

GEOTECHNICAL ENGINEERING REPORT

for

**1568 BROADWAY
NEW YORK, NEW YORK**

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TABLE OF CONTENTS

	<u>Page No.</u>
INTRODUCTION.....	1
SITE DESCRIPTION.....	1
Adjacent Properties	1
Adjacent New York City Transit (NYCT) Structure	3
PROPOSED DEVELOPMENT.....	3
LOCAL GEOLOGIC BACKGROUND	5
SUBSURFACE EXPLORATION	6
Test Pits.....	6
Test Borings	7
Observation Wells	7
Borehole Geophysical Logging.....	8
SUBSURFACE CONDITIONS	8
Fill [Class 7]	8
Weathered Rock [Class 1d]	9
Bedrock [Class 1a to 1c]	9
Groundwater.....	11
SEISMIC EVALUATION.....	11
Liquefaction Analysis	12
EVALUATION.....	12
RECOMMENDATIONS	13
New York City Transit Requirements.....	13
Foundation System	14

Groundwater Control	19
Below Grade Walls	21
SITE PREPARATION AND CONSTRUCTION RECOMMENDATIONS	21
Temporary Support of Excavation	21
Earth Excavation and Retention	21
Rock Excavation and Reinforcement.....	21
Underpinning	24
Fill Material, Placement, and Compaction Criteria.....	26
Structural Stability Analysis of Adjacent Building Prior to Construction.....	27
Landmarks Preservation Commission Requirements	27
Pre-Construction Conditions Documentation and Monitoring During Construction.....	27
Construction Documents and Quality Control	28
Owner and Contractor Obligations	29
LIMITATIONS	29

LIST OF FIGURES

Figure No. 1	Site Layout
Figure No. 2	Proposed 1568 Broadway Building
Figure No. 3	Embedded Footing Diagram
Figure No. 4	Temporary Rock Bolts
Figure No. 5	Temporary Rock Anchors
Figure No. 6	Continuous Concrete (Underpinning) Piers
Figure No. 7	Support of Existing Foundation Walls

LIST OF TABLES

Table No. 1.....Fracture Sets and Foliation
Table No. 2..... Groundwater Monitoring Data
Table No. 3..... Building Code Seismic Design Parameters
Table No. 4..... Typical Tie-down Capacity in Rock

LIST OF DRAWINGS

Drawing No. 1.....Site Location Map
Drawing No. 2.....Historic Viele Sanitary and Topographic Map
Drawing No. 3.....Bedrock Geologic Map
Drawing No. 4.....Boring/Test Pit Location Plan
Drawing No. 5.....Subsurface Profile A
Drawing No. 6.....Subsurface Profile B
Drawing No. 7.....Subsurface Profile C
Drawing No. 8.....Subsurface Profile D
Drawing No. 9.....Borehole Discontinuity Stereonet Plot
Drawing No. 10.....Potential Weathered Rock Limits
Drawing No. 11.....Lateral Earth Pressure Diagram (Soil and Rock)
Drawing No. LS-1.....Langan Standards

APPENDICES

Appendix A.....Test Pit Logs and Photos
Appendix B.....Boring and Well Construction Logs
Appendix C.....Geophysical Borehole Logging Results
Appendix D NYC DOB TPPN #10/88

INTRODUCTION

This report was prepared by Langan Engineering, Environmental, Surveying and Landscape Architecture, D.P.C. (Langan) and presents our geotechnical engineering evaluation for the proposed project located at 1568 Broadway, in Manhattan, New York. The purposes of this report are to provide information on anticipated subsurface conditions, and recommendations for foundations and other geotechnical aspects of design and construction.

This report has been prepared based on information provided by Platt Byard Dovell White Architects, LLP (PBDW), and Severud Associates Consulting Engineers, P.C. (Severud). Ground surface elevations presented in this report were taken from a topographical survey prepared by Earl B. Lovell – S.P. Belcher, Inc., dated 14 December 2015. Elevations from the aforementioned survey are with respect to the North American Vertical Datum (NAVD88)¹. The general sidewalk grade fronting the site varies from about el. 48± to el. 50± NAVD88.

SITE DESCRIPTION

The 1568 Broadway site is located at the southeast corner of West 47th Street and Seventh Avenue, in the Times Square Theater District section of Manhattan. The site is currently occupied by the Landmarked² Palace Theater, and a 45-story hotel that was built both over the theater and to the east of the theater. A single cellar level is located throughout the site footprint that ranges in depth between about 13 to 15 feet below existing sidewalk grade for the theater and hotel, respectively.

A New York City Transit (NYCT) tunnel for the “N”, “Q”, and “R” subway lines is present below Seventh Avenue, directly to the west of the site. Existing structures are located immediately to the south and east of the site. The site is identified as Block 999, Lot 62, with a lot area of about 23,000 square feet. Figure No. 1 presents a general site layout diagram. A site location map is presented as Drawing No. 1.

Adjacent Properties

The southern property line of the site is bordered by a combination of 1560 Broadway, 155 West 46th Street, and portions of the 151 West 46th Street. The entire eastern property line is bordered by 150 West 47th Street. Our understanding of the foundations of the adjacent

¹ The North American Vertical Datum (NAVD88) is 1.1 ft above the U.S. Coast and Geodetic Survey Datum mean sea level at Sandy Hook, New Jersey, 1929, (NGVD).

² Based on the 4th Edition “Guide to New York City Landmarks” prepared by the New York City Preservation Commission, the Embassy Theater, 1556-1560 Broadway, was designated a landmark interior in 1987.

buildings is based on a combination of our recent work on these projects and our review of the Certificate of Occupancies (C/O) for each building posted on The New York City Department of Buildings (NYCDOB) website³; the following was noted:

1560 Broadway (Lot 3): is a 17-story commercial/office building with one-cellar level and was constructed circa 1925. The “L-shaped” building has a footprint of about 14,850 sq-ft, with about 60 feet of frontage along Seventh Avenue/Broadway and about 100 feet of frontage along West 46th Street. The interior of the building is landmarked⁴, and the cellar slab is located about 17-feet from sidewalk grade, corresponding to about el. 31±. Based on our previous involvement at this project site, we understand that the building is supported by shallow foundations bearing on bedrock.

155 West 46th Street (Lot 8): is a 5-story commercial/office building with one-cellar level and was constructed circa 2012. The building has a footprint of about 2,000 sq-ft, with a 20 foot frontage along West 46th Street. The building is joined with the 1560 Broadway building and serves as a lobby/access area for elevators into the 1560 building. The cellar slab within this building is located at about 10-feet below sidewalk grade, at about el. 40±. Based on our previous involvement at this project site, we understand that the building is supported by shallow foundations bearing on bedrock.

151 West 46th Street (Lot 9): is a 14-story mixed-use masonry structure that was constructed circa 1920’s. It is believed that this building has one below grade level. Existing foundation drawings for the building were not available at the time of this investigation; however given the depth to rock at the adjacent sites, we anticipate that the foundations are bearing on or near bedrock.

150 West 47th Street (Lot 54): is a 13-story mixed- use masonry structure that was constructed circa in 1979. It is believed that this building has one below grade level. Existing foundation drawings for the building were not available at the time of this investigation; however given the depth to rock at the adjacent sites, we anticipate that the foundations are bearing on or near bedrock.

³ New York City Department of Buildings website property profile and certificate of occupancy (www.nyc.gov)

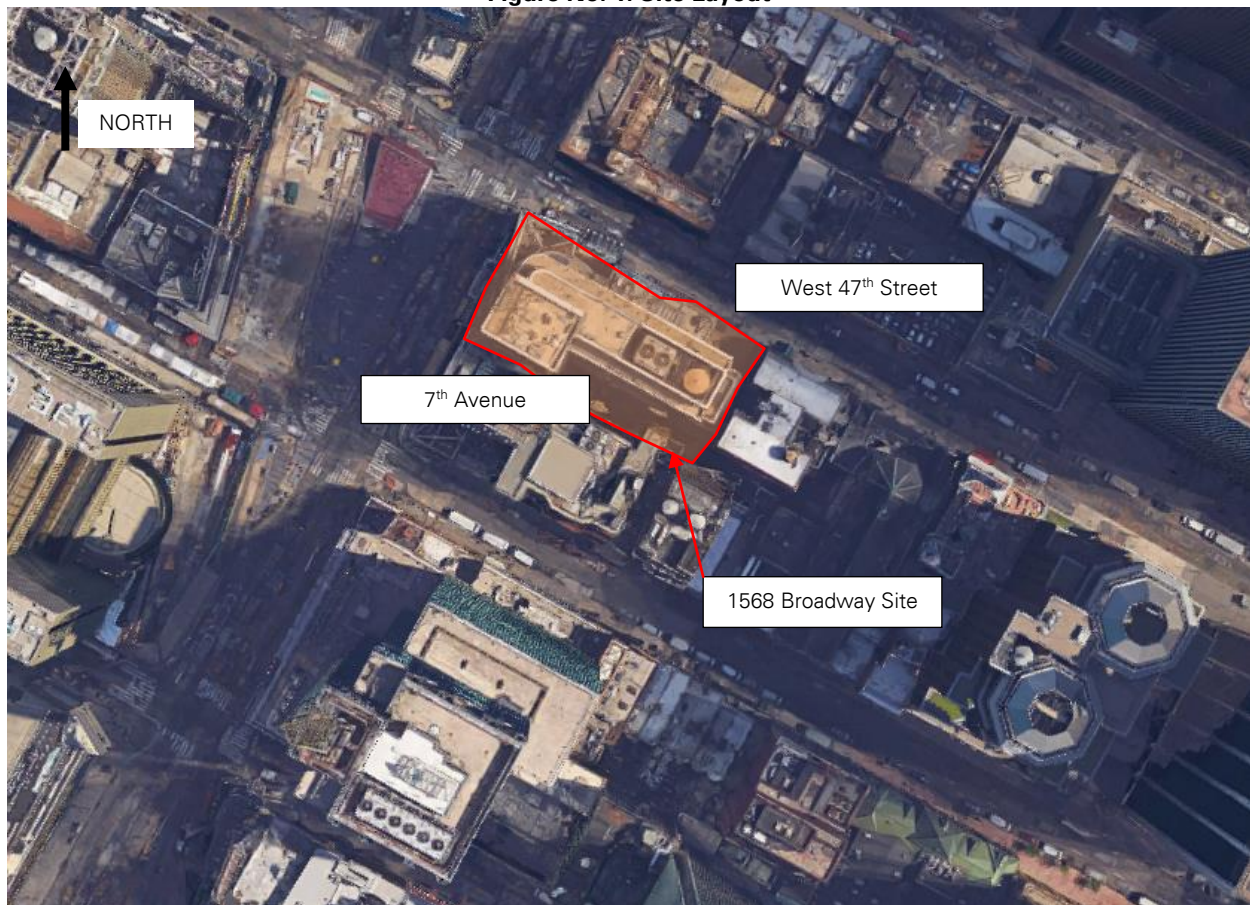
⁴ Based on the 4th Edition “Guide to New York City Landmarks” prepared by the New York City Preservation Commission, the Embassy Theater, 1556-1560 Broadway, was designated a landmark interior in 1987.

Adjacent New York City Transit (NYCT) Structure

As discussed herein, a New York City Transit (NYCT) subway structure is below Seventh Avenue to the west of the site. The NYCT operates and maintains a subway station at the corner of 47th Street and Seventh Avenue. The "N", "Q", and "R" trains run along tracks below Seventh Avenue and travel regularly in the north and south directions. The top of the subway structure is at about 4 feet (el. 44 NAVD) below sidewalk grade and the bottom of the subway structure is at about 24 feet (el. 24 NAVD) below sidewalk grade, with a base-of-rail of about el. 26±.

Due to the proximity of the site to an NYCT tunnel structure, design and construction of the proposed building must conform to the NYCT requirements and restrictions. The Department of Buildings will require NYCT approval prior to issuing building permits.

Figure No. 1: Site Layout



Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

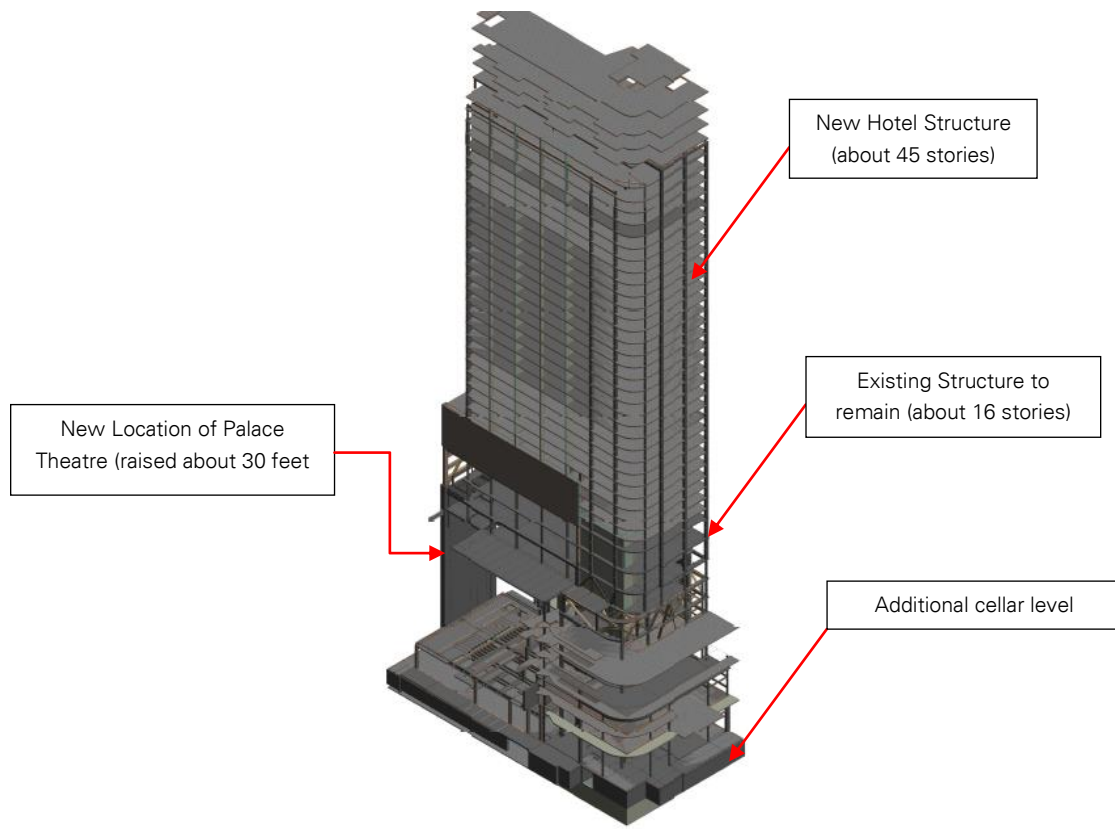
PROPOSED DEVELOPMENT

Our understanding of the proposed building layout and concept is based on discussions with

the project team and project drawings provided by PBDW and Severud. We understand the current scheme includes raising the existing Palace Theatre to be above the existing first floor elevation (to about 30 feet above the current location) and a reconfiguration of the hotel entrances, allowing for a major retail space fronting Broadway. A majority of the existing 45-story hotel structure will be demolished to accommodate the temporary bracing and shoring required to facilitate the raising of the theatre and the excavation below the theater. Specifically, at the completion of demolition, 8 stories of the hotel structure will remain on the east side, and 16 stories of the hotel structure will remain on the west side of the site. One additional sub-cellar is planned to be excavated below the existing cellar and a new foundation system will be installed to support the building expansion.

Once the excavation is completed and the theater has been raised, the hotel will be reconstructed back to the 45 floors it was previously, however with a greater floor to floor clearance. The new retail center will be located within the additional cellar level, with total depth of the new building ranging about 30 feet below sidewalk grade (about el. 18±). Figure No. 2 below presents an overview of the proposed development layout. Severud has provided typical column loads for the single cellar level scheme to be about 3,000 kips, with the loads for the super-columns on the order of about 18,000 kips. We have been informed that the foundations will exhibit localized uplift forces on the order of about 600 kips.

Figure No. 2: Proposed 1568 Broadway Hotel Building



Source: Drawing S-000 – Structural Overview, dated 18 May 2016 provided by Severud Associates Consulting Engineers

LOCAL GEOLOGIC BACKGROUND

The site is on Manhattan Island, which is 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 and schist. Bedrock is overlain by glacial and fluvial soil, as well as extensive fill. Although altered by urban development, original topography within Manhattan typically mimics the contours of the underlying bedrock.

According to Baskerville (1994), bedrock stratigraphy in the vicinity of the site is part of the Hartland formation, with rock of the Lower Cambrian (about 500 to 520 million years ago) to Middle Ordovician (about 461 to 472 million years ago) age and intrusive rock presumably of the Silurian age (about 416 to 444 million years ago), consisting of granite and megacrystalline pegmatite. The geologic map for the site vicinity is included as Drawing No. 3. Boundaries between the intrusive granite and Hartland formation rocks are not well-defined as evidenced by intermittent contacts and inclusions observed in rock cores throughout the area.

Generalized descriptions of the Hartland Formation mapped in the vicinity of the site are reported to be Interbedded units of (1) gray, fine-grained quartz-feldspar granulite containing

minor biotite and garnet; (2) fine-to-coarse grained, gray-to-tan weathering, quartz-feldspar-muscovite-biotite-garnet schist (mica schist); (3) 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.

We reviewed the historical “Sanitary & Topographical Map of the City and Island of New York” (Viele, 1865), identified a major stream channel had previously occupied the site, and that the site appears to lie on a former meadow. Attached as Drawing No. 2, is part of the Viele Map. A major stream channel often suggests deeper fills, a drop in the rock surface, and/or a thick weathered rock layer.

SUBSURFACE EXPLORATION

Our subsurface exploration program included (1) excavating six test pits, (2) drilling eleven test borings with in situ testing and sampling of soil and rock, (3) installing groundwater observation wells, and (4) performing borehole geophysical logging.

Test Pits

Six test pits (TP-1 through TP-6) were excavated adjacent to existing walls and columns within the cellar level of 1568 Broadway. These test pits were performed to identify the type, condition, material, dimensions, and underlying bearing material of the existing building foundations and perimeter walls. The test pits were excavated from 9 to 23 May 2016 by Urban Foundation Engineering, LLC (Urban) using hand tools under the full-time inspection of a Langan engineer.

In general, the existing foundations were noted to be shallow foundations (i.e., footings) bearing on bedrock, which was generally encountered immediately below the cellar slab (average depth of about 3 feet). The conditions encountered within each test pit were documented in the field with sketches and photographs, and those details are presented in Appendix A. The test-pit locations are shown on the subsurface exploration plan included as Drawing No. 4.

Test Borings

Eleven test borings (LB-1 through LB-11) were completed by Warren George Inc. (WGI) under full-time inspection of a Langan engineer. All borings were drilled between 10 May and 9 June 2016 using three limited-access electric drill rigs. The borings were drilled to depths varying between about 21 and 58 feet below the existing cellar level, corresponding to about el. 14± to -23±. All borings were advanced through the overburden using mud-rotary drilling techniques. Steel casing was advanced to the top of rock for supporting overburden during rock coring. The boring locations are shown on the subsurface exploration plan included as Drawing No. 4.

Standard Penetration Test (SPT) N-values⁵ were measured and typically obtained continuously for the upper 12 feet or to the top of rock, and at 5-foot intervals thereafter where soil was encountered deeper than 12 feet. Samples were retrieved using a 2-inch-diameter standard split-spoon sampler in general accordance with ASTM D1586. Recovered soil samples were visually examined and classified in the field in accordance with the Unified Soil Classification System (USCS), and the New York City Building Code (Building Code).

Bedrock was cored using NX-sized core barrel equipped with a diamond cutting bit in general accordance with ASTM D2113. Rock type, percent recovery (REC)⁶ and Rock Quality Designation (RQD)⁷, were determined for each core run. Soil and rock classifications, SPT N-values, and other field observations were recorded on the boring logs included within Appendix B.

Observation Wells

Groundwater observation wells were installed in completed borings LB-7, LB-8, and LB-10. The wells consisted of 10 feet of 2-inch diameter Schedule 40 PVC slotted screen and between 10 and 15 feet of solid riser pipe. For each well, the annulus around the slotted PVC pipe was backfilled with No. 1 filter sand to about 2 feet above the screen, then a 2-foot-thick bentonite pellet seal was placed and the remaining annulus was backfilled with soil cuttings. The well construction logs are included within Appendix B.

⁵ 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-outer-diameter split-barrel sampler 12 inches using a 140 pound hammer falling freely for 30 inches.

⁶ The percent recovery is the ratio of the length of rock recovered over the total rock core length, expressed as a percentage.

⁷ The RQD is defined as the ratio of the summation of each rock piece greater than 4 inches over the total core length, expressed as a percentage.

Borehole Geophysical Logging

Borehole geophysical logging, consisting of optical televiewer (OTV) and acoustic televiewer (ATV) logging, was conducted in five borings, identified as LB-2, LB-3, LB-7, LB-10 and LB-11 by Hager-Richter Geoscience, Inc. (Hager-Richter) on 31 May 2016.

The purpose of the borehole geophysical logging was to characterize in situ conditions of the bedrock, especially to determine depths and orientations of bedrock structures (i.e., fractures, joints, foliation, etc.) intersected by the boreholes. Geophysical results consisting of geophysical logs, bedrock structure statistics plots, tables of bedrock structures, and borehole geophysical logging figures are presented in Appendix C.

SUBSURFACE CONDITIONS

The general subsurface profile consists of uncontrolled fill underlain by weathered rock, overlying competent bedrock. Competent bedrock was observed to be encountered at relatively deep depths, about 30 to 53 feet below existing cellar grade, at boring locations LB-6, LB-7, LB-9, and LB-11. A detailed description of each layer encountered is provided below. Subsurface profiles A through D are shown in Drawing Nos. 5 through 8.

Fill [Class 7]⁸

A layer of fill was encountered in all of the borings immediately below the existing cellar slab. This layer is described as brown, coarse to fine sand with varying amounts of gravel, silt, brick, and concrete. The fill ranged in thickness from about 1 to 12 feet, and averaged about 5 feet thick. The areas of the localized deep fill were observed to be within close proximity to the existing hotel super-columns; the deep fills indicated that over excavation of weathered bedrock and/or bedrock was performed for the installation of the footings for the hotel super-columns and then backfilled.

Standard Penetration Test N-values in the fill ranged from about 2 blows per foot (bpf) to spoon refusal (more than 50 blows over six inches of penetration or 100 blows over one foot of penetration), with an average of about 26 bpf. Refusal occurred where obstructions such as coarse gravel, bricks, and cobbles were encountered. The fill is considered loose to dense and is classified as Building Code Class 7 material, Controlled and Uncontrolled Fills.

⁸ Numbers in brackets that follow the material designation indicate classification of soil and rock materials in accordance with the Building Code.

Weathered Rock [Class 1d]

A layer of weathered rock was encountered below the fill in 7 of the 11 borings drilled (top at about el. 24± to 35±); the weathered rock was not encountered in borings LB-3, LB-4, LB-5, and LB-10. The thickness of the weathered rock, where encountered, was typically about 6 feet; however, we note that in 4 of the 7 borings, the weathered rock was either interbedded within the parent rock (LB-6 and LB-11), or extend the full depth of the boring (LB-7 and LB-9). The weathered rock consisted of highly fractured bedrock, which often displayed the visual characteristics of the parent rock (color, grain size, etc.), but easily breaks apart under a small amount of pressure. Where encountered, the top of the weathered rock was observed at the depth of the existing cellar grades, about 14 to 25 feet below existing sidewalk grade.

In addition, a layer of weathered rock was encountered in borings LB-6 and LB-11 at a depth between about 27 to 33 ft below existing sidewalk grade (about el. 22± and el. 16±, respectively); the weathered rock was observed to be interbedded within competent bedrock. In LB-7, the weathered rock extended down a majority of the bore hole, which was 53 feet of the 58 feet cored. This was confirmed with the borehole geophysics.

For the weathered rock zone, the RQD varied between 0 and 33 percent, and averaged about 10 percent. The weathered rock generally consists of micaceous schist with varying proportions of gravel and silt. N-values within the weathered rock ranged from 7 bpf to spoon refusal, and averaged about 44 bpf. In general, N-values in the weathered rock layer increased with depth, eventually resulting in refusal as the split spoon approached the sound bedrock. The weathered/soft rock is classified as Building Code Class 1d material, Soft Rock.

In summary, we have observed areas where weathered rock is deep (borings LB-7 and LB-9) and where weathered rock seams are present within competent bedrock (boring LB-6 and LB-11). The stream that formerly occupied the site is likely associated with the locations and depths of the weathered rock zones.

Bedrock [Class 1a to 1c]

Below the weathered bedrock layer, where present, is competent bedrock which is characterized as grey mica schist with layers of pegmatite, quartz, and amphibolite. The rock fractures were fresh to highly weathered and had orientations from horizontal to about 60 degrees. The depth to bedrock ranged from about 5 to 53 feet below existing cellar grade and the corresponding top of bedrock elevation ranged from about el. 30± to about el. -19±.

Rock Characterization

Bedrock typically consists of schist with miscellaneous intrusions of pegmatite and granite. The schist is typically comprised of muscovite, biotite, quartz, feldspar, and garnet, and appears to be complexly folded with distinct foliation. Weathering of the bedrock was generally slightly weathered to fresh and fracture spacing was generally close (2.5 to 8 inches) to wide (2 to 5 feet). Isolated zones of highly fractured rock were observed within borings LB-6, LB-7 and LB-11, see Drawing No. 10 for the locations of highly weathered rock. However, the full extent of these highly fractured zones is unknown and these conditions should be considered possible across the site.

Rock-core recovery (REC) values varied between 75 and 100 percent and rock-quality designation (RQD) and averaged about 71 percent. The rock is generally highly competent, with about 70 percent of the RQD values exceeding 70 percent (fair to excellent quality, Building Code Class 1b or better). The bedrock is classified as Building Code Class 1a to 1c, Hard Sound Rock to Intermediate Rock.

Rock Discontinuity Orientations

Bedrock discontinuity orientation data was obtained from borehole geophysical logging consisting of optical televiewer (OTV) and acoustic televiewer (ATV) logging. An equal-area lower-hemisphere stereographic projection (stereonet) of the discontinuity data was developed using the Dips® software program from Rocscience, Inc., and is shown on Drawing No. 9. The stereonet displays a symbolic pole plot of the discontinuities overlain by a Fisher contour distribution. The planes representing the mean orientation of the discontinuities are also shown along with the proposed orientation of the excavation walls. The orientation and dip of discontinuities can vary based on the scatter within the data set.

The stereonet indicates the presence of a prominent fracture set and foliation and a secondary fracture set within the boreholes (displayed as pole clusters), which are summarized in Table 1 following:

Table 1 –Fracture Sets and Foliation

Discontinuity Set	Typical Dip Azimuth	Typical Dip Angle
Prominent Fracture (Set 1)	West to Northwest (250° to 330°)	Moderate to Steep (50° to 80°)
Secondary Fracture (Set 2)	South to Southeast (160° to 180°)	Shallow to Steep (20° to 60°)
Foliation	West to Northwest (270° to 330°)	Moderate to Steep (40° to 80°)

The foliation observed in the bedrock is near parallel to fracture set 1. The orientation of the two prominent fracture sets and foliation is in general agreement with observations made by Hager-Richter.

The data presented above indicates unfavorable conditions (major rock wedges daylighting into the excavation) may be encountered along the east and west sidewalls of the excavation. In addition, the potential for raveling may exist in isolated areas of highly weathered and highly fractured rock, specifically near the bedrock surface, along all sidewalls of the excavation.

Groundwater

Groundwater observation wells were installed in borings LB-7, LB-8, and LB-10 to about 30 feet below grade. Groundwater level was also measured in each borehole during drilling. The water level was measured at about 8.5 feet below existing cellar grade, corresponding to about el. 25± and el. 27±. Based on the subsurface conditions encountered, we believe that the groundwater is perched along the top of the competent bedrock surface. Our measured groundwater levels are included in Table No. 2 below. Details of the groundwater observation wells are presented in Appendix B.

Table No. 2 - Groundwater Monitoring Data

Boring (Ground Surface Elevation)	Date	Depth Below Grade (ft)	Approx. GW Elevation (ft)
LB-7 (OW) (el. 33.6)	06/01/2016	8.5	25.1
	06/02/2016	8.5	25.1
	06/06/2016	8.5	25.1
	06/07/2016	8.5	25.1
LB-8 (OW) (el. 36.2)	06/15/2016	9.2	27.0
LB-10 (OW) (el. 33.6)	06/01/2016	7.5	26.1
	06/02/2016	8.2	25.4
	06/06/2016	8.3	25.3
	06/07/2016	8.3	25.3

SEISMIC EVALUATION

This section presents the results of our seismic evaluation for the site relative to the provisions outlined in the Building Code. The proposed structure has been designated as Structural Occupancy Category III. Table No. 3 below provides our recommended parameters for use in seismic design of the propose structure.

Table No. 3 - Building Code Seismic Design Parameters

Seismic Design Parameter	Recommended Value	2014 NYCBC Reference
Mapped Spectral Acceleration for short periods (S_s)	0.281 g	Section 1613.5.1
Mapped Spectral Acceleration for 1-second period (S_1)	0.073 g	
Site Class	B	Table 1613.5.2
Site Coefficient for short periods (F_a)	1.00	Tables 1613.5.3(1) and 1613.5.3(2)
Site Coefficient for 1-second period (F_v)	1.00	
Design spectral response acceleration at short periods (S_{DS})	0.189 g	Section 1613.5.4
Design spectral response acceleration at 1-sec period (S_{D1})	0.049 g	
Seismic Design Category	B	Section 1613.5.6

Liquefaction Analysis

The seismic provision of the Building Code requires an evaluation of the liquefaction potential of sand, silt, and non-cohesive materials below the groundwater table and up to a depth of 50 feet below the ground surface. Since the lowest level of the building will be at or near bedrock, and the foundation elements will be bearing on sound rock, liquefaction need not be considered in foundation design.

EVALUATION

There are several geotechnical design challenges related to the subsurface conditions, foundation construction, and the adjacent buildings. The challenges include the following:

1. The excavation is planned to extend to a depth of about 15 feet below the existing cellar level. There are also localized elevator pit and hotel ejector pit sections that will be carried deeper into bedrock, up to 14 feet below the proposed sub-cellar level. A substantial part of the excavation will be within the sound bedrock with localized pockets of weathered rock. The excavation will require careful rock remove techniques while limiting vibration levels, and properly supporting the sides of the excavation (i.e., adjacent to streets, subway, adjacent structures, etc.) within both competent and weathered rock zones.
2. Groundwater was encountered at a depth of about 9 feet below existing cellar grade. We believe that the groundwater is perched along the top of the competent bedrock surface, which will need to be properly controlled during foundation construction, and accounted for with the structural design.

3. Unstable rock wedges may daylight requiring temporary support during excavation operations. Also, portions of the site down the center, exhibited areas of soft or weathered rock will likely require support and specific recommendations for new foundation elements.
4. Working within the existing building provides specific foundation challenges and limited choices for foundation support. Based on the results of our subsurface investigation, the existing building is supported by a shallow foundation system with variable bearing capacities. The shallow foundations consist of a combination of spread footings bearing on competent bedrock with allowable bearing capacities ranging from 40 to 50 tons per square foot (tsf) and wall footings bearing on weathered bedrock with an allowable bearing capacity of about 8 tsf. During the construction of new foundations or reinforcing existing foundations, special care must be exercised when working around the existing foundations. It is extremely important that the existing foundations not be compromised by the excavation or proposed construction of the new foundations.
5. Designing and installing new foundations in both competent and weathered rock zones.

Due to the complex nature of the theater lifting, demolition work, and excavation within an existing structure, we believe that it is imperative to have a concise set of plans that are well coordinated between the trades. Typically, demolition and bracing is handled separately from excavation and the new structure; however we recommend that this design work be integrated with the new building scope and theater raising.

Given the depth of the excavation and potential impact on NYCT and adjacent structures, the DOB and NYCT will be reviewing these procedures and design support before permits are issued.

RECOMMENDATIONS

The following provides our recommendations for the foundation system and other geotechnical-related design parameters including below-grade walls, groundwater control, and foundation support. As discussed herein, Severud has provided typical column loads for the single cellar level scheme to be about 3,000 kips, with the loads for the super-columns to be about 18,000 kips. In addition, a few local areas uplift will be acting upon the foundations, with a maximum uplift force of about 600 kips.

New York City Transit Requirements

The design and construction of the foundation system must consider the NYCT Subway structure beneath Seventh Avenue. NYCT regulations do not allow for construction of

foundations bearing within the limits of a theoretical influence line drawn from the base of a NYCT structure. Normally, NYCT regulations dictate that the theoretical line will be taken as 1 vertical to 1.5 horizontal for average soil conditions with water, and 1 vertical to 1 horizontal for average soil conditions without water. We have identified the NYCT theoretical slopes on our cross section shown on Drawing Nos. 7 and 8. The actual influence line will be identified after discussions with the NYCT, which is expected to occur during the design phase of the project.

In addition to the NYCT influence line, we have assumed a soil stability impact line from the base of the NYCT structure as a 1 vertical to 1 horizontal line going downward from that point. According to the soil stability impact line, the proposed foundations will not impact the NYCT structure along Seventh Avenue. Therefore, a shallow foundation element can be constructed outside the NYCT influence line for this project.

Once the architect and structural engineer have developed the building design, the project team will need to meet with the NYCT concerning the proposed design and construction. As indicated herein, and due to the complex nature of the project, NYCT will most likely require review of the demolition bracing, theater support and bracing, support of excavation and the foundation structural drawings as one package; and should be assumed to be submitted together for their review. The results of the meetings will be incorporated into the final foundation design.

Foundation System

As discussed herein, the proposed project includes a major retail expansion and reconfiguration of the hotel and Palace Theatre spaces. In addition, one sub-cellar is planned to be excavated below the existing cellar and a new foundation system will be installed to support the building expansion. We also anticipate that a series of temporary bracing and foundations will be required to support the existing foundations while the theater is raised and the site excavated.

The selection of the foundation type will be governed by the final structural loading on foundation elements, configuration of the proposed structure, economics, and scheduling considerations. Foundation alternatives are discussed below.

Shallow Foundations

Based on the subsurface conditions encountered the lowest cellar level will mostly extend into competent bedrock, with some portions of the site potentially impacted by localized areas of weathered rock. We anticipate the foundation system will primarily consist of shallow foundations (i.e., individual footings, wall footings, and mat foundations). Heavy loaded elements (shear walls, cores, etc.), located within weathered rock areas, may require support

from deep foundations or large mats, depending on structural criteria for allowable settlement.

Allowable Bearing Pressure

Bedrock was encountered above the proposed lowest level for the new building. The bedrock classification at and below the proposed foundation level was generally Building Code Class 1b (Medium Rock). Given the depth of the proposed excavation and the rock encountered at the site, we recommend the footings be designed with an allowable bearing pressure of 40 tons per square foot (tsf), corresponding to Class 1b rock.

However, as discussed herein, zones of weathered rock were observed at borings LB-6, LB-7, LB-9, and LB-11; see Drawing No. 10 for approximate areas of the deep weathered rock. As a result, additional analyses will likely be necessary, especially in heavily loaded areas, to evaluate foundation differential and total settlement. The settlement analysis would be performed after structural loadings and locations are further developed to finalize an alternate design such as:

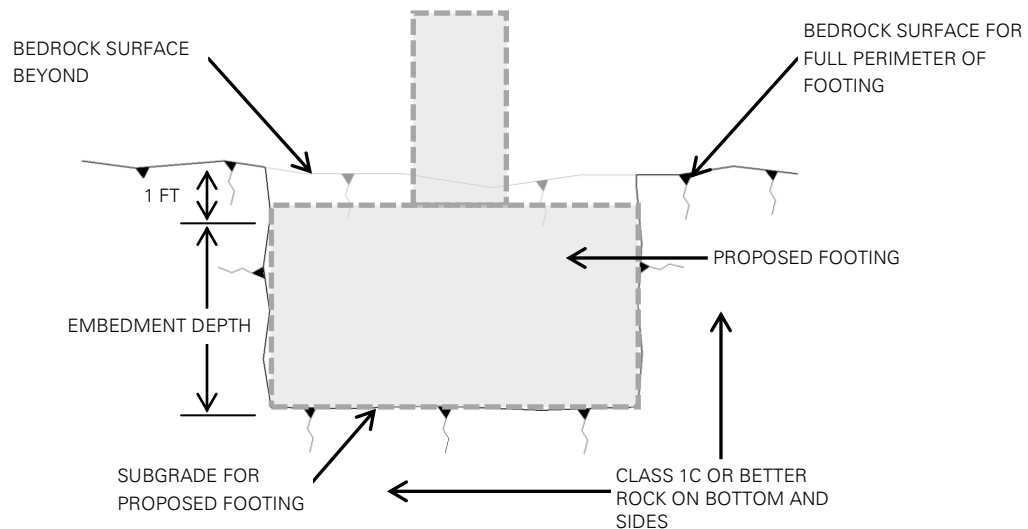
- 1) Footings/mats with an assumed allowable bearing pressure of up to 8 tons per square foot, corresponding to Class 1d rock.
- 2) Drilled caissons socketed into competent rock

The areas of potential weathered rock would also need to be verified in the field during excavation.

According to Building Code Section 1804, the design bearing capacity can also be increased when footings are embedded into the rock surface. The Building Code allows for an increase in bearing pressure within competent bedrock (Class 1c or better rock) of 10 percent for each foot of embedment, but no more than 200 percent of the basic maximum allowable bearing pressure. Although this approach could reduce footing size, excavation for the footings into bedrock will be time consuming and require much more effort from the contractor to be installed properly.

If the footings are planned to be embedded to achieve a higher allowable bearing capacity, the footings must be excavated within locally excavated pits extending to Class 1b or better rock, so the loaded area is below the rock surface and is fully confined by the adjacent rock mass. The adjacent rock mass above the bearing surface must be of the same quality or better. Figure No. 3 below presents a diagram showing the excavation for a footing embedded in rock.

Figure No. 3. Embedded Footing Diagram



Note: Not to scale. Shown for concept only.

Quality of rock within each footing bearing area should be uniform to prevent eccentric loading of the footing. Details pertaining to excavation, excavation support, and preparation of subgrades are outlined in subsequent sections of this report.

Individual footings should be designed assuming a minimum width of 3 feet and continuous footings should have a minimum width of 2 feet for constructability. Design of mat foundations is usually an iterative process, and we will work with the structural engineer during the design development. A uniform modulus of subgrade reactions of about 1,500 and 500 pounds per square inch per inch are recommended for the initial design iteration for Class 1c or better rock and Class 1d rock, respectively.

Settlement

Settlement of the foundations will be the result of elastic compression of the rock mass. Based on our experience from similar sized buildings and rock conditions, we would anticipate that settlements of individual footings and wall footings bearing on weathered rock (Building Code Class 1d) may be as much as 1 inch, possibly higher, depending on the structural loads, while settlements of mat foundations bearing on competent rock (Building Code class 1c or better rock) may be on the order of about 1/4 inch. As discussed herein, settlements are dependent on the structural loadings, bearing area, and quality of the bedrock and thus foundation types and parameters will need to be further evaluated once the structural system is finalized.

Lateral Resistance

For shallow foundations bearing directly on rock, lateral shear from wind and earthquake loads can be resisted by friction on the bottom of the footing. We recommend an ultimate frictional coefficient of 0.70 for mass concrete poured on clean sound rock and a minimum factor of safety of 1.5 when evaluating frictional resistance. If a concrete rock sealant (or mud mat) is used, which is common practice during rock subgrade preparation, friction between the footing bottom and the subgrade should be neglected.

If additional resistance is needed, lateral loads can also be resisted by embedding the 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.

Uplift Resistance

Shallow foundations bearing on rock cannot provide sufficient uplift resistance. If required, we recommend that uplift forces be resisted by post-tensioned tie-down anchors socketed into bedrock (see a subsequent section of this report).

Subgrade Preparation

The top of rock elevation is expected to vary somewhat over relatively short distances. Sloping top of rock and zones of weathered or fractured rock may require local deepening of the footing excavations to achieve the allowable bearing pressure. The foundation subgrades should be level and clear of standing or frozen water, debris, or other deleterious materials. The Building Code requires that a Professional Engineer licensed in the state of New York inspect and approve foundation subgrades prior to placement of concrete, to verify that the subgrade material is adequate to provide the recommended allowable bearing pressure. We recommend that foundation subgrade be inspected by Langan to verify bearing capacity and that footing bottoms have been adequately cleaned.

Deep Foundations

Due to the areas of weathered rock extending to depths between about 9 and almost 40 feet, possibly deeper, below the proposed sub-cellar (see Drawing No. 10), drilled caissons⁹ may be

⁹ A caisson consists of open-ended steel casing sections (unbonded zone) drilled into place down through the overburden soils and extending to the required bearing stratum. An uncased hole is drilled into the rock, down from the unbonded zone, to create the bond zone. After drilling, the entire shaft is filled with cement-grout and steel reinforcement. The structural load is transferred from the mini-caisson to the rock through the bond zone.

required to obtain the required capacity, if differential settlement becomes an structural issue. . . Drilled caissons would be socketed into the rock and rely on side adhesion in the bedrock and that the end bearing capacity of the caissons be neglected for design. The recommended allowable shear resistance corresponding between concrete and Class 1c rock or better rock is 200 pounds per square inch (psi) for compression loads and 100 psi for tension loads. Because of the presence of the fractured/weathered rock, the allowable shear resistance would be reduced, possibly to 50 - 75 psi, where weathered rock layers are expected. Further analyses, including additional field investigations, maybe required to evaluate the shear capacity, once the structural system is finalized.

In general, we recommend the top 2 feet of the rock socket (bond zone) is neglected due to the normally fractured and uneven nature of the bedrock surface encountered. In accordance with Section 1810.7.7 of the Building Code, compressive load tests are not required to be performed on the caissons if rock quality is verified by a Professional Engineer through rock socket video observation.

Permanent Tie Down Anchors

Depending on the building design and dead weight, permanent tie-downs anchored into the rock may be required to resist uplift or overturning forces. Double corrosion protected threaded bars meeting ASTM A-22 requirements can be used for this application. If tie-down anchors are to be used, then we recommend, Grade 150 threaded bars for reinforcement steel. The free stress (unbonded) length should be at least 15 feet long, but additional length may be required to increase rock stability. Global failure of the bedrock must be considered when designing the location and free-length of the anchors. The following table presents the estimated design capacity for three anchor diameter sizes of varying length of bonded lengths assuming competent rock.

Table No. 4 – Typical Tie-down Capacities in Rock

Anchor Diameter	Reinforcement^a	Structural Capacity^b	Bond Length Required^c
(inch)		(kips)	(feet)
4	1 # 14 Bar	200	15
6	1 # 20 Bar	440	20
8	1 # 24 Bar	630	22

a: Grade 150 steel assumed

b: Calculated as $0.6 * [\text{yield strength of steel}] * [\text{cross-sectional area of steel}]$

c: Assuming an allowable peripheral shear of 100 psi obtained with a factor of safety of 2, length required to achieve structural capacity

The design capacity of the anchors should be evaluated once the building design loads and locations are finalized. In areas of weathered rock, the tie-down capacities would be less, possibly one-half the capacities indicated in Table No. 4. Ten percent of the tie-down anchors should be performance tested (creep) to 133% of their design load. The remaining anchors should be proof tested to 133% their design load. Successfully tested anchors should be locked-off at a load exceeding the sum of the design load, seating loss, and long term losses.

Groundwater Control

During our subsurface exploration, the static groundwater level was measured between about el. 25 and el 27, which assumed to be perched on the bedrock surface. We recommend that the permanent design groundwater level be taken at about 4 feet above the highest measured groundwater level, or at about el 31. The elevated design groundwater level should help reduce risks associated with periods of prolonged precipitation, sewers backing up (i.e. clogged or antiquated sewer lines), and/or utility breaks.

Temporary Groundwater Control

Based on our experience on nearby projects, and verified with the groundwater observation wells installed on site, the static groundwater level is close or perched on the top of bedrock. If groundwater is encountered during construction, we expect that it could be controllable with gravel filled sumps and sump pumps, to allow for subgrade preparation and foundation construction.

In order to dispose of groundwater from the excavation into the sewers, The New York City Department of Environmental Protection (NYCDEP) will require laboratory tests of the groundwater to determine water quality prior to allowing construction water to be pumped into the sewers. A groundwater sample can be taken during the subsurface investigation for laboratory testing. We understand that the NYCDEP has a limit of 10,000 gallons per day to be pumped into the sewers, and if this limit is exceeded, then the NYCDEP will charge a fee on the amount of water being pumped. As discussed herein, a boring and well program is needed to study pumping requirements.

Slab Support

We recommend that the lowest floor slab be constructed as a structural slab, designed to resist the uplift of hydrostatic pressure head acting on the bottom of the cellar slab. Alternatively, the lowest floor slab could be designed as a slab-on-grade with an underslab drainage system provided that the lowest slab is isolated from the potentially higher groundwater levels.

Isolation can be achieved by keying the foundation walls a minimum of 2 feet into Building Code Class 1c or better rock to serve as a cutoff, including the perimeter foundation walls. We recommend that a minimum 12-inch thick layer of 3/4-inch, natural crushed stone be placed beneath the lowest floor slab. It should be noted that based on our experience, foundation contractors are reluctant to excavate a vertical "key" into rock, due to the time and expense required to chip/drill vertical faces in very hard sound rock. Therefore, if a water cut-off scheme is selected, the contract documents and pre-bid meetings should carefully present this requirement of vertical excavations in sound rock along the entire site perimeter.

Waterproofing

Given the proposed use of the below-grade space, we recommend that all the below-grade slabs and walls be fully waterproofed with a membrane-type waterproofing such as Preprufe and Bituthene products by Grace.

For all waterproofing applications, diligent inspection of waterproofing materials is critical, especially during placement of reinforcement for the slabs and foundation walls. The vertical waterproofing should be protected with a rigid barrier to prevent damage during backfilling. The substrata to receive horizontal waterproofing should be a 3-inch-thick lean concrete working surface (mud mat). Holes or rips in the waterproofing membranes should be repaired in accordance with the manufacturer's recommendations.

In addition to waterproofing, the foundation walls should have a drainage panel such as Hydroduct 220 by Grace, or an approved equivalent. The drainage panel will provide protection for the waterproofing membrane and minimize water from accumulating against the foundation walls. The use of bentonite waterproofing or negative side crystalline waterproofing is not recommended.

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 waterproofing work. Diligent protection and quality control is critical in producing a final product that limits the potential for seepage. Detailed daily inspections should be performed to document any damage resulting from the contractor's activities. Repairs should be made as soon as possible. Repairs should be made as soon as possible and should be made per the manufacturer's recommendations. A representative of the manufacturer should perform a final inspection and approve all work prior to concrete pours.

Storm Water Detention

The NYCDEP requires a certain amount of on-site detention of storm water for those projects within the Borough of Manhattan. Thus, consideration for roof detention of water and/or

detention tanks should be included in the building design by the architect and the MEP.

Below Grade Walls

Below-grade walls will be subjected to lateral pressures caused by soil loads, surcharge loads, and groundwater (hydrostatic) loads. In the static loading condition, lateral pressures from earth, groundwater and surcharge loads should be considered. The static loading condition will consist of a triangular earth-pressure distribution having an equivalent fluid weight of 60 pounds per square foot per foot of depth (at rest condition) of soil above the groundwater table, and add 63 psf when below the design groundwater level of el. 27±. Lateral pressures caused by a surcharge load have a uniform rectangular distribution equal to 50 percent of the vertical surcharge pressure. Dynamic lateral loads need not be considered because the site is Seismic Site Class B (Building Code Section 1802.2). Our recommended earth-pressure diagram is presented in Drawing No. 11.

SITE PREPARATION AND CONSTRUCTION RECOMMENDATIONS

The following sections discuss typical geotechnical related construction issues including rock excavation, backfill, excavation support and foundation underpinning.

Temporary Support of Excavation

Based on the provided project information, the proposed development is planned to excavate within the existing building to construct a new sub-cellar level for the full building footprint. The contractor must take appropriate measures to stabilize the work area and prevent lateral movements of the adjacent areas during the excavation. The excavation may consist of both soil and rock removal.

Earth Excavation and Retention

The perimeter of the site is surrounded by existing vaults (along the north and west of the site) or by adjacent buildings (to the south and east of the site). It is believed that the adjacent buildings are all founded near or directly on bedrock; therefore, given the presence of a thin soil layer above the bedrock, the support of the perimeter excavation along areas where soil is encountered will most likely consist of continuous concrete (underpinning) piers, see Figure No. 6 below. The rock directly below the existing wall of the adjacent building should be carefully supported, especially if poor quality fractured and/or weathered rock are present.

Rock Excavation and Reinforcement

Based on the current project information, the proposed foundation construction will require a one level deep excavation, about 17 feet below existing cellar grade, for a total depth of about

30 feet below existing sidewalk elevation, corresponding to el. 18±. Rock excavation around the site perimeter will require very sensitive and careful removal techniques due to the close proximity of the adjacent buildings to the south and east, hard rock, possibly street utilities surrounding the site. The bedrock will likely be difficult to excavate, requiring rock chipping and splitting techniques. Channel drilling is recommended, especially around the site perimeter near existing structures, to limit rock overbreak during subsequent chipping and splitting work. Line drilling can be considered adjacent to streets. Line drilling consists of closely spaced drilled holes (say 4 to 6 inches) along the line of the excavation. Channel drilling consists of overlapping drill holes such that a continuous channel is constructed along the excavation line. Due to the close proximity of adjacent structures and the NYCT subway structure below Broadway/Seventh Avenue, blasting operations to remove the bedrock will likely not be permitted.

Given the bedrock discontinuity orientation data obtained from the borehole geophysical logging, there is indication of the presence of one prominent fracture set and foliation and one secondary fracture set. Preliminary kinematic analyses were performed to determine the potential movement of rock blocks by planar-sliding and wedge-sliding failure. The analysis indicates that the excavation stability is more favorable along the north and south site perimeters, and has the potential to be unfavorable along the east and west site perimeters. Therefore, reinforcement for the facades of the rock excavation will be required, and are outlined in the section provided below.

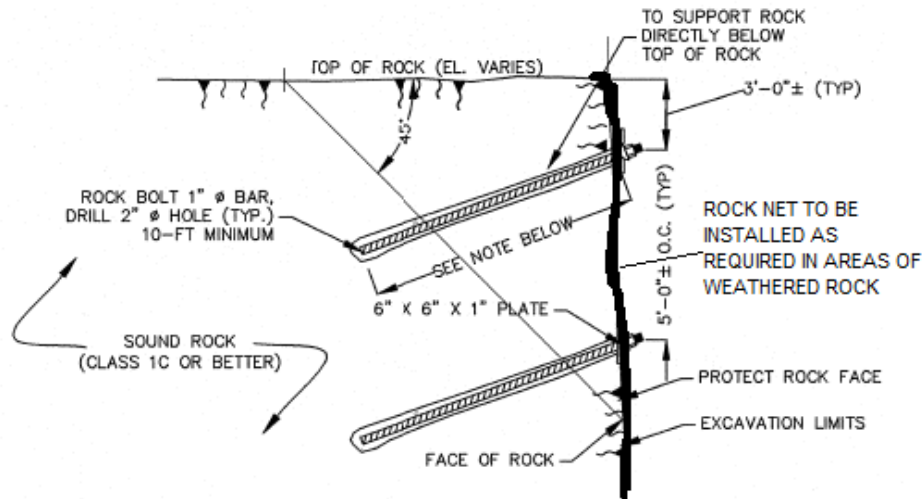
Exposed rock faces should be examined geologically and mapped as the excavation proceeds. Loose, fractured, or soft rock should be secured with mesh and/or excavated and replaced with concrete; rock bolts or pre-stressed rock anchors should be used to secure any potentially unstable rock masses.

Temporary Rock Reinforcement

The temporary rock reinforcement shall consist of a combination of rock bolts and anchors that should extend a minimum of 5 feet beyond potential failure planes of rock wedges; see Figures No. 4 and No. 5 below. Based on the borehole geophysical analysis performed, we expect that temporary rock bolts and anchors will be required along all façades of the excavation; specifically, along facades of the excavation where the adjacent building is not located. However, permission would be required from the adjacent property owners to allow the drilling and installation of temporary rock reinforcement underneath the adjacent buildings. The need for rock bolts and anchors, including spacing and length, must be determined by the Excavation Engineer in the field as excavation proceeds. In addition, for areas where weathered rock and

spalling are encountered, the rock facades may require additional stabilization (i.e. rock nets, mesh, or parging). Rock bench heights should be restricted to 10-foot maximum and stabilized with bolts, anchors, etc. before the next lower rock bench is excavated. A formal design should be provided by the contractor's professional engineer registered in the state of New York.

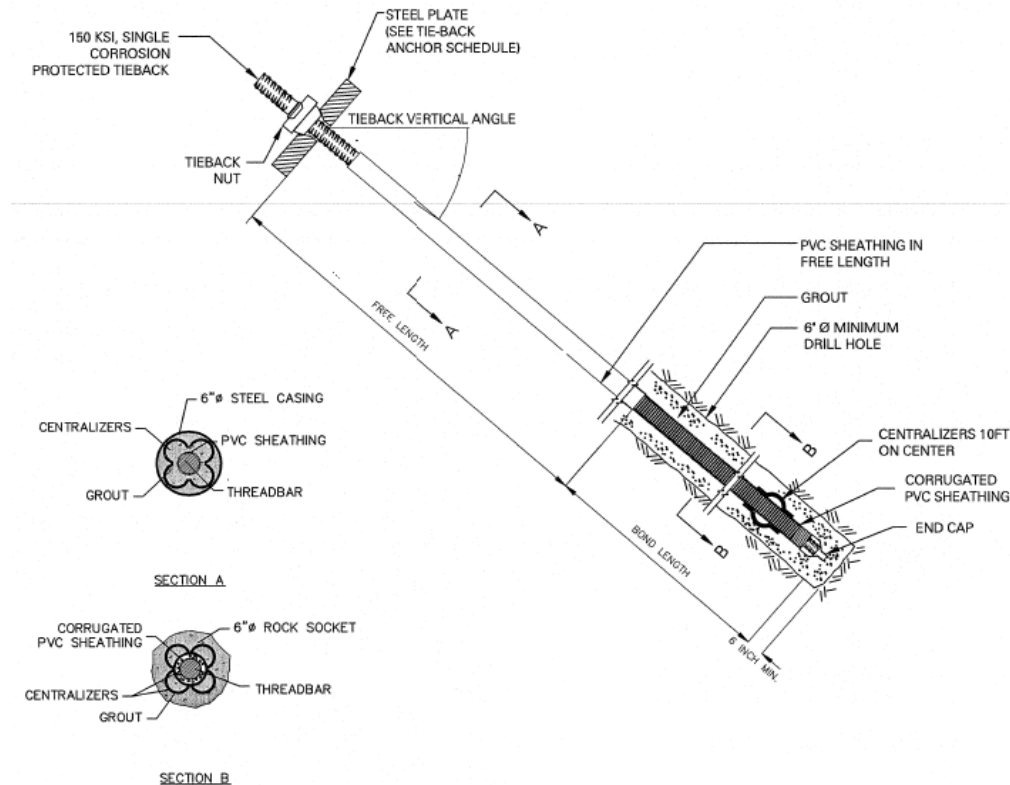
Figure No. 4: Temporary Rock Bolts



NOTES:

1. ROCK BOLT LENGTH MAY BE ADJUSTED BASED ON FIELD CONDITIONS AT DIRECTION OF ENGINEER.
2. ROCK BOLT SPACING SHALL BE TYPICALLY 5-FT ON CENTER; HOWEVER, SPACING MAY BE ADJUSTED BASED ON FIELD CONDITIONS

Figure No. 5: Temporary Rock Anchors

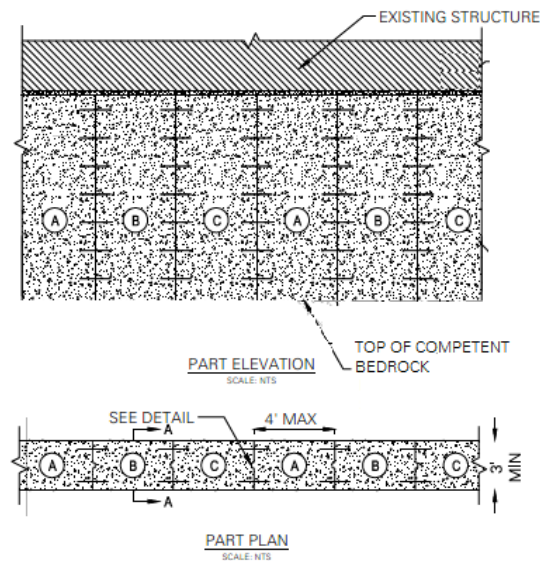


Due to the presence of the NYCT subway structure, we strongly recommend that the excavation support system be extremely stiff in order to provide proper lateral support. The subway structure must be restrained from moving laterally and/or settling. The proposed excavation support system will have to be reviewed and coordinated with the NYCT. There must be careful consideration given to instrumentation monitoring of the NYCT structure during excavation and construction.

Underpinning

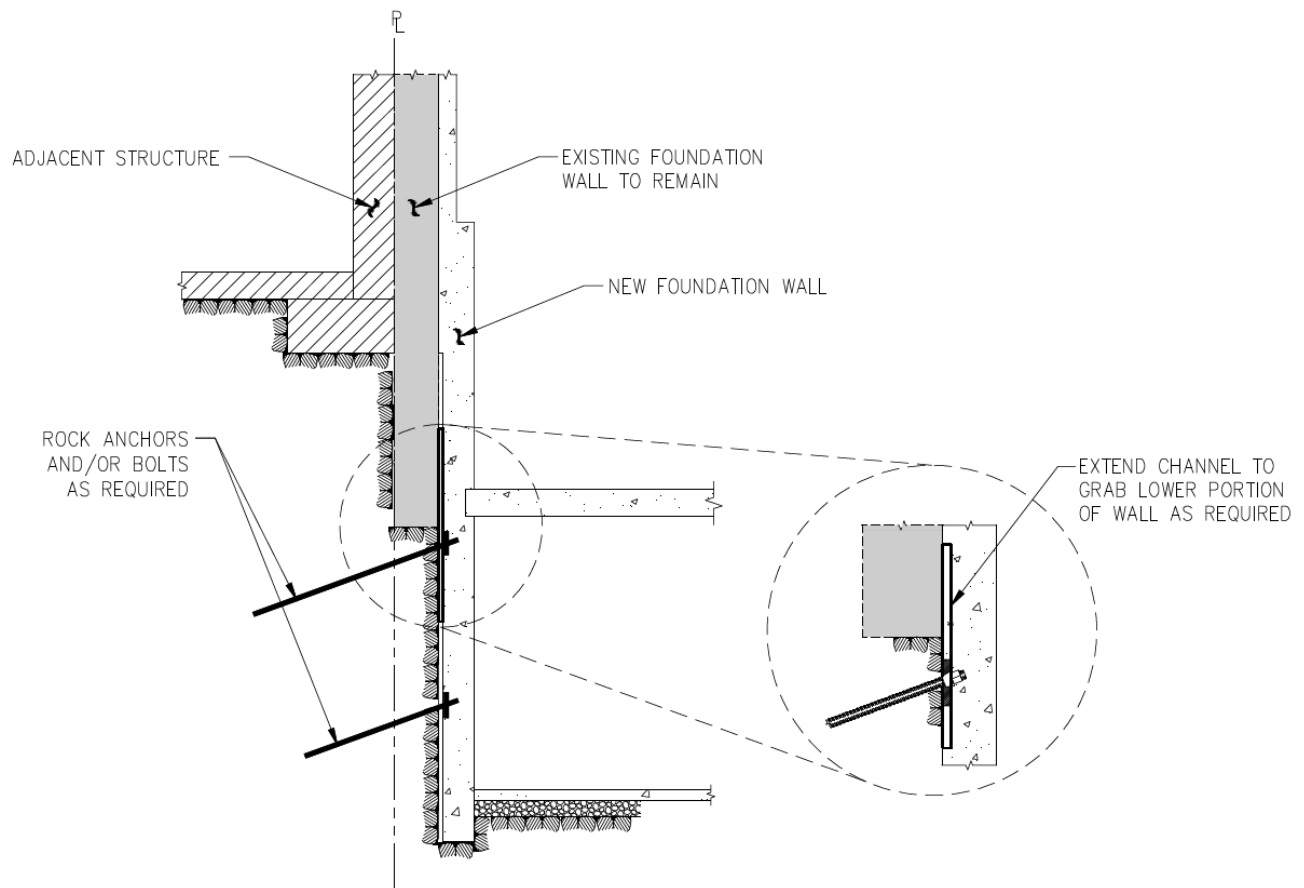
Based on review of existing information, we anticipate that the foundations for the adjacent buildings bear above the proposed foundations. We anticipate that the adjacent buildings are bearing on or near bedrock. Therefore, underpinning is expected to be relatively limited, but if poor or fractured rock is encountered, the poor rock will need to be removed and replaced with concrete in sections (underpinning) as shown in Figure No. 6.

Figure No. 6: Continuous Concrete (Underpinning) Piers



We understand the existing foundation walls are intended to be left in place along the western property line and the western portion of the southern property line (limits of the existing hotel tower), with a new foundation wall to be constructed inboard. Supporting the underlying bedrock below the existing foundation wall and adjacent buildings will be critical. The method selected for supporting the underlying bedrock will be based upon whether permission is granted by adjacent property owners to drill underneath their property. At this time, we believe the underlying bedrock can be supported with a combination of pre-stressed rock anchors and/or bolts for the areas where permission is granted (as described above), and an internal bracing system (i.e. walers, rakes, etc.), where permission may not be granted. A schematic illustrating the rock stabilization is shown below in Figure No. 7. A survey of all adjacent cellar slabs and walls is required by the DOB for underpinning, sheeting, and shoring design.

Figure No. 7: Support of Existing Foundation Walls



Note: Not to scale. Shown for concept only.

The existing foundation walls or the adjacent buildings surrounding the site must not be undermined by the proposed excavation. Measures should be taken to prevent raveling of soil or moving of bedrock wedges beneath the adjacent structures (foundation and slab elements). Underpinning should be designed by the contractor's professional engineer licensed in the state of New York.

Fill Material, Placement, and Compaction Criteria

Any material used for backfill around foundations and walls should consist of controlled fill as defined by the New York City Building Code. Controlled fill should consist of sand, gravel, crushed stone, crushed gravel or a mixture of these and must be free of organic, frozen and other deleterious materials. The top layer of landscaping material should be in accordance with City of New York Parks & Recreation requirements. The fill should have a maximum particle size not greater than 2 inches and have less than 10% by dry weight passing a No. 200 sieve. The structural fill should be compacted to at least 95% of the material's maximum dry density,

as determined by the Modified Proctor Compaction Test (ASTM D1557). The existing fill material may be used, provided it meets the gradation requirements discussed above. The use of recycled concrete aggregate, or the byproduct of blasting/tunneling (commercially known as mole rock), is not recommended for backfill.

Fill should be placed in uniform 12-inch-thick loose lifts. In restricted areas where only hand-operated compactors can be used, the maximum lift thicknesses should be limited to 6 inches. Lightweight compaction equipment should be used adjacent to subgrade walls. The appropriate water content at the time of compaction should be plus or minus 2 percentage points of optimum water content as determined by the laboratory compaction tests of the proposed fill material. No fill should be placed on areas where standing water is observed or on frozen subsoil areas.

Structural Stability Analysis of Adjacent Building Prior to Construction

We recommend a structural stability analysis to be performed on the adjacent buildings to the south and east, to evaluate the existing structural conditions of the building, prior to construction. Specifically, the results of the structural stability analysis will allow for a better understanding of which method would be a feasible option for bracing the building during excavation of the site.

Landmarks Preservation Commission Requirements

The adjacent 1560 Broadway building (about 180-foot frontage of the southern property line as well as the existing Palace Theatre within the 1568 Broadway site) have interior landmarks. General procedures for avoiding damage to Landmark Structures and buildings in historic districts are outlined in The City of New York Department of Buildings Technical Policy and Procedure Notice (TPPN) #10/88, "Procedures for Avoidance of Damage to Historic Structures," (June 6, 1988). TPPN #10/88 defines adjacent properties as being within 90 feet of the site where work is being performed. The monitoring requirements of adjacent properties includes measuring peak particle velocities, monitoring horizontal and vertical deflections of temporary retaining wall structures, monitoring horizontal and vertical deflections of adjacent buildings, groundwater table fluctuations, ground settlements, crack monitoring, preconstruction conditions documentation, and photograph documentation of adjacent buildings. A copy of TPPN #10/88 is attached as Appendix D.

Pre-Construction Conditions Documentation and Monitoring During Construction

A preconstruction construction documentation of all buildings, NYCT subway tunnels and utilities in nearby areas should be performed. The documentation would provide the owner and

foundation contractor and others with documentation of existing conditions in the event of a future damage claim. On the basis of this documentation, an observational and instrumentation program should be designed for monitoring the performance of adjacent structures and evaluating construction procedures.

During active excavation, a precise optical survey program should be implemented to monitor for vertical and horizontal movements of surrounding structures. The survey should be performed weekly, with measurements taken to the nearest 0.005 foot. The survey should be performed by a licensed surveyor. Criteria for allowable movements of structures should be finalized after a building pre-construction survey is completed.

Ground vibrations may develop during construction and excavation. Ground vibrations in nearby structures should be monitored during construction using seismographs. The ground vibrations should be monitored using a threshold-type seismograph capable of measuring to 0.02 inch.

In addition to survey points and seismographs, telltale crack reference gauges should be monitored within the adjacent structures. The crack gauges should be sensitive to 0.001 inch and should be read at least once daily.

We recommend that a monitoring plan and project specifications be completed before construction and excavation. These would detail the methods and equipment required for monitoring vibration and movement, and would provide movement criteria and requirements for frequency of readings and reporting. We anticipate that monitoring of the adjacent NYCT structures will be required.

Construction Documents and Quality Control

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

Excavation and foundation work is subject to various controlled engineering inspections as per the Building Code. A professional engineer familiar with the site subsurface conditions and design intent should perform the engineering inspection and testing of geotechnical-related work during construction. We recommend that Langan perform this work to verify proper implementation of our recommendations and to maintain continuity of our responsibility for this project. Construction activities that require quality-control inspections as required by the

Building Code include, but are not limited to, foundation subgrade inspection, excavation support installation, and compacted fill placement.

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. Construction activities that can alter the existing ground conditions such as excavation, fill placement, foundation construction, ground improvement, pile driving/drilling, dewatering, etc. can also potentially induce stresses, vibrations, and movements 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 given in this geotechnical engineering report are based on subsurface conditions observed through our field explorations, our company database, and the project information provided to us. The preliminary recommendations given herein are contingent upon one another and no recommendation should be followed independent of the others. Any changes should be brought to our attention so that we may determine how such changes may affect our recommendations.

The boring logs provide approximate subsurface conditions only at the indicated locations. Subsurface conditions between boreholes are inferred and may vary from conditions encountered at the boring locations. Groundwater conditions described refer only to those at