	0.92

From the sample calculations, we believe that the capacities of the strut, the tie members and the node are adequate.

10. Conclusions

In conclusion, we find the design of the foundation and superstructure of 11 Hoyt Street to be in general conformance with the structural and foundation design provisions of the Building Code. Where we recommend that the EOR revisit a particular aspect of the design, we have described our recommendation in the body of this report.

The opinions expressed in this report represent our professional view, based on the information made available to us. In developing these opinions, we have exercised a degree of care and skill commensurate with that exercised by professional engineers licensed in the State of New York for similar types of projects. No other warranty, expressed or implied, is made as to the professional advice included in this report.

Respectfully submitted,
LESLIE E. ROBERTSON ASSOCIATES, R.L.L.P.

Sangho Han

WILLIAM J. FASCHAN
Partner-In-Charge

SANGHO HAN
Project Manager

APPENDIX A

List of Reviewed Drawings

STRUCTURAL DRAWINGS LIST – Issued for 85% CD on 10 November 2017

DRAWING No.	**DRAWING TITLE**
FO-001	SITE PLAN
FO-100	FOUNDATION FRAMING PLAN
FO-200	GROUND IMPROVEMENT ELEMENT DETAILS
FO-201	MAT FOUNDATION REINFORCEMENT
FO-202	MAT FOUNDATION SHEAR REINFORCEMENT
FO-203	FOUNDATION DETAILS
FO-204	TYPICAL FOUNDATION DETAILS
FO-205	TYPICAL FOUNDATION DETAILS
FO-206	TYPICAL FOUNDATION DETAILS
FO-210	REINF DEV LENGTH/LAP SPLICE SCHEDULE
FO-300	FOUNDATION SECTIONS
FO-301	FOUNDATION SECTIONS
FO-302	FOUNDATION SECTIONS
S-000	COVER SHEET
S-001	GENERAL NOTES
S-002	PLAN NOTES AND LEGENDS
S-006	CELLAR MEZZANINE FRAMING PLAN
S-010	1ST FLOOR FRAMING PLAN (DEMO)
S-011	1ST FLOOR FRAMING PLAN
S-013	1ST FLOOR MEZZANINE FRAMING PLAN
S-020	2ND FLOOR FRAMING PLAN
S-022	2ND FLOOR MEZZANINE FRAMING PLAN
S-030	3RD FLOOR FRAMING PLAN
S-031	3RD FLOOR GENERAL ARRANGEMENT PLAN
S-040	4TH FLOOR FRAMING PLAN
S-050	5TH - 12TH FLOOR FRAMING PLAN
S-130	13TH - 21ST FLOOR FRAMING PLAN
S-220	22ND - 28TH FLOOR FRAMING PLAN
S-290	29TH FLOOR FRAMING PLAN
S-300	30TH FLOOR FRAMING PLAN
S-310	31ST FLOOR FRAMING PLAN
S-320	32ND - 38TH FLOOR FRAMING PLAN
S-390	39TH FLOOR FRAMING PLAN
S-400	40TH - 44TH FLOOR FRAMING PLAN
S-450	45TH FLOOR FRAMING PLAN

STRUCTURAL DRAWINGS – Issued for 85% CD on 10 November 2017 – Continued

DRAWING No.	DRAWING TITLE
S-460	46TH - 49TH FLOOR FRAMING PLAN
S-500	50TH FLOOR FRAMING PLAN
S-510	51ST FLOOR FRAMING PLAN
S-520	52ND MECHANICAL FRAMING PLAN
S-530	53RD EMR FRAMING PLAN
S-540	EMR ROOF FRAMING PLAN
S-910	COLUMN SCHEDULE
S-911	COLUMN SCHEDULE
S-912	COLUMN SCHEDULE
S-915	TYPICAL COLUMN DETAILS
S-920	TYPICAL SHEAR WALLS AND LINK BEAM DETAILS
S-921	LINK BEAM SCHEDULE
S-922	SHEAR WALL REINFORCING PLAN
S-923	SHEAR WALL REINFORCING PLAN
S-924	SHEAR WALL REINFORCING PLAN
S-925	SHEAR WALL REINFORCING PLAN
S-926	SHEAR WALL REINFORCING PLAN
S-927	SHEAR WALL REINFORCING PLAN
S-928	SHEAR WALL REINFORCING PLAN
S-930	SHEAR WALL ELEVATIONS
S-931	SHEAR WALL ELEVATIONS
S-932	SHEAR WALL ELEVATIONS
S-933	SHEAR WALL ELEVATIONS
S-934	SHEAR WALL ELEVATIONS
S-935	A FRAME ELEVATIONS
S-940	TYPICAL CONCRETE DETAILS
S-941	TYPICAL CONCRETE DETAILS
S-942	TYPICAL CONCRETE DETAILS
S-943	TYPICAL CONCRETE DETAILS
S-944	TYPICAL CONCRETE DETAILS
S-950	TYPICAL STEEL DETAILS
S-951	TYPICAL STEEL DETAILS
S-952	TYPICAL STEEL DETAILS
S-953	TYPICAL STEEL DETAILS
S-954	TYPICAL STEEL DETAILS
S-960	TYPICAL MASONRY DETAILS

STRUCTURAL DRAWINGS – Issued for 85% CD on 10 November 2017 – Continued

DRAWING No.	DRAWING TITLE
S-961	TYPICAL MASONRY DETAILS
S-970	SECTION
S-971	SECTION
S-972	SECTION
S-980	TYPICAL STAIR DETAILS

ARCHITECTURAL DRAWING LIST – Issued for 100% DD on 21 June 2017

DRAWING No.	DRAWING TITLE
A-001.00	DRAWING LIST
A-002.00	GENERAL NOTES 2014
A-003.00	ALTERATION NOTES
A-004.00	ABBREVIATIONS, SYMBOLS AND MATERIALS LEGEND
A-005.00	PARTITION SCHEDULE
A-006.00	DOOR SCHEDULE
A-007.00	DOOR DETAILS
A-008.00	MISC. DETAILS
A-009.00	ELEVATOR
A-010.00	FIRM MAPS (FORMERLY A-009)
A-011.00	SURVEY (FORMERLY A-004)
A-012.00	ADAPTABILITY NOTES (FORMERLY A-003)
A-050.00	3D IMAGES - MODEL PHOTOS
A-051.00	3D IMAGES - AXONOMETRICS
A-081.00	*CELLAR FLOOR PLAN-EXISTING*
A-082.00	*CELLAR FLOOR MEZZANINE PLAN-EXISTING*
A-083.00	*FIRST FLOOR PLAN EXISTING*
A-084.00	*SECOND FLOOR PLAN EXISTING*
A-085.00	*THIRD FLOOR PLAN EXISTING*
A-086.00	*FOURTH FLOOR PLAN EXISTING*
A-087.00	*FIFTH FLOOR PLAN EXISTING*
A-088.00	*ROOF FLOOR PLAN EXISTING*
A-095.00	TOWER GEOMETRY
A-096.00	BASE GEOMETRY
A-100.00	SITE PLAN
A-101.00	CELLAR FLOOR PLAN (FORMERLY A-151)
A-102.00	CELLAR SLAB RETENTION
A-105.00	CELLAR FLOOR MEZZANINE PLAN (FORMERLY A-152)
A-105.00	CELLAR FLOOR MEZZANINE PLAN (FORMERLY A-152)
A-110.00	FIRST FLOOR PLAN (FORMERLY A-153)
A-111.00	FIRST FLOOR SLAB RETENTION
A-113.00	FIRST FLOOR MEZZANINE PLAN
A-115.00	SECOND FLOOR PLAN
A-116.00	2ND FLOOR MEZZANINE
A-118.00	TIER 1 - 3RD AND 4TH FLOOR PLAN
A-120.00	TIER 2A-FLOORS 5-6 PLANS TIER 2B FLOORS 7-9 PLANS

ARCHITECTURAL DRAWING LIST – Issued for 100% DD on 21 June 2017 – Continued

DRAWING No.	DRAWING TITLE
A-120.00	TIER 2A-FLOORS 5-6 PLANS TIER 2B FLOORS 7-9 PLANS
A-121.00	TIER 2C- FLOORS 10-12 PLANS
A-130.00	TIER 3A-FLOORS 13-15 PLANS TIER 3B-FLOORS 16-18 PLANS
A-131.00	TIER 3C- FLOORS 19-21 PLANS
A-140.00	TIER 4A-FLOORS 22-24 PLANS TIER 4B-FLOORS 25-27 PLANS
A-141.00	TIER 4C- FLOOR 28 PLAN
A-145.00	29TH FLOOR PLAN
A-148.00	TIER 5A- FLOORS 30-31
A-152.00	TIER 5A- FLOOR 32 PLAN TIER 5B- FLOORS 33-35 PLANS
A-153.00	TIER 5C- FLOORS 36-38 PLANS
A-154.00	SECOND FLOOR PLAN ALTERATION
A-155.00	THIRD FLOOR PLAN ALTERATION
A-156.00	FOURTH FLOOR PLAN ALTERATION
A-157.00	FIFTH FLOOR PLAN ALTERATION
A-158.00	ROOF PLAN ALTERATION
A-160.00	TIER 6A-FLOORS 39-41 PLANS TIER 6B-FLOORS 42-44 PLANS
A-161.00	TIER 6C-FLOORS 45-47 PLANS TIER 6D-FLOORS 48-49 PLANS
A-170.00	TIER 7 - FLOOR 50 PLAN
A-180.00	51ST AND 52ND FLOOR PLAN
A-185.00	53RD AND 54TH FLOOR PLAN
A-220.00	TIER 2 TYP. FLOORS 5-12 RCP
A-222.00	TIER 3 TYP. FLOORS 13-21 RCP
A-224.00	TIER 4 TYP. FLOORS 22-28 RCP
A-228.00	TIER 5 TYP. FLOORS 30-38 RCP
A-301.00	NORTH ELEVATION
A-302.00	EAST ELEVATION
A-303.00	SOUTH ELEVATION
A-304.00	WEST ELEVATION
A-400.00	SECTION AA
A-401.00	SECTION BB
A-402.00	CELLAR SECTIONS
A-403.00	CELLAR SECTIONS 2
A-404.00	BASE SECTION-ELEVATIONS
A-405.00	BASE SECTION-ELEVATIONS
A-406.00	BASE SECTION-ELEVATIONS
A-407.00	BASE SECTION-ELEVATIONS

ARCHITECTURAL DRAWING LIST – Issued for 100% DD on 21 June 2017 – Continued

DRAWING No.	DRAWING TITLE
A-450.00	STAIR A AND B- SHEET 1
A-451.00	STAIR A AND B- SHEET 2
A-500.00	WALL SECTIONS
A-501.00	WALL SECTIONS
A-502.00	WALL SECTIONS
A-503.00	WALL SECTIONS
A-504.00	WALL SECTIONS
A-510.00	LVL2 ENVELOPE SCHEDULE
A-511.00	TOWER ENVELOPE SCHEDULE ALT
A-512.00	TOWER ENVELOPE PLANS
A-513.00	TOWER ENVELOPE PLANS
A-514.00	TOWER ENVELOPE PLANS
A-515.00	PRECAST SECTION GEOMETRY
A-516.00	TALL DONUTS - GROUP 5A
A-517.00	TALL DONUTS - GROUP 5B
A-518.00	TYPICAL DONUTS - GROUP 0
A-519.00	TYPICAL DONUTS - GROUP 1
A-520.00	TYPICAL DONUTS - GROUP 2
A-521.00	TYPICAL DONUTS - GROUP 3
A-522.00	TYPICAL DONUTS - GROUP 4
A-523.00	SOUTH WEST CORNER DONUTS
A-524.00	NORTH WEST CORNER DONUTS
A-525.00	NORTH EAST CORNER DONUTS
A-526.00	SOUTH EAST CORNER DONUTS
A-530.00	ENVELOPE SCHEDULES
A-531.00	LVL01 LOBBY ENVOLOPE SCHEDULE
A-533.00	LVL01 PRECAST CURTAINS SCHEDULE
A-534.00	LVL01 WALL CEILING INTERFACE
A-540.00	PRECAST CONCRETE SAMPLE PHOTOS
A-541.00	GATES
A-542.00	RETAIL FAÇADE SCHEDULE
A-550.00	ENVELOPE DETAILS
A-551.00	ENVELOPE DETAILS
A-552.00	ENVELOPE DETAILS

ARCHITECTURAL DRAWING LIST – Issued for 100% DD on 21 June 2017 – Continued

DRAWING No.	DRAWING TITLE
A-560.00	SCALLOP BAY DETAILS
A-561.00	ENVELOPE DETAILS
A-562.00	INTERIOR BAY MODEL
A-650.00	BATHROOMS
A-651.00	BATHROOMS
A-652.00	BATHROOMS
A-653.00	BATHROOM VARIATIONS 1
A-654.00	BATHROOM VARIATIONS 2
A-700.00	KITCHEN DETAILS
A-701.00	KITCHEN TYPES 1
A-702.00	KITCHEN TYPES 2
A-703.00	KITCHEN VARIATIONS 1
A-704.00	KITCHEN VARIATIONS 2
A-815.00	MOUNTING DEVICE DETAILS
A-816.00	HEAT PUMP DETAILS
A-817.00	REFUSE ROOM DETAILS

APPENDIX B

Structural Design Criteria

11 Hoyt St.

Brooklyn, NY

STRUCTURAL DESIGN

CRITERIA

Prepared For:
Tishman Speyer

Prepared By

McNamara•Salvia

Bart Sullivan, PE
Principal

Date: Mar 1, 2017

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1 CODES

1.1 Statutory Codes

- New York City Building Code 2014
- ACI-318-11 Building Code Requirements for Structural Concrete and Commentary
- ACI-530-11 Building Code Requirements and Specifications for Masonry Structures and Related Commentaries
- AISC 360-10 – Specification for Structural Steel Buildings

1.2 Design References

- AISC-13th ed. LRFD Manual of Steel Construction
- ASCE7-05 Minimum Design Loads for Buildings and Other Structures

2 DESIGN LOADS

2.1 Dead Load

Dead Loads are calculated based on particular material unit weights for each structural material.

2.2 Super-imposed Dead Load

Occupancy	Distributed Floor Load
Retail Areas (finishes, partitions, ceilings)	30 psf
Public Corridors, Lobby, Amenity (finishes, partitions, ceilings)	30 psf
Mechanical Floors (equipment pads, partitions)	30 psf
Residential Areas (finishes, partitions, ceilings)	15 psf
Terrace Spaces (finishes, landscaping)	150 psf
Ground Floor Drive Lane (paver finish)	50psf
Loading Dock (misc)	10psf

2.3 Façade Load

Precast façade unit weight:	600psf for preliminary calculations *(final precast weights TBD based on selected manufacturer)*
Storefront:	25psf
"Blank wall" façade unit weight:	65psf

2.4 Live Loads

Occupancy	Distributed Floor Load
Retail Areas	100 psf
Public Corridors, Lobby, Amenity	100 psf
Mechanical Floors	150 psf
Residential Areas	40 psf
Private Terrace Spaces	60 psf
Public Terrace Spaces	100 psf
Ground Floor Drive Lane	300psf
Loading Dock	150psf or HS20-44

2.5 Seismic Loads

The building will be designed to resist seismic loads as per the New York City Building Code based the following spectral accelerations:

$S_s = 0.281g$

$S_1 = 0.073g$

Building and site specific seismic parameters are as follows:

Response Modification Factor, R = 5.0

Importance Factor, I = 1.0

Deflection Amplification Factor, Cd = 4.5

Soil Class, S = C

The building will be designed for Seismic Design Category B.

2.6 Wind Loads

Wind loads are based on wind tunnel testing performed by RWDI. Loads are limited to a minimum of 80% of code prescribed forces Exposure C.

3 DESIGN CRITERIA LIMITS

3.1 Vertical Deflections

Concrete Floor Systems:

Total Long Term Deflection Post-Partition Installation: L/480 interior spans
L/240 cantilever spans

Steel Floor Systems:

Live Load: L/360 interior spans
L/180 cantilever spans

Post-Composite: L/240 interior spans
L/120 cantilever spans

Slab edges supporting façade will also be limited to a maximum of 0.5”.

3.2 Horizontal Deflections

<u>Wind</u>, 25-year return period:

Inter-story drift limit: h/400
where h = story height

<u>Seismic</u>, including deflection amplification factor:

Inter-story drift limit: h/50
where h = story height

3.3 Lateral Accelerations

There are no code prescribed limits on lateral acceleration as there is no safety issue associated with them. Based on current professional practice, the lateral accelerations will be limited to 18 milli-g at occupied floors for a 10-year return period wind.

4 STRUCTURAL MATERIALS

4.1 Concrete

CIP Concrete walls and Columns	12,000 to 8,000 psi
CIP Framed Slabs and Beams	8,600 psi to 6,000 psi
Mat Slab	10,000 psi
Slab on metal deck	3,500psi

Grout: Grout shall be non-metallic, non-shrink grout and match the compressive strength of the element being supported

4.2 Concrete Reinforcement

Deformed reinforcing steel bars:	ASTM A615 Gr. 60 or Gr. 75
Reinforcing bars subjected to welding:	ASTM A706 Gr. 60
Welded Wire Fabric	ASTM 185 Gr. 60

4.3 Masonry

CMU Block walls	1500 psi
Grout	2000 psi

4.4 Structural Steel

Wide Flange Sections:	ASTM A992 Gr. 50
Rectangular HSS Sections:	ASTM A500 Gr. B
Circular HSS Sections:	ASTM A500 Gr. C
Plate	ASTM A572 Gr. 50
Angles	ASTM A36
S Sections	ASTM A36

APPENDIX C

Geotechnical Report

GEOTECHNICAL ENGINEERING STUDY

for

11 Hoyt Street
Brooklyn, Kings County, New York

Prepared For:

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23 December 2016
170379401

LANGAN

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INTRODUCTION

This report presents the results of our geotechnical engineering study and provides geotechnical recommendations for the design and construction of a proposed tower at 11 Hoyt Street in Brooklyn, New York. We performed our services in general accordance with our 26 January 2016 contract. Our recommendations are in accordance with the provisions of the New York City Building Code.

Our understanding of the project is based on discussions with the design team, review of the concept design documents provided to us, and our ongoing work on the project. The project architect (Studio/Gang Architects, SGA) provided architectural information, and the project structural engineer (McNamara Salvia Structural Engineers, MSSE) provided structural information.

Elevations herein are with respect to the North American Vertical Datum of 1988 (NAVD88) and were estimated from a progress topographical survey by Langan (13 January 2016).

SITE DESCRIPTION

The site is at 11 Hoyt Street (Block No. 157, Lot No. 1) in downtown Brooklyn (Drawing No. 1). The site is bound by five- to eight-story structures on the north, Elm Place on the east, Hoyt Street on the west, and Livingston Street on the south. The site has a footprint of about 58,800 square feet, excluding an easement for the neighboring building at the northeast corner of the site (1 Hoyt Street); the dimensions of the easement vary with elevation as shown by the differing hatched areas on Drawing No. 2.

A six-story parking garage with one cellar occupied the site at the time of our site investigation but it has since been partly demolished. The adjacent Macy's used the cellar as storage; the cellar is connected to Macy's by a tunnel beneath Hoyt Street at the southwest corner of the site. There is a mezzanine in the cellar. The northeast part of the site has a step out in the foundation wall; the area adjacent to the neighbors is a slab at the same level as the mezzanine, but about 25 feet south of the north wall is a second foundation wall where the space drops down to cellar level. Elevations at the site are:

- Cellar slab: about el 26 to 27 feet;
- Cellar mezzanine: about el 35 to 36 feet;
- Hoyt Street sidewalk (west side of site): about el 46.9 (north) to 45 (south) feet;
- Livingston Street sidewalk (south side of site): about el 47 (east) to 45 (west) feet;
- Elm Place sidewalk (east side of site): about el 44 (north) to 47 (south) feet.

Drawing No. 2 illustrates the site conditions, adjacent buildings and streets.

ADJACENT STRUCTURES

Neighboring Buildings

- 1 Hoyt Street (Block 157, Lot 9) is a designated landmark that borders and has an easement into the northwest corner of the site. The building is eight stories and has a footprint of about 10,000 square feet, including the easement of about 3,400 square feet into 11 Hoyt Street. According to the available Certificate of Occupancy from the New York City Department of Buildings (NYCDOB)[1], the building has one cellar level. The cellar depth is not reported; however, a doorway between the cellar of this building and the existing parking garage cellar at 11 Hoyt Street suggests that the 1 Hoyt Street cellar is at least as deep as the parking garage cellar. The City of New York Technical Policy and Procedure Notice #1088 (TPPN1088) requires all landmarks within 90 feet of the site to be monitored during construction.
- 460 Fulton Street (Block 157, Lot 14) is a five-story commercial building that borders the center of the parking garage. This lot is about 3,700 square feet. The Certificate of Occupancy indicates that the building has one below-grade level with no reported depth.
- 466 Fulton Street (Block 157, Lot 16) is a five-story commercial building that borders the center of the parking garage. This lot is about 2,900 square feet. The Certificate of Occupancy indicates that the building has two below-grade levels with no reported depths.
- 470 Fulton Street (Block 157, Lot 18) is a five-story commercial building that borders the northeast corner of the parking garage site. This lot is about 2,900 square feet. The Certificate of Occupancy indicates that the building has one below-grade level with no reported depth.
- 472 Fulton Street (Block 157, Lot 20) is a four-story commercial building that borders the northeast corner of the parking garage site. This lot is about 2,500 square feet. The Certificate of Occupancy indicates that the building has one below-grade level with no reported depth.

The information obtained from NYCDOB should not be considered absolute or all-inclusive. The foundation contractor should make investigations to properly protect these structures during construction.

[1] New York City Department of Buildings website property profile and certificate of occupancy (www.nyc.gov)

NYCT Subway

A New York City Transit (NYCT) subway tunnel and station (No. 2 and 3 lines) runs northwest-southeast beneath Fulton Street north of the site. The Hoyt Street station has entrances at the corners of Fulton Street with Hoyt Street and Elm Place. The tunnel, with four tracks, is a rectangular steel and concrete structure constructed using cut-and cover-techniques. The subway is about 80 feet from the north property line at its closest point to the site. Historical drawings from NYCT show the base of rail at about el 25.3 and the base of structure at about el 23.3. The cellar of the existing parking garage is entirely below the NYCT subway influence line; and the proposed foundations will also be below the subway influence line. Drawing No. 2 shows the approximate limits of the subway; available drawings from NYCT are included in Appendix A.

Because the site is within 200 feet of the subway, review and approval by NYCT for the new construction is required before obtaining foundation permits. NYCT may require calculations showing that the proposed construction will not affect the subway tunnel. The tunnel must be protected and monitored during construction.

PROPOSED DEVELOPMENT

The proposed development includes demolition of the parking garage and the construction of a 53-story residential building. The existing garage cellar will be persevered as much as possible for reuse as a combination of retail, parking, storage and mechanical spaces, and a driveway will extend west to east in the north part of the site. The proposed tower is trapezoidal shaped and will be centered in the site. The tower will extend through the existing cellar onto a new foundation system. Existing columns surrounding the new tower will remain. The tower footprint will occupy about 13,500 square feet of the site's 58,800 square feet. Concept plans by SGA (5 October 2016) are included in Appendix B.

Preliminary foundation loads from McS (7 November 2016) show the tower perimeter columns' dead loads from 2,000 to 5,800 kips and the shear walls with dead loads of 22,000 to 46,000 kips. Uplift column loads are less than 845 kips. The maximum shear forces are from wind, and will be from about 1,500 to 3,800 kips.

REGIONAL GEOLOGY

The geology at the site consists of glacial deposits commonly referred to as ground moraine, a widespread dense layer consisting of a mixture of clay, silt, sand, gravel and boulders. Several glacial advances into the area significantly influenced the soils in this area of Brooklyn. As the glaciers advanced southward, they scraped soil and rock from the surface and deposited the material at the limit of the ice advance as a wall of soil and rock called the *terminal moraine*, which created a dam and the former glacial Lake Flushing over this area of Brooklyn. Lake

Flushing deposits consist of interlayered fine sand, silt, and clay that overlie the boulder-laden glacial till and bedrock. The New York City area terminal moraine stretches across northern Long Island to Staten Island and to New Jersey. When the climate moderated and the glacier began a gradual retreat north, soil and rock continued to be carried south in the meltwater and was spread over the ground in front of the ice as a veneer called *ground moraine*. The site lies within the bounds of former Lake Flushing and is in the ground moraine (Drawing No. 3).

Hartland formation bedrock underlies the site and generally consists of gneissic schist in this area of Brooklyn. The USGS Bedrock Geology map (Drawing No. 4) shows bedrock about three blocks south of the site at about el -100 feet, and the bedrock becomes shallower heading north toward the parking garage. We estimate that bedrock is likely to be about 150 feet deep at the site (sidewalk grade around el 45 and rock shallower than el -100 feet).

FEMA FLOOD ZONE

We reviewed the Preliminary National Flood Insurance Rate Maps (FIRM) (5 December 2013), which were issued to supersede the Best Available Flood Hazard Data Maps published by the Federal Emergency Management Agency (FEMA) on 10 June 2013 as a result of Hurricane Sandy. According to the Preliminary FIRM, the site is outside the limits of the flood-hazard boundaries—"areas subject to inundation by the 0.2 percent annual chance flood (500-year flood)" (Drawing No. 5).

BASEMENT WALL INVESTIGATION

The purpose of the wall investigation was to find out whether the walls can be reused as support of excavation during construction and as the permanent foundation walls of the proposed building. Our investigation included 12 horizontal cores (WC-1 through WC-12) and four pachometer[2] surveys (PS-1 through PS-4) of the basement walls of the parking garage (Drawing No. 6). Future Tech Consultants of New York (FTC) performed the investigation on 29 February and 1 March 2016 under the full-time observation of Langan. FTC performed three cores and one pachometer survey on each wall. The walls were cored with a 3-inch-diameter thin-wall core barrel and a chipping gun was used to expose the steel reinforcement for the pachometer surveys. FTC also performed unconfined compression strength tests on each core. The FTC report is attached as Appendix C and the thicknesses are described below. We also photo-documented the conditions at each wall (see Appendix D).

The north foundation wall is a split-level wall that "steps back" about 25 feet south from the north property line. The upper level of the wall is at mezzanine level and the lower level is at basement level. The purpose of the step back is unknown but it may have been constructed to avoid underpinning when the parking garage was constructed.

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[2] A pachometer is a device used to measure thickness.

Table 1 includes a summary of the observations and measurements at each wall. Refer to Drawing No. 6 for approximate test locations.

Table 1 – Cellar Wall Measurements

Location	Wall Thickness	Rebar Grid Spacing (v x h)	Vertical Bar	Horizontal Bar	Compressive Strength (PSI)
North Wall Upper	17″	18″ x 18″	0.75″ (#6) 3″ cover	0.625″ (#5) 3.5″ cover	2,750 to 3,730
North Wall Lower	12″	18″ x 18″	Not exposed		4,540
East Wall	17″	8″ x 16″	1″ (#8) 1.5″ cover	0.625″ (#5) 2.5″ cover	4,350 to 4,970
South Wall	16″	8″ x 16″	1″ (#8) 1.5″ cover	0.625″ (#5) 2.5″ cover	6,120 to 7,590
West Wall	16″	8″ x 16″	1″ (#8) 2″ cover	0.625″ (#5) 3″ cover	5,770 to 6,140

SUBSURFACE INVESTIGATION

Our subsurface investigation consisted of drilling seven test borings and excavating six test pits to evaluate the subsurface and foundation conditions. The locations of the borings and test pits are shown on Drawing No. 7.

Sixteen borings are required to meet Building Code requirements for the total development footprint of about 58,800 square feet. Seven borings were completed as part of this study. Uniform dense sands were encountered, consistent with the conditions encountered during our investigation of nearby sites for other projects. Because of the uniform conditions, we have requested a waiver from the Department of Buildings to reduce the total number of borings drilled. Although we have been successful with waiver requests in the past, if the DOB denies the waiver, nine additional borings would be needed.

Geotechnical Test Borings

We drilled seven test borings (LB-1 through LB-7) in the sidewalks of Hoyt Street, Livingston Street, and Elm Place to investigate the subsurface. The test borings were drilled with a combination of truck- and track-mounted drill rigs by Craig Test Boring Co., Inc. between 2 and 12 February 2016 under the full-time special inspection of Langan. The borings were drilled using mud-rotary drilling techniques to depths from 77 to 102 feet. Support of the boreholes was provided by temporary flush-joint steel casing and drilling mud.

The Standard Penetration Test (SPT)[3] was performed in general accordance with ASTM D1586 (Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils). Soils were sampled using a standard 2-inch outer-diameter split-spoon sampler driven by an automatic hammer.

Our engineer recorded SPT N-values and classified the soil in accordance with the Unified Soil Classification System (USCS) and the Building Code. The boring logs from our investigation are provided in Appendix E.

Observation Wells

Two groundwater observation wells were installed in the completed borings LB-2(OW) and LB-5(OW). Protective flush-mounted steel well covers were installed in the sidewalk at each well and we monitored the wells daily throughout the investigation. Detailed observation-well construction logs are included in Appendix E.

Laboratory Testing

Mechanical grain-size determinations (sieve analysis; ASTM D422) were conducted on 10 selected soil samples. The purpose of the testing was to confirm visual field classifications and to define index properties for use in our analysis. The laboratory test results are attached in Appendix F.

Test Pit Investigation

Six test pits (TP-1 through TP-6) were excavated in the basement of the parking garage to observe the existing foundation type, condition, dimensions, and underlying bearing material. The purpose of the test pits to confirm if the existing foundations can be reused for the new building. The test pits were excavated by hand in February 2016 by Red Hook Construction Group. The conditions encountered within each test pit were documented by Langan with sketches and photographs and are presented in Appendix G.

[3] The Standard Penetration Test (SPT) is a measure of soil density and consistency. The SPT N-value is defined as the number of blows required to drive a 2-inch outer diameter split-barrel sampler 1 foot, after an initial penetration of 6 inches, using a 140-pound hammer falling freely from a height of 30 inches.

TEST PIT FINDINGS

Descriptions of the findings in each test pit are provided below. Refer to the sketches and photographs (Appendix G) for additional information at each test-pit location.

Test Pit TP-1

Test pit TP-1 was excavated in the basement along the northwest wall to determine the dimensions and thickness of the footings. The excavation was about 5 feet wide by 6.5 feet long and 4.5 feet deep.

The concrete slab at the test pit was about 7 inches thick and reinforced with welded wire mesh on about a 9-inch grid. The footing extends about 4 feet south of the wall and is about 6 feet wide (east-west) by 3 feet thick, and the bottom of the footing is about 4 feet below the top of the slab.

Soils excavated from TP-1 consisted of light brown, medium- to fine-grain silty sand with gravel. Large boulders and cobbles were also encountered in the test pit. Groundwater was not encountered in this excavation. The test pit was backfilled with the excavated material upon completion.

Test Pit TP-2

Test pit TP-2 was excavated along the upper (mezzanine) part of the north foundation wall to determine the dimensions and thickness of the foundation wall. The excavation was about 6 feet wide by 6 feet long and 6 feet deep.

The concrete slab at the test pit was about 7 inches thick and reinforced with rebar spaced in a grid about 9 inches on center. The wall bears on a strip footing that extends into the site about 3 feet. The footing is at least 5 feet below the top of the slab and extends below the depth of the test pit.

Soils excavated from TP-2 consisted of light brown, medium- to fine-grain silty sand with gravel. Large boulders and cobbles were also encountered in the test pit. Groundwater was not encountered in this excavation. The test pit was backfilled with the excavated material upon completion.

Test Pit TP-3

Test pit TP-3 was excavated along the east basement wall to determine the dimensions and thickness of the column footing and wall foundation. The excavation was about 10 feet wide by 6.5 feet long and 4.5 feet deep.

The concrete slab at the test pit is about 7 inches thick and is reinforced with welded wire mesh on about a 9-inch grid. The footing extends about 4 feet west of the wall, and about 7 feet wide (north-south), and about 3 feet thick and the bottom of the footing is about 4 feet below the slab.

The foundation wall bears on strip footing that extends 21 inches from the face of the wall. The wall footing is about 3 feet thick and the bottom of the footing is about 4 feet below the top of the slab.

Soils excavated from TP-3 consisted of light brown, medium- to fine-grain silty sand with gravel. Large boulders and cobbles were also encountered in the test pit. Groundwater was not encountered during the excavation. The test pit was backfilled with the excavated material upon completion.

Test Pit TP-4

Test pit TP-4 was excavated along the south foundation wall to determine the dimensions and thickness of the column footing. The excavation was about 6 feet wide by 5 feet long and 4.5 feet deep.

The concrete slab at the test pit is about 7 inches thick and reinforced with welded wire mesh on about a 9-inch grid. The column bears on a strip footing that extends about 12 inches from the face of the wall. The footing is about 3 feet thick and the bottom of the footing was about 4 feet below the top of the slab.

Soils excavated from TP-4 consisted of light brown, medium- to fine-grain silty sand with gravel. Large boulders and cobbles were also encountered in the test pit. Groundwater was not encountered during the excavation. The test pit was backfilled with the excavated material upon completion.

Test Pit TP-5

Test pit TP-5 was excavated along the west foundation wall to determine the dimensions and thickness of the column footing. The excavation was about 15 feet wide by 10 feet long and 4.5 feet deep.

The concrete slab at the test pit is about 9 inches thick and reinforced with welded wire mesh on about a 9-inch grid. The slab is about 2 inches thicker than at the other test pits. The column extends about 12 inches below the basement slab and bears on a strip footing that extends 15 inches from the face of the wall. The footing is about 3 feet thick and the bottom of the footing is about 4 feet below the slab.

A grade beam encountered in this test pit extends east towards the adjacent column, and is at least 5 feet wide (full width not exposed). The beam is about 3 feet thick and the bottom of the beam is about 4 feet below the top of the slab.

Soils excavated from TP-5 consisted of dark brown, medium- to fine-grain silty sand with gravel. Large boulders and cobbles were also encountered in the test pit. Groundwater was not encountered during the excavation. The test pit was backfilled with the excavated material upon completion.

Test Pit TP-6

Test pit TP-6 was excavated along the lower part of the north foundation wall to determine the dimensions and thickness of the foundation wall footing. The excavation was about 5 feet wide by 6 feet long by 4.5 feet deep.

The concrete slab at the test pit is about 7 inches thick and reinforced with welded wire mesh on about a 9-inch grid. The column bears on a strip footing that extends about 18 inches from the face of the wall. The footing is about 3 feet thick and the bottom of the footing is about 4 feet below the top of the slab.

Soils excavated from TP-6 consisted of light brown, medium- to fine-grain silty sand with gravel. Large boulders and cobbles were encountered in the test pit. Groundwater was not encountered in this excavation. The test pit was backfilled with the excavated material upon completion.

SUBSURFACE CONDITIONS

The subsurface conditions consist of miscellaneous historical fill (beneath the sidewalk surrounding the site) underlain by medium-dense to dense sand with variable amounts of silt, gravel, cobbles and boulders. Bedrock was not encountered in the borings, which were drilled to 102 feet below the sidewalk. Generalized subsurface profiles showing the borings are included as Drawing Nos. 8 through 10. Detailed descriptions of each subsurface stratum are given below in order of increasing depth.

Uncontrolled Fill [Class 7][4]

Historical uncontrolled fill was encountered immediately below the sidewalk in all borings, and the fill does not appear to extend below the basement level; however, native soils appear to have been used as fill around the foundations. The historical fill is generally a mixture of sand, gravel, and silt with variable amounts of brick, concrete, and wood. The historical fill thickness varies from about 15 to 25 feet, corresponding to about el 32 to 22 feet.

[4] Numbers in brackets indicate classification of soil and rock in accordance with the 2014 New York City Building Code.

The fill density is highly variable, as evidenced by a wide range in SPT N-values that varied from 2 blows per foot (bpf) to greater than 100 bpf (sampler refusal), but is generally loose. The higher SPT N-values are evidence of obstructions such as cobbles, gravel, boulders, and construction debris, and are generally not representative of the fill density.

The fill classifies as Building Code Class 7 (Uncontrolled Fills).

Upper Sand and Boulders [Class 3a and 3b]

A layer of brown and reddish-brown sand with silt, gravel, cobbles and boulders is below the fill. This sand layer is about 25 to 40 feet thick (bottom at about el 20 to 5) and extends to about the groundwater level. SPT N-values in this layer were highly variable, between 15 and >100 bpf (sampler refusal). The higher values appear to be areas where cobbles and boulders may have impeded the sampler.

Three samples of the upper sand were tested in the laboratory for grain-size distribution. Two samples were well-graded coarse to fine sand with gravel and silt (SW-SM) and one sample was silty sand (SM). Percent fines (silt and clay) were from about 8 to 46 percent.

The upper sand classifies as Building Code Class 3a and 3b (Dense Granular Soils and Medium Granular Soils).

Lower Sand [Class 3a]

Beneath the upper sand is brown, fine sand with silt and trace amounts of gravel. The lower sand is distinguished from the upper sand by a change in color, a transition from well-graded to poorly graded grain size, a decrease in silt and gravel content, and less variability in the SPT N-values. The transition generally occurs near groundwater. We encountered the lower sand at about el 0 to -9, and the sand extends to the full depth explored of 102 feet (about el -58). SPT N-values in this layer were between 20 and 78 bpf, and showed a trend of increasing blows and density versus depth.

Seven samples of the lower sand were tested in the laboratory for grain-size distribution. The samples were primarily fine sand with silt and trace gravel (SP) and (SP-SM). Percent fines (silt and clay) were from about 2 to 10 percent.

The lower sand classifies as Building Code Class 3a (Dense Granular Soils).

Groundwater

The groundwater was measured in observation wells LB-2(OW) and LB-5(OW) (see Table 2). Groundwater may fluctuate as a result of precipitation, seasonal variations, construction activities or nearby groundwater pumping.

Table 2 – Groundwater Monitoring Measurements

Observation Well No. (Approx. Surface Elevation)	Date	Depth Below Grade (feet)	Approx. Elevation* (feet) (NAVD88)
LB-2(OW) (about el 45.5±)	2/11/2016	46.5	-1
	2/12/2016	46	-0.5
	2/25/2016	46	-0.5
	2/29/2016	45.75	-0.25
	3/1/2016	45.75	-0.25
	11/16/2016	45.25	0.25
LB-5(OW) (about el 46.5±)	2/10/2016	44	2.5
	2/11/2016	46	0.5
	2/12/2016	45	1.5
	2/25/2016	46.5	0
	2/29/2016	46.5	0
	3/1/2016	46.5	0
	11/16/2016	46	0.5

*Monitoring wells not surveyed, elevations estimated from sidewalk survey

RECOMMENDATION FOR ADDITIONAL INVESTIGATION

Regardless of whether the DOB approves the boring waiver, we recommend drilling at least two borings to bedrock beneath the tower footprint. Our recent experience at a nearby site is that rock may be within about 110 to 140 feet deep. If we can confirm that bedrock is less than about 130 feet below grade, a stiffer foundation response may be justified for design. We believe the benefit to the foundation design is worth the cost of the additional borings.

SEISMIC EVALUATION

Seismic design parameters are in accordance with the 2014 New York City Building Code.

NYC Building Code Seismic Design Parameters

The recommended seismic parameters are based on the average Standard Penetration Resistance (SPT N-value) in the soil above bedrock. The SPT N-values show a general linear increase versus depth. We recommend Site Class C.

We understand the structure is Structural Occupancy/Risk Category II (the architect and structural engineer must confirm). For Structural Occupancy II and Site Class C, the design spectral accelerations result in Seismic Design Category B.

Table 3 – Seismic Design Parameters

Description	Parameter	Recommended Value	Building Code Reference
Mapped Spectral Acceleration for short periods:	Ss	0.281 g	Section 1613.5.1
Mapped Spectral Acceleration for 1-sec period:	S_1	0.073 g	
Site Class	Very Dense Soil Profile	**C**	Table 1613.5.2
Site Coefficient:	Fa	1.20	Table 1613.5.3(1)
Site Coefficient:	Fv	1.70	Table 1613.5.3(2)
5 percent damped design spectral response acceleration at short periods:	$\mathbf{S_{DS}}$	**0.22 g**	Section 1613.5.4
5 percent damped design spectral response acceleration at 1-sec period:	$\mathbf{S_{D1}}$	**0.08 g**	
Seismic Design Category for Structural Occupancy/Risk Category II		**B**	Table 1613.5.6

Liquefaction Potential

Soil liquefaction is a phenomenon primarily associated with saturated, loose, cohesionless soils near the ground surface (generally depths less than 50 feet). As these soils are shaken, pore water pressure may develop that will temporarily force the soil particles apart such that the soil behaves temporarily more like a liquid than a soil mass (hence the term liquefaction). We screened for liquefaction potential using the Building Code Liquefaction Assessment Diagram and performed a liquefaction analysis for the soils and groundwater encountered in the borings.

<u>2014 NYC Building Code Liquefaction Assessment</u>

The seismic provision of the Building Code requires an evaluation of the liquefaction potential of sands, silts, and noncohesive materials below the groundwater table and up to 50 feet below the ground surface. As an initial screening process, we plotted SPT N-values versus depth on the Building Code Liquefaction Assessment Diagram (Drawing No. 11). With groundwater at about 46 feet below the existing grade, some of the SPT N-values fall within the "Liquefaction Probable" category; therefore, we performed a more detailed liquefaction analysis.

<u>Liquefaction Evaluation</u>

The potential for soil liquefaction was evaluated further using the procedure outlined by Youd et al. (2001). This procedure is the general state-of-practice, and is the recommended procedure by the National Earthquake Hazard Reduction Program. The evaluation uses an empirical relationship between the earthquake demand, represented by the Cyclic Stress Ratio (CSR), and the soil's resistance to dynamic loading, represented by the Cyclic Resistance Ratio (CRR). The CSR (demand) is determined by the Peak Ground Acceleration (PGA) of the design earthquake event and the in situ soil stresses. The CRR (resistance) is correlated to SPT N-values obtained in the field. The field-measured SPT N-values are normalized by applying correction factors for such variables as soil overburden pressure, hammer energy and soil fines content.

We analyzed liquefaction potential for a magnitude 5.75 earthquake and a peak ground acceleration of 0.20g (Site Class C) and calculated the factor of safety for SPT N-values below groundwater and within 50 feet of the ground surface (Drawing No. 12). We believe the potential for liquefaction and liquefaction-induced settlement is low, based on our evaluation of the boring data and estimated seismic parameters for the site. Therefore, in our judgment, there is an adequate margin of safety against liquefaction for the site, and liquefaction related phenomena need not be considered in the design.

DESIGN RECOMMENDATIONS

The proposed development includes construction of a 53-story residential building in the center of the site. The existing cellar will be preserved as much as possible. The cellar outside the tower, and a new one-story podium around the tower, will be used for a combination of retail, storage and parking.

Design and Construction Considerations

The key considerations for developing our recommendations for design and construction include:

- A New York City Transit (NYCT) subway tunnel (No. 2 and 3 lines) runs beneath Fulton Street north of the site. NYCT must approve the support of excavation and foundation plans before the Department of Buildings will issue a foundation permit. Although the proposed tower is at least 125 feet away from the NYCT tunnel and station, comments from NYCT may impact the design and means and methods for the support of excavation and possibly foundations.
- The intent is to reuse the existing cellar foundation walls for temporary support during construction, and as the permanent basement walls for the new development. We will analyze the walls during our support of excavation design and determine if temporary bracing will be required in areas where the ground-floor slab (bracing the top of the wall) is removed.
- Reusing the existing foundations for support of the one-story podium appears to be feasible from a geotechnical view. We have included estimates of settlement for

evaluation by the MSSE. The structural integrity of the footings also needs to be evaluated by MSSE.

- Total and differential foundation settlement must be controlled across the tower, and between the tower and the existing cellar slab and foundations that will remain around the tower.
- We believe that a mat foundation is the most feasible foundation to balance cost and settlement performance. The mat may require drilled "settlement reducers" (similar to drilled piles) to reduce settlement beneath heavily loaded areas. Iteration between the geotechnical and structural models will be required during design development, and the need for settlement reducers will be determined.

Podium Foundations

We understand the intent is to reuse the existing ground floor and cellar slabs, and the existing columns and spread footings in the podium area (1-story structure surrounding the tower). The need to add additional loads above the existing loads has not yet been determined. The foundations are bearing in the dense native sands that are favorable for foundation support. The Building Code allows for a maximum bearing pressure of 12 kips per square foot (ksf) [6 tons per square foot (tsf)]. We estimate that settlement of existing foundations loaded to a maximum bearing pressure of 12 ksf will be less than ½ inch; please consult us if loads are above 6 tsf. MSSE must evaluate the structural integrity of the foundations for the proposed loads.

Tower Foundation

Controlling total and differential settlement of the tower is critical; therefore, we recommend supporting the tower on a mat foundation. The anticipated tower bearing pressure will likely be between 8 and 20 ksf. Because of the high bearing pressures, settlement reducers may be needed at strategic locations to limit differential settlement to levels acceptable by MSSE.

<u>Mat Foundation</u>

The tower can be supported on a mat foundation; however, settlement reducers (ground-improvement elements consisting of small diameter drilled elements) may be required to reduce differential settlements across the tower footprint and between the mat and the surrounding existing slab. During our analysis, we considered the following:

1. Bottom of mat assumed to be about el 19 feet (8 feet below cellar el 27)
2. Mat dimensions assumed to be about 160 feet by 95 feet.
3. Because of the large mat size, a significant thickness of soil will be stressed. Therefore, the depth to bedrock impacts the apparent soil stiffness and therefore the total settlement and the soil moduli values. Until additional borings are performed to confirm the depth to rock, we have assumed bedrock at 130 feet below grade (about el -86) for

preliminary design. Even if the building department approves our petition to reduce the number of borings (seven in lieu of 16 total), the depth to bedrock should be confirmed by drilling at least two borings beneath the tower footprint. The moduli values should be adjusted after the depth to rock is confirmed.

4. For our initial analysis, we calculated the settlement at the mat center; the settlement at the edge of the mat is assumed to be half of the center.

Subgrade modulus values for mat design are presented for use by MSSE in the structural model. The subgrade modulus is a simplified representation of the ground response and must be iterated during the foundation design until the settlements estimated by the geotechnical and the structural models (which approximate the soil response via Winkler springs) converge within about 10 percent. Note that the modulus values will change if the mat bearing elevation, or dimensions change (i.e. a wider mat will have a smaller spring value).

For initial design, we recommend using the following moduli values for an assumed bedrock elevation at el -86:

- 50 psi/inch at the center of the mat and
- 100 psi/inch at the edge of the mat

Our initial soil model is based on a rigid mat with uniform bearing conditions. A Finite Element Method (FEM) analysis must be performed to evaluate the settlement response under non-uniform bearing conditions (with or without using settlement reducers) to provide appropriate modulus values for the final mat design.

Ground Improvement Elements (Settlement Reducers)

If the estimated settlements for a mat foundation exceed values acceptable by McS, we recommend using "settlement reducers" below the mat. These pile-like elements are designed to reach their geotechnical capacity, i.e. they are designed to push into the soil but the elements carry load and reduce stresses, and therefore settlements, below the mat. These ground-improvement elements (GIE) typically consist of small diameter (typically 10 to 14 inch diameter) drilled, cast-in-place elements. The GIEs should have a minimum 5-foot-long cased section below the bottom of the mat to allow for load transfer and to aid in constructing the GIEs. The bond zone could be drilled using traditional cased and pressure grouting methods, or with hollow, self-drilling reinforcing bars. Several options for GIE dimensions and reinforcing can be designed to achieve the desired stiffness response. For the initial design, we estimate a GIE consisting of a 10.75-inch-diameter cased section and one #18 reinforcing bar with a 20-foot-long bond zone would have a compression capacity of 500 kips, and a stiffness of about 770 kips/inch. A stiffer or softer response could be achieved using different diameters, bond zones or reinforcement to achieve the desired settlement response. The GIEs should be modeled as discrete elements beneath the mat. We will work with MSSE during design development to provide model input if GIEs are required.

Ground Improvement Element Load Tests

We recommend at least two compression tests be performed on GIEs. If the schedule allows, we recommend performing the load tests during design development so the GIE design can be optimized and redesign delays can be avoided. The load tests should be performed according to the corresponding ASTM procedures for pile load testing but the GIEs should be loaded to geotechnical failure (continued deflection under sustained load – plunging failure). The GIEs should be instrumented with strain gauges and tell-tales during testing so that the design parameters can be optimized.

Lateral Loads

Lateral loads can be resisted by friction between the bottom of spread footings and mat, and the underlying sand. We recommend an allowable friction coefficient of 0.5 times the dead weight for analyzing lateral loads. If additional lateral resistance is required, please contact us to evaluate passive pressure.

Design Groundwater Level

We measured groundwater between about el -1 and 2.5. For design purposes, we recommend a design groundwater level 5 feet above the highest measured groundwater level, corresponding to el 7.5. We do not anticipate groundwater will be encountered during excavation and foundation construction and the basement will not be subject to hydrostatic conditions.

Permanent Groundwater Control

The stabilized groundwater level was measured about xx feet below the cellar and the new cellar level. Therefore, waterproofing to prevent groundwater seepage is not necessary. However, we can't confirm whether the existing cellar walls and slabs to remain are vapor tight or they can resist water intrusion during heavy rain or a utility break. The architect should evaluate the cellar use(s) (retail, parking, etc.) versus potential impacts from moisture intrusion and relative humidity in occupied spaces and determine if vapor mitigation measures are warranted.

Permanent Below-Grade Walls

The existing basement walls and any new below-grade walls (such as elevator pits) should be designed for lateral soil and surcharge pressure (Drawing No. 13).

- Lateral soil pressure: equivalent fluid pressure of 55 pounds per cubic foot
- Hydrostatic: Design groundwater el 7.5

- Surcharge loads: uniform pressure of 0.5 times the surcharge load distributed along the entire below-grade wall height. Large concentrated surcharges, such as loads from adjacent foundations, should be evaluated case-by-case using more detailed analysis.

SITE PREPARATION AND CONSTRUCTION RECOMMENDATIONS

The following sections discuss typical geotechnical related construction issues including excavation, support of excavation, subgrade preparation and backfill.

Excavation

We anticipate the excavation depth for the mat foundation will be about 8 feet and will extend into the native sand. Elevator pit excavations may extend deeper. Excavation should be feasible with conventional earthmoving equipment (i.e., backhoes, etc.) Obstructions such as remnant foundations, slabs, abandoned and live utilities, rubble, and boulders will be encountered (and should be anticipated by the contractor during bidding) and appropriate demolition equipment should be mobilized to remove the obstructions. All excavations must be conducted in accordance with all OSHA requirements including, but not limited to, temporary shoring, using trench boxes, and proper benching.

Temporary Support of Excavation (SOE)

Excavation will primarily be in the center of the site for construction of the tower mat and localized for utilities. Because the mat excavation is expected to be about 8 feet deep, support of excavation will be required. We expect that temporary excavation support will be preferred over sloped excavations so that the extent of demolition of the existing cellar slab and foundations can be minimized. We do not expect that the excavation will extend below water, so a watertight SOE system is not required. We recommend using a drilled soldier pile and lagging system instead of sheet piles because of the significant number of boulders encountered in the upper sand. If boulders obstruct the advancement of soldier piles, the boulders could be cleared with a rock hammer; whereas removing boulders to allow advancement of sheet piles would require deep excavation with a backhoe. Utility trenches can likely be supported with timber shoring or conventional trench boxes and shielding. Existing spread footing foundations adjacent to excavations must also be protected.

Underpinning the adjacent structures will not be required because the current schematic design drawings show that excavation will be localized at the center of the site for the new mat and no new foundations are proposed in the podium area.

Support of Existing Basement Walls

The existing first floor will remain across several areas of the site, which will brace the existing foundation walls against the earth pressure. The central area of the south wall (where the tower will extend through the basement) and the northeast corner (where the loading dock is being demolished) will have the first-floor slab removed, so the existing basement walls may require temporary bracing. We will evaluate the existing walls once the extent of the first-floor slab to be removed is determined during design development. We anticipate that the bracing will consist of rakers to the interior of the site.

Subgrade Preparation

The Building Code requires that a Professional Engineer licensed in the state of New York inspect and approve foundation subgrades before placement of concrete, to verify that the subgrade material is adequate to provide the recommended allowable bearing pressure. We recommend that Langan inspect the foundation subgrade to confirm bearing capacity and that subgrade is adequately prepared to meet design assumptions in this report.

The mat-foundation subgrade should be level and clear of standing or frozen water, debris, or other deleterious materials. The subgrade should be proof-rolled using a minimum 2.5-ton operating weight vibratory roller. Subgrade in utility excavations should be compacted with a 0.75-ton operating weight trench roller. The subgrade should be thoroughly compacted to provide a firm and unyielding surface before placing fill or concrete. Any soft, loose or unsuitable soils identified by the inspecting geotechnical engineering should be removed and replaced with approved structural fill.

The contractor should be responsible for maintaining all subgrades in their as-approved condition until concrete or fill is placed. The mat should be constructed as soon as possible following subgrade approval by the geotechnical engineer or should be protected by a lean concrete "mud mat."

Fill Materials, Placement, and Compaction

We expect only limited amounts of fill to backfill utility trenches and around support of excavation. All fill should be a well-graded granular material having a maximum particle size of 4 inches in any dimension and no more than 10 percent passing the No. 200 sieve. Fill should not be frozen and should be free of trash, debris, roots, vegetation, peat, and other deleterious materials. The geotechnical engineer should approve all fill before placement. Lean concrete or controlled low-strength material (CLSM) may be substituted for structural fill. On-site soils excavated during construction may be reused as structural fill provided the material meets the aforementioned criteria and is approved by the environmental engineer.

Place fill in uniform loose lifts not exceeding 10 inches in open areas where larger compaction equipment may be used (trench roller) and not exceeding 6 inches in confined areas where hand-operated equipment must be used (plate tamper). Compact fill to at least 95 percent of the soil's maximum dry density as determined by ASTM D1557 "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort." The water content of the soil at the time of compaction should be within 2 percent of the optimum value determined by ASTM D1557.

Only place fill on subgrades inspected and approved by the geotechnical engineer. Do not place fill on areas with standing water, on frozen subgrade or any areas not approved by the geotechnical engineer.

Monitoring During Construction

We recommend that a monitoring program be developed and incorporated into the contract documents. Monitoring should include means to measure both structural movement and vibrations from construction operations. The type and locations of specific monitoring equipment, threshold values, and durations should be developed based on review of the anticipated construction means and methods in conjunction with proximity to existing structures and utilities. The purpose of performing monitoring is to provide reasonable feedback to the engineer as to performance of the contractor with respect to protecting existing structures and utilities, and to assess any necessary changes to means and methods of construction.

Specific requirements for monitoring will likely be imposed by governing agencies including NYCT. Critical structures, which are likely to require monitoring, include the NYCT subway and 1 Hoyt Street (the designated landmark at the northwest corner of the site), and the other neighboring buildings on the north.

The monitoring program will likely include optical surveying, seismographs (vibration monitoring), and crack gauges. NYCT may require additional instrumentation. Consideration should be given to installing remote sensors capable of relaying data in real-time via wireless communications. The monitoring plan should address means and methods for measuring ground and structural deformation, and vibration levels.

We recommend that all monitoring be performed by a third-party consultant independent of the contractor; however, the contractor should reserve the right to perform additional monitoring. Monitoring should be performed at a minimum throughout excavation and foundation construction. NYCT may require longer monitoring.

Preconstruction Conditions Documentation

Preconstruction conditions should be documented about one month before construction for the adjacent structures to the north (interior and exterior), the NYCT subway tunnel and platform, and the surrounding public thoroughfares. The purpose of these observations is to provide photographic and video documentation representative of general existing conditions and to identify obvious visual deficiencies prior to construction. The preconditions observations should also identify areas requiring specific monitoring during construction. Structural integrity is not addressed in such documentation. This baseline information is often critical in the event of future damage claims resulting from construction.

Special Inspections

Excavation and foundation work are subject to various Special Inspections per Chapter 17 of the Building Code and the Rules of the City of New York. Construction that requires geotechnical quality control inspections include installation of the foundations, excavation, subgrade preparation, support of excavation, backfilling, and compaction. This work must be performed under the inspection of a qualified geotechnical engineer and should be performed by Langan. The inspecting engineer should be familiar with the subsurface conditions, as well as with the proposed and existing construction on site, and must meet the requisite qualifications outlined in 1RCNY 101-06.

CONSTRUCTION DOCUMENTS AND QUALITY CONTROL

Technical specifications and design drawings should incorporate our recommendations to ensure that subsurface conditions and other geotechnical issues at the site are adequately addressed in the construction documents. Langan should assist the design team in preparing specification sections related to geotechnical issues such as earthwork, GIE installation, and excavation support. Langan should also review foundation drawings and details, and all contractor submission documents and construction procedures related to geotechnical work.

We recommend that the language in foundation and earthwork specifications emphasize the potential for encountering buried obstructions during excavation and foundation drilling with the intent of mitigating change-of-conditions claims arising during construction. All excavation and drilling should be assumed to be unclassified such that the contractor is responsible for providing the necessary performance of the foundation system regardless of conditions encountered.

OWNER AND CONTRACTOR OBLIGATIONS

The contractor is responsible for construction quality control, which includes satisfactorily constructing the foundation system and any associated temporary works to achieve the design intent while not adversely impacting or causing loss of support to neighboring structures. Proper management of excavated soil is also solely the responsibility of the contractor.

Construction that can alter the existing ground conditions such as excavation, fill placement, foundation construction, ground improvement, dewatering, and related activities can also potentially induce stresses, vibrations, and movement in nearby structures and utilities, and disturb occupants of nearby structures. Contractors working at the site must ensure that their activities will not adversely affect the performance of the structures and utilities and will not disturb occupants of nearby structures. Contractors must also take all necessary measures to protect the existing structures during construction. By using this report, the owner agrees that Langan will not be held responsible for any damage to adjacent structures.

The preparation and use of this report is based on the condition that the project construction contract between the owner and their contractors will include (1) Langan being added to the Project Wrap and/or Contractor's General Liability insurance as an additional insured, and (2) language specifically stating the foundation contractor will defend, indemnify, and hold harmless the owner and Langan against all claims related to disturbance or damage to adjacent structures or properties.

LIMITATIONS

The conclusions and recommendations provided in this report are based on subsurface conditions inferred from a limited number of borings, in situ testing, and test pits performed within the site, and information provided by others.

This report has been prepared to assist the owner, architect, and structural engineer in the design process and is only applicable to the envisioned project discussed here. Any proposed changes in structures or their locations should be brought to our attention so we can determine the impact to our recommendations. Langan cannot assume responsibility for use of this report for any areas beyond the limits of this study or for any projects not specifically discussed herein. This report shall not be used for the design of temporary works including scaffolding, construction hoists, and crane pads.

Information on subsurface strata and groundwater levels shown on the logs represents conditions encountered only at the locations indicated and at the time of investigation. If different conditions are encountered during construction, they should immediately be brought to our attention for evaluation as this may impact our recommendations.

Environmental issues (such as potentially contaminated soil and groundwater) are outside the scope of this study.

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APPENDIX D

Wind Tunnel Report

Preliminary Results - Wind-Induced Structural Responses
11 Hoyt Street - Brooklyn, NY - RWDI Project #1701013
August 30, 2017

The wind loads provided in this report include the effects of directionality in the local wind climate. These loads do not contain safety or load factors and are to be applied to the building's structural system in the same manner as would wind loads calculated by code analytical methods.

Table 2: Summary of Predicted Peak Overall Structural Wind Loads

My (lb-ft)	Mx (lb-ft)	Mz (lb-ft)	Fx (lb)	Fy (lb)
1.30E+09	8.17E+08	4.46E+07	3.56E+06	2.08E+06

Notes:

(1) The above loads are the cumulative summation of the wind-induced loads at the structural level '1' (i.e. grade) centered about the reference axis shown in Figure 4, exclusive of combination factors.

(2) Note that the wind loads provided herein are for the overall design of the tower only. The design loads on the linked podium could be estimated based on code calculation.

(3) A total damping ratio of 2.0% of critical was used for structural load calculations.

(4) The above loads are based on the structural properties, "2017-08-02_11HoytSt-Wind Tunnel Prop_v24.6A", as provided on August 2, 2017. The natural building periods were as follows:

Mode 1: 7.22 s (Primarily X-Sway)
Mode 2: 6.19 s (Primarily Y-Sway)
Mode 3: 4.27 s (Primarily Torsion)

(5) The above loads correspond to a 50-year return period basic wind speed (3-second gust) of 98 mph.

Table 3: Effective Static Floor-by-Floor Wind Loads

Floor	Height (ft) Above '1'	Fx (lb)	Fy (lb)	Mz (lb-ft)
1	0	16400	100	55000
1Mezz	9.5	25800	100	34000
2	19.5	30700	11200	139000
2Mezz	29.5	30600	12400	138000
3	41.5	34600	14900	195000
4	52.33	32800	13000	205000
5	63.17	33900	13700	223000
6	74	35500	14400	246000
7	84.83	36000	15300	263000
8	95.67	37000	16100	283000
9	106.5	38100	17000	305000
10	117.33	39400	18100	331000
11	128.17	40500	19000	354000
12	139	43000	20500	389000
13	150.62	44900	21500	417000
14	161.46	44300	21600	426000
15	172.29	45600	22600	452000
16	183.12	47100	23700	480000
17	193.96	48500	24800	507000
18	204.79	49700	25900	533000
19	215.62	51400	27100	563000
20	226.46	52700	28200	590000
21	237.29	55500	30100	634000
22	248.92	56900	31000	656000
23	259.75	56900	31500	673000
24	270.58	58400	32700	702000
25	281.42	60100	34000	732000
26	292.25	61500	35300	760000
27	303.08	63100	36500	788000
28	313.92	66600	38900	841000
29-MEP	325.54	98400	58400	1295000
30	345.54	95200	56300	1142000
31	356.37	67300	39900	867000
32	367.21	68800	41200	903000
33	378.04	70500	42600	936000
34	388.87	71900	43900	968000
35	399.71	73500	45300	1002000
36	410.54	75300	46700	1038000
37	421.37	76800	48000	1071000
38	432.21	80500	50700	1135000
39	443.83	81100	51200	1150000
40	454.67	80000	51000	1158000
41	465.5	81100	52100	1185000

Table 4: Recommended Wind Load Combination Factors

Load Case	Factor for Simultaneous Application of Loads in Table 3: X Forces (Fx)	Y Forces (Fy)	Torsion (Mz)
1	+100%	+45%	+45%
2	+100%	+45%	-30%
3	+100%	-30%	+45%
4	+100%	-30%	-30%
5	-90%	+50%	+45%
6	-90%	+50%	-55%
7	-90%	-30%	+45%
8	-90%	-30%	-55%
9	+50%	+100%	+45%
10	+50%	+100%	-30%
11	+40%	-90%	+35%
12	+40%	-90%	-30%
13	-55%	+100%	+45%
14	-55%	+100%	-30%
15	-45%	-90%	+35%
16	-45%	-90%	-30%
17	+60%	+30%	+100%
18	+30%	+35%	-95%
19	+60%	-30%	+100%
20	+30%	-30%	-95%
21	-35%	+30%	+100%
22	-50%	+35%	-95%
23	-35%	-30%	+100%
24	-50%	-30%	-95%

Note:

(1) Load combination factors have been produced through consideration of the structure's response to various wind directions, modal coupling, correlation of wind gusts, and the directionality of strong winds in the local wind climate.

42	476.33	82900	53600	1221000
43	487.17	84300	55000	1254000
44	498	86100	56500	1291000
45	508.83	87000	57300	1313000
46	519.67	87200	57700	1325000
47	530.5	88900	59000	1356000
48	541.33	90700	60600	1388000
49	552.17	94500	63300	1449000
50	563.79	97900	65600	1491000
51	575.42	111400	76800	1759000
52	587.04	144800	99800	2307000
53 EMR	605.41	128300	87100	1627000
54 EMR RF	617.91	19300	10700	61000
SUMS	-	3.56E+06	2.08E+06	4.46E+07

Notes:

(1) The loads given in this table should be used with the load combination factors given in Table 4.

(2) The loads given in this table are centered about the reference axis shown in Figure 4.

(3) The above loads correspond to a 50-year return period basic wind speed (3-second gust) of 98 mph.

11 HOYT – BROOKLYN, NY

OVERALL STRUCTURAL WIND LOAD STUDY
WIND TUNNEL PHOTOS

PROJECT #1701013
FEBRUARY 2, 2017

SUBMITTED BY

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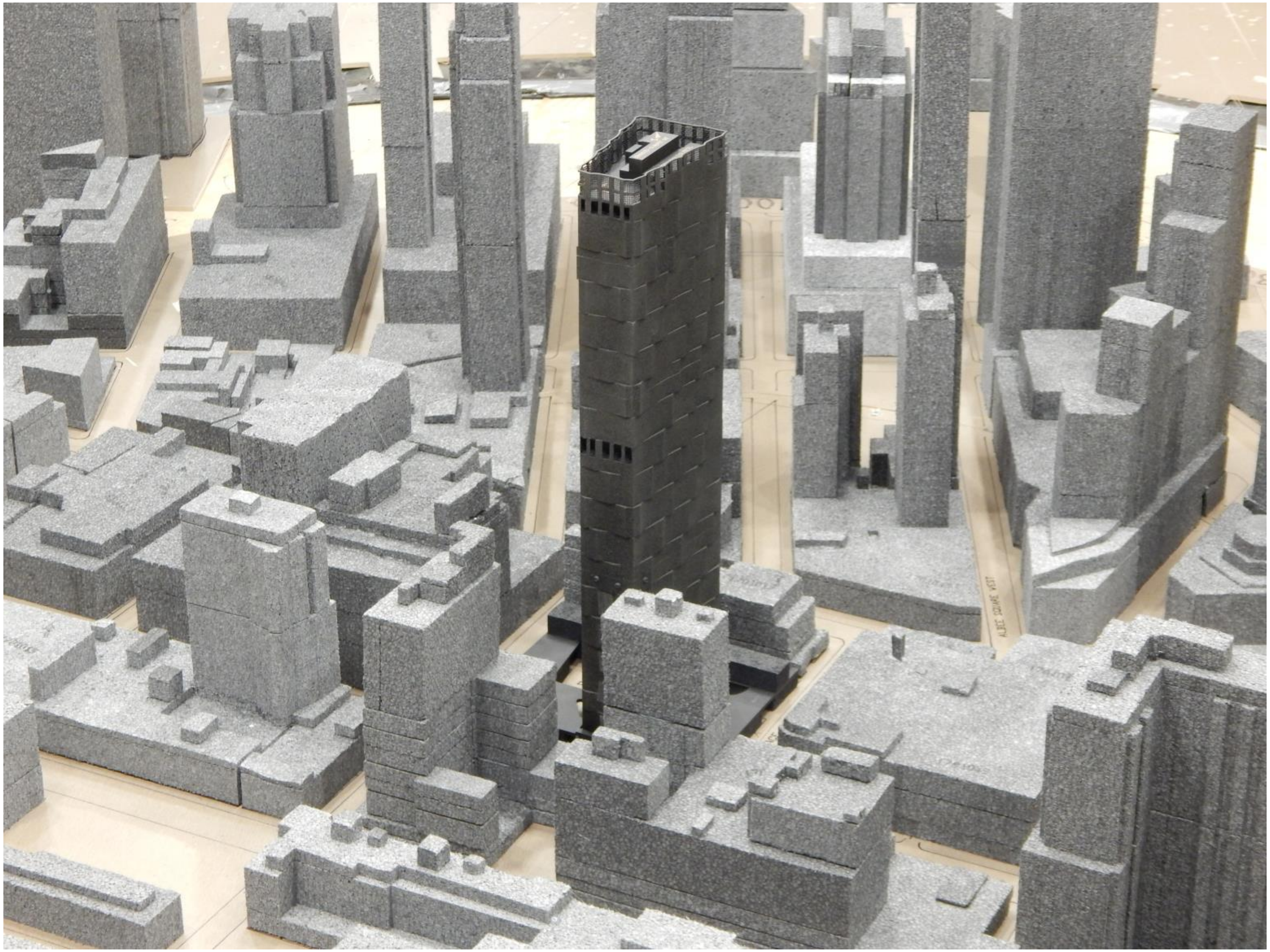


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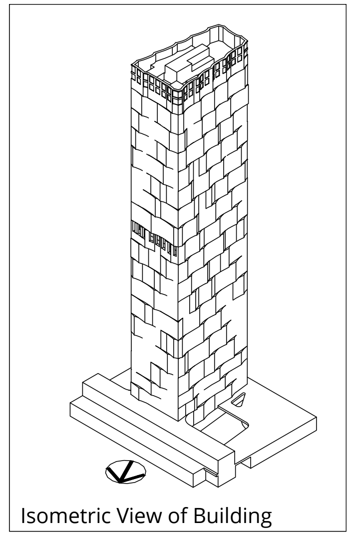
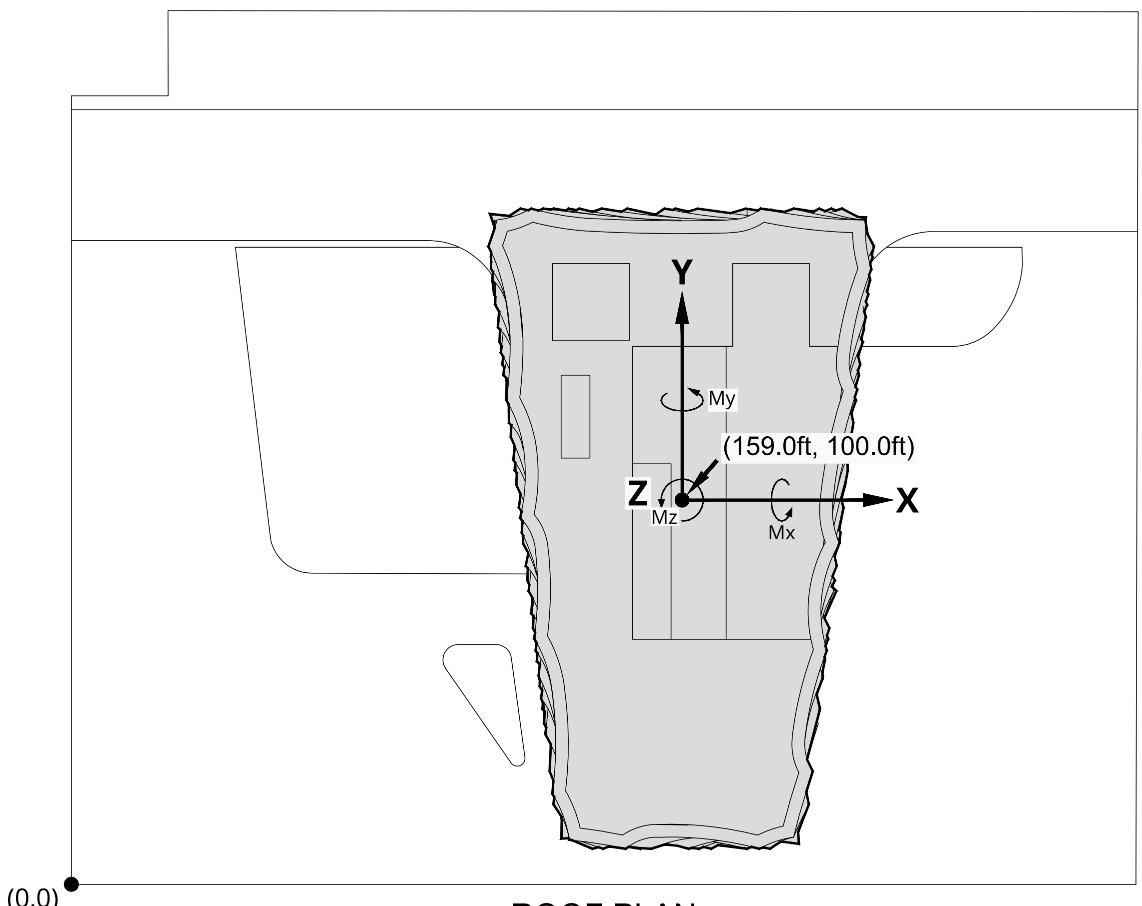


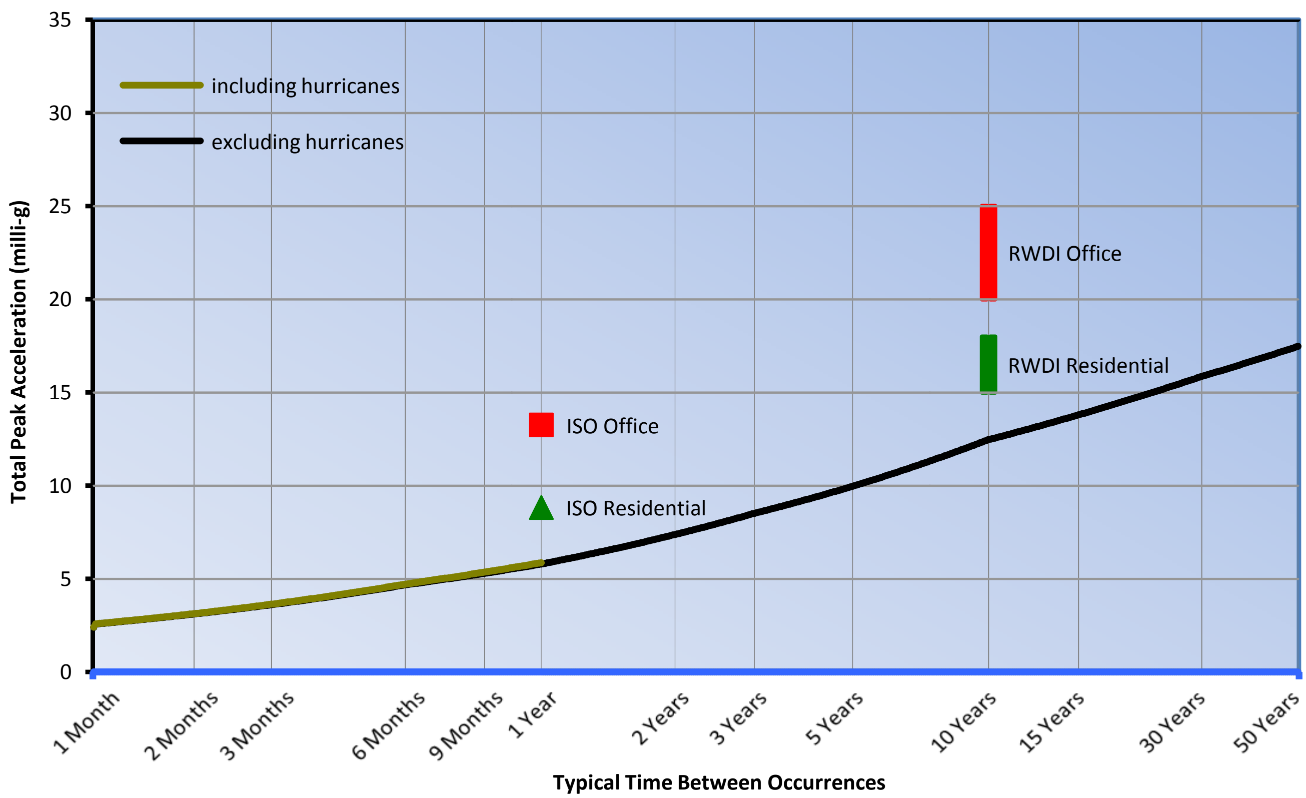


ROOF PLAN

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

0 25 50ft

Co-ordinate System for Structural Loading	True North	Drawn by: JMA	Figure: 4
		Approx. Scale: 1"=50'	
Macy's Development 11 Hoyt Street Tower Project - Brooklyn, NY	Project #1701013	Date Revised: Feb. 7, 2017	



Return Period (Years)	Peak Accelerations(2) (milli-g) Total - [X, Y and torsional components]		Peak Torsional Velocities (milli-rads/sec)		
	without hurricanes	with (6) hurricanes	without hurricanes	with hurricanes	CTBUH(5) Criteria
1	5.8 - [5.4, 4.5, 1.8]	5.9 - [5.5, 4.6, 1.8]	0.68	0.7	1.5
5	10.0 - [9.4, 7.8, 3.0]	-	1.1	-	-
10	12 - [12, 9.6, 3.7]	-	1.4	-	3

Notes:

(1) A damping ratio of 1.5% of critical was used, along with periods of 5.90, 5.64, and 3.94 seconds.

(2) Accelerations are predicted at Structural Level '50' (578 ft above Structural Level '1') at a radial distance of 54.6 ft from the central axis of the tower (given in Figure 4).

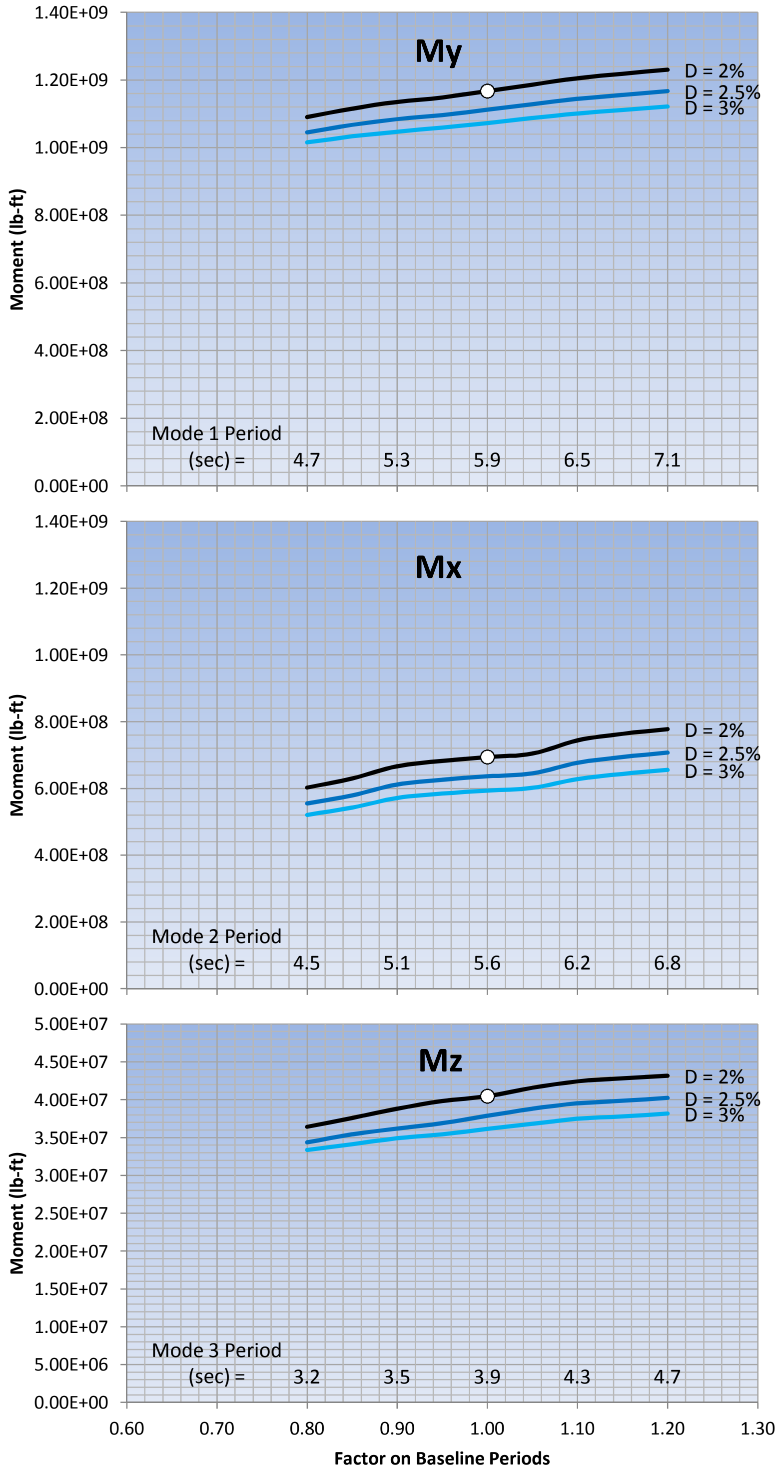
(3) ISO is the International Organization for Standardization, and the current standard (ISO 10137:2007) provides acceleration criteria for buildings at the 1-year return period. The criteria plotted on the graph have been generated based on a response-weighted interpretation of the individual modal 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) The Council on Tall Buildings and Urban Habitat (CTBUH) provides tentative torsional velocity criteria for the 1- and 10-year return periods.

(6) 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.

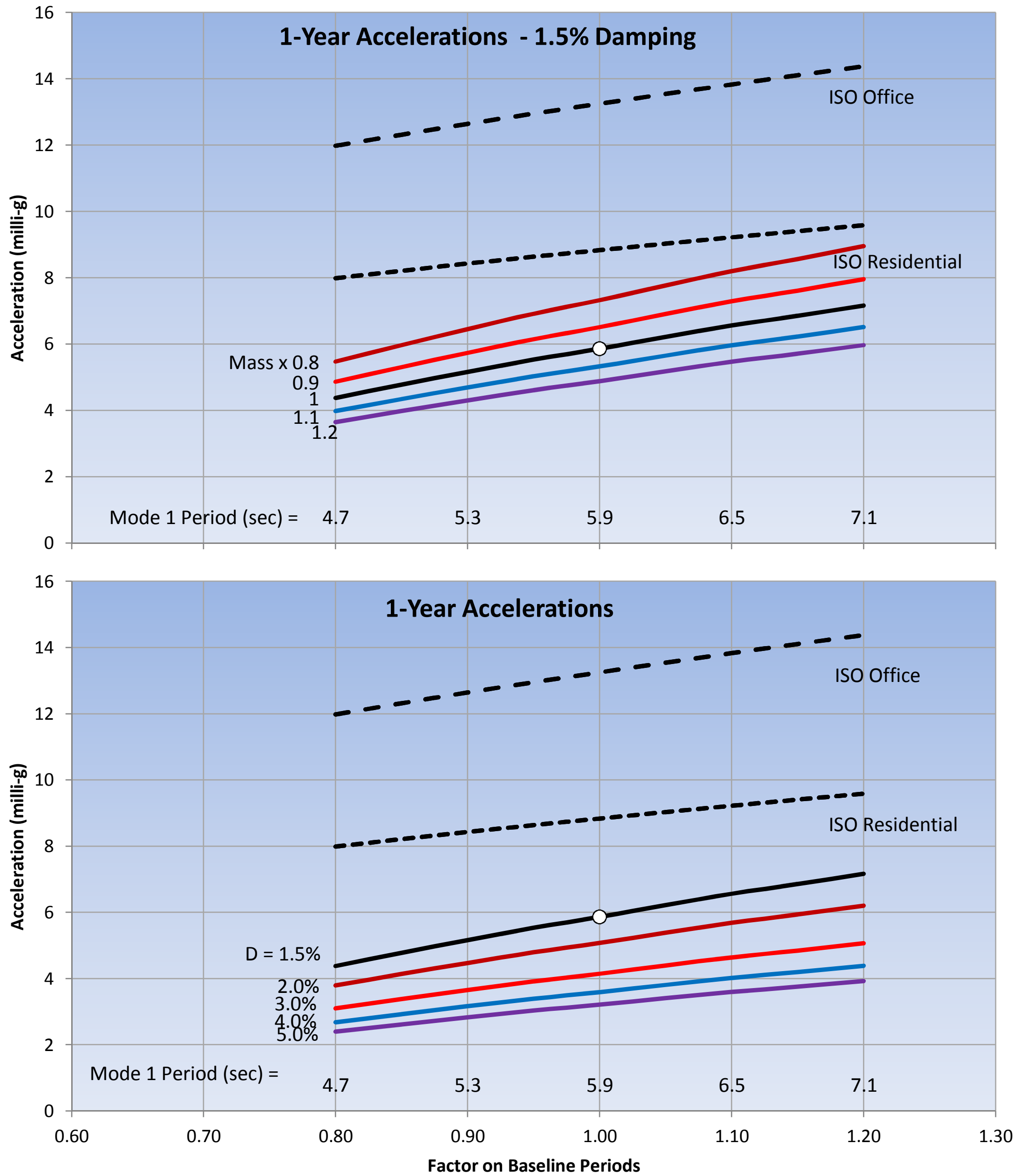
Predicted Peak Accelerations and Torsional Velocities	Figure No. 6	RWDI
Macy's Development 11 Hoyt Street Tower Project - Brooklyn, NY Project #1701013	Date: February 3, 2017	



Notes:

1) The baseline periods for the fundamental modes are 5.90, 5.64, and 3.94 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.

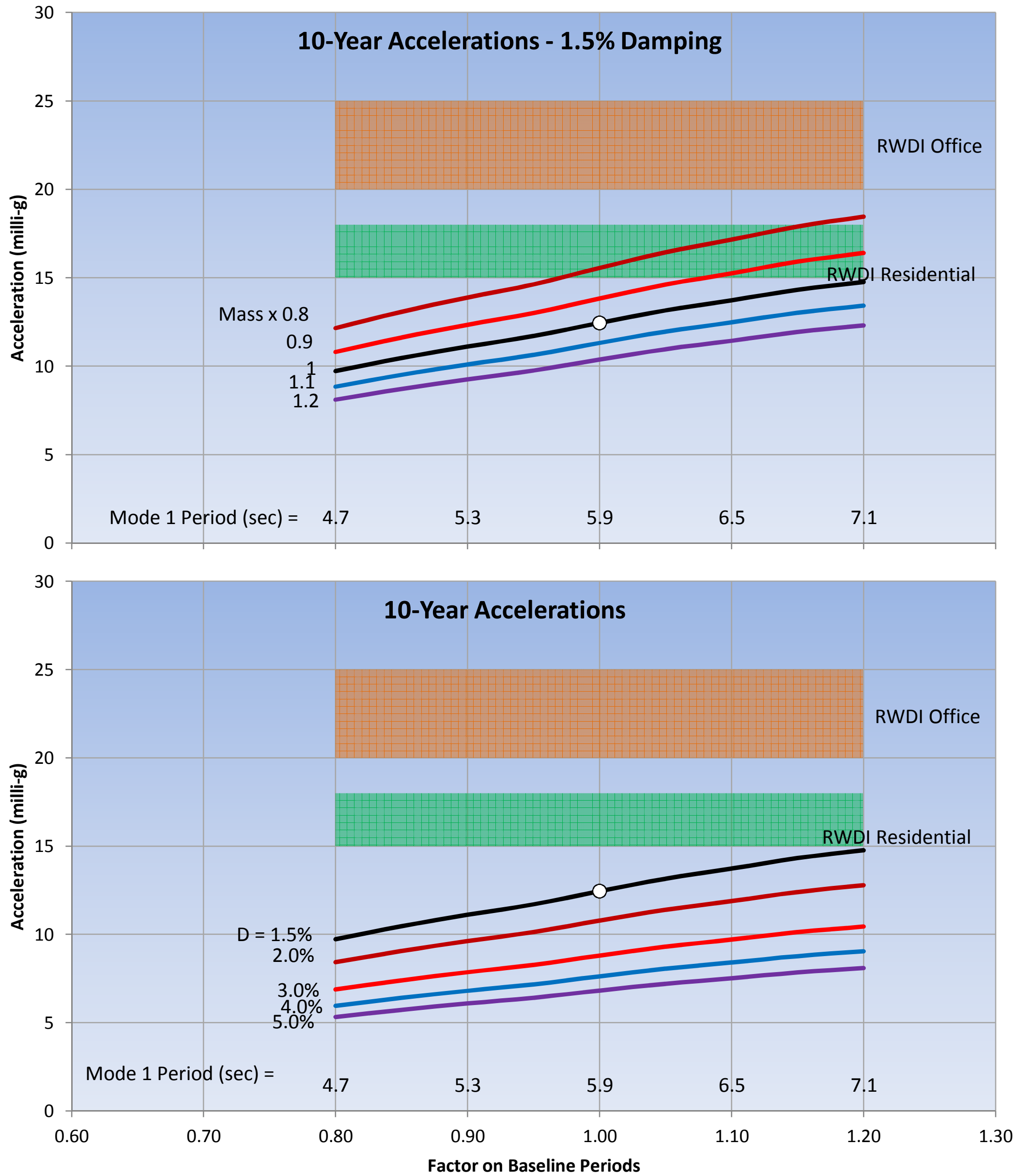
Sensitivity of Base Loads to Period and Damping	Figure No. 7
Macy's Development 11 Hoyt Street Tower Project - Brooklyn, NY Project # 1701013	Date: February 7, 2017



Notes:

1) The baseline periods for the fundamental modes are 5.90, 5.64, and 3.94 sec
2) Accelerations are predicted at Structural Level '50' (578 ft above Structural Level '1') at a radial distance of 54.6 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	Figure No. 8
Macy's Development 11 Hoyt Street Tower Project - Brooklyn, NY — Project # 1701013	Date: February 7, 2017



Notes:

1) The baseline periods for the fundamental modes are 5.90, 5.64, and 3.94 sec
2) Accelerations are predicted at Structural Level '50' (578 ft above Structural Level '1')
   at a radial distance of 54.6 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	Figure No. 9	RWDI
Macy's Development 11 Hoyt Street Tower Project - Brooklyn, NY Project # 1701013	Date: February 7, 2017	