

# Geotechnical Engineering Study

for

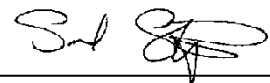
## 55 Hudson Yards Manhattan, New York

*Prepared For:*

**Related Companies  
60 Columbus Circle  
New York, New York 10023**

*Prepared By:*

**Langan Engineering, Environmental, Surveying  
and Landscape Architecture, D.P.C.  
21 Penn Plaza  
360 West 31<sup>st</sup> Street, 8<sup>th</sup> Floor  
New York, New York 10001**



**Saul Shapiro, P.E.**



**Marc Gallagher, P.E., LEED AP  
Professional Engineer License No. 081664-1**

**03 February 2014  
170256702**

# **LANGAN**

---

<b>INTRODUCTION .....</b>	<b>1</b>
<b>PROJECT DATUM AND COORDINATE SYSTEM.....</b>	<b>1</b>
<b>SITE DESCRIPTION .....</b>	<b>1</b>
<b>PROPOSED CONSTRUCTION .....</b>	<b>2</b>
MTA No. 7 Line Structures.....	2
55 Hudson Yards Tower .....	3
<b>ADJACENT SITES, BUILDINGS, STRUCTURES, AND UTILITIES.....</b>	<b>4</b>
No. 7 Line Extension (34 <sup>th</sup> Street Station Cavern and Support Tunnels) .....	4
Eleventh Avenue Viaduct (New York City Department of Transportation) .....	4
West Side Yards (MTA-LIRR) .....	5
Amtrak North Access Tunnel (Empire Line) .....	5
Amtrak North River Tunnels .....	6
Jacob Javits Truck Marshalling Yard .....	7
Utilities.....	8
<b>FUTURE AND ON-GOING CONSTRUCTION.....</b>	<b>8</b>
Street Reconstruction - West 33 <sup>rd</sup> Street .....	9
Hudson Yards .....	9
<b>SITE HISTORY .....</b>	<b>9</b>
<b>LOCAL GEOLOGY .....</b>	<b>10</b>
<b>GEOTECHNICAL INVESTIGATION.....</b>	<b>12</b>
Langan Borings (2007).....	12
Borehole Geophysical Logging (2007) .....	13
<b>PACKER PERMEABILITY TESTING IN ROCK (2007) .....</b>	<b>13</b>
Investigations by Others (Various).....	14
<b>SUBSURFACE CONDITIONS.....</b>	<b>17</b>
Fill [Class 7].....	18
Bedrock [NYCBC Class 1a-d].....	18
Groundwater .....	19
Bedrock Permeability.....	20
<b>FLOOD PLAIN .....</b>	<b>20</b>
<b>SEISMIC EVALUATION.....</b>	<b>21</b>
Mapped Spectral Accelerations.....	21
Site Class .....	21

---

<b>Design Spectral Response Accelerations .....</b>	<b>22</b>
<b>Liquefaction .....</b>	<b>22</b>
<b>DESIGN AND CONSTRUCTION CONSIDERATIONS .....</b>	<b>22</b>
<b>FOUNDATION RECOMMENDATIONS.....</b>	<b>23</b>
<b>Foundation Discussion .....</b>	<b>23</b>
<b>Footings Bearing on Rock .....</b>	<b>23</b>
<b>Drilled Caissons.....</b>	<b>24</b>
<b>Cellar Floor Slab .....</b>	<b>26</b>
<b>Permanent Below Grade Walls.....</b>	<b>27</b>
<b>Waterproofing .....</b>	<b>27</b>
<b>GENERAL CONSTRUCTION RECOMMENDATIONS .....</b>	<b>28</b>
<b>Temporary Excavation Support.....</b>	<b>28</b>
<b>Temporary Control of Groundwater .....</b>	<b>30</b>
<b>Excavation .....</b>	<b>30</b>
<b>Preconstruction Conditions Documentation and Monitoring .....</b>	<b>31</b>
<b>Construction Quality Assurance.....</b>	<b>32</b>
<b>OWNER AND CONTRACTOR OBLIGATIONS .....</b>	<b>33</b>
<b>LIMITATIONS.....</b>	<b>34</b>

## LIST OF FIGURES

Figure 1	Site Location Map
Figure 2	Existing Conditions Plan
Figure 3	Historic Map (Bridges, 1814)
Figure 4	Historic Map (Dripps, 1867)
Figure 5	Geologic Map
Figure 6	Boring Location Plan
Figure 7	Section A-A' (East-West Section)
Figure 8	Section B-B' (East-West Section)
Figure 9	Top of Rock Contour Plan
Figure 10	Stereonet (Scatter Plot of Discontinuity Pole Concentrations)
Figure 11	Stereonet (Fisher Concentrations of Bedrock Discontinuities)
Figure 12	Stereonet (Major Joint Sets Planes)
Figure 13	FEMA Flood Plain Map
Figure 14	FEMA Advisory Base Flood Elevation Map
Figure 15	Earth Pressure Diagrams (Permanent Walls)
Figure 16	Earth Pressure Diagrams (Temporary Walls)

## LIST OF TABLES

Table 1	Elevation Datum Conversions
Table 2	Measured Groundwater Levels
Table 3	Average Bedrock Permeability Rates

## LIST OF APPENDICES

Appendix A – Historic Design Drawings	
•	Eleventh Avenue Viaduct
•	Amtrak North Access Tunnel (Empire Line)
•	Amtrak North River Tunnels
•	Amtrak Emergency Evacuation Tunnel (for NRTs)
•	MTA No. 7 Line Extension
Appendix B	Boring Logs (Langan 2007)
Appendix C	Rock Core Photographs
Appendix D	Borehole Geophysical Logs
Appendix E	Boring and Test Data (by others)

## **INTRODUCTION**

This report presents our geotechnical engineering study for the proposed development of 55 Hudson Yards in Manhattan, New York. The study was performed to evaluate the subsurface conditions within the site and develop recommendations for foundation design and construction. As described herein, our study included review of available information, geotechnical engineering analyses, and preparation of this report. Our understanding of the project is based review of available project plans, and discussions and meeting attendance with the following:

- 1) The Related Companies (Related);
- 2) Kohn Pederson Fox Associates (the project architect);
- 3) WSP Cantor Seinuk (the project structural engineer); and
- 4) the No. 7 Line Extension design team comprised of representatives from the PB Team and Metropolitan Transportation Authority (MTA).

All work was performed in general accordance with our proposal to Related dated 15 November 2013.

## **PROJECT DATUM AND COORDINATE SYSTEM**

All elevations herein refer to the New York City Transit (NYCT) Datum. See Table 1 for typical datum conversions. All coordinates and directional descriptions provided herein refer to New York State Plane, Long Island (3104), U.S. Eleventh Avenue is oriented approximately N29°E; the avenues are typically referred to locally as Manhattan Meridian North.

## **SITE DESCRIPTION**

The project site is located between West 33<sup>rd</sup> and West 34<sup>th</sup> Streets, on the east side of 11<sup>th</sup> Avenue in Manhattan, New York. A site location plan is attached as Figure 1.

The project site is bound by West 34<sup>th</sup> Street to the north, West 33<sup>rd</sup> Street to the south, the Eleventh Avenue viaduct on the west and Hudson Boulevard Park (under construction) on the east. The lot is mapped by the Department of City Planning (DCP) as Block 705, Lot 1 and comprises an area of about 40,000 square feet.

West 34<sup>th</sup> Street and the Eleventh Avenue viaduct are generally level at about el 130 ft in the vicinity of the site. West 33<sup>rd</sup> Street generally slopes down from a high point of about el 130 ft at the intersection with Eleventh Avenue to about el 124 ft at the east end of the site. Surface grades below the viaduct are generally level at about el 107 ft and varying only about 1 to 2 ft in the vicinity of the site.

## **PROPOSED CONSTRUCTION**

The project site houses a subway facility recently constructed as part of the MTA No. 7 Line Extension project. The No. 7 Line facilities generally include a below grade station entrance and support space, a ventilation building, and entrance tunnels that connect to the 34<sup>th</sup> Street Station Cavern. The 55 Hudson Yards building will sit atop and adjacent to the MTA structure.

Additional details pertaining to the No. 7 Line construction and the 55 Hudson Yards project is provided below.

### **MTA No. 7 Line Structures**

A subway entrance structure was constructed throughout the majority of the project site and extends below Hudson Boulevard Park as shown in Figure 2. In addition, an above grade ventilation building is located on the south side of site and extends about 80 ft above grade.

The below grade structure extends to depths ranging from about 30 ft below grade at the east (near Hudson Boulevard) to about 125 ft below grade at the west (connecting to the main subway tunnel). Three floor levels are present below grade and are referenced on architectural drawings as B-1, B-3, and B-5. These areas generally house escalators, mechanical systems, and other support facilities for the No. 7 Line Extension. A portion of the B-1 level at the northwest corner of the site is comprised of an unexcavated area. This area will be occupied

by the cellar of the 55 Hudson Yards overbuild. Two entrance tunnels, designated as E1 and E3, connect the subway entrance structure to the No. 7 Line Extension 34<sup>th</sup> Street Station Cavern located to the west of the site below Eleventh Avenue; a third tunnel connects to the ventilation structure exiting the site below West 33<sup>rd</sup> Street.

The No. 7 Line ventilation building is designed to operate independently of the 55 Hudson Yards tower; however structural columns and shear walls were incorporated into the ventilation building to accommodate the 55 Hudson Yards tower overbuild.

### **55 Hudson Yards Tower**

The proposed overbuild generally consists of a tower that will extend to a height of about 780 ft above grade with the first-floor elevation at about 131 feet. The building will be anticipated to be a concrete framed structure with a conventional concrete core. The tower will be integral with the MTA ventilation building and subterranean station entrance, sharing columns and shear walls. The overbuild will have two cellar levels for mechanical and storage spaces in the northwest quadrant of the site, with a lowest finished floor at about el 104 feet.

An easement governs the design and construction of the overbuild. The maximum allowed service loads for existing columns established in the easement vary from about 300 kips to 20,700 kips in compression, and about 250 kips to 4,600 kips in tension. The allowable bearing pressures for new footings constructed within the unexcavated zone at the northwest quadrant of the site are also dictated by the easement and vary from 8 ksf to 40 ksf. The lowest allowable bearing pressure permitted is located on the east end of the site where the E1 and E2 tunnels are shallowest. Additional information regarding the distribution of bearing pressure within the site follows.

Service loads for new caissons, provided by WSP, vary from about 6,000 kips to about 36,000 kips in compression, up to about 1,400 kips in tension, and about 25 kips to 1,600 kips lateral. All proposed loads comply with the easement as confirmed by WSP.

---

## **ADJACENT SITES, BUILDINGS, STRUCTURES, AND UTILITIES**

The following section presents brief descriptions of notable property, buildings, structures, and utilities in the vicinity of the site.

### **No. 7 Line Extension (34<sup>th</sup> Street Station Cavern and Support Tunnels)**

The project site is located east of the 34<sup>th</sup> Street Station Cavern and contains the ventilation building and support tunnels, as part of the MTA No. 7 Line Extension project. The ventilation building is located on the south side of the site, and two entrance tunnels are located north of the ventilation building. The entrance tunnels, designated as E1 and E2, daylight within the site and connect to station entrance located below Hudson Boulevard Park. A support tunnel designated T2 is located near the southwest corner of the site and connection to the ventilation building. The approximate location and extent of the No. 7 Line structures are shown in Figure 2.

The station cavern was excavated by a combination of bored excavation via a tunnel boring machine (TBM) and controlled blasting, the running tunnels outboard of the station were excavated via a TBM, and the support tunnels were excavated via drilling and controlled blasting. The crown of the T1B tunnel is about 80 feet below existing grade. The crown of the E1 and E2 tunnels vary from 80 feet below existing grade on the western border of the site to approximately 35 feet below existing grade where they connect to the lower mezzanine of the subway station entrance. Select drawings from the No. 7 contract documents are included in Appendix A.

The MTA No. 7 Line will remain and must be protected during construction. MTA approval of the design and construction will be required to obtain New York City Department of Buildings (NYC DOB) permits.

### **Eleventh Avenue Viaduct (New York City Department of Transportation)**

The Eleventh Avenue viaduct is located on the west border of the site. The viaduct runs from West 30<sup>th</sup> Street to West 37<sup>th</sup> Street. The viaduct generally consists of a steel-frame structure with a reinforced concrete deck. Four steel columns supporting the viaduct are located



immediately west of the site. These columns are supported by plain concrete piers bearing on bedrock. The viaduct is anticipated to be reconstructed in the future in the area fronting the site; a reconstruction date is not known at this time. The Eleventh Avenue viaduct is shown on the existing conditions plan, Figure 2.

The viaduct will remain and must be protected during construction.

### **West Side Yards (MTA-LIRR)**

The LIRR West Side Yards encompass the superblock bound by West 33<sup>rd</sup> Street to the north, former West 31<sup>st</sup> Street to the south, Tenth Avenue to the east, and Twelfth Avenue to the west. The yards were built in 1983, but the area has been used as rail yards for more than 100 years. Currently, the property is primarily used for storage and maintenance of LIRR commuter trains. Several structures and facilities are present at grade throughout the site, serving various uses including maintenance and cleaning facilities, electrical substations, and control towers. A concrete retaining wall is located along the north side of the yards supporting West 33<sup>rd</sup> Street.

The West Side Yards encompass the Hudson Yards development which will include construction of a structural platform over the yard. In addition, several towers will be constructed along the streets bounding the property. Two towers, designated as A and E, are proposed along the West 33<sup>rd</sup> Street side of the property between Tenth and Eleventh Avenue. Additional information pertaining to Hudson Yards follows.

LIRR reviews all projects within 200 ft of their facilities; we expect they will issue a letter of no impact for the project, but may require monitoring.

### **Amtrak North Access Tunnel (Empire Line)**

Amtrak North Access Tunnel (NAT), also known as the "Empire Line", was constructed in late 1980s by Amtrak to provide rail access through the Westside of Manhattan to points north. The tunnel runs west-northwest below the West Side Yards in a sweeping arc before heading north-northeast below the Javits Truck Marshalling Yard (Block 679, Lot 1) and Eleventh Avenue viaduct. The tunnel is relatively shallow, with its deepest point located within the

yards; ground cover decreases to the north and the tunnel day-lights into a U-shaped, reinforced concrete portal north of West 34<sup>th</sup> Street.

The eastern edge of the NAT is located about 70 to 110 ft from the west property boundary of the site and is closest at the northwest corner of the site. Adjacent to the site, the NAT consists of an approximately 22-ft wide by 24-ft tall reinforced concrete box and was built using cut and cover construction techniques. The tunnel is partially embedded in soil and partially embedded in bedrock. The tunnel is relatively shallow; ground cover decreases to the north and the tunnel daylight into a U-shaped, reinforced concrete portal north of West 34<sup>th</sup> Street. The top of rail grades near the site vary from about elevation 86 to 89 feet. The crown of the tunnel is located approximately 17 feet above top of rail, or about 1 to 3 feet below the ground surface. A vent/emergency egress enclosure is located at grade below the Eleventh Avenue viaduct and above the tunnel (under West 33<sup>rd</sup> Street at the west side of Eleventh Avenue). Select "As-Built" drawings are included in Appendix A.

Amtrak must review and approved the proposed work because the tunnel is within 200 ft of the site. We anticipate that a letter of no impact can be obtained given the distance between the building and the tunnel.

### **Amtrak North River Tunnels**

The Amtrak North River Tunnels (NRTs) run below the West Side Yards and are located about 250 feet south of the site (roughly coincident with the former West 32<sup>nd</sup> Street). The NRTs were constructed by the Pennsylvania Railroad in the early 1900's to provide rail access to Manhattan via Penn Station. The tunnels currently carry commuter trains for Amtrak and New Jersey Transit.

The tunnels were built using drilling and blasting techniques and tunneling shields, and consist of arch sections (i.e. inverted U-shape) with concrete and brick liners. The majority of the tunnel is fully embedded in bedrock; however, there is a 170-foot section beneath the West Side Yards between 10<sup>th</sup> and 11<sup>th</sup> Avenue that is partially embedded in soil. Part of the tunnel adjacent to Tenth Avenue may also be partially embedded in soil as a result of excavation that took place during construction of the West Side Yards in the 1980's. Both the north and south

tunnels slope down to the west with a gradient of about 1.9 percent. Top of rail grades vary from about elevation 76 to 57 feet from Tenth Avenue to Eleventh Avenue.

#### Amtrak Emergency Evacuation Tunnel (for NRTs)

A new emergency evacuation tunnel from the NRTs was constructed in 1982 in conjunction with the West Side Yards project. The emergency evacuation tunnel runs in a roughly southwest-northeast alignment. The evacuation tunnel connects to an egress enclosure located immediately adjacent to the sidewalk south of West 33<sup>rd</sup> Street directly across the street from the proposed development.

The evacuation tunnel was built using cut and cover construction and consists of reinforced concrete liner walls. The tunnel exits the NRT at about elevation 90 feet, runs under the LIRR yard to about elevation 92 feet and is almost entirely embedded in bedrock. A small part of the tunnel adjacent to the egress at West 33<sup>rd</sup> Street, as well as the egress itself, is partially embedded in soil and partially embedded in bedrock. The emergency evacuation tunnel is shown on the existing conditions plan, Figure 2. Select "As-Built" drawings are included in Appendix A.

The emergency evacuation tunnel egress point is expected to be relocated as part of Related's Hudson Yards project. Amtrak will review the 55 Hudson Yards design with respect to the evacuation tunnel.

#### **Jacob Javits Truck Marshalling Yard**

The Jacob Javits Truck Marshalling (JTM) Yard is located west of the site and is bound by West 34<sup>th</sup> Street to the north, West 33<sup>rd</sup> Street to the south, Eleventh Avenue to the east, and Twelfth Avenue to the West. The JTM yard consists of an at-grade trailer storage facility serving the Javits Convention Center. Maintenance access to the NAT is provided through the JTM yard.

## Utilities

Public and private utilities are present within the rights-of-way fronting the site. Noteworthy existing utilities potentially affecting design and construction in the vicinity of the site include:

- A 15-inch sewer located roughly in the center of West 33<sup>rd</sup> Street, connecting to an inverted siphon below the viaduct and Amtrak NAT;
- An inverted sewer siphon located below the viaduct at West 33<sup>rd</sup> Street and spanning under the Amtrak NAT;
- A 16-inch storm sewer located approximately 10 ft west of the site. The 16-inch storm sewer flows south below the viaduct and ties to the West 33<sup>rd</sup> Street sewer siphon;
- A 3' x 3' concrete box sewer below the viaduct, extending below the viaduct from West 34<sup>th</sup> Street on a slight diagonal south-southwest to the West 33<sup>rd</sup> Street siphon;
- A 2'-4" by 4'-6" egg shaped sewer in West 34<sup>th</sup> Street. The West 34<sup>th</sup> Street sewer flows west to a manhole located beneath the viaduct;
- An electrical substation located below the viaduct at West 33<sup>rd</sup> Street (adjacent to the north side of the West 33<sup>rd</sup> Street sewer siphon);
- A catch basin located adjacent to the southwest corner of the site. The culvert picks up flow from a leader on the viaduct and drains to the West 33<sup>rd</sup> Street siphon.

Numerous other utilities are present in the areas fronting the site including: 1) electrical lines and vaults, 2) low and high pressure water mains; 3) telecommunications; and 4) natural gas lines. Several utilities are anticipated to be reconstructed at West 33<sup>rd</sup> Street as discussed below.

## FUTURE AND ON-GOING CONSTRUCTION

This section presents a brief overview of adjacent projects which may impact design and construction within the site; most notably: 1) Reconstruction of West 33<sup>rd</sup> Street; and 2) Hudson Yards Development. Numerous other public and private construction projects are

expecting near or adjacent to the site, but are expected to have less impact from a design standpoint and more on construction logistics.

### **Street Reconstruction - West 33<sup>rd</sup> Street**

Reconstruction of West 33<sup>rd</sup> Street is currently in design. Construction of West 33<sup>rd</sup> Street will include replacement of existing utilities, regrading of the road (grades being raised along a substantial length of the road between Tenth and Eleventh Avenues), and reconstructing the retaining wall along the south side of the road to accommodate increased grades.

West 33<sup>rd</sup> Street will be closed to traffic during reconstruction. If the West 33<sup>rd</sup> Street reconstruction is concurrent with the construction of 55 Hudson Yards, limited site access, and increased traffic volume on adjacent cross streets should be expected.

### **Hudson Yards**

Related's development over the West Side Yard (East Rail Yard) is currently in the construction phase. The rail yard will remain active during and after construction. The development generally includes:

- Tower A (65-story steel framed office structure) – corner of 33<sup>rd</sup> Street and Tenth Avenue
- Tower E (900-foot tall reinforced concrete mixed-use structure) – corner of 33<sup>rd</sup> Street and Eleventh Avenue
- Platform – encompasses entire Eastern Rail Yards to provide support for commercial and residential buildings and landscaping.

Construction on the adjacent side of the street should not significantly impact proposed construction however site access should be coordinated during the construction phases given the potential for lane closures that may affect construction logistics.

### **SITE HISTORY**

Historic maps (Figures 3 and 4) show the original Hudson River shoreline located about 50 to 100 ft west of the site's western property line. The old shoreline was closest to the site at the

southwest corner. The area west of the site was filled in stages between the mid to late 1800s, extending the shoreline west to its current configuration. No historic streams or marshes are known to existing within or immediately adjacent to the site.

Available resources show the site has undergone several evolutions of development throughout its history. Historic topographic, real-estate, fire insurance, and land-use maps suggest development began in the early 1800s. Early use of the site was primarily of an agricultural character with various commercial and industrial uses following through the mid-1800s. Between the 1850s and early 1900s, portions of the site served as storage yards, an iron works, various foundries and factories, garages, a lard refinery, and various storehouses and warehouses.

Several infrastructure improvements were made in the in the 1930s as part of the City of New York and New York Central Railroad (NYCRR) Westside Improvements Project. Improvements were generally made to facilitate rail traffic through the Eleventh Avenue corridor and included construction of the Eleventh Avenue viaduct and alterations to subsurface utilities. Grades below Eleventh Avenue were lowered about 6 to 8 ft and raised about 16 ft on West 33<sup>rd</sup> and West 34<sup>th</sup> Street for construction of the viaduct.

Three buildings were constructed on the block in the early 1900s and generally occupied the site until the No. 7 Line development began. These buildings typically served as commercial centers, warehouses, distribution centers and garages.

Construction at the site, recently performed by the MTA, generally included demolition of several buildings, installation of support of excavation (SOE) systems, excavation of the site to rock, construction of the No. 7 Line structures described previously, and backfilling of the undeveloped areas of the site to about el 130 ft in conjunction with construction of a mechanically stabilized earth (MSE) wall.

## **LOCAL GEOLOGY**

The site is located on Manhattan Island which falls within the southern terminus of the Manhattan Prong of the New England Upland province. Bedrock in the vicinity of the site

generally consists of granite, schist, and gneiss overlain by glacial and fluvial soil deposits as well as extensive manmade fill. Although altered by urban development, topography within Manhattan historically mimicked the underlying bedrock.

According to Baskerville<sup>1</sup>, bedrock stratigraphy in the vicinity of the site consists of rock of the Lower Cambrian to Middle Ordovician Age (Hartland Formation) and intrusive rock presumably of the Silurian age, consisting of granite and megacrystalline pegmatite; see Figure 5. A large sill of intrusive granite is mapped north of the site; however, historic boring data indicates that this granite sill extends further south than mapped. Boundaries between the intrusive granite and Hartland Formation rocks are not well defined as evidenced by intermittent contacts and inclusions observed in rock cores.

Generalized descriptions of rocks mapped the vicinity of the site follow:

1. Hartland Formation – Interbedded units of: a) gray, fine grained quartz-feldspar granulite containing minor biotite and garnet; b) Fine to coarse grained, gray to tan weathering, quartz-feldspar-muscovite-biotite-garnet schist (mica schist); c) Dark greenish-black quartz-biotite-hornblende amphibolite. Intrusions of granite and pegmatite are common (Baskerville 1994).

Metamorphism has resulted in foliation, a distinct planar alignment of mineral grains, within rocks of the Hartland Formation. This grain alignment is commonly referred to as schistosity in the more platy schistose rock or compositional banding in gneissic rocks. Foliation is typically oriented either northwest or southeast and dips steeply within Manhattan as discussed by Baskerville, but may be altered locally as a result of folding.

2. Granite and Pegmatite – Gray-white-pink medium to coarse grained, biotite-muscovite-microcline-quartz granite and megacrystalline pegmatite in dikes less than 3 feet thick and sills greater than 3 feet thick. Accessory minerals include tourmaline, pyrite, garnet,

---

<sup>1</sup> I-MAP 2306, Sheets 1&2. "Bedrock and Engineering Geology Maps of New York County, and parts of Kings and Queens Counties, New York, and parts of Bergen and Hudson Counties, New Jersey". C.A. Baskerville, USGS 1994.

and epidote. A thick sill cross-cuts rock the Hartland Formation as shown in Figure 5 (Baskerville 1994).

## **GEOTECHNICAL INVESTIGATION**

### **Langan Borings (2007)**

Langan performed a field investigation in 2007 that included drilling five borings, designated as L-SJ-1 through L-SJ-5. The boring locations were distributed throughout the site. The borings were located in the field by our engineer by measuring from existing structures. The elevations at the boring locations were measured using an automatic level referenced to a surveyed benchmark. A boring location plan is attached as Figure 6.

The borings were drilled between 14 August 2007 and 28 November 2007 by Warren George, Inc., under the full-time inspection of a Langan engineer. The test borings were drilled to depths varying from about 99 to 167 ft below the existing ground surface using a DK-525 track mounted drill rig. The borings were advanced through soil using mud rotary drilling techniques with a tri-cone roller bit and drilling fluid, consisting of a mixture of bentonite and water. Temporary steel casing was installed through fill and native overburden soils as required to stabilize the boreholes and prevent fluid loss during drilling. Rock cores were obtained using 5-ft and 10-ft Type NQ double wall core barrel. Soil samples were obtained using a standard 2-inch OD split-spoon sampler driven by a donut-type hammer.

The Standard Penetration Test (SPT)<sup>2</sup> was performed with soil overburden in accordance with ASTM D-1586-99; SPT N-values were recorded by our inspecting engineer. All soil and rock samples were visually examined and classified in the field in accordance with ASTM D2487 and the NYCBC. Rock cover recovery (REC)<sup>3</sup> and rock quality designation (RQD)<sup>4</sup> were logged in the field by our engineer.

---

<sup>2</sup> The Standard Penetration Test is a measure of soil density and consistency. The SPT N-value is defined as the number of blows required to drive a 2-inch OD split-spoon sampler 12-inches, after an initial "seating" penetration of 6-inches. The split-spoon sampler is advanced by driving using a 140-lb hammer free falling from a height of 30-inches.

<sup>3</sup> Rock core recovery (REC) is defined as the length of all core pieces recovered divided by the total core run length.

<sup>4</sup> Rock Quality Designation (RQD) is defined as the sum of all recovered sound rock core pieces measuring 4-inches or more in



Groundwater was measured at the completion of drilling each boring. A temporary groundwater observation well was installed in boring L-SJ-4 upon completion of drilling. The groundwater well consists of a solid PVC standpipe over 10 ft of slotted PVC well screen. The groundwater levels were measured periodically during our time on-site and following completion of the field work.

Logs of all drilling activities are provided in Appendix A. Photographs of recovered rock cores are provided in Appendix B.

### **Borehole Geophysical Logging (2007)**

A borehole geophysical survey was conducted in each 2007 Langan boring upon completion of drilling. The purpose of this survey was to evaluate the orientation and spacing of discontinuities (fractures, joints, foliation, etc.) of bedrock within the borings. Geophysical logging included acoustic televiwer (ATV) and optical televiwer (OTV) methods. Geophysical logging was performed on 18 September 2007 and 25 September 2007 by Hager Richter Geoscience of Fords, New Jersey. A discussion of the ATV and OTV methods, recorded borehole data, and data interpretation is presented in Hager Richter's final report, attached as Appendix C.

### **Packer Permeability Testing in Rock (2007)**

Packer permeability tests were conducted in two borings (L-SJ-1 and L-SJ-4) in general accordance with the procedures outlined the US Bureau of Reclamation Earth Manual. The purpose of the tests was to evaluate the permeability of the rock and the potential for groundwater flow into the excavation through fissures in the rock mass. The packer tests were performed by Warren George, Inc. following completion of drilling at each respective boring.

Constant head injection tests were performed at 10 ft intervals from the borehole bottom to the top of rock. Two inflatable packers were used to isolate each 10 ft section of the borehole. The water was injected between the packers using sequential pressure increments and the

---

length (for type NX or NQ cores) divided by the total core run length. RQD is a relative indicator of rock quality.

resulting flow was recorded by our inspecting engineer at one minute intervals until a roughly steady state condition was reached.

### **Investigations by Others**

The following section presents a brief summary of available data which was reviewed with respect to design and construction at the site. The information reviewed included publicly available documents and documents from our in-house archive for projects in the vicinity of the site. The following data sources were reviewed:

- No. 7 (Flushing) Line Extension - RFP documents for Contract C-26503, Construction of Running Tunnels and Station Structures, Volumes 1-6;
- No. 7 (Flushing) Line Extension – RFP documents for Contract C-26510, Excavation/Mining/Lining of Vertical Shaft, E1 and E2 Inclined Tunnels, and T1 Connector Tunnel; and Construction of a Ventilation Building and Station Entrance Structure at Site J, Volumes 1-6;
- Jacob Javits Convention Center Expansion and Renovation – Historic reports and data, and ongoing Langan Geotechnical Studies;
- Amtrak North Access Tunnel – Geotechnical report, as-built drawings;
- NYCDOT Eleventh Avenue Viaduct – Historic plans, boring and test pit data, and preliminary foundation layouts for proposed reconstruction;
- Site P - 2007 Langan Geotechnical Study, historic drawings and boring information;
- Hudson Boulevard Garage - 2006 Langan Geotechnical Study;
- John D. Caemmerer Yards – Historic boring and test pit data, 2005 Langan Geotechnical Study for proposed Jets Stadium;
- NYCDDC – Historic boring and excavation data sets;
- NYCDEP – Historic plans, Stormwater and Sanitary Drainage Management plan (2007);

- Maps – Various historic, real estate, fire insurance, geologic, and topographic maps; Federal Emergency Management Agency (FEMA) flood maps

Our review was focused on subsurface bedrock data immediately adjacent to the site with regard to composition, continuity, and depth below grade. Much of this information was presented the No. 7 Line Extension documents (Volume 4 – Contract C-26503 and Volume 3 – Contract-26510). Boring logs and representative data is provided in Appendix E. The following summarizes major points obtained from the available data sources:

- Borings drilled adjacent to the site indicate that bedrock elevations vary from about el 91 ft to el 112 ft. The bedrock surface is irregular, but generally slopes downward from east to west. Bedrock is shallowest with respect to surface grades near the west end of the site; this is due in large part to an abrupt change in the ground surface elevation at the Eleventh Avenue viaduct (about 24 feet).
- As indicated in the Geotechnical Baseline Report for the No. 7 line, bedrock in the vicinity of the site is comprised of rock of two groups; the “Granite Group” and the “Schist Group”. The Granite Group is composed of intrusive igneous rocks including granite, granitic gneiss, pegmatite, and aplite. The Schist Group includes mica and hornblende schist, schistose gneiss, and gneissic schist. Similar observations were presented in boring data for other projects in the area. Boring data suggests that the majority of rock within the site will consist of rocks of the Granite Group; however, a significant amount of rock of the Schist Group was observed within several borings near the northwest corner of the site.
- Boring and geophysical data indicates that increased weathering and fracturing is common at contacts between the two rock groups defined above (i.e. schist-granite, schist-pegmatite, etc.). In general, contacts will be near vertical at lateral boundaries of the Schist and Granite Groups; however, distinct boundaries are not likely. Inclusions, observed as alternating layers in boreholes, are likely present throughout the site.

- Geophysical logging performed for the No. 7 Line Extension indicates that four predominant joint sets are present in the vicinity of the site. Although rock of both the Schist and Granite Groups are present in each joint set, preferential grouping is indicated by the data. The most prevalent of the sets (Joint Set 1) strikes about N10°E and dips about 40° to 70° west. Joint Set 1 is primarily composed of steeply dipping foliation and fracturing within the schist, granitic gneiss and pegmatite. The second most prevalent set (Joint Set 2) is comprised chiefly of nearly horizontal, large aperture, persistent fractures of the Granite Group which typically dip less than 20°. The third joint set (Joint Set 3) is chiefly comprised of steeply dipping gneissic banding within the Granite Group. Joint Set 3 strikes about N5°W and dips 30° to 40° east-northeast. The fourth joint set (Joint Set 4) generally contains discontinuities found in pegmatite and schist rocks. Joint Set 4 strikes about N70°E and dips steeply at about 65° to 80° southeast.
- As discussed in the No. 7 Line Baseline Geotechnical Report, relatively high horizontal in-situ stresses are expected within the rock based on the results of hydro-fracturing tests, core logging, and previous tunneling experience within Manhattan. Stresses are apparently higher in rock of the more massive Granite Group. The stresses are typically oriented in a north-south orientation. High stresses have reportedly caused deformation (i.e. bulging, slabbing, heaving, “popping”, etc.) during past tunneling projects in Manhattan.
- Groundwater elevations measured in monitoring wells adjacent to the site vary substantially. Groundwater elevations measured in wells installed for the No. 7 Line Extension (FD-08W, FD-406W), south and west of the site, varied from about el 62 ft to el 65 ft. Attempts to measure groundwater levels in two deep borings (L-SP-5, L-SP-7) performed by Langan at Site P, north of West 34<sup>th</sup> Street, showed water to be greater than 100 ft deep (i.e. water was deeper than could be measured with a 100 ft long water level indicator). Elsewhere within Site P, water was found to be perched atop rock at about el 108 ft. The lower groundwater levels appear to be related to the presence of wide aperture nearly horizontal fractures of the aforementioned Joint Set 1.

- Bedrock permeability rates measured in boreholes for the No. 7 Line extension vary from  $10^{-7}$  to  $10^{-4}$  cm/sec. The permeability rates are largely affected by the presence of discontinuities in the rock. Permeability rates tend to be highest in rock containing wide aperture fractures, typical of Joint Set 1 and more-highly weathered and fractured contacts between the Schist and Granite groups. Groundwater flow rates will be governed by connections of the fractures beyond the excavation line. The Geotechnical Baseline report indicates that highest estimated groundwater flow for the No. 7 Line Extension project is expected to occur in the vicinity the 34<sup>th</sup> Street Cavern, adjacent to the site.
- Lab testing performed for the No. 7 Line Extension and adjacent projects measured unconfined compressive strengths (UCS) from about 4,000 psi to about 40,000 psi within the site. In general, rock of the Granite Group is stronger than that of the Schist Group. Structural failure (i.e. fracture along discontinuity planes) is predominant in rocks of the Schist Group, and non-structural failure (i.e. shattering) is more prevalent in rock of the Granite Group. Data from the No. 7 Line Extension indicates that rock of the Granite Group is considerably more abrasive than that of the Schist Group.
- Shear-wave velocities were measured in bedrock at the Jacob Javits Convention Center and at locations along the route of the No. 7 Line Extension. Shear wave velocities were all in excess of 5,000 ft/sec indicating the rock classifies as Seismic Site Class A in accordance with the NYSBC and NYCBC.

## **SUBSURFACE CONDITIONS**

The general subsurface stratigraphy in the vicinity of the site consists of fill overlying bedrock; a thin mantle of decomposed rock may be present above competent bedrock at some locations. A brief description of each layer is presented below in order of increasing depth. Subsurface profiles are presented in Figures 7 and 8.

### **Fill [Class 7]<sup>5</sup>**

A layer of controlled fill is present below the undeveloped portion of the site (i.e. northwest quadrant). This fill was placed following excavation of the site to rock by the MTA's No. 7 Line Site J contractor. The fill was generally placed coincident with the construction of the MSE wall located immediately east of the Eleventh Avenue Viaduct, north of the ventilation building. The thickness of the fill layer is estimated to vary from about 17 to 38 ft.

### **Bedrock [NYCBC Class 1a-d]**

Bedrock is present underlying the fill. The bedrock surface elevations vary from about el 91 ft to el 112 ft (about 20 to 40 ft below street grade at Eleventh Avenue) within the site. A thin layer of decomposed rock was observed in boring L-SJ-2 and several borings performed for the No. 7 Line Extension. The bedrock surface is irregular but generally trends slightly downward toward the west; a trough in the rock surface is apparent within the north-central portion of the site, trending in a north direction (this trough continues across West 34<sup>th</sup> Street). A rock contour plan showing approximate elevations of the bedrock surface is presented in Figure 9.

Bedrock within the site typically consists of rocks of the previously defined Granite and Schist Groups. Rock type is not consistent across the site and commonly alternates with depth, but rock of the Granite Group was predominant in the borings performed. Relatively small inclusions of mica schist were observed within otherwise granitic rock throughout the site with the most pronounced concentration observed in borings SJ-6A and L-SJ-3 where schist with interbedded seams of gneiss and pegmatite was up to about 60 ft thick.

Rock core recovery for borings performed within the site varied from 51 to 100 percent and the rock quality designation varied from 11 to 100 percent. The lower RQD values were generally observed near the rock surface or at contacts (i.e. granite/schist, pegmatite/schist, etc.) where increased fracturing and weathering was more prevalent.

Rock of the predominant Granite Group varied from slightly weathered to fresh, fracture spacing varied from very close to wide, and the rock generally varied from hard to very hard.

---

<sup>5</sup> New York City Building Code 2007 soil/rock classification designation.

Inclusions of the Schist Group tended to display increased weathering and fracturing predominantly near contacts (i.e. schist to granite, granite to pegmatite, pegmatite to schist, etc.). Iron staining was common along fractures. Disking (very close uniform fracturing) of recovered rock cores was noted in several borings and is indicative of high in-situ horizontal stresses as discussed earlier.

Bedrock discontinuity orientation data obtained from borehole geophysical surveys (ATV/OTV) indicates the presence of four predominant joint sets which are in general agreement with observations made in the Geotechnical Baseline Report for the No. 7 Line Extension. A lower hemisphere scatter plot (Figure 10) shows the general concentration of discontinuity poles with respect to the proposed excavation walls. A lower hemisphere Fisher Concentration Plot is shown in Figure 11. The data presented in Figures 10 and 11 is inclusive of all borings performed within the site, and borings performed along the site perimeter for the No. 7 Line Extension. Planes representing the highest concentrations of discontinuity sets are illustrated as relative to proposed orientation of excavation walls and slopes in Figure 12. We note that the orientation and dip of possible planes will actually vary widely based on the scatter within the data set. The predominant joint sets may affect excavation locally through the creation of planes, blocks, or wedges.

The rock is generally designated NYCBC Class 1a - Hard Sound Rock; areas containing more highly fractured and weathered zones the rock is locally designated as Class 1b - Medium Hard Rock, or Class 1c – Intermediate Rock. Highly weathered and fractures zones observed at some locations near the bedrock surface are designated as Class 1d – Decomposed Rock.

## **Groundwater**

Groundwater was measured in one observation well installed as part of Langan's subsurface investigation (L-SJ-4). Groundwater levels were also measured in boreholes L-SJ-1, L-SJ-2, L-SJ-3 and L-SJ-5 after completion of drilling and prior to geophysical logging. The measured groundwater levels vary from about el 65 ft to 105 ft as shown in Table 2.

As indicated in Table 2, the groundwater level varies substantially within the site. Groundwater is perched atop rock in borings L-SJ-3, L-SJ-4, and L-SJ-5, but is considerably lower in L-SJ-1.

This difference is likely due to drainage through fractures within the bedrock near L-SJ-1. ATV logs of L-SJ-1 show wide aperture fractures below the measured groundwater level and fluid circulation was lost during drilling at about the measured groundwater level. We note that the groundwater elevation at L-SJ-1 is consistent with that observed in several borings performed for the No. 7 Line Extension (FD-08W and FD-406). The consistency of the groundwater elevation between the three borings indicates interconnected fractures which may extend significant distances.

### **Bedrock Permeability**

The results of packer permeability tests performed within the site are in general agreement with the data presented in the Geotechnical Data report for the No. 7 Line Extension Project. Permeability rates varied from about  $10^{-7}$  cm/sec to about  $10^{-3}$  cm/sec with the highest rates observed in borehole sections containing wide aperture fractures typical of Joint Set 1. In several instances, back pressure could not be developed during testing, indicating the fractures could accept water as fast as the pump could supply. The results of permeability testing are presented in Table 3.

Local rock permeability rates are likely to be higher during construction due to the increased exposure of fractures. Additionally, permeability rates can increase because of erosion and/or the opening of fractures resulting from excavation/blasting activities.

### **FLOOD PLAIN**

The current Federal Emergency Management Agency (FEMA) flood insurance rate map (FIRM) which dates from 2007 indicates that the site falls outside the limits of the 100-year flood plain. However, site elevations below the Eleventh Avenue viaduct are not accurately addressed in the FIRM and the site would be subject to flooding under the viaduct during the 100-year flood event. The current 100-year flood elevation is mapped at about el 107.4 ft (el 10 ft NGVD) in the vicinity of the site. The approximate limits of the 100-year flood zone are depicted in Figure 13.

“Preliminary FIRM’s” were recently released by FEMA in December 2013 and represent the best available data for use in design following Hurricane Sandy. Similar to the 2007 FIRM, the



2013 Preliminary FIRM does not correctly address site grade below the viaduct, and indicates that the site falls outside the 100-year flood plain. Because the site may be subject to inundation along its western boundary, we recommend that the site be designed to accommodate a 100-year design flood elevation of at least el 110.5 ft (el 12 ft NAVD) which equals the nearest mapped base flood elevation plus 1-foot of freeboard. An excerpt of the Preliminary FIRM map illustrating the site location is attached as Figure 14.

We understand that Related has opted to design the building to be flood resistant to el 113.5 ft. This value is in excess of the minimum design flood elevation (el 110.5 ft) as determined from the mapped flood zone nearest the site.

## **SEISMIC EVALUATION**

This section presents the results of our seismic evaluation for the site relative to the provisions outlined in the New York City Building Code (NYCBC) 2008. The seismic parameters determined from the NYCBC are more conservative than those of the New York State Building Code (NYSBC) 2012 which is used for design of MTA structures. The following subsections provide recommended parameters for use in seismic design of the proposed structure. The following seismic parameters comply with the requirements outlined in the NYCBC.

### **Mapped Spectral Accelerations**

Per section 1615.1 of NYCBC, the mapped maximum considered earthquake response spectra for the short period ( $S_s$ ) and 1-second period ( $S_1$ ) are 0.365g and 0.071g, respectively.

### **Site Class**

The NYCBC require assignment of a Site Class in accordance with the procedures outlined section 1615.1.1. The subsurface conditions at the anticipated foundation level generally consist of hard granitic and schist bedrock corresponding to Site Class A (hard rock profile). To substantiate Site Class A, the codes require that shear wave velocity ( $v_s$ ) be measured either onsite or within the same formation containing similar fracture and weathering characteristics. As discussed earlier, shear wave velocities were previously measured within the same formation and having values exceeding 5,000 ft/sec corresponding to Site Class A conditions.

Per Table 1615.1.1 of the NYCBC, the site coefficients for short period ( $F_a$ ) and 1-second period ( $F_v$ ) are 0.8 and 0.8, respectively.

### **Design Spectral Response Accelerations**

Design spectral accelerations were determined in accordance with section 1615.1.3 of the NYCBC. The design spectral response accelerations at short periods ( $S_{DS}$ ) and 1-second periods ( $S_{D1}$ ) are 0.194g and 0.037g, respectively.

### **Liquefaction**

Liquefaction need not be considered for the design given that all soils are to be removed from the site and that the all structure will be supported directly on rock.

## **DESIGN AND CONSTRUCTION CONSIDERATIONS**

The following summarize issues related to design and construction of foundation systems for the site.

- As previously indicated, two entrance tunnels connect the site to the 34<sup>th</sup> Street Station Cavern. Foundations systems over and adjacent to the tunnels have been coordinated with the MTA/PB design team and will consist of a combination of spread footings and caissons. Spread footings and caissons will comply with the MTA C-26510 Contract Drawings load plans and NYCT's General Notes so as not to impact MTA structures.
- Some dewatering will be required during construction because of the presence of perched groundwater and surface water runoff.
- Waterproofing of the below grade structure will be required because of groundwater conditions and the critical nature of the structure.
- Temporary soil excavation support will be required along the northern and western side of the site. The No. 7 Line structure surrounds the remainder of the excavation area and extends to or below bedrock.

## **FOUNDATION RECOMMENDATIONS**

Our recommendations for foundation related design parameters follow.

### **Foundation Discussion**

The 55 Hudson Yards overbuild will be supported by foundation elements previously installed within the No. 7 Line structures and new foundations located in the in the northwestern quadrant of the site. The new foundations will be comprised of a combination of shallow footings and a mat bearing on rock, and caissons socketed into rock below the influence lines of adjacent tunnels. All new foundations must comply with the easement established with the MTA and described on the reference drawings.

### **Footings Bearing on Rock**

The subsurface conditions within the site are favorable for supporting foundation loads using spread footings or mat foundations bearing on sound rock. The MTA/PB have established maximum bearing pressures for shallow foundation systems for both strip and mat foundations. These prescribed bearing pressures are significantly lower than NYCBC allowable bearing pressure for Class 1a bedrock (60 tsf) and address the presence of the No. 7 E1 and E2 entrance tunnels being below or adjacent to the new shallow foundations for the overbuild.

Spread footings must be designed assuming a linearly-varying allowable bearing pressure envelope of 40 ksf at the western site border to 20 ksf on the eastern edge of the overbuild cellar area. These limits are depicted on Drawing No. ST-40702 of Contract C-26510. Strip footings are not allowed to bear directly over the MTA entrance tunnels; however mat foundations may be utilized and can be designed assuming a linearly-varying allowable bearing pressure envelope of 16 ksf on the western site border to 8 ksf on the eastern building edge. At least 15 ft of cover must be maintained above the E1 and E2 tunnels per MTA requirements.

Lateral shear from wind and earthquake loads can be resisted using shear pins doweled into rock, and/or by embedding footings to develop passive resistance from the surrounding rock. The allowable passive resistance provided by the rock will be dictated by the depth of embedment and the presence of discontinuities (fractures, foliation, etc.) at a particular

location. Alternatively, floor slabs and mat foundations can be used as diaphragms to transfer loads to the exterior walls.

Individual footings should be designed assuming a minimum width of at least 3 ft and foundation wall footings should have a minimum width of at least 2 ft regardless of the allowable bearing pressure. Bedrock subgrades for isolated footings, mat foundations and wall footings should consist of clean, sound rock representing NYCBC Class 1a, 1b, or 1c.

All bedrock subgrades for spread footings should be level. Where required, slopes should be benched to provide level bearing conditions. In addition, benches should be provided below columns or walls within mat foundations located in slopes. Benches for individual footings should extend a minimum of 5 ft beyond the downslope edge of the footing. Footings adjacent to the tops of slopes should be overexcavated as directed by the geotechnical engineer where unfavorable bedrock discontinuities exist that may reduce allowable bearing pressure or present a potential sliding condition.

All bedrock subgrades should be scaled of loose rock and cleaned of debris using compressed air. Concrete should not be poured in standing water or on frozen ground. If concrete is to be placed in freezing conditions, appropriate protection must be employed. All subgrades should be inspected by the geotechnical engineer to verify that the subgrade material is adequate for providing the design bearing pressure. Where required, subgrades should be overexcavated to achieve the required bearing conditions, but only as directed by the inspecting geotechnical engineer.

### **Drilled Caissons**

Caissons are recommended to support remaining loads and uplift forces that are not accommodated by existing MTA foundations or cannot be supported by new shallow foundations. Caissons consist of a permanent isolation steel casing drilled through bedrock below established MTA influence lines, with an uncased socket extending below the cased area. The casing and rock socket are filled with steel reinforcing and concrete. The annulus between the isolation casing and the rock must be filled with structural grout. Steel reinforcing may consist of rolled steel sections, built-up plate steel shapes, or rebar cages. Caissons

develop axial load capacity through a combination of peripheral shear resistance between the concrete and rock, and end-bearing on the rock.

The caissons anticipated to support the 55 Hudson Yards overbuild have rock sockets varying from 5 to 6-ft in diameter. The caissons are proportioned assuming an allowable side shear of 200 psi for compression and 100 psi for tension. These values assume rock meeting NYCBC Class 1c or better.

#### Bond Breakers

Bond breakers will be necessary for all caissons located within the 1H:2V influence zone of the MTA No. 7 Line Extension tunnels to limit imparting new loads on these structures. Means to achieve a bond breaker include coating the permanent isolation casing with materials such as bitumen, Slickcoat™, plastic tape or sheathing, etc.

#### Drilling Methods

Caissons are expected to be drilled entirely through bedrock. Because the rock is expected to be hard, using a down-the-hole hammer or cluster drill to advance the rock socket will likely be required. The caissons should be flushed using water or compressed air (or other approved methods) upon completion of drilling rock sockets to remove all debris accumulated on the bottom of the rock socket. Cleaning of the bottom of the rock socket is critical for caissons designed with end-bearing, and proper cleaning must be verified through inspection, as discussed below.

#### Rock Socket Verification

The NYC Building Code requires that all rock sockets be inspected to verify the quality of the bedrock before installing reinforcing steel and concreting. We recommend that verification be performed through video inspection with a down-the-hole camera, as opposed to entering the caisson.

#### Reinforcing Steel Splices

The NYC Building Code requires that core-beam splices be milled and full-depth welded. Given

that very large core beams will likely be required to achieve the highest load capacities, a waiver for this requirement may be necessary for constructability. The connection must be capable of achieving the necessary stress and moment transfer at the splice depth. We note that mechanical connections could inhibit constructability because the splice can require significant volume within the caisson section, thus potentially limiting concrete flow or installation of concrete tremie tubes. Deformed bar and threadbar cages can be spliced using staggered mechanical or lap splices. We recommend that only mechanical couplers capable of developing full capacity of the bars be used for tension elements.

#### Centralizers

All reinforcing steel must be centered within the caisson. Where rebar cages are implemented, centralizers should be spaced no more than 10 feet on center. Steel core beams should be provided with at least one centralizer at the base. The tops of core beams should be aligned at the top of the casing using either a template or by manual wedges.

#### Concrete Placement

Concrete should be placed as soon as possible following cleaning and within four hours of final inspection of the rock socket. If placement is delayed the socket must be reinspected. Concrete must be placed using tremie methods, and must be performed in a continuous operation. Concrete must consist of a flowable mixture and must remain workable throughout the anticipated duration of the pour.

#### **Cellar Floor Slab**

We recommend that the cellar slab be designed as a pressure slab and that it be designed to accommodate hydrostatic pressure corresponding to at least el 113.4 ft. The pressure slab should be keyed into the foundation walls and be cast with integral water-stops and grout tubes. The pressure slab should be waterproofed as per the recommendations presented herein.

### **Permanent Below Grade Walls**

Permanent below grade walls should be designed to resist static earth pressures, hydrostatic pressures, and foundation and surface surcharge loadings.

The lateral earth pressure diagram for static loading is presented in Figure 15. We recommend the permanent below grade walls in soil be designed using a triangular earth pressure distribution having an equivalent fluid weight of 55 psf per foot of depth above the design groundwater level, and 90 psf per foot of depth below the design groundwater level. Below the top of bedrock, a uniform pressure of  $10H$  psf added to a triangular hydrostatic pressure distribution of 62.4 psf per foot. Lateral pressures from surface surcharge loads should be added as a uniform soil pressure equal to one-half the vertical pressure applied over the first 15 ft of the wall. Surcharge loads from adjacent foundations should be added as a uniform lateral pressure equal to 0.15 times the vertical pressure. The lateral effect of foundation surcharge loads can be neglected at depths greater than 15 ft below the top of bedrock.

### **Waterproofing**

Waterproofing will be required for all below grade structure. Waterproofing must extend one foot above the design flood elevation along the west side of the site and should extend to grade or the top of structure elsewhere.

Waterproofing should consist of a membrane liner such as Grace Preprufe 300R, Bituthene System 4000 or equal and all products should be coordinated with the MTA. Coordination and special detailing will be required to tie dissimilar waterproofing systems together. We note that the No. 7 Line Extension Project utilized PVC waterproofing liners. We recommend that all products and detailing be reviewed by the MTA prior to construction. Additionally, we recommend that the waterproofing manufacturer consult on areas where dissimilar waterproofing materials are to be implemented.

We recommend that a warrantee be obtained from the manufacturer and installer to cover materials and workmanship; only certified installers should be used to perform the work. Detailed daily inspections must be performed to document any damage resulting from the contractor's activities. Repairs should be made as soon as possible. A representative of the

manufacturer should perform a final inspection where possible and approve all work prior to casting concrete.

We recommend that water-stops and grout injection tubes be provided at all concrete joints located below the static water table in addition to the waterproofing membrane.

## **GENERAL CONSTRUCTION RECOMMENDATIONS**

The following sections present recommendations relative to below grade construction within the site.

### **Temporary Excavation Support**

Temporary soil excavation support will be required along the north and west side of the site. Previously installed temporary soil excavation support located along the northern side of the site will require removal if it is found to be in conflict with the new construction. The southern and eastern side of the excavation will adjoin existing No.7 Line station and ventilation structures. Boring data indicates soil depths ranging from about 20 to 45 ft along the north side of the site and less than 10 feet along the west side of the site. We anticipate all soil will be removed within the footprint of the below grade structure to achieve proposed grades.

Solider pile and lagging walls with tie-backs are considered feasible along the north side of the site. Driving of piles will be difficult due to the presence of construction debris and boulders within fill soil; predrilling will likely be necessary to prevent twisting or vertical misalignment of the piles. Toe-pins or socketing of the piles into rock will be required to prevent sliding/kick-out.

Concrete piers and lagging walls are considered feasible along the west side of the site. Concrete piers can be hand dug in localized sheeted pits and constructed in a manner to limit impact on adjacent utilities. A tie-down rock anchor will be required to anchor piers to rock and resolve sliding and overturning forces.

Temporary soil excavation support should be installed such that a minimum of 5-ft wide lateral bench is provided between the soil support system and the edge of any rock excavation extending below the SOE.



Recommended lateral earth pressure diagrams for the design of temporary support are presented in Figures 16a and 16b. Temporary excavation support should be designed assuming the following minimum loading conditions:

- Free draining or dewatered walls should be designed using a uniform pressure distribution of  $28H$  psf, where  $H$  is the total height of the wall. Lagging walls can be assumed to be free draining provided louvers are installed.
- Walls which are not free draining or are not dewatered should be designed using a uniform pressure of  $28H$  psf above the design groundwater level; below the design groundwater level these walls should be designed using a uniform pressure of  $28H$  psf added to a triangular hydrostatic pressure of 62.4 psf per ft, where  $H$  is the total height of the wall.
- Lateral pressures from surface surcharge loads should assume roadway vehicular loading. Surface surcharges should be added as an inverted triangle having a maximum pressure at the ground surface equal to one-half of the vertical surface load (minimum 600 psf). Lateral surcharge pressure can be reduced to zero at depth of 15 ft below ground surface.

#### Rock Stabilization

Stabilization of excavated rock faces may be required. As discussed earlier, four predominant joint sets, faulting, and high horizontal stresses have potential for creating local instability within the exposed rock faces. We recommend that geologic mapping be performed during excavation to identify areas requiring stabilization. Adequate stabilization should be installed prior to advancing the excavation. In addition, exposed rock faces should be inspected regularly to evaluate the necessity for additional stabilization.

All rock stabilization measures should be designed by the Contractor's professional engineer. Particular stabilization measures will depend on local rock conditions encountered. We expect that stabilization would typically consist of rock bolts. The Contractor should evaluate the need specific stabilization during construction and should provide vigilant observation during excavation to identify areas of potential instability.

## **Temporary Control of Groundwater**

Temporary dewatering will be required to control groundwater inflow and surface water runoff within the excavation. The results of our investigation indicate that groundwater is likely perched atop rock throughout much of the site. We expect that the site can be dewatered during excavation using gravel filled sump pits and sump pumps.

The Contractor's dewatering system should be adequate for maintaining a "dry" site during normal operating conditions and should be capable of handling temporary high volume (flush) flows which may present themselves during excavation. All groundwater discharged from the site into NYC sewers will require temporary dewatering permits from the NYCDEP.

## **Excavation**

### Soil

All soil is expected to be removed from within the building footprint. Soil may be excavated using conventional methods. All soil must be handled in accordance with the approved remedial action plan and will require on-site environmental characterization to meet requirements of the selected disposal facility. Obstructions such as boulders or remnant foundations may be encountered along the northern area of the excavation and may require larger demolition equipment for removal. Excavation should progress such that the existing MSE wall is not destabilized. Soil should be removed from the site in uniform horizontal lifts such that the MSE wall is deconstructed in a controlled fashion.

### Rock

Up to 12 feet of bedrock excavation is anticipated to achieve proposed grades. As indicated earlier, rock within the site consists of granite, granitic gneiss, schistose gneiss, gneissic schist, and pegmatite; the majority of the rock encountered within the borings performed within and adjacent to the site was of a granitic nature. The granitic rock is generally extremely hard and will likely prove difficult to excavate via mechanical methods (hoe-rams, splitting, etc.).

Rock excavation within the site will require careful removal techniques because of the close proximity of existing structures. The bedrock was generally high quality and hard (Class 1a and

1b) and will likely be difficult to excavate, requiring rock chipping and splitting techniques. Channel drilling is recommended around the perimeter of the proposed cellar excavation and within elevator pits to minimize rock overbreak during subsequent chipping and splitting work. Channel drilling consists of overlapping drill holes such that a continuous channel is constructed along the excavation line. Because of the close proximity of adjacent MTA structures, blasting operations to remove the bedrock will likely not be permitted. To limit vibrations and assist in excavation, mechanical splitting techniques or chemical splitting agents should be considered.

Ground vibrations should be limited to a maximum of 2 inches per second as measured at the closest adjacent structure. Audible air-overpressure should not exceed 94 dB at the perimeter of the site to comply with NYCDEP construction noise mitigation rules. Monitoring should be performed using arrays of seismographs capable of measuring both vibration and air-overpressure. The Contractor's excavation design should be modified as necessary in the event of exceedances.

#### Rock Verification

The NYC Building Code requires that all rock subgrade be inspected to verify the quality of the bedrock before installing reinforcing steel and concreting. Rock subgrade must be inspected by a professional engineer to verify bearing capacity and ensure bearing rock has been adequately cleaned.

#### **Preconstruction Conditions Documentation and Monitoring**

We recommend that preconstruction conditions documentation be performed for all surrounding structures about one month prior to commencing excavation. The purpose of these observations is to provide a photographic and/or video documentation representative of general existing conditions and identify obvious visual deficiencies. The preconditions observations should also identify areas requiring specific monitoring during construction. Structural integrity is not commonly addressed in such documentation. This baseline information is often critical in the event of future damage claims resulting from construction activities. We note that Amtrak, MTA and LIRR may require documentation of their adjacent facilities prior to construction.

A comprehensive monitoring plan is both warranted and recommended. Specific requirements for monitoring are likely to be imposed by governing agencies including the NYCDOT, MTA, Amtrak, and NYCDEP. Critical structures which require monitoring include:

- 1) the Eleventh Avenue Viaduct;
- 2) the Amtrak North Access Tunnel;
- 3) No. 7 Subway 34<sup>th</sup> Street Cavern
- 4) MTA 34<sup>th</sup> Street Entrance Substructure and Tunnels
- 5) MTA Ventilation Building

We recommend that a dialog be established with all governing agencies prior to construction to determine specific monitoring requirements.

The monitoring program will likely include borehole inclinometers, optical surveying, vibration monitoring, and tiltmeters, crack gages, etc. The monitoring plan is currently in development and will be submitted separated along with the Support of Excavation (SOE) drawings. Given the expected duration for excavation, consideration should be given to installing remote sensors capable of relaying data in real-time via wireless communications. The monitoring plan will address means and methods for measuring ground and structural deformation, and vibration levels. We recommend that all monitoring be performed by a third-party consultant independent of the Contractor; however, the Contractor should be allowed to perform additional monitoring. Monitoring should be performed throughout excavation and foundation construction.

### **Construction Quality Assurance**

Excavation and foundation work are subject to various Special Inspections as per the requirements outlined in Chapter 17 of the NYCBC and the Rules of the City of New York (RCNY). Construction activities that require geotechnical quality control inspections include installation of the footing subgrades, caisson foundations, excavation, SOE systems, backfilling, and compaction. This work must be performed under the inspection of a qualified geotechnical engineer. The inspecting engineer should be familiar with the subsurface conditions as well as the

proposed and existing construction onsite. We recommend that all inspectors meet the requisite qualifications outlined in 1RCNY 101-06. In addition, while not required by the NYCBC, we recommend that full-time inspection of waterproofing be performed to mitigate the potential for leaks resulting from damaged or improperly installed materials.

MTA representatives may require access during construction to perform independent inspections.

## **OWNER AND CONTRACTOR OBLIGATIONS**

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

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

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

## **LIMITATIONS**

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

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

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

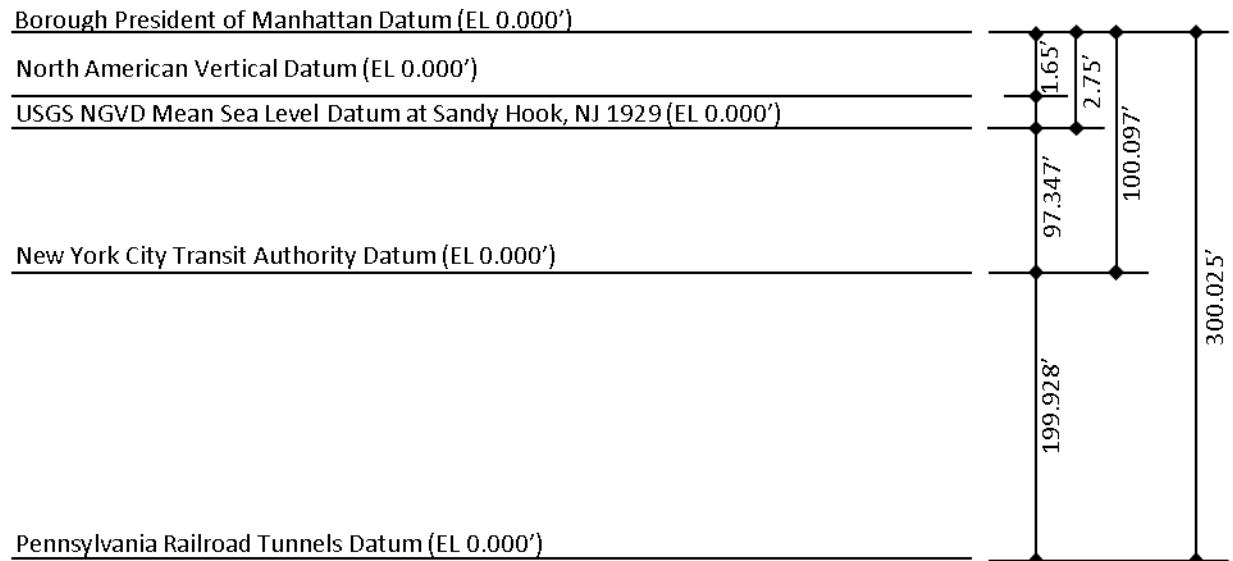
Environmental issues (such as potentially contaminated soil and groundwater) are outside the scope of this study and are being addressed in a separate study.

\\Langan.com\data\NY\data\7\170256702\Office Data\Reports\Geotechnical\2014-01-23-Georpt OHB.doc

**Table 1 – Elevation Conversions from NYCT Datum**

Borough President of Manhattan Datum (BPMD) <sup>1</sup>	BPMD = NYCT - 100.097 ft
United States Geologic Survey NGVD 1929 (USGS) <sup>2</sup>	NGVD = NYCT - 97.347 ft
Pennsylvania Railroad Tunnel Datum (PENN) <sup>3</sup>	PENN = NYCT + 199.928 ft
North American Vertical Datum 1988 (NAVD) <sup>4</sup>	NAVD = NYCT - 98.447 ft

- 1) New York Central Railroad (NYCRR), Eleventh Avenue Viaduct, Adopted Legal Street Grades (NYCDOT)
- 2) Mean Sea Level as measured at Sandy Hook, NJ
- 3) Amtrak North River and North Access Tunnels, LIRR Westside Storage Yard
- 4) United States Geological Survey - North American Vertical Datum



**Table 2: Measured Groundwater Levels**

Boring	Date	Depth to Water (ft)	Groundwater Elevation (ft)	Depth to Rock (ft)	Rock Elevation (ft)
L-SJ-1	8/28/07	64	65.4	23.5	105.9
	9/18/07	64.5	64.9		
	9/25/07	64.8	64.6		
L-SJ-2	11/28/07	5.0	105.8	9.5	101.3
L-SJ-3	9/25/07	31.1	103.8	30	104.9
L-SJ-4	9/7/07	29	104.5	37	95.5
	9/18/07	28.3	104.2		
	9/25/07	28.5	104.0		
L-SJ-5	9/18/07	27.4	105.1	41	91.5
	9/25/07	27.4	105.1		



**Table 3 – Packer Permeability Test Results**

Boring	Depth Upper Packer	Depth Lower Packer	Average Permeability	Notes
	feet	feet	cm/sec	
L-SJ-1	24	34	1.00 E-7	No water take at full pressure
	34	44	4.11 E-5	
	44	54	2.51 E-5	
	54	64	5.95 E-5	
	64	74	1.67 E-3	Could not develop test pressure, full flow from pump
	74	84	1.39 E-3	Could not develop test pressure, full flow from pump
	84	94	8.70 E-4	
	94	104	1.44 E-3	Could not develop test pressure, full flow from pump
	104	114	1.65 E-3	Could not develop test pressure, full flow from pump
	114	124	3.33 E-06	Minimal water take at max. pressure step
	124	134	1.00 E-7	No water take at full pressure
L-SJ-4	51	61	2.77 E-4	
	61	71	3.16 E-5	
	71	81	1.26 E-4	
	81	91	1.00 E-7	No water take at full pressure
	91	101	1.00 E-7	No water take at full pressure
	101	111	1.00 E-7	No water take at full pressure
	111	121	1.00 E-7	No water take at full pressure
	121	131	1.00 E-7	No water take at full pressure
	131	141	1.00 E-7	No water take at full pressure
	141	151	1.00 E-7	No water take at full pressure
	151	161	2.76 E-6	Minimal water take at max. pressure step