GEOTECHNICAL INVESTIGATION

PROPOSED STORAGE FACILITIES - 125 COLONNADE ROAD SOUTH, OTTAWA, ONTARIO

ARCHITECTURE49 INC.

GEOTECHNICAL REPORT

PROJECT NO.: 211-11099-00
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1 INTRODUCTION

1.1 CONTEXT

WSP Canada Inc. (WSP) was retained by Architecture49 Inc. (A49) to provide geotechnical engineering services for the proposed development to be located on 125 Colonnade Road South in Ottawa, Ontario (hereinafter referred as "site") as shown on Drawing No. 1 in Appendix A. It is understood that the existing building will be converted into two separate storage facilities. To support the design services, WSP completed a geotechnical investigation at the proposed building locations.

The scope of work for this investigation is outlined in WSP's Proposal, dated September 10, 2021 and subsequent project correspondence.

1.2 OBJECTIVES AND LIMITATIONS

The current report was prepared at the request and for the sole use of A49 according to the specific terms of the mandate given to WSP. The use of this report by a third party, as well as any decision based upon this report, is under this party's sole responsibility. WSP may not be held accountable for any possible damages resulting from the third party's decisions based on this report.

Furthermore, any opinions regarding conformity with laws and regulations expressed in this report are technical in nature; the report is not and shall not, in any case, be considered as a legal opinion.

Information in this report is only valid for the borehole locations as described.

Reference should be made to the Limitations of this Report, attached in Appendix D, which follows the text but forms an integral part of this document.

1.3 PROJECT UNDERSTANDING AND SITE DESCRIPTION

1.3.1 SITE DESCRIPTION

The project site is located on 125 Colonnade Road South, southwest of the intersection of Colonnade Road and Prince of Wales Drive in the City of Ottawa, Ontario. Access to the project site is from Colonnade Road South. The existing Canadian National Railway corridor is located south of the subject site. An abandoned railway exists north of the active rail corridor and crosses part of the site. The general site location is shown on Drawing No. 1 in Appendix A.

Currently there is an existing warehouse, with several loading dock bays on the northern half of the site. Asphalt paved parking lots are present north and south of the existing building. The southern half of the site is surrounded by a steel wired fence. A large diameter sanitary sewer is present along the southern and eastern perimeters of the site; however, the exact location of the sewer could not be identified (by the City of Ottawa).

The existing ground surface at the subject site is generally flat and approximately at grade with Colonnade Road South to the north, and slightly below grade of the neighbouring property to the west. The ground surface east of the site slopes down toward a swale parallel with Prince of Wales Drive. A grass covered berm is located along the south perimeter of the property parallel to the active rail corridor. Tall trees exist on the southeastern perimeter of the site.

1.3.2 PROJECT DESCRIPTION

The proposed development includes construction of an addition abutting the south side of the existing building, which will contain self-storage units. This addition will include both 2- and 3-storey portions and have a footprint of approximately 50 m by 70 m. Further south on the property, a new single-storey standalone warehouse, 11 m in height, will be constructed with a footprint of approximately 50 m by 90 m. It is assumed that the existing paved areas will be rehabilitated as part of the new construction. The conceptual plan is shown on Drawing No. 2 provided in Appendix A.

Although preliminary in nature, the conceptual plan on Drawing No. 2 indicates that the abandoned railway spur will likely encroach on the footprint for both of the proposed storage facilities.

In general, the geotechnical scope of work included the following tasks:

- A desktop study reviewing the existing geotechnical information available in the general area;
- Drilling of exploratory boreholes within the study area;
- In-situ soil sampling and testing, including Standard Penetration Testing (SPT) and in-situ undrained shear vane testing;
- Obtaining soil samples for further review and laboratory testing;
- Geophysical testing;
- Geotechnical laboratory testing;
- Chemical analysis;
- Geotechnical analysis; and
- Preparation of this report, which presents the results of the investigation and provides geotechnical recommendations related to the design and construction of the proposed building addition.

Environmental investigations related to contaminated soil, bedrock or groundwater are not included as part of the current project.

2 SITE INVESTIGATION

2.1 DESKTOP STUDY

The study area is situated within the Physiographic Region known as the Ottawa Valley Clay Plains as identified in "The Physiography of Southern Ontario, Third Edition", by Chapman and Putnam.

Published Ontario Geological Survey (OGS) surficial geology maps indicate that the project area is underlain by fine-textured glaciomarine (glacial till) deposits, consisting of predominantly silt and clay with some sand and gravel. Bedrock geology maps indicate the bedrock in the general area includes dolostone and sandstone of the Beekmantown Group.

The Ministry of the Environment and Conservation and Parks (MOECP) well record database indicates 16 well records within an approximately 150 m radius of the project area. These records indicate clay, fine sand and gravel overburden soil. Limestone bedrock was noted at five of the well locations at depths ranging from approximately 14 m to 22 m below the ground surface.

Frost penetration depth within the study area is 1.8 m based on Ministry of Transportation's (MTO) Ontario Provincial Standard Drawing (OPSD) 3090.101.

2.2 FIELD INVESTIGATION

The geotechnical field exploration program was completed by WSP between November 17 and 30 2021 to supplement the geotechnical information acquired in the desktop study. This exploration program included overburden drilling and laboratory testing of selected soil samples.

Utility clearances were obtained for each of the borehole locations prior to the start of drilling.

A total of 8 boreholes (BH21-01 to BH21-08) were drilled near or within the footprint of the proposed buildings as shown on Drawing No. 2 in Appendix A. The borehole locations were selected based on the site accessibility and location of the known underground services.

- BH21-01 to 21-04 were advanced near or within the footprint of the proposed 2- to 3-storey (north) building; and,
- BH21-05 to 21-08 were advanced within the footprint of at the proposed single storey (south) building.

The boreholes were advanced using truck- and track-mounted CME hydraulic drill rigs, equipped with hollow stem augering equipment, supplied and operated by Ohlmann Geotechnical Services (OGS) of Almonte, Ontario and George Downing Estate Drilling (Downing) of Hawkesbury, Ontario.

The boreholes were advanced through the overburden soils to depths ranging from approximately 9.0 m to 9.8 m below the existing ground surface. In-situ tests including Standard Penetration Testing (SPT) and field shear vane testing were carried out at regular intervals, where possible.

During the field investigation, the drilling operations were supervised on a full-time basis by a member of WSP's geotechnical staff, who logged and visually classified the samples retrieved in the field, and documented the subsurface conditions encountered during the drilling process. This member of WSP's geotechnical staff then transported the soil samples collected during the drilling to WSP's geotechnical materials testing laboratory in Ottawa for further review by project engineer and laboratory testing.

A monitoring well consisting of a 50 mm diameter HDPE standpipe piezometer was installed at BH21-04 to permit monitoring of the stabilized groundwater subsequent to the drilling investigation. The groundwater level was measured on December 22, 2021.

The borehole logs from the current exploration program are included in Appendix B of this report. The ground surface elevation of each of the boreholes were not established, which is beyond the scope of this work.

2.3 LABORATORY TESTING

Upon completion of drilling and in-situ testing, soil samples were transported to WSP's laboratory for further examination, classification and testing. A laboratory testing program was carried out on selected representative soil samples, which included the determination of natural water content, grain size distribution and Atterberg limits (plasticity).

The results of natural water content tests and Atterberg limits (plasticity) are included on the relevant borehole logs in Appendix B. The results of Atterberg limits and grain size distribution testing are also presented in Appendix C.

One sample of soil was submitted to Eurofins Environmental Ltd. for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements. The results of the chemical analysis are provided in Appendix C.

2.4 GEOPHYISCAL TESTING

In addition to the above noted geotechnical investigation activities, a geophysical survey was carried out on November 9, 2021 to provide a seismic site classification to support the structural design of the proposed storage facilities. A Multichannel Analysis of Surface Waves (MASW) test was completed to provide a seismic shear wave velocity profile for this site. A full discussion of the geophysical survey and results is provided in Appendix B.

3 SUBSURFACE CONDITIONS

3.1 GENERAL

The following provides a general description of the major soil types encountered during the current geotechnical investigation. It should be noted that the following discussion includes some simplifications for the purposes of discussing broadly similar soil strata. It should also be noted that the differences in soil types and changes between various soil strata are often gradational, as opposed to precise boundaries of geological change.

A detailed description of the soil stratigraphy encountered at each borehole location is shown on the borehole log sheets in Appendix B. Please note that the factual descriptions shown in each borehole log takes precedence over the generalized (and simplified) descriptions presented below.

3.2 PAVEMENT STRUCTURE

Boreholes 21-1 to 21-5 and BH21-7 were advanced through the parking lot south of the existing building. The pavement structure at the northern portion of the parking lot, where the 2- to 3-storey building is being proposed (near BH21-1 to 21-04), consists of hot mix asphalt overlying sand and gravel fill. The pavement structure at the southern portion of the parking lot, where the single storey building will be located (near BH21-05 and 21-07) consists of sand and gravel fill at the existing ground surface.

Where encountered, the asphalt thickness ranged from 80 mm to 180 mm. The underlying granular fill within the northern portion of the parking lot extended to depths ranging from 0.2 m to 1.5 m below the existing ground surface, with thicknesses varying widely from 100 mm of 1,300 mm. The surficial granular fill encountered at the boreholes advanced within the southern portion of the parking lot is about 300 mm thick. The granular material was relatively uniform and a distinction between granular base and granular subbase could not be made in the boreholes.

One SPT was carried out within the granular fill layer. A SPT 'N' value of 20 blows per 305 mm of penetration was recorded, indicating a compact relative density.

Grain size distribution analysis was carried out on two selected samples of the granular fill. The results of the analyses are presented in Appendix C and summarized in Table 3.1 below. Note the soil samples were recovered using a 50 mm diameter split spoon sampler, which does not include coarse gravel, cobbles and boulder; therefore, the results of this distribution test only represent the samples with soil particles smaller than 50 mm in diameter.

Table 3.1 Results of Grain Size Analyses for Granular Fill

Borehole No.	Sample No.	Description	Grain Size Distribution		
			% Gravel	% Sand	% Fines (Silt & Clay)
BH21-04	AS-1	Sand and Gravel, Fill	41	44	15
BH21-07	AS-1	Gravel and Sand, Fill	61	37	2

Natural water content of the two tested samples of the granular fill was determined to be 5 percent and 9 percent, respectively.

3.3 TOPSOIL

Topsoil was encountered at the existing ground surface at boreholes BH21-06 and BH21-08, which were advanced through the landscaping areas in the southern portion of the site. The thickness of topsoil was approximately 150 mm at both boreholes.

3.4 SILTY CLAY

A deposit of sensitive silty clay was encountered below the granular fill and topsoil layers at all boreholes. This deposit generally consists of interlayered clay, silty clay and silt. For simplicity this deposit is referred to in this report as silty clay, as this is the predominant soil type. Some thin (i.e., approximately 50 mm thick) sand layers were encountered intermittently within the silty clay deposit. The silty clay deposit extended to the borehole termination depth in all boreholes except BH21-02, BH21-04 and BH21-05.

The upper portion of the silty clay has been weathered to form a brown-grey crust. The weathered zone extended to depths ranging from approximately 2.1 m to 4.4 m below existing ground surface. The SPT 'N' values within the weathered crust ranged from 3 blows to 18 blows per 305 mm of penetration (generally decreasing with depth). In-situ shear vane tests were attempted within the weathered crust, however the undrained shear strength of weathered silty clay exceeded the maximum limit of the field vane (i.e., greater than 116 kPa). Based on field observations, the weathered crust has a very stiff consistency.

The silty clay below the depth of weathering was brown-grey to grey in colour. SPT 'N' values recorded in the unweathered silty clay ranged from 0 blows to 2 blows per 305 mm of penetration. In-situ shear vane tests yielded undrained shear strength values within the unweathered silty clay ranging from 42 kPa to 116 kPa, indicating a firm to very stiff consistency.

Four samples of the silty clay were selected for Atterberg limits testing; two samples from the weathered crust and the other two from the unweathered silty clay. The Atterberg limits test results are summarized in Table 3.2 below and provided in Appendix C. Based on the results of the Atterberg Limit testing, the samples of both the weathered crust and unweathered silty clay would be classified as low-plasticity clay (CL).

Natural water content of selected samples of the silty clay ranged from 40 percent to 80 percent, generally in excess of its liquid limit values.

Table 3.2 Results of Atterberg Limits Testing for Silty Clay

Borehole No.	Sample No.	Description	Atterberg Limits (% moisture)			
			Liquid Limit	Plastic Limit	Plasticity Index	Water Content
BH21-04	SS-4	Weathered Crust	50	19	31	62
BH21-07	SS-2	Weathered Crust	39	15	24	46
BH21-04	SS-7B	Unweathered Silty Clay	34	17	17	61
BH21-07	SS-6	Unweathered Silty Clay	29	13	16	58

3.5 CLAYEY SAND

Layers of clayey sand were encountered at varying depths within and/or below the silty clay deposit in all boreholes. The clayey sand layers contained some silt and a trace of gravel, and interbedded with the silty clay deposit. The clayey sand extended to the borehole termination depth in boreholes BH2-02, BH21-04 and BH21-05.

SPT 'N' values recorded in the clayey sand ranged from 0 blows to 4 blows per 305 mm of penetration, indicating very loose to loose relative density. The very low SPT 'N' values recorded in the clayey sand layers are likely due to the presence of hydrostatic pressure and blow black of the soils.

One sample of the clayey sand was selected for grain size distribution analysis. The analysis results are summarized in Table 3.3 below and presented in Appendix C.

Table 3.3 Results of Grain Size Distribution Analysis for Clayey Sand

Borehole No.	Sample No.	Description	Grain Size Distribution			
			% Gravel	% Sand	% Silt	% Clay
BH21-04	SS-6	Clayey Sand	1	58	18	23

One sample of the clayey sand was selected for Atterberg limits testing. The Atterberg limits test results are summarized in Table 3.4 below and provided in Appendix C. Based on the results of the Atterberg Limit testing, the sample was non-plastic.

Natural water content of four samples of the clayey sand was determined to range from 25 percent to 41 percent.

Table 3.4 Results of Atterberg Limits Testing for Clayey Sand

Borehole No.	Sample No.	Description	Atterberg Limits (% moisture)			
			Liquid Limit	Plastic Limit	Plasticity Index	Water Content
BH21-07	SS-5	Clayey Sand	Non-plastic			31

3.6 GROUNDWATER CONDITIONS

A monitoring well consisting of a 50mm diameter HDPE standpipe piezometer was installed at BH21-04 to permit monitoring of the stabilized groundwater level subsequent to the drilling investigation. The groundwater level was measured at 2.1 m below the existing ground surface on December 22, 2021, about 3 weeks following completion of drilling.

Water level was also measured in the open boreholes following drilling at BH21-01, BH21-03, BH21-05, BH21-07 and BH21-08 at depths varying from 1.3 m to 6.1 m below the existing ground surface. Minor cave-in of the borehole sidewalls at borehole BH21-02 occurred following drilling and prevented water level measurement in the open borehole at this location. It should be noted that given the composition of the soils at this site, water infiltration rate is expected to be low and therefore the observed water levels in the open boreholes may not be representative of long-term stabilized groundwater levels at this site.

It should also be also noted that groundwater levels can vary and are subject to seasonal fluctuations as well as fluctuations in response to major weather events.

3.7 SUMMARY

A summary of the soil and groundwater conditions encountered within the borehole locations is presented in Table 3.4 below. It should be noted that sand seams and interbedded clayey sand layers of varying thicknesses (approximately 50 mm to 2.1 m) were encountered within the silty clay; for simplicity these have been omitted in the stratigraphy summary below. Refer to the borehole logs in Appendix B for further information.

Table 3.4 Simplified Stratigraphy and Groundwater Depths

Borehole No.	Simplified Stratigraphy (Depth in metres below existing ground surface)					Measured Groundwater Depth (m)	Notes
	Topsoil	Asphalt	Granular Fill	Silty Clay			
				Weathered Crust	Unweathered		
21-01	-	0 – 0.2	0.2 – 1.5	1.5 - 4.4	4.4 - 9.0	1.3[1]	Borehole terminated at 9.0 m. Water level measured in open borehole. Clayey sand layers observed between approximately 5.3 m and 7.0 m depth.
21-02	-	0 - 0.2	0.2 – 0.8	0.8 - 4.4	4.4 - 8.4	-	Borehole terminated at 9.0 m. Clayey sand layers observed from approximately 4.9 m to 5.2 m and 8.4 m to 9.0 m depth. Borehole caved and prevented water level measurement in open borehole.
21-03	-	0 – 0.1	0.1 – 0.8	0.8 – 4.4	4.4 – 9.0	6.1[1]	Borehole terminated at 9.0 m. Water level measured in open borehole. Clayey sand layer observed between approximately 5.3 and 7.5 m depth.
21-04	-	0 – 0.1	0.1 – 0.2	0.2 – 3.7	3.7 – 8.2	2.1[2]	Borehole terminated at 9.0 m. Water level measured in piezometer. Clayey sand layers observed from approximately 4.9 m to 6.4 m and 8.2 m to 9.0 m depth.
21-05	-	-	0 – 0.3	0.3 – 3.7	3.7 – 7.5	6.0[1]	Borehole terminated at 9.0 m. Water level measured in open borehole. Clayey sand layers observed between approximately 5.3 m to 6.9 m and 7.5 m to 9.0 m depth.
21-06	0 – 0.2	-	-	0.2 – 3.7	3.7 – 9.0	-	Borehole terminated at 9.0 m. Clayey sand layer observed from approximately 6.9 m to 7.5 m depth.
21-07	-	-	0 – 0.3	0.3 – 2.1	2.1 – 9.8	5.9[1]	Borehole terminated at 9.8 m. Water level measured in open borehole. Clayey sand layer observed from approximately 4.4 m to 5.9 m depth.
21-08	0 – 0.2	-	-	0.2 – 3.7	3.7 – 9.8	6.1[1]	Borehole terminated at 9.8 m. Water level measured in open borehole. Clayey sand layer observed from approximately 6.1 m to 6.7 m depth.

Notes:
[1] Water level measured in open borehole following drilling.
[2] Water level measured in standpipe piezometer installed within borehole.

4 RECOMMENDATIONS

4.1 GENERAL

This section of the report provides engineering guidance related to the geotechnical design aspects of the project based on our interpretation of the available information described herein and our understanding of the project requirements. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the factual information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities. Reference should be made to the Limitations of this Report, attached in Appendix D, which follows the text but forms an integral part of this document.

The general subsurface conditions encountered in the boreholes within the project limits include topsoil or flexible pavement structure, which in turn is underlain by silty clay deposits with sand seams and interbedded clayey sand layers.

4.2 SEISMIC CONSIDERATIONS

4.2.1 LIQUEFACTION

The soils at the site may be susceptible to seismic liquefaction based on the soil types encountered at the site, the SPT 'N' values collected within these soils and the groundwater level observed at the site. The clayey sand layer may liquefy under seismic loading. Further analysis and testing is recommended to confirm.

Based on the results of the current investigation, the site is underlain by a deposit of silty clay, interlayered with very loose to loose clayey sand.

Seismic liquefaction occurs when earthquake vibrations cause an increase in pore water pressures within the soil. The presence of increased/excess pore water pressures reduces the effective stress between the soil particles, and the soil's frictional resistance to shearing. This phenomenon, which leads to a temporary reduction in the shear strength of the soil, may cause:

- Large lateral movements of even gently sloping ground, referred to as 'lateral spreading'.
- Reduced shear resistance (i.e., bearing capacity) of soils which support foundations, as well as reduced resistance to sliding.
- Reduced shaft resistance for deep foundations as well as reduced resistance to lateral loading.
- Buoyant uplift of buried structures (such as tanks or sewer pipes).

In addition, 'seismic settlements' may occur once the vibrations and shear stresses have ceased. Seismic settlement is the process whereby the soils stabilize into a denser arrangement after an earthquake, causing potentially large surface settlements.

The following conditions are more prone to experiencing seismic liquefaction:

- Granular soils, rather than cohesive soils (i.e., more probable for sands and silts than for clay).
- Soils having a loose state of packing.
- Soils located below the groundwater level.

An assessment of the liquefaction potential of the clayey sand deposits was carried out using the Seed and Idriss (1971) simplified procedure based on SPT N60-values from the boreholes. The SPT N-values reported on the borehole records were corrected for overburden stress, rod length during sampling, and hammer energy efficiencies. The results of this assessment suggest that the native submerged clayey sand would be classified as liquefiable under an earthquake with a magnitude of 6.4 (Ottawa area specified design value) and a peak 'firm ground' acceleration of 0.30 g.

The anticipated settlement of the liquefiable native clayey sand under the analyzed earthquake event could be up to 125 mm. The amount of settlement is highly dependant on the earthquake event, the thickness of the deposit and its liquefaction potential, and therefore settlements could be highly variable and differential across the building footprint. If the foundations are founded above or within these materials, then the structure should be designed to accept this differential settlement without experiencing collapse. It should be noted that guarding against collapse (i.e., allowing for 'safe exit') is considered to be the objective of design for earthquake conditions (recognizing that the 'design' earthquake has a return period of 2,475 years), though the structure may be damaged and rendered unserviceable.

The seismic settlements would be in addition to the anticipated settlements under static loading, which are discussed in Section 4.6 of this report.

4.2.2 SEISMIC SITE CLASSIFICATION

The seismic design provisions of the Ontario Building Code (OBC) depend, in part, on the shear wave velocity of the upper 30 m (V_s30) of soil and/or rock below founding level. The OBC also permits the Site Class to be estimated from the standard penetration test N values.

Multichannel Analysis of Surface Waves (MASW) geophysical testing was completed at the subject site to develop a seismic shear wave velocity profile of the upper 30 m of the soil or bedrock. This testing was used to develop a site-specific seismic site classification to support the structural design of the proposed storage facilities.

Two MASW Lines (numbered 1 and 2) were completed for V_s30 determination from the existing ground surface. Following field data collection and data analysis, the average V_s30 values for Lines 1 and 2 were estimated to be 229 m/s and 208 m/s, respectively.

According to Table 4.1.8.4.A of the current OBC, the site classification for seismic site response would be Site Class D for foundations placed on engineered fill or directly native weathered silty clay. Reference should be made to the technical memorandum in Appendix B for full discussion of geophysical testing methodology and results.

Regardless of the liquefaction potential of the clayey sand layer, a Site Class D can still be used if the proposed structure will have a fundamental period of vibration of less than or equal to 0.5 seconds. If the fundamental period of the proposed structure is greater than 0.5 seconds, then a Site Class F would apply.

4.3 SITE PREPARATION AND GRADING

At this time, only preliminary design details of the proposed buildings are available. It has been assumed that the desired finished grade elevation will be within 1 m of the existing ground surface, therefore it is anticipated that limited or no engineered fill will be required to be placed and compacted within the building areas. However, for design purposes, a grade raise restriction of 1.5 m should be applied based on our assessment of the subsurface conditions encountered at this site to limit the potential post-construction settlement within the underlying silty clay.

The existing fill and topsoil are not considered suitable for support of the building foundations and should be removed from the entire building footprint. At the completion of the topsoil stripping and fill removal and prior to any placement of new fill, the exposed clay subgrade within the proposed building footprint should be reviewed by qualified geotechnical personnel. Any loose, disturbed or unsuitable areas should be removed and replaced with compacted engineered fill meeting the requirement described later in this report and to the satisfaction of geotechnical personnel.

In addition, all stripping and earthwork activities should be performed in a manner consistent with good erosion and sediment control practices. Prior to placement of any new granular material, the exposed subgrade should be inspected and approved by qualified personnel to ensure drainage is maintained across the proposed building foundation footprint.

Proof-rolling could disturb the exposed silty clay subgrade within the proposed building footprint and is therefore not recommended.

4.4 MATERIAL REUSE

The native soils at this site are not considered to be suitable for reuse as structural engineered fill. However, these soils could be reused as general earth borrow in non-structural areas (i.e., landscaping) depending upon its environmental suitability, which is not included as part of this assignment.

4.5 ENGINEERED FILL

Prior to placing engineered fill, where required, the exposed subgrade should be inspected by qualified geotechnical personnel to confirm that the exposed soils are suitable, undisturbed and have been adequately cleaned of ponded water and all disturbed, loosened, softened, organic and other deleterious material. Remedial work (i.e., further sub-excavation and replacement) should be carried out as directed by geotechnical personnel.

Imported materials to be used for engineered/structural fill should be approved by geotechnical personnel. In this regard, the imported materials, which meet the requirements for OPSS Granular A or Granular B Type II, would be suitable for use as engineered fill below footings or other foundation elements.

The approved materials for engineered fill should be placed in maximum 300 mm loose lifts and be uniformly compacted to at least 98 percent of its Standard Proctor Maximum Dry Density (SPMDD) using suitable vibratory compaction equipment. The placement of engineered fill must be monitored by qualified geotechnical personnel on a full-time basis. The top surface of the engineered fill should be protected as necessary from construction traffic and should be sloped to provide positive drainage for surface water during the construction period.

The upper surface of the engineered fill should extend to a minimum of 1 m outside of the outer edge of the exterior building foundation envelope (in all directions) and should be sloped downward and outward at no steeper than 1 horizontal to 1 vertical (1H:1V). Engineered fill slopes that will become permanently exposed fill slopes at the development, if any, should be flattened to 2H:1V or flatter, and should be covered with topsoil and sodded or otherwise treated to reduce erosion. Maintenance will be required over the first several years until the vegetative mat has taken root.

Imported materials, which meet the requirements for OPSS earth borrow, would be suitable for use as general fill in non-structural areas. This fill should be compacted to at least 95 percent of its SPMMD. The placement of the fill should be monitored by geotechnical personnel on a regular basis. Placement of the upper 450 mm should be monitored on a full-time basis.

4.6 FOUNDATIONS

4.6.1 FROST PROTECTION

The native soils at this site are considered frost susceptible. Therefore, the bearing surfaces below all footings should be protected against frost action by providing sufficient earth cover. Minimum earth cover of 1.5 m is considered acceptable for foundation elements attached to heated structures. Isolated, unheated footings adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum of 1.8 m of earth cover.

Insulating the bearing surface with high density insulation could be considered as an alternative to earth cover for frost protection. Further details can be provided if and when required

In the event that foundations are to be constructed during the winter months, foundation soils are required to be protected from freezing temperatures using suitable construction techniques. Therefore, the base of all excavations should be insulated immediately upon exposure, until the time that heat can be supplied to the building interior and/or the foundations have sufficient earth cover to prevent freezing of the subgrade soils, following review by qualified geotechnical personnel.

4.6.2 BEARING RESISTANCES

It is understood that the proposed addition, abutting the south side of the existing building, will have 2- and 3-storey portions. The proposed new warehouse further south will be a single storey structure. Based on the results of the subsurface exploration program, the proposed buildings could be supported on shallow spread footing foundations within 1.5 m of the existing ground surface. At this depth, it is anticipated that the underside of the footing level will be on or within the native, weathered silty clay crust.

For strip and pad footing widths between 0.6 m and 1.5 m, the following bearing resistances may be assumed:

- The unfactored ultimate geotechnical bearing resistance can be taken as 300 kilopascals (kPa). A resistance factor of 0.5 should be applied to this value, yielding a factored bearing resistance of 150 kPa at Ultimate Limit States (ULS).
- The geotechnical resistance at the Serviceability Limit State (SLS) can be taken as 100 kPa.

Provided that the foundation subgrade is properly prepared, and not unduly disturbed by construction activities, the total and differential post-construction settlements associated with the above SLS resistance value are expected to be less than 25 mm and 15 mm, respectively. The settlement of the new footings will be 100 percent differential with respect to the existing structure, which should be considered, especially wherever the addition will be connected to the existing structure.

It should be noted that where the new foundation elements are located in the vicinity of the existing exterior footings, the existing foundation wall backfill may exist. The existing fill material is not considered suitable for support of the new foundation elements. This backfill should be removed and replaced with engineered fill consisting of OPSS Granular B Type II, placed in maximum 300 mm lifts and compacted to at least 98 percent of the material's SPMDD.

In addition, although preliminary in nature, the conceptual plan on Drawing No. 2 in Appendix A indicates that the abandoned railway spur will likely encroach the footprint of the proposed storage facilities. The existing fill material along the railway spur is not considered suitable to support the future buildings and should therefore be removed from within the building footprint. Where the resulting excavation leaves the native subgrade level below the proposed underside of footing level, the grade should be raised, within the zone of influence of the footing, with engineered fill placed and compacted in accordance with Section 4.5.

The silty clay subgrade will be sensitive to disturbance by construction traffic, especially in the presence of water. To reduce the potential disturbance to the underlying silty clay, it should be planned to place a mud slab consisting of lean concrete on the subgrade surface upon completion of excavation (and subgrade approval of) the silty clay subgrade.

All bearing surfaces should be reviewed and approved at the time of construction by a geotechnical engineer, who is familiar with the findings of this investigation and the design and construction of similar projects prior to placement of any concrete, backfill, etc.

4.6.3 SLIDING RESISTANCES

The sliding resistance can be calculated using the following unfactored friction coefficients given in the table below:

Table 4.1 Unfactored Friction Coefficients

Condition	Unfactored Friction Coefficient, tan σ	Adhesion, c_a
Between Concrete and native very stiff to stiff weathered Silty Clay	0.40	45 kPa
Between Concrete and Engineered Fill	0.55	-

4.6.4 ADJACENT EXISTING FOUNDATIONS

The new addition foundations should be sized in such a way that the loads imposed on the soil by the new footings will not increase the stress on soil under the existing building foundations. At the time of preparation of this report, the foundation information for both the existing and new buildings are not available.

Where new foundations are located within the zone of influence of the existing building foundations, the existing foundations will likely be subjected to a new round of settlement which will be 100 percent differential with respect to the remainder of the original structure. The magnitude of that additional settlement is dependent on the size, depth, design bearing pressure of the existing footings as well as the size, depth, location, and design bearing pressure of the new footings. A settlement assessment could be carried out if the above information is provided and requested.

Care should be exercised during construction to avoid undermining the existing foundations. Temporary shoring or underpinning of the existing foundations maybe required depending on the depth of the new foundations.

4.6.5 ALTERNATIVE FOUNDATIONS – HELICAL PILES

If shallow spread footing foundations cannot provide sufficient bearing resistances for the required foundation loads, helical piles could be a suitable foundation alternative. Helical piles are a type of piled foundation that is bored into the subsoils, rather than driven as with conventional pile foundations.

Helical piles are a less conventual foundation system for new construction, although they have been in use since the 1830's and are mainly used to underpin existing foundations. They can offer significant advantages in terms of speed of installation and reduction in the offsite disposal of drilling spoil. Most helical piles/piers are proprietary foundation systems with each supplier providing their own structural capacity design. The helical pile installation contractor will need to provide an engineered stamped shop drawing to the design team for review prior to installation.

Helical piles are steel shafts with one or more helices welded to the shafts that provide a self-tapping mechanism during installation. Helical piles can be used to resist both compression and tension loads. There are several different shaft sections available. They can be made of either circular or square steel shafts sections, which could be either hollow or solid. The hollow shafts could be filled with concrete or grout following installation to increase their capacity. Shaft diameters range from 50 mm up to 600 mm and helix diameters range from 150 mm up to 1,200 mm depending on capacity requirements. After the piles are installed, the piles are then structurally connected to a perimeter grade beam or interior column pad typically using brackets, plates or other structural connection, or could be casted within these concrete elements.

For this project, depending on the structural loads and based on past experience, it is anticipated that either square shaft size SS5 or round shaft size RS2875 or equivalent helical piles would be feasible to support the required structural loads. On site testing during installation can confirm the capacity of the individual helical piles. As a preliminary guidance, these helical piles are expected to have the following geotechnical resistances within the upper silty clay below frost depth:

Table 4.2 Preliminary Helical Pile Capacities

Bearing Strata	Helical Pile Section	Compression Capacity (kN)		Tension Capacity (kN)	
		ULS	SLS	ULS	SLS
Stiff Silty Clay below frost depth, 1.8 m	SS5 (38 mm x 38 mm)	150	100	65	50
	RS2875 (73 mm dia.)				

It is recommended that the helical piles extend through any potentially liquefiable soil layers (e.g. loose to very loose clayey sand) to resist settlements caused by seismic liquefaction, should they occur.

4.7 SLABS-ON-GRADE

For predictable performance of the floor slab, the underslab subgrade and engineered fill should be prepared as previously described in Sections 4.3 and 4.5 of this report. The subgrade of the slab-on-grade should be reviewed by qualified geotechnical personnel prior to placement of any granular fill or concrete.

Provision should be made for at least 200 mm of OPSS Granular A to form the base for the floor slab.

A modulus of subgrade reaction value for the slab subgrade may be required by the structural engineer. A value of 15,000 kN/m^3 may be used provided at least 200 mm of OPSS Granular A is placed beneath the floor slab.

4.8 FOUNDATION WALL BACKFILL

The native soils at this site are frost susceptible and should not be used as backfill against exterior or unheated foundation elements (e.g., footing, foundation walls, etc.). To avoid problems with frost adhesion and heaving, these foundation elements should be backfilled with non-frost-susceptible sand and/or gravel, which meets that gradation requirements for OPSS Granular A or Granular B Type I or II; and,

It should be noted that the use of 19 mm clear crushed stone, even with the use of non-woven geotextile, as foundation backfill may result in unfavourable growing conditions for plant matter placed in overlying topsoil and therefore is not recommended.

Foundation backfill should be placed in shallow lifts, not exceeding 200 mm loose thickness, and compacted to at least 98 percent of its SPMDD where it is supporting any structures or services, or at least of 95 percent of its SPMDD in other areas using suitable vibratory compaction equipment.

To avoid damaging or laterally displacing the structures, care should be exercised when compacting fill adjacent to new structures. Heavy equipment should be kept a minimum of 1 m away from the structure during backfilling. The 1 m width adjacent to the wall should be compacted using hand-operated equipment unless otherwise authorized.

In areas where pavement or other hard surfacing will be in contact with the buildings, differential frost heaving could occur between the granular fill and other areas. To reduce this differential heaving, the backfill adjacent to the wall should be placed to form a frost taper. The frost taper should be brought up to pavement subgrade level from 1.5 m below finished exterior grade at a slope of 3H:1V or flatter, away from the wall. The backfill should be placed in maximum 200 mm thick lifts and should be compacted to at least 95 percent of the material's SPMDD using suitable vibratory compaction equipment.

The pavement or hard surfacing could be expected to perform better in the long term if the granular backfill against the foundation walls is positively drained. Drainage of the wall backfill can be provided by means of a perforated pipe subdrain in a surround of 19 mm clear stone, fully wrapped in geotextile, which leads by gravity drainage to an adjacent storm sewer or sump pit.

4.9 LATERAL EARTH PRESSURES

4.9.1 STATIC LATERAL EARTH PRESSURE

The static lateral earth pressure acting on foundation walls or retaining walls, if required, may be calculated using the following expression:

$$\sigma_h(z) = K(\gamma z + q)$$

Where:

$\sigma_h(z)$ = Lateral earth pressure (kPa) acting at depth z;

K = Earth pressure coefficient; for unrestrained walls and structures where some movement is acceptable (such as retaining walls) use a coefficient of active earth pressure (K_a) equal to 0.3, for restrained walls use the coefficient of earth pressure at rest (K_0) equal to 0.5;

γ = Unit weight of the backfill; use 21.5 kN/m^3 for compacted granular backfill;

z = Depth to the point of interest (m); and,

q = The magnitude of uniform surcharge at the ground surface to account for traffic, equipment, or stockpile soil (use 15 kPa).

The above values assume free-draining granular backfill will be used. If this is not the case, then the above values may need to be adjusted based on the soil type used, and hydrostatic pressures should be considered in the calculation of lateral earth pressures. WSP can provide additional guidance based on actual building plans if required.

The passive resistance offered by the foundation wall backfill soils could also be considered in evaluating the lateral earth pressure applied to the foundations. The magnitude of that lateral resistance will depend on the backfill materials and backfill conditions adjacent to the foundation walls. If the backfill materials consist of compacted sand or sand and gravel (meeting OPSS Granular B Type I) as discussed herein, then the passive resistance acting on the foundation wall may be taken as:

$$\sigma_h(z) = K_p (\gamma z + q)$$

Where:

$\sigma_h(z)$ = Lateral earth resistance (kPa) applied to the foundation wall at depth z (kPa);

K_p = Passive earth pressure coefficient, use 3.0;

γ = Unit weight of retained soil, use 21.5 kN/m^3;

z = Depth below top of wall (m);

q = The magnitude of uniform surcharge at the ground surface to account for traffic, equipment, or stockpile soil (use 15 kPa).

This resistance is provided in an unfactored format. Factoring of the calculated resistance value will be required if the design is being carried out using Limit States Design.

Movement of the backfill and wall is required to mobilize the passive resistance. As a preliminary guideline, approximately 75 mm of movement would be required to mobilize the passive resistance.

4.9.2 SEISMIC LATERAL EARTH PRESSURE

Lateral earth pressures will be higher under seismic loading conditions. In order to account for earthquake-induced dynamic pressures, the combined pressure distribution during a seismic event (including both the seismic and static components) may be assumed to be:

$$\sigma_h(z) = K \gamma z + (K_{AE} - K) \gamma (H-z)$$

Where:

$\sigma_h(z)$ = Total lateral earth pressure at depth z (kPa);

K = Static earth pressure coefficient; at-rest earth pressure coefficient (0.5) for restrained walls and active earth pressure coefficient (0.3) for unrestrained wall;

γ = Unit weight of soil, use 21.5 kN/m^3 for granular fill;

K_{AE} = Seismic earth pressure coefficient (use 0.53 for non-yielding walls, 0.4 for yielding walls);

H = Total height of the wall (m); and,

z = Depth below the top of the wall (m).

The above lateral earth pressure values (both static and seismic) are unfactored values. Factoring of the calculated resistance value will be required if the design is being carried out using Limit States Design.

4.10 SITE SERVICES

Construction of new site services and/or relocation of existing site services are expected to be within the existing fill or native silty clay. Details of the proposed site services are not available at this time; however, it is assumed that they will include localized trenches throughout the site. Trenches can be temporarily supported using sloped excavations or trench boxes as outlined in Section 4.12.1 of this report.

Bedding for site services should consist of at least 150 mm of OPSS Granular A compacted to at least 95 percent of its SPMDD. Where wet or disturbed conditions are encountered in the base of the trench, it may be necessary to over-excavate and place a sub-bedding layer consisting of 300 mm of compacted OPSS Granular B Type II beneath the Granular A or thicken the Granular A bedding. The use of clear stone as a bedding and cover material is not recommended as the finer particles of the native soils and backfill may migrate into the voids of the clear stone, resulting in loss of pipe support.

Cover material above the spring line should consist of OPSS Granular A or Granular B I material with a maximum particle size of 25 mm. Cover material should be compacted to a minimum of at least 95 percent of the material's SPMDD.

Trench backfill may consist of additional granular fill, or properly moisture conditioned native silty clay, and should be placed in maximum 300 mm loose lifts and compacted to at least 95 percent of its SPMDD (98 percent if below structures). Where backfill is within the frost depth, the backfill profile (above the minimum cover required to 1.8 m depth) in the trench should be made to match the native soils on either side as much as is practical in order to minimize the potential for differential frost heave. As a result, portions of the silty clay above the water table may be retained, moisture conditioned (if necessary) and re-used.

The deeper and unweathered portions of the silty clay may be too wet to be compactable. If these materials are excavated from the service trenches, they should ideally be wasted.

Any service trenches which extend below the water table should have clay cut-offs installed across the trench at regular intervals (typically 100 m) to prevent the trench acting as a drain and lowering the groundwater table in the general area. These cut-offs should extend the full width of the trench and must completely penetrate the bedding, cover and any other granular materials in the trench.

The above are general guidelines for typical site services. All services installations should be completed in accordance with the relevant OPSS's and OPSD's for the particular application and size. WSP can provide additional review during detailed design based on the actual services proposed if required.

4.11 PAVEMENT DESIGN

It is understood that pavement construction will be required for the parking lot and access lanes at the subject site. A heavy-duty pavement structure is anticipated to be required for heavier vehicles (e.g., trucks). A light duty pavement structure is generally acceptable for areas where only light vehicles are anticipated.

Table 4.3 Recommended Pavement Structure Thickness

Pavement Layer	Heavy Duty Vehicles	Light Vehicles Only
Hot Mix Asphalt	40 mm SP 12.5 50 mm SP 19.0	50 mm HL3 or SP 12.5
OPSS Granular A Base	150 mm	150 mm
OPSS Granular B Subbase	400 mm	300 mm

All pavement granular materials should be placed in maximum 300 mm thick loose lifts and compacted to minimum 98% SPMDD. Placement and compaction of all granular and asphalt materials should be reviewed at the time of construction by qualified geotechnical personnel.

4.12 CORROSIVITY POTENTIAL

Two soil samples were submitted to Eurofins for testing to assess soil corrosivity and potential exposure of buried concrete elements to sulphate attack. The results of these tests are summarized below.

Table 4.4 Results of Soil Corrosivity Testing

Borehole/ Sample No.	Soil Type	Chloride (%)	pH	Conductivity (ms/cm)	Resistivity (ohm-cm)	Sulphate (%)
BH21-02 / SS2	Silty Clay	0.036	7.50	1.00	1000	0.05
BH21-08 / SS2	Silty Clay	0.022	7.91	0.58	1720	<0.01

The test results obtained from the soil samples suggest a moderately to severely corrosive environment for buried steel structures. These values must be taken into consideration during design of below-grade steel elements, such as piling or underground services.

The test results indicate a low soluble sulphate content and therefore sulphate resistant Portland cement is not required.

The Eurofins Certificate of Analysis and testing results are included in Appendix C.

4.13 CONSTRUCTION CONSIDERATIONS

It is understood that excavation work will be required as part of the overall construction of the new buildings. Where work is required near the existing and new structural elements, the following recommendations are provided:

4.13.1 TEMPORARY EXCAVATIONS

All excavations should be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA), Part III of Ontario Regulation 213/91.

The soils within the expected excavation include topsoil, fill and silty clay above the groundwater level (about 2 m depth). These soils above the groundwater level or depth of dewatering can be classified as Type 3 soils, or Type 4 soils if excavation extends below the groundwater table. Excavations within Type 3 soil require side slopes with a minimum gradient of 1H:1V and excavations within Type 4 soil require side slopes of 3H:1V or flatter. These classifications must be reviewed and confirmed by a qualified person during excavation.

If space restrictions exist, the excavations could be carried out within fully braced steel trench box, which would reduce the width of excavation and therefore the backfill reinstatement that is required.

Stockpiling of soil beside excavations made in the silty clay should be avoided; the weight of the stockpiled soil could lead to basal instability of braced excavations or slope instability for unsupported excavations.

Layers of clayey sand was encountered within and/or below the silty clay deposit in all boreholes between depths of 4.9 m and at least 8.9 m (borehole termination depth). If excavation is deeper than 2 m depth and extend near the underlying clayey sand, basal heave could potentially be an issue. Basal heave can result when only a limited thickness of low-permeability soil (e.g., clay) beneath the base of the excavation is underlain by higher permeability soil (e.g., clayey sand at this site) with high groundwater pressures. This condition can result in a disturbed/destabilized subgrade, which would not be suitable for support of the sewer pipe, and the recompression of which, upon backfilling, would lead to unacceptable ground settlement. Depressurization of the clayey sand may be required to be carried out in advance of the excavation to prevent basal heave, which should be confirmed once the servicing plan becomes available.

4.13.2 TEMPORARY DEWATERING

The groundwater levels observed in the open boreholes following drilling were found to range from 1.3 m to 6.1 m below the existing ground surface. This groundwater level may not accurately represent the stabilized groundwater level, since insufficient time was available for the groundwater level to stabilize. It is anticipated that the observation taken in the monitoring well at BH21-04 of 2.1 m below existing ground surface is a closer representation of long-term groundwater level, however as previously noted in Section 3.5 groundwater levels can fluctuate seasonally and in response to weather events.

Assuming that excavation for the new construction will be above the observed groundwater level (about 2 m depth), any groundwater inflows encountered within the excavations would be expected to be low and manageable by pumping from closely spaced, properly filtered sumps.

The actual rate of groundwater inflow to the base of excavation will depend on many factors, including the contractor's schedule and rate of excavation, the size of the excavation, the number of working areas being excavated at one time, and the time of year at which the excavation is made. Also, there may be instances where volumes of precipitation, surface runoff and/or groundwater collects in an open excavation, and must be pumped out. Ministry of the Environment, Conservation and Parks (MOECP) Environmental Activity and Sector Registration (EASR) will be required if construction dewatering is more than 50,000 L/day to up to 400,000 L/day. A Permit to Take Water (PTTW) will be required for dewatering in excess of 400,000 L/day. A hydrogeological assessment is beyond the scope of this work.

It should be noted that this discussion applies to groundwater flows. An assessment of the requirements for surface water diversion should be made by others.

The soils present at the site are expected to be sensitive to disturbance and proper control of the groundwater infiltration (by construction of sumps, use of well points, etc.) will be required to prevent excessive disturbance. Failure to adequately control groundwater inflows may result in disturbance of the subgrade and a need for over-excavation and replacement of disturbed subgrade soil.

4.13.3 SUBGRADE PREPARATION

The geotechnical bearing resistances provided in Section 4.6.2 assume that the foundation soils will not be disturbed by construction activities. Proper dewatering and protection of the exposed soil subgrades will be important during the construction of the foundations. All excavated surfaces should be kept free of frost, water, etc. during the course of construction. All excavated surfaces should be inspected by a qualified geotechnical engineer, who is familiar with the findings of this investigation and the design and construction of similar structures.

The foundations soils at the site are expected to be sensitive to disturbance from ponded water and construction traffic. If the subgrade for the foundations and floor slab will be exposed for a prolonged duration and/or exposed to construction traffic, then placement of a mud slab directly on the subgrade may be required to protect the subgrade from disturbance.

4.13.4 WINTER CONSTRUCTION

Should construction be carried out during freezing temperatures, the exposed frost susceptible subgrade should be protected immediately using one or a combination of: straw, propane heaters, polystyrene insulation, insulated tarpaulins, or other suitable means that prevent the underlying soil from freezing, which could cause frost heave.

5 CLOSURE

WSP should be retained for a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not accorded this opportunity to make this review, WSP assumes no responsibility for the interpretation of the recommendations contained in this report.

The geotechnical fieldwork for this assignment was completed by M. Banhoro, EIT and O. Benkirane, CPI. Reporting was completed by N. Christie, P.Eng. A technical review was completed by C. Ko, P.Eng. and M. Elsayed, P.Eng.

Recommendations presented in the body of this report have been made based on our present understanding of the project requirements.

We trust this information satisfies the requirements of A49 at this time. Should you have any questions regarding the contents of this report, please do not hesitate to contact the undersigned.

WSP Canada Inc.

Report prepared by:

Nathan Christie, P. Eng.
Geotechnical Engineer

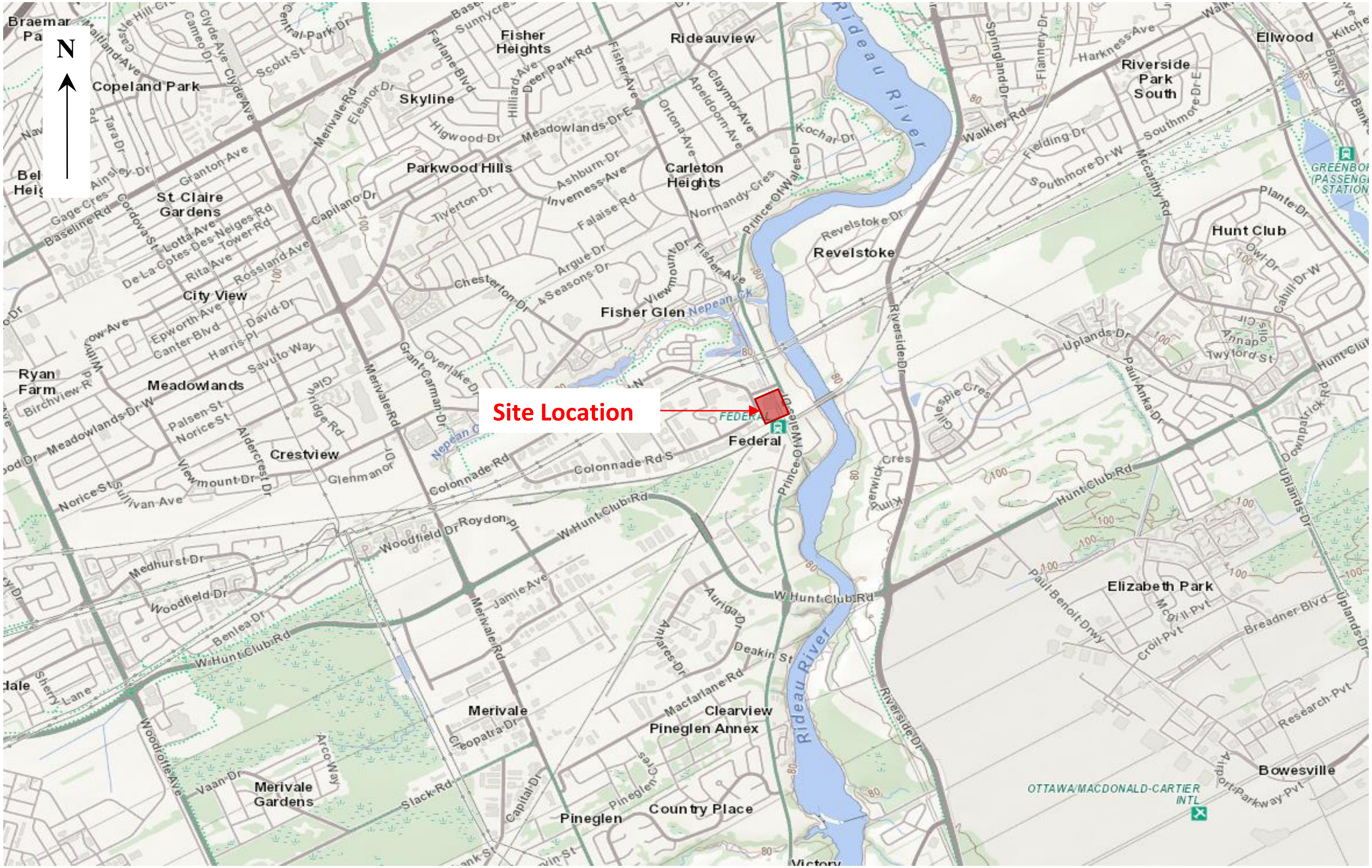
Reviewed by:

Mo Elsayed, P. Eng.
Senior Geotechnical Engineer

APPENDIX

A

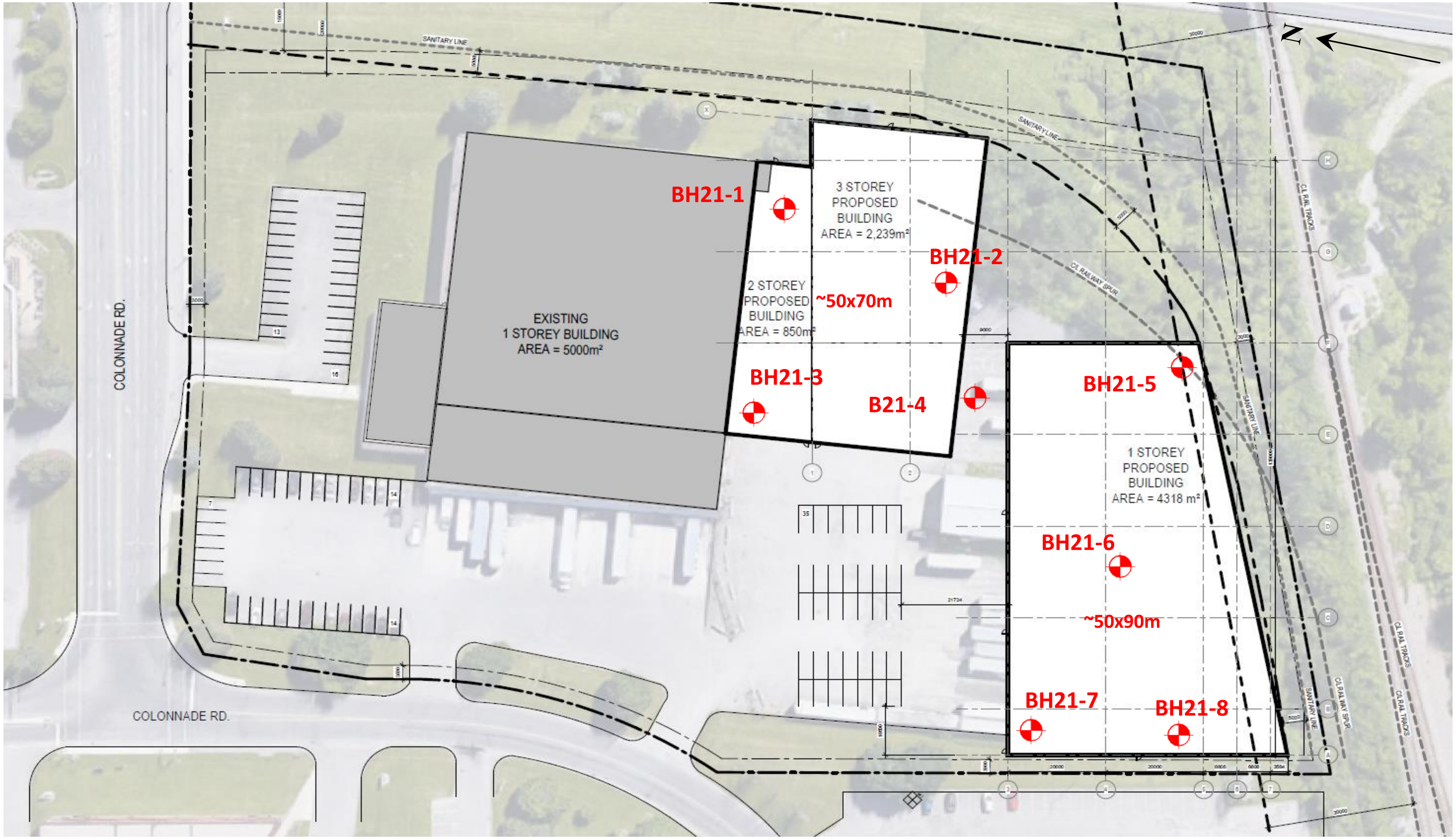
DRAWINGS



Client:	A49	Title:	Site Location Plan
Project#:	211-11099-00	DWG #:	1
Drawn:	NC	Approved:	BG
Date:	November 2021	Scale:	N. T. S.
Size:	Letter	Rev:	1

Project: Geotechnical Investigation
125 Colonnade Road

wsp



Client:		A49		Title:	Borehole Location Plan
Project#:	211-11099-00	DWG #:	2	Project:	Geotechnical Investigation 125 Colonnade Road
Drawn:	NC	Approved:	BG		
Date:	November 2021	Scale:	N. T. S.		wsp
Size:	Letter	Rev:	1		

APPENDIX

B

BOREHOLE LOGS

EXPLANATION OF TERMS USED IN REPORT

GEOPHYSICAL SURVEY RESULTS

APPENDIX B

BOREHOLE LOGS
EXPLANATION OF TERMS USED IN REPORT

BOREHOLE DRILLING RECORD : **BH21-01**

Prepared by: **Othmane Benkirane** Date (Start): **2021-11-29**
Reviewed by: **Christine Ko** Date (End): **2021-11-29**

Project Name: **Geotechnical Investigation**
Site: **125 Colonnade Road South**
Sector: **Proposed Storage Facilities**
Client: **Architecture49**

Project Number: **211-11099-00**
Geographic Coordinates: X = 445054.58 mE
Y = 5021531.03 mN
Surface Elevation: Not measured
Plunge / Azimuth:

Drilling Company: OGS
Drilling Equipment: CME 75
Drilling Method: Hollow Stem Auger
Borehole Diameter: 152.4 mm
Drilling Fluid: N/A

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
Water Level Free Phase

SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

GEOLOGY / LITHOLOGY | ANALYSIS | GEOTECHNICAL | WELL

GEOTECHNICAL plot scales: R □ Shear (kPa) I ■ 30 60 90 120; SPT=N Value ▲; PENTEST △; RQD (%); PLASTIC LIMIT w (%) LIQUID 20 40 60 80

DEPTH / ELEVATION (m)	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
	Ground surface.							
0.15	**ASPHALT (150 mm).**							
1.45	**FILL: SAND AND GRAVEL,** trace fines, brown, moist, compact.				AS-1	Remoulded		
					SS-1	Remoulded	30	12 10 10 9 (20)
4.42	**WEATHERED CRUST: SILTY CLAY,** trace sand, brown-grey, moist, very stiff.				SS-2	Remoulded	50	1 2 2 5 (4)
					SS-3	Remoulded	100	2 4 5 6 (9)
					SS-4	Remoulded	100	3 4 3 6 (7)
					SS-5	Remoulded	100	2 3 3 5 (6)
5.33	**SILTY CLAY,** trace sand, brown-grey, moist, stiff to very stiff. *Thin sand layer (50 mm).* *Thin sand layer (50 mm).*							
	CLAYEY SAND, some silt, grey, wet, loose to very loose.				SS-6	Remoulded	80	2 2 2 3 (4)

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0

BOREHOLE DRILLING RECORD : **BH21-01**

Prepared by: **Othmane Benkirane** Date (Start): **2021-11-29**
Reviewed by: **Christine Ko** Date (End): **2021-11-29**

Project Name:	**Geotechnical Investigation**	Project Number:	**211-11099-00**
Site:	**125 Colonnade Road South**	Geographic Coordinates:	X = 445054.58 mE Y = 5021531.03 mN
Sector:	**Proposed Storage Facilities**	Surface Elevation:	Not measured
Client:	**Architecture49**	Plunge / Azimuth:	

Drilling Company:	OGS
Drilling Equipment:	CME 75
Drilling Method:	Hollow Stem Auger
Borehole Diameter:	152.4 mm
Drilling Fluid:	N/A

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
Water Level   Free Phase

SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

DEPTH / ELEVATION (m)	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
	CLAYEY SAND, some silt, grey, wet, loose to very loose.	SS-7A			SS-7		100	0 0 0 0 (0)
6.40	**SILTY CLAY,** trace sand, grey, wet, stiff to very stiff.	SS-7B						
6.71	**CLAYEY SAND,** some silt, grey, wet, very loose.	SS-8A			SS-8		100	0 1 0 1 (1)
7.01	**SILTY CLAY,** trace sand, grey, wet, stiff to very stiff.	SS-8B						
	Thin sand layer (50 mm).				SS-9		100	1 0 1 0 (1)
8.99								

GEOTECHNICAL: R □ Shear (kPa) I ■ 30 60 90 120; SPT=N Value ▲; PENTEST △; RQD (%); PLASTIC LIMIT w (%) LIQUID 20 40 60 80

Depth scale (m): 6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10.0, 10.5, 11.0, 11.5, 12.0

COMMENTS:
- Water level was measured in open borehole at a depth of 1.3 m after borehole termination.
- The undrained shear strength of the silty clay at depths of 5.0 m and between 7.8 m and 8.1 m exceed the maximum limit of the vane (116 kPa).
- The low blow counts in the clayey sand layers are assumed to be due to hydrostatic pressure and blow back of the soils.

End of borehole at 8.99 m.

BOREHOLE DRILLING RECORD : **BH21-02**

Prepared by: **Othmane Benkirane**
Reviewed by: **Christine Ko**

Date (Start): **2021-11-30**
Date (End): **2021-11-30**

Project Name: **Geotechnical Investigation**
Site: **125 Colonnade Road South**
Sector: **Proposed Storage Facilities**
Client: **Architecture49**

Project Number: **211-11099-00**
Geographic Coordinates: X = 445052.76 mE, Y = 5021493.26 mN
Surface Elevation: Not measured
Plunge / Azimuth:

Drilling Company: OGS
Drilling Equipment: CME 75
Drilling Method: Hollow Stem Auger
Borehole Diameter: 152.4 mm
Drilling Fluid: N/A

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
Water Level — Free Phase

SAMPLE TYPE
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ANALYSIS
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GSA - Grain Size Analysis
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PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

GEOTECHNICAL: R □ Shear (kPa) 30 60 90 120 I ■; SPT=N Value ▲; PENTEST △; RQD (%); PLASTIC LIMIT w (%) LIQUID 20 40 60 80

DEPTH / ELEVATION (m)	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)	WELL DIAGRAM
	Ground surface.								
0.00–0.18	**ASPHALT (180 mm).**								
0.18–0.76	**FILL: SAND AND GRAVEL,** trace fines, brown, moist.				AS-1	Remoulded			
0.76–4.42	**WEATHERED CRUST: SILTY CLAY,** trace sand, brown-grey, moist, very stiff.				SS-1	Remoulded	80	2 4 6 9 (10)	
					SS-2	Remoulded	80	5 8 10 11 (18)	
					SS-3	Remoulded	90	4 6 8 7 (14)	
					SS-4	Remoulded	100	3 4 5 5 (9)	
					SS-5	Remoulded	100	3 4 4 5 (8)	
4.42–4.88	**SILTY CLAY,** trace sand, brown-grey, moist, very stiff to stiff.	SS-6A			SS-6	Remoulded	100	2 2 2 4 (4)	
4.88–5.18	**CLAYEY SAND,** some silt, brown-grey, wet, loose to very loose.	SS-6B							
5.18–	**SILTY CLAY,** trace sand, grey, wet, stiff to very stiff. *Thin sand layer (50 mm).*				SS-7	Remoulded	100	1 1 0 1 (1)	

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0

BOREHOLE DRILLING RECORD : **BH21-02**

Prepared by: **Othmane Benkirane** Date (Start): **2021-11-30**
Reviewed by: **Christine Ko** Date (End): **2021-11-30**

Project Name: **Geotechnical Investigation**
Site: **125 Colonnade Road South**
Sector: **Proposed Storage Facilities**
Client: **Architecture49**

Project Number: **211-11099-00**
Geographic Coordinates: X = 445052.76 mE, Y = 5021493.26 mN
Surface Elevation: Not measured
Plunge / Azimuth:

Drilling Company: OGS
Drilling Equipment: CME 75
Drilling Method: Hollow Stem Auger
Borehole Diameter: 152.4 mm
Drilling Fluid: N/A

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
Water Level / Free Phase

SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

DEPTH / ELEVATION (m)	DESCRIPTION	TYPE & NO.	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
	SILTY CLAY, trace sand, grey, wet, stiff to very stiff.	SS-8	100	0 0 0 1 (0)
		SS-9	100	4 3 2 2 (5)
8.38	**CLAYEY SAND,** some silt, grey, wet, loose to very loose.	SS-10	15	3 2 2 1 (4)
8.99				

GEOTECHNICAL: R □ Shear (kPa) 30 60 90 120 I ■; SPT=N Value ▲; PENTEST △; RQD (%); PLASTIC LIMIT w (%) LIQUID 20 40 60 80

Depth scale: 6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10.0, 10.5, 11.0, 11.5, 12.0

COMMENTS:
- The borehole caved after auger removal preventing water level measurement.
End of borehole at 8.99 m.

BOREHOLE DRILLING RECORD : **BH21-03**

Prepared by: **Mohamadou Banhoro** Date (Start): **2021-11-18**
Reviewed by: **Christine Ko** Date (End): **2021-11-18**

Project Name: **Geotechnical Investigation**
Site: **125 Colonnade Road South**
Sector: **Proposed Storage Facilities**
Client: **Architecture49**

Project Number: **211-11099-00**
Geographic Coordinates: X = 445013.39 mE, Y = 5021520.11 mN
Surface Elevation: Not measured
Plunge / Azimuth:

Drilling Company: George Downing Estate Drilling Ltd
Drilling Equipment: CME 75
Drilling Method: Hollow Stem Auger
Borehole Diameter: 152.4 mm
Drilling Fluid: N/A

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
Water Level · Free Phase

SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

DEPTH ELEVATION (m)	DESCRIPTION	TYPE & NO.	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
	Ground surface.			
0.10	**ASPHALT (100 mm).**			
0.76	**FILL: SAND AND GRAVEL,** trace fines, brown, moist.	AS-1		
4.42	**WEATHERED CRUST: SILTY CLAY,** trace sand, brown-grey, moist, very stiff.	SS-1	100	2 3 2 3 (5)
		SS-2	100	1 2 3 4 (5)
		SS-3	100	2 2 2 2 (4)
		SS-4	100	1 2 2 2 (4)
		SS-5	100	1 2 2 2 (4)
5.33	**SILTY CLAY,** trace sand, brown-grey, moist, firm.			
	CLAYEY SAND, some silt, brown-grey, wet, loose to very loose.	SS-6	100	1 2 2 2 (4)

GEOTECHNICAL: R □ / Shear (kPa) 30 60 90 120 / I ■; SPT=N Value ▲; PENTEST △; RQD (%); PLASTIC LIMIT w (%) LIQUID 20 40 60 80

BOREHOLE DRILLING RECORD : **BH21-03**

Prepared by: **Mohamadou Banhoro** Date (Start): **2021-11-18**
Reviewed by: **Christine Ko** Date (End): **2021-11-18**

Project Name:	**Geotechnical Investigation**	Project Number:	**211-11099-00**
Site:	**125 Colonnade Road South**	Geographic Coordinates:	X = 445013.39 mE Y = 5021520.11 mN
Sector:	**Proposed Storage Facilities**	Surface Elevation:	Not measured
Client:	**Architecture49**	Plunge / Azimuth:	

Drilling Company:	George Downing Estate Drilling Ltd
Drilling Equipment:	CME 75
Drilling Method:	Hollow Stem Auger
Borehole Diameter:	152.4 mm
Drilling Fluid:	N/A

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
Water Level Free Phase

SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

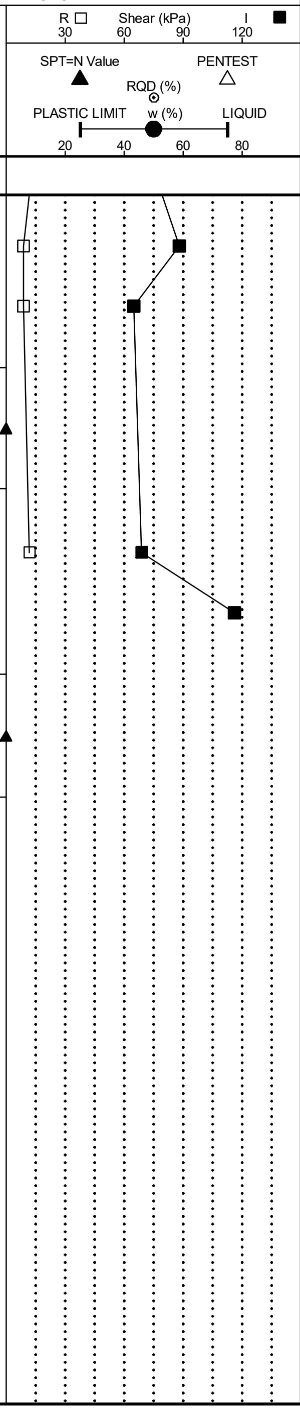
ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

DEPTH / ELEVATION (m)	STRATIGRAPHY	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
		CLAYEY SAND, some silt, brown-grey, wet, loose to very loose.				SS-7		100	0 0 0 0 (0)
7.47		**SILTY CLAY,** trace sand, grey, wet, stiff to very stiff.				SS-8		100	0 0 0 0 (0)
8.99									

COMMENTS:
- Water level was measured in open borehole at a depth of 6.1 m.
- The field vane tests between depths of 5.3 m and 7.5 m were probably completed in clayey sand layers and are not representative of the consistency of silty clay.
- The undrained shear strength of the silty clay at 8.1 m exceed the maximum limit of the vane (116 kPa).
- The low blow counts in the clayey sand layers are assumed to be due to hydrostatic pressure and blow back of the soils.
End of borehole at 8.99 m.

GEOTECHNICAL



WELL
DIAGRAM

Depth scale (m): 6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10.0, 10.5, 11.0, 11.5, 12.0

BOREHOLE DRILLING RECORD : **BH21-04**

Prepared by: **Othmane Benkirane** Date (Start): **2021-11-29**
Reviewed by: **Christine Ko** Date (End): **2021-11-30**

Project Name: **Geotechnical Investigation**
Site: **125 Colonnade Road South**
Sector: **Proposed Storage Facilities**
Client: **Architecture49**

Project Number: **211-11099-00**
Geographic Coordinates: X = 445033.19 mE, Y = 5021478.01 mN
Surface Elevation: Not measured
Plunge / Azimuth:

Drilling Company: OGS
Drilling Equipment: CME 75
Drilling Method: Hollow Stem Auger
Borehole Diameter: 152.4 mm
Drilling Fluid: N/A

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth : 7.92 m
Length : 1.52 m
Opening : 20 mm
WATER Elevation: m
WATER Date: 2021-12-22
Water Level / Free Phase

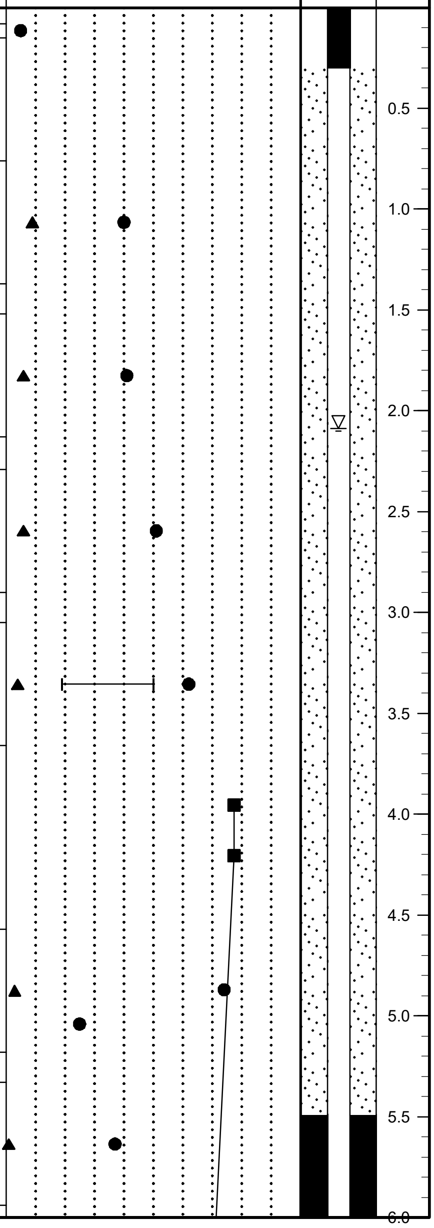
SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

GEOTECHNICAL: R / I Shear (kPa) 30, 60, 90, 120; SPT=N Value; PENTEST; RQD (%); PLASTIC LIMIT, w (%), LIQUID 20, 40, 60, 80

DEPTH / ELEVATION (m)	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
	Ground surface.							
0.08	**ASPHALT (80 mm).**							
0.18	**FILL: SAND AND GRAVEL,** some fines, brown, moist.				AS-1			
	WEATHERED CRUST: SILTY CLAY, trace sand, brown-grey, moist, very stiff.				SS-1		60	1 4 5 5 (9)
					SS-2		100	2 3 3 5 (6)
					SS-3		100	2 3 3 3 (6)
					SS-4		100	1 2 2 2 (4)
3.66	**SILTY CLAY,** trace sand, brown-grey, moist, very stiff.							
	Becoming wet, grey.	SS-5A			SS-5		100	0 2 1 2 (3)
4.88	**CLAYEY SAND,** some silt, grey, wet, very loose.	SS-5B						
					SS-6		100	1 0 1 1 (1)



wsp

Prepared by: **Othmane Benkirane** Date (Start): **2021-11-29**
Reviewed by: **Christine Ko** Date (End): **2021-11-30**

Project Name:	**Geotechnical Investigation**	Project Number:	**211-11099-00**
Site:	**125 Colonnade Road South**	Geographic Coordinates:	X = 445033.19 mE Y = 5021478.01 mN
Sector:	**Proposed Storage Facilities**	Surface Elevation:	Not measured
Client:	**Architecture49**	Plunge / Azimuth:	

Drilling Company:	OGS
Drilling Equipment:	CME 75
Drilling Method:	Hollow Stem Auger
Borehole Diameter:	152.4 mm
Drilling Fluid:	N/A

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth : 7.92 m
Length : 1.52 m
Opening : 20 mm
WATER Elevation: m
WATER Date: 2021-12-22
Water Level Free Phase

SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

DEPTH / ELEVATION (m)	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
6.40	**CLAYEY SAND,** some silt, grey, wet, very loose.	SS-7A			SS-7	100	0 0 1 1 (1)
8.23	**SILTY CLAY,** trace sand, grey, wet, stiff to very stiff.	SS-7B					
					SS-8	100	3 3 2 1 (5)
8.99	**CLAYEY SAND,** some silt, grey, wet, very loose.				SS-9	100	2 1 1 2 (2)

GEOTECHNICAL: R □ Shear (kPa) 30 60 90 120 I ■; SPT=N Value ▲; PENTEST △; RQD (%); PLASTIC LIMIT w (%) LIQUID 20 40 60 80

WELL: DIAGRAM

Depth scale: 6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10.0, 10.5, 11.0, 11.5, 12.0

COMMENTS:
- Groundwater level in standpipe piezometer was measured at 2.1 m below existing ground surface on December 22, 2021.
- The undrained shear strength of the silty clay between depths of 3.9 m and 4.2 m exceed the maximum limit of the vane (116 kPa).
- The low blow counts in the clayey sand layers are assumed to be due to hydrostatic pressure and blow back of the soils.
End of borehole at 8.99 m.

BOREHOLE DRILLING RECORD : **BH21-05**

Prepared by: **Mohamadou Banhoro** Date (Start): **2021-11-18**
Reviewed by: **Christine Ko** Date (End): **2021-11-18**

Project Name:	**Geotechnical Investigation**	Project Number:	**211-11099-00**
Site:	**125 Colonnade Road South**	Geographic Coordinates:	X = 445054.35 mE Y = 5021440.92 mN
Sector:	**Proposed Storage Facilities**	Surface Elevation:	Not measured
Client:	**Architecture49**	Plunge / Azimuth:	

Drilling Company:	George Downing Estate Drilling Ltd
Drilling Equipment:	CME 75
Drilling Method:	Hollow Stem Auger
Borehole Diameter:	152.4 mm
Drilling Fluid:	N/A

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
Water Level   Free Phase

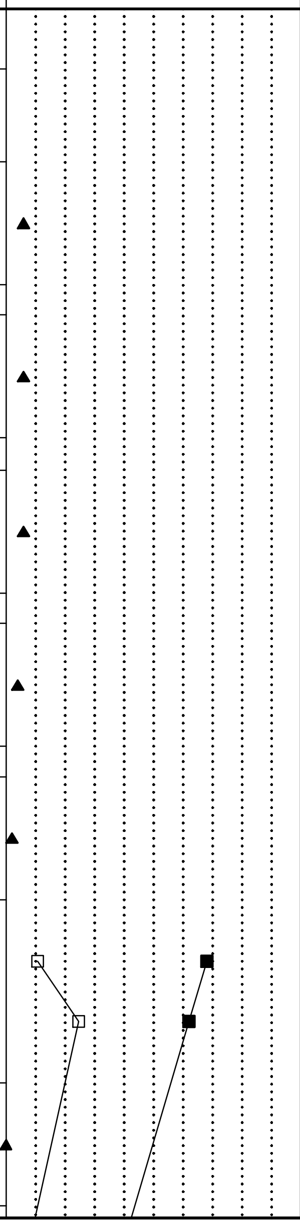
SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

DEPTH *ELEVATION* (m)	STRATIGRAPHY	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
		Ground surface.							
0.30		**FILL: SAND AND GRAVEL,** trace fines, brown, moist.				AS-1			
3.66		**WEATHERED CRUST: SILTY CLAY,** trace sand, brown-grey, moist, very stiff.				SS-1		100	2 2 4 6 (6)
						SS-2		100	2 3 3 4 (6)
						SS-3		100	2 3 3 3 (6)
						SS-4		100	1 2 2 2 (4)
5.33		**SILTY CLAY,** trace sand, brown-grey, moist, very stiff to stiff.				SS-5		100	1 1 1 1 (2)
		CLAYEY SAND, some silt, brown-grey, wet, very loose.				SS-6		100	0 0 0 0 (0)

GEOTECHNICAL
R □ Shear (kPa) I ■
30 60 90 120
SPT=N Value PENTEST
RQD (%)
PLASTIC LIMIT w (%) LIQUID
20 40 60 80



WELL
DIAGRAM

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0

BOREHOLE DRILLING RECORD : **BH21-05**

Prepared by: **Mohamadou Banhoro**
Reviewed by: **Christine Ko**
Date (Start): **2021-11-18**
Date (End): **2021-11-18**

Project Name:	**Geotechnical Investigation**	Project Number:	**211-11099-00**
Site:	**125 Colonnade Road South**	Geographic Coordinates:	X = 445054.35 mE Y = 5021440.92 mN
Sector:	**Proposed Storage Facilities**	Surface Elevation:	Not measured
Client:	**Architecture49**	Plunge / Azimuth:	

Drilling Company:	George Downing Estate Drilling Ltd
Drilling Equipment:	CME 75
Drilling Method:	Hollow Stem Auger
Borehole Diameter:	152.4 mm
Drilling Fluid:	N/A

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
Water Level   Free Phase

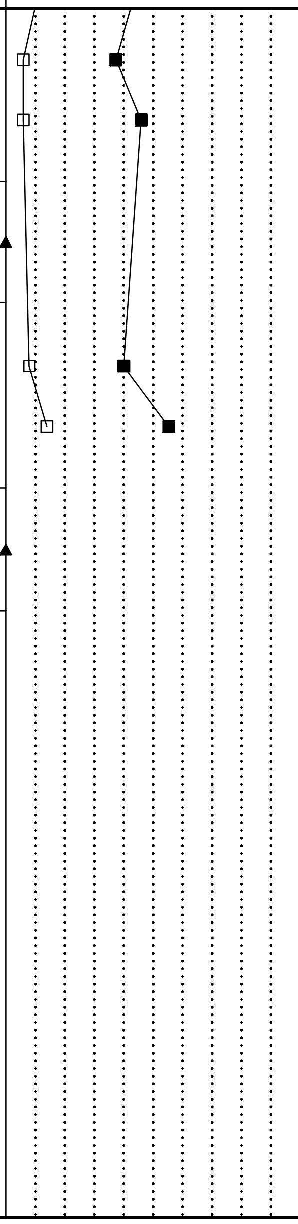
SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

DEPTH / ELEVATION (m)	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
6.86	**CLAYEY SAND,** some silt, brown-grey, wet, very loose.							
7.46	**SILTY CLAY,** trace sand, grey, wet, stiff.				SS-7		100	0 0 0 0 (0)
8.99	**CLAYEY SAND,** some silt, brown-grey, wet, very loose.				SS-8		100	0 0 0 0 (0)

GEOTECHNICAL
R □ Shear (kPa) I ■
SPT=N Value PENTEST
RQD (%)
PLASTIC LIMIT w (%) LIQUID



Depth scale (m): 6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10.0, 10.5, 11.0, 11.5, 12.0

COMMENTS:
- Water level was measured in open borehole at a depth of 6.0 m after borehole termination.
- The field vane tests between depths of 6.2 m and 8.1 m were probably completed in clayey sand layers and are not representative of the consistency of the silty clay.
- The low blow counts in the clayey sand layers are assumed to be due to hydrostatic pressure and blow back of the soils.

End of borehole at 8.99 m.

BOREHOLE DRILLING RECORD : **BH21-06**

Prepared by: **Mohamadou Banhoro**
Reviewed by: **Christine Ko**
Date (Start): **2021-11-17**
Date (End): **2021-11-17**

Project Name: **Geotechnical Investigation**
Site: **125 Colonnade Road South**
Sector: **Proposed Storage Facilities**
Client: **Architecture49**

Project Number: **211-11099-00**
Geographic Coordinates: X = 445011.11 mE, Y = 5021435.91 mN
Surface Elevation: Not measured
Plunge / Azimuth:

Drilling Company: George Downing Estate Drilling Ltd
Drilling Equipment: CME 75
Drilling Method: Hollow Stem Auger
Borehole Diameter: 152.4 mm
Drilling Fluid: N/A

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
Water Level — Free Phase

SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
▨ Undisturbed
☒ Remoulded
■ Lost
◧ Cored

GEOTECHNICAL: R □ Shear (kPa) I ■ — 30, 60, 90, 120; SPT=N Value ▲; PENTEST △; RQD (%); PLASTIC LIMIT, w (%), LIQUID — 20, 40, 60, 80

DEPTH / *ELEVATION* (m)	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)	WELL DIAGRAM
	Ground surface.								
0.15	**TOPSOIL (150 mm).**								
	WEATHERED CRUST: SILTY CLAY, trace sand, brown-grey, moist, very stiff.				AS-1	☒			
					SS-1	☒	100	2 5 5 5 (10)	
					SS-2	☒	100	2 4 6 5 (10)	
					SS-3	☒	100	2 3 3 3 (6)	
					SS-4	☒	100	2 2 2 2 (4)	
3.66	**SILTY CLAY,** trace sand, grey, wet, stiff.								
					SS-5	☒	100	1 1 1 1 (2)	
					SS-6	☒	100	0 0 0 0 (0)	

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0

BOREHOLE DRILLING RECORD : **BH21-06**

Prepared by: **Mohamadou Banhoro** Date (Start): **2021-11-17**
Reviewed by: **Christine Ko** Date (End): **2021-11-17**

Project Name:	**Geotechnical Investigation**	Project Number:	**211-11099-00**
Site:	**125 Colonnade Road South**	Geographic Coordinates:	X = 445011.11 mE Y = 5021435.91 mN
Sector:	**Proposed Storage Facilities**	Surface Elevation:	Not measured
Client:	**Architecture49**	Plunge / Azimuth:	

Drilling Company:	George Downing Estate Drilling Ltd
Drilling Equipment:	CME 75
Drilling Method:	Hollow Stem Auger
Borehole Diameter:	152.4 mm
Drilling Fluid:	N/A

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
Water Level   Free Phase

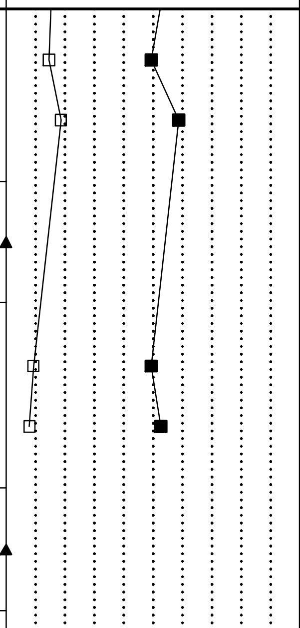
SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

DEPTH *ELEVATION* (m)	STRATIGRAPHY	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
6.5		**SILTY CLAY,** trace sand, grey, wet, stiff.							
6.86		**CLAYEY SAND,** some silt, brown-grey, wet, very loose.				SS-7		100	0 0 0 0 (0)
7.46		**SILTY CLAY,** trace sand, grey, wet, stiff.				SS-8		100	0 0 0 0 (0)
8.99		***COMMENTS:*** *- The low blow counts in the clayey sand layers are assumed to be due to hydrostatic pressure and blow back of the soils.* End of borehole at 8.99 m.							

GEOTECHNICAL
R □ Shear (kPa) I ■
30 60 90 120
SPT=N Value PENTEST
RQD (%)
PLASTIC LIMIT w (%) LIQUID
20 40 60 80



WELL
DIAGRAM

Depth scale (m): 6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10.0, 10.5, 11.0, 11.5, 12.0

BOREHOLE DRILLING RECORD : **BH21-07**

Prepared by: **Mohamadou Banhoro** Date (Start): **2021-11-18**
Reviewed by: **Christine Ko** Date (End): **2021-11-18**

Project Name:	**Geotechnical Investigation**	Project Number:	**211-11099-00**
Site:	**125 Colonnade Road South**	Geographic Coordinates:	X = 44972.88 mE Y = 5021438.64 mN
Sector:	**Proposed Storage Facilities**	Surface Elevation:	Not measured
Client:	**Architecture49**	Plunge / Azimuth:	

Drilling Company:	George Downing Estate Drilling Ltd
Drilling Equipment:	CME 75
Drilling Method:	Hollow Stem Auger
Borehole Diameter:	152.4 mm
Drilling Fluid:	N/A

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
Water Level   Free Phase

SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

DEPTH / ELEVATION (m)	DESCRIPTION	TYPE & NO.	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
	Ground surface.			
0.30	**FILL: SAND AND GRAVEL,** trace fines, brown, moist.	AS-1		
2.13	**WEATHERED CRUST: SILTY CLAY,** trace sand, brown-grey, moist, very stiff.	SS-1	100	2 2 2 2 (4)
		SS-2	100	1 2 1 2 (3)
4.42	**SILTY CLAY,** trace sand, brown-grey, moist, very stiff to stiff.	SS-3	100	1 1 1 1 (2)
		SS-4	100	1 0 1 0 (1)
5.94	**CLAYEY SAND,** some silt, brown-grey, wet, very loose.	SS-5	100	1 0 1 0 (1)

GEOTECHNICAL: R □ Shear (kPa) I ■ 30 60 90 120; SPT=N Value ▲; PENTEST △; RQD (%); PLASTIC LIMIT w (%) LIQUID 20 40 60 80

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0

Column headings: GEOLOGY / LITHOLOGY; STRATIGRAPHY; ANALYSIS: NUMBER, LABORATORY TESTING, DUPLICATE; STATE; WELL: DIAGRAM

BOREHOLE DRILLING RECORD : **BH21-07**

Prepared by: **Mohamadou Banhoro** Date (Start): **2021-11-18**
Reviewed by: **Christine Ko** Date (End): **2021-11-18**

Project Name:	**Geotechnical Investigation**	Project Number:	**211-11099-00**
Site:	**125 Colonnade Road South**	Geographic Coordinates:	X = 44972.88 mE Y = 5021438.64 mN
Sector:	**Proposed Storage Facilities**	Surface Elevation:	Not measured
Client:	**Architecture49**	Plunge / Azimuth:	

Drilling Company:	George Downing Estate Drilling Ltd
Drilling Equipment:	CME 75
Drilling Method:	Hollow Stem Auger
Borehole Diameter:	152.4 mm
Drilling Fluid:	N/A

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
Water Level Free Phase

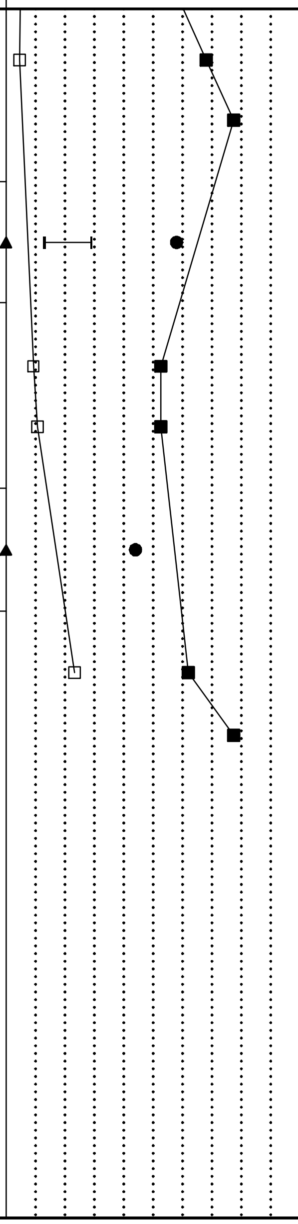
SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

DEPTH *ELEVATION* (m)	STRATIGRAPHY	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
6.0 – 9.75		**SILTY CLAY,** trace sand, grey, wet, stiff to very stiff.							
7.0						SS-6		100	0 0 0 0 (0)
8.5						SS-7		100	0 0 0 0 (0)
9.75									

GEOTECHNICAL: R □ Shear (kPa) I ■ 30 60 90 120; SPT=N Value ▲; PENTEST △; RQD (%); PLASTIC LIMIT w (%) LIQUID 20 40 60 80



COMMENTS:
- Water level was measured in open borehole at a depth of 5.9 m.
- The field vane tests between 4.7 m and 5.0 m were probably completed in clayey sand layers and are not representative of the consistency of the silty clay.
- The low blow counts in the clayey sand layers are assumed to be due to hydrostatic pressure and blow back of the soils.
End of borehole at 9.75 m.

BOREHOLE DRILLING RECORD : **BH21-08**

Prepared by: **Mohamadou Banhoro**
Reviewed by: **Christine Ko**

Date (Start): **2021-11-17**
Date (End): **2021-11-17**

Project Name:	**Geotechnical Investigation**	Project Number:	**211-11099-00**
Site:	**125 Colonnade Road South**	Geographic Coordinates:	X = 44982.44 mE Y = 5021410.42 mN
Sector:	**Proposed Storage Facilities**	Surface Elevation:	Not measured
Client:	**Architecture49**	Plunge / Azimuth:	

Drilling Company:	George Downing Estate Drilling Ltd
Drilling Equipment:	CME 75
Drilling Method:	Hollow Stem Auger
Borehole Diameter:	152.4 mm
Drilling Fluid:	N/A

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
Water Level — Free Phase

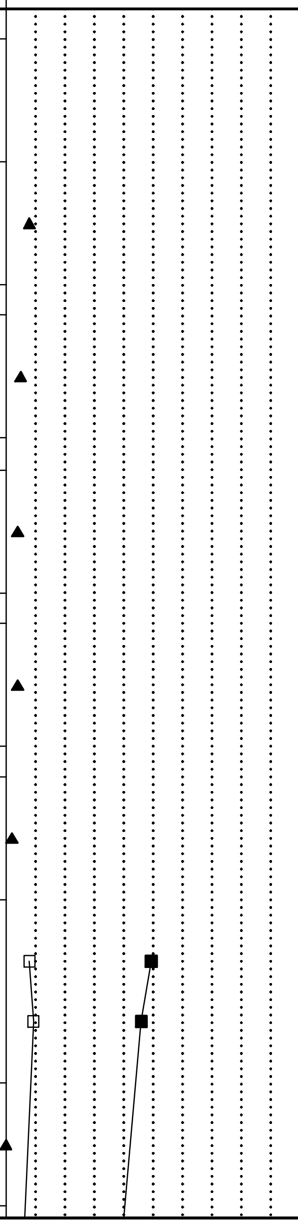
SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

DEPTH / *ELEVATION* (m)	STRATIGRAPHY	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
		Ground surface.							
0.15		**TOPSOIL (150 mm).**							
		WEATHERED CRUST: SILTY CLAY, trace sand, brown-grey, moist, very stiff to stiff.				AS-1			
						SS-1		50	3 4 4 5 (8)
						SS-2		100	2 2 3 2 (5)
						SS-3		100	2 2 2 2 (4)
						SS-4		100	2 2 2 2 (4)
3.66		**SILTY CLAY,** trace sand, brown-grey, wet, stiff.				SS-5		100	1 1 1 1 (2)
						SS-6		100	0 0 0 0 (0)

GEOTECHNICAL: R □ Shear (kPa) I ■ 30 60 90 120; SPT=N Value ▲; PENTEST △; RQD (%); PLASTIC LIMIT w (%) LIQUID 20 40 60 80



WELL: DIAGRAM

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0

BOREHOLE DRILLING RECORD : **BH21-08**

Prepared by: **Mohamadou Banhoro**
Reviewed by: **Christine Ko**
Date (Start): **2021-11-17**
Date (End): **2021-11-17**

Project Name:	**Geotechnical Investigation**	Project Number:	**211-11099-00**
Site:	**125 Colonnade Road South**	Geographic Coordinates:	X = 44982.44 mE Y = 5021410.42 mN
Sector:	**Proposed Storage Facilities**	Surface Elevation:	Not measured
Client:	**Architecture49**	Plunge / Azimuth:	

Drilling Company:	George Downing Estate Drilling Ltd
Drilling Equipment:	CME 75
Drilling Method:	Hollow Stem Auger
Borehole Diameter:	152.4 mm
Drilling Fluid:	N/A

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
▽ Water Level ▼ Free Phase

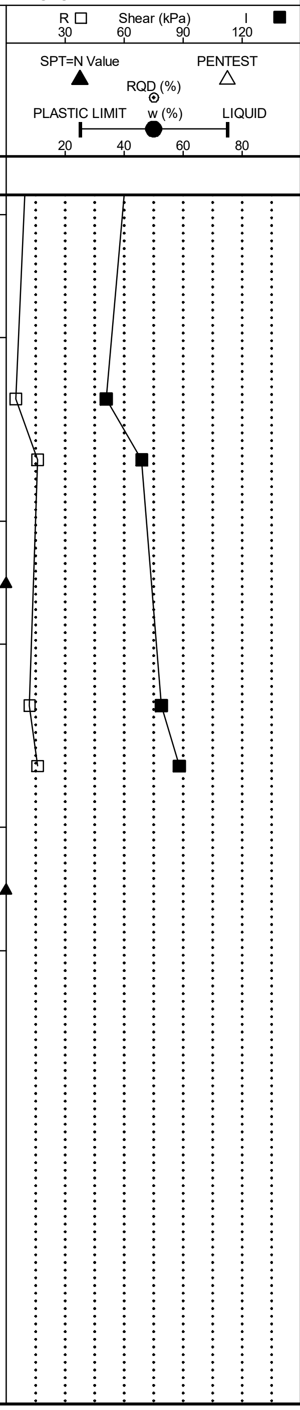
SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

DEPTH / *ELEVATION* (m)	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
6.10	**CLAYEY SAND,** some silt, brown-grey, wet.				ST-1		0	
6.71	**SILTY CLAY,** trace sand, grey, wet, stiff.				SS-7		100	0 0 0 0 (0)
					SS-8		100	0 0 0 0 (0)
9.75	**COMMENTS:** - Water level was measured in open borehole at a depth of 6.1 m. - The Shelby Tube was empty after sampling; inferred clayey sand layer between depths of 6.1 m and 6.7 m. End of borehole at 9.75 m.							

GEOTECHNICAL



WELL
DIAGRAM

Depth scale (m): 6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10.0, 10.5, 11.0, 11.5, 12.0

Explanation of Terms Used in the Record of Boreholes

Sample Type

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Dimension type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Spoon sample
SH	Shelby tube Sample
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

Penetration Resistance

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) required to drive a 50 mm (2 in) drive open sampler for a distance of 300 mm (12 in).

WH – Samples sinks under "weight of hammer"

Dynamic Cone Penetration Resistance, N_d:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) to drive uncased a 50 mm (2 in) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in).

Textural Classification of Soils

Classification	Particle Size
Boulders	> 200 mm
Cobbles	75 mm - 200 mm
Gravel	4.75 mm - 75 mm
Sand	0.075 mm – 4.75 mm
Silt	0.002 mm-0.075 mm
Clay	<0.002 mm

Coarse Grain Soil Description (50% greater than 0.075 mm)

Terminology	Proportion
Trace	0-10%
Some	10-20%
Adjective (e.g. silty or sandy)	20-35%
And (e.g. sand and gravel)	> 35%

Soil Description

a) Cohesive Soils(*)

Consistency	Undrained Shear Strength (kPa)	SPT "N" Value
Very soft	<12	0-2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very stiff	100-200	15-30
Hard	>200	>30

(*) Hierarchy of Shear Strength prediction

1. Lab triaxial test
2. Field vane shear test
3. Lab. vane shear test
4. SPT "N" value
5. Pocket penetrometer

b) Cohesionless Soils

Density Index (Relative Density)	SPT "N" Value
Very loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Soil Tests

w	Water content
w_p	Plastic limit
w_l	Liquid limit
C	Consolidation (oedometer) test
CID	Consolidated isotropically drained triaxial test
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement
D_R	Relative density (specific gravity, Gs)
DS	Direct shear test
ENV	Environmental/ chemical analysis
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified proctor compaction test
SPC	Standard proctor compaction test
OC	Organic content test
U	Unconsolidated Undrained Triaxial Test
V	Field vane (LV-laboratory vane test)
γ	Unit weight

APPENDIX B

GEOPHYSICAL SURVEY RESULTS

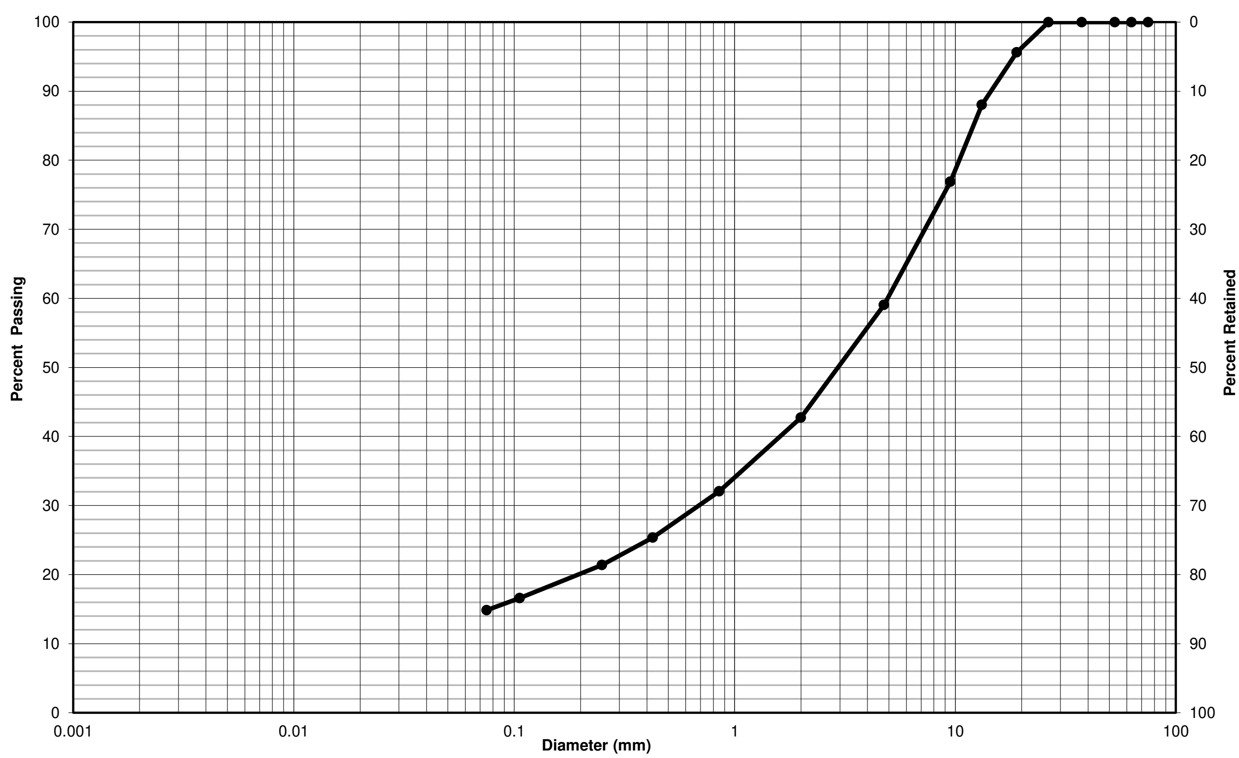
APPENDIX

C

LABORATORY TESTING RESULTS

Particle-Size Analysis of Soils
LS-702

Client:	A49	**Lab no.:**	OL1001-4
Project/Site:	125 Colonnade Rd	**Project no.:**	211-11099-00
Borehole no.:	BH21-4	Sample no.:	AS1
Depth:	0.1-0.3 m		



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent %	Gravel	Sand	Clay & Silt	Silt	Clay
	40.9	44.2	14.8	-	-

Remarks: sample contains trace asphalt particles

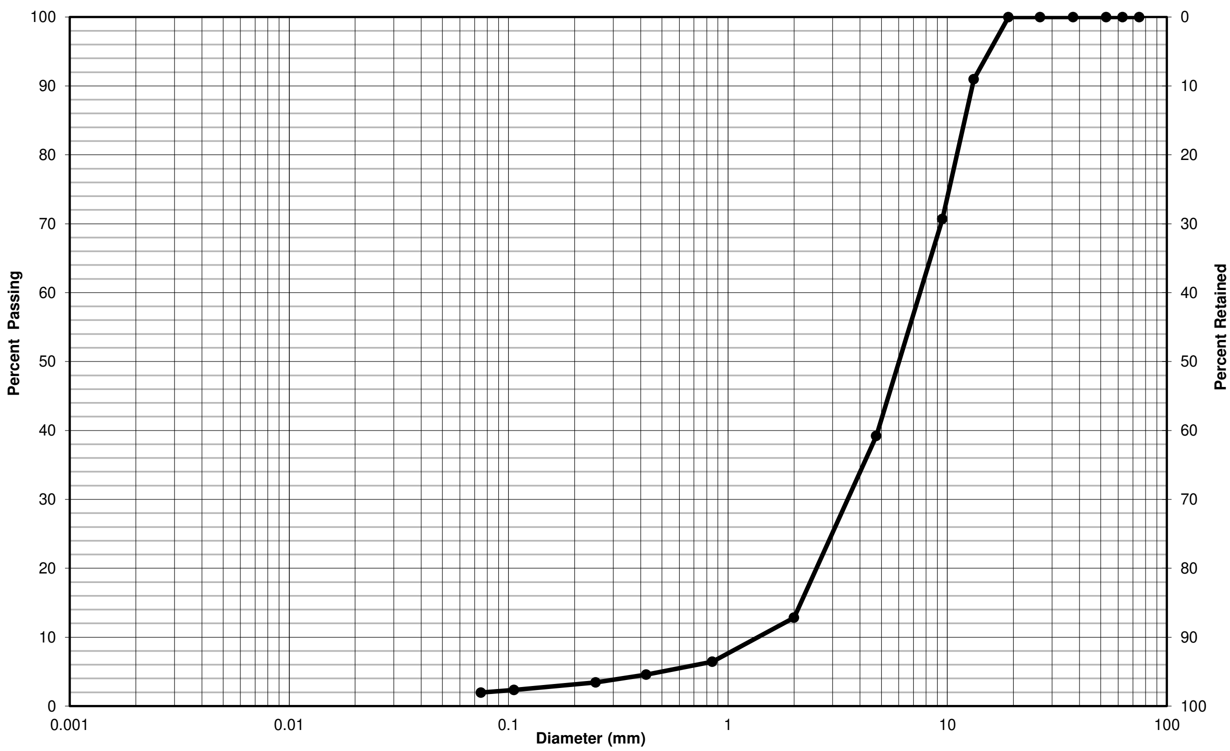
**More information available upon request*

Performed by:	M.Tippett	**Date:**	December 22, 2021
Verified by:	N.Krebs	**Date:**	December 24, 2021

Particle-Size Analysis of Soils
LS-702

Client:	A49	**Lab no.:**	OL 1001-1
Project/Site:	125 Colonnade Rd	**Project no.:**	211-11099-00

Borehole no.:	BH21-7	Sample no.:	GS1
Depth:	0.1-0.3 m		



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

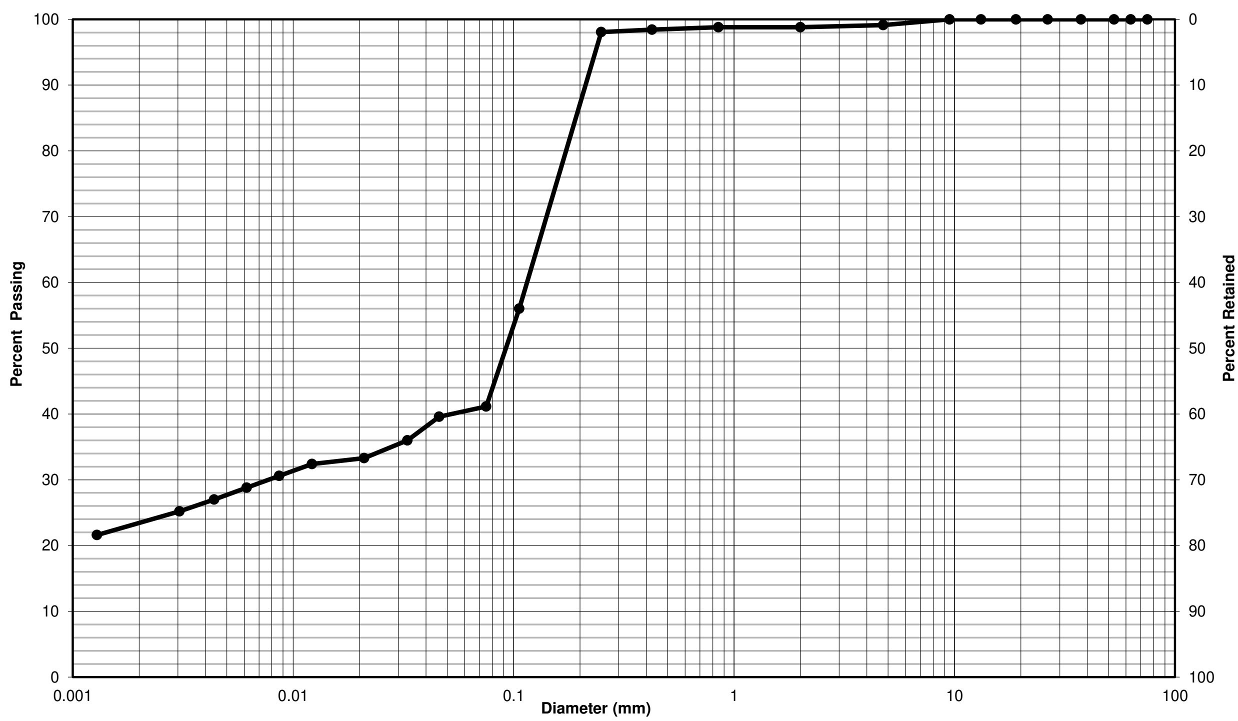
Percent %	Gravel	Sand	Clay & Silt	Silt	Clay
	60.8	37.3	2.0	-	-

Remarks:

Performed by:	M.Tippett	**Date:**	December 22, 2021
Verified by:	N.Krebs	**Date:**	December 24, 2021

Particle-Size Analysis of Soils
LS-702

Client:	A49	**Lab no.:**	OL1001-6
Project/Site:	125 Colonnade Rd	**Project no.:**	211-11099-00
Borehole no.:	BH21-4	Sample no.:	SS6
Depth:	5.3-5.9m		



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

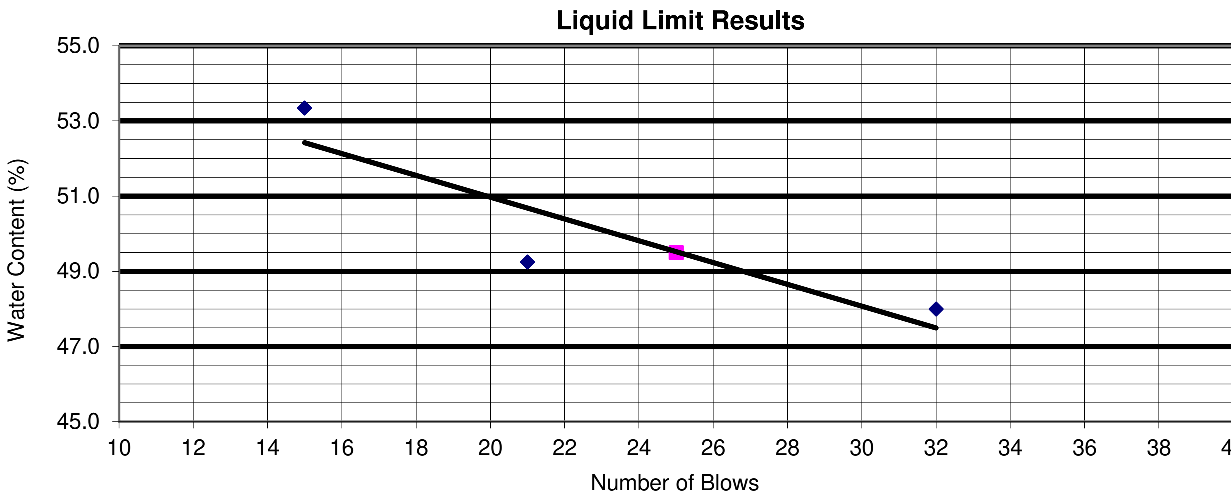
Percent %	Gravel	Sand	Clay & Silt	Silt	Clay
	0.9	58.0	41.1	18.1	23.0

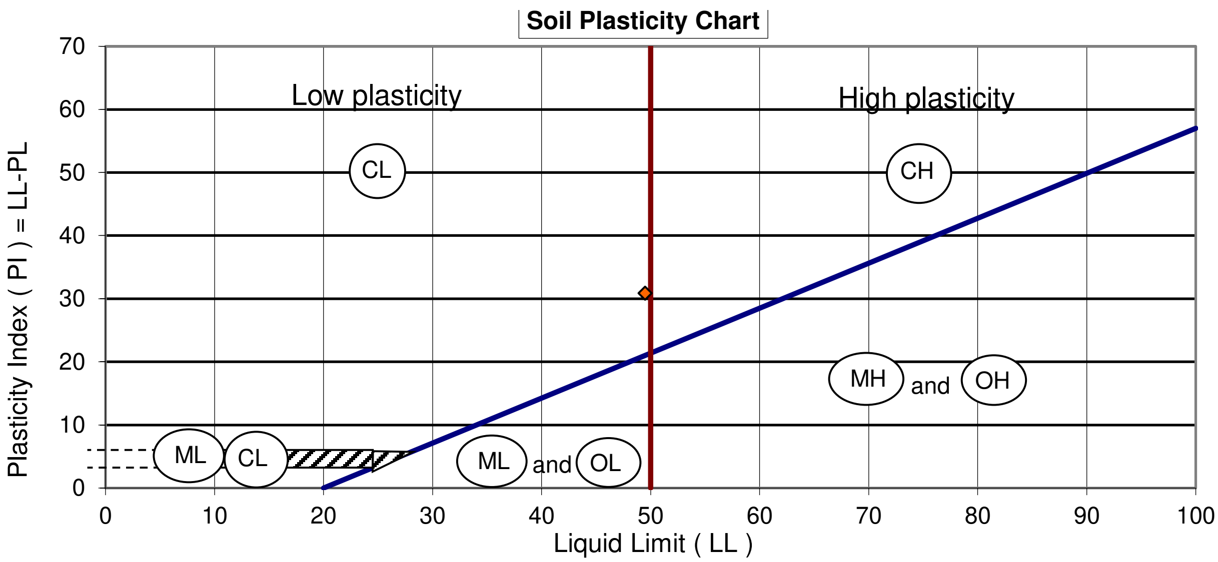
Remarks:

Performed by:	M.Tippett	**Date:**	January 5, 2022
Verified by:	N.Krebs	**Date:**	January 11, 2022

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

Client:	A49	**Lab No.:**	OL1001-5
Project/Site:	125 Colonnade Rd.	**Project No.:**	211-11099-00
Borehole No.:	BH21-4	Sample No.:	SS4
Sample Depth:	3.1-3.7m		





Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content %
50	19	31	62.4

Sample Description: **CL - Low plasticity, inorganic clay**

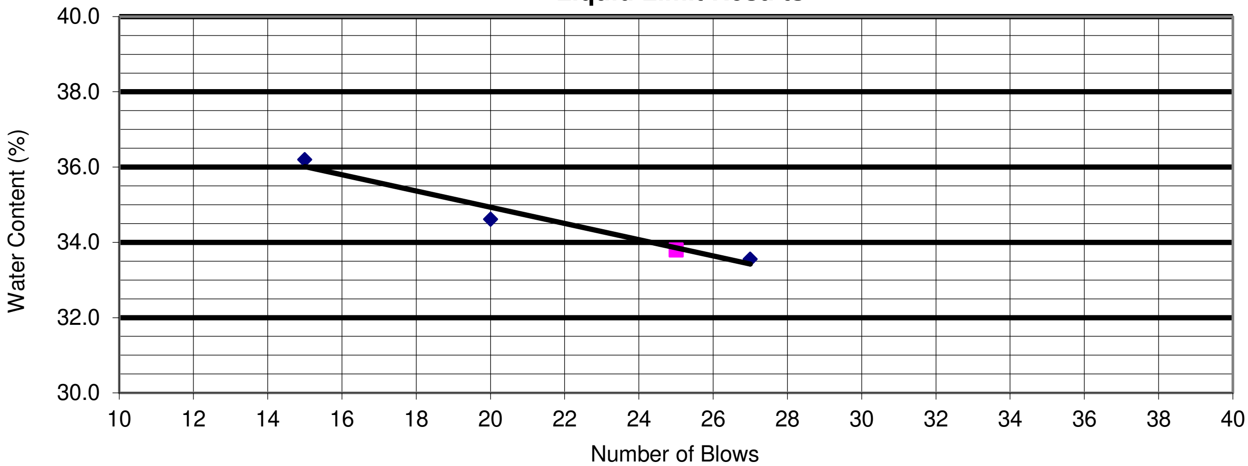
Performed By:	M.Tippett	**Date:**	January 5, 2022
Verified By:	N.Krebs	**Date:**	January 11, 2022

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

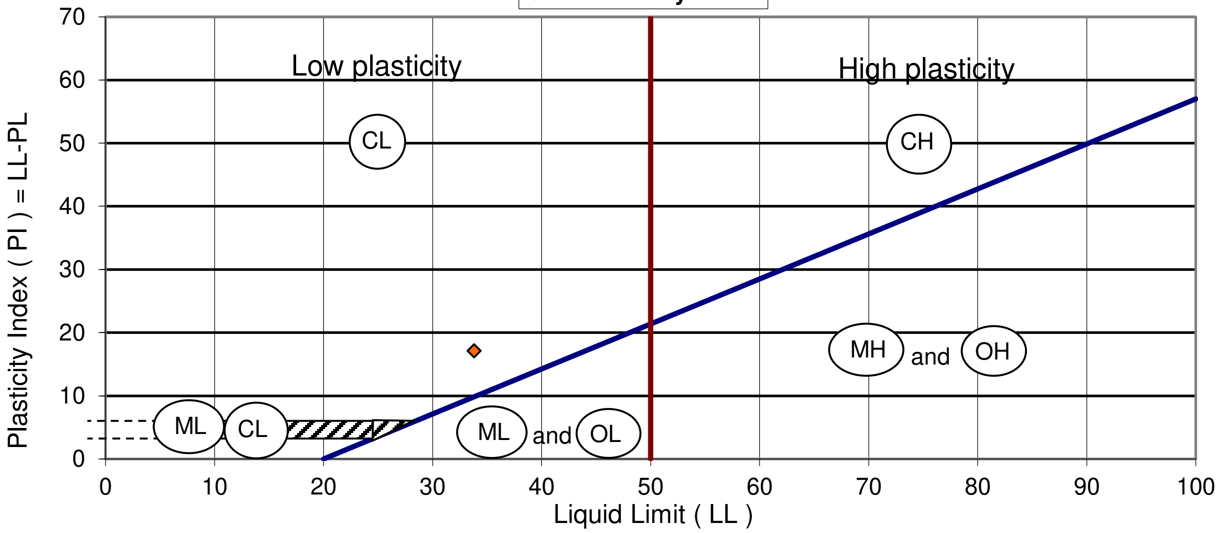
Client:	A49	**Lab No.:**	OL1001-7
Project/Site:	125 Colonnade Rd.	**Project No.:**	211-11099-00

Borehole No.:	BH21-4	Sample No.:	SS7B
Sample Depth:	6.4-6.7m		

Liquid Limit Results



Soil Plasticity Chart



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content %
34	17	17	61.2

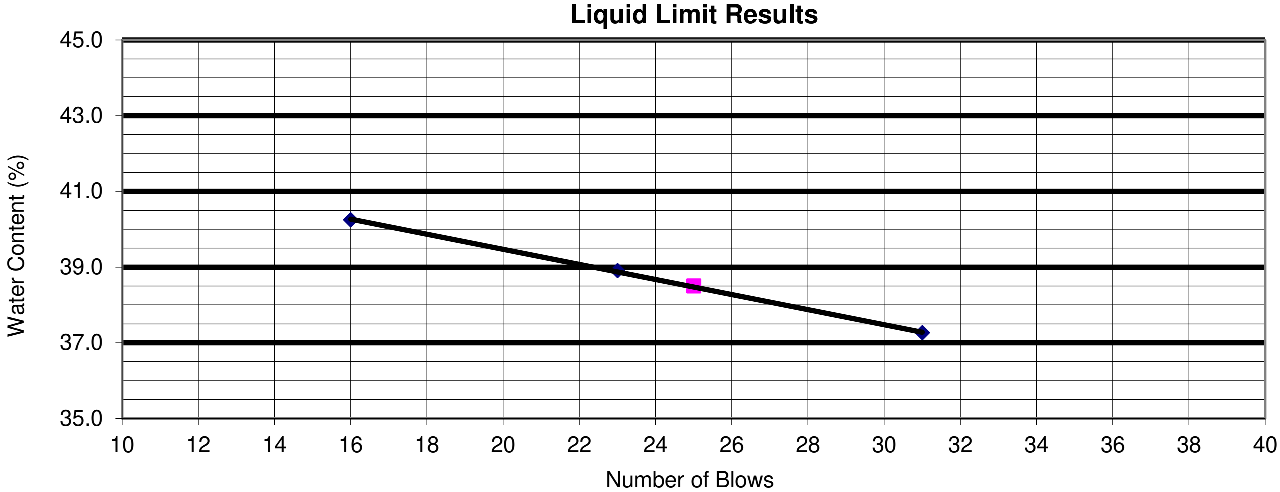
Sample Description: **CL - Low plasticity, inorganic clay**

Performed By:	M.Tippett	**Date:**	January 6, 2022
Verified By:	N.Krebs	**Date:**	January 11, 2022

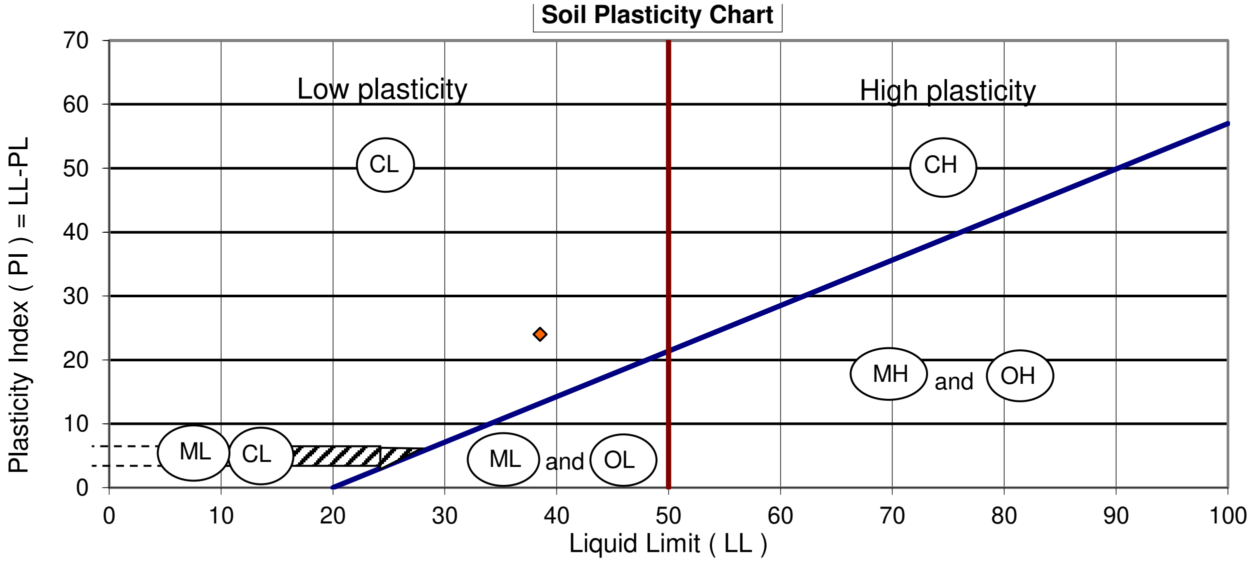
Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client:	A49	**Lab No.:**	OL1001-2
Project/Site:	125 Colonnade Rd.	**Project No.:**	211-11099-00
Borehole No.:	BH21-7	Sample No.:	SS2
Sample Depth:	1.5-2.1m		

Liquid Limit Results



Soil Plasticity Chart



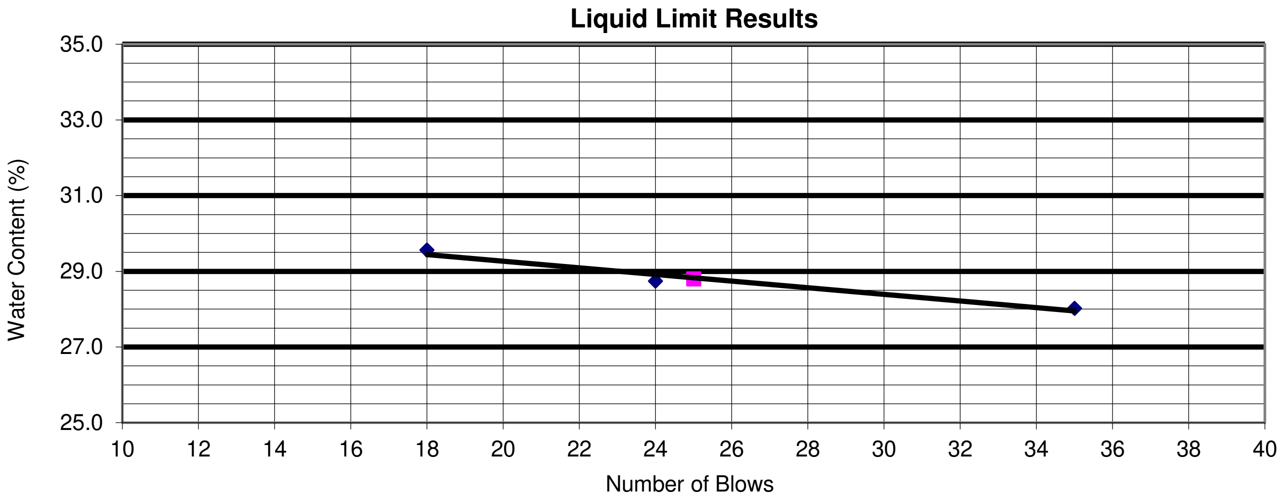
Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content %
39	15	24	45.5

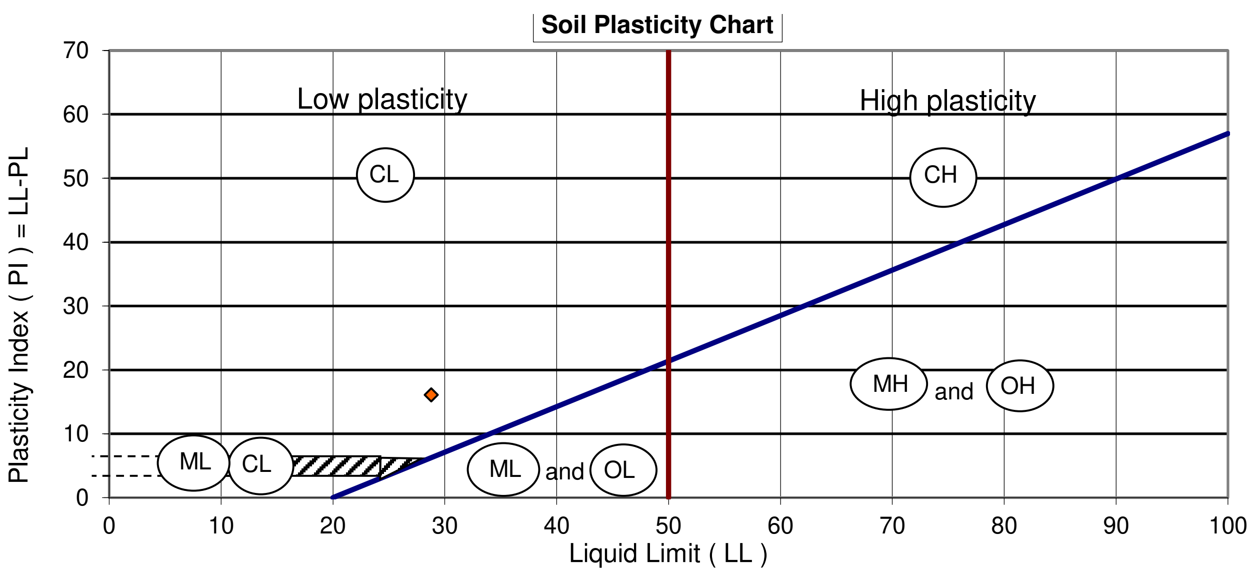
Sample Description: **CL - Low plasticity, inorganic clay**

Performed By:	M.Tippett	**Date:**	December 22, 2021
Verified By:	N.Krebs	**Date:**	December 24, 2021

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

Client:	A49	**Lab No.:**	OL1001-3
Project/Site:	125 Colonnade Rd.	**Project No.:**	211-11099-00
Borehole No.:	BH21-7	Sample No.:	SS6
Sample Depth:	6.9-7.5m		





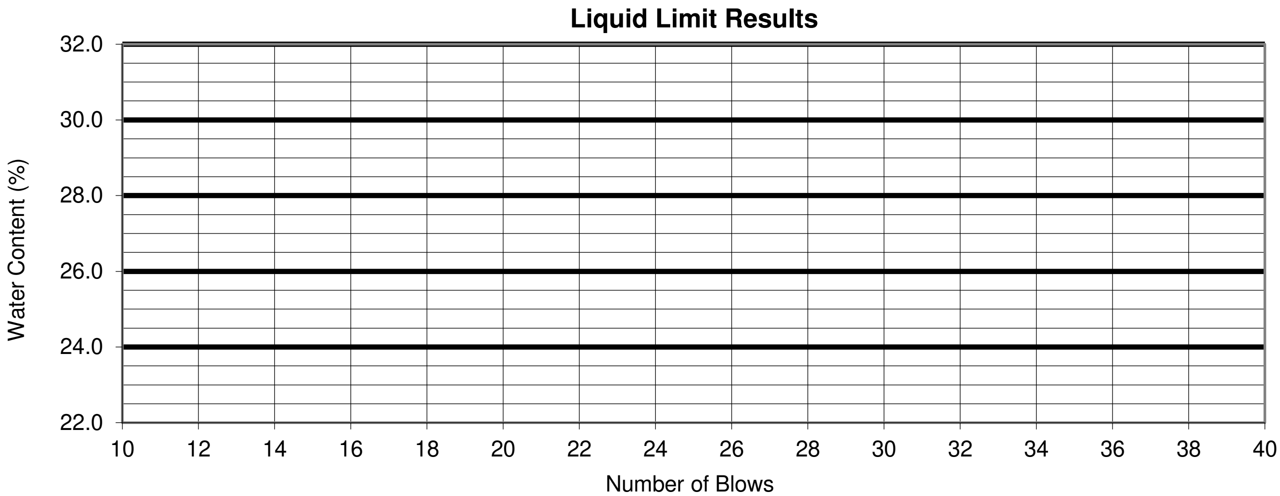
Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content %
29	13	16	58.2

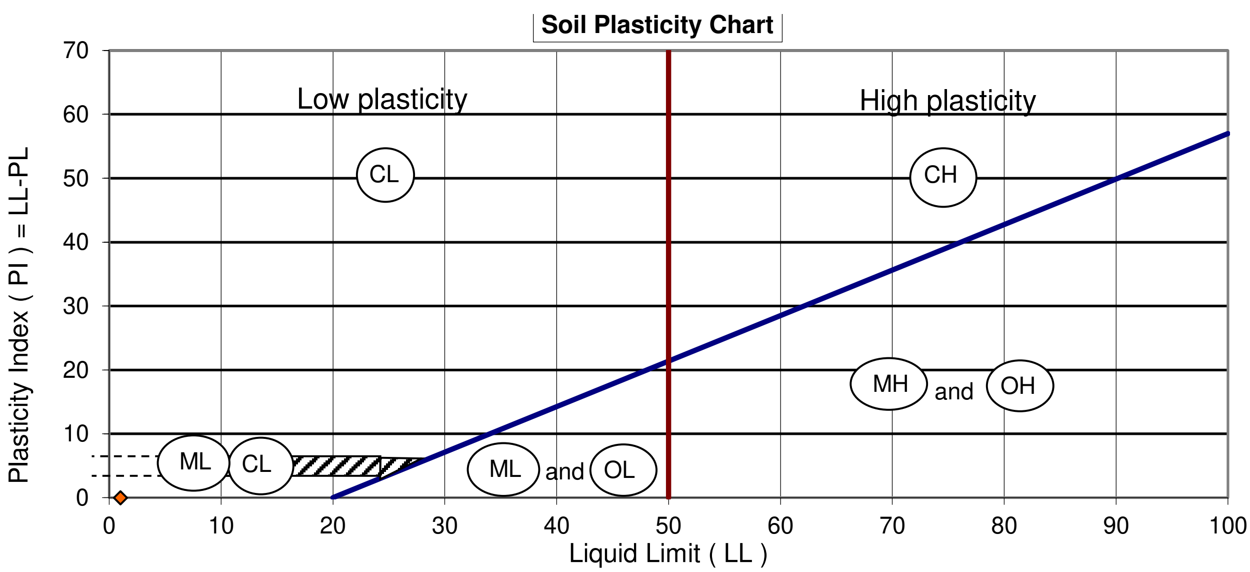
Sample Description: **CL - Low plasticity, inorganic clay**

Performed By:	M.Tippett	**Date:**	December 22, 2021
Verified By:	N.Krebs	**Date:**	December 24, 2021

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

Client:	A49	**Lab No.:**	OL 0019
Project/Site:	125 Colonnade Rd	**Project No.:**	211-11099-01
Borehole No.:	BH7	Sample No.:	SS5
Sample Depth:	5.3-5.9 m		





Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content %
-	-	-	-

Sample Description: **Non-Plastic**

Performed By:	M.Tippett	**Date:**	April 12, 2022
Verified By:	N.Krebs	**Date:**	April 12, 2022

Certificate of Analysis

Client:	WSP (Ottawa) 2611Queensview Dr. Ottawa, ON K2B 8K1	Report Number:	1967946
		Date Submitted:	2021-12-02
		Date Reported:	2021-12-09
		Project:	211-11099-00
		COC #:	215773
Attention:	Nathan Christie		
PO#:	211-11099-0		
Invoice to:	WSP Canada Inc.		

Dear Nathan Christie:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

APPROVAL: ______________________________

Sarah Horner, Inorganics Technician

All analysis is completed at Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) unless otherwise indicated.

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Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by the Ontario Ministry of Agriculture, Food, and Rural Affairs for specific tests in agricultural soils.

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required. Unless otherwise stated, measurement uncertainty is not taken into account when determining guideline or regulatory exceedances.

Certificate of Analysis

eurofins | Environment Testing

Client:	WSP (Ottawa) 2611Queensview Dr. Ottawa, ON K2B 8K1	Report Number:	1967946
Attention:	Nathan Christie	Date Submitted:	2021-12-02
PO#:	211-11099-0	Date Reported:	2021-12-09
Invoice to:	WSP Canada Inc.	Project:	211-11099-00
		COC #:	215773

Group	Analyte	MRL	Units	Guideline	1599462	1599463
				Lab I.D.	1599462	1599463
				Sample Matrix	Soil	Soil
				Sample Type		
				Sampling Date	2021-11-17	2021-11-30
				Sample I.D.	BH21-08 SS2	BH21-02 SS2
Anions	Cl	0.002	%		0.022	0.036
	SO4	0.01	%		<0.01	0.05
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.58	1.00
	pH	2.00			7.91	7.50
	Resistivity	1	ohm-cm		1720	1000

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Certificate of Analysis

eurofins | Environment Testing

Client: WSP (Ottawa)
2611Queensview Dr.
Ottawa, ON
K2B 8K1
Attention: Nathan Christie
PO#: 211-11099-0
Invoice to: WSP Canada Inc.

Report Number: 1967946
Date Submitted: 2021-12-02
Date Reported: 2021-12-09
Project: 211-11099-00
COC #: 215773

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 413570 **Analysis/Extraction Date** 2021-12-07 **Analyst** MW **Method** AG SOIL			
SO4	<0.01 %	110	70-130
Run No 413676 **Analysis/Extraction Date** 2021-12-08 **Analyst** MW **Method** C CSA A23.2-4B			
Chloride	<0.002 %		90-110
Run No 413808 **Analysis/Extraction Date** 2021-12-09 **Analyst** MW **Method** Ag Soil			
pH	7.64	100	90-110
Run No 413813 **Analysis/Extraction Date** 2021-12-09 **Analyst** MW **Method** Resistivity - soil			
Resistivity			
Run No 413815 **Analysis/Extraction Date** 2021-12-09 **Analyst** MW **Method** Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	98	90-110

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

APPENDIX

D

LIMITATIONS OF THIS REPORT

LIMITATIONS OF REPORT

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The report is intended to be used in its entirety. No excerpts may be taken to be representative of the findings in the assessment.

The conclusions presented in this report are based on work performed by trained, professional and technical staff, in accordance with their reasonable interpretation of current and accepted engineering and scientific practices at the time the work was performed.

The content and opinions contained in the present report are based on the observations and/or information available to WSP at the time of preparation, using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by WSP and other engineering/scientific practitioners working under similar conditions, and subject to the same time, financial and physical constraints applicable to this project.

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Design recommendations given in this report are applicable only to the project and areas as described in the text and then only if constructed in accordance with the details stated in this report. The comments made in this report on potential construction issues and possible methods are intended only for the guidance of the designer. The number of testing and/or sampling locations may not be sufficient to determine all the factors that may affect construction methods and costs. We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.

Overall conditions can only be extrapolated to an undefined limited area around these testing and sampling locations. The conditions that WSP interprets to exist between testing and sampling points may differ from those that actually exist. The accuracy of any extrapolation and interpretation beyond the sampling locations will depend on natural conditions, the history of Site development and changes through construction and other activities. In addition, analysis has been carried out for the identified chemical and physical parameters only, and it should not be inferred that other chemical species or physical conditions are not present. WSP cannot warrant against undiscovered environmental liabilities or adverse impacts off-Site.

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