DEPT OF BLDGS 121190772 Job Number

ES875292092 Scan Code

STRUCTURAL PEER REVIEW STATEMENT

This structural peer review and report, dated 24 February 2017, is complete for the foundation submission.

Structural Peer Reviewer Name: Benjamin M. Cornelius
Leslie E. Robertson Associates

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

Project Address: 45 Broad Street, New York City, Block #25, Lot #7, 10

Department Application Number for Structural Work: #121190772

Structural Peer Reviewer Statement:

I , Benjamin M. Cornelius , am a qualified and independent NYS licensed and registered engineer in accordance with BC Section 1627.4, and I have reviewed the structural plans, specifications, and supplemental reports for 45 Broad St., Block #25, Lot #7, 10, Application #121190772 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 2014 NYC Construction Codes. The Structural Peer Review Report is attached.

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

Name Benjamin M. Cornelius

Signature ____________________ Date 02/24/17

Cc: Project Owner: Madison Equities; Andrew Harris
Project Registered Design Professional: WSP; Johan Leonard

LERA

45 Broad Street
Structural Peer Review Report
Foundations

Leslie E. Robertson Associates, RLLP

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24 February 2017

45 Broad Street Structural Peer Review Report – Foundation

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1. Introduction

At the request of Mr. Andrew Harris of Madison Equities, Leslie E. Robertson Associates, R.L.L.P. (LERA) has conducted a Structural Peer Review of the foundation design of 45 Broad Street, as required by the New York City Building Code section 1617. This report presents our findings and conclusions.

The building is located at 45 Broad Street in New York City, and the structural design was prepared by WSP.

1.1 Documents Reviewed

We have reviewed the following documents:

- Structural and Architectural Drawings, listed in Appendix A.
- Structural Design Criteria listed in Drawing FO-001.00 and attached as Appendix B.
- Structural Design Criteria Narrative document provided by WSP and attached as Appendix C.
- The geotechnical report prepared by Langan Engineering, titled *Amended Geotechnical Engineering Study for 45 Broad Street* dated 29 April 2016, attached to this report as Appendix D.
- SOE Plan Drawing, attached as Appendix E.
- Caisson and Secant Pile Wall Capacity Summary Tables provided by WSP and attached as Appendix F.

2. Design Criteria

We reviewed drawing FO-001 *Foundation Notes*, the Structural Design Criteria document provided by WSP, as well as the geotechnical report. Our observations are discussed below.

2.1 Geotechnical Report

We note that, while the geotechnical report recommends the use of a mat foundation and rock anchors, the drawings show the foundation system to be caissons. Upon request, WSP provided documentation from the geotechnical engineer substantiating the foundation system they have selected (caissons) and the allowable caisson loads they used in their design. We recommend that these criteria be shared with the DOB, that the geotechnical report be updated as appropriate, and that the title and date of the report be updated in Drawing FO-001.00.

In addition, we note that the earthquake design data shown on FO-001.00 are different than what is provided in the geotechnical report dated 29 April 2016. We recommend that the EOR coordinate this data with the recommendations of the geotechnical report.

- Site Class: B (per geotechnical report), D (shown on dwg FO-001.00)
- S_{ds}: 0.187g (per geotech report), 0.295g (shown on dwg FO-001.00)
- S_{d1}: 0.049g (per geotech report), 0.117g (shown on dwg FO-001.00)

It should be noted, however, that the design data shown on FO-001.00 is more conservative than what is recommended in the geotechnical report.

We further note that the following information was missing from the geotechnical report:

- Recommendation for earthquake loads on permanent foundation walls are not included in the report.
- Recommendation for loads from adjacent buildings on permanent foundation walls are not included in the report.

2.2 Structural Design Criteria

Drawing FO-001 *Foundation Notes* generally includes the necessary design loads and other information pertinent to the structural design; however, we recommend the EOR review the following list of items that should also be included and provide the following additional design criteria:

- A loading schedule for different floor occupancy, including floor live load, partition loads and other superimposed dead loads (NYCBC 1603.1.1, 1603.1.2, 1603.1.10)
- Permissible Live load reductions, where allowed by code
- Roof snow loads (NYCBC 1603.1.4)
- Base shear for wind loads (NYCBC 1603.1.5)
- Flood design loads (or water head) (NYCBC 1603.1.8)
- Design criteria loading of foundation walls due to static and seismic earth pressures, surcharge, and hydrostatic pressures. (NYCBC 1603.1.9)

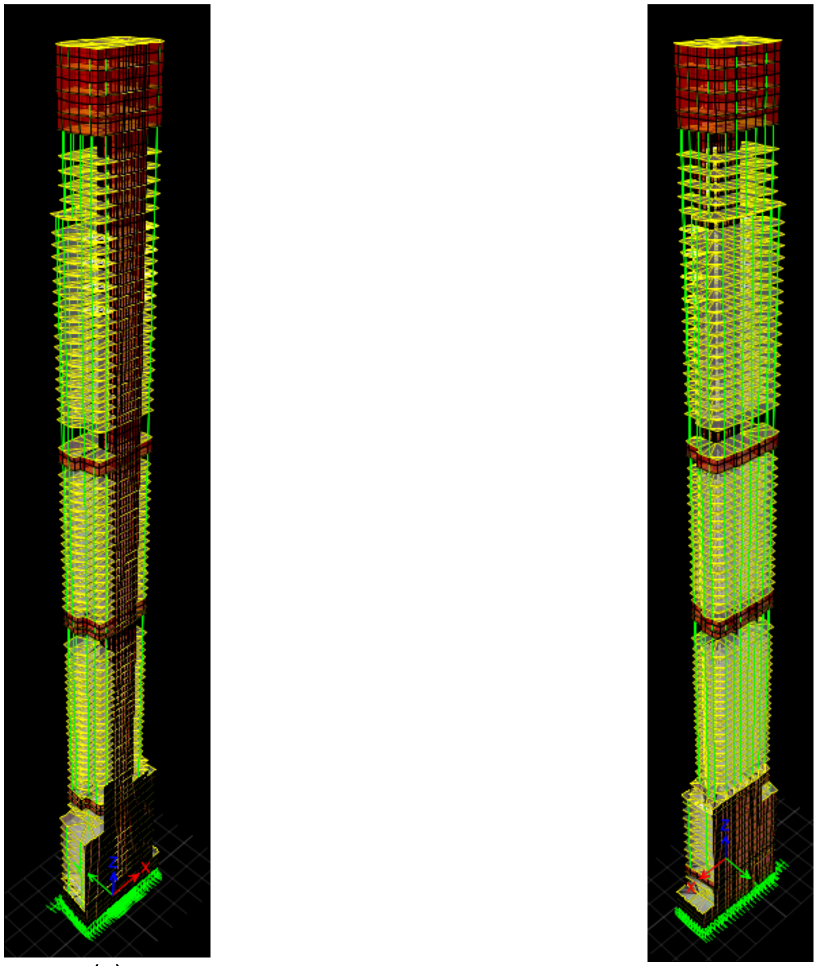
3. Superstructure Review

3.1 Architectural and Structural Drawings

We reviewed and compared the architectural drawings and WSP's structural foundation drawings, and found that the structural foundation drawings were in general conformance with the architectural drawings.

3.2 ETABS Model

A global building model developed using ETABS was provided by WSP. This model was reviewed, compared with the structural and architectural drawings, and updated as necessary to be consistent with the submitted structural and architectural drawings and code requirements. The model was used to obtain loads for the design checks of foundation elements. Figure 1 below shows different views of the ETABS model.



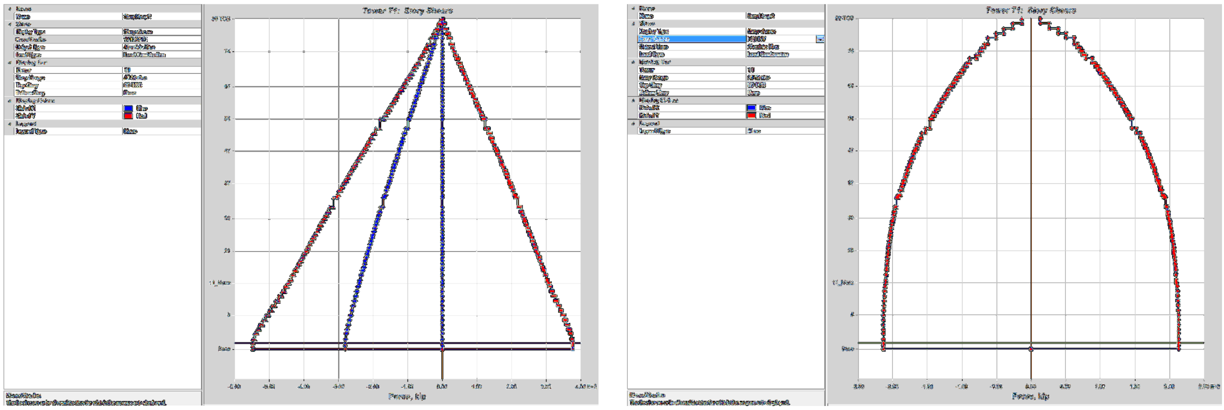
(a) (b)

Figure 1 – Global ETABS Model (a) Looking North, (b) Looking South

3.2.1 Base Shear Check

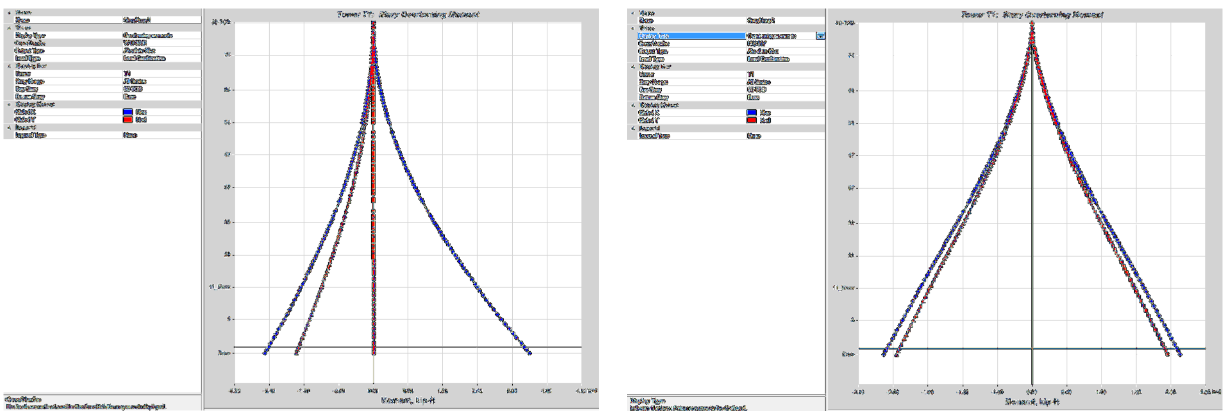
The ETABS model was used to compare the building base shear from earthquake loads shown on drawing FO-001.00. We could not compare wind load base shears as they were not provided, however, we generated our own code-conforming wind loads for use in our analysis and review.

We found that the base shear from code-prescribed earthquake loads match the base shear values reported in the design criteria drawing FO-001.00, and that base shear from code-prescribed wind loads generated are in scale for the size and type of building. Figures 2 and 3 below present the global shears and overturning moments taken from our ETABS model for wind and earthquake loads.



(a) (b)

Figure 2 - Global Model Shears for (a) Wind Loads (Envelope), and (b) Earthquake Loads (Envelope)



(a) (b)

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

3.3 Load Path

We reviewed a sampling of typical floors, walls, columns as well as foundation walls, caisson caps and caissons, and found they generally were acceptably proportioned for the size and type of building. The superstructure appears to have a continuous load path.

4. Pressure Slab

The pressure slab was checked for its global behavior with the building against the buoyancy force as well as locally for the reinforcement provided. A SAFE model was developed to obtain loads from the water uplift load combinations. Upward water pressures obtained using the information on the water table provided in the geotechnical report and downward loads from the ETABS model were combined in the SAFE model.

4.1 Global Behavior

The geotechnical report recommends the design ground water at El. +12 ft., meaning that the water head is equal to 20' – 71/2" ft.

A hand calculation was made to review the net uplift of the building, and it was found that the self-weight of the building (CDL+SDL) is greater than the buoyancy force. From this results, we believe that there will be no uplift issues under gravity-alone load cases.

4.2 Pressure Slab Reinforcement

The slab reinforcement of the 24" pressure slabs was checked using the SAFE model described above, using the information provided in the notes on drawing FO-100.00 and additional reinforcement shown in plan. Table 1 below summarizes the findings, while Figure 4 indicates the locations where the capacity of the pressure slab was reviewed.

Table 1 – Summary of Pressure Slab Capacity Check

Pressure Slab	Reinforcement	DCR Flexure	DCR Shear
M_{bottom} @ 1	#11 @ 6 + 3 - #11 @ 12	0.58	1.15
M_{bottom} @ 2	#11 @ 6 + 14 - #11 @ 6	0.33	0.99
M_{top} @ 3	#11 @ 6	0.60	0.26
M_{bottom} @ 4	#11 @ 6	0.61	0.72

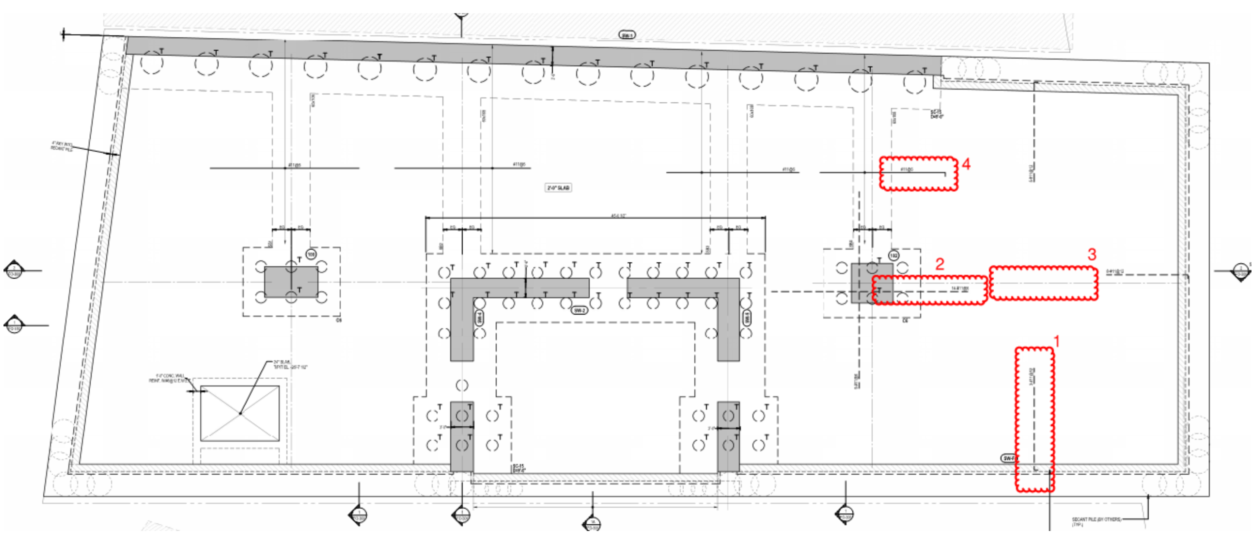


Figure 4 – Location of Pressure Slab Reinforcement Checks (FO-100.00)

To address the overstress identified in Table 1, we recommend the EOR revise the design of the pressure slab where it frames into the perimeter wall and to columns 100 and 102 for shear.

5. Secant Pile Wall

The ETABS model described in Section 3.2 of this report was used to obtain the axial loads in the secant pile wall and we compared these loads to the secant pile wall capacities listed in the summary table provided by WSP. Tables 2 to 5 below summarize the findings, while Figure 5 indicates the locations of the overstresses observed in the secant piles. It is important to note that only the maximum DCR per secant pile is shown on the figure.

Table 2 – Summary of Secant Pile Type "A" Axial Capacity Check

Caisson	Compression (kip)	DCR	Tension (kip)	DCR
370	916.98	0.92	-91.46	0.30
372	830.42	0.83	111.76	-
4298	968.44	0.97	124.57	-
8602	1116.07	1.12	99.34	-
8610	1267.66	1.27	47.76	-
8797	912.69	0.91	26.01	-
8801	927.64	0.93	115.21	-
8805	954.12	0.95	77.98	-
8809	1096.50	1.10	-25.93	0.09
8829	909.06	0.91	-127.77	0.43
9008	1323.81	1.32	-585.12	1.95
9014	1181.60	1.18	-419.90	1.40
9021	1044.29	1.04	-255.07	0.85
9058	779.96	0.78	83.27	-
9062	719.44	0.72	97.45	-
12201	915.79	0.92	-288.64	0.96

Table 3 – Summary of Secant Pile Type "B" Axial Capacity Check

Caisson	Compression (kip)	DCR	Tension (kip)	DCR
20	2761.70	0.66	-1421.93	1.05
21	3403.48	0.81	-1441.34	1.07
25	3547.47	0.84	-1513.88	1.12
66	3523.62	0.84	-1521.11	1.13
214	3532.99	0.84	166.21	-
219	3697.31	0.88	-827.35	0.61
261	3509.88	0.84	-1487.30	1.10
4105	3491.52	0.83	-1524.51	1.13
4351	3232.33	0.77	127.37	-
8308	2873.89	0.68	-1418.95	1.05
8713	2543.81	0.61	-263.43	0.20
8719	3020.64	0.72	-490.56	0.36
8753	2094.89	0.50	-1318.86	0.98
8757	1867.33	0.44	-917.90	0.68
8895	2717.14	0.65	-1423.93	1.05
8899	2794.59	0.67	-1419.78	1.05
8901	2845.28	0.68	-1425.34	1.06
9112	3824.78	0.91	164.42	-
9129	2728.08	0.65	27.04	-
9131	2971.84	0.71	81.21	-
9139	3720.56	0.89	175.64	-

Table 4 – Summary of Secant Pile Type "C" Axial Capacity Check

Caisson	Compression (kip)	DCR	Tension (kip)	DCR
48	3998.63	0.63	-1697.08	0.51
113	3887.29	0.61	-1666.54	0.50
339	3715.19	0.59	-1626.41	0.49
4062	3523.88	0.55	-1591.66	0.48
4078	3327.25	0.52	-1562.06	0.47
8739	2755.30	0.43	-1787.20	0.53
8741	2870.24	0.45	-1718.14	0.51
8743	2979.51	0.47	-1697.44	0.51
8745	3080.05	0.49	-1696.92	0.51
8747	3168.43	0.50	-1696.46	0.51

Table 5 – Summary of Secant Pile Type "24" Axial Capacity Check

Caisson	Compression (kip)	DCR	Tension (kip)	DCR
1	1613.58	0.70	-789.24	0.99
2	1889.33	0.82	-836.24	1.05
8959	1621.92	0.71	-786.82	0.98
8963	1629.94	0.71	-784.86	0.98
8967	1641.15	0.71	-784.18	0.98
8971	1655.76	0.72	-784.77	0.98
8975	1673.95	0.73	-786.72	0.98
8979	1694.13	0.74	-789.71	0.99
8983	1716.90	0.75	-793.84	0.99
8987	1742.37	0.76	-799.19	1.00
8991	1771.68	0.77	-806.05	1.01
8995	1805.21	0.78	-814.39	1.02
8999	1842.70	0.80	-824.02	1.03

We observed that there are compression and tension overstresses in the secant pile wall under combined gravity and wind loads. We recommend the EOR to revise the secant pile layout to address overstresses.

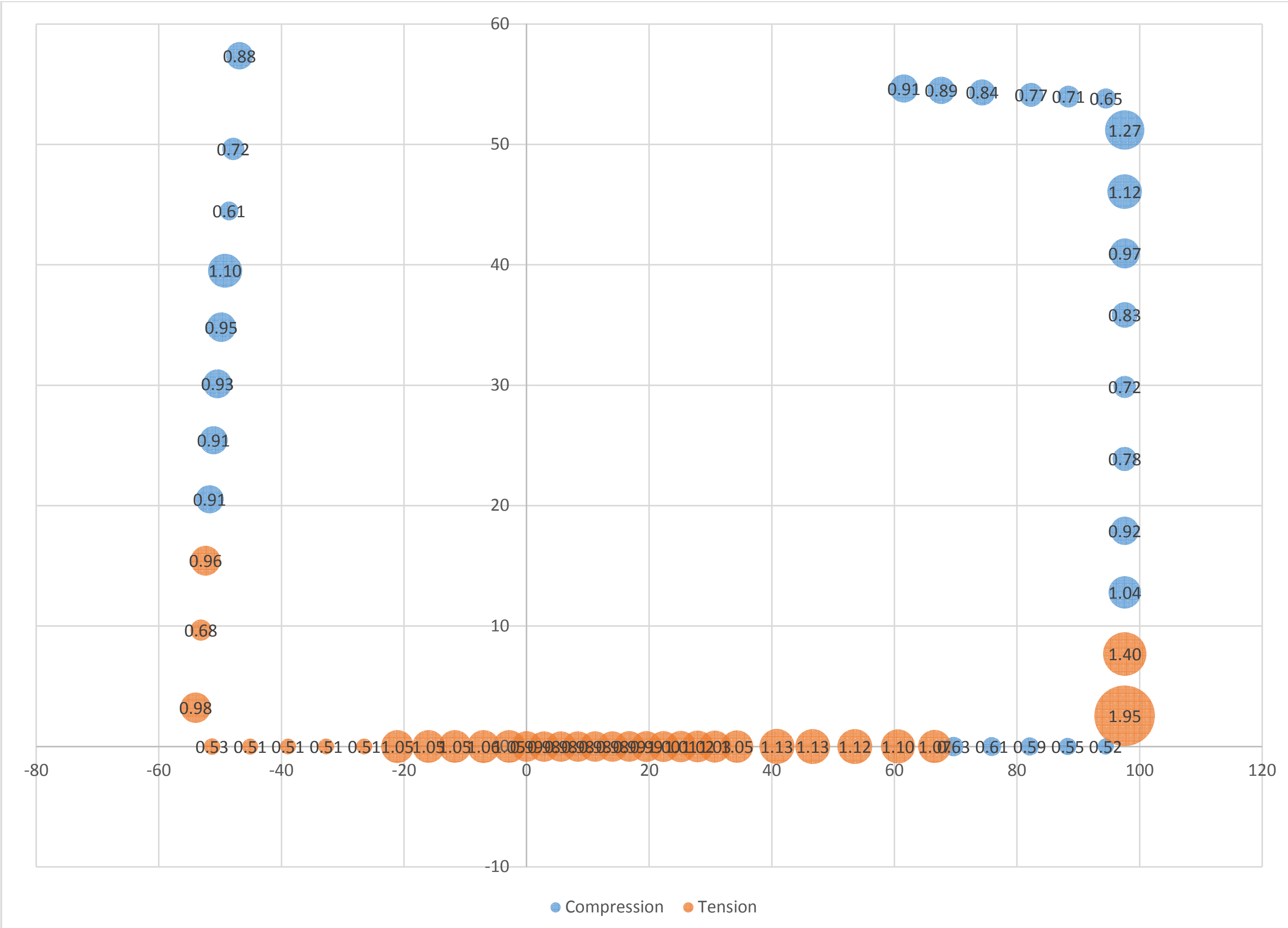


Figure 5 – Location of Secant Pile Overstresses

6. Caissons

6.1 Axial Capacity

The ETABS model described in Section 3.2 of this report was used to obtain the axial loads in the caissons. The caisson groups shown in plan were reviewed and we found that caisson designs were sufficient for their axial loads. Table 6 below summarizes the finding, while Figure 6 indicates the locations of caissons reviewed.

Table 6 – Summary of Caisson Axial Capacity Check

Caisson	Compression (kip)	DCR	Tension (kip)	DCR
3960	2217.68	0.92	192.55	0.16
4133	1868.59	0.78	183.26	0.15
8294	2197.47	0.92	94.13	0.08
8345	1665.12	0.69	377.30	0.31
8351	1717.23	0.72	387.68	0.32
8362	2243.08	0.93	20.62	0.02
8414	1217.23	0.51	40.65	0.03
8551	1844.71	0.77	6.98	0.01
8560	1833.62	0.76	50.87	0.04
300	5234.67	0.87	399.20	0.13
8491	4856.41	0.81	882.74	0.29
8512	5078.51	0.85	681.88	0.23
10253	5135.37	0.86	93.05	0.03
10485	5300.24	0.88	136.80	0.05
11222	5204.29	0.87	26.83	0.01

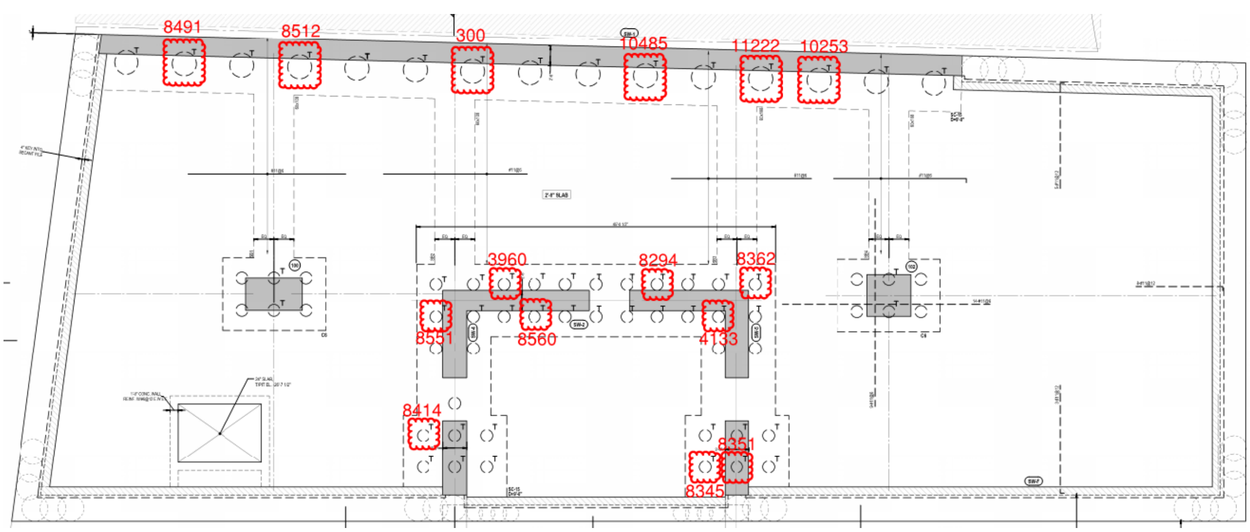


Figure 6 – Location of Caissons Checked (from FO-100.00)

6.2 Lateral Capacity

The average lateral load on the caissons was reviewed by comparing the total base shear from seismic and wind loads and the total horizontal capacity of the caissons. Table 7 below summarizes the results obtained.

Table 7 – Summary of Caisson Lateral Load Capacity Check

Load	Base Shear (kip)	Total Capacity (kip)	DCR
EQx	2100	5650	0.37
EQy	2100	5650	0.37
Wind x	2800	5650	0.5
Wind y	5477	5650	0.97

From the sample calculations, we believe that the caissons capacities provided in Drawing FO-200.00 are adequate for the building demand. We recommend that the geotechnical report be updated to include the allowable caisson loads. See also comments in Section 2.1.

7. Caisson Caps

7.1 Flexural Capacity

The flexural capacity of caisson cap type C6 under columns 100 and 102 was reviewed using the information shown on FO-200.00. Table 8 summarizes the results obtained.

Table 8 – Summary of Caisson Caps Flexural Capacity Check

Caisson Cap	Reinforcement	DCR
C6 Long Way	29 – #11	0.45
C6 Short Way	22 - #11	0.45

Per ACI 9.6.1.2 the minimum area of flexural reinforcement shall be the greater of (a) and (b).

$$\text{(a)} \quad \frac{3\sqrt{f'_c}}{f_y} b_w d$$

$$\text{(b)} \quad \frac{200}{f_y} b_w d$$

In this case, the governing equation is (a), requiring a minimum reinforcement area of 0.5%. The current area of flexural reinforcement shown for type C6 caisson cap is 0.19% in the short way and 0.36% in the long way. We recommend the EOR revise the caisson cap design of all caps to meet the requirement of minimum area of flexural reinforcement.

7.2 Shear Capacity

All caissons are within distance *d* from the face of the wall or column above, therefore not contributing to shear in the caisson caps.

8. Strap Beams

8.1 Flexural Capacity

The flexural capacity of strap beams was reviewed using the information shown on FO-100.00. Table 9 summarizes the results obtained.

Table 9 – Summary of Strap Beam Flexural Capacity Check

Strap Beam	Reinforcement	DCR
SB1	Top: 126 - #11 (7 layers)	0.67

From the sample calculations, we believe the flexural design of the strap beams is adequate.

8.2 Shear Capacity

The shear capacity of strap beams was reviewed using the information shown on FO-100.00. Table 10 summarizes the results obtained.

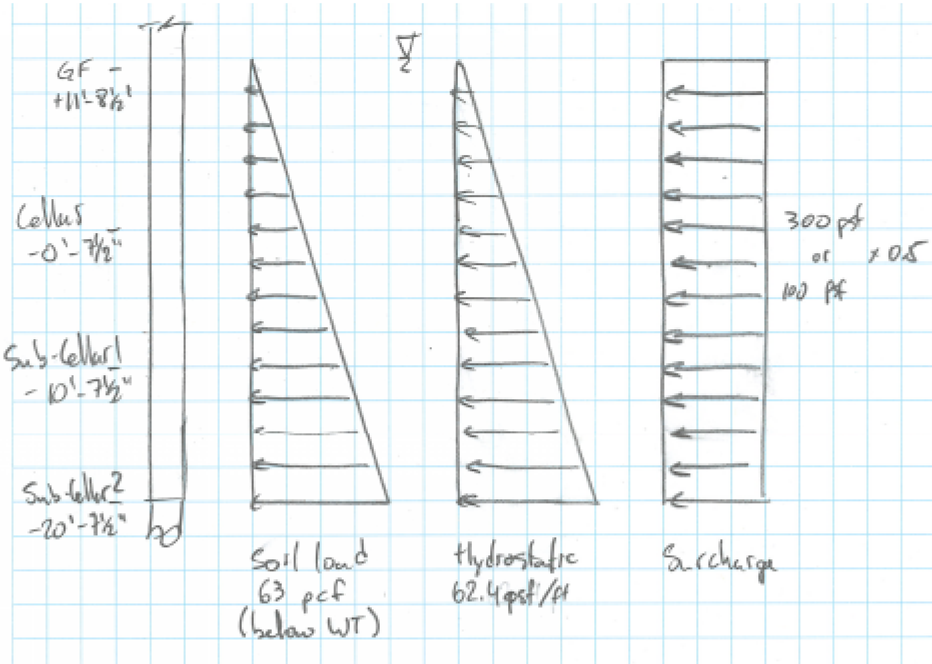
Table 10 – Summary of Strap Beam Shear Capacity Check

Strap Beam	Legs	Size	Spacing	DCR
SB1	6	#5	6”	0.27

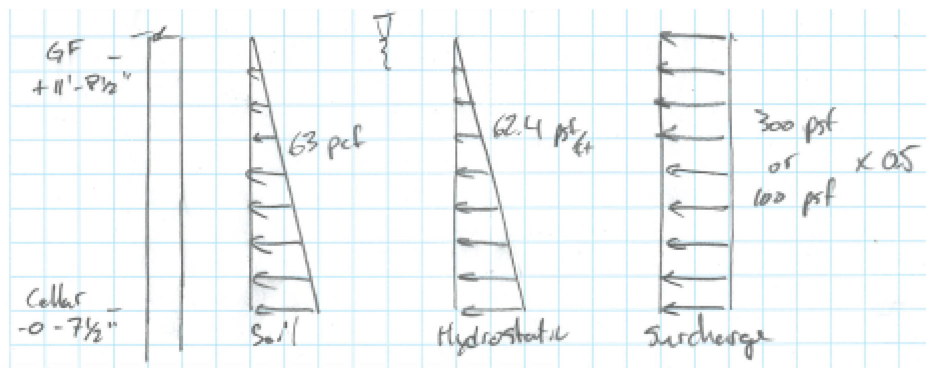
From the sample calculations, we believe the shear design of the strap beams is adequate.

9. Perimeter Foundation walls Acting as Retaining Walls

The design of the perimeter foundation retaining wall (East side of the building) and of the shear wall acting as a retaining wall (North side of the building) were reviewed based on the lateral pressure design data (see Figure 7) provided in the geotechnical report prepared by Langan, dated 29 April 2016, and the reinforcement details shown on 2/FO-300.00 and S-940.00.



(a)



(b)

Figure 7 – Lateral Soil, Hydrostatic and Surcharge Pressures (a) North Wall, (b) East Wall

9.1 Flexural Capacity

Table 11 below summarizes the results of our foundation wall flexural capacity checks.

Table 11 – Summary of Perimeter Retaining Walls Flexural Capacity Check

Wall	Reinforcement	DCR
North Wall	#9 @ 10	0.23
East Wall	#7 @ 12	0.78

From the sample calculations, we believe the flexural design of the retaining walls is adequate.

9.2 Shear Capacity

Table 12 below summarizes the results of our foundation wall shear capacity checks.

Table 12 – Summary of Perimeter Retaining Walls Shear Capacity Check

Wall	Reinforcement	DCR
North Wall	-	0.36
East Wall	-	0.45

From the sample calculations, we believe the shear design of the retaining walls is adequate.

10. Conclusions

In conclusion, we find that the geotechnical recommendations need be updated to include the design criteria used by the EOR in the design of the foundations and secant pile wall. Provided that the final geotechnical recommendations confirm the design criteria given in the drawings and other documents provided to us, we find the design of the foundation of 45 Broad Street to be in general conformance with the structural and foundation design provisions of the New York City Building Code. In some cases, however, we have noted overstresses and detailing issues that need to be addressed by the Engineer of Record. In these instances, we have recommended that the EOR revisit the design as indicated 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.

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BENJAMIN M. CORNELIUS
Partner-In-Charge

Appendix A Drawing List

STRUCTURAL SHEET LIST

SHEET NUMBER	SHEET NUMBER	DATE
FO-001	GENERAL NOTES & LEGEND	2016.11.07
FO-100	FOUNDATION PLAN	2016.11.07
FO-110	SUB-CELLAR 1 FRAMING PLAN	2016.11.07
FO-120	CELLAR FRAMING PLAN	2016.11.07
FO-150	PILE CAP REINFORCEMENT PLAN	2016.11.07
FO-200	TYPICAL FOUNDATION DETAILS 1	2016.11.07
FO-201	TYPICAL FOUNDATION DETAILS 2	2016.11.07
FO-202	TYPICAL FOUNDATION DETAILS 3	2016.11.07
FO-203	TYPICAL FOUNDATION DETAILS 4	2016.11.07
FO-300	FOUNDATION SECTIONS 1	2016.11.07
FO-301	FOUNDATION SECTIONS 2	2016.11.07
FO-310	FOUNDATION SECTIONS 3	2016.11.07
S-010	GROUND FLOOR FRAMING PLAN	2016.11.07
S-940	SHEAR WALL PLAN	2016.11.07
S-945	TYPICAL SHEARWALL DETAILS	2016.11.07
S-950	COLUMN SCHEDULE 1	2016.11.07
S-951	COLUMN SCHEDULE 2	2016.11.07
S-955	TYPICAL COLUMN DETAILS	2016.11.07
S-960	TYPICAL SUPERSTRUCTURE DETAILS 1	2016.11.07
S-961	TYPICAL SUPERSTRUCTURE DETAILS 2	2016.11.07
S-962	TYPICAL SUPERSTRUCTURE DETAILS 3	2016.11.07
S-963	TYPICAL SUPERSTRUCTURE DETAILS 4	2016.11.07
S-964	TYPICAL MANSONRY DETAILS	2016.11.07
S-980	TYPICAL STAIR SECTIONS	2016.11.07

ARCHITECTURAL SHEET LIST

SHEET NUMBER	SHEET NUMBER	DATE
A-098.00	SUB-CELLAR 2	2016.12.09
A-099.00	SUB-CELLAR 1	2016.12.09
A-100.00	CELLAR	2016.12.09
A-101.00	GROUND FLOOR	2016.12.09
A-102.00	GROUND FLOOR MEZZANINE	-
A-103.00	2ND FLOOR PLAN - MACHANICAL	2016.04.22
A-104.00	3RD TO 8TH FLOOR COMMERCIAL	2016.04.22
A-109.00	9TH FLOOR - COMMERCIAL	2016.04.22
A-110.00	10TH FLOOR - COMMERCIAL	-
A-111.00	11TH FLOOR - MECHANICAL	2016.04.22
A-112.00	11TH FLOOR - MEZZANINE	2016.04.22
A-113.00	12TH FLOOR - AMENITY	2016.04.22
A-114.00	13TH FLOOR - AMENITY	2016.04.22
A-115.00	14TH TO 29TH FLOOR - TIER 1	2016.04.22
A-130.00	30TH FLOOR - TIER 1	-
A-132.00	32TH FLOOR - TIER 1 - TRANSFER	2016.04.22
A-133.00	33RD FLOOR - OUTDOOR	2016.04.22

ARCHITECTURAL SHEET LIST

SHEET NUMBER	SHEET NUMBER	DATE
A-134.00	34TH FLOOR - MECHANICAL	2016.04.22
A-135.00	34TH FLOOR - MEZZANINE	2016.04.22
A-136.00	35TH - 50TH FLOOR AND 54TH - 63RD FLOOR TIER 2	2016.04.22
A-151.00	51ST FLOOR - TIER2 - TRANSFER	2016.04.22
A-152.00	52ND FLOOR - MECHANICAL	2016.04.22
A-153.00	53RD FLOOR - WINDBREAK	-
A-164.00	64TH, 66TH, 68TH, 70TH FLOOR - TIER 2 - LOWER	2016.04.22
A-165.00	65TH, 67TH, 69TH FLOORS - TIER 2 - UPPER	2016.04.22
A-171.00	71ST FLOOR - TIER 2 - TRANSFER	2016.04.22
A-172.00	72ND FLOOR - TIER 3	2016.04.22
A-176.00	76TH FLOOR - TIER 3 - TRANSFER	-
A-177.00	77TH FLOOR - MECHANICAL	2016.04.22
A-178.00	78TH FLOOR - MECHANICAL	2016.04.22
A-179.00	78TH FLOOR - MEZZANINE	-
A-180.00	79TH FLOOR - ELEVATOR MACHINE ROOM	2016.04.22
A-181.00	BULKHEAD	-

Appendix B Structural Design Criteria from WSP

1. CODES:

1.0 THE FOLLOWING CODES SHALL APPLY TO THE DESIGN, CONSTRUCTION, QUALITY CONTROL AND SAFETY AND STABILITY OF THE WORK PERFORMED ON THIS PROJECT. LATEST EDITIONS IN EFFECT AT THE TIME OF DESIGN SHALL BE USED UNLESS OTHERWISE NOTED.
1.1 ALL WORK SHALL CONFORM TO THE REQUIREMENTS OF THE 2014 NEW YORK CITY BUILDING CODE, WITH ALL GOVERNMENTAL AGENCIES HAVING JURISDICTION, AND WITH THE PROJECT SPECIFICATIONS.
1.2 SEISMIC DESIGN CONFORMS TO THE REQUIREMENTS OF THE NEW YORK CITY BUILDING CODE.
1.3 "BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE", ACI 318, AMERICAN CONCRETE INSTITUTE, AS AMENDED BY THE NEW YORK CITY BUILDING CODE.
1.4 "MANUAL OF STEEL CONSTRUCTION – LOAD AND RESISTANCE FACTOR DESIGN (LFRD)", THIRD EDITION, AMERICAN INSTITUTE OF STEEL CONSTRUCTION.
1.5 "BUILDING CODE REQUIREMENTS FOR MASONRY STRUCTURES (ACI 530/ASCE 5)" AND "SPECIFICATIONS FOR MASONRY STRUCTURES (ACI 530.1/ASCE 6)"

2. SEISMIC AND WIND CRITERIA

1. THE STRUCTURE HAS BEEN DESIGNED IN ACCORDANCE WITH THE LATEST NEW YORK CITY BUILDING CODE (NYCBC 2014).

2. WIND DESIGN DATA:

– BASIC WIND SPEED (3 SECOND GUST) = 98 mph
– WIND IMPORTANCE FACTOR = 1
– WIND EXPOSURE = C
– INTERNAL PRESSURE COEFFICIENT = ±0.18

3. EARTHQUAKE DESIGN DATA:

– SEISMIC IMPORTANCE FACTOR = 1
– STRUCTURAL OCCUPANCY CATEGORY: II
– S_S = 0.281g, S_1=0.073g
– SITE CLASS = D
– S_{DS} = 0.295g, S_{D1}=0.117g
– SEISMIC DESIGN CATEGORY = B
– SEISMIC FORCE RESISTING SYSTEM = ORDINARY REINFORCED CONCRETE SHEARWALLS
– DESIGN BASE SHEAR (V): E/W = 2100 kips; N/S = 2100 kips
– SEISMIC RESPONSE COEFFICIENT (C_S): E/W = 0.013; N/S = 0.013
– RESPONSE MODIFICATION FACTOR: R = 5
– ANALYSIS PROCEDURE USED = EQUIVALENT LATERAL FORCE PROCEDURE

4. STRUCTURAL SEPARATIONS, (NYCBC–1613.7): ALL STRUCTURES SHALL BE SEPARATED FROM ADJACENT STRUCTURES. WHEN A STRUCTURE ADJOINS A PROPERTY LINE NOT COMMON TO A PUBLIC WAY (TYPICALLY SIDE OR REAR LOT LINES), THAT STRUCTURE SHALL ALSO BE SET BACK FROM THE PROPERTY LINE BY AT LEAST 1 INCH FOR EACH 50 FEET OF HEIGHT AND A MINIMUM OF 1 INCH FOR STRUCTURES WITH HEIGHTS LESS THAN 50 FEET. SMALLER SEPARATIONS OR PROPERTY LINE SETBACKS SHALL BE PERMITTED WHEN JUSTIFIED BY RATIONAL ANALYSIS BASED ON MAXIMUM EXPECTED GROUND MOTIONS WITH A MINIMUM SEPARATION OF 1 INCH ALONG THE FULL HEIGHT OF THE STRUCTURE.

3. INSPECTIONS:

SPECIAL INSPECTIONS	NYC CODE REFERENCES
STRUCTURAL STEEL: WELDING	1704.3.1
STRUCTURAL STEEL: ERECTION & BOLTING	1704.3.3
STRUCTURAL COLD FORMED STEEL	1704.3.4
CONCRETE: CAST–IN–PLACE	1704.4
CONCRETE: PRE–CAST	1704.4
CONCRETE: PRE–STRESSED	1704.4
*CONCRETE TEST CYLINDERS (TR–2)	1905.6
*CONCRETE DESIGN MIX (TR–3)	1905.3
MASONRY	1704.5
SOILS: SITE PREPARATION	1704.7.1
SOILS: FILL PLACEMENT & IN–PLACE DENSITY	1704.7.2. & 1704.7.3
SOILS: INVESTIGATIONS (BORINGS/TEST PITS) (TR–4)	1704.7.4
PILE FOUNDATIONS & DRILLED PIER INSTALLATION (TR–5)	1704.8
PIER FOUNDATIONS	1704.9
UNDERPINNING	1704.9.1
WALL PANELS, CURTAIN WALLS & VENEERS – (ATTACHMENT TO BUILDING)	1704.10
SPRAYED FIRE–RESISTANT MATERIALS	1704.11
STRUCTURAL SAFETY–STRUCTURAL STABILITY	1704.19
EXCAVATION: SHEETING, SHORING & BRACING	1704.19 & 3304.4.1
FIRESTOP, DRAFTSTOP & FIREBLOCKING SYSTEMS	1704.25

*–THESE TESTS MUST BE PERFORMED BY A LICENSED CONCRETE TESTING LABORATORY.

PROGRESS INSPECTIONS	NYC CODE REFERENCES
FOOTING & FOUNDATION	109.3.1
FINAL	28–116.2.4.2 & 109.5 AND DIRECTIVE 14 (1975)

4. CONSTRUCTION – GENERAL

4.0 STRUCTURAL SEPARATIONS PER NYCBC 1617.3.2:
AT ANY PROPERTY LINE NOT COMMON TO A PUBLIC WAY,
"THE STRUCTURE SHALL BE SET BACK FROM THE PROPERTY LINE BY AT LEAST ONE INCH FOR EACH 50 FEET OF HEIGHT"
"SMALLER SETBACKS SHALL BE PERMITTED WHEN JUSTIFIED BY RATIONAL ANALYSIS BASED ON MAXIMUM EXPECTED GROUND MOTIONS"
4.1 THE CONTRACTOR SHALL BE SOLELY RESPONSIBLE FOR THE CORRECTNESS OF DIMENSIONS AND QUANTITIES, FOR FABRICATION PROCEDURES, FOR THE MEANS METHODS, TECHNIQUES, AND PROCEDURES OF CONSTRUCTION, AND FOR THE COORDINATION OF STRUCTURAL WORK WITH THE WORK OF ALL OTHER TRADES. REVIEW OF THE CONTRACTOR'S SUBMISSIONS DOES NOT RELIEVE THE CONTRACTOR OF THESE RESPONSIBILITIES.
4.2 PRIOR TO CONSTRUCTION, THE CONTRACTOR SHALL DETERMINE EXISTING CONDITIONS AND VERIFY DIMENSIONS, ELEVATIONS AND CLEARANCES. NOTIFY THE ARCHITECT AND STRUCTURAL ENGINEER IMMEDIATELY OF ANY DISCREPANCIES.
4.3 THE CONTRACTOR SHALL TAKE ALL NECESSARY MEASURES TO PREVENT DAMAGE TO THE EXISTING ADJACENT STRUCTURES.
4.4 THE CONTRACTOR SHALL COMPARE STRUCTURAL DRAWINGS WITH THE ARCHITECTURAL DRAWINGS; COORDINATE LOCATIONS OF OPENINGS, SLEEVES, SLAB DEPRESSIONS, DRAINS, INSERTS AND OTHER EMBEDDED ITEMS REQUIRED BY ALL TRADES. NOTIFY THE ARCHITECT AND STRUCTURAL ENGINEER OF ANY DISCREPANCIES REQUIRING CLARIFICATION PRIOR TO INSTALLATION OF AFFECTED WORK.
4.4.1 SEE ARCHITECTURAL DRAWINGS FOR FINISHES, FIREPROOFING (INCLUDING FIRESTOPPING), DETAILS AND LOCATIONS OF NON–LOAD BEARING MASONRY AND DRYWALL PARTITIONS, CURBS, DRAINS, SLOPES, EMBEDS AND RECESSES FOR CLADDING ANCHORAGE, ETC.
4.4.2 SEE MECHANICAL, PLUMBING AND ELECTRICAL DRAWINGS FOR DUCT AND PIPE OPENINGS AND SLEEVES, HANGERS, TRENCHES, ANCHOR BOLTS OR CONCRETE INSERTS FOR EQUIPMENT AND FIXTURES, SIZE AND LOCATION OF EQUIPMENT BASES (HOUSEKEEPING PADS). NOTE THAT HOUSEKEEPING PADS SHOWN ON THE STRUCTURAL DRAWINGS ARE APPROXIMATE AND FOR REFERENCE ONLY.
4.5 THIS PROJECT HAS BEEN DESIGNED FOR THE WEIGHTS OF MATERIALS AND SUPERIMPOSED LOADS INDICATED ON THE DRAWINGS. IT IS THE CONTRACTOR'S RESPONSIBILITY TO DETERMINE ALLOWABLE CONSTRUCTION LOADS AND TO PROVIDE PROPER DESIGN AND CONSTRUCTION OF FALSEWORK, FORMWORK, STAGING, BRACING, SHORING, ETC.
4.6 THE CONTRACTOR SHALL SUBMIT SHOP DRAWINGS FOR APPROVAL PER SPECIFICATION REQUIREMENTS. SUCH DRAWINGS SHALL INCLUDE ERECTION PLANS AND SEQUENCE OF OPERATIONS.
4.6.1 REPRODUCTION OF ANY PORTION OF THE STRUCTURAL CONTRACT DRAWINGS FOR RE–SUBMITTAL AS SHOP DRAWINGS IS PROHIBITED. SHOP DRAWINGS PRODUCED IN SUCH A MANNER WILL BE REJECTED AND RETURNED.
4.6.2 IN THE EVENT THAT SOME DETAILS OF THE STRUCTURE ARE NOT EXPLICITLY SHOWN OR NOTED ON THE DRAWINGS, THEIR CONSTRUCTION SHALL BE OF THE SAME TYPE AS FOR SIMILAR CONDITIONS WHICH ARE SHOWN AND NOTED, SUBJECT TO THE ENGINEER'S APPROVAL OF THEIR SUBMISSION.
4.7 WHERE MECHANICAL AND OTHER SUSPENDED LOADS EXCEED 100 POUNDS, DETAILS OF PROPOSED SYSTEMS SHALL BE SUBMITTED FOR REVIEW BY THE STRUCTURAL ENGINEER PRIOR TO INSTALLATION, UNLESS THEY ARE SPECIFICALLY INDICATED ON THE STRUCTURAL DRAWINGS.
4.8 NON–STRUCTURAL ITEMS SHOWN ON THE STRUCTURAL/FOUNDATION DRAWINGS
4.8.1 THE FOLLOWING NON–STRUCTURAL ITEMS MAY BE SHOWN ON THE STRUCTURAL AND/OR FOUNDATION DRAWINGS FOR THE PURPOSE OF CLARITY IN INTERFACE WITH STRUCTURAL AND/OR FOUNDATION WORK. ITEMS BELOW MAY NOT BE FULLY DEFINED ON THE STRUCTURAL/FOUNDATION DRAWINGS. THE INFORMATION FOR NON–STRUCTURAL ELEMENTS IS FURNISHED BY OTHER CONSULTANTS AS LISTED BELOW. ALL RFI AND SHOP DRAWINGS RELATED TO THESE NON–STRUCTURAL ITEMS SHALL BE SUBMITTED TO THE CONSULTANTS LISTED BELOW FOR THEIR REVIEW AND APPROVAL.

GEOTECHNICAL ENGINEER:

- FOUNDATION/UNDERSLAB WATERPROOFING, DAMPROOFING SYSTEMS
- WALL AND UNDERSLAB DRAINAGE SYSTEM, INCLUDING SUMP PITS, GRAVEL & PIPING, CLEANOUTS
- ROCK ANCHORS
- CAISSONS AND PILES, INCLUDING REINFORCEMENT, SECANT PILE WALL
- ROCK CONTOURS

ARCHITECT OF RECORD:

- SUMP PITS
- WATERPROOFING/DAMPROOFING APPLIED TO EXPOSED SURFACES, ELEVATOR OR SUMP PIT INTERIOR SURFACES
- PAINT
- FIREPROOFING
- CONCRETE CURBS: HEIGHT, WIDTH, EXTENT, LOCATION
- BRICK, BLOCK, TILE MASONRY, METAL PANELS, PRECAST FACADE PANELS, CURTAIN WALLS AND OTHER FACADE SYSTEMS.
- ROOFING SYSTEMS, DRAIN LOCATIONS, SLOPES TO DRAINS, FILLS, INSULATION, PAVERS OR GRAVEL
- FLOATING/SECONDARY SLABS

5. FOUNDATIONS – CAISSON SUPPORTED

5.1 GENERAL: FOUNDATIONS, FOUNDATION WALLS, FOUNDATION DRAINAGE, ETC. HAVE BEEN DESIGNED IN ACCORDANCE WITH THE RECOMMENDATIONS OF GEOTECHNICAL CONSULTANTS LANGAN REPORT ENTITLED GEOTECHNICAL ENGINEERING STUDY, FOR 45 BROAD STREET NEW YORK N.Y. DATED 12/30/2015.
5.2 SEE GEOTECHNICAL REPORT AND PROJECT SPECIFICATIONS FOR ADDITIONAL INFORMATION AND FOR REQUIREMENTS FOR EXCAVATION, BACKFILLING, FILLING AND PREPARATION OF THE FOUNDATION AND SLAB–ON–GROUND SUBGRADE. REQUIREMENTS CONTAINED IN THE GEOTECHNICAL REPORT ARE PART OF THIS WORK.
5.3 EARTHWORK:
5.3.1 THE CONTRACTOR SHALL VERIFY ALL EXISTING FIELD CONDITIONS THAT MAY AFFECT THE INSTALLATION OF FOUNDATIONS, AND SHALL REPORT ANY DISCREPANCIES OR CONFLICTS PRIOR TO STARTING WORK.
5.3.2 WHERE NEW FOUNDATION IS LOWER THAN EXISTING ADJACENT FOUNDATION, THE CONTRACTOR IS TO MAINTAIN SUPPORT OF EXISTING FOUNDATION. THE CONTRACTOR IS TO DETERMINE AND REPORT EXISTING CONDITIONS BEFORE STARTING WORK.
5.3.3 ALL UNDERPINNING, SHEETING, SHORING, OR OTHER CONSTRUCTION REQUIRED FOR THE SUPPORT OF ADJACENT PROPERTIES, BUILDINGS, SIDEWALKS, UTILITIES, ETC. SHALL BE SUBJECT TO SPECIAL INSPECTION AS REQUIRED BY CODE. THE CONTRACTOR SHALL RETAIN A LICENSED PROFESSIONAL ENGINEER ACCEPTABLE TO THE ENGINEER OF RECORD TO PROVIDE THE NECESSARY DESIGN AND THE REQUIRED INSPECTION. THE CONTRACTOR'S PROFESSIONAL ENGINEER SHALL PREPARE AND FILE THE REQUIRED FORMS FOR THE WORK WITH THE DEPARTMENT OF BUILDINGS.
5.3.4 MAINTAIN GROUND WATER LEVEL A MINIMUM OF ONE FOOT BELOW BOTTOM ELEVATION OF LOWEST FOUNDATION EXCAVATION. SEE GEOTECHNICAL REPORT FOR ANTICIPATED GROUND WATER LEVEL. THE CONTRACTOR IS RESPONSIBLE FOR ALL DEWATERING DURING CONSTRUCTION, REGARDLESS OF ACTUAL GROUND WATER LEVEL. EXCAVATION, CONCRETE WORK, AND BACKFILLING SHALL BE PERFORMED IN THE DRY.
5.3.5 WATER SHALL NOT BE ALLOWED TO STAND IN FOUNDATION EXCAVATIONS BEFORE OR AFTER CONCRETE IS PLACED.
5.3.6 NO BACKFILL SHALL BE PLACED AGAINST FOUNDATION WALLS UNTIL FLOORS BRACING THE WALLS HAVE BEEN IN PLACE FOR SEVEN DAYS OR ADEQUATE LATERAL BRACING IS INSTALLED.
5.3.7 IN NO CASE SHALL TRUCKS, BULLDOZERS, OR ANY OTHER HEAVY EQUIPMENT BE PERMITTED CLOSER THAN 8'–0" FROM ANY FOUNDATION WALL UNLESS APPROVED BY THE ENGINEER.
5.3.8 COMPACT SUBGRADE TO RECEIVE COMPACTED GRANULAR BASE (AND VAPOR RETARDER OR MEMBRANE WATERPROOFING) FOR SUPPORT OF CONCRETE SLABS ON GROUND (INCLUDING REINFORCED FRAMED SLABS WHERE APPLICABLE).
5.3.9 THE CONTRACTOR SHALL MAINTAIN MINIMUM REQUIRED BACKFILL ADJACENT TO EXPOSED FOUNDATIONS DURING CONSTRUCTION TO PREVENT FROST HEAVE OF FOUNDATIONS.
5.3.10 SEE TYPICAL DETAILS AND ARCHITECTURAL DRAWINGS, AND SPECIFICATIONS, FOR WATERPROOFING REQUIREMENTS (AND PERMANENT DEWATERING SYSTEM WHERE APPLICABLE).

CAISSON NOTES:

5.4 THE DESIGN AND INSTALLATION OF CAISSONS, CAISSONS CAPS, AND RELATED CONSTRUCTION IS TO CONFORM TO THE REQUIREMENTS SET FORTH IN THE NEW YORK CITY BUILDING CODE AND THE SPECIFICATIONS.
5.5. DRILLED CAISSONS SHALL HAVE THE FOLLOWING PARAMETERS AS PER RECOMMENDATION FROM GEOTECHNICAL ENGINEER:
A. FOR CAISSON (SERVICE) LOAD CAPACITIES
COMPRESSION, TENSION, LATERAL SEE CAISSON SCHEDULE ON FO–200.
B. UPLIFT AND LATERAL FIELD TEST ARE REQUIRED.
5.6 CAISSON INSTALLATION TO BE SUPERVISED BY A LICENSED PROFESSIONAL ENGINEER.
5.7 CAISSON OPERATIONS TO BE IN ACCORDANCE WITH THE NEW YORK CITY BUILDING CODE, AND ARE SUBJECT TO SPECIAL INSPECTION IN ACCORDANCE WITH NEW YORK CITY BUILDING CODE.
5.8 A PLAN SHOWING THE IDENTIFICATION OF ALL CAISSONS AND A CAISSONS NUMBERING PLAN IS TO BE SUBMITTED TO THE ENGINEER OF RECORD FOR FILING WITH THE BUILDING DEPARTMENT PRIOR TO COMMENCEMENT OF DRIVING OPERATIONS.
5.9 LOAD TESTS (IF NECESSARY) SHALL BE PERFORMED AS PER THE REQUIREMENTS OF THE NEW YORK CITY BUILDING CODE. LOCATION OF TEST CAISSONS TO BE APPROVED BY THE ENGINEER OF RECORD.
5.10. ALL CAISSON GROUPS AND CAISSON CAPS TO BE CONCENTRIC WITH COLUMNS AND WALLS ABOVE UNLESS OTHERWISE NOTED ON PLAN.
5.11. RECORDS OF PENETRATION OF EVERY CAISSON AND THE BEHAVIOR OF SAME DURING DRILLING ARE TO BE SUBMITTED TO THE ENGINEER OF RECORD.
5.12 AN "AS–DRIVEN" CAISSON LOCATION PLAN AND CAISSON LOGS ARE TO BE SUBMITTED TO THE ENGINEER OF RECORD FOR APPROVAL, NO CAISSON CAPS ARE TO BE PLACED BEFORE THIS IS DONE.
5.13 ESTIMATED AVERAGE CAISSON LENGTH IS PER GEOTECHNICAL CONSULTANT. CAISSON LENGTH COULD VARY DUE TO ACTUAL SOIL CONDITION.
5.14 FOR DETAILS REFER TO GEOTECHNICAL CONSULTANT REPORT AND SPECIFICATIONS.

CAISSON INSTALLATION PROCEDURE

5.15 MOBILIZE TO SITE.
5.17 SET UP RIG ON PROPER LOCATION AND PLUMB MAST. 5.
5.18 DRILL IN CAISSON USING DUPLEX DRILLING METHODS. CLEAN WITH WATER ONLY. NOTE: OUTSIDE CASING TO BE ADVANCED 2–DIAMETERS OR 2–FOOT MINIMUM PRIOR TO CLEANING.
5.19 CASING IS DRILLED–IN TO BEDROCK AS INDICATED ON DRAWINGS. DRILL ROCK SOCKET AS SHOWN ON DRAWINGS.
5.20 FLUSH HOLE CLEAN OF SPOILS. IF PILE TIP IS BELOW GWT, FLUID LEVEL INSIDE CASING TO BE MAINTAINED AT TOP OF PILE DURING CLEAN OUT. A BUCKET OR AUGER MAY BE USED TO CLEAN HOLE. (AIR MAY BE USED IN COMPACTED TILL OR ROCK).
5.21 INTRODUCE REINFORCING THREADBAR WITH SPACERS AND PUSH TO THE BOTTOM OF THE PILE.
5.22 PLACE 3/4–INCH DIAMETER PVC GROUT TUBE TO THE BOTTOM OF ROCK SOCKET AND GROUT THE CAISSON FROM THE BOTTOM TO DISPLACE THE DRILLING FLUID. CONTINUE GROUTING UNTIL GOOD GROUT FLOWS OUT THE TOP OF THE PILE.
5.23 CUT THREADBAR TO PROPER ELEVATION AS SHOWN ON CONTRACT DRAWINGS.

6. FOUNDATION CONCRETE

6.1 ALL CONCRETE SHALL BE NORMAL WEIGHT CONTROLLED CONCRETE, UNLESS OTHERWISE NOTED, AND SHALL COMPLY WITH THE 2014 NEW YORK CITY BUILDING CODE AND WITH THE ACI BUILDING CODE (ACI 318), AS AMENDED BY THE 2014 NEW YORK CITY BUILDING CODE. THE CONTRACTOR SHALL SUBMIT CONCRETE MIX DESIGNS TO THE STRUCTURAL ENGINEER FOR APPROVAL PRIOR TO PLACING ANY CONCRETE.
6.2 CONTRACTOR SHALL MAKE HIMSELF THOROUGHLY FAMILIAR WITH THE SUPERSTRUCTURE DRAWINGS RELATED TO THE FOUNDATION CONTRACT INCLUDING BUT NOT LIMITED TO THE FOLLOWING:
1. FIRST FLOOR FRAMING PLAN AND SECTIONS.
2. SHEAR WALLS, DOWELS
3. COLUMN SCHEDULE
4. TYPICAL MASONRY DETAILS
6.2.1 NO CONCRETE FOOTING, FOUNDATION PIER, OR FOUNDATION WALL SHALL BE PLACED UNTIL SUBGRADE FOR SAME HAS BEEN APPROVED BY A LICENSED PROFESSIONAL ENGINEER.
6.3 CONCRETE STRENGTH AT 28 DAYS SHALL BE AS FOLLOWS, UNLESS OTHERWISE NOTED:

f'c (PSI)*	LOCATION IN THE STRUCTURE
[10,000 PSI]	FOUNDATION WALLS INCLUDING BUTTRESSES, GRADE BEAMS, AND WALLS
[10,000 PSI]	SOIL SUPPORTED OR FRAMED SLABS ON GROUND (AIR–ENTRAINED)

*f'c CONCRETE STRENGTH VALUES GIVEN ABOVE ARE ALL MINIMUM VALUES – HIGHER STRENGTHS MAY BE CALLED FOR ON THE PLANS.
6.3.1 ALL CONCRETE EXPOSED TO THE ELEMENTS OR SUBJECT TO VEHICULAR TRAFFIC AFTER COMPLETION OF THE STRUCTURE SHALL BE AIR–ENTRAINED 6% ±1%
6.4 IF THE SLAB ON GROUND (SOIL SUPPORTED OR FRAMED) IS PLACED BEFORE THE COLUMNS ABOVE, EITHER:
A. PROVIDE OPENINGS IN THE SLAB TO RECEIVE COLUMNS. NOTE THAT OPENINGS ARE REQUIRED WHEN COLUMN CONCRETE f'c EXCEEDS 1.4 TIMES SLAB CONCRETE f'c. (E.G. WHEN COLUMN CONCRETE f'c=6,000 OR GREATER AND SLAB CONCRETE f'c=4,000).
B. SLAB IS PLACED WITHOUT OPENINGS (MAY BE POSSIBLE DUE TO LOWER STRENGTH COLUMN CONCRETE).
6.4.1 SHOULD OPTION "B" BE FOLLOWED, DOWELS EXTENDING ABOVE FOOTINGS, PILE CAPS, OR PIERS ARE TO BE LENGTHENED TO PROVIDE REQUIRED LAP SPLICE ABOVE TOP OF SLAB.
6.5 THE CONTRACTOR MUST SUBMIT REINFORCING STEEL SHOP DRAWINGS TO THE STRUCTURAL ENGINEER FOR APPROVAL. NO CONSTRUCTION IS TO BE STARTED UNTIL THE SHOP DRAWINGS ARE REVIEWED BY THE ENGINEER.
6.6 VERTICAL CONSTRUCTION JOINTS IN WALLS SHALL BE SPACED A MAXIMUM OF FORTY FEET APART, AND SHALL BE LOCATED AT LEAST 4'–0" AWAY FROM ANY SUPPORTING COLUMN OR WALL OPENING. DETAILS OF JOINTS SHALL CONFORM TO THE PROJECT SPECIFICATIONS AND STRUCTURAL DRAWINGS. LOCATIONS SHALL BE SUBMITTED FOR REVIEW PRIOR TO PREPARING REINFORCING SHOP DRAWINGS. NO HORIZONTAL CONSTRUCTION JOINTS WILL BE ALLOWED IN FOUNDATIONS, PIERS, SLABS, GRADE BEAMS, OR STRAP BEAMS.
6.7 NO CONCRETE SHALL BE PLACED UNTIL THE CONTRACTOR HAS INSTALLED ALL EMBEDDED ITEMS SUCH AS, BUT NOT LIMITED TO: BOXED OPENINGS, DOWELS, (ANCHOR) BOLTS, INSERTS, ANCHORS, PLATES, DOVETAIL ANCHOR SLOTS, SLEEVES, WATERSTOPS, JOINT MATERIALS, AND SIMILAR ITEMS FURNISHED BY OTHER TRADES. NO ELECTRICAL DUCTS MAY BE EMBEDDED IN FOOTINGS, PILE CAPS, OR MAT FOUNDATIONS.
6.7.1 WHERE EMBEDDED PLATES OR INSERTS HAVE NOT BEEN CAST INTO THE CONCRETE, POST–INSTALLED ANCHORS WILL BE PERMITTED ONLY WHEN IT IS PROVEN TO THE SATISFACTION OF THE STRUCTURAL ENGINEER THAT THE ANCHORING SYSTEM WILL NOT DAMAGE THE CONCRETE. THE CONTRACTOR'S P.E. MUST SUBMIT STRENGTH DESIGN METHOD CALCULATIONS DEMONSTRATING THAT THE PROPOSED INSTALLATION PROVIDES MORE THAN THE REQUIRED LOAD CARRYING CAPACITY AS A GROUP.
6.7.2 POST–INSTALLED ANCHORS SHALL BE AS CALLED FOR ON THE PLANS AND DETAILS, OR, IN THE ABSENCE OF SPECIFIC REQUIREMENTS, ANCHORS MAY BE UNDERCUT, ADHESIVE, OR EXPANSION TYPES AS MANUFACTURED BY HILTI, POWERS, ITW RED HEAD, OR SIMPSON.
6.7.3 WHEN INSTALLING EXPANSION BOLTS OR ADHESIVE ANCHORS THE CONTRACTOR SHALL LOCATE REBARS TO AVOID DRILLING OR CUTTING OF ANY EXISTING REINFORCING BARS, OR DAMAGING CONCRETE. FOLLOW MANUFACTURER'S INSTRUCTIONS FOR INSTALLATIONS.
6.8 WATERSTOPS SHALL BE INSTALLED AT ALL CONSTRUCTION JOINTS BELOW THE HIGHEST GROUNDWATER ELEVATION CITED IN GEOTECHNICAL REPORT, OR HIGHER IF REQUIRED BY THE STRUCTURAL DRAWINGS OR FIELD CONDITIONS.
6.9 FOR SOIL SUPPORTED SLABS ON GROUND (AREAWAYS, RAMPS, ETC. INCLUDED), SEE FOUNDATION DRAWINGS FOR TYPICAL DETAILS OF POUR STRIPS, SAWCUT JOINTS, CONSTRUCTION JOINTS, VAPOR RETARDER, POROUS FILL, SUB–SLAB DRAINS, FILTER FABRIC, ETC. WHERE APPLICABLE, PROVIDE ISOLATION JOINTS AROUND COLUMNS.
6.10 FOR PIER SIZES SEE FOUNDATION DRAWINGS. WHERE PIER IS REQUIRED BUT NOT SHOWN ON THE DRAWINGS, THE PIER SHALL PROJECT 8 INCHES FROM ALL SIDES OF THE COLUMN ABOVE, AND PIER CONCRETE STRENGTH SHALL BE AT LEAST 70 PERCENT OF COLUMN CONCRETE STRENGTH.
6.11 WHERE A PIER IS INDICATED ON THE FOUNDATION PLAN BUT IS ELIMINATED IN THE FILED (GOOD BEARING MATERIAL HIGHER THAN ASSUMED) THE STRUCTURAL ENGINEER SHALL BE NOTIFIED, SINCE DEPTH OF FOOTING MAY NEED TO BE INCREASED.
6.12 CHAMFER ALL EXPOSED CONCRETE CORNERS ¾" X ¾" MINIMUM UNLESS OTHERWISE NOTED ON ARCHITECTURAL DRAWINGS.
6.13 FOR CONCRETE SURFACES WHICH ARE TO RECEIVE FINISHES, SEE ARCHITECTURAL DRAWINGS. COORDINATE EXPOSED CONCRETE FINISHES WITH THE ARCHITECTURAL DRAWINGS AND PROJECT SPECIFICATIONS.
6.14 CORE DRILLING OF FOUNDATIONS, WALLS, PIERS, BEAMS OR SLABS SHALL NOT BE PERMITTED UNLESS AUTHORIZED IN WRITING BY THE STRUCTURAL ENGINEER.
6.15 WHEN PLACING FOUNDATION CONCRETE AGAINST AN ADJACENT EXISTING STRUCTURE'S FOUNDATION, INSTALL AT LEAST A ONE INCH THICKNESS OF EXTRUDED OR EXPANDED POLYSTYRENE (SEE NOTE 7.16) IN THE INTERFACE BETWEEN EXISTING AND NEW STRUCTURES.
6.16 LOAD BEARING VOID FORMERS SHOWN ON THE STRUCTURAL DRAWINGS SHALL BE RIGID CLOSED CELL EXTRUDED POLYSTYRENE CONFORMING TO ASTM C578. RIGID VOID FORMERS SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH, AT YIELD OR AT 10 PERCENT DEFORMATION, OF 100 POUNDS PER SQUARE INCH. NON–LOAD BEARING POLYSTYRENE BOARDS PLACED BETWEEN NEW AND EXISTING ADJACENT STRUCTURES SHALL BE EXPANDED OR EXTRUDED POLYSTYRENE SIMILAR TO ABOVE, BUT COMPRESSIVE STRENGTH SHALL BE BELOW 25 POUNDS PER SQUARE INCH.

7. SUPERSTRUCTURE CONCRETE

7.1 ALL CONCRETE SHALL BE NORMAL WEIGHT CONTROLLED CONCRETE, UNLESS OTHERWISE NOTED, AND SHALL COMPLY WITH THE 2014 NEW YORK CITY BUILDING CODE AND WITH THE ACI BUILDING CODE (ACI 318), AS AMENDED BY THE 2014 NEW YORK CITY BUILDING CODE. THE CONTRACTOR SHALL SUBMIT CONCRETE MIX DESIGNS TO THE STRUCTURAL ENGINEER FOR APPROVAL PRIOR TO PLACING ANY CONCRETE.
7.2 CONCRETE STRENGTH AT 28 DAYS SHALL BE AS FOLLOWS, UNLESS OTHERWISE NOTED:

f'c (PSI)	LOCATION IN THE STRUCTURE
[5,000* PSI]	SLABS AND BEAMS *EXCEPT WHERE A HIGHER STRENGTH IS CALLED FOR ON THE PLANS
[***]	COLUMNS – **SEE COLUMN SCHEDULE
[***]	SHEARWALLS AND LINK BEAMS – **SAME STRENGTH AS COLUMNS

7.2.1 ALL CONCRETE EXPOSED TO THE ELEMENTS AFTER COMPLETION OF THE STRUCTURE SHALL BE AIR–ENTRAINED 6% ±1% .
7.2.2 FOR SLABS ON GROUND, SOIL SUPPORTED OR FRAMED, SEE "FOUNDATION CONCRETE" NOTE 7.4.
7.3 NO CONCRETE SHALL BE PLACED UNTIL THE CONTRACTOR HAS INSTALLED ALL EMBEDDED PLATES, ANCHORS, INSERTS, DOVETAIL SLOTS, ETC. NECESSARY TO PROVIDE SUPPORT FOR MULLIONS, APPLIED FINISHES, PARTITIONS, PIPES, DUCTS, EQUIPMENT, ETC. AS REQUIRED BY THE ARCHITECTURAL, M.E.P., AND STRUCTURAL DRAWINGS.
7.3.1 WHERE EMBEDDED PLATES OR INSERTS HAVE NOT BEEN CAST INTO THE CONCRETE, POST–INSTALLED ANCHORS WILL BE PERMITTED ONLY WHEN IT IS PROVEN TO THE SATISFACTION OF THE STRUCTURAL ENGINEER THAT THE ANCHORING SYSTEM WILL NOT DAMAGE THE CONCRETE. THE CONTRACTOR'S P.E. MUST SUBMIT STRENGTH DESIGN METHOD CALCULATIONS DEMONSTRATING THAT THE PROPOSED INSTALLATION PROVIDES MORE THAN THE REQUIRED LOAD CARRYING CAPACITY AS A GROUP.
7.3.2 POST–INSTALLED ANCHORS SHALL BE AS CALLED FOR ON THE PLANS AND DETAILS, OR, IN THE ABSENCE OF SPECIFIC REQUIREMENTS, ANCHORS MAY BE UNDERCUT, ADHESIVE OR EXPANSION TYPES AS MANUFACTURED BY HILTI, POWERS, ITW RED HEAD, OR SIMPSON.
7.3.3 WHEN INSTALLING EXPANSION BOLTS OR ADHESIVE ANCHORS THE CONTRACTOR SHALL LOCATE REBARS TO AVOID DRILLING OR CUTTING OF ANY EXISTING REINFORCING BARS, OR DAMAGING CONCRETE. FOLLOW MANUFACTURER'S INSTRUCTION FOR INSTALLATION.
7.4 WHERE BRICK VENEER EXCEEDS 16 INCHES IN HEIGHT, PROVIDE VERTICAL DOVETAIL SLOTS AT 24 INCHES MAXIMUM SPACING IN ALL BACKUP CONCRETE SURFACES.
7.5 THE CONTRACTOR SHALL VERIFY LOCATIONS AND DIMENSIONS OF ALL SLOTS, PIPE SLEEVES, DUCT OPENINGS, AND ANY OTHER CONCRETE PENETRATIONS AS REQUIRED FOR THE OTHER TRADES.

SHOP DRAWINGS SHOWING COMPOSITE LAYOUT OF ALL PENETRATIONS MUST BE SUBMITTED TO THE STRUCTURAL ENGINEER FOR APPROVAL PRIOR TO CONSTRUCTION.

7.6 ALL PLUMBING AND ELECTRICAL SLOTS (OPENINGS) SHALL BE FILLED WITH CONCRETE TO THE FULL DEPTH OF THE SLAB AFTER PIPES AND/OR CONDUITS, AND THEIR SLEEVES, HAVE BEEN INSTALLED.
7.7 NO PIPES OR CONDUITS EXCEEDING ⅓ SLAB THICKNESS IN OUTER DIAMETER, NOR OVER 2 INCH OUTSIDE DIAMETER, SHALL BE EMBEDDED IN A STRUCTURAL CONCRETE SLAB. NO PIPES OR CONDUITS SHALL BE SPACED CLOSER THAN 3 DIAMETERS ON CENTER, NOR PASS WITHIN 24 INCHES OF COLUMN FACE, UNLESS LAYOUT IS SUBMITTED FOR REVIEW AND APPROVAL BY THE STRUCTURAL ENGINEER. JUNCTION BOXES MAY BE PLACED IN THE CONCRETE SLAB, BUT SHALL NOT EXCEED 4½ x 4½ x 3½ INCHES IN DEPTH, AND SHALL BE SEPARATED FROM OTHER JUNCTION BOXES BY NOT LESS THAN 8 INCHES OF CONCRETE.
7.8 CHAMFER ALL EXPOSED CONCRETE CORNERS ¾ INCH X ¾ INCH MINIMUM UNLESS OTHERWISE NOTED ON ARCHITECTURAL DRAWINGS.
7.9 FOR CONCRETE SURFACES WHICH ARE TO RECEIVE FINISHES, SEE ARCHITECTURAL DRAWINGS. COORDINATE EXPOSED CONCRETE FINISHES WITH THE ARCHITECTURAL DRAWINGS AND PROJECT SPECIFICATIONS. FOR FLOOR FLATNESS/LEVELNESS TOLERANCES SEE SPECIFICATIONS.
7.10 NO CORE DRILLING OF CONCRETE STRUCTURE (WALLS, COLUMNS, BEAMS, SLABS, ETC.) SHALL BE PERMITTED UNLESS AUTHORIZED IN WRITING BY THE STRUCTURAL ENGINEER.
7.11 ALL MEMBERS IN THE FLOOR SYSTEM, INCLUDING BEAMS, BRACKETS, COLUMN CAPITALS, AND HAUNCHES SHALL BE PLACED MONOLITHICALLY. WHERE NECESSARY, VERTICAL CONSTRUCTION JOINTS MAY BE MADE AT MID–SPAN OF BEAM OR SLAB, USING APPROVED POUR JOINTS AND ADDITIONAL REINFORCEMENT AS SHOWN ON DETAILS.
7.12 NO CONCRETE FLOOR SYSTEM IS TO BE PLACED UNTIL AT LEAST TWO HOURS HAVE ELAPSED AFTER COMPLETION OF PLACEMENT OF THE SUPPORTING COLUMNS AND WALLS.
7.13 WHEN PLACING SUPERSTRUCTURE CONCRETE ADJACENT TO AN EXISTING STRUCTURE, MAINTAIN THE GAP DEFINED ON THE PLANS BETWEEN NEW AND EXISTING STRUCTURES (FINISH TO FINISH SURFACES), WHICH IS REQUIRED TO PREVENT "POUNDING" BETWEEN STRUCTURES.
7.14 LOAD BEARING VOID FORMERS SHOWN ON THE STRUCTURAL DRAWINGS SHALL BE RIGID CLOSED CELL EXTRUDED POLYSTYRENE CONFORMING TO ASTM C578. RIGID VOID FORMERS SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH, AT YIELD OR AT 10 PERCENT DEFORMATION, OF 100 POUNDS PER SQUARE INCH. NON–LOAD BEARING VOID FORMERS MAY BE EXPANDED OR EXTRUDED POLYSTYRENE SIMILAR TO ABOVE, BUT COMPRESSIVE STRENGTH SHALL BE BELOW 25 POUNDS PER SQUARE INCH.
7.15 ALL WORK MARKED S.S. (SUPERSTRUCTURE) IN FOUNDATION DRAWINGS SHALL BE PART OF THE SUPERSTRUCTURE CONTRACT.
7.16 TEMPORARY SHORING AND RESHORING OF FLOOR CONSTRUCTION SHALL CONFORM TO REQUIREMENTS OF THE SPECIFICATIONS AND/OR CONTRACT DRAWINGS. TEMPORARY SHORING AND RESHORING SHALL REMAIN IN PLACE AT LEAST 28 DAYS AFTER PLACEMENT OF CONCRETE.
7.17 NO DEVIATION FROM THE CONSTRUCTION DOCUMENTS SHALL BE PERMITTED WITHOUT THE EXPRESS WRITTEN CONSENT OF THE STRUCTURAL ENGINEER.

LEGEND:

a.	*	INDICATES ADDITIONAL WIND BARS
b.	[...]	INDICATES THE BOTTOM OF FOUNDATION WALL ELEVATION
c.	<...>	INDICATES THE TOP OF FOUNDATION WALL ELEVATION
d.	(...)	INDICATES BOTTOM OF PILE CAP ELEVATION
e.	Fxxxx	INDICATES FOOTING MARK 10 T/SF
f.	(XXxXX)	INDICATES SIZE OF PIER IN INCHES, FIRST DIMENSION SHOWN IS IN THE EAST–WEST DIRECTION.
g.	⊕	INDICATES ROCK ANCHOR.
h.	←	INDICATES DRAIN DIRECTION
i.		INDICATES CLEANOUT
j.		INDICATES ADDITIONAL TOP REINFORCEMENT AT SUPPORTS
k.		INDICATES ADD'L BOTTOM REINFORCING AT SUPPORTS
l.		INDICATES ADDITIONAL TOP REINFORCEMENT CONTINUOUS BETWEEN SUPPORTS
m.		INDICATES ADDITIONAL BOTTOM REINFORCEMENT CONTINUOUS BETWEEN SUPPORTS
n.	1ST & 4TH LAYERS / 2ND & 3RD LAYERS	INDICATES ORDER OF BAR PLACEMENT AS SHOWN ON PLAN.
o.		INDICATES CHANGE IN ELEVATION
p.		INDICATES CONCRETE COLUMN/SHEARWALL/FOUNDATION WALL
q.		INDICATES CONCRETE COLUMN/SHEARWALL BELOW
r.		INDICATES CONCRETE WALL
s.		INDICATES SLAB OPENING INDICATES SLAB OPENING AND/OR SLOT OR ZONE FOR SLEEVES (FIRST DIMENSION IS IN EAST–WEST DIRECTION)
t.		INDICATES COLUMN ABOVE OR BELOW
u.		INDICATES COLUMN ABOVE AND BELOW
v.		INDICATES SLOPING COLUMN ABOVE OR BELOW
w.	XX	INDICATES COLUMN DESIGNATION
x.	P	INDICATES POST DESIGNATION
y.	SW–X	INDICATES SHEARWALL DESIGNATION
z	S	INDICATES TEMPORARY SHORING

8. STEEL REINFORCEMENT

8.1 DEFORMED STEEL REINFORCING BARS, INCLUDING STIRRUPS AND TIES, SHALL BE GRADE 60 WITH A MINIMUM TENSILE STRENGTH OF 90,000 PSI PER ASTM A615.
8.1.1 HIGHER YIELD STRENGTHS MAY BE REQUIRED AT LOCATIONS DESIGNATED ON THE DRAWINGS, SUCH AS GRADE 75 PER ASTM A615 OR GRADE 80 PER ASTM A615 OR ASTM A706.
8.1.2 VERTICAL REINFORCEMENT IN COLUMNS AND/OR SHEARWALLS MAY BE SPECIFIED TO HAVE A YIELD STRENGTH OF 97 KSI, PURSUANT TO THE NEW YORK CITY DEPARTMENT OF BUILDINGS BULLETIN 2010–003. SEE THE COLUMN SCHEDULE AND/OR SHEARWALL DETAILS.
8.2 WELDED WIRE FABRIC SHALL CONFORM TO ASTM A185 AND ASTM A82.
8.2.1 WELDED WIRE FABRIC SHALL BE SUPPLIED IN SHEETS EXCEPT FOR SLAB ON GROUND CONSTRUCTION, WHERE ROLLS MAY BE USED.
8.2.2 LAP WELDED WIRE FABRIC ONE FULL MESH PLUS 2 INCHES, OR 6 INCHES, WHICHEVER IS GREATER.
8.3 WELDING OF REINFORCING BARS SHALL NOT BE PERMITTED UNLESS SPECIFICALLY CALLED FOR OR APPROVED BY THE STRUCTURAL ENGINEER. IN SUCH APPLICATIONS, WELDABLE DEFORMED REINFORCING BARS CONFORMING WITH ASTM A706, GRADE 60, SHALL BE USED.
8.3.1 WELDING ELECTRODES USED IN WELDING OF REINFORCING BARS SHALL BE E90XX, LOW HYDROGEN. CONFORM TO REQUIREMENTS OF AWS D1.4, STRUCTURAL WELDING CODE – REINFORCING STEEL.
8.4 EPOXY COATING OF REINFORCEMENT MAY BE REQUIRED AT LOCATIONS WHERE EXPOSURE TO CORROSIVE ENVIRONMENT IS LIKELY (SUCH AS PARKING AREAS, DRIVEWAYS, SIDEWALKS, SPLASH ZONES, ETC.). SEE PLANS, SECTIONS, AND DETAILS FOR LOCATIONS WHERE EPOXY COATED BARS ARE REQUIRED.
8.5 THE CONTRACTOR SHALL FURNISH AND INSTALL ALL CHAIRS, BOLSTERS, SPACERS, TIES, REBARS, CONCRETE BRICKS, ETC., NECESSARY TO SECURE AND SUPPORT REINFORCEMENT WHILE PLACING CONCRETE. PROVIDE SUPPORTS AND SPACERS WITH NON–CORROSIVE [PLASTIC] TIPS AT ALL LOCATIONS WHERE THE CONCRETE SURFACE IN CONTACT WITH SUCH TIPS IS EXPOSED.
8.6 ALL REINFORCEMENT MARKED CONTINUOUS SHALL BE LAPPED AT SPLICES AND CORNERS IN CONFORMANCE WITH LAP SPLICE TABLES IN THE TYPICAL DETAILS, EXCEPT WHERE OTHERWISE SHOWN ON THE DRAWINGS. STAGGER SPLICES AT HEAVILY REINFORCED LOCATIONS. SPLICE TOP BARS AT CENTER BETWEEN SUPPORTS AND BOTTOM BARS AT SUPPORTS. HOOK TOP BARS AT DISCONTINUOUS ENDS. (SPLICE LOCATIONS DO NOT APPLY TO COMBINED FOOTINGS AND MAT FOUNDATIONS, WHERE UPLIFT FORCES MAY CAUSE A REVERSAL OF STRESSES. SEE PLANS FOR SPLICE REQUIREMENTS.)
8.7 SEE GENERAL NOTES REGARDING SUBMISSION OF REINFORCEMENT SHOP DRAWINGS.
8.8 THE STRUCTURAL ENGINEER OR HIS QUALIFIED FIELD REPRESENTATIVE MUST CHECK AND APPROVE ALL REINFORCEMENT PRIOR TO CONCRETE PLACEMENT.
8.9 MINIMUM CONCRETE COVER FOR REINFORCEMENT SHALL BE AS FOLLOWS:
8.9.1 INTERIOR SLABS AND INTERIOR WALL SURFACES
¾" #11 BAR AND SMALLER
1 ½" #14 & #18
8.9.2 INTERIOR BEAM, GIRDERS, STIRRUPS AND TIES 1 ½"
8.9.3 COLUMNS:
1 ½" TIES
2" VERTICAL BARS #14 AND SMALLER
2 ½" VERTICAL BARS #18
3" VERTICAL BARS #20
8.9.4 CONCRETE EXPOSED TO WEATHER OR EARTH:
1 ½" #5 AND SMALLER, W.W.F.
2" #6 THROUGH #14
2 ½" #18
3" #20
8.9.5 CONCRETE PLACED AGAINST EARTH 3"

9. MASONRY

9.1 NOTES FOR MASONRY ARE CONTAINED ON THE TYPICAL MASONRY DETAILS DRAWING IN THE S–960 SERIES.

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PRELIMINARY, NOT FOR CONSTRUCTION

XXXX-XX-XX	XXXX	
2016-11-07	FOUNDATION AND EXCAVATION FOR BID	2
2016-04-22	ISSUED FOR DESIGN DEVELOPMENT	1

CETRARUDDY

FOUNDATION NOTES

FO-001.00

As Noted

1590109

Appendix C Structural Design Criteria Narrative from WSP

45 Broad Street

Madison Equities, LLC

Pizzarotti IBC, LLC

CetraRuddy Architecture

December 30, 2016

WSP

QM

Issue/revision	Issue 1	Revision 1	Revision 2	Revision 3
Remarks				
Date	12-30-2016			
Prepared by	JL			
Signature				
Checked by	HM			
Signature				
Authorized by	SM			
Signature				
Project number	B1590-109			
File reference				

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Contents

1 Design Criteria

DESIGN LOADS

1.1 DESIGN DEAD LOADS

Dead loads are calculated from the known self-weight of the materials used for the construction of the frame.

1.2 DESIGN SUPERIMPOSED DEAD LOADS

Additional allowance is made for fixed finishes and services as follows:

Residential Floors (ceiling, partitions, finishes)	15 psf
Balconies	30 psf
Lobby/public spaces (ceiling, dense finishes)	30 psf
Retail (ceiling, dense finishes)	30 psf
Mechanical room (ceiling, suspended services, partitions)	30 psf
Elevator/stair lobbies within core (Ceiling, suspended services and dense finishes)	30 psf
Roof (Finishes, insulation, tapered slab, ceiling)	30 psf

1.3 DESIGN LIVE LOADS

The following loads have been adopted in the design:

Residential	40 psf
Balconies	100 psf
Staircases	100 psf
Main roof (access for maintenance only)	40 psf
Mechanical areas	150 psf
Public areas (lobby)	100 psf

The building structure will be checked for the loadings applied from the proposed temporary cranes and hoists by the Contractor, with the capacity of the structure being adjusted where necessary.

1.4 WIND LOADS

Wind loads acting on the main building frame and the various elements of cladding were determined by requirements from NYCBC 2014.

1.5 EARTHQUAKE LOADS

Static Analysis using New York City Building Code – 2014 Edition

S_s = 0.281 g
S_1 = 0.073 g
Seismic Importance Factor = 1.0
Site Class = D

Ordinary Reinforced Concrete Shearwalls

R = 5

1.6 CLADDING LOADS

Unitised window wall 30 psf

1.7 TEMPORARY HOIST AND CRANE LOADS

The permanent structure will be designed to support the design loads provided by the Contractor from the temporary cranes and hoists.

1.8 ELEVATOR LOADS

All elevator shaft walls and elevator machine room slabs will be designed for elevator loadings provided by the elevator consultant.

1.9 FAÇADE ACCESS EQUIPMENT LOADS

The structures will be designed to support the window cleaning equipment loads to be provided by the façade access consultant.

1.10 MECHANICAL EQUIPMENT REPLACEMENT LOADS

All equipment replacement in and around the building is to be undertaken in such a manner as not to exceed the imposed loadings indicated on the WSP PB loading plans.

1.11 CONSTRUCTION LOADS

To be determined by the Contractor.

2 Other Design Criteria

2.1 DEFLECTION, GENERALLY

Vertical floor deflections for concrete floor construction:

Calculation of deflections includes long-term effects after installation of partitions/façade:

Spandrel beam/slab edge live load + super-imposed dead load deflection: ½”

Super-imposed + live load deflection:	span / 480 (beam supported at each end)
	span / 240 (cantilever beam)

2.2 HORIZONTAL SWAY

Earthquake

Sway deflection of any one story: minimum of h/250 or ½”

2.3 DURABILITY OF THE STRUCTURE

The structure is to have a design life of 50 years. Some structural elements, such as those with concrete wearing surfaces and corrosion protection will require periodic inspection and maintenance.

2.4 FIRE RESISTANCE PERIODS

The following fire resistance periods are adopted in the design of the building:

Beams and slabs:	2 hours
Columns:	3 hours

Material Properties

Concrete

Shearwalls:	f’c = 14,000psi to 8,000psi	E = 7,080ksi to 5,100ksi
Columns:	f’c = 14,000psi to 8,000psi	E = 7,080ksi to 5,100ksi
Link Beams:	f’c = 14,000psi to 8,000psi	E = 7,080ksi to 5,100ksi
Floor Slabs:	f’c = 6,000psi to 8,000psi	E = 4,415ksi to 5,100ksi
Bearing Foundations:	f’c = 10,000psi	E = 5,700ksi
Foundation Walls:	f’c = 14,000psi to 10,000psi	E = 7,080ksi to 5,700ksi

Grout:

Grout around anchor bolts and under base plates is to be a non-metallic non-shrink or expansive grout.

Reinforcement:

Deformed reinforcing bars	ASTM A615 Gr.60

Structural Steelwork:

Hot rolled steel sections	ASTM A992 Gr.50
HSS sections	ASTM A500 Gr.B
Plate and misc. steel	ASTM A572 Gr.50
Channels and angles	ASTM A36

3 Design Standards

3.1 STATUTORY CODES OF PRACTICE

New York City Building Code - 2014

ACI-318 Building Code Requirements for Structural Concrete and Commentary

ACI-530-08: Building Code Requirements and Specifications for Masonry Structures and Related Commentaries

AISC-13thed.: LRFD Manual of Steel Construction

4 Design References

Other publications used include:

IBC-2009: International Building Code

ASCE7-05 Minimum Design Loads for Buildings and Other Structures

AISC Design Guide 11 – Floor Vibrations Due To Human Activity

4.1 COMPUTER PROGRAMS

RAM Structural System Version 14.02.01

SAFE Version 12.0

ETABS non-linear Version 2015

4.2 UNITS

The structural calculations will be completed using the following units.

Length:	feet and inches
Mass:	Kip / g
Force:	Kip
Stress:	K/in^2
Moment:	Kip-ft
Velocity:	Miles/hour
Acceleration:	ft/s^2 and milli-g

Appendix D Geotechnical Report

AMENDED GEOTECHNICAL ENGINEERING STUDY for

45 Broad Street
New York, New York

Prepared For:

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29 April 2016
170394201

LANGAN

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FIGURES

APPENDICES

INTRODUCTION

This amended report updates the results of our amended geotechnical engineering study for the proposed development of 45 Broad Street in Manhattan, New York. The purpose of this study was to develop recommendations for foundations and other geotechnical aspects of design and construction. Our work was performed in accordance with our approved 19 November 2015 proposal. Our study included a review of available information, field investigations, engineering evaluation, and development of geotechnical recommendations in accordance with the 2014 New York City Building Code. Amendments to our 23 November 2010 report were made to:

1. Include information from a supplementary subsurface investigation performed in January and February of 2016;
2. Account for new design drawings prepared by the architect (CetraRuddy) and subsequent discussions with the project team and Madison 45 Broad Development;
3. Account for new foundation drawings prepared by the structural engineer (WSP) in March 2016.

Elevations given are based on the survey prepared by Empire State Layout, Inc., dated 21 January 2016, and are with respect to the North American Vertical Datum (NAVD88) unless otherwise noted.

SITE DESCRIPTION

The 45 Broad Street site is on the east side of Broad Street between Exchange Place and Beaver Street in lower Manhattan, New York. The site is identified as Block 25, Lot 7 on the New York City Tax Maps and is currently vacant. The site is within the block bound by Exchange Place on the north, Beaver Street on the south, Broad Street on the west, and William Street on the east. Existing buildings are adjacent to the site on the north, south, and east. Broad Street borders the site on the west. A New York City Transit (NYCT) tunnel is located under Broad Street. A site location map is presented in Figure 1.

The vacant site is T-shaped with about 63 feet of frontage on Broad Street and a site area of about 12,600 square feet (SF), with surface elevation varying from about el 9 to el 11. An eight-story structure with one cellar level was demolished in 2007 to make way for the previous owner's proposed redevelopment. The former cellar was backfilled with demolition debris to sidewalk grade with the former foundations, including piles and pile caps and basement slab, left in place.

Adjacent Buildings

Existing structures adjacent to the site on the north, south, and east are shown in Figure 6:

41 Broad Street – Claremont Preparatory School

The Claremont Preparatory School (41 Broad Street) north of the site is a nine- to twelve-story brick and stone structure with a footprint of about 11,000 SF built in 1929. Available architectural drawings indicate that 41 Broad Street has two below-grade levels with the subcellar level having a finished-floor elevation about 28 feet below the adjacent sidewalk grades(about el -17.5). Available foundation drawings show the structure supported by spread footings. Bearing capacity was not indicated on the available plans. Construction drawings appear to indicate that, along the southern end of the site (adjacent to 45 Broad), the foundations consist of piers bearing on bedrock constructed by way of a continuous cofferdam.

25 Broad Street

25 Broad Street is a T-shaped lot to the east occupied by a 20-story brick and stone structure with a 263-foot frontage along Exchange Place, built around 1900. The building previously had an about 50-foot-wide section that extended to the south, adjacent to 41 and 45 Broad Street to the east. This 4,200-square-foot extension was demolished to be part of the previous 45 Broad Street development scheme. Available architectural drawings show that the entire building footprint of 25 Broad Street, including the demolished southern part, has one cellar level. The finished-floor elevations of the below-grade levels are not known, and no foundation drawings are available for this structure. A steam-line easement running in the north-south exists within the part of 25 Broad Street that was demolished.

40 Exchange Place

Beyond 25 Broad Street to the east is 40 Exchange Place, a 20-story brick and stone commercial building with one below-grade level, built in 1902. The finished-floor elevations of the below-grade levels are not known, and no foundation drawings are available for this structure.

15 William Street

Adjacent to 25 Broad Street to the southeast is 15 William Street, a 44-story concrete residential structure with below-grade levels that extend about 45 feet below the surrounding grades (about el -34.5) built in 2005. The foundation wall and excavation support system for 15 William Street consists of a permanent reinforced secant pile wall drilled into the underlying bedrock.

55 Broad Street

55 Broad Street, adjacent to the south, is a brick building varying from 6 to 31 stories, built in 1968. A one-story extension borders the project site to the southeast. Available drawings show that the building has one below-grade level at about el -7.5 and that the structure is supported on driven H-piles bearing on bedrock.

Adjacent NYCT Subway Structure

The existing NYCT subway tunnels and structures for the BMT and IND J, M, and Z lines run beneath Broad Street about 20 feet west of the site; in addition, the Broad Street station (servicing lines J and Z) is nearby. NYCT drawings (Broad Street Station, South-End, 1928) show that the subway consists of a reinforced concrete box constructed using cut-and-cover methods. Vents in the Broad Street sidewalk are as close as about 10.5 feet to the property line. The base of the rail closest to the site is at about el -12.5. The tunnel foundation level is at about el -16.5, which is about 28 feet below the adjacent sidewalk grades. Because the proposed construction will be within 200 feet of the subway tunnel, NYCT approval of excavation and foundation construction is required to obtain building permits.

PROPOSED DEVELOPMENT

According to CetraRuddy's architectural drawings, the project will consist of about 8,950 square feet of development with an 83-story (plus mechanical penthouse) tower. The tower will extend to about 1,150 feet above grade and will have about 30-foot setback from the south property line along Broad Street. The top of the ground floor slab will be about el 11.4. The development in the rear "hammerhead" portion of the site is not proposed.

The building will include three cellar levels below the podium to be used for storage and amenities, including a swimming pool. The top of lowest cellar slab will be about 32 feet below sidewalk grade; the corresponding elevation is about el -20.7.

The tower will be concrete and will have a central structural core extending the entire height of the structure, with perimeter columns carrying the remaining load. The foundation loads and contact pressure at the base of the tower is not yet available at the time of this report; however WSP expects the contact pressure to be below 40tsf.

REVIEW OF PUBLISHED INFORMATION

Regional Geology

The United States Geological Survey "Bedrock and Engineering Geologic Maps of New York County and Parts of Kings and Queens Counties, New York, and Parts of Bergen and Hudson Counties, New Jersey" (see Figure 2) shows the bedrock formation underlying the site is Manhattan Schist.

Pleistocene glacial activity modified the landscapes and surficial features of Manhattan, Brooklyn, Queens, and Long Island. Glaciers scoured uplands and deposited varying amounts of till (an unsorted mixture of sand, clay and boulders) across the lowlands and valleys. The USGS surficial geology map indicates that the site is underlain by glacial outwash deposits generally consisting of sand and gravel. See Figure 3 for the USGS surficial geology map.

Historical Land Use

We reviewed the "Sanitary & Topographical Map of the City and Island of New York" (Viele, 1856), which indicates the east portion of the site near Broad Street is on manmade land and the west part of the site was a meadow. Before being filled, Broad Street was an inlet from the East River known as Broad Canal. See Figure 4 for the relevant part of the Viele Map.

Flood Hazard

We reviewed the Federal Emergency Management Agency (FEMA) Preliminary Flood Insurance Rate Map (FIRM), dated 5 December 2013 (Community Panel No. 360497 0088 G). According to the Preliminary FIRM, the western part of the site is within Zone X (areas within the 0.2 percent annual chance floodplain, i.e., 500-year flood). The eastern part of the site is within Zone AE (areas within the 1 percent annual chance floodplain, i.e., 100-year flood), which has a base flood elevation of el 11 NAVD88. Design of the building must follow the flood protection requirements of the NYCT and ASCE-24. The relevant part of the Preliminary FIRM is presented in Figure 5.

SUBSURFACE EXPLORATION

A summary of our subsurface explorations performed in August 2007 and February 2016 are presented below.

2007 Borings

Six borings (B-1 through B-6) were drilled as part of our 2007 subsurface exploration. All borings were drilled by Craig Test Boring, Inc. with a CME track-mounted drill rig, under Langan's full-time special inspection. The borings were advanced using mud rotary drilling techniques and a tricone roller bit with drilling fluid and steel casing providing soil support. Borings were advanced to between 59 and 65 feet below grade.

The upper 10 feet of each boring was drilled without sampling to permit the boring to be advanced through demolition debris and the remnant cellar-floor slab. Standard Penetration Test (SPT)[1] N-values were measured and soil samples were typically obtained beginning at about 10 feet below the existing site grades and at 5-foot intervals thereafter. Samples were retrieved using a standard 2-inch outside-diameter split-spoon sampler driven by a 140-pound automatic hammer in accordance with ASTM D1586. NX-size rock cores were obtained at each boring location in accordance with ASTM D2113. Rock core recovery[2] and rock quality designation (RQD)[3] was recorded for each core run.

Recovered soil samples were visually examined and classified in the field in accordance with the Building Code. Soil classifications, N-values, and other field observations were recorded on field logs. See Appendix A for the boring logs and Figure 6 for the boring location plan.

2016 Borings

Two borings (B-7 and B-8) were drilled in the rear of the lot ("hammerhead") as part of our 2016 supplemental subsurface exploration program. The borings were drilled by Craig Geotechnical Drilling Co., Inc. with a truck-mounted drill rig under Langan's full-time special inspection. . The borings were advanced using mud-rotary drilling techniques and a tricone roller bit with drilling fluid and steel casing providing soil support. Both borings were advanced to 55 feet below grade.

[1] The Standard Penetration Test is a measure of the soil density and consistency. The SPT N-value is defined as the number of blows required to drive a 2-inch outside diameter split-barrel sampler 12-inches, after an initial penetration of 6-inches, using a 140-pound hammer free falling from a height of 30-inches.

[2] Core recovery is defined as the ratio of the total length of rock recovered to the total core run length, expressed as a percent.

[3] The RQD is defined as the ratio of the summation of each rock piece greater than 4-inches in length for NX cores to total core run length, expressed as a percent.

The upper 10 feet of each boring was drilled without sampling to permit the boring to be advanced through demolition debris and the remnant cellar floor slab. SPT N-values were measured and soil samples were typically obtained beginning at about 10 feet below the existing site grades and at 5-foot intervals thereafter. Samples were retrieved using a standard 2-inch outside-diameter split-spoon sampler driven by a 140-pound automatic hammer in accordance with ASTM D1586. NX-size rock cores were obtained at each boring location in accordance with ASTM D2113. Rock core recovery and RQD were recorded for each core run.

Recovered soil samples were visually examined and classified in the field in accordance with the Building Code. Soil classification, N-values, and other field observations were recorded on field logs. See Appendix A for the boring logs and Figure 6 for the boring location plan.

2016 Cone Penetration Tests (CPTs)

Two Cone Penetration Tests (CPT-1, CPT-2) were performed on 1 February 2016 in accordance with ASTMD-5778 as part of our supplemental subsurface exploration. The CPTs were performed by Craig Geotechnical Drilling Co., Inc. under the special inspection of Langan. A truck-mounted CPT rig was used to hydraulically push a 1.4-inch-diameter (36mm) electric cone penetrometer to about 35 feet (CPT-1) and 38 feet (CPT-2).

The upper 15 feet of each CPT was pre-drilled to penetrate through the demolition debris and the remnant cellar-floor slab. The cone penetrometer was pushed at an estimated rate of about 0.75 in/sec (20mm/s) and readings were taken every 0.5 to 2.0 inch. Seismic shear-wave velocity tests were performed approximately every 5 feet. Seven shear-wave tests were performed at CPT-1, and eight at CPT-2. See Figure 6 for CPT locations and Appendix E for the CPT report prepared by Craig Geotechnical Drilling Co., Inc.

2016 Test Pit

One test pit (TP-1) was excavated by J. Coffey Contracting Inc., Flushing, New York, from 17 through 22 February 2016 under the full-time special inspection of Langan. The purpose of the test pit was to explore the adjacent foundation condition at 55 Broad Street. The test-pit indicated the cellar slab for 55 Broad Street extends to about el -5.25 (which appears to be slightly higher than el -7.5 depicted on available drawings), and that foundation pile caps extend to about el -12.25. The test pit was backfilled to existing grade with excavated material upon completion of the exploration.

See Figure 6 for the test pit location and Appendix D for the test pit sketch and selected photographs.

Groundwater Observation Wells

Three groundwater monitoring wells were installed in completed borings B-1, B-6, and B-7 to monitor the groundwater level at the site. The wells consisted of 1¼-inch or 2-inch diameter PVC riser pipes and 10-foot- or 20-foot-long well screens with well depths ranging between about 26 and 49 feet. The water levels were measured during the exploration. Observation well construction logs are provided in Appendix B.

Laboratory Testing

Samples obtained during our 2007 and 2016 subsurface explorations were brought to our office for further analysis and laboratory tests. Soil classifications were verified by a senior engineer and selected soil and rock samples were sent to our laboratory for testing. Six grain-size analyses, 11 Atterberg Limits determinations, 17 moisture-content measurements, 4 unconfined compression tests, 2 elastic moduli determinations, and 2 splitting tensile strength tests were performed. See Appendix C for laboratory test results.

SUBSURFACE CONDITIONS

The subsurface conditions generally consist of about 13 to 17 feet of uncontrolled fill and demolition debris, about 21 to 27 feet of silt with discontinuous sand and clay seams, and about 3 to 15 feet of decomposed rock. Schist bedrock was encountered between about 38 to 49 feet below grade. Stabilized groundwater levels were observed at depths of about 13.5 feet in 2016 and 20 feet in 2007. A more detailed description of each layer is provided below. Representative subsurface profiles are presented on Figures 7 and 8.

Fill [Class 7][4]

A layer of uncontrolled fill and demolition debris ranging in thickness between 13 and 17 feet was encountered in the borings, test pits and CPTs. The upper fill generally consisted of brick, concrete, and rebar debris from previous demolition at the site. The former basement floor slab was encountered about 12 feet below the existing site grade. Fill encountered below the basement slab generally consisted of coarse to fine sand with varying amounts of silt, gravel, and debris. No soil sampling was performed within the upper 10 feet of each borehole because of obstructions within the fill from the demolition operations. In addition to the floor slab, former foundation elements and other large obstructions should be anticipated within the fill. The piles and pile caps from the former structure are also present below the slab.

[4] Numbers in brackets that follow the material designation indicate classification of soil and rock materials in accordance with the NYC Building Code.

The fill is highly variable and is designated as Building Code Class 7, "uncontrolled fill."

Silt and Clay [Class 5b, 4c, and 6]

A layer of low-plasticity silt about 21 to 27 feet thick was encountered below the fill layer. This silt is regionally known as "Bull's Liver". The silt is generally loose to medium-dense with varying amounts of fine sand and clay, and is known for having unconventional engineering properties because of its silt-sized particles with little to no plasticity. In a saturated state, this silt has been observed to behave like a gel or even flow like liquid under shock or vibration. The foundation contractor should consider this soil behavior because it can introduce significant challenges during excavation and foundation construction.

Discontinuous layers of fine silty sand were encountered within the silt in borings B-2, B-3, B-4, and B-8 (discussed below). In addition, pockets with more clay content were encountered within the silt layer in borings B-4, B-5, and B-7.

Standard Penetration Test (SPT) N-values for the silt ranged between 1 and 29 blows per foot. CPT results indicated that this layer has the behavior of "Clayey silt to silty clay" or "Silty sand to sandy silt" with small pockets of "Clay to silty clay" and "Clean sand to silty sand". In general terms the SPT sampling and CPT results correlate well.

Laboratory testing of collected samples yielded natural moisture contents from 27 to 40 percent. The liquid limit ranged between 26 and 33 (average about 30); the plastic limit ranged from 20 to 25 (average about 23); and the plasticity index ranged from 4 to 11 (average about 7). In most tests the water content is near or above the liquid limit indicating that the silt could behave similarly to a viscous liquid when disturbed by construction.

The silt is generally classified as ML, CL, and ML-CL, in accordance with Unified Soil Classification System (USCS). The silt is designated as Building Code Class 5b and 6 material, "medium dense silts" and "loose silts," respectively. The pockets with higher clay content are designated as Building Code Class 4c and 6 material, "medium stiff clays" and "soft clays," respectively.

Clayey Sand [Class 6]

Four to 7 feet thick pockets of clayey fine to coarse sand were encountered within the silt in borings B-2, B-3, B-4, and B-8. Typical N-values for these sand pockets ranged between 1 and 8 bpf. These thin pockets of "Clean sand to silty sand" were also encountered at CPT-1 and CPT-2.

The clayey sand is generally classified as SC in accordance with USCS and is designated as Building Code Class 6 material, "loose granular soils."

Decomposed Rock [Class 1d]

Decomposed rock, ranging in thickness between about 3 and 15 feet, was encountered below the silt. The top of the decomposed rock was found about 34 to 41 feet below the existing ground surface (about el -24 to el -32). The decomposed rock generally consisted of micaceous silt with varying proportions of gravel and sand, and gravel-sized fragments of schist. SPT N-values within the decomposed rock generally met split-spoon refusal at 100 blows over 3 inches.

The decomposed rock layer is classified as Building Code Class 1d material, "soft rock."

Bedrock [Class 1a, 1b, and 1c]

The site is underlain by Manhattan schist bedrock, and the top of rock was encountered at depths of about 38 to 49 feet below the existing site grades. The corresponding top or rock elevations range between about el -28 and el -40. Rock-core recoveries range between 58 and 100 percent. Rock quality designation (RQD) values range between 37 and 100 percent. Both core recoveries and RQD generally improve with depth.

The bedrock at the site is classified as Building Code Class 1a, 1b, and 1c material, "hard sound rock," "medium hard rock," and "intermediate rock," respectively. Laboratory testing performed on select rock cores show intact compressive strength ranging from 8,400 to 16,800 psi, with an average compressive strength of about 13,500 psi. The rock Elastic Modulus test results range from 6,500 to 9,100 ksi, with an average of about 7,800 ksi. Splitting Tensile test results range from 1,300 to 2,300 psi, with an average of about 1,600 psi.

Groundwater

Groundwater levels were measured between about 18 and 20 feet below the existing grades during our 2007 exploration (about el -8 and el -10). Groundwater levels were measured at about 13.5 feet below the existing grade (about el -3.5) during our 2016 exploration. Groundwater can be expected to fluctuate with weather, seasonal conditions, construction activity, or groundwater pumping. The NYCT tunnels in Broad and William streets may be causing a local depression of the groundwater table. Nearby construction or pumping activity can also affect groundwater elevations on this site. We recommend the groundwater level be monitored throughout the design phase.

EVALUATION AND DISCUSSION

The subsurface and surrounding conditions present several geotechnical design challenges:

1. The uncontrolled fill and low-plasticity silt are unsuitable to support the proposed high-rise tower.
2. Existing structures (buildings, a subway tunnel, and a steam tunnel) are adjacent to the site on all four sides; the excavation and foundations construction methods must not overstress or damage the adjacent structures.
3. Driven piles are not recommended because of the proximity to adjacent buildings and NYCT tunnel.

The building will include three cellar levels with the top of the lowest cellar slab at about 32 feet below sidewalk grade. Therefore, we recommend a mat foundation bearing directly on the underlying bedrock combined with permanent tie-down anchors to resist wind and hydrostatic uplift. Where the top of competent rock (Building Code Class 1b or better) is below the proposed bottom of the mat, the mat should rest on clean, concrete fill with a minimum 28-day strength of 4,000 psi, casted atop the rock. The excavation will require installing a permanent rigid support of excavation (SOE) system to provide groundwater cut-off. The rigid SOE system can be appropriately sized and reinforced to carry compression and tension perimeter building loads. Geotechnical parameters for the mat foundation, tie-down anchors, and support of excavation design are provided in subsequent sections.

Because the site is long-narrow shaped and the excavation will extend about 50 feet below existing grades, equipment access and material storage through the site during foundation construction could be challenging. Traditional bottom-up construction would require rather dense temporary bracing, which could restrict access and congest traffic. Therefore, top-down construction has been considered and discussed with Madison 45 Broad Development and the design team as a viable alternative. During the top-down (or up-down) construction the perimeter wall is installed first (as a drilled secant wall) and the cellar floors are constructed as the excavation progresses. When in place, the ground floor slab will be used as a lay-down area and allow equipment access across the site.

Because of the site's proximity to the adjacent subway tunnel, NYCT review and approval will be required to obtain an excavation and foundation permit from the NYC Department of Buildings. We expect that the interaction with NYCT will be extensive and that permitting process can take four to six months or more, which must be accounted for in the project schedule.

FOUNDATION DESIGN RECOMMENDATIONS

The following sections present our liquefaction evaluation, a discussion of the seismic design parameters, and our recommendations related to the design and construction of the foundation system for the proposed development. All discussions reference the 2014 Building Code.

Seismic Design Parameters

The proposed structure will be founded directly on rock; therefore, the Site Class is B. The Building Code seismic design parameters are summarized in Table 1.

Table 1 – Seismic Design Parameters

Description	Parameter	Recommended Value	Building Code Reference
Risk Category (Assumed; to be confirmed by structural engineer)		**II**	Section 1604.5
Site Class	**Rock**	**B**	Section 1613.5.2
Mapped Spectral Acceleration for short periods:	$\mathbf{S_s}$	**0.281 g**	Section 1613.5.1
Mapped Spectral Acceleration for 1-sec period:	$\mathbf{S_1}$	**0.073 g**	
Site Coefficient:	$\mathbf{F_a}$	**1.00**	Section 1613.5.3
Site Coefficient:	$\mathbf{F_v}$	**1.00**	
5%damped design spectral response acceleration at short periods:	$\mathbf{S_{DS}}$	**0.187 g**	Section 1613.5.4
5% damped design spectral response acceleration at 1-sec period:	$\mathbf{S_{D1}}$	**0.049 g**	Section 1613.5.4
Maximum considered Earthquake geometric mean (MCEG) peak ground acceleration	$\mathbf{PGA_M}$	**0.17g**	Section 1813.2.1
Seismic Design Category (Based on assumed Risk Category)		**B**	Tables 1613.5.6 (1) & 1613.5.6 (2)

Based on the design spectral accelerations in Table 1 and the anticipated structural occupancy/risk category of the structure (identified as Structural Occupancy/Risk Category II) and in accordance with the Building Code, we have estimated that the design will be subject to the requirements of Seismic Design Category B. The Structural Occupancy/Risk Category must be confirmed by the architect and structural engineer.

Liquefaction Evaluation

The Building Code requires an evaluation of the liquefaction potential of noncohesive soil and cohesive soil with plasticity index 20 or less below the groundwater table and up to 50 feet below the ground surface. In accordance with the Building Code screening process for liquefaction, the SPT N_{60} values from the borings are plotted versus depth on the Liquefaction Assessment Diagram, presented as Figure 9. This plot shows a significant amount of soil in the "Liquefaction Probable" zone.

The proposed construction involves excavation and removal of all soil to support the structure directly on rock. Therefore, the risk of liquefaction is mitigated and a site-specific study is not required. If the development plan changes and excavation and removal of all liquefiable soil is no longer considered, the design team should address this change and re-evaluate the site classification and soil liquefaction potential.

Foundation System

We recommend the building be supported by a mat foundation bearing on bedrock. The recommended allowable rock bearing capacity is 40 tsf (Building Class 1b bock). The top of rock was encountered at depths of about 38 to 49 feet below the existing site grades and generally dips north to south. The corresponding top or rock elevations range from about el -28 to el -40. The bottom of a 9 to 12-foot-thick mat foundation as shown on preliminary design drawings prepared by WSP, will be at about el -29.5 to el -33. Therefore, the bottom of the proposed mat will not bear directly on rock at the majority of the site.

Wherever Building Class 1b rock is not encountered at the bottom of mat foundation elevation, all soil and decomposed rock should be excavated to the top of Building Class 1b rock and backfilled with 4,000 psi concrete fill. All rock bearing surfaces should have a maximum 10-percent slope as required by the Building Code. Otherwise, horizontal benches 10 feet long and wide, with vertical faces, should be created to satisfy the maximum slope requirement. Because the difference in the bottom of the mat elevation and the estimated top of rock can be as much as 8 feet or more, WSP should evaluate whether the concrete fill should be reinforced.

For initial design development, we recommend an average modulus of subgrade reaction of 1,500 psi/inch for Class 1b rock. The mat foundation design should be compatible with half and twice of this value. The subgrade modulus must be iterated until the geotechnical model and the structural model (which approximates the subgrade response via Winkler springs) converge (i.e., the spring value must be iterated until the settlement predicted by the geotechnical model matches that predicted by the structural model).

Foundation Settlement

The settlement of foundations is a function of the structural loads and are dependent on the layout of columns and shear walls and stiffness of the foundation. For the proposed building loads, we anticipate that the total and differential foundation settlements below the thick foundation mat will be ¾ inch or less.

Lateral Resistance

For a mat bearing directly on rock, lateral loads can be resisted by friction on the bottom of the mat. We recommend an ultimate frictional coefficient of 0.70 for mass concrete poured on clean sound rock. Where concrete fill underlies the mat foundation, WSP should confirm that the concrete fill-to-foundation concrete-to-rock interfaces can resist the proposed lateral loading. If additional resistance is needed, shear keys may be embedded into rock or concrete. We should be contacted to evaluate passive pressure if needed.

Rigid Perimeter Excavation Support

Below grade construction will require excavating to the top or rock or about 38 to 49 feet below the existing grades (about el -28 to el -40). To provide excavation support and temporary groundwater cut-off we recommend installing a rigid, continuous secant pile wall system on the south, east, and west foundation perimeter. The secant pile walls will abut the foundation wall of 41 Broad Street, which extends into the bedrock according to historic construction plans.

The secant pile wall installation begins with the construction of a guide wall at the ground surface. The guide wall ensures that the position, alignment and required overlap of subsequent secant piles are maintained. After the guide wall is formed, the primary piles (every other pile location) are installed by advancing steel casing to top of rock and continuing the rock socket to the design depth. The casing is then withdrawn as the pile is grouted. Secondary piles are then drilled in between such that they overlap with the primary piles. Reinforcing steel is added to the secondary piles based on the structural loading and excavation support requirements. These systems are relatively stiff soil retention systems, necessary to limit wall deflection and movement of adjacent structures, and assist in groundwater control. To accommodate access of the drilling equipment close to the property line, the edge of casing is positioned at least 12 inches from the face of adjacent buildings. The contractor should note that obstructions such as remnant slabs and foundations including piles and pile caps exist within and below the fill and should be removed prior to or bypassed during the installation of the perimeter excavation support.

In addition to serving as temporary excavation support and water cut-off, the secant pile wall can serve as the permanent foundation wall and carry part of the foundation loads according to the foundation design. The structural loads on the secant pile wall were not available at the time of this report. If the secant piles are used to rest tension capacity, they must also be evaluated for global stability. In addition, the top level of the secant pile wall must be coordinated with the structural engineer to account for the continuous ring beam.

For top-down construction, lateral bracing is provided by the ground and cellar floors slabs, which are constructed as the excavation progresses. The Owner and design team are considering creating additional headroom during construction by constructing one of the cellar slabs after the foundation construction is complete; therefore additional temporary lateral support will be necessary at the bypassed slab elevation. Lateral support could consist of tiebacks on the east and west (below the NYCT tunnel influence line) and rakers or buttresses (additional secant piles perpendicular to the perimeter walls).

The NYC Department of buildings (DOB) requires that project-specific excavation support drawings be prepared as part of the new-building submission. The project-specific plans must be fully developed, in conjunction with developed structural building plans, to be reviewed and approved by DOB so that a construction permit for the new building (or foundations) can be issued. Excavation support plans will also need to be reviewed by the NYCT for potential impacts on the adjacent subway structures.

Permanent Rock Anchors

Permanent post-tensioned tie-downs anchored into bedrock will be required to resist uplift forces resulting from wind, buoyant, and seismic loads. We recommend using double corrosion-protected Grade 150 threaded bars meeting ASTM A-722 requirements or Grade 270 strand tendons meeting ASTM A-416 requirements for reinforcement steel. Double corrosion protection should consist of PVC sheathing and grout encapsulation around the anchor bar or tendons. The anchor bar diameter should not exceed 3 inches; if higher capacity is required, strand anchors should be used. The anchor bond length should be proportioned using an allowable peripheral shear resistance in uplift of 100 psi. The free stress (un-bonded) length should be a minimum of 10 feet long, but additional length may be required for group effects and global uplift stability.

The free-stressing length of reinforcement should be proportioned such that the dead weight and tensile strength of the engaged rock mass is greater than the individual anchor load or the sum of the group anchor loads. Group and global stability analysis must be performed by Langan during design development. The free length of adjacent anchors can be alternated in a staggered pattern, if required by the group analysis. Table No. 2 and Table No. 3 present the estimated design capacity with corresponding bond lengths for both threaded bars and strand tendon options.

Table 2 – Threaded Bar Rock Anchor Capacities

Design Uplift Load (kips)	Threaded Bar Diameter (inch)	Threaded Bar Grade	Min. Drill Hole Diameter (inch)	Min. Free Length[1] (ft)	Min. Bond Length[2] (ft)
110	1-1/4	150	5	10	10
615	3	150	7	10	25

[1] The free stressing length will be defined by the global stability and group effect analysis

[2] This table represents minimum lengths for single anchors. Group effects must be analyzed during DD phase and may require longer anchors.

Table 3 – Strand Tendon Rock Anchor Capacities

Design Uplift Load (kips)	No. of Strand Tendons	Strand Tendon Cross Sectional Area (sq-inch)	Strand Tendon Grade	Min. Drill Hole Diameter (inch)	Min. Free Length[1] (ft)	Min. Bond Length[2] (ft)
110	4	0.868	270	5	10	10
615	18	3.906	270	7	10	25

[1] The free stressing length will be defined by the global stability and group effect analysis

[2] This table represents minimum lengths for single anchors. Group effects must be analyzed during DD phase and may require longer anchors.

A minimum of 10 anchors or two percent of the tie-down anchors (whichever is greater) should be performance-tested (creep) to 133% of their design loads in accordance with Post-Tensioning Institute (PTI) standards. The remaining anchors should be proof tested to 133% their design load per PTI standards. Lift-off testing should be performed to all anchors. Successfully tested anchors should be locked off at a load exceeding the sum of the design load, seating loss, and long-term losses.

Pressure Slabs

The lowest floor level will extend below groundwater and should be designed as a pressure slab. We recommend that the pressure slabs be designed assuming hydrostatic uplift corresponding to the design groundwater el 12 (BFE + 1ft). Where possible, pressure slabs should be keyed into the foundation walls and should be cast with integral water stops (PVC "dumbbells" and post construction grout tubes). Pressure slabs should be waterproofed according to the recommendations presented herein.

Permanent Groundwater Control

This section describes our recommendations for permanent groundwater control at the site.

<u>Design Groundwater Level</u>

During the 2007 subsurface exploration, the static groundwater was observed at about 18 to 20 feet below existing grade (about el -8 to el -10). During the 2016 subsurface exploration, the static groundwater was observed at about 13.5 feet below existing grade (about el -3.5). This fluctuation could be related to seasonal variations, nearby construction or pumping activities.

Because the site is partially located within the Flood Zone AE, the foundation walls, ground level, and below-ground slabs should be flood-proofed and designed to resist hydrostatic pressure for groundwater rising to el 12. This Design Flood Elevation (DFE) corresponds to the base flood elevation of el 11 (BFE) plus 1-foot freeboard as per Chapter G5 Table 6.1 of the Building Code.

Foundation Waterproofing

To limit water seepage we recommend that the foundation raft and the perimeter secant pile wall be fully waterproofed to at least the design flood elevation (DFE). We recommend installing a membrane-type, positive-side waterproofing (installation on outside of structure). For horizontal applications, the waterproofing membrane should be installed on a two-inch-minimum concrete working surface (mud-slab), which will create a uniform substrate. For one-face wall vertical applications (conventional foundation wall and pit walls), plywood or other acceptable flat surfaces should be used to secure the waterproofing membrane. The membrane should be protected against damage during rebar placement, concrete placement, and general construction traffic.

Groundwater can be expected to seep through the joints in the secant pile wall. One scheme to accommodate the water leakage is to create a cavity wall using masonry block. The water is collected behind the partition walls via a series of scupper drains and directed to the lowest cellar level. The water is then ejected and discharged into the city sewer system.

An alternate scheme is to waterproof the inside face of the secant pile wall. This can be accomplished by installing a waterproofing membrane on the secant pile wall and casting an interior liner wall. Prior to the membrane application the secant wall surface should be purged and leveled. A concrete facing wall would then be cast against the secant piles to provide the necessary bond to the waterproofing and to hold the membrane in place. The minimum wall thickness is 4 inches (or as otherwise recommended by the waterproofing manufacturer) as needed for structural integrity. Special waterproofing details will need to be developed for locations of the secant pile wall – intermediate slabs interface and at the bracing locations. For the horizontal and vertical applications we recommend using Preprufe products by W.R. Grace or other equivalent. As a supplementary measure, waterproofing concrete admixtures such as

Hycrete's products can be added to the secant pile grout mix (for water control and corrosion protection) and the liner wall grout mix.

We recommend that warranties are obtained from the manufacturers and installers to cover materials and workmanship. Material and system compatibility needs to be confirmed if products from multiple manufacturers are selected. Only certified installers should be used to perform the work. Detailed oversight should be performed and a representative of the manufacturer should perform a final inspection of the waterproofing prior to concrete pours.

Depending on the use of the cellar space, installing a secondary control system may be warranted. For this purpose the following secondary measures can also be considered.

1. Install a second mud slab on top of the installed horizontal waterproofing membrane. This mud slab would protect the installed waterproofing from construction traffic during placement for the steel reinforcement.
2. Use a waterproofing additive in the foundation concrete. Addatives typically react with water to block pours and small cracks.
3. Install a connection layer and concrete slab over the mat slab. The draining layer can be gravel with collection pipes or a heavy duty prefabricated drainage board. This system will collect groundwater (that could intrude through damaged waterproofing) and guide it to a drain system.

Permanent Below-Grade Walls

Permanent below-grade walls including perimeter foundation and elevator pit walls should be designed to resist lateral loadings from static earth pressure, water pressure, and vertical surcharge. Backfill should not be placed against below-grade walls until the concrete has reached its 28-day compressive design strength and after adequate lateral bracing has been provided to prevent rotation of the wall, or as otherwise directed by the structural engineer. We recommend the following design parameters in Table 3 and subsequent paragraphs.

Table 3 – Horizontal Earth Pressure Parameters

Layer	Unit Weight Above WT (pcf)	Effective Unit Weight Below WT (pcf)	At Rest Earth Pressure Coefficient K_o
Fill [Class 7]	120	63	.50
Silt and Clay [Class 5b, 4c, 6]	110	57	.60
Decomposed Rock [Class 1d]	135	72	.35

- Hydrostatic pressures should be added as a triangular pressure distribution having an equivalent fluid weight of 62.4 pounds per square foot per foot of depth below the design groundwater level.

Surcharge loads should be considered in the design of below-grade walls. The walls should be designed for an additional uniform pressure distribution equal to 0.50 times the anticipated surcharge load. We recommend the following minimum surcharges be considered:

- Surficial traffic loads should be considered for the west perimeter walls (along Broad Street). We recommend a surcharge load of 300 psf for the street side walls to account for large trucks and emergency vehicles.

- Surficial loads should be considered for the east perimeter walls (along hammerhead). We recommend a surcharge of 100 psf for these walls.

- Construction surcharge loads should be considered along the west and east perimeter walls if they exceed the recommended values above.

- Walls must also be designed for surcharge loads from adjacent structures where the walls extend below the area of influence of the adjacent foundations. We understand 41 Broad Street is founded on rock, and 55 Broad Street is founded on piles such that only the surcharge from the neighboring slab needs to be considered.

GEOTECHNICAL CONSTRUCTION RECOMMENDATIONS

Our recommendations for excavation, subgrade preparation, temporary groundwater control, and pre-construction activities and construction monitoring are provided below.

Excavation

Site excavation within the fill and underlying silt and clay can be performed using conventional earth-moving equipment (e.g., backhoes, excavators, dozers, etc.). All excavations should be conducted in accordance with all OSHA requirements including, but not limited to, temporary shoring, trench boxes, and proper benching. Obstructions such as old foundations, slabs, pile caps and piles, and demolition debris should be expected and may require heavy demolition equipment to remove.

Note that obstructions such as remnant slabs and foundations including piles and pile caps exist within and below the fill. Specifically, the remnant cellar slab was encountered about 12 feet below existing grade. The contractor should be prepared to demolish and excavate through the existing slab and all obstructions, and remove the existing pile caps, piles, and slabs.

An alternative method to perform the foundation construction would be the "top-down" construction method. In general terms this option involves construction of the ground and cellar floor levels as the excavation progresses. Top-down construction begins with installation of exterior walls and load bearing elements to support subsequent floor slabs. The ground floor is then cast. The excavation is performed below the cast slab to the next slab level, with excavation spoils removed through shafts and access openings in the slabs. The process is repeated to the final mat level.

Subgrade Preparation for Foundation Mat on Rock

The foundation mat bearing surface should be level and clear of debris, standing or frozen water, and other deleterious materials. All rock bearing surfaces should have a maximum 10-percent slope as required by the Building Code. Otherwise, horizontal benches at least 10 feet long and wide with vertical faces should be created to satisfy the maximum slope requirement. Compressed air should be used to clean all rock surfaces. Rock, joints, foliation, and local zones of weathered or fractured rock may require locally deepening the excavations further into rock. The Building Code requires that all rock subgrade be inspected by Professional Engineer to verify the quality of the bedrock before installing reinforcing steel and concreting. The rock subgrade must be inspected to verify bearing capacity and that foundations have been adequately cleaned and prepared.

Temporary Groundwater Control

Groundwater was encountered in the 2016 investigation at 13.5 feet below grade. The proposed deep excavation will require dewatering. The proposed SOE system using secant piles and tangent piles will provide groundwater cutoff such that the interior of the excavation can be locally dewatered. Collection of rainwater runoff will also be needed during the excavation and subgrade preparation work. Water runoff should be controlled with the use of gravel-lined collection trenches or pits and submersible pumps. Care should be taken to ensure that drainage is provided during all phases of excavation work so as to limit the disturbance of the subgrade materials and provide a workable surface. Any necessary environmental pre-treatment of groundwater should be coordinated with the applicable environmental regulations for the site. A DEP discharge permit will need to be furnished to discharge groundwater into the DEP combined sewer. It is the contractor's responsibility to estimate the daily groundwater discharge volume and to furnish all paperwork for the permit application.

Preconstruction Conditions Survey and Monitoring During Construction

A preconstruction-conditions survey report should be prepared for the adjacent buildings and the existing NYCT subway tunnel adjacent to the site. We recommend that a monitoring program be developed to observe the response of the existing buildings and subway tunnel adjacent to the site during foundation construction activities (i.e., excavation, SOE installation, bracing, etc.). According to our past discussions with NYCT, this program could consist of monitoring horizontal and vertical movements by optical surveying and inclinometers, and vibration monitoring using seismographs. The NYCT typically requires that the vibration monitoring data is collected manually, or at least has on site observation of an automated system.

Construction Documents and Quality Control

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

Geotechnical assessment and design is an ongoing process as additional information becomes available, including during construction. A geotechnical engineer familiar with the site subsurface conditions and design intent should perform the quality assurance observations and testing of geotechnical-related work during construction. According to the Building Code, construction of foundations (i.e., earthwork, subgrade preparation, etc.) and support of excavation require special inspection by a Professional Engineer licensed in the state of New York.

Owner and Contractor Obligations

Construction activities that alter the existing ground conditions such as excavation, fill placement, foundation construction, ground improvement, pile driving/drilling, dewatering, etc. can induce stresses, vibrations and movements on nearby structures. The Owner and all Contractors must ensure that these impacts will not adversely affect the performance of the structures and take adequate measures to protect the existing structures during construction.

Unless otherwise agreed to by Langan in writing, by using this report, the owner agrees to the following:

1) That Langan will not be held responsible for damage to adjacent structures caused by the actions of contractors involved in the project;

2) To have Langan added to the Foundation Contractor's General Liability insurance as an additional insured;

3) To require the Foundation Contractor to defend, indemnify and hold harmless the Owner and Langan against all claims related to 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, as well as information provided by Madison 45 Broad Development LLC, February 2016 concept design drawings and sketches provided by CetraRuddy, and subsequent discussions with the project team. Recommendations provided are dependent upon one another and no recommendation should be followed independent of the others.

Any proposed changes in structures or their locations should be brought to Langan's attention as soon as possible so that we can determine whether such changes affect our recommendations. Information on subsurface strata and groundwater levels shown on the logs represent 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 Langan's attention for evaluation, as they may affect our recommendations.

This report has been prepared for 45 Broad Street, New York, New York, to assist the owner, architect, and structural engineer in the design process and is only applicable to the design of the specific project identified. The information in this report cannot be utilized or depended on by engineers or contractors who are involved in evaluations or designs of facilities (including underpinning, grouting, stabilization, etc.) on adjacent properties, which are beyond the limits of that which is the specific subject of this report.

Environmental issues (such as potentially contaminated soil and groundwater) are outside the scope of this study and should be addressed in a separate study.

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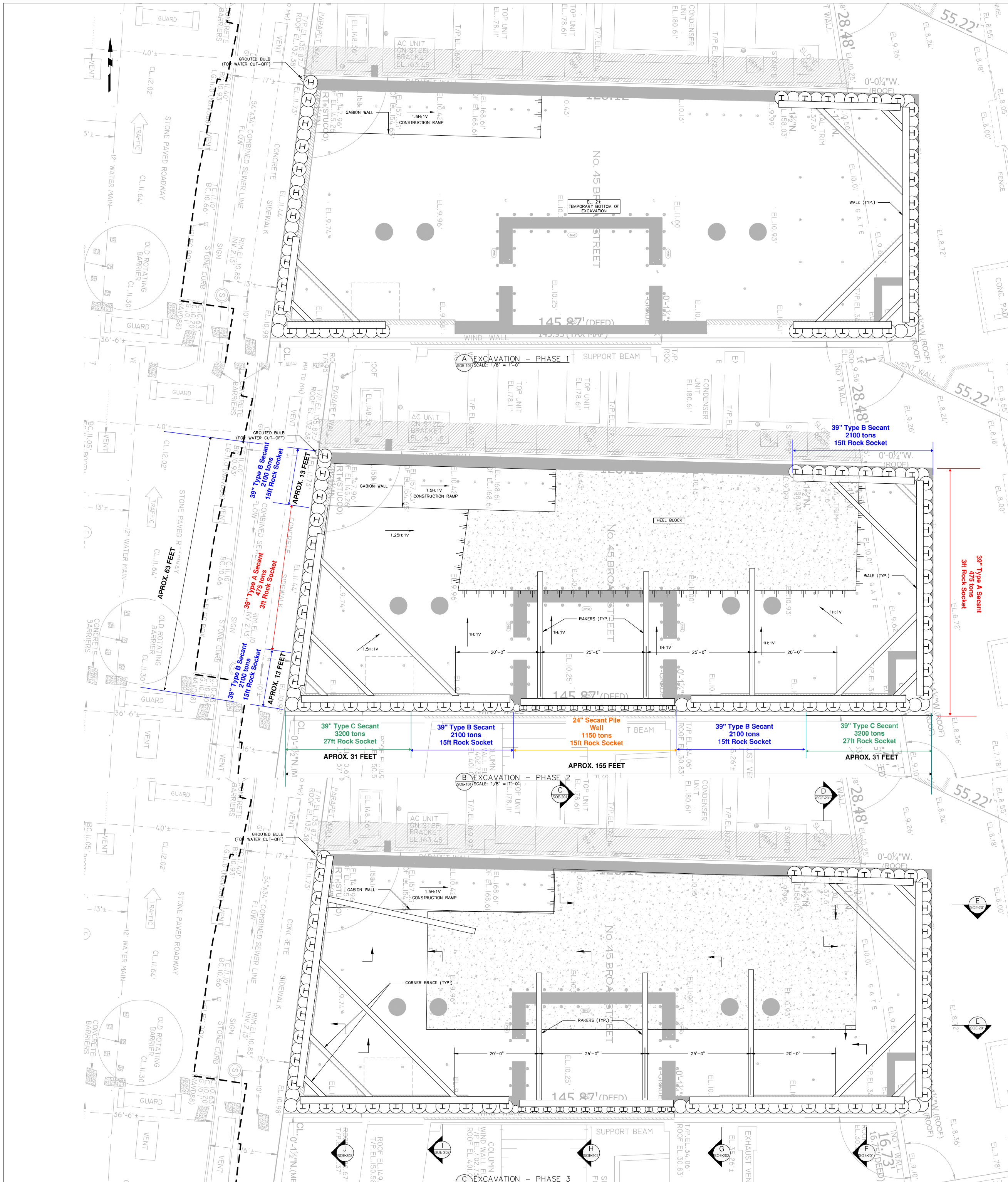
Appendix E SOE Plan Drawing



PHASE 1 – EXCAVATION SEQUENCE:

1. DRILL ALL SECANT PILES SHOW ON PHASE 1 DRAWING, FROM GROUND SURFACE.
2. CONSTRUCT A 10-FEET WIDE EGRESS BERM AT 1.5H:1V SLOPE FROM SIDEWALK, AS SHOWN ON THE PHASE 1 DRAWING. THE BERM SHALL BE SUPPORTED ON THE SOUTH SIDE BY GABION WALLS.
3. EXCAVATE THE ENTIRE SITE TO EL. 2±.
4. INSTALL THE FIRST TIER CORNER BRACES AT ALL THREE CORNERS AS SHOWN, AT EL. 3±.

PHASE 2 – EXCAVATION SEQUENCE:

1. CONSTRUCT BERMS TO EL. –23± (FINAL EXCAVATION LEVEL) AT SLOPES SHOWN ON THE DRAWING. IMPROVE GROUND BY MIXING WITH GROUT AS NEEDED TO STABILIZE SLOPES.
2. INSTALL MIDDLE AND BOTTOM TIERS OF WALERS AND CORNER-BRACES (EL. –7± AND EL. –17±) AT THE NORTH-EAST CORNER.
3. CONSTRUCT HEEL BLOCK (SUB-CELLAR 2 SLAB), THE EXTENTS OF WHICH ARE SHOWN ON THE PHASE 2 DRAWING. CONNECT HEEL BLOCK STRUCTURALLY TO NORTH WALL PILE CAPS.
4. INSTALL THE TOP TIER RAKERS AND WALERS ALONG WEST WALL (EL. +3±) AND SOUTH WALL (EL. +0.5±).

PHASE 3 – EXCAVATION SEQUENCE:

1. EXCAVATE REMAINDER OF SITE TO FINAL GRADE (BESIDES EGRESS BERM) AND EXPAND THE HEEL BLOCK, AS SHOWN ON THE PHASE 3 DRAWING. CONNECT THE HEEL BLOCK STRUCTURALLY TO EXPOSED PILE CAPS.
2. AS SITE IS EXCAVATED:
   2.1. INSTALL MIDDLE AND BOTTOM TIERS OF WALER AND RAKER AT THE WEST WALL (EL. –7± AND EL. –17±).
   2.2. INSTALL THE BOTTOM TIER OF WALERS AND RAKERS AT THE SOUTH WALL (EL. –9.5±).
   2.3. INSTALL THE MIDDLE AND BOTTOM TIER OF CROSS-BRACES AT THE SOUTHEAST AND SOUTHWEST CORNERS (EL. –7± AND EL. –17±).
3. BRACE THE WEST WALL AT THE EGRESS BERM LATERALLY USING RAKERS, AS THE BERM IS REMOVED.

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Appendix F Caisson and Secant Pile Wall Capacity Summary Tables from WSP

Caisson Design Summary

Project: 45 Broad Street, NY, NY
Project No.: 170394201
Date: 10/21/2016

Compression Capacity (tons)	Tension Capacity (tons)	Casing Diameter (inches)	Caisson Reinforcing (Grade 75)	Concrete/ Grout Strength (ksi)	Reinforcement Steel Strength Fy (ksi)	Spring Constant K-down (kips/inch)	Spring Constant K-up (kips/inch)
1200	600	18 x 0.5 wall	8 - #24 Thread Bar	**10**	75	4200	3700
3000	1500	36 x 0.5 wall	10 - #28 Thread Bar	**10**	75	11200	7500

Caisson Design Summary

Project: 45 Broad Street, NY, NY
Project No.: 170394201
Date: 11/2/2016

Secant Size & Type	Compression Capacity (tons)	Tension Capacity (tons)	Casing Diameter (inches)	Caisson Reinforcing (Grade 75)	Concrete/ Grout Strength (ksi)	Reinforcement Steel Strength Fy (ksi)	Spring Constant K-down (kips/inch)	Spring Constant K-up (kips/inch)
24" Secant	**1150**	**400**	18 x 0.5 Wall	9 - #24 bars	10	75	3400	2500
39" Secant - Type A	**500**	**150**	N/A	W24x176 (Grade 50)	10	50	3100	1500
39" Secant - Type B	**2100**	**675**	30 x 0.5 Wall	10 - #20 bars	10	75	6100	3200
39" Secant - Type C	**3175**	**1675**	30 x 0.5 Wall	14 - #28 bars	10	75	7300	5500
39" Secant - Type D	**2250**	**1400**	30 x 0.5 Wall	14 - #28 bars	10	75	7000	5800