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
45 Broad Street
Structural Peer Review Report
Foundations

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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)
- $S_{D_s}$: 0.187g (per geotech report), 0.295g (shown on dwg FO-001.00)
- $S_{D_1}$: 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.
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.
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 result, 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

<table>
<thead>
<tr>
<th>Pressure Slab</th>
<th>Reinforcement</th>
<th>DCR Flexure</th>
<th>DCR Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{\text{bottom}} @ 1 )</td>
<td>#11 @ 6 + 3 - #11 @ 12</td>
<td>0.58</td>
<td>1.15</td>
</tr>
<tr>
<td>( M_{\text{bottom}} @ 2 )</td>
<td>#11 @ 6 + 14 - #11 @ 6</td>
<td>0.33</td>
<td>0.99</td>
</tr>
<tr>
<td>( M_{\text{top}} @ 3 )</td>
<td>#11 @ 6</td>
<td>0.60</td>
<td>0.26</td>
</tr>
<tr>
<td>( M_{\text{bottom}} @ 4 )</td>
<td>#11 @ 6</td>
<td>0.61</td>
<td>0.72</td>
</tr>
</tbody>
</table>

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

To address the over stress 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

<table>
<thead>
<tr>
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<th>DCR</th>
<th>Tension (kip)</th>
<th>DCR</th>
</tr>
</thead>
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<tr>
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<td>0.97</td>
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</tr>
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<td>99.34</td>
<td>-</td>
</tr>
<tr>
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<td>1.27</td>
<td>47.76</td>
<td>-</td>
</tr>
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<td>0.91</td>
<td>26.01</td>
<td>-</td>
</tr>
<tr>
<td>8801</td>
<td>927.64</td>
<td>0.93</td>
<td>115.21</td>
<td>-</td>
</tr>
<tr>
<td>8805</td>
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<td>0.95</td>
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<td>97.45</td>
<td>-</td>
</tr>
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### Table 3 – Summary of Secant Pile Type “B” Axial Capacity Check

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<td>164.42</td>
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<td>9129</td>
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### Table 4 – Summary of Secant Pile Type “C” Axial Capacity Check

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

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</table>

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

<table>
<thead>
<tr>
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<th>Tension (kip)</th>
<th>DCR</th>
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</thead>
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<td>882.74</td>
<td>0.29</td>
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<td>8512</td>
<td>5078.51</td>
<td>0.85</td>
<td>681.88</td>
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<td>0.86</td>
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<td>0.88</td>
<td>136.80</td>
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<td>11222</td>
<td>5204.29</td>
<td>0.87</td>
<td>26.83</td>
<td>0.01</td>
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</table>

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

<table>
<thead>
<tr>
<th>Load</th>
<th>Base Shear (kip)</th>
<th>Total Capacity (kip)</th>
<th>DCR</th>
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<tbody>
<tr>
<td>EQx</td>
<td>2100</td>
<td>5650</td>
<td>0.37</td>
</tr>
<tr>
<td>EQy</td>
<td>2100</td>
<td>5650</td>
<td>0.37</td>
</tr>
<tr>
<td>Wind x</td>
<td>2800</td>
<td>5650</td>
<td>0.5</td>
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<tr>
<td>Wind y</td>
<td>5477</td>
<td>5650</td>
<td>0.97</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Caisson Cap</th>
<th>Reinforcement</th>
<th>DCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>C6 Long Way</td>
<td>29 - #11</td>
<td>0.45</td>
</tr>
<tr>
<td>C6 Short Way</td>
<td>22 - #11</td>
<td>0.45</td>
</tr>
</tbody>
</table>

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

\[
\frac{3\sqrt{f_c}}{f_y}b_u d \\
\frac{200}{f_y}b_u d
\]

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

<table>
<thead>
<tr>
<th>Strap Beam</th>
<th>Reinforcement</th>
<th>DCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>SB1</td>
<td>Top: 126 - #11 (7 layers)</td>
<td>0.67</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Strap Beam</th>
<th>Legs</th>
<th>Size</th>
<th>Spacing</th>
<th>DCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>SB1</td>
<td>6</td>
<td>#5</td>
<td>6”</td>
<td>0.27</td>
</tr>
</tbody>
</table>

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.
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>
<thead>
<tr>
<th>Wall</th>
<th>Reinforcement</th>
<th>DCR</th>
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</thead>
<tbody>
<tr>
<td>North Wall</td>
<td>#9 @ 10</td>
<td>0.23</td>
</tr>
<tr>
<td>East Wall</td>
<td>#7 @ 12</td>
<td>0.78</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Wall</th>
<th>Reinforcement</th>
<th>DCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>North Wall</td>
<td>-</td>
<td>0.36</td>
</tr>
<tr>
<td>East Wall</td>
<td>-</td>
<td>0.45</td>
</tr>
</tbody>
</table>

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,
LESLEI E. ROBERTSON ASSOCIATES, R.L.L.P.

_____________________________
BENJAMIN M. CORNELIUS
Partner-In-Charge
Appendix A   Drawing List
### STRUCTURAL SHEET LIST

<table>
<thead>
<tr>
<th>SHEET NUMBER</th>
<th>SHEET NUMBER</th>
<th>DATE</th>
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<tbody>
<tr>
<td>FO-001</td>
<td>GENERAL NOTES &amp; LEGEND</td>
<td>2016.11.07</td>
</tr>
<tr>
<td>FO-100</td>
<td>FOUNDATION PLAN</td>
<td>2016.11.07</td>
</tr>
<tr>
<td>FO-110</td>
<td>SUB-CELLAR 1 FRAMING PLAN</td>
<td>2016.11.07</td>
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<tr>
<td>FO-120</td>
<td>CELLAR FRAMING PLAN</td>
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<tr>
<td>FO-150</td>
<td>PILE CAP REINFORCEMENT PLAN</td>
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### ARCHITECTURAL SHEET LIST

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Appendix B  Structural Design Criteria from WSP
Appendix C  Structural Design Criteria Narrative from WSP
45 Broad Street
Madison Equities, LLC
Pizzarotti IBC, LLC
CetraRuddy Architecture
December 30, 2016
<table>
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<th>Revision 1</th>
<th>Revision 2</th>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td></td>
<td></td>
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# Contents

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1.2 Design Superimposed Dead Loads  
1.3 Design LIVE Loads  
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2.2 Horizontal Sway  
2.3 Vibration Limits  
2.4 Durability of the Structure  
2.5 Fire Resistance Periods  

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3.1 Statutory Codes of Practice  

4 **Design References**  
4.1 Computer Programs  
4.2 UNITS
STRUCTURAL DESIGN CRITERIA

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

\[ S_s = 0.281 \, g \]
\[ S_I = 0.073 \, g \]
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:

\[
\text{span} / 480 \quad \text{beam supported at each end} \\
\text{span} / 240 \quad \text{cantilever beam}
\]

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

<table>
<thead>
<tr>
<th>Element</th>
<th>f'c (psi)</th>
<th>E (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shearwalls</td>
<td>14,000psi to 8,000psi</td>
<td>7,080ksi to 5,100ksi</td>
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<tr>
<td>Columns</td>
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<td>7,080ksi to 5,100ksi</td>
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<td>14,000psi to 8,000psi</td>
<td>7,080ksi to 5,100ksi</td>
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<tr>
<td>Floor Slabs</td>
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<td>4,415ksi to 5,100ksi</td>
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<tr>
<td>Bearing Foundations</td>
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<tr>
<td>Foundation Walls</td>
<td>14,000psi to 10,000psi</td>
<td>7,080ksi to 5,700ksi</td>
</tr>
</tbody>
</table>
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-13th ed.: LRFD Manual of Steel Construction
4 Design References

Other publications used include:
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/in²
Moment: Kip-ft
Velocity: Miles/hour
Acceleration: ft/s² 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
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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.

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\(^1\) 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>
<thead>
<tr>
<th>Description</th>
<th>Parameter</th>
<th>Recommended Value</th>
<th>Building Code Reference</th>
</tr>
</thead>
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<tr>
<td>Risk Category (Assumed; to be confirmed by structural engineer)</td>
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<td>II</td>
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<tr>
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<td>Section 1613.5.3</td>
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<tr>
<td>Site Coefficient:</td>
<td>(F_v)</td>
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</tr>
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<td>Section 1813.2.1</td>
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<tr>
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<td>B</td>
<td>Tables 1613.5.6 (1) &amp; 1613.5.6 (2)</td>
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</table>
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 rock). 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

<table>
<thead>
<tr>
<th>Design Uplift Load (kips)</th>
<th>Threaded Bar Diameter (inch)</th>
<th>Threaded Bar Grade</th>
<th>Min. Drill Hole Diameter (inch)</th>
<th>Min. Free Length(^1) (ft)</th>
<th>Min. Bond Length(^2) (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>110</td>
<td>1-1/4</td>
<td>150</td>
<td>5</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>615</td>
<td>3</td>
<td>150</td>
<td>7</td>
<td>10</td>
<td>25</td>
</tr>
</tbody>
</table>

\(^1\) 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

<table>
<thead>
<tr>
<th>Design Uplift Load (kips)</th>
<th>No. of Strand Tendons</th>
<th>Strand Tendon Cross Sectional Area (sq-inch)</th>
<th>Strand Tendon Grade</th>
<th>Min. Drill Hole Diameter (inch)</th>
<th>Min. Free Length¹ (ft)</th>
<th>Min. Bond Length² (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>110</td>
<td>4</td>
<td>0.868</td>
<td>270</td>
<td>5</td>
<td>10</td>
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<tr>
<td>615</td>
<td>18</td>
<td>3.906</td>
<td>270</td>
<td>7</td>
<td>10</td>
<td>25</td>
</tr>
</tbody>
</table>

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.

**Design Groundwater Level**

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. Additives 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

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

- 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.

%Langan.com\data\NY\data2\170394201\Office Data\Reports\Geotechnical\Updated Geotechnical Report\2016-03-03 Geotechnical Engineering Study.docx
Appendix E    SOE Plan Drawing
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

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

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

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<th>Casing Diameter (inches)</th>
<th>Caisson Reinforcing (Grade 75)</th>
<th>Concrete/Grout Strength (ksi)</th>
<th>Reinforcement Steel Strength Fy (ksi)</th>
<th>Spring Constant K-down (kips/inch)</th>
<th>Spring Constant K-up (kips/inch)</th>
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<tr>
<td>24&quot; Secant</td>
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<td>2500</td>
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<td>30 x 0.5 Wall</td>
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<td>75</td>
<td>7000</td>
<td>5800</td>
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