STRUCTURAL PEER REVIEW STATEMENT

This structural peer review and report is complete for the whole building, ~~or~~
~~For phase ____ of ____ phased submissions~~ based upon the current design phase.
Peer review will be updated as the design evolves.

Structural peer reviewer name: JEFFREY H SMILOW LICENSE #060728

Structural peer reviewer address: 228 EAST 45TH STREET - 3RD FLOOR
NEW YORK, NY 10017

Project address: 220 CENTRAL PARK SOUTH

Department application number for structural work: 121184592

Structural Peer Reviewer Statement

I (insert name) JEFFREY H SMILOW ***am a qualified and independent NYS licensed and registered engineer in accordance with BC Section 1627.4, and I have reviewed the structural plans, specifications, and supplemental reports for (Insert address and DOB application # for structural work)*** 220 CENTRAL PARK SOUTH - JOB #121184592 ***and found that the structural design shown on the plans and specifications generally conforms to the foundation and structural requirements of Title 28 of the Administrative Code and the 2008 NYC Construction Codes. The Structural Peer Review Report is attached.***

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

Name (please print) JEFFREY H SMILOW

Signature ____________ Date 4/17/14

PE/RA Seal (apply seal, then sign and date over seal)

JEFFREY H. SMILOW LICENSED PROFESSIONAL ENGINEER No. 060728 STATE OF NEW YORK

cc: Project Owner
Project Registered Design Professional

WSP

228 East 45th Street
3rd Floor
New York, NY 10017
Main: 212 687 9888
www.wspgroup.com/usa

Building Structures

17 March 2014

Martin Rebholz
Manhattan Borough Commissioner
New York City Department of Buildings
280 Broadway, 3rd Floor
New York, NY 10007

RE: 220 Central Park South – WSP No. B1490-008
Structural Peer Review

Dear Mr. Rebholz:

At the request of Vornado Reality, WSP has conducted a Structural Peer Review of the proposed building at 220 Central Park South, as required by New York City Building Code Section 1627. This report summarizes the extent and findings of our review.

We have reviewed the drawings listed in Appendix A, as well as the available geotechnical information, a copy of which is attached to this report as Appendix B.

Through our review, we have confirmed the following aspects of the structural design, as required by Section 1627.6.1:

- the design loads conform to the Building Code;
- the design criteria and design assumptions conform to the Building Code;
- the design conforms to the Structural Integrity provisions of the Building Code;
- the design properly incorporates the recommendations of the geotechnical engineer;
- the structure has a complete load path;
- based on our independent calculations of representative footings and foundation wall sections, we find that the design of the foundations have adequate strength;
- the structural plans are in general conformance with the architectural plans regarding loads and other conditions that affect the structural design; and
- the structural foundation plans are generally complete.

This review is consistent with the current phase of the project. It is understood that additional reviews will follow as the project advances into the future phases of design.

Accordingly, we find the structural design to be in general conformance with the structural design provisions of the Building Code.

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

Very truly yours,

WSP Building Structures

Jeffrey Smilow, P.E., F.ASCE
Executive Vice President
USA Director of Building Structures
New York State Registration No. 060728-1

cc: Mr. Mukesh Parikh, DCE via email: mukesh.parikh@de-simone.com

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APPENDIX A

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17 March 2014

220 Central Park South
Foundation Permit Application
Structural Peer Review

Appendix A: Drawing List

Reviewed Structural Drawings

Drawing Number	Drawing Title	Date
S-001	GENERAL NOTES, DRAWING INDEX, AND DESIGN CRITERIA/LOADS	01.29.2014
S-002	NYC TRANSIT NOTES AND INSURANCE CLAUSES	01.29.2014
FO-101	FOUNDATION PLAN AND MTA LAYOUT	01.29.2014
FO-102	FOUNDATION AND SUB CELLAR 2 FLOOR PLAN	01.29.2014
FO-103	MAT FOUNDATION LAYOUT AND REINFORCEMENT PLANS	01.29.2014
FO-103	MAT FOUNDATION LAYOUT AND REINFORCEMENT PLANS	01.29.2014
FO-104	MAT FOUNDATION LAYOUT AND REINFORCEMENT PLANS	01.29.2014
FO-111	TYPICAL FOUNDATION SECTIONS AND DETAILS	01.29.2014
FO-112	FOUNDATION SECTIONS AND DETAILS	01.29.2014
FO-113	FOUNDATION SECTIONS AND DETAILS	01.29.2014
FO-114	FOUNDATION SECTIONS AND DETAILS	01.29.2014
S-201	GROUND FLOOR PLAN	01.29.2014
S-411	SHEAR WALL PARTIAL PLAN	01.29.2014
S-412	SHEAR WALL PARTIAL PLAN	01.29.2014
SK-33.1	3RD FLOOR PLAN	02.04.2014
SK-33.2	4TH FLOOR PLAN	02.04.2014
SK-33.3	5TH FLOOR PLAN	02.04.2014
SK-33.4	6TH FLOOR PLAN	02.04.2014
SK-27.3	10TH – 38TH FLOOR PLAN	12.19.2013
SK-27.4	39TH – 63RD FLOOR PLAN	11.12.2013
SK-24.1	64TH FLOOR/MECHANICAL FLOOR PLAN	12.12.2013
SK-24.2	ROOF OVER MECHANICAL FLOOR PLAN	12.12.2013
SK-24.3	EMR/ROOF FLOOR PLAN	12.12.2013
SK-24.4	ROOF OVER TMD ROOM FLOOR PLAN	12.12.2013

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Architectural Drawings (For Reference Only)

Drawing Number	Drawing Title	Date
A-000	COVER SHEET	01.14.2014
A-001	LIST OF DRAWINGS	01.14.2014
Z-001	ZONING CALCULATIONS	01.14.2014
Z-002	SITE SURVEY & AIR RIGHTS PARCEL AREAS	01.14.2014
Z-003	GROSS FLOOR AREA & DEDUCTIONS	01.14.2014
Z-004	GROSS FLOOR AREA & DEDUCTIONS	01.14.2014
Z-005	REAR YARD ANALYSIS	01.14.2014
Z-006	HEIGHT & SETBACK TOWER COVERAGE ANALYSIS	01.14.2014
Z-007	STREETSCAPE REQUIREMENTS	01.14.2014
A-002	GENERAL NOTES, LEGENDS & ABBREVIATIONS	01.14.2014
A-003	HOUSING MAINTENANCE NOTES	01.14.2014
A-004	ACCESSIBILITY DETAILS	01.14.2014
A-005	DETAILS & BUILDING DATA	01.14.2014
A-010	CELLAR LIFE SAFETY PLANS & CODE DATA	01.14.2014
A-011	1ST & 2ND FLOOR LIFE SAFETY PLANS	01.14.2014
A-012	3RD & 4TH FLOOR LIFE SAFETY PLANS	01.14.2014
A-013	5TH & 6TH FLOOR LIFE SAFETY PLANS	01.14.2014
A-014	7TH & 8TH THRU 10TH FLOOR LIFE SAFETY PLANS	01.14.2014
A-015	11TH THRU 64TH FLOOR LIFE SAFETY PLANS	01.14.2014
A-016	64TH FLOOR THRU E.M.R. LIFE SAFETY PLANS	01.14.2014
A-102A	CELLAR FLOOR PLAN	01.14.2014
A-103	1ST FLOOR PLAN	01.14.2014
A-103A	1ST FLOOR PLAN ENLARGED	01.14.2014
A-104	2ND FLOOR PLAN	01.14.2014
A-104A	2ND FLOOR PLAN ENLARGED	01.14.2014
A-105	3RD FLOOR PLAN	01.14.2014
A-106	4TH FLOOR PLAN	01.14.2014
A-107	5TH FLOOR PLAN	01.14.2014
A-108	6TH FLOOR PLAN	01.14.2014
A-109	7TH FLOOR PLAN	01.14.2014
A-110	8TH FLOOR PLAN	01.14.2014
A-111	11TH THRU 38TH FLOOR PLAN	01.14.2014
A-112	39TH THRU 64TH FLOOR PLAN	01.14.2014
A-113	65TH FLOOR MECHANICAL LEVEL PLAN	01.14.2014
A-114	66TH FLOOR MECHANICAL LEVEL PLAN	01.14.2014
A-115	E.M.R./ROOF LEVEL PLAN	01.14.2014
A-116	ROOF OVER E.M.R. PLAN	01.14.2014

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Architectural Drawings (For Reference Only) - continued

Drawing Number	Drawing Title	Date
A-200	OVERALL BUILDING ELEVATIONS	01.14.2014
A-201	NORTH ELEVATIONS	01.14.2014
A-202	SOUTH ELEVATIONS	01.14.2014
A-203	EAST ELEVATIONS	01.14.2014
A-204	WEST ELEVATIONS	01.14.2014
A-210	SCHEMATIC BUILDING SECTIONS SHEET 1	01.14.2014
A-211	SCHEMATIC BUILDING SECTIONS SHEET 2	01.14.2014
A-250	MISCELLANEOUS DETAILS	01.14.2014
A-251	MISCELLANEOUS DETAILS	01.14.2014
A-300	PARTITION SCHEDULE	01.14.2014
A-301	MISC. DETAILS SHEET 1	01.14.2014
A-302	MISC. DETAILS SHEET 2	01.14.2014
A-303	MISC. DETAILS SHEET 3	01.14.2014
A-320	ELEVATOR PLANS & SECTIONS	01.14.2014
A-330	KITCHEN PLANS AND ELEVATIONS SHEET 1	01.14.2014
A-331	KITCHEN PLANS AND ELEVATIONS SHEET 2	01.14.2014
A-332	KITCHEN PLANS AND ELEVATIONS SHEET 3	01.14.2014
A-333	KITCHEN PLANS AND ELEVATIONS SHEET 4	01.14.2014
A-334	KITCHEN DETAILS SHEET 5	01.14.2014
A-340	RESID. BATHROOM PLANS AND ELEVATIONS SHEET 1	01.14.2014
A-341	RESID. BATHROOM PLANS AND ELEVATIONS SHEET 2	01.14.2014
A-342	RESID. BATHROOM PLANS AND ELEVATIONS SHEET 3	01.14.2014
A-350	EGRESS STAIR SECTIONS	01.14.2014
A-351	EGRESS STAIR SECTION	01.14.2014
A-352	EGRESS STAIR SECTION	01.14.2014

APPENDIX B

Interim Geotechnical Engineering Study

for

220 Central Park South
New York, New York

Prepared For:

Vornado Development
Vornado Realty Trust
888 Seventh Avenue
New York, New York 10019

Prepared By:

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

Arthur J. Alzamora, Jr, PE, LEED AP
Professional Engineer License No. 085419

Alan R. Poeppel, P.E.
Professional Engineer License No. 080220

20 December 2013
170177701

LANGAN

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Page No.

LIST OF DRAWINGS

APPENDICES

INTRODUCTION

This interim report was prepared by Langan Engineering, Environmental, Surveying & Landscape Architecture DPC (Langan) and presents a geotechnical evaluation for the new construction at 220 Central Park South, located in Manhattan, New York. The purpose of this study was to investigate subsurface conditions and develop preliminary recommendations for foundations and other geotechnical aspects of the proposed development. This report summarizes the results of our subsurface investigation and provides our engineering evaluation and recommendations. A summary of our findings and recommendations are presented herein.

The recommendations outlined herein have been prepared based on input and coordination with the design architect, SLCE Architects (SLCE), and the project structural engineer, Desimone Consulting Engineers, PC (Desimone). Elevations given herein are based on the air rights easement boundary survey prepared by New York City Land Surveyors, PC, dated 18 July 2013, and are relative to the Borough President of Manhattan Datum (BPMD)[1].

SITE DESCRIPTION

The site is located about mid-block, on the city block bordered by Central Park South to the north, Seventh Avenue to the east, Broadway to the west and West 58th Street to the south. The development site is comprised of four property lots and has a total footprint of about 27,610 square feet (SF.); extending from Central Park through to West 58th Street. The site is irregularly shaped, with a frontage of about 200 feet along West 58th Street, and a frontage of about 75 feet along Central Park South. A site location map is presented as Drawing No. 1.

Per the New York City Tax Maps, the combined lots are identified as block 1030, Lots 15, 16, 17, and 19. Tax Lots 15, 16, and 17 are currently open-air lots, consisting of demolition debris from the former buildings. We understand that the former buildings all had one below grade, typically extending about 10 feet below sidewalk grade. The exception is that Lot 19 was once occupied by a 20-story building that was demolished down to the ground floor level, and leaving the two basement levels, which extend to about 20 feet below sidewalk grade. We understand that the remaining portions of the building are planned to be demolished within the next few weeks. The general sidewalk grade along West 58th Street as well as along Central Park South is about el. 80.

[1] Elevations are with respect to the Borough President of Manhattan Datum (BPMD) which is 2.75 ft above the U.S. Coast and Geodetic Survey Datum (Mean Sea Level at Sandy Hook, New Jersey, 1929) (BPMD = USGS – 2.75)

Adjacent Structures

The project site is bordered by adjacent structures to the north, west and east of the site. Additionally there is a New York City Transit (NYCT) subway structure below Broadway and Seventh Avenue. The Broadway subway structure is located about 160 feet to the west, while the Seventh Avenue subway structure is located more than 200 feet to the east. Based on our review of The New York City Landmarks Preservation Commission (LPC) website[2], we noted that there are six landmarked structures within a 90 foot radius of site. The location of the adjacent landmark structures with respect to our site are shown on Drawing No. 2. The following provides brief information on the adjacent structures:

New York City Transit Subway Structure

The "1, 2 and 3" trains run along tracks below Broadway and travel regularly in the north and south directions. Based on our review of the NYCT drawings the portions of the subway structure are about 160 feet from the western property line of our site.

Since the site is located within 200 linear feet from a NYCT subway, The New York City Department of Buildings (NYC DOB) will require NYCT approval prior to issuing building permits. Given the minimal potential impact on the subway structure and recent experience one block to the south, we believe the NYCT will issue a "Letter of No Impact" for excavation and foundation construction for our site.

Adjacent Structures

- 210 Central Park South (Lot 39): is a 24-story commercial building that abuts a 100 foot portion of the eastern property line. We understand there is one basement level, which extends about 15 feet below the existing sidewalk grade. Specific building foundation information was not available at the time of this report.
- 230 Central Park South (Lot 48): is a 19-story residential building that abuts a 75 foot portion of the northern property line. The building is offset from the property line, with an open-air yard having a general grade of el. 74. We understand there is one basement level, which extends about 10 feet below the existing sidewalk grade. There is also a stone retaining wall that exists along the southern property line of this lot and borders our site. Specific building foundation information was not available at the time of this report.

---

[2] LPC website: http://www.nyc.gov/html/lpc/html/home/home.shtml

- 240 Central Park South (Lot 58): is a landmarked building, designated in 2002. The property consists of two main buildings, a 28-story and 15-story brick mixed residential and commercial buildings. The property abuts a 100 foot portion of the western property line, however the building is offset from the property line with an open air yard that has a general grade of el. 80. There is one basement level which extends to about 10 feet below the existing sidewalk grade. Specific building foundation information was not available at the time of this report.
- 222 Central Park South - The Gainsborough Studios - (Lot 46): is a landmarked building, designated in 1988. The property consists of a narrow double 8-story multi-family elevator building and abuts about 75 feet along the western property line. We understand there is one basement level. Specific building foundation information was not available at the time of this report.
- 215 West 58th Street - Fire House Engine Company No. 23 - (Lot 23): is a landmarked building, designated in 1968. The property consists of a 3-story brick fire house, and abuts 100 feet along the eastern property line. There is one basement level which extends to about 10 feet below the existing sidewalk grade. Specific building foundation information was not available at the time of this report.

Landmarked Structures (Not Adjacent)

As discussed herein, there are several landmarked building within a 90 foot radius of the project site, but not directly adjacent. These specific landmarked buildings are as follows:

- 1784 Broadway - Former United States Rubber Company Building - (Block 1029 Lot 53): was built between 1911 & 1912 and became a designated landmark building in 2000. The property is a 20-story limestone and brick commercial and office building, and is located to the south west of the site, across West 58th Street. There are two basement levels with vaulted space under West 58th Street and Broadway. Specific building foundation information was not available at the time of this report.
- 213 West 58th Street - Helen Miller Gould Carriage House - (Block 1030, Lot 24): was built between 1902 & 1903 and became a designated landmark building in 1989. The property consists of a 4-story limestone and load bearing brick public facility/institutional building that is located directly east of 215 West 58th Street, which abuts the eastern property line of the site. There is one basement level which extends about 10 feet below the existing sidewalk grade. Specific building foundation information was not available at the time of this report.

- 215 West 57th Street - The Art Students League of New York - (Block 1029, Lot 23): The Art Students League of New York was built between 1891 & 1892 and became a designated landmark building in 1968. The property is a 4-story stone public facility/institution building that has a 75 foot frontage on West 57th Street and on West 58th Street, and covers a lot area of about 15,000 sq. ft. The property is located across West 58th Street, south of the site. There is one basement level which extends to about 10 feet below the existing sidewalk grade. Specific building foundation information was not available at the time of this report.

PROPOSED DEVELOPMENT

Our understanding of the proposed project is based on discussions with the project team and a review of schematic level architectural plans. In general, the project pertains to the development of a 65-story, 950-ft-tall residential building and separate 15-story building. The 65-story tower will be constructed along West 58th Street portion of the property, while the portion of the site fronting Central Park South is planned to consist of a separate 15-story residential building. The current program calls for three below grade levels, to a total depth of about 50 feet below sidewalk grade, and extending to the full site limits, based on the 18 December 2013 architectural plans, this corresponds to about el. 30. At the time this report was written typical column loads have not been provided. We assume the building will be reinforced concrete slabs, columns, and shear walls. The total building load is taken at about 300 pounds per square foot (psf).

LOCAL GEOLOGIC BACKGROUND

The USGS "Bedrock and Engineering Geologic Maps of New York County and Parts of Kings and Queens Counties, New York, and Parts of Bergen and Hudson Counties, New Jersey", commonly known as the Baskerville Map, shows the site is underlain by mica schist of the Manhattan Formation, known locally as Manhattan Schist. The schist is a metamorphic rock formed under the effects of heat and pressure during burial within the earth's crust. The USGS bedrock map is shown as Drawing No. 3.

The predominant feature of the Manhattan Schist is the parallel alignment of the mineral grains, which is technically referred to as schistosity or foliation. The dominant regional structure of the Manhattan Schist in the site area is approximately parallel to the Manhattan Meridian North with local variations due to asymmetrical folding. The foliation of the Manhattan Schist generally dips steeply to the west or to the east, depending on local conditions of folding, although foliation or dipping to other directions or nearly horizontal foliation has been observed. The quality of the Manhattan Schist is generally fair to good, and tends to improve with depth.

However, localized shear zones and zones of decomposed rock are known to exist, sometimes to significant depths.

We reviewed the historical "Sanitary & Topographical Map of the City and Island of New York" (Viele, 1865); which shows that no identified stream channels previously occupied the site, and that the site appears to lie on a former meadow. Attached as Drawing No. 4, is a portion of the Viele Map.

2006 Test Borings (By Others)

A geotechnical engineering report was prepared by Future Tech Consultants of NY, Inc. (FTC), dated 28 August 2006. The subsurface investigation was performed within the existing building parking garage on lot 19. Six borings (identified as FTC-1 through FTC-6) were completed between 10 and 21 July 2006. Borings were drilled from the basement slab, but were advanced to depths ranging from about 18 to 27 feet below existing sidewalk grade. Five feet of rock was cored in each of the borings performed.

The FTC boring logs report that the basement and sub-basement slabs were typically 4 inches thick and underlain by a 8-inch feet thick gravel sub-base layer. The gravel sub-base in each boring was observed to bear on about a two foot layer of decomposed/weathered rock, which in turn was underlain by bedrock. Top of bedrock was observed to vary between either 13 feet or 22 feet below grade depending on the location within the existing building and concurrent basement levels therein. The bedrock at the site was reported as Gneissic Mica Schist. Copies of the FTC boring logs are included in Appendix D.

SUBSURFACE EXPLORATION

A total of Thirteen Langan borings are planned for the proposed development. Four borings were performed on Lot 17 between 18 and 30 December 2008, and were advanced to depths ranging from about 24 to 35 feet below existing sidewalk grade. Three borings were drilled on Lots 15 and 16, and completed in between 18 and 26 November 2013, and were advanced to depths ranging from about 48 to 50 feet below existing sidewalk grade (el. 30 to 32). All of the Langan borings were drilled at the site by Warren George, Inc., under the full-time special inspection of Langan. The remaining six borings will be drilled after the lot 19 building has been completed demolished.

A total of seven Langan test pits are planned for the proposed development. Four test pits identified as TP-1 through TP-4, were excavated to identify the type, depth, and bearing materials of the neighboring foundations and perimeter walls on 12 November 2013. The test pits were excavated under the full-time inspection of a Langan Engineer. The conditions encountered within each test pit were documented in the field with sketches and photographs, and are presented in Appendix A. test pits were excavated by J. Coffey Contracting, Inc., (Coffey) on 12 November 2013. The remaining three test pits will be excavated after the Lot 19 building has been completed demolished. A boring and test-pit location plan is included as Drawing No. 5.

Test Borings

All test borings were advanced through soil using mud rotary drilling techniques and a tri-cone roller bit with drilling fluid along with steel casing for providing soil support. Standard Penetration Tests (SPT)[3] (N-values) were measured and typically obtained continuously through the fill layer. Spoon 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 double tube core barrel. The core barrel was equipped with a diamond-cutting bit in accordance with ASTM D-2113 (Rock Core Drilling). Rock type, percent recovery (REC)[4] and Rock Quality Designation (RQD)[5], were determined for each core run. Soil and rock classifications, SPT N-values, and other field observation were recorded on the boring logs, which are included in this report as Appendix B.

Borehole geophysical logging, which consisted of optical televiewer (OTV) and acoustic televiewer (ATV) logging, was conducted in the recent borings (LB-6, LB-7 and LB-8) by Hager – Richter Geoscience, Inc. (Hager) on 27 November 2013. 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

[3] 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.

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

[5] 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.

are presented in Appendix C.

Observation Wells

To groundwater observation wells were installed at the site; one within boring LB-7 and one within boring LB-4. The groundwater observation well at LB-7 consisted of 10 feet of 1-1/4-inch diameter Schedule 40 PVC slotted screen and about 40 feet of solid riser pipe. The groundwater observation well at LB-4 consisted of 10 feet of 1-1/4-inch diameter Schedule 40 PVC slotted screen and about 23.5 feet of solid riser pipe. For both observation wells the annulus around the slotted PVC pipe was backfilled with No.1 filter sand to about 2 foot above the screen, then a 2-foot thick bentonite pellet seal was placed, and the remaining annulus was backfilled with soil cuttings. The groundwater observation well logs are included within Appendix B.

TEST PIT FINDINGS

The test pits are identified as TP-1 through TP-4. Test pit sketches and photographs, are provided in Appendix A as mentioned herein, and a detailed description of each test pit is provided below.

Test Pit TP-1 – West Side of Lot 15

Test Pit TP-1 was excavated adjacent to the brick wall along the western property line of the site. The brick wall appears to be within the site property. The test pit was excavated using a mini-excavator within vacant tax lot 15 from about sidewalk grade (el. 80). The plan dimensions of the test pit were about 7 feet in the north-south direction by about 13 feet in the east-west direction. TP-1 was excavated to a maximum depth of about 6 feet below grade.

The soil directly below grade consisted of miscellaneous fill and was underlain by a brown/grey silt and sand that extended down to bedrock. The brick wall appears to be a remnant of the former building that occupied the site and was supported by a rubble foundation, which extended about 20 inches into the test pit and about 2 feet below grade. The rubble foundation was observed to be bearing directly on class 1c or better bedrock. The bedrock was sloping downward from the base of the rubble wall as you moved east into site. Groundwater was not encountered during the excavation of TP-1. The test pit was backfilled with the excavated material up to existing grade.

Test Pit TP-2 – North Side of Lot 16

Test Pit TP-2 was excavated adjacent to the fencing along the northern property line of the site.

The test pit was excavated using a mini-excavator within vacant tax lot 16 from about sidewalk grade (el. 80). The plan dimensions of the test pit were about 10 feet in the north-south direction by about 10 feet in the east-west direction. TP-2 was excavated to a maximum depth of about 7 feet 6 inches below grade.

The soil directly below grade consisted of miscellaneous fill and extended down to the top of a demolished, 4 inch thick, concrete slab about 7 feet below grade. The northern perimeter fencing was observed to bear directly on a concrete retaining wall directly below grade that also extend to about 7 feet below grade to the top of slab. Beneath the concrete slab two steel pipes (possibly former utility pipes) were observed and the concrete slab was to bearing on the fill material. Bedrock and Groundwater were not encountered during the excavation of TP-2. The test pit was backfilled with the excavated material up to existing grade.

Test Pit TP-3 – North Side of Lot 17

Test Pit TP-3 was excavated adjacent to the brick and CMU wall along the northern property line of the site. The test pit was excavated using a mini-excavator within vacant tax lot 17 from about 10 feet to 12 feet below sidewalk grade (el.70 to 68 respectively). The plan dimensions of the test pit were about 5 feet in the north-south direction by about 5 feet in the east-west direction. TP-3 was excavated to a depth of about 1 foot below grade before encountering bedrock.

The soil directly below grade consisted of miscellaneous fill and within 1 foot below grade bedrock was encountered. In the location of the test pit was both a CMU wall and a brick wall. The CMU wall and brick wall, as well as sections of their concrete supports, were fully exposed prior to excavation of the test pit. The CMU wall was about 6 feet tall and was bearing on a 30 inch concrete wall and/or footing, which was observed to be underlain by a 3 inch thick concrete mud mat bearing directly on bedrock. The brick wall was about 10 feet 8 inches tall and was also founded on about a 18 inch thick concrete wall and/or footing which was observed to be bearing directly on bedrock. Groundwater was not encountered during the excavation of TP-3. The test pit was backfilled with the excavated material up to the existing grade prior to excavation.

Test Pit TP-4 – West Side of Lot 15

Test Pit TP-4 was excavated adjacent to the brick wall along the western property line of the site. The test pit was excavated using a mini-excavator within vacant tax lot 15 from about sidewalk grade (el. 80). The plan dimensions of the test pit were about 5 feet in the north-south direction by about 8 feet in the east-west direction. TP-4 was excavated to a maximum depth of about 5 feet below grade.

The soil directly below grade consisted of miscellaneous fill and was underlain by a brown/grey silt and sand with varying amounts of clay that extended down to bedrock. The west perimeter brick wall extended about 20 inches into the test pit directly below grade and about 1 foot below grade. The brick wall and adjoining brick step out was observed to be founded on a rubble wall that extended about an additional 1 foot below grade, and was bearing on highly weathered bedrock. The weathered bedrock extended through the bottom of the test pit about 5 feet below grade. Groundwater was not encountered during the excavation of TP-4. The test pit was backfilled with the excavated material up to existing grade.

SUBSURFACE CONDITIONS

The general subsurface soil profile consists of fill, underlain in some thin layers of sand, and extending down to bedrock. A detailed description of each layer encountered is provided below. Representative subsurface profiles are included as Drawing Nos. 6, 7, & 8.

Fill [Class 7][6]

A layer of fill was encountered in all the Langan borings immediately below the ground surface. This layer is described as a brown medium to fine sand with varying amounts of coarse sand, gravel, brick, and concrete. The fill extended from about 8 feet to 15 feet below grade (corresponding to el. 72 and el. 65 respectively). SPT N-values in the fill ranged from 2 blows per foot (bpf) to spoon refusal (more than 100 blows over 6 inches of penetration), with an average of about 39 bpf. Refusal occurred where obstructions, or the top of bedrock, were encountered. The fill is considered dense and is classified as New York City Building Code Class 7 material, Controlled and Uncontrolled Fills.

Sand [Class 3]

A layer of sand was encountered directly beneath the fill layer in four of the borings (LB-2, LB-4, LB-6 and LB-9). This layer generally consists of brown fine to medium sand with varying amounts of silt, fine gravel and micaceous fragments. SPT N-values within the sand layer ranged from 25 bpf to spoon refusal, and averaged about 61 bpf. Refusal occurred where the sand layer was adjacent to the top of bedrock. The layer of sand was given a USCS

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

classification of SP-SM and is classified as Building Code Class 3a material, Dense Granular Soil.

Weathered Rock/Soft Rock [Class 1d]

A layer of weathered/soft rock was encountered below the fill and/or sand layer in all but one Langan borings (LB-7), and also reported in two of the FTC borings (FTC-1 and FTC-6). The top of the weathered/soft rock layer was encountered at depths ranging from about 9 to 17 feet below the existing sidewalk grade (el. 71 to el. 63 respectively) and ranged in thickness from about 1 to 5 feet. No samples of the weathered rock were retrieved, but it is expected to generally consist of micaceous schist with varying proportions of gravel and silt. The weathered rock has the matrix of the parent rock, but easily breaks apart under pressure. N-values within the decomposed rock were at refusal. The decomposed rock layer is classified as Building Code Class 1d material, Soft Rock.

Bedrock [Class 1a to 1b]

Bedrock was encountered in all boreholes at site and consists of fresh to slightly weathered, gray garnet Gneissic Mica Schist with pegmatite, quartzite, and granite intrusions. The depth of top of rock varied from about 10 feet below grade in boring LB-1 to about 20 feet in boring LB-2A, corresponding to elevations of about el. 70 and el. 60, respectively. Rock-core recovery (REC) values ranged from about 23 to 100 percent, with an average value of 94 percent. Rock-Quality Designation (RQD) values generally ranged from about 10 to 100 percent, indicating generally poor to very good quality rock, however the average RQD was 84 percent and bedrock quality generally increases with depth. Two low RQD values of 10 and 12 percent were encountered from el. 66 to el. 61 in boring LB-2 and el. 66 to el. 62 in boring LB-3, which corresponds to Class 1d, soft bedrock. All other bedrock cores completed onsite consist of sound competent rock and are classified as Building Code as Class 1b to 1a, Medium Hard Rock to Hard Sound 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. 10. The stereonet displays a symbolic pole plot of the discontinuities overlain by a Fisher contour distribution. The plane representing the mean orientation of the foliation is also shown along with the proposed orientation of the excavation walls. The plane representing the prominent joint set is not shown because of the variability of the fracture dip angles. The orientation and dip of discontinuities can vary based on the scatter within the data set.

The stereonet indicates the presence of one prominent joint set and foliation within the boreholes, which are displayed as pole clusters. The prominent joint set exhibits a westerly to southwesterly dip azimuth of about 225 to 285 degrees (based on geodetic north) and a dip angle varying from 10 to 80 degrees from horizontal. The foliation observed in the bedrock exhibits a southwesterly dip azimuth of about 255 to 270 degrees and a steep dip angle varying from about 55 to 70 degrees from horizontal. The foliation within the bedrock is sub parallel to the prominent joint set. The orientation of the prominent joint set and foliation is in general agreement with observations made by Hager-Richter.

The data presented above indicates unfavorable conditions (rock wedges day-lighting into the excavation) may be encountered along the east and north sidewalls of the excavation. The potential for raveling may also exist in isolated areas of highly weathered and highly fractured rock near the bedrock surface.

Groundwater

The groundwater levels were monitored in the two observation wells during the investigation periods. Groundwater levels measured at the site are presented in Table No. 1 below.

Table No. 1: Groundwater Elevations

2008 Langan Boring LB-4			2013 Langan Boring LB-7		
Date	Depth (ft)	Approx. GW Elevation (ft)	Date	Depth (ft)	Approx. GW Elevation (ft)
12/30/2008	11.5	68.5	11/27/2013	30.0	50.0
12/30/2008	11.9	68.1	11/27/2013	14.6	65.4
-	-	-	12/3/2013	10.8	69.2
-	-	-	12/10/2013	11.0	69.0

Based on the subsurface conditions encountered, the groundwater levels measured at the site, and that fact that no groundwater was encountered within the test pits during excavation, we believe that the groundwater is perched on the top of the bedrock surface.

SEISMIC EVALUATION

This section presents the results of our seismic evaluation for the development site relative to the provisions outlined in the New York City Building Code. The following subsections provide recommended parameters for use in seismic design of the proposed structure.

Mapped Spectral Accelerations

Per section 1615.1 of the Building Code, the mapped maximum considered earthquake response spectra for the short period (S_s) is 0.365g and the 1-second period (S_1) is 0.071g.

Site Class

Section 1615.1.1 of the Building Code requires assignment of a Site Class. Based on the bedrock that will be directly beneath the proposed lowest level slab, the site is assigned to Site Class B (Rock Profile) in accordance with Table 1615.1.1 of the Building Code, and the site coefficients for short period (F_a) is 1.00 and for 1-second period (F_v) is 1.00.

Design Spectral Response Accelerations and Seismic Design Category

Design spectral accelerations were determined in accordance with section 1615.1.3 of the Building Code. The design spectral acceleration at short periods (S_{DS}) is 0.243g and 1-second period (S_{D1}) is 0.0473g.

Based on the above design spectral acceleration and the use group/occupancy category of the structure (assumed as Seismic Use Group I and Structural Occupancy Category II, to be confirmed by the Design Team), the correspondent Seismic Design Category (SDC) is identified as "B", in accordance with section 1616.3 of the Building Code.

Liquefaction Potential

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 within bedrock, and the foundation elements will be bearing on sound rock, liquefaction need not be considered in foundation design.

FOUNDATION DESIGN RECOMMENDATIONS

The following provides our recommendations for the foundation system and other geotechnical-related design parameters follow. At the time of this report typical column loads and uplift demands have not been developed.

Foundation System

The proposed three cellar levels will extend into the competent bedrock, based on all the borings drilled to date. Both the 65-story building and the 15-story building can be supported by individual piers or a continuous mat foundation bearing on bedrock. We recommend an allowable bearing pressure of up to 60 tons per square foot (tsf) for foundations bearing on the bedrock. For a 5 foot square footing, and a 1,500 ton column load, the estimated settlement for the individual pier is less than 1/2-inch. Settlements and associated subgrade modulus has to be calculated once loads are established. Individual footings should be designed assuming a minimum width of at least 3 feet and foundation wall footings should have a minimum width of at least 2 feet for constructability. For the combined footings or larger rafts under the tower core, the settlement and subgrade modulus should be determined by Langan once the loads have been established.

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 additional resistance is needed, lateral loads can also be resisted 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 or excavated slopes.

Subgrade Preparation

The top of rock elevation can vary considerably over relatively short distances. Sloping top of rock and zones of weathered or fractured rock may require local deepening of the footings and/or piers to achieve the 40 tsf allowable bearing pressure. The foundation subgrades should be level and clear of standing or frozen water, debris, or other deleterious materials. Compressed air should be used to clean all rock bearing surfaces. 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 foundation subgrade be inspected by Langan to verify bearing capacity and that footing bottoms have been adequately cleaned.

Permanent Tie-down Anchors

Uplift forces can be resisted by post-tensioned tie-down anchors socketed into bedrock. If tie-down anchors are to be used we recommend double corrosion protected Grade 150 threaded bars meeting ASTM A-22 requirements for reinforcement steel. The anchor bar diameter should not exceed 3 inches. The free stress (unbonded) length should be at minimum 15 feet long, but additional length may be required to transfer load below existing or proposed foundations and increase rock stability. The free stressing length of bar should be proportioned such that the dead weight of the engaged rock mass is greater than the individual anchor load or the sum of the group anchor loads. The engaged rock mass should be defined as the wedge formed by extending a plane 45 degrees from vertical from the midpoint of the bond length. Where multiple anchors are installed in a group, the rock mass wedge should extend upward from the outermost anchors, and the bottom of the wedge should be a level plane through the midpoint of the anchors. The anchor bond length should be proportioned using an allowable peripheral shear resistance in uplift of 100 psi[7]. The following Table No. 2 presents the estimated design capacity for three anchor diameter sizes of varying bond lengths.

Table No. 2: Typical Tie-down Capacity in Rock

Anchor Hole Diameter	Reinforcement[a]	Structural Capacity[b]	Bond Length Required[c]
(inch)		(kips)	(feet)
4	1 # 14 Bar	300	30
6	1 # 18 Bar	400	20
8	1 # 18 Bar	400	15

a: Grade 150 steel assumed
b: Calculated as 0.9 * [yield strength of steel] * [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 actual design capacity of the anchors should be evaluated once the building design loads are finalized. Ten percent of the tie-down anchors should be performance-tested (creep) to 133% of their design loads. 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

Groundwater Level

During our subsurface investigation, the static groundwater level was measured at a depth of about 11 feet below the ground surface, corresponding to an elevation of about el. 69. The groundwater level is judged to be perched on the bedrock surface. The perched water level is subject to the seasonal fluctuations and could rise several feet above the current level. Vertical walls and slabs that extend below the observed level will be subject to lateral and vertical

[7] Assuming an allowable peripheral shear of 100 psi obtained with a factor of safety of 2, based on 200 psi peripheral shear for compressive loading in Class 1a to Class 1b bedrock.

hydrostatic pressures as well as potential for water seepage into the occupiable spaces. The currently proposed three cellar scheme extends to about el. 30 or 40 feet below the perched water level.

Outlined below are viable strategies for permanent and temporary control of the groundwater, and with the excavation into bedrock.

Permanent Groundwater Control

The currently proposed three level deep excavation has a lowest level top of slab would be located about 48 feet below the existing sidewalk grade, about 40 feet below the measured water level. Permanent control of the groundwater run off can be accomplished by

1. Designing the lowest level cellar slab and below grade walls to resist hydrostatic forces. For this option, we recommend a double slab at the lowest level, consisting of the pressure slab, at least 12 inch sand/gravel layer, and a top wearing slab.

2. Utilize the below grade walls and a rock key to provide a vertical cut-off and relieveing the hydrostatic pressure below the slab.

3. Relieve horizontal and vertical hydrostatic pressures by linking a perimeter drainage system with the underslab drainage system.

Waterproofing

Exterior waterproofing is recommended for basement walls and the lowest slab, for all three options provided above. We recommend that they be fully waterproofed using a membrane type waterproofing, such as the Preprufe and Bituthene products by Grace. A layer of lean concrete, approximately three inches thick, should be placed over subgrade prior to installing the waterproofing layer. Plywood or similar protection materials (cement parging, etc.) should be placed on the perimeter rock and sheeting surface prior to the installation of the waterproofing membrane (as per the manufacturer's specifications). Any joints in the filter material should be carefully taped to prevent concrete or fines from entering the drainage panel. Punctures or tears in the drainage material must also be repaired, and penetration of the drainage material by utilities should be carefully sealed.

Underslab Drainage System

For option 2 and 3 listed above, we recommend the underslab drainage system consist of a minimum 18-inch-thick layer of ¾-inch clean gravel bedding be used. Recycled concrete aggregate (RCA) should not be used. The drainage pipes should be a minimum 6-inch in diameter, Schedule 40, slotted PVC conforming to ASTM D1785. The pipes should outlet into a sump pit for discharge to the city sewer system. The piping network should be installed beneath the slab with a maximum spacing of 20 ft on center. Typical clean-out fitting should be incorporated into the underslab drainage system.

Drainage piping should extend to a suitable gravity outlet or to a sump pit for removal by pumping. The sump and pumping system should be designed by the Mechanical Engineer. As this system is a permanent control measure, a duplex pump scheme is recommended to account for maintenance and emergency situations. Power loss and mechanical equipment failures should be considered. For preliminary design of the sump and pumping system, a flow rate of 60 gallons per minute (gpm) may be assumed. Once the excavation is open, and if groundwater is encountered, the rate of seepage can be observed and the actual daily inflow can be measured by the pumping required to keep the site dry.

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.

Temporary Control of Groundwater

Controlling the groundwater will be critical in order to allow for subgrade preparation and general foundation construction. Based on our experience with similar projects, we anticipate that sump pumping from gravel filled trenches and local sump pits should be suitable to temporarily control groundwater during construction.

Permanent Below-Grade Walls

Below grade walls will be subjected to lateral pressures due to soil loads, surcharge loads, and groundwater (hydrostatic) loads. In the static loading condition, lateral pressures from earth, groundwater and surcharge loads should be considered. This will consist of a triangular earth pressure distribution having an equivalent fluid weight of 60 psf per foot of depth (at rest condition) of soil above the groundwater table, an equivalent fluid weight of 18 psf per foot of depth (at rest condition) of rock above the groundwater table, and add 63 psf when below the design groundwater level. Water pressures should be added when designing for hydrostatic pressures. Lateral pressures due to 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.

The adjacent buildings are presumed to be supported directly on the bedrock, based on the test pits performed to date. Since the excavation is planned to be about 50 feet below grade, and located well below the adjacent building foundation levels, the surcharge loading from the neighboring structures needs to be accounted for with the structural design of the below grade walls. Once the structural loads from the adjacent structure have been established we can then advise on the lateral loads to be accounted for the below grade walls.

ADDITIONAL RECOMMENDATIONS

The following sections present our considerations related to excavation, monitoring, backfill and compaction, special inspections, and construction documents.

Additional Borings and Test Pits

Once the remaining portions of the building on Lot 19 have been completed demolished and removed, we plan to drill the remaining six borings and three test pits at the site. We plan that two deeper borings will be performed in the proposed tower core area. The test pits will help to confirm the adjacent building foundation conditions, while the borings will confirm bedrock depth, and allow for Langan to provide final geotechnical recommendations.

Rock Excavation

Rock excavation around the site perimeter will require very sensitive and careful removal techniques due to the close proximity of the adjacent buildings, hard rock, and possibly street utilities surrounding the site. The bedrock will likely be difficult to excavate, requiring rock chipping and splitting techniques. Channel drilling is recommended (overlapping drill holes at 4 inch diameter), especially around the site perimeter near existing structures, to minimize 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. Due to the close proximity of adjacent and landmarked structures, blasting operations to remove the bedrock will likely not be permitted. The buildings surrounding the site are expected shallower than the current proposed excavation level; therefore we anticipate the bedrock face to be exposed around the site perimeter. Given the bedrock discontinuity orientation data obtained from the borehole geophysical logging, there is indication of the presence of one prominent joint set and foliation. The data indicates that the excavation stability is more favorable along the south and west site perimeters, and has the potential to be unfavorable along the north and east site perimeters. It should be anticipated that excavation perimeters, particularly along the north and east sides of the site, will need proper support of the rock face during excavation.

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. The rock bolts and anchors should extend at least 5 feet beyond potential failure planes of rock wedges. The need for rock bolts and anchors, including spacing and length, must be determined by the Excavation Engineer in the field as excavation proceeds. However, permission would be required from adjacent property owners to allow the drilling and installation of rock bolts and anchors underneath each property. Rock bench heights should be restricted to 10-feet 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.

Temporary Support of Excavation

Since the proposed excavation is going to be significantly deep, the contractor must take appropriate measures to stabilize the work area and prevent lateral movements of the adjacent areas during the excavation. Given the presence of the adjacent buildings the soil above the bedrock, and along the streets will most likely be supported by solider piles or concrete buttons with internal bracing (i.e., walers, rakes, etc.). The rock directly below the existing walls of the adjacent buildings should be carefully supported, especially if poor quality and/or fractured rock are present. Due to the presence of the adjacent landmarked structures, we strongly recommend that the excavation support system be extremely stiff in order to provide proper lateral support.

Rock Bolting and Rock Reinforcement

Rock stabilization measures should also be anticipated during excavation. As previously discussed, the predominant rock discontinuities strike north-easterly to easterly and dip in a south-westerly to westerly direction. All excavation faces, may experience local wedge/block instabilities, however the north and east excavation faces are most susceptible to potential stability issues. Exposed rock faces should be examined geologically and mapped as the excavation proceeds. Loose, fractured, or soft rock should be secured with mesh or excavated and replaced with concrete; rock bolts or pre-stressed rock anchors should be used to secure any potentially unstable rock masses. The rock bolts and anchors should extend at least 5 feet beyond potential failure planes of rock wedges. Any rock bolts or anchors going below adjacent buildings will require the neighbor's permission.

Underpinning

We expect the excavation to extend at least 50 feet below current site grade. We anticipate that the buildings adjacent to the site are bearing on or near bedrock. Therefore, underpinning is expected to be relatively limited but if poor and/or fractured rock is encountered, the poor rock will need to be removed and replaced with concrete in sections (underpinning). Also, supporting the underlying bedrock below 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, and an internal bracing system (i.e. walers, rakes, etc.), where permission is may not be granted. A survey of all adjacent basement slabs, and walls is required by the DOB for underpinning, sheeting, and shoring design.

The existing foundation walls or the adjacent buildings surrounding the site must not be undermined by the proposed deep 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 is meets the gradation requirements discussed above. The use of recycled concrete aggregate, or the byproduct of blasting/tunneling (commercially known as mole rock), for backfill is not recommended.

Fill should be placed in uniform 8-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.

Preconstruction Documentation and Monitoring During Construction

Pre-construction conditions documentation of all adjacent and landmarked buildings should be performed. The documentation would provide the owner and foundation contractor, etc., with documentation of existing conditions in the event of a future damage claim.

As discussed herein, there are six (6) landmarked structures within 90 feet of our project site. The City of New York Department of Buildings Technical Policy and Procedure Notice (TPPN) #10/88, "Procedures for Avoidance of Damage to Historic Structures", dated June 6, 1988, requires special monitoring of all landmarked structures within 90 feet of the site. We have provided a copy within the attached Appendix F.

A pre-construction documentation, an observational and instrumentation program was implemented for demolition of the existing structures on site. However, a supplemental observation and instrumentation program based on the findings of the pre-construction documentation should be designed for monitoring the performance of adjacent structures and evaluating construction procedures during excavation and foundation construction. Landmarked structures require a building specific monitoring and protection plan be submitted and approved.

The monitoring program will need to consist of a precise optical survey program should be implemented to monitor for vertical and horizontal movements of surrounding structures. The survey should be performed biweekly, with measurements taken to the nearest 0.005 ft. The survey should be performed by a licensed surveyor. Excavation work should be temporarily stopped if movements (vertical or horizontal) exceed about 1/4-inch over two readings or a movement trend develops over several readings. Criteria for allowable movements of structures should be finalized for the supplemental observation and instrumentation program.

During excavation and foundation construction ground vibrations within the adjacent buildings should also be monitored using a threshold-type seismograph capable of measuring to 0.02 inches. The ground vibrations should be monitored full-time while rock excavation and foundation construction work is performed. TPPN #10/88 indicates the maximum permissible vertical and horizontal movement shall be 1/4-inch, while the maximum permissible peak particle velocity (vibrations) shall be 0.5 inches per second.

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, deep foundations, 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 provided in this preliminary report are based on subsurface conditions inferred from a limited number of borings, as well as architectural and structural information provided by SLCE Architects and Desimone Consulting Engineers, PC, respectively. Recommendations provided are dependent upon one another and no recommendation should be followed independent of the others.

Any proposed changes in structures or their locations should be brought to Langan's attention as soon as possible so that we can determine whether such changes affect our recommendations. Information on subsurface strata and groundwater levels shown on the logs represent conditions encountered only at the locations indicated and at the time of investigation. If different conditions are encountered during construction, they should immediately be brought to Langan's attention for evaluation, as they may affect our recommendations. The recommendations presented in this report assume that the subsurface conditions do not deviate appreciably from those disclosed by the borings.

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

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