REPORT

Geotechnical Investigation

2700 Swansea Crescent
Ottawa, ON K1G 6R8

Submitted to:

IST Properties Inc
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Ottawa, ON
K1G 6R8

Submitted by:

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Distribution List

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was retained by IST properties Inc. to carry out a geotechnical investigation to support the proposed design of the Phase 1 (south side) expansion of the building located at 2700 Swansea Crescent in Ottawa, Ontario. The approximate location of the site is shown on the Key Map inset on the attached Site Plan (Figure 1). The investigation and reporting were carried out in general accordance with the scope of work provided in our proposal dated February 15, 2022 as well as subsequent correspondence.

The purpose of this investigation was to assess the general subsurface and groundwater conditions within the study area by means of two boreholes and associated laboratory testing. Based on an interpretation of the factual information obtained during the current investigation, a general description of the soil and groundwater conditions is presented. These interpreted subsurface conditions and available project details were used to prepare engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

The reader is referred to the 'Important Information and Limitations of This Report' which follows the text but forms an integral part of this document.

2.0 PROJECT DESCRIPTION

Plans are being prepared for the Phase 1 expansion of an existing two-storey light manufacturing building located at 2700 Swansea Crescent in Ottawa, Ontario (see Figure 1 – Site Plan).

The site is bordered to the north and east by commercial buildings, and south and west by Swansea Crescent Road. There is a two-storey light manufacturing building located within the northern portion of the site. As a part of Phase I building expansion, a structure will be constructed above the parking lot as an addition to the south side of second storey of the existing building. The phase I expansion will have an approximately 1,420 sq. m footprint.

In addition to the building expansion, it is understood that an approximately 2 m thick layer of clean gravel will be placed in the area, replacing the in-situ native soil beneath the pavement structure for the purpose of mass thermal storage. The proposed area of the thermal storage includes the hard landscaping areas extending to the east and west sides of the parking area, including the parking area under the proposed building expansion. Based on the information provided to Golder, geothermal ground loops will also be placed within the thermal storage layer at depths of 1.8 m (6') and 2.7 m (9') below the final grade.

Based on the results of previous investigations carried out near the site, as well as a review of the available published geological mapping, the subsurface conditions at the site are indicated to consist of a layer of surficial fill overlying a deposit of silty clay to clay which is underlain by glacial till above bedrock. The bedrock at this site is indicated to be about 10 m below the ground surface and consists of interbedded shale and limestone of the Carlsbad formation.

3.0 PROCEDURE

The fieldwork for this geotechnical investigation was carried out on July 5, 2022. A total of two boreholes (numbered 22-01 and 22-01A) were advanced at the approximate locations shown on the Site Plan.

The boreholes were advanced using a truck-mounted drill rig supplied and operated by CCC Group, Ottawa, Ontario. Borehole 22-01 was advanced to 6.9 m (elevation of 75.8 m) below the existing ground surface. Standard Penetration Tests (SPTs) were carried out in the boreholes within the overburden at regular intervals of depth where possible. Samples of the soils encountered were recovered using 35 mm inside diameter split-spoon sampling equipment in general accordance with ASTM D1586-18. In-situ vane testing was carried out, where possible, in the silty clay to clay deposit to measure the undrained shear strength of this soil. Borehole 22-01A was augered to 1.9 m below ground surface without recovering the soil samples and then in-situ vane testing was carried out in the inferred silty clay to clay deposit to 4.0 m below the ground surface.

One monitoring well was sealed into borehole 22-01 to allow for measurement of the stabilised groundwater levels. The groundwater level measurement in this well was carried out by Golder personnel on August 12, 2022.

The fieldwork was supervised by personnel from our engineering staff who located the boreholes, directed the drilling and in-situ testing operations, logged the borehole logs and samples, and took custody of the soil samples retrieved. On completion of the drilling operations, the soil samples were transported to our laboratory for further examination and for laboratory testing, which included determination of natural water content, grain size distribution and Atterberg limits on selected soil samples. One sample of soil was submitted to Eurofins Environment Testing for basic chemical analyses related to potential sulphate attack on buried concrete elements and potential corrosion of buried ferrous elements.

The borehole locations were selected in consultation with IST properties Inc., marked in the field, and subsequently surveyed by Golder Associates personnel.

4.0 SUBSURFACE CONDITIONS

4.1 General

Information on the subsurface conditions is provided as follows:

- The Record of borehole logs are provided in Appendix A.
- Laboratory test results are provided in Appendix B.
- Results of the basic chemical analyses are provided in Appendix C.
- Golder Associate Technical Memorandum No. 07-1121-0135 is provided in Appendix D

The Record of Borehole sheets describe the subsurface conditions at the particular borehole locations only. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling in some cases, observations of drilling progress as well as results of SPT tests and, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface soil, and groundwater conditions will vary between and beyond the borehole locations.

4.2 Overview of Subsurface Conditions

In general, the subsurface stratigraphy at the borehole locations consists of pavement structure, underlain by deposits of silty clay to clay, underlain by glacial till and weathered bedrock. The following sections present a detailed overview of the subsurface conditions encountered in the boreholes advanced during the current investigation.

4.2.1 Pavement Structure

The pavement structure at the location of borehole 22-01 consists of approximately 80 mm layer of asphaltic concrete overlying 530 mm of granular base/subbase.

4.2.2 Silty Clay to Clay

At the location of borehole 22-01, the pavement structure is underlain by a silty clay to clay deposit with trace to some amounts of gravel. The upper portion of the silty clay to clay is weathered to a grey-brown crust. At this location, the weathered silty clay to clay crust extends to a depth of approximately 2.1m (elevation of 80.6 m) below the existing ground surface.

Standard Penetration test ‘N’ values within the weathered crust yielded ‘N’ values of 4 and 5 blows per 0.3 m of penetration. The results of in-situ vane testing carried out within the upper portion of inferred weathered silty clay to clay in borehole 22-01A gave undrained shear strength of more than 96 kPa, indicating a very stiff consistency.

The measured natural water content of two samples of the weathered silty clay to clay was 51 and 58%. The result of Atterberg limit testing carried out on a single sample from weathered silty clay to clay deposits gave a plasticity index value of 56 and liquid limit value of 79, indicating a high plasticity soil. The results of the Atterberg limit testing are provided on Figure B-1 in Appendix B.

Beneath the weathered zone, the clay is grey in colour. The unweathered grey silty clay to clay extends to a depth of 3.7 m (elevation of 79.0 m) below the existing ground surface.

The results of in-situ vane testing carried out within the unweathered portion of silty clay to clay in borehole 22-01 and inferred silty clay to clay in borehole 22-01A gave undrained shear strengths in the range of 67 to 81 kPa, indicating a stiff consistency.

The measured natural water content of a single sample of the unweathered silty clay to clay was 48%. The results of Atterberg limit testing carried out on a single sample from the unweathered silty clay gave a plasticity index value of 46 and a liquid limit value of 67, indicating a high plasticity soil. The results of the Atterberg limit testing are provided on Figure B-2 in Appendix B.

4.2.3 Glacial Till

A deposit of glacial till exists beneath the grey silty clay to clay at the location of borehole 22-01.

The glacial till typically consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt to silty sand. The glacial till extends to depth of 5.2 m (elevation of 77.5 m) beneath the existing ground surface.

The SPT “N” values within the glacial till layer ranged from 8 to 21 blows per 0.3 m of penetration, indicating a loose to compact state of packing.

The measured natural water content of two samples of glacial till were 8 and 9%. The results of grain size distribution testing carried out on two samples of the glacial till are provided on Figure B-3 in Appendix B. The result of Atterberg limit testing carried out on fine particles of two samples from glacial deposits gave plasticity index values of 5, and liquid limit values of 17 and 19, indicating low plasticity fines. The results of the Atterberg limit testing are provided on Figure B-4 in Appendix B.

4.2.4 Highly Weathered Bedrock and Refusal

Highly weathered bedrock was encountered below the glacial till at the location of borehole 22-01. The bedrock was encountered at a depth of 5.2 m (77.5 m elevation) and penetrated to a depth of 1.7 m by augering. Sampler refusal was encountered at a depth of about 6.9 m (elevation of 75.8 m) below the existing ground surface.

4.3 Groundwater

The groundwater level in the monitoring well installed at this site was measured on August 12, 2022. During that time, the water level was observed at a depth of 6.4 m below existing ground surface. The measured water levels are summarized as follows:

Monitoring Well Number	Geologic Unit screened	Groundwater Level Depth (m)	Ground Surface Elevation (m)	Groundwater Level Elevation (m)
22-01	Glacial Till & Weathered Bedrock	6.4	82.7	76.3

It should be noted that groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring and fall.

4.4 Corrosion Testing

One sample of soil from borehole 22-01 was submitted to Eurofins Environment Testing for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements. The result of this testing is provided in Appendix C and is summarized below:

Borehole Number	Sample Number	Depth Interval (m)	Chlorides (%)	Sulphates (%)	pH	Resistivity (Ohm-cm)
22-01	3	1.5 - 2.1	0.043	0.05	7.88	1920

5.0 DESIGN AND CONSTRUCTION CONSIDERATIONS

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the proposed building expansion based on our interpretation of the borehole information and project requirements.

The information in this portion of the report is provided for planning and design purposes for the guidance of the design engineers and architects. Where comments are made on construction, they are provided only to highlight aspects of construction which could affect the design of the project. 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, costs, sequencing and the like.

Pursuant to the following recommendations, the subsurface conditions encountered during the investigations indicate that there are no concerns from a geotechnical standpoint for the site development that cannot be managed using routine and accepted design and construction approaches for commercial development.

The reader is referred to the "Important Information and Limitations of This Report" which follows the text of this report but forms an integral part of this document.

5.2 Excavation and Groundwater Control

Based on the information provided to Golder, it is anticipated that excavations will be require for the foundations, the thermal storage pad and possibly for site services. It is assumed that the foundations will extend to typical frost depth and excavations for the thermal storage pad will be to approximately 3 m depth. At these depths the excavations will be in the silty clay, or possibly into the upper portions of the till in some locations. Deeper excavations (if required) for site services may also be in glacial till or the underlying bedrock.

Measurements taken during the current investigation suggest that the groundwater level is generally at about 6.4 m below the existing ground surface, and within the upper portion of the bedrock (but likely below the depth of excavation for foundations, the granular pad and typical site services).

No unusual problems are anticipated in excavating the overburden using conventional hydraulic excavating equipment, recognizing that cobbles and boulders could be present in the glacial till.

In accordance with the Occupational Health and Safety Act (OHSA) of Ontario, the soils that will be encountered within the excavations would be generally classified as Type 3 soils. The side slopes in the overburden above the water table, which is the case for this site, could be sloped no steeper than 1 horizontal to 1 vertical. Boulders larger than 0.3 m in diameter should be removed from the excavation side slopes for worker safety.

Where site conditions (such as presence of soft or weak soils, proximity of existing structure and utilities, or space restriction) do not allow for the above noted side slopes, then suitable safety and support measures must be undertaken according to the requirement of OSHA. These measures include installation of a suitable shoring system to create and maintain positive supports to the side walls of the excavation. Guidelines on excavation shoring are provided in section 6.8.3 and design parameters are provided in section 6.6.1.

The groundwater levels at the site were measured to be below the anticipated general excavation depth. However, some groundwater infiltration into the excavation (such as perched water) should still be expected. Also, there may be instances where significant volumes of precipitation, surface runoff, and/or groundwater collects in an open excavation must be pumped out. Water in the open excavations should feasibly be handled by pumping from sumps within the excavations. Assuming the excavations are predominantly in silty clay, and are above the groundwater level, a PTTW is not expected to be required.

The silty clay and glacial till will be easily disturbed during construction. Any disturbed soil will need to be removed prior to placing the geotextile and 2 m thick gravel layer. This gravel layer should be placed immediately following inspection and approval of the subgrade. The period of time between exposure of the subgrade and covering with the gravel layer should be limited to as brief as possible and, in the interim, construction traffic on the subgrade should be minimized. In addition to this, this thermal gravel layer should be wrapped (top, bottom and sides) with a non-woven geotextile material to limit the migration of fine particles from pavement structure into voids of clean gravel.

5.3 Foundations

Shallow foundations may be used to support the new addition. It is assumed that the foundation to support the proposed building expansion will be on footings up to 3 m in width, placed at a depth of around 1.8 m below existing ground surface. Based on our understanding of the current design intent, the footings will be resting on the lower portions of the layer of gravel placed for mass thermal storage, underlain by grey silty clay to clay.

Pad footings up to 3 m in width, may be assumed to have a bearing resistance of 150 kPa at serviceability limit states (SLS) and a factored bearing resistance at ultimate limit state (ULS) of 200 kPa. It would be possible to accommodate larger foundations, but the bearing resistance values would need to be re-assessed.

The post-construction total and differential settlements of footings supported on soil and sized using the above maximum allowable bearing pressures would be expected to be less than 25 and 10 mm, respectively, assuming the foundations were properly constructed.

5.4 Foundation Wall Backfill

The soils at this site are frost susceptible and should not be used as backfill directly against foundation elements. To avoid problems with frost adhesion and heaving, foundation elements should be backfilled with non-frost susceptible sand and gravel conforming to the requirements for OPSS Granular B Type I or, alternatively a bond break such as the Platon system sheeting could be placed against the foundation walls. The 2 m thick gravel layer for mass thermal storage will fulfill this function.

5.5 Site Class

The seismic design provisions of the 2012 Ontario Building Code (OBC) depend, in part, on the shear wave velocity of the upper 30 m of soil and/or rock below founding level. The OBC also permits the Site Class to be specified based solely on the stratigraphy and in situ testing data (i.e., shear strengths and standard penetration test results), rather than from direct measurements of the shear wave velocity.

The results of the shear wave velocity assessment are provided in Golder Associates previous memo titled "Seismic Study – Site Classification, MASW Data Processing and Results, 2700 Swansea Crescent, Ottawa, Ontario". This memo is provided in Appendix D.

The results of the testing indicate an average shear wave velocity for the upper 30 m of soil and bedrock of 470 m per second which, according to the 2012 Ontario Building Code site classification for seismic site response, classifies this site as Site Class C.

5.6 Frost Protection

The soils at this site are frost susceptible. All 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 based on Ontario Provincial Standard Drawing (OPSD) 3090.101.

Consideration could be given to insulating the bearing surface with high density insulation as an alternative to earth cover. Further geotechnical input can be provided in this regard, if required.

5.7 Trees

Silty clay to clay soils in some areas in Ottawa are highly sensitive to water depletion by trees of high water demand during periods of dry weather. When trees draw water from the silty clay or clay, the soil undergoes a significant amount of volume change (i.e., shrinkage) which can result in settlement of adjacent structures.

Based on the results of Atterberg limit testing, silty clay to clay layers has high plasticity. Therefore, these materials are likely to undergo significant volume changes as a result of variation in water content.

Tree planting setback restrictions are required at this site as per City guideline, *"Tree Planting in Sensitive Marine Clay Soils – 2017 Guidelines" draft version 2.0 (dated January 7, 2019).*

5.8 Site Servicing

At least 150 mm of OPSS Granular A should be used as pipe bedding for sewer and water pipes. Where unavoidable disturbance to the subgrade surface occurs below the invert of the pipe, it will be necessary to remove the disturbed material, and place a sub-bedding layer consisting of compacted OPSS Granular B Type II beneath the Granular A. The bedding material should in all cases extend to the spring line of the pipe and should be compacted to at least 95 percent of the material's standard proctor maximum dry density (SPMDD). The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the sandy backfill materials or surrounding soil could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from spring line of the pipe to at least 300 mm above the top of pipe, should consist of OPSS Granular A or Granular B Type I with a maximum particle size of 25 mm. The cover material should be compacted to at least 95 percent of the SPMDD.

It should generally be possible to re-use the existing native in-organic soil, silty clay and glacial till as trench backfill, provided that they are not too wet to handle, place, and compact. Where the trench will be covered with hard surfaced areas, the type of material placed in the frost zone (between subgrade level and 1.8 m depth) should match the soil exposed on the trench walls for frost heave compatibility. Trench backfill should be placed in maximum 300 mm thick lifts and should be compacted to at least 95 percent of the material's SPMDD using suitable vibratory compaction equipment.

5.9 Pavement Design

In preparation for pavement construction, all topsoil, fill, disturbed, or otherwise deleterious materials (i.e., those materials containing organic material) should be removed from the roadway areas.

Pavement areas requiring grade raising to proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material (SSM). These materials should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of Standard Proctor Maximum Dry Density (SPMDD) using suitable compaction equipment.

The surface of the pavement subgrade should be crowned to promote drainage of the roadway granular structure. Perforated pipe sub-drains should be provided at subgrade level extending from the catch basins for a distance of at least 3 m longitudinally, parallel to the curb in two directions.

The pavement structure for new car parking areas may consist of:

Pavement Component	Thickness (mm)
Asphaltic Concrete OPSS Granular A Base OPSS Granular B Type II Subbase	50 150 300

The pavement structure for new access roadways and truck traffic areas should consist of:

Pavement Component	Thickness (mm)
Asphaltic Concrete OPSS Granular A Base OPSS Granular B Type II Subbase	90 150 400

The granular base and subbase used on this site should consist of Granular A and B Type II, respectfully, in conformance with OPSS.MUNI 1010 or City of Ottawa specification F-3147. The granular base and subbase materials should be uniformly compacted to 100 percent of the material's SPMDD using suitable vibratory compaction equipment. The asphaltic concrete should be compacted in accordance with Table 10 of OPSS.MUNI 310.

The composition of the asphaltic concrete pavement in car parking areas should be as follows:

- Superpave 12.5 Surface Course – 50 mm.

The composition of the asphaltic concrete pavement in access roadways and truck traffic areas should be as follows:

- Superpave 12.5 Surface Course – 40 mm.
- Superpave 19.0 Binder Course – 50 mm.

The pavement design should be based on a Traffic Category of Level B. The asphalt cement used on this project should be made with PG 58-34 asphalt cement on all lifts.

The above pavement designs are based on the assumption that the pavement subgrade has been acceptably prepared (i.e., where the trench backfill, and grade raise fill have been adequately compacted to the required density and the subgrade surface not disturbed by construction operations or precipitation). In this case, the pavement subgrade will be a compacted gravel layer placed for mass thermal storage. It could be necessary to place a gravel layer wrapped with woven geotextile to limit the migration of fine particles from pavement structure or surrounding soil into the voids in the gravel layer.

At the limits of construction or the end of the curb "return" (i.e., the start of the constant width portion of the access road, the asphaltic concrete should be milled back an additional 300 mm to a depth of 40 or 50 mm to accept the surface course asphaltic concrete.

The granular courses and subbase level should be tapered between the new and existing pavements by using 10 horizontal to 1 vertical tapers up or down as required, starting from beyond the limits of construction. Butt joints can be used along joints of new and existing parking areas.

5.10 Corrosion and Cement Type

One sample of soil from borehole 22-01 was submitted to Eurofins Environment Testing for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements. The results of this testing are provided in Appendix C and are summarized in Section4.4.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The sulphate results were compared with Table 3 of

Canadian Standards Association Standards A23.1-14 (CSA A23.1) and generally indicate a low degree of sulphate attack potential on concrete structures at this site. Accordingly, Type GU Portland cement should be acceptable for buried concrete substructures.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. Generally, the test results indicate an elevated potential for corrosion of exposed ferrous metal at the site which should be considered in the design of substructures.

6.0 ADDITIONAL CONSIDERATIONS

The soils at this site are sensitive to disturbance construction traffic, and frost when wet (i.e., saturated). Cobbles and boulders are present in the glacial till.

All footing and subgrade areas should be inspected by experienced geotechnical personnel of Golder Associates prior to filling or concreting to document that the correct/expected strata exist and that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill, pipe bedding, and pavement base and subbase materials should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction point of view.

The groundwater level monitoring device (i.e., monitoring well) installed at the site will require decommissioning at the time of construction in accordance with Ontario Regulation 903. It is therefore proposed that decommissioning of this device be made part of the construction contract. Some of the device may be useful during the initial stages of dewatering, if required, for monitoring the progress of the groundwater level lowering.

At the time of the writing of this report, only preliminary details for the proposed development were available. Golder Associates should review the final drawings and specifications for this project prior to tendering to confirm that the guidelines in this report have been adequately interpreted and to review some of our preliminary recommendations.

Signature Page

Golder Associates Ltd.

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KEY MAP

SCALE: 1:50,000 m

LEGEND

APPROXIMATE BOREHOLE/COREHOLE LOCATION, CURRENT INVESTIGATION

REFERENCE(S)

1. PROJECTION: TRANSVERSE MERCATOR, DATUM NAD 83, COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM CGVD28

DRAFT

0 10 20

1:500 METRES

CLIENT

IST PROPERTIES INC.

PROJECT

GEOTECHNICAL INVESTIGATION, 2700 SWANSEA CRESCENT, OTTAWA, ONTARIO

TITLE

SITE PLAN

CONSULTANT

WSP GOLDER

YYYY-MM-DD	2022-08-18
DESIGNED	----
PREPARED	ZS
REVIEWED	----
APPROVED	----

PROJECT NO.	CONTROL	REV.	FIGURE
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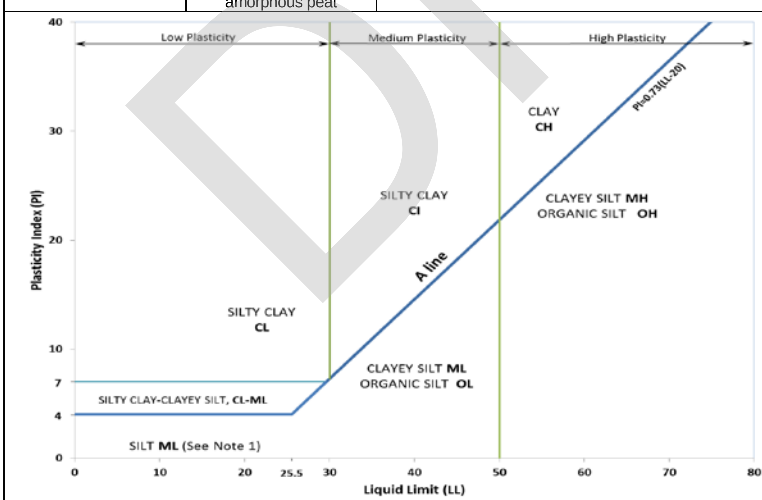
APPENDIX A

Record of Borehole Logs

The Golder Associates Ltd. Soil Classification System is based on the Unified Soil Classification System (USCS)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10} x D_{60}}$	Organic Content	USCS Group Symbol	Group Name
INORGANIC (Organic Content ≤30% by mass)	COARSE-GRAINED SOILS (>50% by mass is larger than 0.075 mm)	GRAVELS (>50% by mass of coarse fraction is larger than 4.75 mm)	Gravels with ≤12% fines (by mass)	Poorly Graded	<4	≤1 or ≥3	≤30%	GP	GRAVEL
				Well Graded	≥4	1 to 3		GW	GRAVEL
			Gravels with >12% fines (by mass)	Below A Line	n/a			GM	SILTY GRAVEL
				Above A Line	n/a			GC	CLAYEY GRAVEL
		SANDS (≥50% by mass of coarse fraction is smaller than 4.75 mm)	Sands with ≤12% fines (by mass)	Poorly Graded	<6	≤1 or ≥3		SP	SAND
				Well Graded	≥6	1 to 3		SW	SAND
			Sands with >12% fines (by mass)	Below A Line	n/a			SM	SILTY SAND
				Above A Line	n/a			SC	CLAYEY SAND

Organic or Inorganic	Soil Group	Type of Soil		Laboratory Tests	Field Indicators: Dilatancy	Dry Strength	Shine Test	Thread Diameter	Toughness (of 3 mm thread)	Organic Content	USCS Group Symbol	Primary Name
INORGANIC (Organic Content ≤30% by mass)	FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm)	SILTS	(Non-Plastic or PI and LL plot below A-Line on Plasticity Chart below)	Liquid Limit <50	Rapid	None	None	>6 mm	N/A (can't roll 3 mm thread)	<5%	ML	SILT
					Slow	None to Low	Dull	3mm to 6 mm	None to low	<5%	ML	CLAYEY SILT
					Slow to very slow	Low to medium	Dull to slight	3mm to 6 mm	Low	5% to 30%	OL	ORGANIC SILT
				Liquid Limit ≥50	Slow to very slow	Low to medium	Slight	3mm to 6 mm	Low to medium	<5%	MH	CLAYEY SILT
					None	Medium to high	Dull to slight	1 mm to 3 mm	Medium to high	5% to 30%	OH	ORGANIC SILT
		CLAYS	(PI and LL plot above A-Line on Plasticity Chart below)	Liquid Limit <30	None	Low to medium	Slight to shiny	~ 3 mm	Low to medium	0% to 30% (see Note 2)	CL	SILTY CLAY
				Liquid Limit 30 to 50	None	Medium to high	Slight to shiny	1 mm to 3 mm	Medium		CI	SILTY CLAY
				Liquid Limit ≥50	None	High	Shiny	<1 mm	High		CH	CLAY
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures								30% to 75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat								75% to 100%		PEAT



Note 1 – Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.
Note 2 – For soils with <5% organic content, include the descriptor "trace organics" for soils with between 5% and 30% organic content include the prefix "organic" before the Primary name.

Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML.
For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between "clean" and "dirty" sand or gravel.
For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML.
A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to indicate a range of similar soil types within a stratum.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (*i.e.,* SAND and GRAVEL)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable
> 5 to 12	some
≤ 5	trace

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.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); 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.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
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
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

Term	SPT 'N' (blows/0.3m)[1]
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.
2. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grainsize. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1,2] (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$ - u)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c′	effective cohesion
c_u, s_u	undrained shear strength (ϕ = 0 analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$
2 shear strength = (compressive strength)/2

PROJECT: 22514086EX

LOCATION: N 5026760.78; E 374229.05

RECORD OF BOREHOLE: 22-01

BORING DATE: July 5, 2021

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

HAMMER TYPE: AUTOMATIC

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢ ○W ⊣ Wl : 20 40 60 80

DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
GROUND SURFACE	82.69					
ASPHALTIC CONCRETE	0.00					
FILL - (SP) GRAVELLY SAND, some silt; brown to black (PAVEMENT STRUCTURE); non-cohesive, moist	0.08	1	AS	-		
(CI/CH) SILTY CLAY to CLAY, trace to some gravel; grey brown (WEATHERED CRUST); cohesive, w~PL, stiff to very stiff	82.08 0.61	2	SS	5		
		3	SS	4		Silica Sand
(CI/CH) SILTY CLAY to CLAY, some gravel; grey; cohesive, w>PL, stiff	80.56 2.13	4	SS	WR		
(SM/ML) gravelly SILTY SAND to sandy SILT, some low plastic fines; grey, possible cobbles and boulders (GLACIAL TILL); slightly cohesive, moist, loose to compact	79.03 3.66	5	SS	21		Bentonite Seal
		6	SS	8	MH	Silica Sand
Highly Weathered BEDROCK	77.51 5.18	7	SS	33	MH	PVC #10 Slot Screen
		8	SS	37		
		9	SS	>50		
End of Borehole	75.77 6.92					

Note(s):

1. Water level measured at a depth of 6.39 m (Elev. 76.3 m) on August 12, 2022

DEPTH SCALE

1 : 50

WSP GOLDER

LOGGED: PAK/EIT

CHECKED:

RECORD OF BOREHOLE: 22-01A

PROJECT: 22514086EX

LOCATION: N 5026771.85; E 374221.65

BORING DATE: July 5, 2021

SHEET 1 OF 1

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (20, 40, 60, 80) nat V. + Q - ● rem V. ⊕ U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20, 40, 60, 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		82.65							
0		For Soil Stratigraphy refer to Record of Borehole 22-01		0.00							
2				80.77 / 1.88				>96 + ; >96 + ; >96 + ; ⊕ + ; ⊕+ ; ⊕+ ; >96 +			
4		End of Borehole		78.69 / 3.96							

DEPTH SCALE

1 : 50

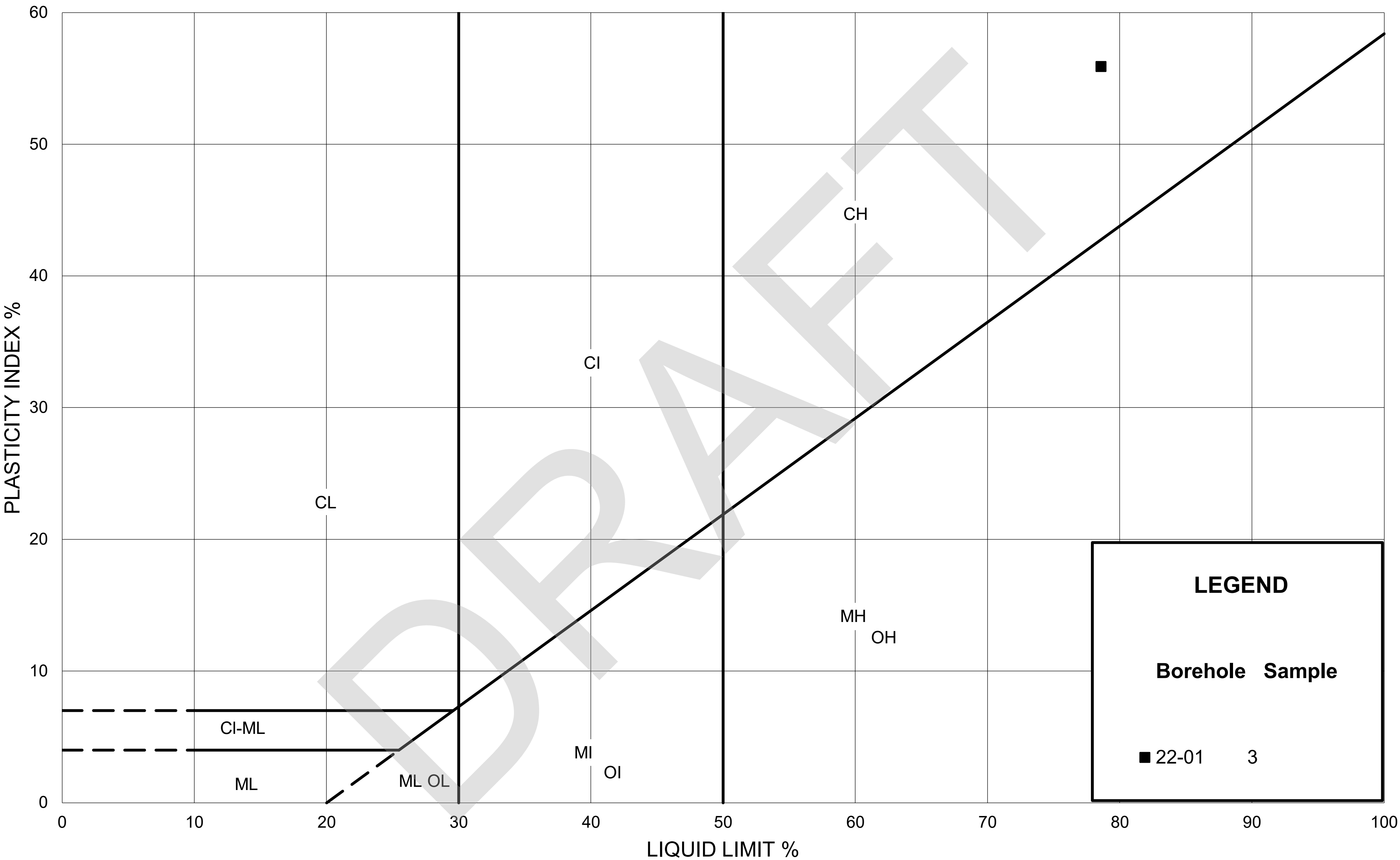
WSP GOLDER

LOGGED: PAK/EIT

CHECKED:

APPENDIX B

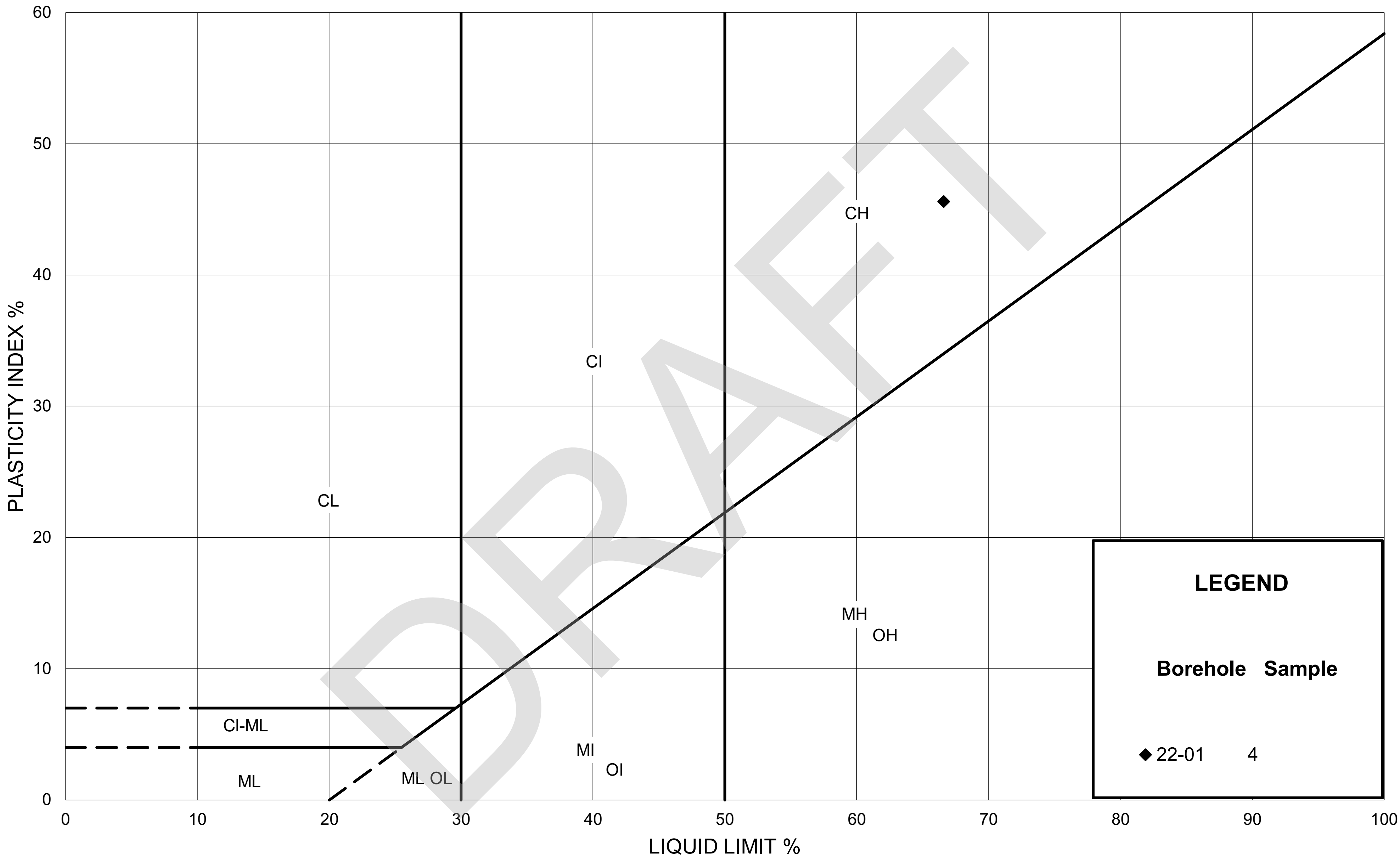
Results of Laboratory Testing



GOLDER
MEMBER OF WSP

PLASTICITY CHART
SILTY CLAY TO CLAY (WEATHERED CRUST)

Figure:	B-1
Project:	22514086EX
Created By: MI	Checked By: JB



GOLDER
MEMBER OF WSP

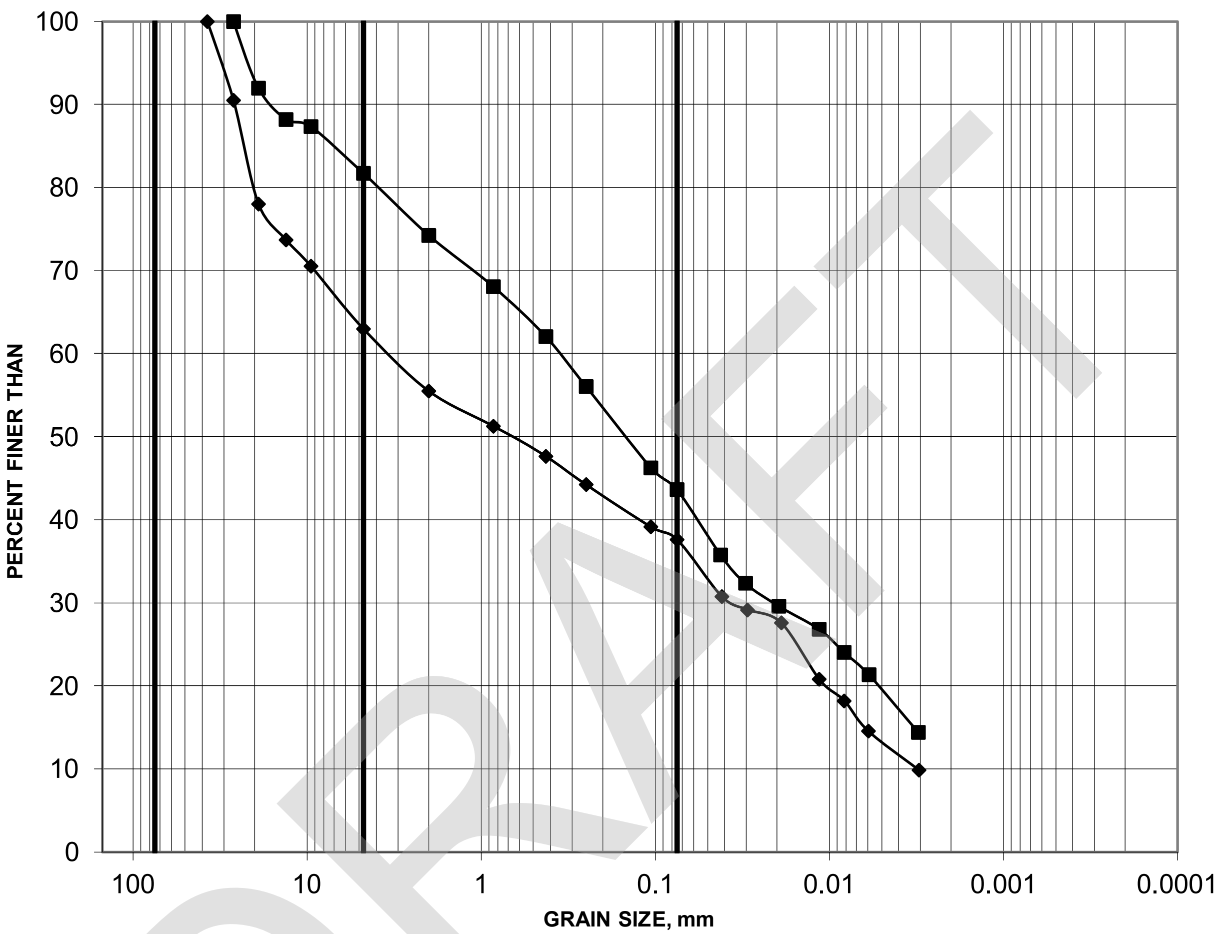
PLASTICITY CHART
SILTY CLAY TO CLAY

Figure:	B-2
Project:	22514086EX
Created By: MI	Checked By: JB

GRAIN SIZE DISTRIBUTION

FIGURE B-3

GLACIAL TILL



COBBLE SIZE	GRAVEL SIZE		SAND SIZE			SILT AND CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

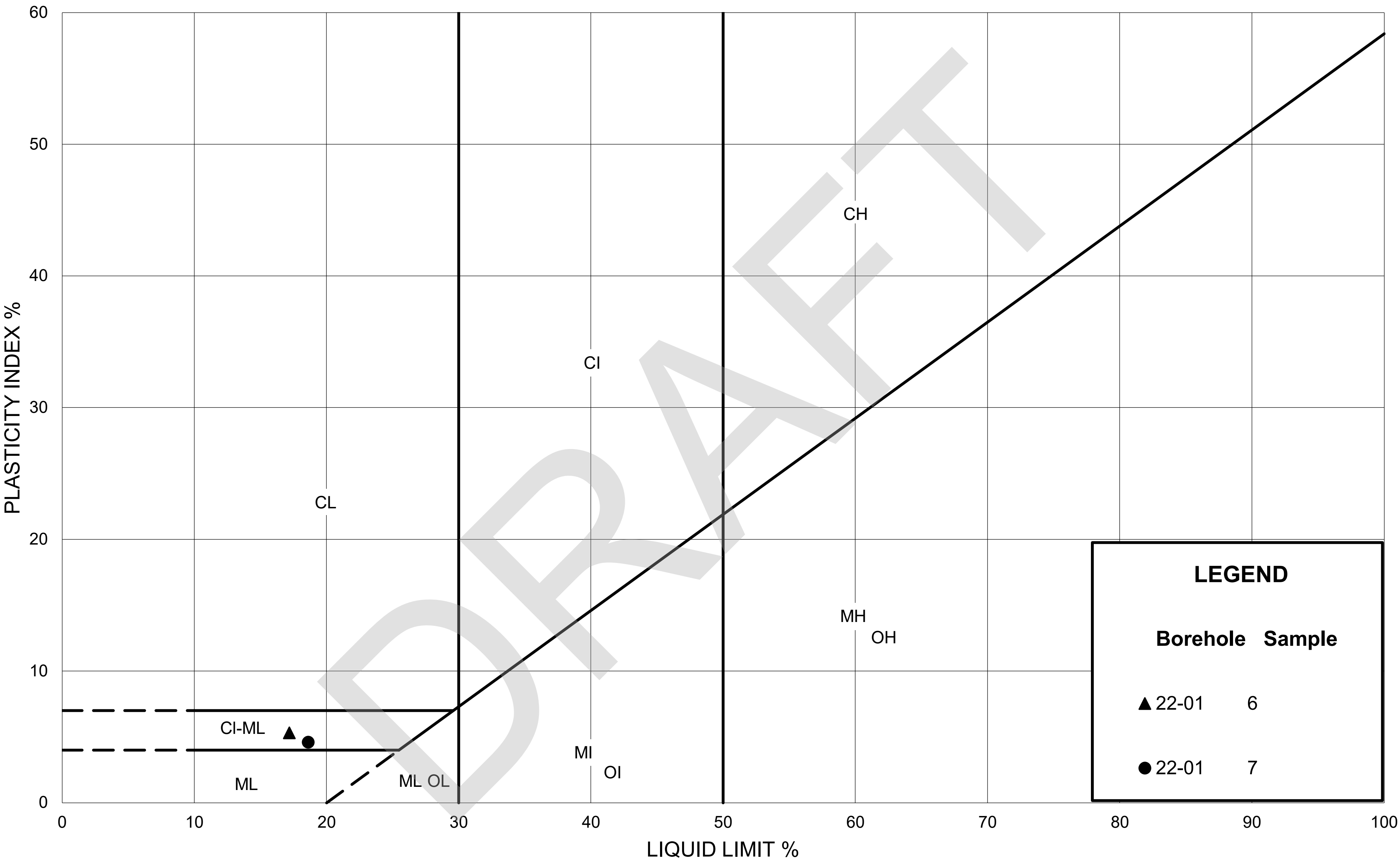
Borehole	Sample	Depth (m)	Constituents (%) Gravel	Sand	Silt	Clay
■ 22-01	6	3.81-4.42	18	38	32	12
◆ 22-01	7	4.57-5.18	37	25	30	8

Project: 22514086EX

GOLDER
MEMBER OF WSP

Created by: MI
Checked by: JB

https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2022/22514086EX/Figures/



GOLDER
MEMBER OF WSP

PLASTICITY CHART
GLACIAL TILL

Figure:	B-4
Project:	22514086EX
Created By: MI	Checked By: JB

APPENDIX C

Results of Chemical Analysis

Certificate of Analysis

Client:	Golder Associates Ltd. (Ottawa) 1931 Robertson Road Ottawa, ON K2H 5B7	Report Number:	1981832
		Date Submitted:	2022-07-18
		Date Reported:	2022-07-26
		Project:	22514086EX
		COC #:	893464
Attention:	Chaitanya Raj Goyal		
PO#:			
Invoice to:	Golder Associates Ltd. (Ottawa)		

Dear Chaitanya Raj Goyal:

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: ______________________________

Addrine Thomas, Inorganics Supervisor

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

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on the scope of accreditation. The scope is available at: https://directory.cala.ca/.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is licensed by the Ontario Ministry of the Environment, Conservation, and Parks (MECP) for specific tests in drinking water (license #2318). A copy of the license is available upon request.

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:	Golder Associates Ltd. (Ottawa) 1931 Robertson Road Ottawa, ON K2H 5B7	Report Number:	1981832
Attention:	Chaitanya Raj Goyal	Date Submitted:	2022-07-18
PO#:		Date Reported:	2022-07-26
Invoice to:	Golder Associates Ltd. (Ottawa)	Project:	22514086EX
		COC #:	893464

Group	Analyte	MRL	Units	Guideline	Lab I.D. 1638137 Sample Matrix Soil Sample Type Sampling Date 2022-07-05 Sample I.D. 22-01 Sa3 / 5-7'
Anions	Cl	0.002	%		0.043
	SO4	0.01	%		0.05
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.52
	pH	2.00			7.88
	Resistivity	1	ohm-cm		1920

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:	Golder Associates Ltd. (Ottawa) 1931 Robertson Road Ottawa, ON K2H 5B7	Report Number:	1981832
Attention:	Chaitanya Raj Goyal	Date Submitted:	2022-07-18
PO#:		Date Reported:	2022-07-26
Invoice to:	Golder Associates Ltd. (Ottawa)	Project:	22514086EX
		COC #:	893464

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 425856 **Analysis/Extraction Date** 2022-07-19 **Analyst** IP **Method** Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	81	90-110
pH	6.27	101	90-110
Resistivity			
Run No 426022 **Analysis/Extraction Date** 2022-07-21 **Analyst** AsA **Method** C CSA A23.2-4B			
Chloride	<0.002 %		90-110
Run No 426257 **Analysis/Extraction Date** 2022-07-26 **Analyst** IP **Method** AG SOIL			
SO4	<0.01 %	97	70-130

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

Golder Associate Technical Memorandum No. 07-1121-0135

TECHNICAL MEMORANDUM

Golder Associates Ltd.
32 Steacie Drive
Kanata, ON, Canada K2K 2A9

Telephone: 613-592-9600
Fax Access: 613-592-9601

TO: Mr. Sean Montgomery
Project Manager
Canderel Stoneridge Equity Group Inc.

DATE: August 28, 2007

FROM: Christopher Phillips, GAL - Mississauga
Michel St-Louis, GAL - Ottawa
Michael Snow, GAL - Ottawa

JOB NO: 07-1121-0135

EMAIL: cphillips@golder.com

RE: **SEISMIC STUDY – SITE CLASSIFICATION**
MASW DATA PROCESSING AND RESULTS –
2700 SWANSEA CRESCENT, OTTAWA, ONTARIO

This memorandum presents the processing and results of the MASW test performed for the proposed warehouse building located at 2700 Swansea Crescent, Ottawa, Ontario for Canderel Management Inc. The MASW survey was performed as part of the geotechnical investigation for the site.

A geotechnical investigation was carried out by Golder Associates in 2002 at 2700 Swansea Crescent.

The information gathered to date on this property (2700 Swansea Crescent) and adjacent lands, suggests that sensitive silty clays extend to depths greater than four (4) metres from the present ground surface. Regional geologic maps indicate bedrock to be of the order of 10 metres deep.

1.0 METHODOLOGY

The Multichannel Analysis of Surface Waves (MASW) method measures variations in surface wave velocity with increasing distance and wavelength and can be used to infer the rock/soil types, stratigraphy and soil conditions. A typical MASW survey requires a seismic source, to generate surface-waves, and a minimum of two geophone receivers, to measure the ground response at some distance from the source. Surface waves are a special type of seismic wave whose propagation is confined to the near surface medium. The depth of penetration of a surface-wave into a medium is directly proportional to its wavelength. In a non-homogeneous medium surface-waves are dispersive, i.e. each wavelength has a characteristic velocity owing to the subsurface heterogeneities within the depth interval that particular wavelength of surface-wave propagates through. The relationship between surface-wave velocity and wavelength is used to obtain the shear-wave velocity and attenuation profile of the medium with increasing depth.

The seismic source used can be either active or passive, depending on the application and location of the survey. Examples of active sources include explosives, weight-drops, and vibrating pads. Examples of passive sources are road traffic, micro-tremors, and water-wave action (in near-shore environments). The geophone receivers measure the wave-train associated with the surface wave travelling from a seismic source at different distances from the source.

The participation of surface-waves with different wavelengths can be determined from the wave-train by transforming the wave-train results into the frequency domain. The surface-wave velocity profile with respect to wavelength (called the 'dispersion curve') is determined by the delay in wave propagation measured between the geophone receivers. The dispersion curve is then matched to a theoretical dispersion curve using an iterative forward-modelling procedure. The result is a shear-wave velocity profile of the tested medium with depth, which can be used to estimate the dynamic shear modulus of the medium as a function of depth.

2.0 FIELD WORK

The MASW field work was conducted on July 19th, 2007, by Golder personnel. The approximate location of the MASW test line was approximately 44 meters east of Swansea Crescent, along a line approximately parallel to the road. The MASW test line was oriented in a North-South direction along the approximate centre line of the proposed building footprint.

A series of 22 low frequency (4.5 Hz) geophones were laid out at 2 metre intervals. A seismic gun was used as the seismic source for this investigation. The seismic source location was offset a distance of 30 m from the end and collinear with the geophone array. A total of 3 shots were collected for the MASW line.

3.0 DATA PROCESSING

Processing of the MASW test results consisted of the following main steps:

1. Transformation of the time domain data into the frequency domain using a fast-Fourier transform (FFT) for each source location;
2. Calculation of the phase for each frequency component;

3. Linear regression to calculate phase velocity for each frequency component;
4. Filtering of the calculated phase velocities based on the Pearson correlation coefficient (r^2) between the data and the linear regression best fit line used to calculate phase velocity;
5. Generation of the dispersion curve by combining calculated phase velocities for each shot location of a single MASW test; and
6. Generation of the stiffness profile, through forward iterative modelling and matching of model data to the field collected dispersion curve.

Processing of the MASW data was completed using the SeisImager/SW software package (Geometrics Inc.). The calculated phase velocities for each shot point for a given test were combined and the dispersion curve generated by choosing the minimum phase velocity calculated for each frequency component. Shear wave velocity profiles were generated through inverse modelling to best fit the calculated dispersion curves.

The minimum measured surface wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was in the range of 9 Hz for the MASW test location.

4.0 RESULTS

The MASW test results are presented in Table 1, which presents the calculated shear wave velocity profile measured from the field testing, and on Figure 1, which present a graphical representation of the shear wave velocity profile with depth.

The MASW results indicate a near surface layer with a shear wave velocity in the range of 280 meters per second, present to a depth of approximately 7.0 meters below ground surface (mbgs). Velocity increases quickly at a depth of 8.9 mbgs, to approximately 650 m/s, and is relatively consistent to a depth of 30 mbgs.

The field collected dispersion curves are compared with the model generated dispersion curves in Figure 2. There is excellent correlation between the field collected and model calculated dispersion curves, with a root mean squared error of 3.99%.

The MASW results report an average shear wave velocity, calculated from the time taken for the shear wave to travel from surface to a depth of 30 meters, of 470 m/s, which according to the National Building Code of Canada, 2005 (NBCC2005) site classification for seismic site response classifies this site as Site Class C (Very Dense Soil and Soft Rock), which would be appropriate.

5.0 CLOSURE

We trust that these results meet your current needs. If you have any questions or require clarification, please contact Michael Snow at your convenience.

CRP/MSTL/MSS/crp/lb
n:\active\2007\1121 - geotechnical\07-1121-0135 canderel stoneridge\07-1121-0135 tech memo masw results.doc

Attachments:

Table 1 – Shear Wave Velocity Profile

Figure 1 – MASW Shear Wave Velocity Profile

Figure 2 – MASW Field Data and Model Dispersion Curve Comparison

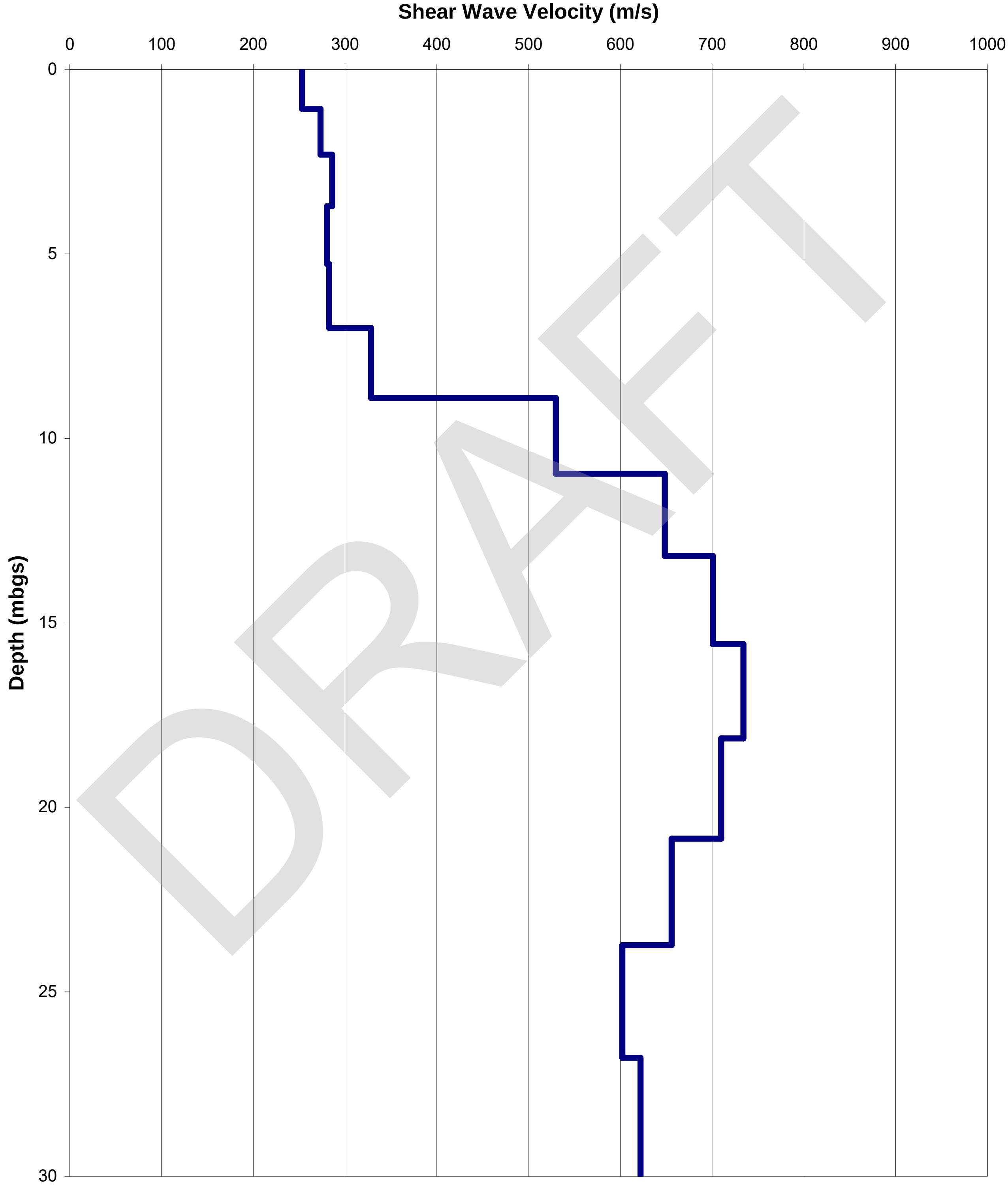
TABLE 1
Shear Wave Velocity Profile - MASW Test Results
2700 Swansea Crescent
Ottawa, Ontario

Model Layer (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom			
0.0	1.1	1.1	253	0.00423
1.1	2.3	1.2	273	0.00452
2.3	3.7	1.4	286	0.00490
3.7	5.3	1.6	280	0.00558
5.3	7.0	1.7	283	0.00613
7.0	8.9	1.9	328	0.00577
8.9	11.0	2.1	530	0.00389
11.0	13.2	2.2	648	0.00343
13.2	15.6	2.4	701	0.00341
15.6	18.1	2.6	734	0.00348
18.1	20.9	2.7	710	0.00383
20.9	23.7	2.9	656	0.00440
23.7	26.8	3.0	602	0.00506
26.8	30.0	3.2	622	0.00517
			Vs Average to 30 mbgs (m/s)	**470**

Inputted By: CRP

Checked By: [signature]

FIGURE 1
Shear Wave Velocity Profile - MASW Test Results
2700 Swansea Crescent
Ottawa, Ontario



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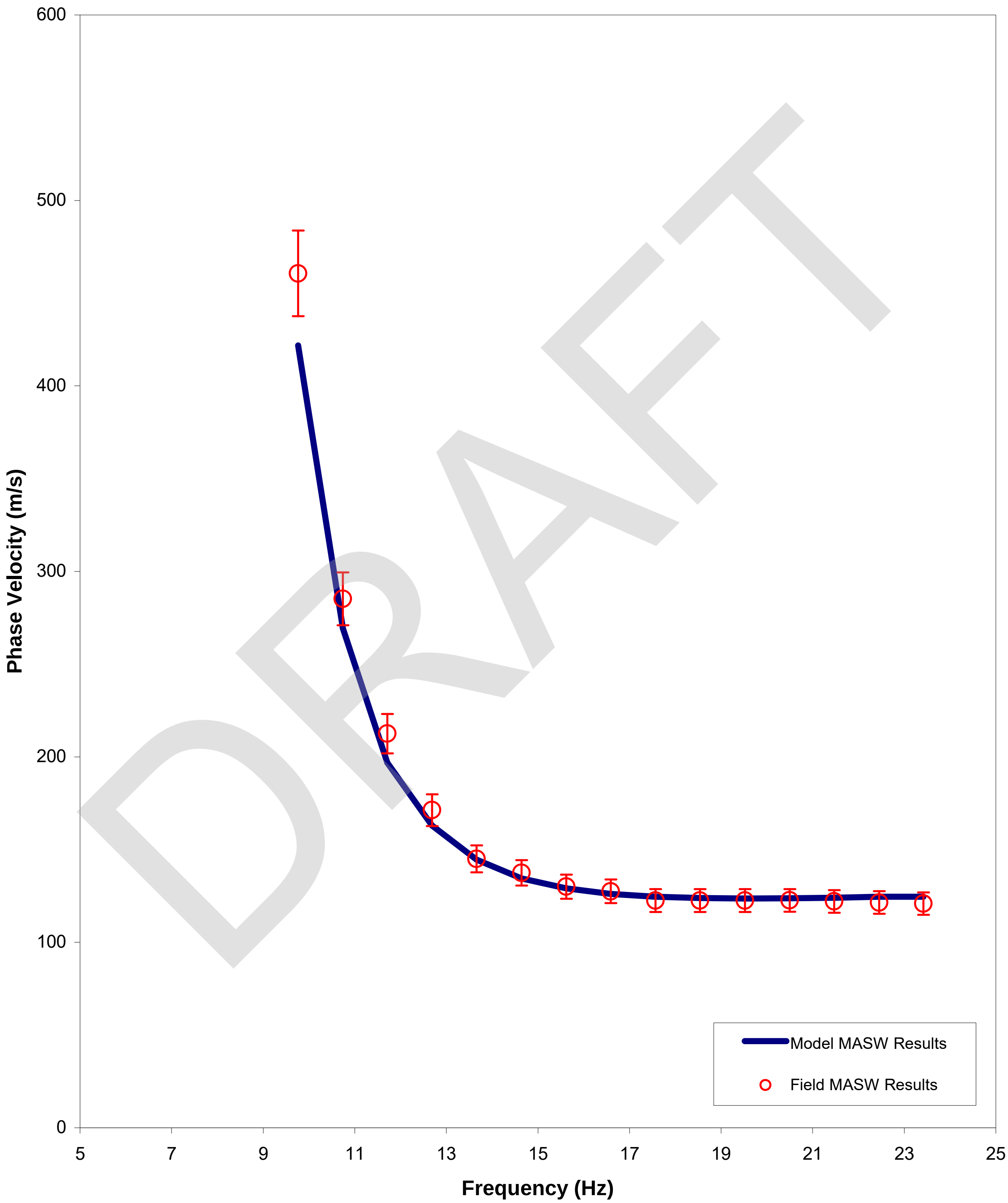
Golder Associates

Checked By:

August 2007

07-1121-0132

FIGURE 2
Dispersion Curve Comparison
Field Measured vs. Modelled Results
2700 Swansea Crescent
Ottawa, Ontario



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Golder Associates