

Geoengineering Constraints on Foundation: Case Study from Queens, New York City, USA

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Abstract

This paper deals with regional geologic information coupled with geoengineering and soil characterization aspects of a facility site to be developed by New York City Agency in Maspeth (40° 43' 28" North, 73° 55' 10" West), Queens. Soil and sediment samples, collected from depths close to the surface to over 60 m into the bedrock near the Maspeth site in Queens, consist of a zone of non-compact fill materials (3–8 m thick), underlain by a compressible peat and a partially decomposed highly plastic organic layer (liquid limit around 85) associated with calcareous clay and shell fragments (1–3 m thick). The presence of the shell-bearing unit close to the surface may be indicative of a buried estuarine complex in this area. In planning construction projects near waterways in older coastal cities, it is important to consider this possibility. The organic clay and peat layer were underlain by loose-to-firm glacial sand with gravels often intercalated with thin silty clay lenses. The current upper soil horizons are not sufficiently strong to withstand the required loading, which is estimated at near 1200 kips in some locations. The foundation support system will therefore have to be established in the glacial sand, possessing N (blow count) around 50 and liquid limit close to 30 (low plasticity).

Keywords: Compressible soil, geoengineering, Atterberg limits, estuarine soils, glacial sand, New York City

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1 Introduction

Regional geologic information coupled with geoengineering and routine soil characterization aspects related to a facility site to be developed by New York City Agency in Maspeth (40° 43' 23" North, 73° 54' 47" West), Queens, were critically assessed, and recommendations obtained from the studies were provided to this agency for consideration (Figure 1). The current investigation highlights the importance of regional geologic and sedimentologic investigation prior to undertaking geoengineering, design, and construction phases to build the facility for the New York City Agency. Maspeth (Figure 2) lies adjacent to Manhattan.

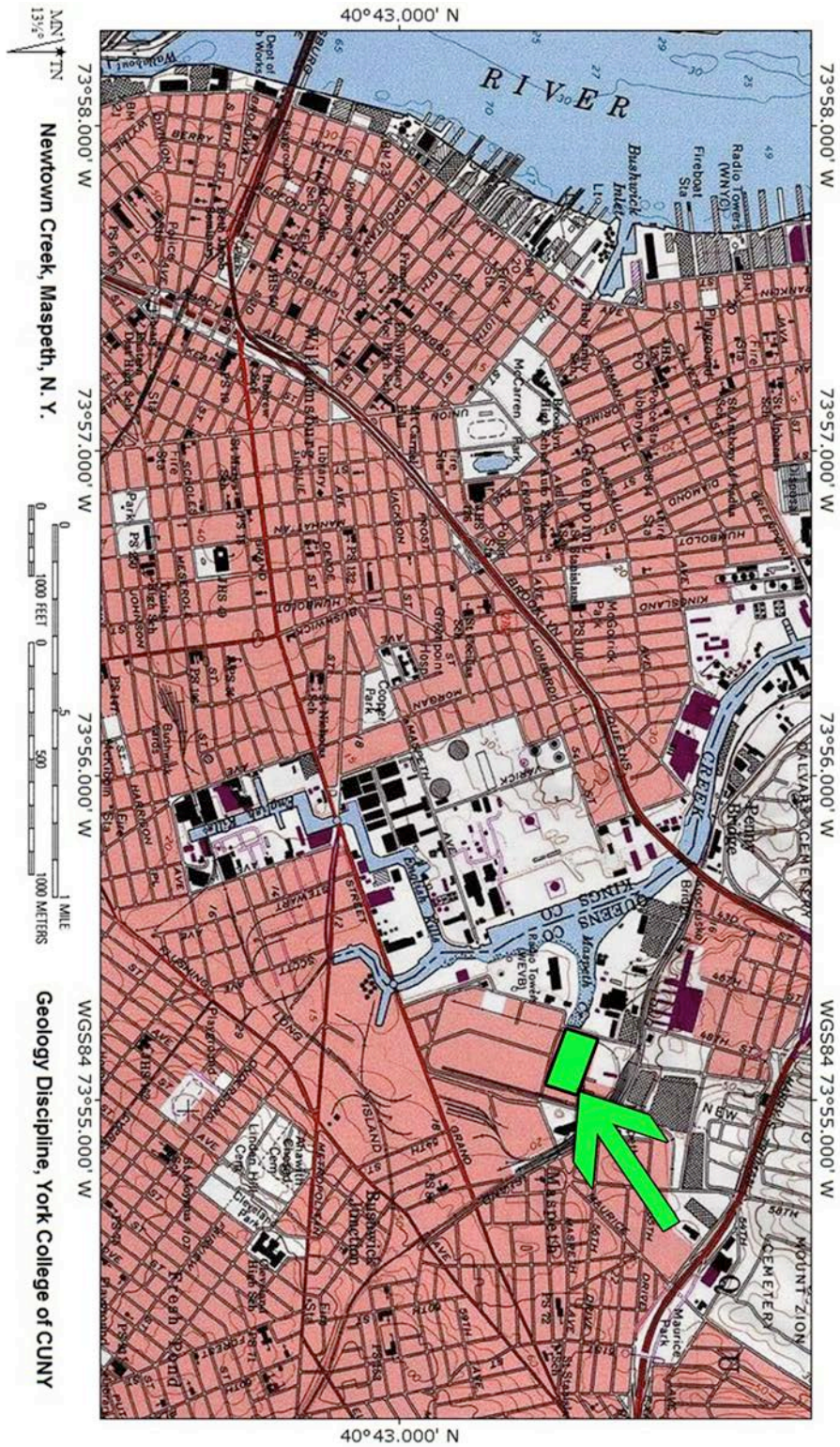


Figure 1: Location map showing Maspeth, Queens, New York City. (arrow pointing to the borehole locations)

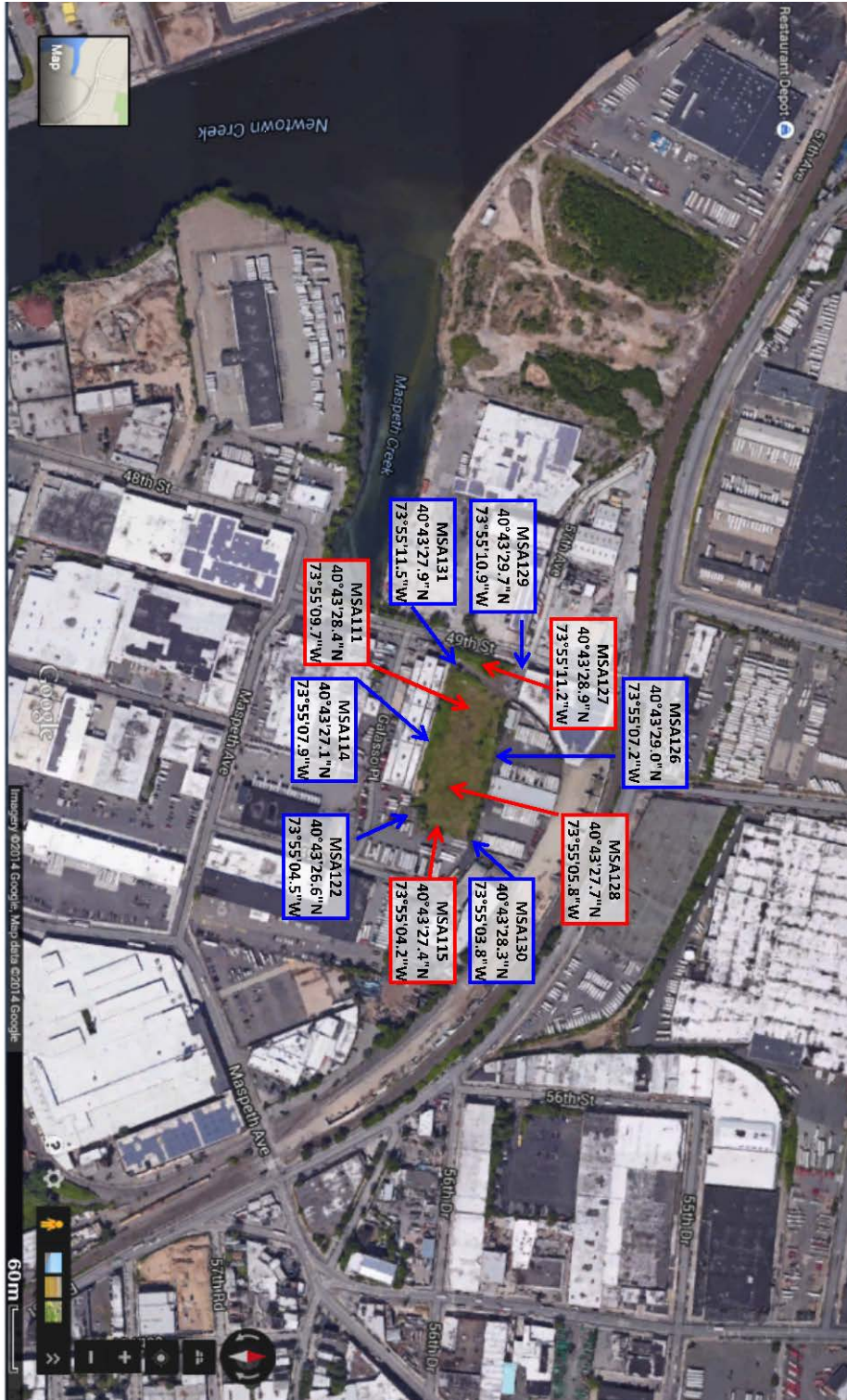


Figure 2: A closer view of the representative borehole locations with geographic coordinates. Maspeth Creek, west and southwest of the site, is one of the tributaries of Newtown Creek.

The surficial geology is dominated by anthropogenic fill consisting of discarded construction materials, abandoned and demolished buildings (Figure 3), and other landfill materials. A comprehensive soil mapping program was initiated by the New York City Soil and Water Conservation District (NYCSWCD), and this agency's most updated soil classification can be obtained by visiting http://www.nycswcd.net/soil_survey.cfm [1]. Surviving examples of these soils are not widely distributed. Much of the area is covered with glacial deposits. Wherever the bedrock is exposed, weathering and rapid erosion have also prevented large deposits from being accumulated in recent times. At one of the shaft sites of City Water Tunnel 2 in Brooklyn, the upper part of the bedrock was covered by concretionary sandstone, believed to be a residual soil derived from the weathering of the underlying gneiss [2]. Residual soils were also found in some of the deep borings in Queens and Brooklyn, advanced for the proposed City Water Tunnel 3. In the eastern part of the Bronx, explorations for the Cross Bronx Expressway revealed some deep weathering of the Manhattan Schists [3].



Figure 3: Shallow excavation showing fills comprising discarded construction materials.

Maspeth sits in close proximity to Newtown Creek (Figure 4), which has apparently been partially filled in by humans and is, therefore, not in its original configuration. Surface elevation in close proximity to the boring locations is slightly higher than 4 m above sea level and water table is found to be approximately 5 m below the ground. Newtown Creek, one of the still-existing estuaries, is a 5.6-kilometer waterway between Brooklyn and Queens. It empties into the East River near Long Island City. The current beginning of Newtown Creek is at Grand and 47th streets in Queens, although indications are that the creek once ran much farther inland.



Figure:4

Newtown Creek, one of the still-existing estuaries, is a 5.6 km waterway between Brooklyn and Queens. Notice Maspeth Creek located within few hundred meters close to the site

Newtown Creek is large enough to have had tributaries of its own, and at one time had small islands within it. Newtown Creek forms a portion of the boundary between the Boroughs of Brooklyn and Queens, in Kings and Queens Counties, New York. Newtown Creek includes the tributaries of Dutch Kills, Maspeth Creek, Whale Creek, East Branch, and English Kills. It flows through a highly industrialized area, is estuarine, and experiences tidal fluctuations of approximately 60 cm to 2 m. Groundwater flow beneath the site occurs under unconfined conditions and generally flows in a southerly direction towards Newtown and Maspeth Creeks and the groundwater flow patterns are generally similar under low and high tide conditions [4], [5], and [6].

The Newtown Creek area of Brooklyn and Queens has a history of extensive industrial development stretching back to the 1800s. This development resulted in major reworking

of the banks and channel for drainage, municipal discharges, and use for navigation purposes. The channelizing and deepening of Newtown Creek and its tributaries was largely completed to its current configuration by the 1920s and 1930s. This historical development has resulted in changes in the nature of Newtown Creek and its tributaries from a natural drainage condition to one that is largely governed by engineered and institutional systems. Currently, the predominant land use around Newtown Creek and its tributaries includes industrial, manufacturing, transportation, and utility facilities [7], [8], and [9].

Today, Newtown Creek is best known for the pollution that has plagued it for decades. Another prominent still-existing stream is Flushing Creek, which meanders through salt marshes in central northern Queens before emptying into the East River at Flushing Bay. The freshwater aquifer beneath Kings and Queens Counties in New York City (Figure 2) is bounded on the top by the water table and on the bottom by relatively impermeable crystalline bedrock. The southern, western, and northern lateral boundaries of the freshwater are bodies of saline ground water and the saline tidal water that surround Long Island [10], [11], [12], and [13].

2 Objectives

Primary objectives of this research were manifold. Geotechnical and sedimentological factors were provided to ascertain and evaluate several sedimentological and geotechnical parameters, including grain size, Unified Soil Classification System (USCS), Atterberg limits (liquid limit, plastic limit, plasticity index), pH, moisture, organic concentration, SPT (standard penetration test or N values), % recovery, and unconfined compressive tests (UCTs) on both soil and sediment samples to establish the area's overall geoengineering characteristics, which were useful for subsequently designing the proposed construction for this area. Restricting the aerial and geoengineering scope of this study to Maspeth (Figure 2) allows more precise observations to be made on a much smaller scale than was feasible in more regional investigations. The increased details of this study promises to lend a greater resolution to the general conclusions made in previous work.

Representative samples collected from several shallow-to-intermediate depth (15 to 70 meters) drill holes including MSA 111, MSA 114, MSA 115, MSA 122, MSA126, MSA127, MSA128, MSA129, MSA130, and MSA131 (Figure 2) enabled us to address and evaluate these strategic geoengineering attributes and come up with a viable baseline geological report associated with Maspeth site. Consideration of regional geologic information and structural settings were incorporated with the obtained geoengineering data for validating and reinforcing some of our conclusions drawn in the end.

3 Methodology

3.1 Field and Laboratory Investigations

In order to assess the significance for geoengineering in Maspeth, New York City Agency, in collaboration with a local sub-soil drilling company, carried out numerous closely-spaced (5 to 10 m apart) shallow (i.e., 15 to 23 m) and intermediate-depth (i.e., 70

m) exploratory borings. Closely-spaced boring locations also provided a clear window to the subsurface stratigraphy and samples collected from several intermediate drill holes including MSA 111, MSA 114, MSA 115, MSA 122, MSA126, MSA127, MSA128, MSA129, MSA130, and MSA131 (Figure 5 and Figure 6) were carefully selected in the field to conduct several geoen지니어ing and routine sedimentological investigations to obtain critical information pertaining to subsoil nature of the Maspeth site. Traditional split-spoon and cuttings were obtained to conduct various laboratory investigations and were included in grain size analysis (mechanical sieving, both dry and wet), Atterberg limits (liquid limit, plastic limit, and plasticity index), unconfined compressive tests, pH, organic concentration, and moisture content determination. Visual soil classification was conducted by following ASTM D 2487 (Casagrande's Classification System). Furthermore, detailed cross-sections (Figure 5 and Figure 6) depicting subsurface geology were constructed by utilizing borehole information, well site geologist's daily log, and lithological characterization of representative samples in the laboratory.

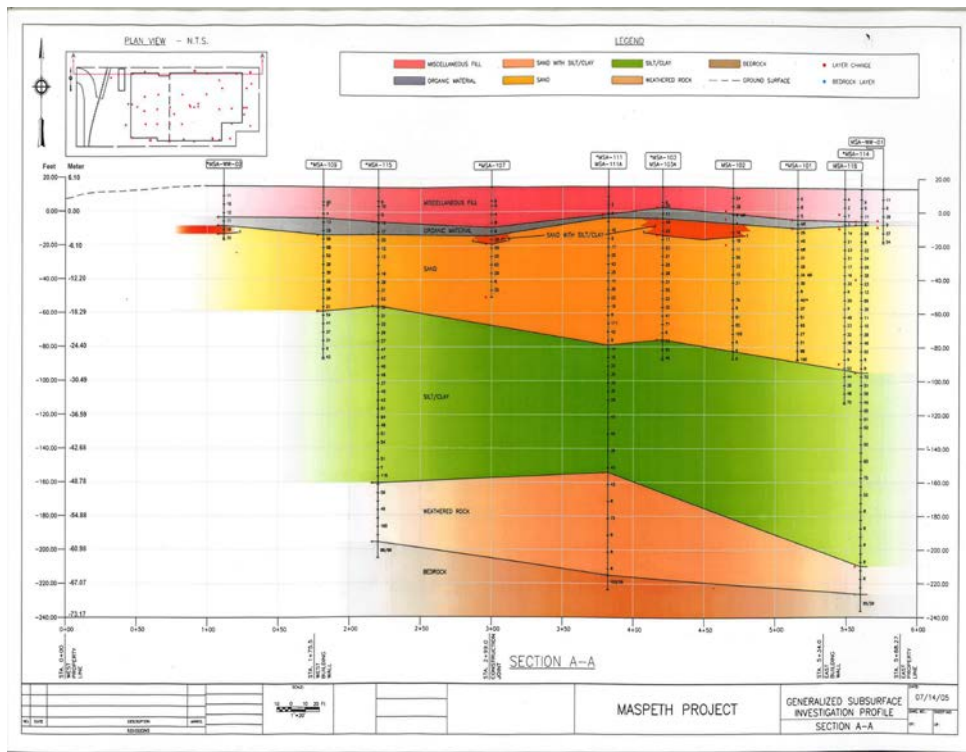


Figure 5: Generalized subsurface investigation profile showing bore holes and encountered fills and dominant sedimentary rock types. Cross-section is drawn along A-A' (west-east, northern perimeter of the site). Notice laterally continuous Organic-rich, highly compressible layer (black) around - 3 meter. It is underlain by thick glacial sand (- 10 m to -27 m), followed by massive silt/clay (-27 m to - 45 m; green). Crystalline basement is capped by weathered rock of variable thicknesses. Borehole spacing is variable (4m to 20 m).

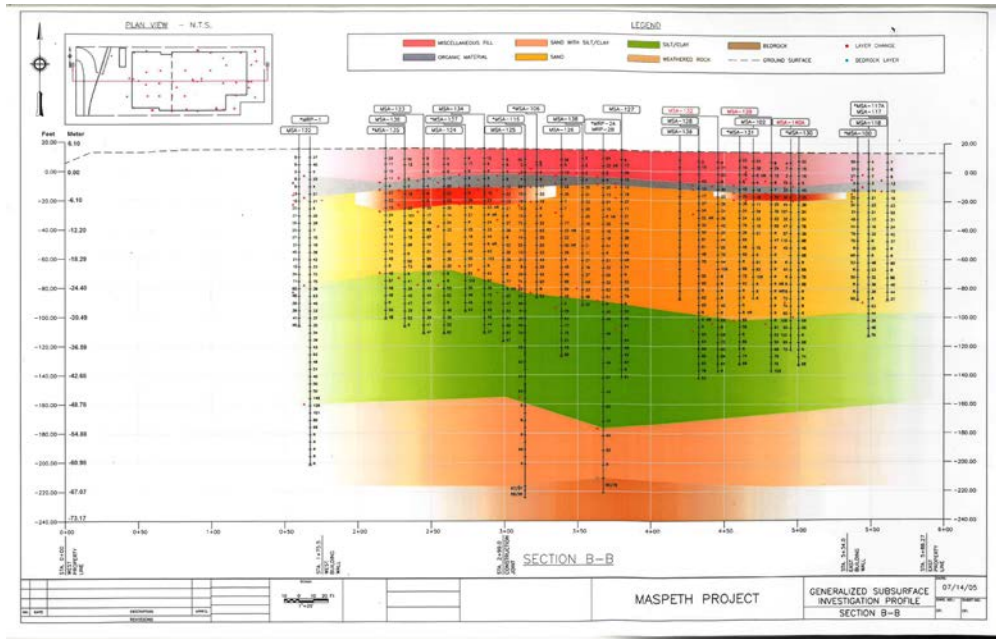


Figure 6. Cross-section is drawn along B-B” (west-east; center of the site, south of A-A”). It is representing additional bore holes and depth of the solid crystalline bedrock was encountered around – 83.8 meter. Notice variable thicknesses associated with overlying weathered rock, silt/clay, and glacial sandy layers. Compressible organic layer (black) extends laterally. Thickness of Fill materials is also extensive.

3.1.1 Maspeth Soil/Sediment Characterization Based on Unified Soil Classification System (USCS)

USCS classifies soils from any geographic location into categories representing the results of prescribed laboratory tests to determine the particle-size characteristics, the liquid limit, and the plasticity index [14]. The assigning of a group name and symbol(s) along with descriptive information required in routine geoenvironmental investigation and can be used to describe a soil to aid in the evaluation of its significant properties for engineering use (Figure 7A and Figure 7B; Unified Soil Classification System for fine-grained and coarse-grained soils respectively; <http://matrix.vtrc.virginia.edu/DATA/GINT/vdotusc.PDF>).

FINE-GRAINED SOILS
(50% or more of material is smaller than No. 200 sieve size.)

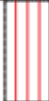






SILTS AND CLAYS Liquid limit less than 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit 50% or greater		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS		PT	Peat and other highly organic soils

Figure 7A. Unified Soil Classification System for fine-grained soils.
<http://matrix.vtrc.virginia.edu/DATA/GINT/vdotusc.PDF>









COARSE-GRAINED SOILS (more than 50% of material is larger than No. 200 sieve size.)		
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	Clean Gravels (Less than 5% fines)	
	 GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	 GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
	Gravels with fines (More than 12% fines)	
	 GM	Silty gravels, gravel-sand-silt mixtures
	 GC	Clayey gravels, gravel-sand-clay mixtures
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	Clean Sands (Less than 5% fines)	
	 SW	Well-graded sands, gravelly sands, little or no fines
	 SP	Poorly graded sands, gravelly sands, little or no fines
	Sands with fines (More than 12% fines)	
	 SM	Silty sands, sand-silt mixtures
 SC	Clayey sands, sand-clay mixtures	

Figure 7B: Unified Soil Classification System for coarse grained soils.

Use of this standard enables us to devise a single classification group symbol and group name, except when a soil contains 5 to 12 % fines or when the plot of the liquid limit and plasticity index values falls into the crosshatched area of the plasticity chart. In these two cases, a dual symbol is used (e.g., CL-ML). When laboratory test results indicate that the soil is close to another soil classification group, the borderline condition can be indicated with two symbols separated by a slash, and the first symbol should be based on this standard (e.g., CL/CH, SC/SM, SC/CL). Borderline symbols are particularly useful when the liquid limit value of clayey soils is close to 50. These soils can have expansive characteristics, and the use of a borderline symbol (e.g., CL/CH, CH/CL) will alert the user of the assigned classifications of expansive potential (Figure 7A and Figure 7B).

This standard is the ASTM version of the USCS. The basis for the classification scheme is the Airfield Classification System developed by A. Casagrande in the early 1940s. It became known as the USCS when several U.S. government agencies adopted a modified version of the Airfield System in 1952 (Table 1).

Table 1: Categories of ground beneath a site.

Material type	Strength	Compressibility	Permeability
Rock	Very High	Very Low	Medium to high
Granular soil	High	Low	High
Cohesive soil	Low high	Very Low	
Organic soil	Very Low	Very High	High
Made ground	Medium to very low	Medium to very high	Low to high

Early British practice was summarized in Cooling et al.'s discussion of Casagrande's 1947 plasticity chart [15].

3.1.2 Grain Size Distribution

Routine grain size distribution method covers the quantitative determination of the distribution of particle sizes in soils. US standard sieves from 6 (3.36 mm opening) to 200 (0.074 mm opening) were arranged to follow 0.25 phi interval for recording precise grain size data. The distribution of particle sizes larger than 75 micrometers or 0.075 mm (retained on the No. 200 sieve) is determined by dry sieving, while the distribution of particle sizes smaller than 75 micrometers is determined by a sedimentation process using a hydrometer. Humboldt sieving machine was used in the laboratory to conduct grain size investigation. Humboldt sieves are well-known for adhering to ASTM standards and for use in all types of sieve testing applications, from sampling and classification of soils, aggregates and other powdered and granular materials to specific ASTM standard tests.

3.1.3 Atterberg Limits

To conduct this laboratory investigation to determine the plastic and liquid limits of fine-grained soil standard, procedures described in ASTM 4318 were followed. The liquid limit (LL) is arbitrarily defined as the water content, in percent, at which a portion of soil in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm (0.5 in.) when subjected to 25 shocks from the cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second. The plastic limit (PL) is the water content, in percent, at which a soil can no longer be deformed by rolling into 3.2 mm (1/8 in.) diameter threads without crumbling. ASTM D 4318 was used to determine Atterberg limits for liquid limit (LL), plastic limit (PL, measured in percentage), and plasticity index (plasticity index $PI=LL-PL$, also measured in percentage) in order to assess the geotechnical properties of several layers from -5 m extending to - 45 m considering their direct bearing with settling of foundation or consolidation factor. Both disturbed and undisturbed samples were used based on the availability of field samples and recovery percentage.

3.1.4 Estimation of Water or Moisture Content

For many soils, water content may be an extremely important index used for establishing the relationship between the way a soil behaves and its properties. The consistency of a fine-grained soil largely depends on its water content. The water content is also used in expressing the phase relationships of air, water, and solids in a given volume of soil. The

steps below were followed to accomplish determination of moisture content concerning representative Maspeth soil/sediment samples [16] and [17].

- 1) Label the moisture can and lid number.
- 2) Determine and record the mass of the empty, clean, and dry moisture can with its lid (M_C).
- 3) Securely insert the collected samples in the moisture can and close the lid.
- 4) Determine and record the mass of the moisture can (now containing the moist soil) with the lid (M_{CMS}).

Remove the lid and place the moisture can (containing the moist soil) in the drying oven that is set at 105 °C and let it stay on overnight.

- 5) Remove the moisture can.
- 6) Carefully but securely, replace the lid on the moisture can using gloves and allow it to cool to room temperature.
- 7) Determine and record the mass of the moisture can and lid (containing the dry soil) (M_{CDS}).
- 8) Empty the moisture can and then clean the can and lid.

Data Analysis:

- (1) Determine the mass of soil solids.

$$M_S = M_{CDS} - M_{SC}$$

- (2) Determine the mass of pore water.

$$M_W = M_{CMS} - M_{CDS}$$

- (3) Determine the water content.

$$W = \frac{M_W}{M_S} \times 100$$

3.1.5 Distribution of Organic Matter

The percent organic matter is important in the following tasks: (1) classifying peat or other organic soil, (2) establishing more information for geotechnical and general classification purposes. The presence of organics in soils is generally associated with high compressibility, significant secondary compression, often unsatisfactory strength characteristics, and low unit weight. The loss on ignition or commonly known as LOI test is the most widely used technique for measuring organic content, especially for low organic content soils [18].

The loss on ignition method is based on the thermal decomposition of organic matter. High temperature heating is used to remove organic matter. An oven-dried soil sample (typically about 10 g in mass and entirely passing the No. 10 sieve or 2.00 mm in diameter) is placed in a crucible and set in a well-temperature-control furnace. The weight loss of the soil sample due to the oxidation of the organic matter, termed the loss on ignition (LOI [%]) is obtained from the following:

$$SOM(\%) = LOI(\%) = \frac{Weight_{105} - Weight_{440}}{Weight_{105}} \times 100$$

where SOM (%) stands for the soil organic matter content; Weight 105 is the weight of the soil after oven-drying to 105°C for 24 hours and Weight 440 is the weight of the soil

after ignition at 440°C for a given period of time. The temperature of 440°C used in this equation is based on ASTM D 2974 Method C.

3.1.6 Unconfined Compression Test (UCT)

The primary purpose of the unconfined compression test is to quickly obtain a measure of compressive strength for those soils that possess sufficient cohesion to permit testing in the unconfined state (emphasis was provided on samples collected from 30 m below the surface and also underlying glacial sand). This test method is applicable only to cohesive materials that will not expel or bleed water (i.e., water expelled from the soil due to deformation or compaction) during the loading portion of the test and that will retain intrinsic strength after removal of confining pressures, such as clays or cemented soils. Research has shown that the strength of a soil determined by compression testing varies with extremes of the length-to-diameter ratio and the rate of strain. It is generally accepted that ratios of length-to-diameter of 1.5 to 3.0 are satisfactory. Ratios of 2.0 and 2.5 are commonly employed. Similarly, satisfactory rates of strain are 0.5 to 2.0% per minute. Most samples use 0.5 to 1.0% per minute [18].

Unconfined compressive strength is calculated the same as for any material, with an additional calculation of the area change from bulging.

$$q_u = P/A$$

q_u = unconfined compressive strength (psi)

P = compressive force (lbs)

A = cross-sectional area (inch²)

The shear strength is defined as half the compressive strength [19].

The pocket penetrometer is designed as a lightweight instrument for use by field personnel to check visual classification of soils. The objective of the unconfined compression test is to determine the UU (unconsolidated, undrained) strength of a cohesive soil in an inexpensive manner. It can be used to verify whether excavation side walls require shoring, based on OSHA cohesive soils classifications. This instrument also indicates consistency, shear strength, and approximate unconfined shear strength. Direct-reading scale—in tons/sq. ft, or kg/sq. cm—corresponds to equivalent unconfined compressive strength.

Type A soils are cohesive soils with an unconfined compressive strength of 1.5 tons per sq. ft (tsf) (144 kPa) or greater. Examples of Type A cohesive soils are often designated as follows:

clay, silty clay, sandy clay, clay loam and, in some cases, silty clay loam and sandy clay loam.

3.1.7 Vane Shear Test

This test method covers the miniature vane test in very soft to stiff, saturated fine-grained clayey soils ($[\phi] = 6$ or greater; 0.044 mm or lower). Knowledge of the nature of the soil in which each vane test is to be made is necessary for assessment of the applicability and interpretation of the test results. Samples collected from several intermediate drill holes including MSA126, MSA127, MSA128, MSA129, MSA130 and MSA131 were carefully selected in the field to adhere to this assumption. For in-situ field accuracy, vane shear tests were performed on sub-samples with respect to each boring at a regular interval ranging

from 1 to 3 m. The miniature vane test conducted largely in fine-grained, predominately clayey soils encountered around 35 m (underlying the glacial sand) recorded an undrained shear strength of less than 1.0 tsf (100 kPa), which are defined as stiff according to ASTM Standard Practice D2488.

3.1.8 Determination of pH

The pH of the soil is a useful variable in determining the solubility of soil minerals and the mobility of ions in the soil and assessing the viability of the soil-plant environment. The pH measurements are made in both water and a calcium chloride solution, because the calcium displaces some of the exchangeable aluminum. Therefore, both measurements are required to fully define the character of the soil's pH. For the purpose of this test method the soil samples were sieved through a 2-mm (No. 10) sieve. Current investigation included representative soil samples from the Organic-rich upper layer (-3 m to -7 m), Glacial sand (-7 m to -25 m), and Silty clay layer (-25 m to -42 m) underlying the Glacial sand. This test method covers the measurement of the pH of soils for uses other than for corrosion testing. This measurement determines the degree of acidity or alkalinity in soil materials suspended in water and in a 0.01 *M* calcium chloride solution.

3.1.9 Standard Penetration Test (SPT) or N (blow counts)

SPT or blow counts were retrieved from the daily log prepared by the site geologists and recorded in this paper to conduct geoengineering investigation. SPT (standard penetration test) or N (blow counts) was performed by using a 64 kg (140-pounds) hammer and letting it free-fall for 76.2 cm (30 inches) with a 15.2-cm interval (6-inch), reaching a maximum depth of 61 cm (24 inches). For better accuracy of estimating N values, only second 30-cm (12-inch) and third 46-cm (18-inch) measurement were used in this research. The value recorded for the first round of advance is usually discarded because of fall-in and contamination in the borehole. This value is reported as the SPT blow count value, commonly termed "N". After the penetration test is completed, the sampler is retrieved from the hole. The split barrel is opened, the soil is classified, and a moisture specimen is obtained. After the test, the borehole is extended to the next test depth and the process is repeated. SPT soil samples are disturbed during the driving process and cannot be used as undisturbed specimens for laboratory testing.

4 Main Results

4.1.1 Geoengineering Characterization

ASTM D 2974 and ASTM D 4972 were used to evaluate organic content and pH values, respectively. In-situ unconfined compressive strength investigation was conducted on a few representative samples using the pocket penetrometer to assess its mechanical strength (Table 1). Recovery percent (based on a 60-cm or 2-ft recovery) was retrieved from the field data provided by the drillers. Samples were collected via split-spoon samplers and boring depths ranged from 15 m to more than 70 m. Crystalline rocks belonging to late Precambrian to early Cambrian bedrock, consisting primarily of schist, gneiss, and marble, was recorded around this depth [20] and [21]. Overall, the subsurface

geology below the anthropogenic fill (up to 3 m) can be described generally based on Figure 5 and Figure 6. Compilation of several lithologic logs based on visually inspected soil and sediment revealed an approximate subsurface geologic condition in the vicinity of the site, which also provided critical stratigraphic information in relation to lateral continuity, consistency of thickness, and lithologic homogeneity of various strata. Two generalized subsurface east-west profile labeled as A-A'' (Figure 5) and B-B'' (Figure 6) were constructed based on drilled soil and sediment data to approximately reveal subsurface stratigraphy. Considering closely spaced drill holes (20 m to 4m) and covering a length of over 200 m in east-west direction, construction of two detailed stratigraphic profiles (A-A'' and B-B'') enabled us to depict high resolution subsurface conditions and thoroughly assess the geoenvironmental characteristics related to this project. An organic layer comprising decomposed woody and plant materials, rootlets, and leaves, often resembling peat and amounting to a thickness of 2.5 m to 4 m, underlies the anthropogenic fill (Figure 8A and Figure 8B).

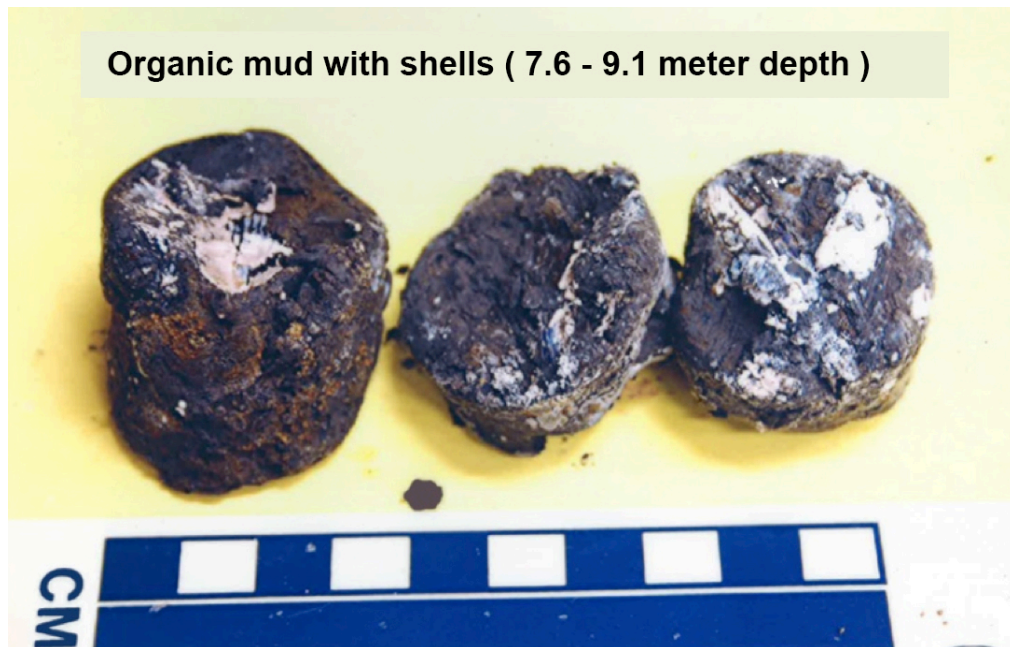


Figure 8A: Suggested buried estuary as evidenced by the presence of Organic-rich, compressible peat-like materials with clam shells. (7.6 to 9.1 meter below sea level).



Figure 8B: A close inspection of well-preserved clam shells within the peat-like materials.

Organic content and percent moisture distribution within this organic layer was analyzed by following ASTM 2974 and ASTM 2216. Representative samples used to determine organic content and pH are recorded in Figure 9A and Figure 9B. An overall alkaline condition was noticeable in all conditions except in sample 3 (-6 m to -7 m), where the soil is more acidic (pH = 4.0). Generally speaking, there is a good correlation with higher organic concentration in respect to lower pH; however, local anomaly does exist as can be seen in sample 2. A silty clay to clay layer, mostly lenticular type, lies below the organic layer and reaches a maximum thickness of 5 m.

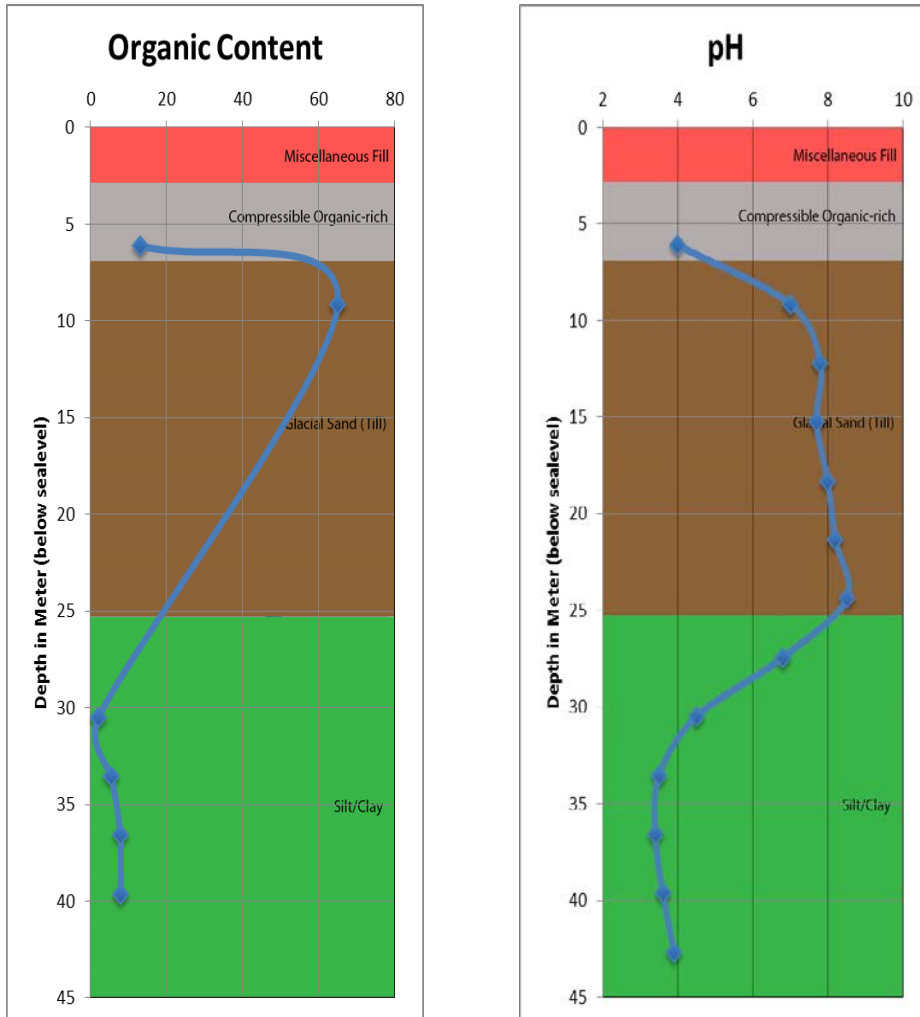


Figure 9A: The amount of organic content registered a higher number in Organic zone and displayed a dramatic decline within the Glacial Sand.

Figure 9B: Measured pH values indicated acidic setting within the Organic-rich zone followed by being slightly alkaline (Sand) and becoming acidic within the Silty Clay unit encountered below – 27 meter.

Plasticity index displayed a general increasing trend from being 12 at depths of 7 m to 20 at a depth of 40 m, whereas plastic limit decreased from 70 at 7 m deep to 20 at a depth of 38 m. Liquid limit displayed a similar trend with 83 within samples collected from 7 m and gradually diminishing to 35 when reaching a depth of 38 m.

4.1.2 Soil Index

Grain size distribution involving organic silt layer (-3 m to - 7 m) revealed black clayey silt with trace amount of sand, and using the USCS, it was labeled “OH” as the dominant soil type. The work led to propose a system for classifying organic soils that is based on the percentage of organic matter present: soils with organic content <3% are termed

“mineral soils”; if the organic content is >3% and <15%, then soils are classified as “mineral soils with organics”;

when the organic content exceeds 15% but is <30%, the term “organic soil” is employed. Finally, soils with organic content higher than 30% are termed “highly organic soils” or “peats.”

Presence of clam shells and decomposed woody materials often resembling peat indicated an existence of estuarine or swampy condition in the past, and extensive anthropogenic fills were used to reclaim this site, which was once was part of the Newtown Creek. As result of anthropogenic fill over the highly compressible and mechanically weak organic layer, consolidation of underlying sediments was not significant, and it was quite apparent from the higher moisture content (over 80%) and liquid limit exceeding 80 (Figure 10). The presence of soil organic matter, a complex mixture of organic compounds, differentiates organic soils from inorganic soils. The amount of soil organic matter significantly affects overall soil index, physico-chemical, and engineering properties of soils, including specific gravity, water content, liquid limit, plastic limit, density, cation exchange capacity, hydraulic conductivity, compressibility, and strength.

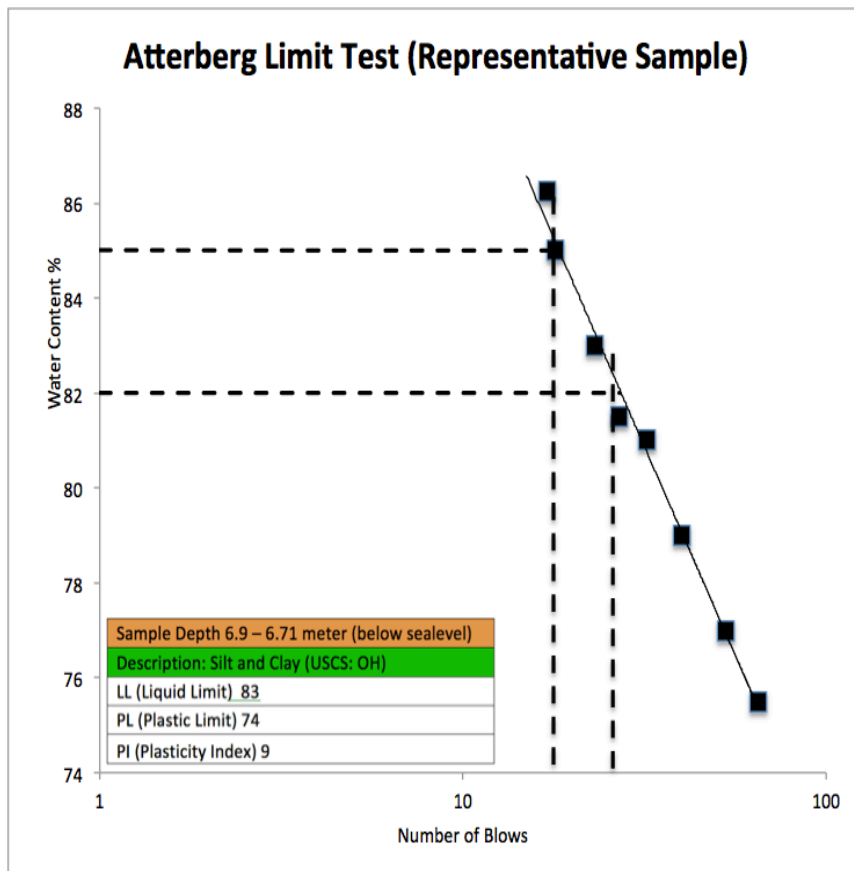


Figure 10: Representative Atterberg Limit Test conducted on Organic-rich clay layer (- 6.7 to 6.9 m). Notice very high Liquid Limit (83) and Plastic Limit (74). USCS designation “OH” is given to this highly plastic and compressible clay layer underlying just below the Miscellaneous Fill.

4.1.3 Standard Penetration Test (N) & Unified Soil Classification System (USCS)

SPT (standard penetration test) conducted on this compressible organic layer provided the N (blow count) value to be between 5 (5 blows per 30 cm penetration) and 12 (12 blows per 30 cm penetration). Below the compressible silty clay to clay layer is thick, massive, fine- to medium-grained, well-graded sand mixed with rock fragments largely containing medium to fine gravel-sized quartzite, basalt, and schist (-7 m to -27 m). In places massive sand is intercalated with a trace to subordinate amount of silty clay. This sandy unit is 20 to 27 m thick and is presumed to be of glacial origin, Wisconsinan age. Based on the USCS, glacial sand has been designated as primarily SM with occasional SC and SP. N (blow count) values in this massive sandy unit range from 40 to 55, and recovery of sand is between 50% and 65%. Glacial sand encountered in several bore holes correlates with respect to till reported elsewhere by several authors. Generally speaking, till based on regional distribution is composed of clay, sand, gravel, and boulders, and forms the Harbor Hill and Ronkonkoma moraines. Outwash consisting mainly of brown fine-to-coarse sand and gravel is stratified and interbedded with clays. The till has relatively poor permeability. The sand and gravel part of the outwash is highly permeable; yields of individual wells in it are as much as 1,700 gal/min. Specific capacities of wells are as much as 330 gal/min per meter of drawdown. Groundwater is fresh, except near the shorelines. Horizontal hydraulic conductivity is 7–23 m/d and 65–95 m/d in moraine and outwash deposits, respectively. Horizontal-to-vertical anisotropy is 10:1. Specific yield is 0.25 and 0.3 in moraine and outwash deposits, respectively [21] and [22].

4.1.4 Assessment of Cohesive Silty Clay

The massive sandy unit is underlain by a thick, silty clay to clay layer, often reaching a maximum thickness of 40 m (70 m below sea level). Roughly, unconfined compressive strength of the clay layer recorded a moderate-to-medium strength (Figure 11) with N values ranging from 60 to 75 and total core recovery also ranging from 75 to almost 100%.



Figure 11: Major lithologic unit above the solid bedrock is dominantly composed of variegated, moderately to highly compact silty clay; often recording higher N or SPT counts. (61 to 67 meter below sea level).

Field investigations of in-situ sediment cores from this unit revealed clay-rich, variegated, dense, and in places, pyritized zones and are suggestive of reduced/anoxic deposition presumably restricted from paludal to an aerially extensive lacustrine environment. This is often associated with saprolites displaying relict foliations and evidence of kaolinitization and chloritization. Weathering of the parent rock, consisting mostly of schists, pegmatites, granites, schistose-gneiss, and marble of late Proterozoic to early Cambrian age, has apparently contributed to the development of the saprolitic units. The Grenville rocks (late Proterozoic) form the basement for all of New York State and lie buried, unexposed under the younger rocks, over most of the state. However, they are exposed at places in the southeastern New York State, particularly in the Hudson Highlands. Evidences of extensive weathering of crystalline basement rock have been suggested by many researchers and generally linked with bedrock heterogeneity, structural weakness, degree of alteration, and lowering of sea level associated with a past glacial episode (Pleistocene Glaciation). Present-day Hudson River Canyon is one of the primary examples of being largely affected by lowering sea level due to a glacial episode. Lowering sea level in response to glacial episode contributed to subsequent drop in base level, resulting in rapid downcutting and forming incised deep valleys, which were eventually filled in by younger sediments. Correlation of this clayey horizon with similar sedimentological characteristics is perhaps indicative of Raritan Clay (Upper Cretaceous), which is locally known to be associated with the laterally extensive Cretaceous aquifer called Llyod sand. The Raritan unit is a relatively impermeable confining unit. Locally, it is of lenticular type, with admixtures of sand and gravel displaying moderate-to-high permeability. Vertical hydraulic conductivity is 0.03 cm/d. Yields are as much as 2,000 gal/min to individual

wells and have been recorded elsewhere. Specific capacities as high as 120 gal/min per meter of drawdown typically characterize this unit. There is some evidence of artesian pressure in some wells. Water is of good quality, except for its high iron content. Horizontal hydraulic conductivity is 15–25 m/d. Horizontal-to-vertical anisotropy is 10:1. Specific storage is 1×10^{-2} per m [23] and [24].

5 Discussion

5.1.1 Suggested Load Bearing Capacity Based on Geoengineering Attributes

Several key sedimentological aspects, such as an association of compressible organic layer and silty clay to clayey layer at a depth reaching 7 m, must be considered as an important geoengineering challenge since any foundation design involving these two shallow layers will pose a threat to the stability of the foundation. Based on the very low N value (between 5 and 12), as determined by the SPT (standard penetration test) and compressible nature of the current upper ground horizons (-3 to -7 m), these values were not considered to be sufficiently strong enough to withstand the required loadings, which were estimated at near 1200 kips for some locations. Calculation of Atterberg limits including liquid limit, plastic limit, and plasticity index for organic-rich clayey layer provided a very highly plastic designation (i.e., liquid limit ranging between 70 and 85%) and raised serious questions as to the stability of foundation on the organic and primarily cohesive clayey layer (Figure 12A, Figure 12B, and Figure 12C). Considering the very highly plastic nature (LL being 70–85%) of this cohesive organic-rich clay layer, a serious dewatering phase can be envisioned during the construction phase to include interference with high groundwater table, which is apparently in existence very close to the site, and this will certainly create a major problem.

The foundation support system will therefore be established in the till, comprising thick, primarily non-cohesive sandy horizon and registering N value between 40 and 55 (Table 2).

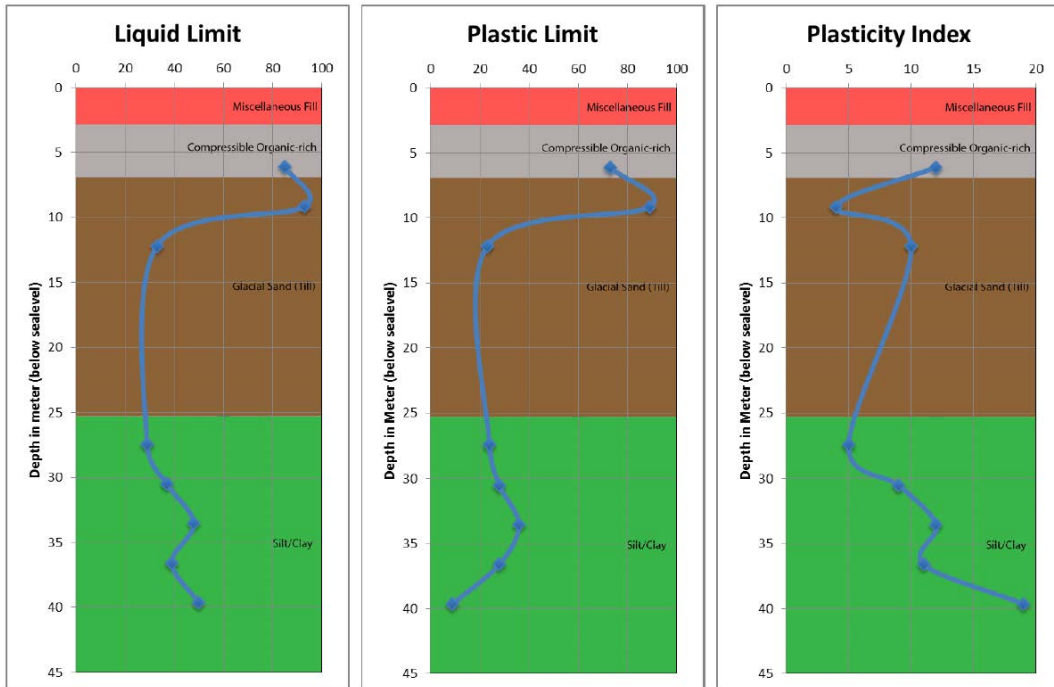


Figure 12A: Liquid Limits were calculated for Organic-rich clay, Silty clay, and Glacial sand. Notice higher Liquid Limit (greater than 85; very highly plastic) for Organic-rich zone below the Miscellaneous Fill. Liquid Limit for Glacial Sand is around 30 (low plasticity).

Figure 12B: Plastic Limit showing greater number for Organic-rich zone located around -3 to -8 meters below the Miscellaneous Fill.

Figure 12C: Plasticity Index markedly varied being higher in Organic-rich zone compared to Glacial Sand.

Table 2: Representative borehole data displaying Recovery (in %), Unconfined Compressive Test (UCT), Vane Shear Test (VCT), and Standard Penetration Test (N or blow counts) belonging to the silty clay layer underlying the glacial sand. Weight of the thicker glacial sandy horizon above exerted pressure and caused greater compaction and subsequent increase in blow counts (N). Depth in meter (below sealevel).

Boring	Depth in meter (below sealevel)	Measured Sediment Core Recovery (%)	UCT (unconfined compressive test, measured in kgf/cm ²)	Vane Shear Test (measured in kgf/cm ²)	N (standard penetration test)
MSA126	35	95	4.5	1.25	64
	37	100	4	1.08	77
	38	100	3.6	2.5	75
	40	100	3.83	0.8	63
	41	100	4.5	0.4	78
	43	100	4	0.38	88
MSA127	38	100	4.01	0.93	68
	41	90	4.5	0.87	98
	43	100	4.5	0.78	92
	44	95	4.43	0.94	87
	47	100	4.08	0.82	81
MSA128	38	92	1.2	0.41	90
	40	85	4.13	0.84	64
	41	90	4.07	1.03	88
	43	100	4.5	0.7	86
	44	100	3.23	0.4	80
	46	100	4.26	0.82	78
	47	92	4.5	0.83	72
MSA129	37	85	2.77	0.53	61
	38	100	3.87	0.9	53
	40	80	3.9	0.98	50
	41	82	4.1	0.64	68
	43	100	4.5	0.34	79
	44	100	4.2	0.57	79
	46	95	4.01	0.36	103
MSA130	37	47	3.9	0.6	52
	40	75	3.5	0.85	74
	41	85	4.5	0.57	79
	43	75	4.25	0.65	61
	44	85	3.8	0.85	75
MSA131	35	90	3.3	0.8	69
	37	95	3.3	0.75	56
	38	70	3.3	0.78	55
	40	75	4.2	0.82	88
	41	62	4.5	0.7	77
	43	77	4.2	0.6	74
	44	85	4.3	0.75	68

Liquid limit (25 to 30; low plasticity) and plastic limit (mostly confined to 20) associated with sandy unit (Figure 12A through Figure 12C) further corroborates the notion of the nearly homogeneous lithological nature of glacial sand (dominantly SM; occasionally SP based on USCS) with insignificant association of any dominant silty or clayey unit (Table 3). However, in places, small lenses of silty clay (USCS: ML) are known to occur due to the effect of degree of weathering involving bedrock. The other significant geoenvironmental aspect of this sandy layer is the lesser likelihood of large-scale dewatering issue due to piling and overlying weight.

Table 3: Representative Soil/Sediment Types based on ASTM, Unified Soil Classification System (USCS) and Natural Moisture Content.

Sample Depth (meters, below sealevel)	Description ASTM 2487 Visual and ASTM D422	Unified Soil Classification System (USCS)	Natural Moisture Content (in %) ASTM 2216
6.1 - 6.7	Black, clayey silt, trace fine sand	OH	85
7.6 - 8.2	Black organic silt	OH	88
9.1 - 9.8	Dark brown, coarse to fine sand and silt, trace fine gravel	SC	22
10.7 - 11.3	Dark brown silt and coarse to fine sand; trace fine gravel	ML	23
12.2 - 12.8	Brown coarse to fine sand, little silt; some medium to fine gravel	SM	11
13.7 - 14.3	Brown coarse to fine sand, little silt; some coarse to fine gravel	SM	10
15.2 - 15.9	Dark yellowish brown coarse to fine sand, little silt, some medium to fine gravel	SM	10.7
16.8 - 17.4	Brown coarse to fine sand, some silt and coarse to fine gravel	SM	11.7
18.3 - 18.9	Yellowish brown coarse to fine sand, little silt, some medium to fine gravel	SM	12.6
19.8 - 20.4	Light brown coarse to fine sand, some silt; little medium to fine gravel	SM	14.2
21.3 - 22	Brown coarse to fine sand, trace silt and little medium to fine gravel	SP-SM	16.3
22.9 - 23.5	Brown coarse to fine sand, some silt and some medium to fine gravel	SM	10.8
24.4 - 25	Light yellowish to brown coarse to fine sand, trace silt and little coarse to fine gravel	SP-SM	14
25.9 - 26.5	Brown coarse to fine sand, some silt and some medium to fine gravel	SP-SM	11.8
27.4 - 28	Light gray silt and clay, little fine sand	OL	17.3
29 - 29.6	Gray silt and clay, little fine sand	OL	22.3
30.5 - 31.1	Gray silt and clay, little fine sand	OL	30.8
32 - 32.6	Light gray silt and clay, little fine sand	OL	28.7
33.5 - 34.1	Light gray to dark silt and clay, little fine sand	OL	29

* **PT**: Peat and other highly organic soil; **OH**: Organic clays of medium to high plasticity, organic silts; **M**: Inorganic silts and very fine sand; **C**: Inorganic clays; **O**: Organic silts and clays and organic matter; **L**: Silts and clays of low compressibility (Having a liquid limit less than 35); **ML**: Inorganic silt with low to medium compressibility; **H**: Silts and clays of high compressibility (Having a liquid limit greater than 50); **SC**: Clayey sand; sand-clay mixtures; **SM**: Silty sands; sand-silt mixtures; **SP**: Poorly graded sands; gravelly sands; little or no fines.

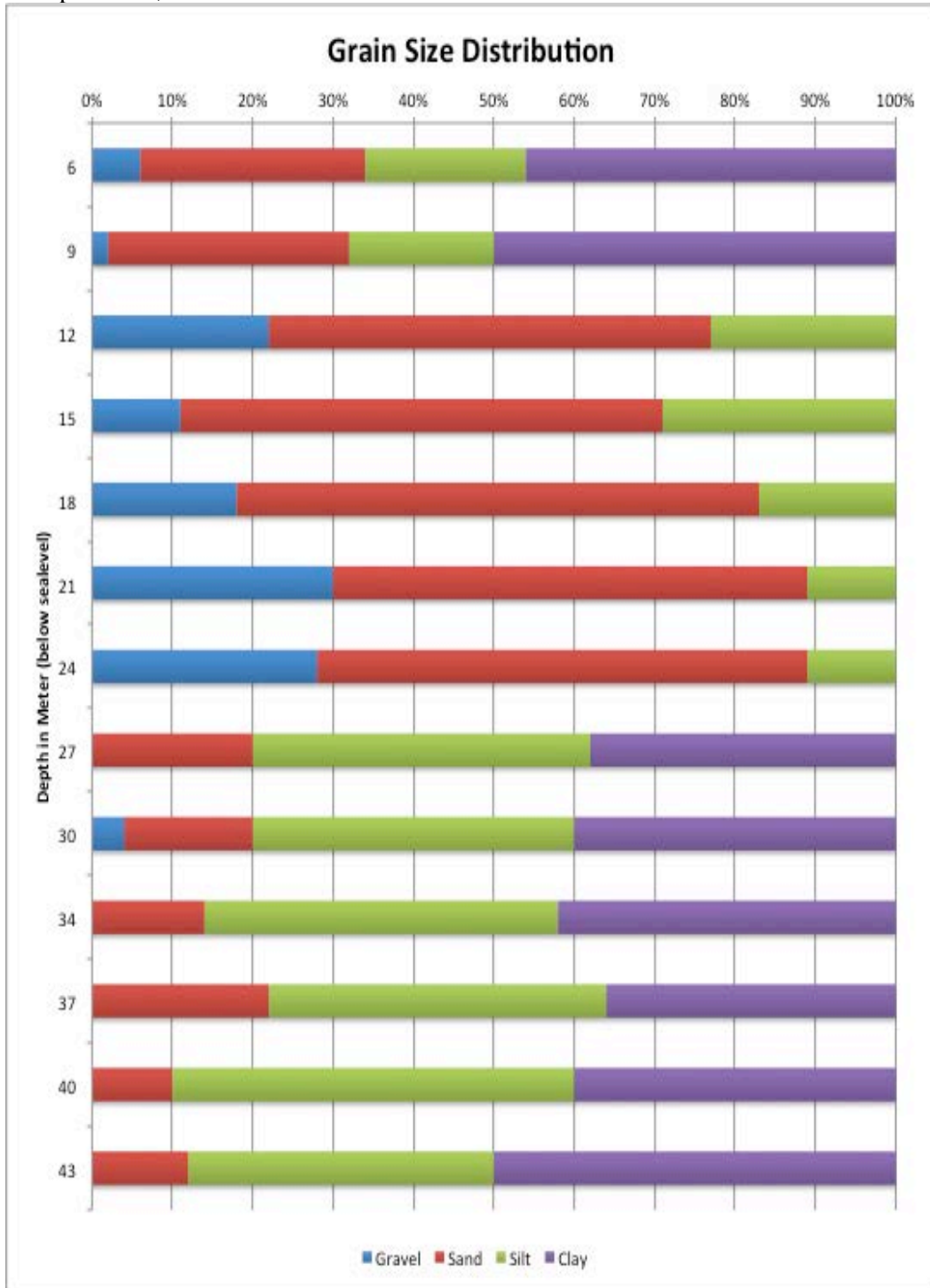


Figure 13: An overall grain size analysis involving representative samples showing the distribution of gravel-sand-silt-clay with depth (- 6 to - 43 meter). Hydrometer technique was used to determine silt and clay fraction. Notice a general decrease of gravel and sand from - 27 to -43 meter.

There seems to be a slight variation in the distribution of moisture and silt content in sandy layer (Figure 13). Typically, moisture percentage is between 20 and 10, while silt content ranges from 30 to 10 and is considered to be mechanically strong enough to withstand a load amounting to 1200 kips. A system of piles is planned to be used for foundation support. The placement of the piles may either be in clusters at each column location or through a distribution of piles over the site with rigid slabs and grade beams used to distribute the heavy loads among the pile supports.

5.1.2 Hydrogeological Constraints on Foundation Design

Furthermore, considering the shallow water table in close proximity to this facility site, pilings or a deep foundation will intersect the groundwater table and a dewatering phase should be considered once the construction begins [25], [26], and [27]. The significance of the current study is that it blends anthropogenic and soil data with sedimentology, regional geology and geoengineering parameters to assist the design and construction team to come up with the best engineering practice in order to ensure a stable foundation for the proposed New York City Agency facility to be built in Maspeth. Finally, New York City's overburden exhibits extreme variations in geotechnical parameters, as well as in the physical character of buried paleo-glacial and peri-glacial streams; these are a constant and ubiquitous challenge to construction. It is quite well known that the depth of the overburden deposits (mainly Pleistocene glacial, and in a few instances, early Cretaceous lacustrine) in close proximity to this site was largely constrained by the mechanical strength of the bedrock and inherent structural weaknesses associated with bedrock. Based on this consideration, weathered zones over the basement rocks (late Precambrian to early Cambrian) were found to vary extensively (16 m to 76 m and more) and certainly warranting detailed geoengineering consideration in order to arrive at a plausible and economically feasible construction design. Locked-in paleo-tectonic bedrock stresses have been observed and encountered during construction. Evidence has been found indicating pre-historic liquefaction and ground-rupture motions related to the major regional paleo-earthquakes. The presence of compressible soils has been found in pockets of glacio-lacustrine origin. Anthropogenic interference, including uncontrolled dumping of unconsolidated fills and toxic waste, has altered the natural physiography. When undertaking any new and large-scale capital construction projects in this area, one must address anthropogenic fill and resculpturing of surface topography [28], [29], [30], [31], and [32].

5.1.3 Dewatering and Potential Adverse Impacts

Taking variables such as soil type, consolidation factor, plasticity, and groundwater flow characteristics into geotechnical consideration are quite significant while attempting to arrive at a feasible design construction such as New York City Agency is contemplating. Soil types are often constrained by their parent rock composition and degree of weathering; regional geology information should be considered as one of the key geotechnical aspects, as evidenced from this investigation. Highly compressible soils, as noticed in this investigation, testify to the importance of delineating subsurface conditions before construction begins. Highly compressible (i.e., very weak) soils having very low density, high moisture content, and plasticity, as determined in this investigation, are susceptible to large settlements [33] and [34]. Due to weak mechanical strength, serious large-scale settlements can pose geoengineering hazards to a foundation. Mechanically

viable design construction requires further inspection of the subsurface rocks before undertaking any engineering project. Furthermore, even deep piling foundation design-anchored into the mechanically durable glacial sandy layer should take the high groundwater table into account, as some of the moisture-saturated overlying organic clay layer may dewater when exposed during the construction phase, and this may result in ponding of water or local flooding [35], [36], and [37].

6 Conclusions

Geoengineering data of the soil samples, including ASTM Classification, USCS, Atterberg limits, unconfined compressive strength (Figure 14), moisture content, organic constituents, and pH of the representative soil samples were evaluated prior to submitting a feasible design plan for a foundation facility in this area.

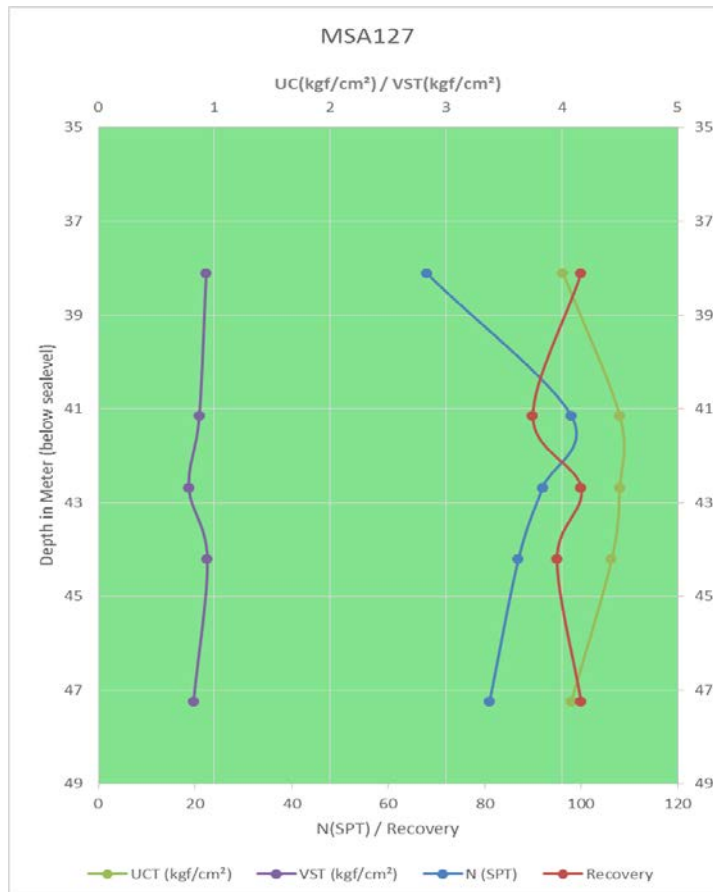


Figure 14: A summary plot showing distribution of Recovery % (based on 0.6 meter or 2 ft of coring), UCT (unconfined compressive test, measured in kilogram force per square centimeter), Vane Shear Test and N or SPT (standard penetration test or blow counts) conducted on Silty Clay layer below the Glacial Sand. Notice higher N or SPT for Silty Clay from – 41 meter suggesting more compaction and dewatering phenomena. Depth in meter (below sealevel).

Based on distinctive geoengineering attributes, including higher liquid limit, markedly low N (blow counts) values, increased organic and moisture contents, the upper highly compressible, cohesive, and plastic soil horizons (6 to 8 m below sea level) are not sufficiently strong to withstand the required loading, which is estimated at near 1200 kips for some locations (Figure 15). Presence of several feet of anthropogenic fill over organic-rich silt and clay layers does not have sufficient consolidation factor and further exacerbates the situation from a geotechnical point of view. The foundation support system will therefore have to be established in the glacial sand, possessing N (blow count) around 50 and liquid limit close to 30 (low plasticity). Considering greater homogeneity of glacial sand in this area in terms of moisture, liquid limit, plasticity index, and N values, contemplated sustainable foundation design with greater mechanical durability seemed to a viable option. However, sporadic presence of clayey to silty clay lenses intercalated with the glacial sand should be taken into account, and proper geotechnical adjustments need to be made before a final design proposal is submitted.

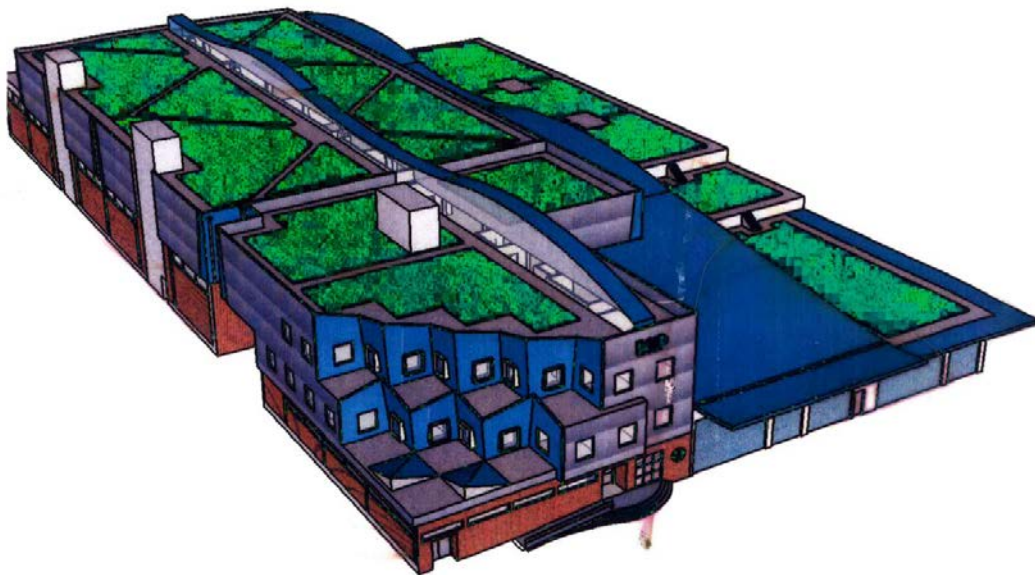


Figure 15: It is proposed to use the above architectural design plan for construction based on subsurface conditions. Foundation needs to be on Glacial Sand located below the compressible Organic layer to avoid structural failure.

Geoengineering characterization of representative samples coupled with regional geologic history and paleo-depositional environments also revealed an extensive weathered zone resembling saprolitic type capping the crystalline rocks. It was found to underlie the silty clay to clay layer, and its thickness is varied from a maximum of 19 m to a minimum of 6 m. An abnormal lateral variation of bedrock weathering can be attributed to the mechanical strength of the crystalline rock and associated structural fabric. Another notable factor associated with extensive weathering of bedrock is the lowering sea level that existed in this region during the last ice age (Pleistocene Glaciation), which in turn caused rapid erosion and forming of incised valleys into the bedrock due to lowering of base level as well. Soft, clayey weathered zone at top of the crystalline basement

belonging to the late Proterozoic to early Cambrian rocks, as thick as 32 m, has been known to occur in this region. Most of the bedrock (late Proterozoic to early Cambrian) in this geologic setting was subjected to at least two major tectonic events, namely, the late Ordovician Taconic and late Devonian Acadian episodes. These events were manifested in complex deformation, crumpling, medium to high-grade metamorphism, jointing, and secondary mineralization. Also, the authors observed a pronounced gradation in terms of bedrock composition, particularly associated with schists. Often it grades from muscovite-biotite-chlorite schist to garnetiferous-sillimanite schist, reflecting variable degrees of metamorphism and variation in composition of protoliths.

When construction, particularly of high rise buildings, is planned near estuaries in or around older municipalities, the probability of encountering buried estuary deposits should be considered because of possible effects on the load bearing ability of foundation soils. This experience with the Maspeth project illustrates the need for thorough geoscientific reconnaissance, investigation, and evaluation of the data obtained before planning such construction activities.

ACKNOWLEDGEMENTS: The current research was made possible by the involvement of Nazrul Khandaker and Stanley Schleifer from the City University of New York, York College as in-house geotechnical consultants of the New York City Department of Environmental Protection (NYCDEP). Detailed sedimentological and geotechnical parameters were analyzed at York College, and sincere appreciation is extended to the Geology discipline for providing usage of these facilities. Authors appreciate critical reviews and useful recommendations provided by anonymous reviewers concerning the improvement of the manuscript. Finally, the authors deeply acknowledge the late Dr. Charles Baskerville, an engineering geologist, educator, and author of four geologic quadrangles showing the greatest mapped detail yet produced for New York City, for his guidance and support in understanding the complexity of the overburden deposits and geoengineering aspects associated with bedrock geology of New York City.

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