October 2017

GEOTECHNICAL EXPLORATION

PROPOSED OBSERVATION TOWER POINT PELEE NATIONAL PARK LEAMINGTON, ONTARIO

Submitted to:
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REPORT

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1.0 INTRODUCTION

This report presents the results of the geotechnical exploration and testing carried out for the design of the proposed observation tower to be constructed within Point Pelee National Park near Leamington, Ontario. The general location of the site is shown on the Key Plan, Figure 1. The proposed tower is to be constructed in the southern portion of the park adjacent to the current staff parking lot.

The purpose of the exploration and testing was to assess the subsurface soil and groundwater conditions at the proposed observation tower location and to provide geotechnical engineering recommendations for the design of the tower foundation. Authorization to proceed with the work was provided in an e-mail from Mr. Patrick Robitaille, P. Eng. of Dillon Consulting Limited (Dillon) on April 20, 2017.

Important information on the limitations of this report is attached.

2.0 FIELD PROCEDURES

The field work was carried out on May 31, 2017 at which time a single borehole, identified as borehole (BH)-101 was drilled at the approximate location shown on the Location Plan, Figure 1, to a depth of about 15.7 metres (m) below the existing ground surface. The soil stratigraphy encountered in BH-101 is shown in detail on the Record of Borehole sheet following the text of this report. Standard penetration testing and sampling was carried out at appropriate intervals of depth in the borehole using conventional 38 millimetre (mm) inside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures (ASTM D 1586) with an automatic hammer.

Groundwater seepage conditions were observed in the borehole during drilling and these observations are provided on the Record of Borehole sheet. Upon completion of drilling, sampling and in situ testing, the borehole was backfilled in accordance with current regulations and the interlocking concrete pavers replaced.

All of the samples obtained during the exploration were brought to our laboratory in Windsor, Ontario for further examination and representative routine classification testing. The results of the field and laboratory testing are shown on the Record of Borehole and on Figures 2 and 3, attached.

The borehole was located in the field by a member of our engineering staff who also supervised the drilling, sampling and testing, logged the borehole, cared for the samples obtained and provided temporary traffic and pedestrian control.

The ground surface elevation at the borehole location was surveyed by Golder Associates Ltd. (Golder) staff and referenced to a temporary benchmark. The benchmark is described as the finished floor of the existing washroom facility south of the borehole. The location of this point was assigned an elevation of 100.0 m for the purposes of this project.

Subsequent to the investigation, the proposed observation tower was relocated about 45 metres to the west of the initially proposed location.

3.0 SUBSURFACE CONDITIONS

3.1 General

The subsurface conditions encountered in the borehole advanced at the site are shown on the attached Record of Borehole sheet. The following paragraphs have been simplified in terms of major soil strata for the purposes of geotechnical design. The soil boundaries indicated have been inferred from non-continuous samples and observations of sampling and drilling resistance and typically represent a transition from one soil type to another. They should not necessarily be interpreted to represent exact planes of geological change. Further, the subsurface conditions should be expected to vary beyond the borehole location.

3.2 Soil Conditions

The soil conditions encountered in the borehole generally consisted of the existing concrete pavers and related granular fill overlying layers of sand and silty sand which were, in turn, underlain by silty clay till.

Concrete pavers were present at the ground surface at the borehole location. Beneath the pavers, granular fill was present and extended to a depth of about 0.5 m.

Underlying the fill, a layer of sand about 1.7 m thick was encountered. The sand had 'N' values, as determined in the standard penetration testing, of 2 and 22 blows per 0.3 m with water contents of about 18 to 20 per cent.

A layer of silty sand, which was about 3.5 m thick, was encountered beneath the sand. The silty sand had 'N' values of 32 to 52 blows per 0.3 m with water contents of about 11 to 21 per cent. A grain size distribution curve for a sample of the silty sand recovered from the borehole is provided on Figure 2.

A subsequent layer of sand was encountered beneath the silty sand. The lower sand layer was about 3.1 m thick. The lower sand had 'N' values of 21 and 30 blows per 0.3 m and water contents of about 19 and 20 per cent.

Beneath the lower sand, a subsequent layer of silty sand about 2.6 m thick was encountered. The lower silty sand had 'N' values of 8 and 9 blows per 0.3 m with water contents of about 21 and 22 per cent.

Silty clay till was encountered beneath the granular deposits at a depth of about 11.3 m. BH-101 was terminated in the silty clay till at a depth of about 15.7 m after exploring it for about 4.4 m. The silty clay till had SPT 'N' values of 5 to 11 blows per 0.3 m and water contents of about 16 to 25 per cent. Field vane shear testing in the silty clay till indicated an undrained shear strength greater than 96 kilopascals (kPa). The silty clay till had plastic and liquid limits of about 20 and 32 per cent, respectively, based on a single Atterberg limits determination. These data are shown on the Plasticity Chart, Figure 3, and indicate an inorganic clay of intermediate plasticity.

The available bedrock mapping is sparse for this area; however, for preliminary consideration, bedrock may be assumed to be about 30 metres below ground surface.

3.3 Groundwater Conditions

Groundwater seepage conditions were observed in the borehole during drilling and these observations are provided on the Record of Borehole sheet. Groundwater was encountered at a depth of about 0.7 m or at about elevation 98.8 m during drilling on May 31, 2017.

Groundwater levels should be expected to fluctuate seasonally and in response to significant precipitation events and changes in the Lake Erie water level.

4.0 DISCUSSION

Based on information provided by Dillon, the proposed observation tower will consist of a combination of steel and wood construction with three primary supports connected to the foundation system. Preliminary design information indicates that the proposed observation tower will be supported on piles. Drilled concrete piles (caissons) or a mass concrete footing (raft foundation) are considered to be appropriate from a geotechnical perspective. In addition, consideration could be given to supporting the proposed tower on a series of micropiles socketed and grouted into the bedrock. It is understood that the vertical support reactions can vary from about 2,680 kilonewtons (kN) in compression (downward loading) to 1,550 kN in tension (upward loading).

This section of the report provides our interpretation of the factual geotechnical data obtained during the exploration and it is intended for the guidance of the design engineer. Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project. Contractors bidding on or undertaking the works should make their own interpretation of the subsurface information provided as it affects their proposed construction methods, equipment selection, scheduling and the like.

4.1 Foundations

4.1.1 Shallow Foundations

A mass concrete raft foundation bearing on the undisturbed native upper sand or silty sand between about elevation 96 and 98 m may be designed for geotechnical resistance at Serviceability Limit States (SLS) of 200 kPa and a factored geotechnical resistance at Ultimate Limit States (ULS) of 300 kPa. The SLS resistance is based on total settlements less than about 25 mm and differential settlements being about half this value.

For detailed analyses, an unfactored unit modulus of subgrade reaction, $k_{0.3}$, of 40 meganewtons per cubic metre (MN/m^3) may be used for a raft foundation bearing on the undisturbed and properly dewatered sand and silty sand between about elevation 96 and 98 m. The unfactored modulus of subgrade reaction should be modified based on the actual geometry of the raft using the following equation:

$$k = k_{0.3}\left(\frac{B + 0.3}{2B}\right)^2$$

where k = unfactored modulus of subgrade reaction (MN/m^3);

$k_{0.3}$ = unfactored unit modulus of a 0.3 m by 0.3 m loaded area (MN/m^3); and

B = maximum horizontal dimension of the continuous raft (m).

The analyses should also consider unit moduli of half and twice the above-noted value. Should the design be sensitive to the subgrade reaction modulus, additional analyses will be required to refine the value. The design of a raft foundation should be such that the resultant total reaction load remains within the middle third of the raft and that the maximum edge pressure does not exceed the geotechnical resistances provided above and should incorporate any overturning moments due to lateral loads.

Resistance to sliding between the raft foundation and the sand to silty sand may be assessed using an unfactored coefficient of interface friction, tan δ, of 0.58.

Upward loading would be resisted by the mass of concrete raft as transmitted to the structure supports through appropriate mechanical anchors.

4.1.2 Deep Foundations

To achieve the resistance to overturning loads ("pull out" resistance), driven steel H-piles are not considered appropriate for this site. Deep foundations should consist of cast-in-place concrete piles or drilled shafts ("caissons"). The unfactored vertical resistance in compression can be calculated using the following:

$$P = \sum_{z=0}^{L} C q_s \Delta z + A_t q_b$$

where C = circumference of pile (m);

L = embedded length of pile (m) divided into segments of length Δz (m);

A_t = area of pile base;

q_s = unit shaft friction (kPa); and

q_b = bearing resistance at pile base (kPa).

The unit shaft friction, q_s, and bearing resistance, q_b, in the sands and silty sands can be calculated as:

$$q_s = \beta \, \sigma_v' \qquad q_b = N_t \, \sigma_b'$$

where β = 0.3;

N_t = 30;

σ_v' = vertical effective stress adjacent to the pile; and

σ_b' = vertical effective stress at the base of the pile.

The effective stresses should be based on a soil unit weight of 19 kN/m^3 and a groundwater level at ground surface.

For any portions of the pile within the silty clay till, the unit shaft friction, q_s, and bearing resistance, q_b, can be calculated as:

$$q_s = \alpha\, S_u \qquad q_b = N_t\, S_u$$

where $\alpha = 0.6$;

$S_u = 100$ kPa; and

$N_t = 6$.

The unfactored uplift resistance of the pile can be assessed by removing the contribution of toe resistance, $A_t\, q_b$, from the above equations.

Based on our preliminary calculations using the available data, three 2.0 m diameter caissons about 18.5 m in length will be required to support the compressive and tension loads. The designer should confirm that these are adequate for the load cases provided and any other loading combinations considered and revise the caisson depth and diameter, if and as appropriate.

The unfactored resistance to lateral loads for each pile can be assessed using a triangular earth pressure distribution acting over an area equivalent to two pile diameters using the following:

$$R = \frac{1}{2} K_p\, \gamma\, L^2\, d$$

where R = unfactored lateral resistance (kN);

$K_p = 3.0$;

$\gamma = 9$ kN/m^3;

L = embedded length of pile; and

d = two times pile diameter.

For modelling of soil-foundation interaction using an equivalent spring approach, the following horizontal moduli of subgrade reaction may be used:

Sand – 4 MPa/m

Silty Clay – 15 MPa/m

4.1.3 Micropiles

Micropiles may also be considered for the support of the observation tower. Micropiles are commonly constructed using heavy-wall, threaded-joint, steel pipe on the order of 100 to 200 millimetre diameter. Once drilled to depth, grout is injected under pressure to cement the casing in place and pressurize the surrounding ground. In some cases, multiple stages of grouting are used to develop higher vertical capacities. Depending on vertical loads, centralized steel reinforcing rods can also be installed to increase the cross-section area of steel and strength of the micropile. Axial capacity of micropiles is highly dependent upon diameter, how many stages of grouting are completed, grouting pressures and installation methods. Commonly, specialist micropile installation contractors will prepare designs based on their particular equipment and methods in accordance with design loads and performance specifications and detailed field-scale load testing is completed to prove final capacities in tension and/or compression. One disadvantage of micropiles is that, because of their small diameter and slenderness, lateral loading is usually addressed by installing the micropiles on a batter and lateral load transfer is not as efficient compared to other foundation systems.

For this site and the loads being considered, micropile groups (with some battered piles) grouted into the bedrock will likely be required to develop the appropriate resistances. The specialist micropile contractor should be responsible for the detailed design of their selected foundation system based on their particular equipment and methodologies.

4.1.4 Seismic Design

The site classification for seismic response presented in Table 4.1.8.4 of the 2012 Ontario Building Code (OBC) relates to the average properties of the upper 30 m of soil beneath the structure. Based on information obtained during the geotechnical exploration, available well records and our knowledge of the subsurface conditions in the area of the site, the subsurface soil conditions are generally comprised of sand and silty sand underlain by an extensive deposit of silty clay till. The characteristics of the soils in the upper 30 m correspond to ‘stiff soil’ according to the OBC. Thus, Site Class ‘D’ is appropriate for this site.

For Leamington, Table 1.2 of the 2012 OBC Supplementary Standard SB-1 defines the Peak Ground Acceleration (PGA) as 0.091 g (g = acceleration due to gravity) and the damped spectral response accelerations as Sa(0.2) = 0.170, Sa(0.5) = 0.092, Sa(1.0) = 0.047 and Sa(2.0) = 0.015 for the reference ground conditions. Based on the above spectral response acceleration values and the Site Class ‘D’ designation, the Fa and Fv values would be 1.3 and 1.4, respectively, for this site.

4.2 Excavations

Based on the explorations, excavations for the proposed observation tower raft foundation will encounter native sand and silty sand. All excavations should be completed in accordance with the current Occupational Health and Safety Act (OHSA) criteria and, in particular, OHSA Regulation 213/91 which specifically addresses Construction Projects. The sand and silty sand would be classified as Type 4 soils given the shallow groundwater level. If these granular materials are properly dewatered using proactive dewatering techniques (e.g. vacuum well points, eductors, deep wells), they could be classified as Type 3 soils under the act.

Care will be required to ensure that adequate support is provided for all existing utilities located within the zone of influence of the excavations as defined by a line drawn upwards and outwards from the base of the excavation at an inclination of 2 horizontal to 1 vertical.

Should an excavation support system be used to reduce the lateral extent of the excavations and/or maintain stability, it should be noted that these systems only provide protection for the workmen once in place. The design of any temporary support systems should be the responsibility of the contractor and take into account line, point and area loads acting on the system, surcharge loads, soil loads and hydrostatic loads.

Should drilled piles be used to support the new observation tower, temporary steel liners should be advanced in conjunction with the augering to support the sides of the auger holes. It will be important to maintain a head of water within the liner coincident with the ground surface while augering through the sands and silty sands. This head of water should be maintained until such time that the liner has been advanced at least 1 metre into the silty clay till. If the caissons do not extend to the silty clay till, the water head should be maintained and the concrete placed using tremie methods following appropriate cleaning of the hole bottom.

4.3 Temporary Control of Groundwater

Excavations at the site for a raft foundation will require either proactive dewatering using appropriate systems which may include vacuum well points, eductors, large diameter drilled wells or the like. Alternatively, a diaphragm wall system may be used to isolate the tower foundation area from the groundwater. Such a system could consist of a concrete secant pile wall enclosure or driven steel sheet pile wall enclosure installed at least 1 metre into the silty clay till. Depending on the type and size of support system required, bracing, tie backs or the like may be required for overall stability. As indicated above, the design of all temporary support systems should be the responsibility of the contractor.

If vacuum well points or the like are to be used, a Permit to Take Water will likely be required.

4.4 Backfill

Backfill adjacent to the foundations should consist of free draining Granular ‘B’, Type II. The Granular ‘B’ backfill should be placed in loose lift thicknesses not exceeding 300 mm and be uniformly compacted to at least 95 per cent of standard Proctor maximum dry density.

4.5 Additional Geotechnical Explorations, Inspection and Testing

It is recommended that geotechnical involvement continues throughout the design, tender and construction phases of this project. In addition to a review of the geotechnical aspects of the contractor's work plans, a regular program of geotechnical inspections and materials testing should be carried out during construction to confirm that the subsurface conditions encountered are consistent with those encountered during the exploration, that the intent of this report is met, and that the various material and project specifications are being achieved.

If micropiles to rock or very deep drilled concrete piles are to be used for the observation tower foundations, it would be beneficial to advance a borehole to about 3 pile diameters or 3 metres below the foundation termination elevation to assist in managing risks for changed conditions and to confirm the assumptions regarding the founding soil/bedrock.

We trust that this report provides sufficient geotechnical information presently required. Should any point require further clarification, or when we can be of additional assistance, please contact this office.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
Oct. 11/17
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The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

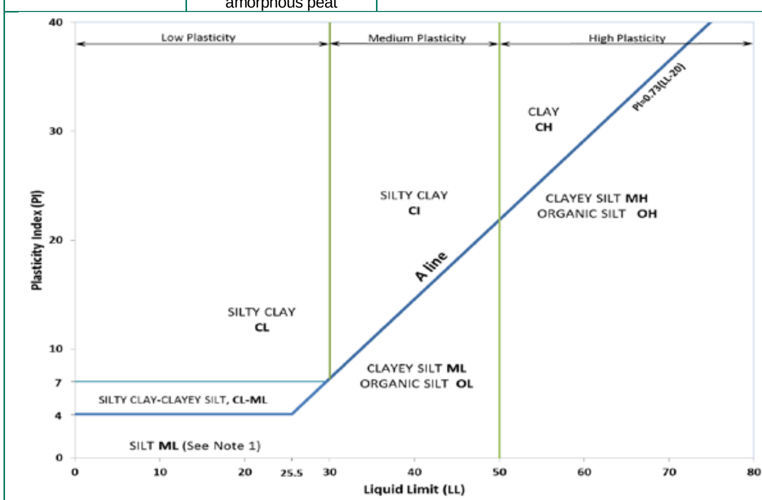
Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

METHOD OF SOIL CLASSIFICATION

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, SAND and CLAY)
> 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.).

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
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
FS	Foil sample
GS	Grab Sample
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size
TP	Thin-walled, piston – note size
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 which are 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 - 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 overburden pressure effects.
2. Definition of compactness descriptions based on SPT 'N' ranges from Terzaghi and Peck (1967) and correspond to typical average N_{60} values.

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 $= (w_l - w_p)$
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 $= \sigma'_p / \sigma'_{vo}$

(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: 1776745

RECORD OF BOREHOLE BH-101

LOCATION: REFER TO LOCATION PLAN

BORING DATE: May 31, 2017

DRILLING CONTRACTOR: Henderson Drilling Inc.

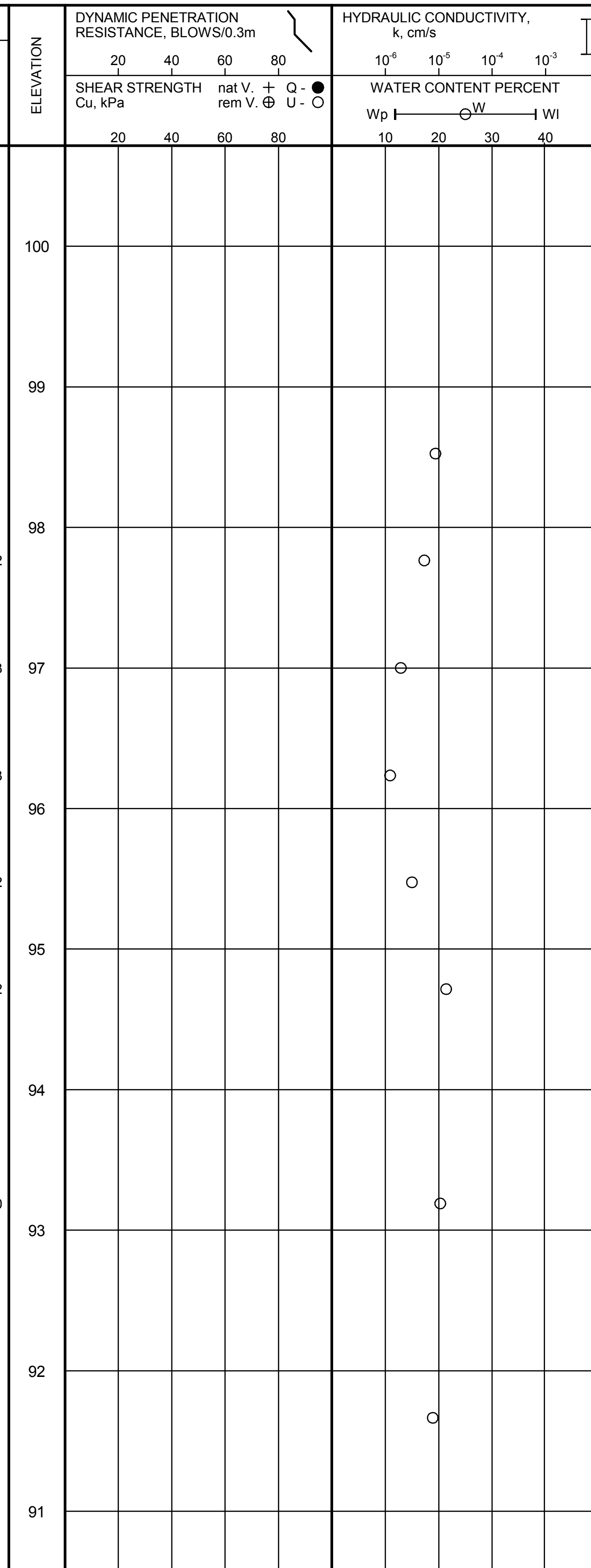
DATUM: LOCAL

HAMMER TYPE: Auto Hammer

BORING METHOD: POWER AUGER, 210mm ID HOLLOW STEM

SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)
CONCRETE SURFACE	99.52 / 0.00
CONCRETE PAVERS	0.06
FILL, gravelly sand; grey	99.06 / 0.46
(SW) **SAND**, fine to medium, trace to some gravel; brown; very loose to compact	97.39 / 2.13
(SM) **SILTY SAND**, fine to medium, with gravel; brown; dense to very dense	93.94 / 5.58
(SW-SM) **SAND**, fine, some gravel, some silt; brown to grey; compact	90.83 / 8.69
(SM) **SILTY SAND**, trace gravel; grey; loose	

SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	SS	2	
2	SS	22	
3	SS	33	
4	SS	33	
5	SS	52	MH
6	SS	32	
7	SS	30	
8	SS	21	



INSTALLATION AND GROUNDWATER OBSERVATIONS:

May 31/17

Water level measured at about elev. 98.83m upon completion of drilling on May 31, 2017

--- CONTINUED NEXT PAGE ---

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: NC

CHECKED: LV

PROJECT: 1776745

RECORD OF BOREHOLE BH-101

LOCATION: REFER TO LOCATION PLAN

BORING DATE: May 31, 2017
DRILLING CONTRACTOR: Henderson Drilling Inc.

DATUM: LOCAL

HAMMER TYPE: Auto Hammer

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	ELEVATION	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 (10 20 30 40)	ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS
		--- *CONTINUED FROM PREVIOUS PAGE* ---										
9–11.28	POWER AUGER 210mm ID HOLLOW STEM	(SM) **SILTY SAND**, trace gravel; grey; loose		88.24 / 11.28	9	SS	9	90				
					10	SS	8	89				
11.28–15.70		(CI) **SILTY CLAY**, some sand, trace gravel; grey, **TILL**; stiff to firm		83.82 / 15.70	11	SS	11	87				
					12	SS	7	86	>96.5			
					13	SS	5	84				
		END OF BOREHOLE										

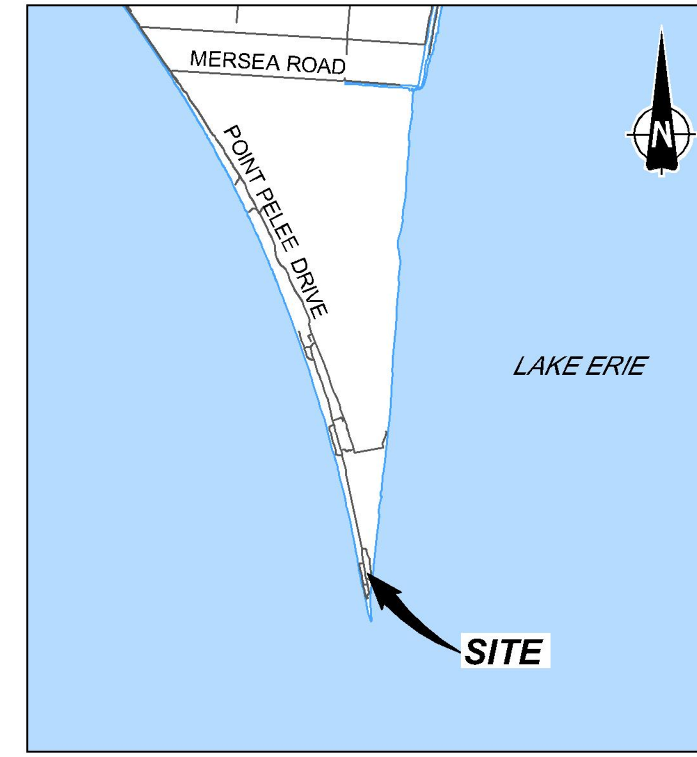
Depth scale: 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19

Elevation scale: 90, 89, 88, 87, 86, 85, 84, 83

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: NC
CHECKED: LV



KEY PLAN

LEGEND

BOREHOLE

REFERENCE

DRAWING BASED ON 2015 AERIAL IMAGERY FROM THE COUNTY OF ESSEX INTERACTIVE WEB MAPPING SITE, BY PERMISSION;
AND CANMAP STREETFILES V2008.4.

NOTES

THIS DRAWING IS SCHEMATIC ONLY AND IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

ALL LOCATIONS ARE APPROXIMATE.

PROJECT

GEOTECHNICAL EXPLORATION
POINT PELEE OBSERVATION TOWER
POINT PELEE, ONTARIO

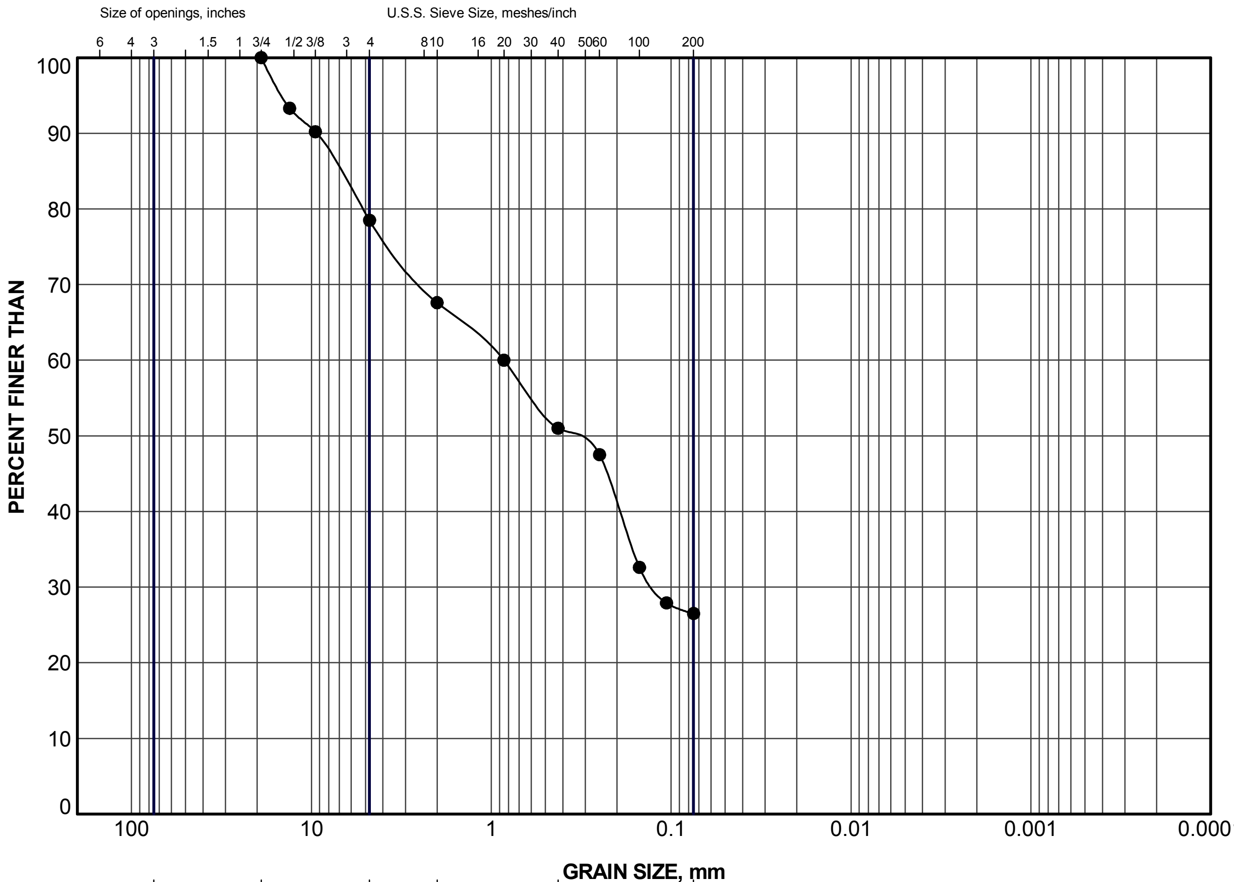
TITLE

LOCATION PLAN

Golder Associates

PROJECT No.	1776745	FILE No.	1776745-R01001
		SCALE	AS SHOWN
CADD	ZJB/LMK	Oct. 9/17	REV.
CHECK			FIGURE 1

Drawing file: 1776745-R01001.dwg Oct 10, 2017 – 10:31am



Cobble Size	GRAVEL SIZE		SAND SIZE			SILT AND CLAY
	coarse	fine	coarse	medium	fine	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-101	5	95.5

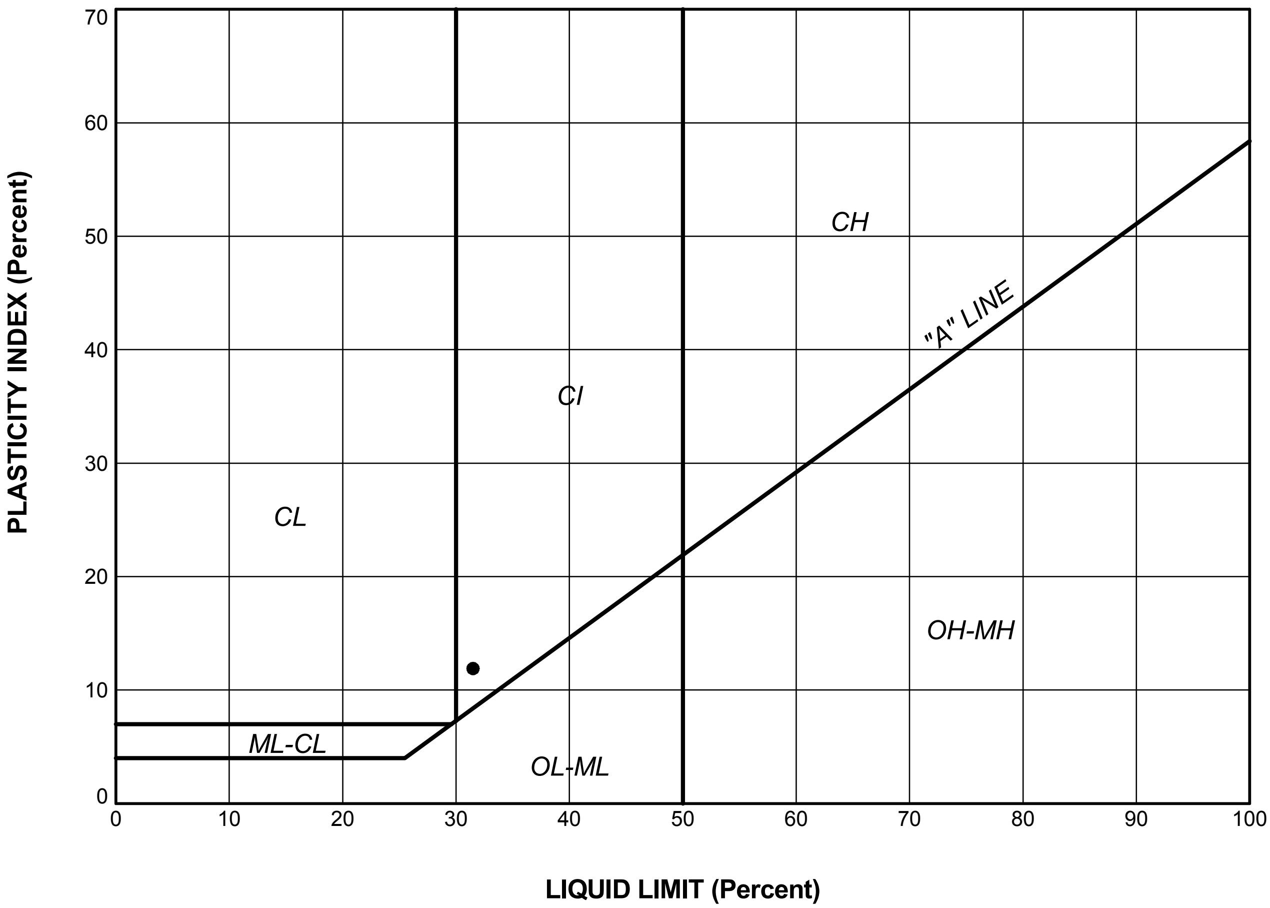
PROJECT
GEOTECHICAL EXPLORATION
POINT PELEE OBSERVATION TOWER
POINT PELEE, ONTARIO

TITLE
GRAIN SIZE DISTRIBUTION
gravelly SAND

Golder Associates

PROJECT No.	1776745	FILE No.	1776745-R01002
		SCALE N/A	REV.
DRAWN ZJB	Jun 09/17	FIGURE 2	
CHECK			

LDN_GSD GLDR_LDN.GDT 09/06/17



SOIL TYPE	**PLASTICITY**
C = Clay	L = Low
M = Silt	I = Intermediate
O = Organic	H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BH-101	12	31.5	19.6	11.9

PROJECT
GEOTECHICAL EXPLORATION
POINT PELEE OBSERVATION TOWER
POINT PELEE, ONTARIO

TITLE
PLASTICITY CHART

Golder Associates

PROJECT No.	1776745	FILE No.	1776745-R01003
		SCALE N/A	REV.
DRAWN ZJB	Jun 09/17	**FIGURE 3**	
CHECK			

LDN_PI GLDR_LON.GDT 09/06/17

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