



May 19, 2017

## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

### Simcoe Street Underpass Structure Site No. 22-176 Highway 401 Improvements from Brock Road to Courtice Road Regional Municipality of Durham W.O. 10-20011

**Submitted to:**

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REPORT



**GEOCRES No.: 30M14-451**

**Report Number: 11-1184-0143-9**

**Distribution:**

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**PRELIMINARY FOUNDATION REPORT  
SIMCOE STREET UNDERPASS, W.O. 10-20011**

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# **PART A**

**PRELIMINARY FOUNDATION INVESTIGATION REPORT  
SIMCOE STREET UNDERPASS  
STRUCTURE SITE NO. 22-177  
HIGHWAY 401 IMPROVEMENTS FROM BROCK ROAD TO COURTICE ROAD  
REGIONAL MUNICIPALITY OF DURHAM  
G.W.P. 10-2011**



## **1.0 INTRODUCTION**

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the future improvements and widening of Highway 401 from Brock Road to Courtice Road in the Regional Municipality of Durham, Ontario. This report addresses the proposed replacement of the existing Simcoe Street underpass.

The terms of reference for the preliminary foundation engineering services are outlined in MTO's Request for Proposals (RFP) for Assignment No. 2010-E-0062, dated June 2011. The scope of work for the preliminary foundation engineering services is presented in Section 5.8 of URS's *Technical Proposal* for this assignment, as well as Golder's Scope Change for Foundations Engineering Services letter dated December 8, 2014.

## **2.0 SITE DESCRIPTION**

The Simcoe Street underpass is located in the City of Oshawa, in the Regional Municipality of Durham. The existing Simcoe Street underpass is a two-span structure supported on spread footings with a span of about 28.6 m.

The natural ground surface at this site is at approximately Elevation 100 m to 102 m. Highway 401 has been constructed in a cut with its grade at approximately Elevation 96.5 m in the vicinity of the underpass. The grade of Simcoe Street varies from about Elevation 101.5 m to 102.5 m, rising northward, at the structure site. A commercial business is located in the northeast quadrant of the structure site, and the Oshawa Visitor Information Centre is located in the southeast quadrant. Greenspace areas are present in the northwest and southwest quadrants of the structure site.

## **3.0 INVESTIGATION PROCEDURES**

The field investigation was carried out in March 2015, at which time two boreholes, designated as Boreholes S1 and S2, were advanced to a depth of approximately 6.8 m and 15.7 m below the existing ground surface, respectively, at the structure site.

The boreholes were advanced using a CME-55 truck-mounted drill rig supplied and operated by DBW Drilling Ltd. of North York, Ontario. The boreholes were advanced using 150 mm diameter solid stem augers, with soil samples obtained at approximately 0.75 m and 1.5 m intervals of depth, using 50 mm outside diameter split-spoon samplers driven by an automatic hammer, in accordance with the Standard Penetration Test (SPT) procedure. (ASTM D1586-11 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of the Soil).

The groundwater conditions were observed in the open borehole during and immediately following the drilling operations. A standpipe piezometer was installed in Borehole S2, consisting of a 50 mm diameter slotted screen installed within a sand filter pack at a selected depth within the borehole; the details of this piezometer installation are shown on the borehole record in Appendix B. Borehole S1 was backfilled with bentonite on completion, in accordance with Ontario Regulation 903 (as amended).

The field work was observed by members of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations,



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logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Index and classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples. The results of the geotechnical laboratory testing are included in Appendix B.

The borehole locations, including MTM NAD 83 northing and easting coordinates and the ground surface elevations referenced to geodetic datum, are summarized in the following table, presented on the Record of Borehole sheets in Appendix A and are summarized on Drawing 1.

Borehole Number	UTM NAD83 Northing (m)	UTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
S1	4,860,398.8	356,528.6	101.3	6.8
S2	4,860,328.0	356,527.1	100.4	15.7

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

This section of Highway 401 is located within the Iroquois Plain physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984) and *Urban Geology of Canadian Cities* (Brennand, 1998). The Iroquois Plain extends around the western shores of Lake Ontario. The Plain is comprised of the flat to undulating lakebed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession.

The surficial soils in this area of the Iroquois Plain are typically comprised of glaciolacustrine clays, silts and sands to gravelly sands, which are underlain by an extensive till deposit that is mapped in this area as the Bowmanville Till. More recent alluvial deposits of gravel, sand, silt and/or clay are present in the creek valleys.

### 4.2 Subsurface Conditions

Boreholes S1 and S2 were advanced in the vicinity of the proposed north and south abutments, respectively, at the Simcoe Street underpass site. The detailed subsurface soil and groundwater conditions encountered in the boreholes, and the results of in situ and laboratory testing, are presented on the borehole records contained in Appendix A. The results of geotechnical laboratory testing are also presented in Appendix B.

The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. The interpreted stratigraphic profile along the structure, shown on Drawing 1, is a simplification of the subsurface conditions. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

At the site, a thin layer of topsoil and fill is underlain by a deposit of stiff to very stiff silty clay to clay, which is relatively thin at the north abutment, and which becomes thicker at the south abutment. The silty clay to clay is underlain by an upper till deposit that varies in composition from silt and sand to clayey silt. This upper till is underlain by a deposit of dense to very dense silt and sand to sand and gravel, which is in turn underlain by a



lower till deposit that varies in composition from silty sand to clayey silt. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

#### **4.2.1 Topsoil**

Approximately 400 mm to 500 mm of topsoil was encountered immediately below the existing ground surface in Boreholes S1 and S2, respectively.

#### **4.2.2 Fill**

An approximately 0.9 m thick layer of fill material was encountered underlying the topsoil in Borehole S2, in the vicinity of the south abutment. The surface of the fill was encountered at a depth of about 0.5 m, corresponding to Elevation 99.9 m. The fill consists of silty clay containing trace to some sand and trace organics. A hydrocarbon odour was noted in this layer.

The Standard Penetration Test (SPT) "N"-value measured within the fill was 12 blows per 0.3 m of penetration, suggesting a stiff consistency.

#### **4.2.3 Silty Clay to Clay**

A 0.9 m thick deposit of silty clay was encountered below the topsoil in Borehole S1 near the north abutment; its surface was encountered at a depth of about 0.4 m (Elevation 100.9 m). In Borehole S2 near the south abutment, the silty clay to clay deposit thickens to about 4.7 m, with its surface encountered below the topsoil and fill at a depth of about 1.4 m (Elevation 99.0 m). The base of the silty clay to clay deposit varies from about Elevation 99.8 m in Borehole S1 near the north abutment, to about Elevation 94.3 m in Borehole S2 near the south abutment.

The deposit consists of brown silty clay to clay containing trace sand and gravel, as well as trace organics and rootlets. The result of a grain size distribution test completed on one selected sample of the deposit is presented on Figure B1 in Appendix B. Atterberg limits testing was conducted on two selected samples of the deposit and measured plastic limit of 16 and 19 per cent, liquid limits of 37 and 52 per cent, and corresponding plasticity indices of 21 and 33 per cent. The test results, which are plotted on a plasticity chart on Figure B2 in Appendix B, confirm that the deposit consists of silty clay to clay of intermediate to high plasticity. The natural water content measured on two selected samples of the deposit are 23 and 36 per cent.

The measured SPT "N"-values within the silty clay to clay deposit range from 10 to 21 blows per 0.3 m of penetration, suggesting a stiff to very stiff consistency.

#### **4.2.4 Upper Silt and Sand Till to Clayey Silt Till**

An upper till deposit was encountered below the silty clay to clay deposit. In Borehole S1 near the north abutment, a 4.0 m thick deposit of silt and sand till was encountered underlying the silty clay at a depth of 1.5 m (Elevation 99.8 m). In Borehole S2 near the south abutment, a 0.9 m thick deposit of clayey silt till was encountered at a depth of about 6.1 m (Elevation 94.3 m).

The till deposit varies from silt and sand containing trace to some clay and trace gravel, to clayey silt containing some sand and trace gravel. The results of a grain size distribution test completed on one sample of the silt and sand till deposit is presented on Figure B3 in Appendix B. The natural water content measured on two selected samples of the silt and sand till were 9 per cent and 10 per cent.



The measured SPT “N”-values within the silt and sand till ranged from 24 blows per 0.3 m of penetration to 100 blows for 0.13 m of penetration, indicating a compact to very dense relative density. The measured SPT “N”-value within the clayey silt till was 38 blows per 0.3 m of penetration, suggesting a hard consistency.

#### **4.2.5 Silt and Sand to Sand and Gravel**

In Borehole S1 near the north abutment, a 1.1 m thick deposit of sand and gravel was encountered underlying the upper till at a depth of about 5.5 m, extending from Elevation 95.8 to 94.7 m. In Borehole S2 near the south abutment, the upper till is underlain by a 1.5 m thick layer of silt and sand and a 3.7 m thick layer of sand and gravel, extending from a depth of 7.0 m to 12.2 m (Elevation 93.4 m to 88.2 m).

The silt and sand layer contains trace to some gravel, and the sand and gravel layer contains trace to some silt and trace clay. The results of grain size distribution tests completed on three sample of the deposit are presented on Figure B4 in Appendix B. Natural water contents of 3 and 6 per cent were measured on two selected samples of the sand and gravel, and a natural water content of 17 per cent was measured on one selected sample of the silt and sand deposit.

The measured SPT “N”-values within the sand and gravel portion of the deposit are 90 to greater than 100 blows for 0.3 m of penetration, indicating a very dense relative density. One SPT “N”-value of 32 blows per 0.3 m of penetration was measured in the silt and sand, indicating a dense relative density.

#### **4.2.6 Lower Silty Sand to Clayey Silt Till**

A lower till deposit was encountered below the silt and sand to sand and gravel deposit, at a depth of about 6.6 m (Elevation 94.7 m) in Borehole S1 near the north abutment, and at a depth of about 12.2 m (Elevation 88.2 m) in Borehole S2 near the south abutment. The boreholes terminated in the lower till, penetrating it for a thickness of 0.2 m and 3.5 m in Boreholes S1 and S2, respectively.

The lower till deposit consists of silty sand containing trace gravel and trace clay in Borehole S1 near the north abutment, varying to clayey silt containing some sand and trace to some gravel in Borehole S2 near the south abutment. Atterberg limits testing was conducted on one sample of the clayey silt till and measured a plastic limit of 9 per cent, a liquid limit of 19 per cent and a corresponding plasticity index of 10 per cent. The test result, which is plotted on a plasticity chart on Figure B8 in Appendix B, confirms that the deposit consists of clayey silt of low plasticity. The natural water content measured on this same sample was 9 per cent, near the plastic limit of the material. A natural water content of 4 per cent was measured on one selected sample of the silty sand till.

The measured SPT “N”-values within the lower till deposit range from 89 to greater than 100 blows per 0.3 m of penetration, indicating a very dense relative density/hard consistency.

### **4.3 Groundwater Conditions**

The groundwater levels in the open boreholes were measured upon completion of drilling operations. Details of the measured groundwater levels are shown on the borehole records in Appendix A. The groundwater levels recorded in the open boreholes and piezometers are summarized below:





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Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
S1	101.3	Dry	-	March 9, 2015	Open Borehole
S2	100.4	7.6	92.8	March 11, 2015	Open Borehole
		6.8	93.6	June 7, 2016	Piezometer

The groundwater level will be subject to seasonal fluctuations and precipitation events; therefore the water levels should be expected to be higher during the spring season or following periods of heavy precipitation and snow melt.

### 5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Ms. Andrea Walsh and was reviewed by Ms. Nikol Kochmanová, P.Eng., a geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., a Designated MTO Foundations Contact, conducted an independent review of this report.

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**PRELIMINARY FOUNDATION REPORT  
SIMCOE STREET UNDERPASS, W.O. 10-20011**

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## **PART B**

**PRELIMINARY FOUNDATION DESIGN REPORT  
SIMCOE UNDERPASS  
STRUCTURE SITE NO. 22-176  
HIGHWAY 401 IMPROVEMENTS FROM BROCK ROAD TO COURTICE ROAD  
REGIONAL MUNICIPALITY OF DURHAM  
G.W.P. 10-2011**



## **6.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS**

### **6.1 General**

This section of the report provides preliminary foundation recommendations in support of the proposed replacement of the existing Highway 401-Simcoe Street underpass (MTO Structure Site 22-176). These preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the preliminary subsurface investigation at this site. This Preliminary Foundation Design Report, including the interpretations and recommendations contained herein, are intended for the use of MTO to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. This Preliminary Foundation Design Report shall not be used or relied upon for any other purpose or by any other parties, including contractors. Further investigation and design will be required during the detailed design stage.

Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the contract documents. Contractors must make their own interpretation of the factual information provided in the Preliminary Foundation Investigation Report, as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

### **6.2 Foundation Options**

The existing two-span rigid frame underpass structure was constructed in 1941. Based on the design drawings (Contract No. 39-01: "Simcoe Street Underpass, Toronto to Oshawa Dual Highway, Division No. 6, Co. of Ontario, Twp. Whitby", prepared by the Department of Highway Ontario, Bridge Office, dated 1940), the existing structure is supported on spread footings founded at approximately Elevation 93.9 m. Based on visual observations, the existing abutments are considered to have performed satisfactorily.

It is understood that as part of the future improvements and widening of Highway 401 from Brock Road to Courtice Road in the Regional Municipality of Durham, the existing Simcoe Street underpass will be replaced. Based on the preliminary General Arrangement drawing provided by AECOM, the proposed replacement structure is to consist of a three-span structure with a total span length of approximately 87 m. Highway 401 will be widened from the existing six lanes to ten lanes, plus an additional two lanes to accommodate the new eastbound off-ramp, and one lane for the new westbound off-ramp. Additionally, the Highway 401 grade is proposed to be lowered to approximately Elevation 95.5 m, while the Simcoe Street grade will be maintained.

Both shallow and deep foundation options have been considered for support of the abutments and piers for the proposed Simcoe Street underpass replacement structure. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 1 following the text of this report.

- **Strip footings founded on the upper till deposit:** Shallow footings are feasible at this site provided that they are extended through the silty clay to clay deposit to found on the upper till deposit at the abutments, although this foundation type will preclude the use of integral abutments. This option would require excavation to a depth of about 1.2 m below the proposed Highway 401 grade at the pier locations, and about 7.5 m of excavation at the abutment locations. Depending on the construction staging approach, temporary



protection systems may be required along Highway 401 and along Simcoe Street to facilitate the construction of the new abutments and the removal of the existing structure. Temporary protection systems will also be required on Highway 401 for the pier foundation construction. The proposed founding level will be near the groundwater level at the site, and groundwater control is expected to be required to enable shallow foundations to be constructed in “dry” conditions.

- **Footings “perched” on a compacted granular pad in the approach embankment:** Due to the presence of stiff silty clay to clay soils in the vicinity of the proposed abutments, the geotechnical resistances are inadequate for support of perched footings without significant subexcavation (particularly in the vicinity of the south abutment). In addition, this foundation type would require a longer structure span, which would not work with the local road/retaining wall configuration adjacent to the new north abutment location. Therefore, this alternative is not addressed further in this report.
- **Driven steel H-piles or pipe piles founded on the very dense upper till / sand and gravel deposits:** Driven steel H-piles or steel pipe (tube) piles are feasible for support of the abutments, and would permit design of conventional abutments, semi-integral abutments (for tube piles) or integral abutments (for H-piles). The abutments may be constructed with a pile cap perched above the Highway 401 grade in a false abutment configuration, with reinforced soil system (RSS) walls. Driven steel piles are feasible and but not recommended at the pier locations, due to the shallow depth to “100-blow” soil, which would require pre-augering prior to pile placement. Pile driving shoes are recommended to protect the pile tips from damage during driving into the very dense/hard deposits.
- **Caissons founded on the very dense upper till / sand and gravel deposits:** Caissons are considered feasible for the support of the abutments; however this option would preclude integral abutment design. A perched pile cap would minimize excavation and groundwater control requirements at the new abutments; however, it is understood that a closed abutment configuration is required for this structure, so this advantage likely cannot be realized for this site. Caissons are also feasible at the piers, and may be geometrically desirable as they may be constructed in a smaller footprint than a strip footing or pile cap. This option will be more expensive than either shallow foundations or pile foundations, although fewer caisson elements would be required in comparison to the number of steel piles that would be required. If caissons are adopted for support of the abutments or piers, they would extend into and through water-bearing cohesionless soil deposits; temporary liners would be required during construction to control potential ground losses and/or disturbance at the caisson base.

Based on the above considerations, both shallow and deep foundation options are considered feasible for the support of the new abutments. Typically at the abutments, pile foundations would be preferred with a perched pile cap in a false abutment configuration, to permit integral abutments and minimize excavation and groundwater control requirements. However, for this site, a closed structure type is preferred due to the proximity of the local road on the north side of the highway, and therefore shallow foundations are preferred given the shallow depth to “100-blow” soil below the Highway 401 grade and abutment founding level. At the piers, shallow foundations are preferred from a geotechnical/foundations perspective due to the presence of a suitable bearing stratum at shallow depth. However, short caisson foundations may also be adopted at the piers; although more expensive, they may occupy a smaller footprint than other foundation types.



## **6.3 Shallow Foundations**

### **6.3.1 Founding Elevation and Frost Protection Requirements**

For support of the abutments, piers and associated wing walls for the proposed new underpass, strip footings should be founded on the very dense silty sand till and hard clayey silt till deposits. Strip or spread footings should be founded at a minimum depth of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario*). If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

The Highway 401 grade is proposed to be lowered to approximately Elevation 95.5 m below the underpass structure; as such the maximum (highest) founding elevation recommended for the preliminary design of the footings is Elevation 94.2 m.

Due to the variability encountered between Boreholes S1 and S2 at the north and south abutments, the subsurface conditions at the pier locations will need to be investigated and confirmed during the detail design stage. Based on interpolation between these boreholes, it is anticipated that dense to very dense soils will be encountered immediately below the Highway 401 grade; however, there is some risk that the surface of the till deposit will be deeper, and that stiff silty clay to clay soils may be present in the area.

### **6.3.2 Geotechnical Axial Resistance and Reaction**

The following factored geotechnical axial resistances at Ultimate Limit States (ULS) and geotechnical resistance at Serviceability Limit States (SLS, for 25 mm of settlement) may be used for preliminary design of strip footing founded on the properly prepared silty sand till and hard clayey silt till deposits. These values are based on a 3 m wide footing.

<b>Foundation Alternative</b>	<b>Factored Geotechnical Axial Resistance at ULS (kPa)</b>	<b>Geotechnical Reaction at SLS for 25 mm of Settlement (kPa)</b>
Footing on properly prepared very dense silty sand till or hard clayey silt till	500	350

The geotechnical resistances provided herein are given for loads will that be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with the Table 10.2 in *CFEM* (2006).

## **6.4 Driven Steel H-Pile or Steel Pipe (Tube) Pile Foundations**

### **6.4.1 Founding Elevation**

If geometrically suitable, the abutments and associated retaining walls for the replacement structure may be supported on steel piles driven to found within the very dense (“100-blow”) silty sand till or sand and gravel at the north abutment, and the very dense sand and gravel/hard lower clayey silt till soils at the south abutment. Typically, it would be recommended that the abutment pile caps be perched above the Highway 401 grade to minimize



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excavation and groundwater control requirements, and to start pile driving at a sufficient height above the surface of the “100-blow” soil to minimize requirements to pre-auger to place piles. However, at this site, it is understood that the Highway 401 cut must be widened using retaining walls at the north abutment (i.e., closed abutment), due to the proximity to the local road along the north side of Highway 401. This excavation requirement offsets some of the benefits of maintaining a perched pile cap with pile-supported abutments.

For preliminary design purposes, if integral abutments were to be adopted for the design of the structure replacement, it has been assumed that the pile caps would be “perched” within the embankment. The following pile tip elevations are recommended for preliminary design, assuming about 2 m to 3 m of penetration into “100-blow” soil:

Foundation Element	Estimated Pile Cap Elevation (m)	Approximate Surface Elevation of “100-Blow” Soil (m)	Estimated Design Tip Elevation (m)	Approximate Pile Length (m)	Founding Soil at Tip Elevation
North Abutment	100	96.5	94.5	5.5	Silty sand till
South Abutment	98	91	88	10	Sand and gravel

Based on the above elevations, the proposed piles are estimated to be approximately 5.5 m to 10 m long. Given the relatively shallow depth to the very dense (“100-blow”) silty sand till at the north abutment and very dense/hard soils at the soil abutment, pre-augering may be required, with the piles subsequently installed within the CSP and pre-augered hole using centralizers.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the glacially-derived soils at this site, as well as the potential for damage to the pile tips during seating on the bedrock. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of experiencing refusal on boulders or being deflected away from the vertical/battered orientation during installation due to their larger end area. Piles should be reinforced at the tip with driving shoes and/or flange plates in accordance with OPSD 3000.100 (*Steel H-Pile Driving Shoe*) or OPSD 3001.100 (*Steel Tube Pile Driving Shoe*) Type II, as appropriate, to reduce the potential for damage to the piles during driving. In very dense strata containing cobbles and/or boulders, as encountered at this site, driving shoes (such as Titus Standard ‘H’ Bearing Pile Points) are preferred over flange plates.

### 6.4.2 Geotechnical Axial Resistance/Reaction

For HP 310x110 piles driven to the design tip elevations given above, the factored axial geotechnical resistance at ULS may be taken as 1,400 kN. The axial geotechnical resistance at SLS may be taken as 1,200 kN for 25 mm of settlement. The same axial resistances may be used in the preliminary design of closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.). These preliminary geotechnical resistances/reactions will have to be re-evaluated and modified, as necessary, during detail design in consideration of additional subsurface investigation at the foundation elements.



Pile installation should be in accordance with OPSS 903 (*Deep Foundations*). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO's Standard Drawing SS103-11, *Pile Driving Control*) during the final stages of driving to verify that the required ultimate capacity has been achieved.

## **6.5 Caisson Foundations**

### **6.5.1 Founding Elevation**

Caissons founded within the very dense silty sand till and hard clayey silt till deposits may be considered for support of the abutments and piers for the proposed replacement structure; however, as very dense/hard ("100-blow") soils are present at a shallow depth below the Highway 401 cut grade, this foundation option is not considered cost-effective. If adopted, the following caisson founding elevations may be used for preliminary design purposes, assuming about 2 m to 3 m of penetration into "100-blow" soil:

<b>Foundation Element</b>	<b>Estimated Pile Cap Elevation (m)</b>	<b>Approximate Surface Elevation of "100-Blow" Soil (m)</b>	<b>Estimated Design Base Elevation (m)</b>	<b>Founding Soil at Tip Elevation</b>
North Abutment	100	96.5	94.5	Silty sand till
South Abutment	98	91	88	Sand and gravel

Due to the variability in the elevation of the 100-blow soil layer between the boreholes advanced north and south of the highway, it is difficult to establish design caisson base elevations for the piers. Further investigation will be required during detail design if caisson foundations are adopted for support of the piers.

If caisson foundations are adopted, a temporary liner and a head of water/drilling slurry will be required to support the overburden soils during construction and balance groundwater pressures to minimize disturbance to the side walls and to control base disturbance/basal heave. In addition, placement of concrete by tremie methods would be required.

### **6.5.2 Geotechnical Axial Resistance/Reaction**

The following factored geotechnical axial resistance at ULS and geotechnical resistance at SLS (for 25 mm of settlement) may be used for preliminary design of caisson foundations:

<b>Caisson Diameter (m)</b>	<b>Factored Geotechnical Axial Resistance at ULS (kN)</b>	<b>Geotechnical Reaction at SLS for 25 mm of Settlement (kN)</b>
1.2	4,500	3,500
1.5	6,500	5,500





The preliminary geotechnical resistances/reactions provided above will need to be re-evaluated and modified, as necessary, during detail design in consideration of additional subsurface investigation at the foundation elements. Lower geotechnical resistance values may apply at the piers, depending on the depth to the 100-blow soil layer and the groundwater conditions within the highway cut.

## **6.6 Retained Soil System (RSS) Walls**

### **6.6.1 Founding Elevations**

If perched pile caps are used in a false abutment configuration, and for retaining walls adjacent to the abutments and wing walls at this site, retained soil system (RSS) walls are a suitable and feasible alternative to conventional concrete retaining walls supported on shallow foundations. Such walls would be constructed parallel to the Highway 401 cut. Retaining walls beyond the structure replacement are addressed in a separate Foundation Investigation and Design Report for all retaining walls associated with this assignment.

The front facing panels and the reinforced soil mass of the RSS wall should be founded below any existing topsoil or unsuitable fill soils. In addition, it is recommended that the RSS wall be founded below the stiff portions of the silty clay to clay deposit that was encountered in the vicinity of the south abutment. This would require founding the reinforced soil mass and the front facing footing below approximately Elevation 94.0 m for RSS walls adjacent to the south abutment; alternatively, the stiff silty clay to clay deposit within the footprint of the reinforced soil mass could be subexcavated down to this elevation prior to construction of the RSS wall.

Typically, the front facing panels are supported on a footing and/or granular levelling pad at a shallow depth below the ground surface in front of the wall; it is recommended that the facing panels be founded at a minimum depth of 0.5 m below the lowest surrounding grade, in accordance with MTO's *RSS Design Guidelines*. The levelling pad should consist of a minimum thickness of 0.3 m of compacted Granular A, which should extend at least 0.5 m beyond the outside edge of both sides of the facing footing, then outward/downward at 1H:1V.

### **6.6.2 Geotechnical Resistance/Reaction**

For the RSS facing panels founded on compacted granular fill as described above, preliminary design may be completed based on a factored geotechnical resistance at ULS of 150 kPa, and a geotechnical reaction at SLS (for 25 mm of settlement) of 100 kPa.

Assuming that the RSS wall (up to approximately 6 m high) acts as a unit and uses the full width of the reinforced soil mass (which can be taken as approximately 0.8 times the wall height for preliminary design), a factored geotechnical resistance at ULS of 600 kPa and a geotechnical reaction at SLS of 400 kPa (for 25 mm of settlement) may be used for preliminary design. The preliminary geotechnical resistance/reaction values should be reviewed and revised during detail design, after the RSS wall configuration and any "step" elevations are confirmed, taking into account any additional subsurface information at that time.

### **6.6.3 Global Stability of RSS Walls**

Preliminary slope stability analyses have been performed for a conceptual RSS wall adjacent to the south abutment, using the commercially available program *Slide 6.0*, produced by Rocscience Inc., to check that a minimum factor of safety of 1.5 is achieved for the proposed maximum retaining wall heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed walls on this site, considering the design requirements and the available field and laboratory testing data.





## PRELIMINARY FOUNDATION REPORT SIMCOE STREET UNDERPASS, W.O. 10-20011

The following parameters have been used in the analyses, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990):

Soil Deposit	Bulk Unit Weight (kN/m <sup>3</sup> )	Effective Friction Angle	Undrained Shear Strength (kPa)
Granular backfill in RSS wall	21	35°	-
Hard upper clayey silt till	21	32°	-
Dense silt and sand	21	32°	-
Very dense sand and gravel	21	35°	-
Hard clayey silt till	21	35°	-

The results of the static global stability analysis indicate that a minimum factor of safety of 1.5 is achieved for a conceptual RSS wall up to approximately 7 m in retained soil height, assuming level ground in front of and behind the wall. This preliminary assessment of the global stability of the retaining walls should be reviewed and confirmed as part of the detail design, if this type of wall is adopted; this assessment should include any sloping ground in front of or behind the wall.

It should be noted that the internal stability of a reinforced earth structure is to be assessed by the proprietary product designer to ensure the internal and external stability of the wall is adequate.

## 6.7 Construction Considerations

The following sections identify future construction considerations that may impact the future detail design, and for which provision may be required in the contract documents produced as part of detail design.

### 6.7.1 Open-Cut Excavation and Temporary Protection Systems

The construction of the new strip footings for the replacement structure will require excavations up to about 1.2 m below the existing Highway 401 grade at the pier locations. Widening of Highway 401 and construction of the abutments will require excavations of up to about 6 m to 7 m through the existing cut slopes. The existing fill and the stiff to very stiff silty clay to clay deposits are classified as Type 3 soils, while the native dense to very dense and very stiff to hard deposits are classified as Type 2 soils, according to the Occupational Health and Safety Act (OHSA). Where space permits, temporary open-cut excavations above the groundwater level should be made with side slopes no steeper than 1H:1V. All excavations must be carried out in accordance with Ontario Regulation 213 (Ontario Occupational Health and Safety Act for Construction Projects) (as amended).

It is anticipated that due to space constraints, temporary protection systems will be required along the existing Highway 401 eastbound and westbound lanes to facilitate the removal of the existing bridge foundations and walls, and excavation for the widened Highway 401 and new abutments. Temporary protection systems are also likely to be required for the shallow foundation excavations within Highway 401 at the pier locations. Temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539, provided any adjacent utilities can tolerate this magnitude of deformation.

The selection and design of the protection system will be the responsibility of the Contractor.



### **6.7.2 Groundwater Control**

The excavations for shallow foundations at the abutments and piers, together with excavations for the highway cut widening, will extend near but should remain above the groundwater level, which was measured in the standpipe piezometer to be at about Elevation 93.6 m. Some groundwater control (dewatering) may be required within the sand and gravel and/or silt and sand till deposits, depending on the seasonal variations in the water level at the time of construction; further assessment of such seasonal variation will be required as part of detailed design.

At this preliminary stage, it is anticipated that an active dewatering system (such as the use of a system of well points) could be required to lower the groundwater in the silt and sand till and sand and gravel soils to at least 0.5 m to 1 m below the foundation level during excavation and footing construction. An accurate prediction of the groundwater pumping volumes cannot be made at this stage. However, it is considered that it may be possible to maintain pumping volumes at less than 50 m<sup>3</sup>/day, but that groundwater pumping volumes could exceed 50 m<sup>3</sup>/day during initial drawdown stages and/or if multiple excavation areas are being dewatered at one time.

At this preliminary stage, it is anticipated that the zone of influence for any dewatering operations would be relatively localized at the structure site. Assuming the dewatering system is properly constructed and operated such that there is no loss of fine soil particles, the dewatering operations are not expected to cause excessive settlement in the compact to very dense/hard till or silt and sand to sand and gravel deposits at the site. However, the potential for settlement impacts on the existing or new structure foundations and any adjacent utilities should be re-assessed at the detailed design phase.

### **6.7.3 Subgrade Protection**

The native soils that will be exposed within the excavations at the foundation subgrade level will be susceptible to disturbance from construction traffic and/or precipitation and ponded water. To limit the effects of this disturbance, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the subgrade. The minimum thickness of the concrete working slab should be 100 mm and the concrete should have a minimum 28-day compressive strength of 20 MPa.

### **6.7.4 Obstructions**

The soils at this site are glacially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of deep foundations or protection systems. Further observation is recommended in future investigation as part of the detailed design assignment for this site, to further assess the presence of cobbles and boulders and permit the contractor to assess the impact on foundation construction and protection system installation.

### **6.7.5 Vibration Monitoring During Pile or Caisson Installation**

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition; lower thresholds are applicable for nearby residential and commercial facilities (between 25 mm/s and 50 mm/s). If pile driving is adopted at the abutments, or if caissons are adopted at the piers, then vibration monitoring is recommended adjacent to the abutment areas to demonstrate/confirm that vibration levels do not exceed the threshold levels.

## **6.8 Recommendations for Future Work during Detail Design**

During the detail design phase, additional investigation is recommended to confirm or assess the following:



## PRELIMINARY FOUNDATION REPORT SIMCOE STREET UNDERPASS, W.O. 10-20011

- the subsurface conditions and geotechnical/foundation recommendations at the pier locations;
- the subsurface conditions at greater depth below footings in the vicinity of the north abutment (as Borehole S1 terminated at Elevation 94.5 m, three samples into “100-blow” soil); and
- the groundwater level for more detailed assessment of groundwater control requirements during construction.

### 7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Ms. Nikol Kochmanová, P.Eng.. Ms. Lisa Coyne, P.Eng., a Designated MTO Foundations Contact, conducted an independent review of this report.

#### GOLDER ASSOCIATES LTD.



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Geotechnical Engineer



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Designated MTO Foundations Contact

AW/NK/LCC/sm

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## PRELIMINARY FOUNDATION REPORT SIMCOE STREET UNDERPASS, W.O. 10-20011

### REFERENCES

- Bowles, J.E., 1984. *Physical and Geotechnical Properties of Soils*, Second Edition, McGraw Hill Book Company, New York.
- Brennand, T.A. 1998. Urban Geology Note: Oshawa Ontario. In P.F. Karrow, and O. L. White (Eds.), Geological Association of Canada, Special Paper 42: Urban Geology of Canadian Cities, p. 353-364.
- Canadian Geotechnical Society, 1992. *Canadian Foundation Engineering Manual*, 3<sup>rd</sup> Edition. The Canadian Geotechnical Society, BiTech Published Ltd., British Columbia.
- Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual*, 4<sup>th</sup> Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.
- Canadian Standards Association (CSA), 2006. *Canadian Highway Bridge Design Code and Commentary on CAN/CSA S6 06*. CSA Special Publication, S6.1 06.
- Chapman, L.J., and Putnam, D.F., 1984. *The Physiography of Southern Ontario*, 3<sup>rd</sup> Edition. Ontario Geological Survey, Special Volume 2. Ontario Ministry of Natural Resources.
- Kulhawy, F.H. and Mayne, P.W., 1990. *Manual on Estimating Soil Properties for Foundation Design*. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.
- NAVFAC, 1982. *Design Manual DM 7.2: Soil Mechanics, Foundation and Earth Structures*. U.S. Navy. Alexandria, Virginia.

### ASTM International

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

### Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

### Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile, Driving Shoe
OPSD 3001.100	Foundation, Piles, Steel Tube Piles, Driving Shoe
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement

### Other

Ontario Regulation 213	Construction Projects (as amended)
Ontario Regulation 903	Wells (as amended)



## PRELIMINARY FOUNDATION REPORT SIMCOE STREET UNDERPASS, W.O. 10-20011

**TABLE 1 – COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES**

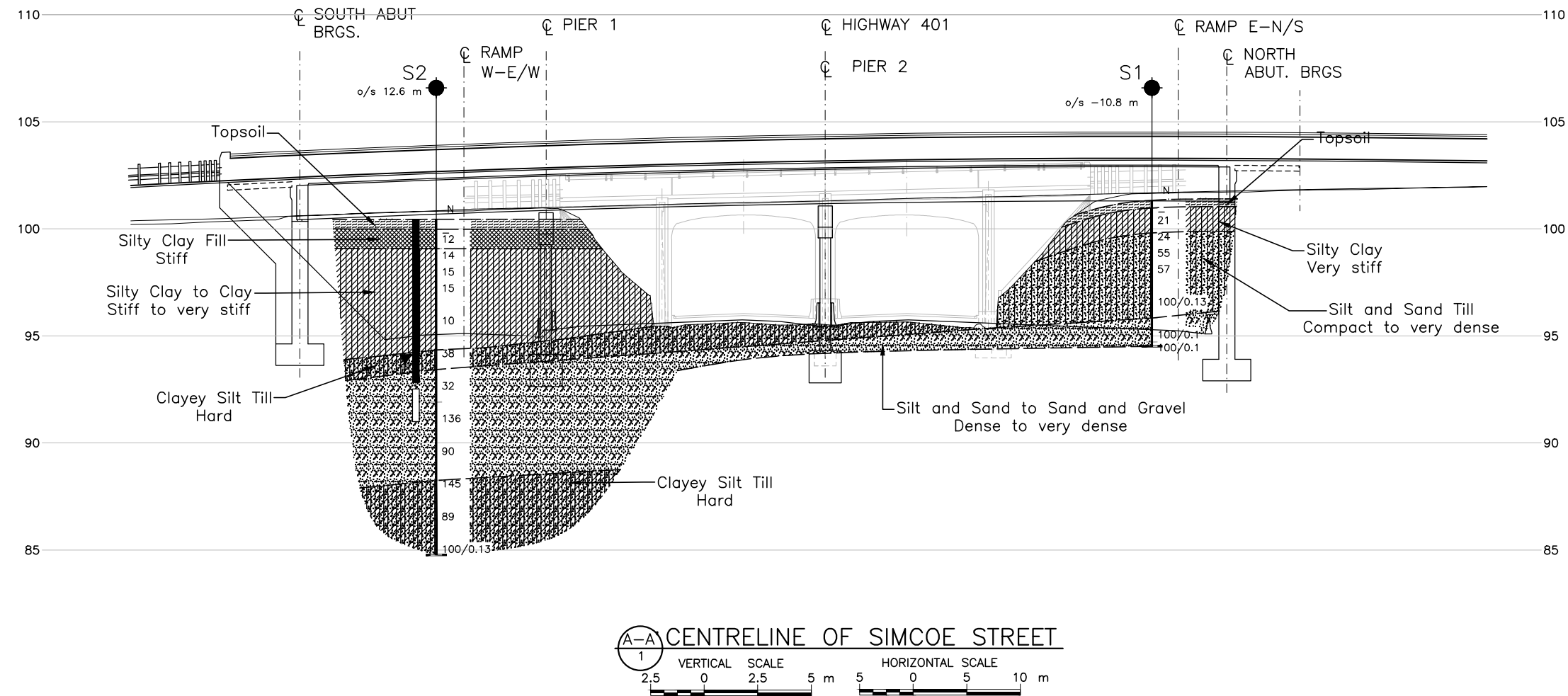
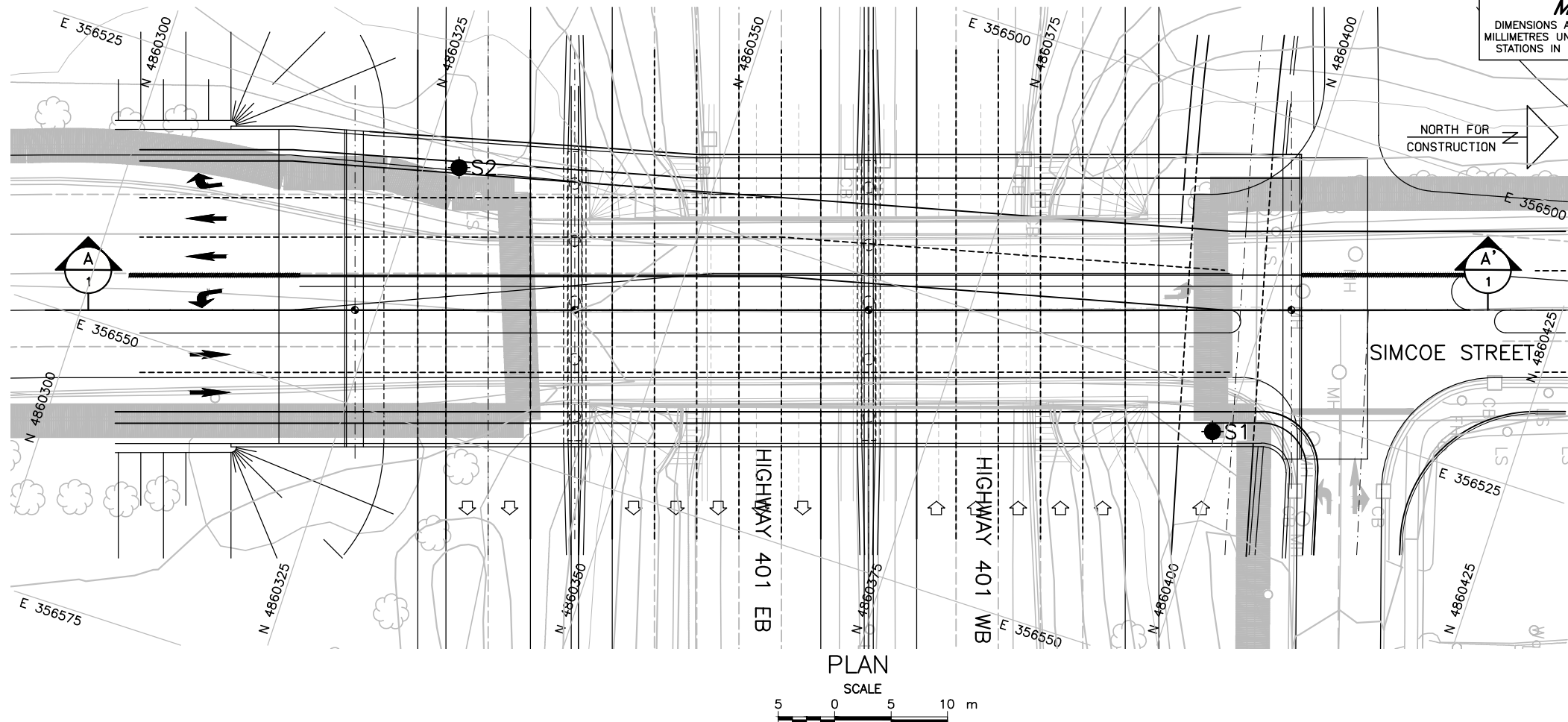
Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Strip footings	<ul style="list-style-type: none"> <li>Feasible for support of the new abutments and piers</li> </ul>	<ul style="list-style-type: none"> <li>Conventional excavation and construction techniques</li> <li>Very dense soils (with SPT “N” values greater than 100 blows per 0.3 m of penetration) present at shallow depth below Highway 401 cut grade, with good geotechnical resistance and settlement performance</li> </ul>	<ul style="list-style-type: none"> <li>Excavation will be required to extend 6 m to 7 m below Simcoe Street grade; however, this is required at any rate for Highway 401 cut widening</li> <li>Protection systems required for deep excavations at abutments, and at piers</li> </ul>	<ul style="list-style-type: none"> <li>Estimated cost is approximately \$600/m<sup>3</sup> for construction of shallow foundations</li> </ul>	<ul style="list-style-type: none"> <li>Some groundwater control may be required to maintain water level 0.5 m to 1 m below excavation/founding level</li> </ul>
Steel H-piles or pipe piles	<ul style="list-style-type: none"> <li>Feasible for support of abutments, particularly if abutment pile caps can be “perched” above Highway 401 grade in false abutment configuration</li> <li>Not recommended for the support of the piers due to shallow depth to 100-blow soil</li> </ul>	<ul style="list-style-type: none"> <li>Conventional construction methods</li> <li>Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation, protection system and groundwater control requirements</li> <li>Steel H-piles allow for integral abutment configuration, and pipe piles for semi-integral abutment configuration</li> </ul>	<ul style="list-style-type: none"> <li>Temporary excavation support required to facilitate removal of existing structure and widening of Highway 401</li> <li>Design geotechnical resistance may not be achieved if piles refuse on cobble and boulder layers</li> </ul>	<ul style="list-style-type: none"> <li>Estimated cost is approximately \$200/m length for pile installation and \$600/m<sup>3</sup> for pile cap construction</li> </ul>	<ul style="list-style-type: none"> <li>Minor potential for pile damage / deflection if cobbles and boulders are encountered during pile driving</li> <li>Slightly greater risk in this regard for pipe piles as compared with H-piles if boulders are encountered during pile driving</li> </ul>



## PRELIMINARY FOUNDATION REPORT SIMCOE STREET UNDERPASS, W.O. 10-20011

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Caissons	<ul style="list-style-type: none"><li>• Feasible but not recommended for support of abutments or piers, due to shallow depth to 100-blow soils and feasibility of shallow foundations at piers, and requirement for highway cut widening at abutments</li></ul>	<ul style="list-style-type: none"><li>• Abutment pile caps could be maintained higher than spread footings in an open structure configuration, potentially reducing depth of excavation, protection system and groundwater control requirements</li><li>• Higher capacity than for driven piles, so reduced number of deep foundation elements compared to piles</li></ul>	<ul style="list-style-type: none"><li>• Caissons would extend below the groundwater level at the site into water-bearing non-cohesive soils, with potential for loss of ground or base disturbance</li><li>• Temporary liners would be required, plus special measures such as use of drilling mud and tremie placement of concrete; likely not possible to inspect caisson base</li><li>• Precludes use of integral abutments</li></ul>	<ul style="list-style-type: none"><li>• Estimated cost is approximately \$1,000/m length for caisson installation and \$600/m<sup>3</sup> for pile cap construction; the cost may be higher to account for temporary liners</li></ul>	<ul style="list-style-type: none"><li>• Risk of loosening or disturbing founding soils at base of caissons</li></ul>



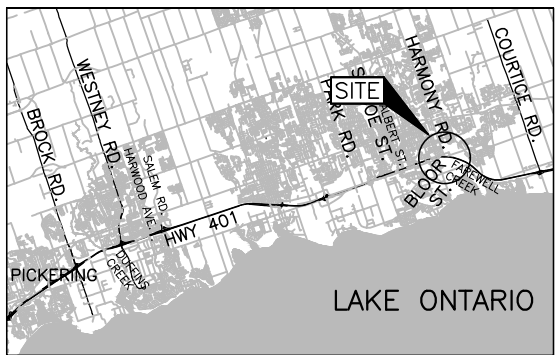


CONT No. WO No. 10-20011

SIMCOE STREET UNDERPASS  
HIGHWAY 401 IMPROVEMENTS  
BOREHOLE LOCATIONS AND  
SOIL STRATA



SHEET



KEY PLAN

SCALE

4 0 4 8 km

LEGEND

- Borehole - Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer, measured in June 2016

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
S1	101.3	4860398.8	356528.6
S2	100.4	4860328.0	356527.1

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Preliminary Design Report.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by URS, drawing file nos. X-Base.dwg, X-Property.dwg and Street Names.dwg, and the Proposed Design obtained from drawing file x-design\_130625.dwg, all dated July 05, 2013, received April 11, 2014. General Arrangement provided in digital format by AECOM, drawing file ACAD-01\_GA\_Simcoe St\_Underpass.dwg, received October 25, 2016.



NO.	DATE	BY	REVISION
Geocres No. 30M14-451			
HWY. 401	PROJECT NO. 11-1184-0143		DIST. CENTRAL
SUBM'D. MWK	CHKD. PKS	DATE: 4/5/2017	SITE: 22-176
DRAWN: JFC	CHKD. PKS	APPD. LCC	DWG. 1



# **APPENDIX A**

## **Borehole Records**





## LIST OF SYMBOLS

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

### I. GENERAL

$\pi$	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
FoS	factor of safety

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

<b>(a)</b>	<b>Index Properties</b>
$\rho(\gamma)$	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
$\gamma'$	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

### (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
$C_\alpha$	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
$\sigma'_p$	pre-consolidation stress
OCR	over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$

### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	coefficient of friction = $\tan \delta$
$c'$	effective cohesion
$c_u, s_u$	undrained shear strength ( $\phi = 0$ 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

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

Notes: 1  
2

$$\tau = c' + \sigma' \tan \phi'$$
$$\text{shear strength} = (\text{compressive strength})/2$$



## LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

### I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

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

#### Dynamic Cone Penetration Resistance; $N_d$ :

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

**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

#### Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ( $Q_t$ ), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

### III. SOIL DESCRIPTION

#### (a) Non-Cohesive (Cohesionless) Soils

Density Index	N
Relative Density	Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

#### (b) Cohesive Soils Consistency

	$c_u, s_u$	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

### IV. SOIL TESTS

w	water content
w <sub>p</sub>	plastic limit
w <sub>l</sub>	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
D <sub>R</sub>	relative density (specific gravity, $G_s$ )
DS	direct shear test
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 <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

**Note:** 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

### V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

+ 3, × 3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

PROJECT		11-1184-0143		<b>RECORD OF BOREHOLE No A2</b>		SHEET 1 OF 1		<b>METRIC</b>									
G.W.P.		10-20011		LOCATION		N 4860377.6 ; E 356679.4		ORIGINATED BY		TD							
DIST		Central HWY 401		BOREHOLE TYPE		150 mm O.D. Continuous Flight Solid Stem Augers		COMPILED BY		PKS							
DATUM		Geodetic		DATE		March 10, 2015		CHECKED BY		LCC							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
102.0	GROUND SURFACE																
0.0	TOPSOIL																
101.5																	
0.5	Silty clay, trace to some organics, trace gravel, trace sand (FILL) Stiff Dark to light brown Moist		1	AS	-												
100.6			2A	SS	15												
1.4	CLAYEY SILT, trace gravel, trace sand Hard Brown Moist		3A	SS	35												
100.2			3B	SS													
1.8	SILT and SAND, trace to some clay, trace gravel (TILL) Dense to very dense Brown to grey, oxidation staining Moist		4	SS	47												
			5	SS	135												
97.1	- becoming grey and wet below a depth of 4.6 m		6A	SS	84												
4.9	SAND, trace gravel, trace silt Very dense Brown Moist		6B	SS													
96.5																	
5.5	SILT and SAND, trace gravel, trace clay Very dense Grey Wet		7	SS	132												
94.0			8	SS	100/0.0												
8.0	END OF BOREHOLE																
NOTES:																	
1. Borehole caved to a depth of 7.3 m below ground surface (Elev. 94.7 m) upon completion of drilling, March 10, 2015. Borehole dry above this depth on completion of drilling.																	

PROJECT 11-1184-0143		RECORD OF BOREHOLE No S1				SHEET 1 OF 1		METRIC									
G.W.P. 10-20011		LOCATION N 4860398.8 ; E 356528.6				ORIGINATED BY TD											
DIST Central HWY 401		BOREHOLE TYPE 150 mm O.D. Continuous Flight Solid Stem Augers				COMPILED BY PKS											
DATUM Geodetic		DATE March 9, 2015				CHECKED BY LCC											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
101.3	GROUND SURFACE																
0.0	TOPSOIL																
100.9																	
0.4	SILTY CLAY, trace gravel, trace sand, trace organics, containing rootlets Very stiff Brown Moist		1	AS	-												
			2	SS	21												
99.8																	
1.5	SILT and SAND, trace to some clay, trace gravel (TILL) Compact to very dense Brown, oxidation staining Moist		3	SS	24												
			4	SS	55												
			5	SS	57												
	- auger grinding on inferred cobble or boulder between a depth of 3.1 m and 6.1 m - becoming grey below a depth of 3.4 m																
			6	SS	100/0.13												
95.8																	
5.5	SAND and GRAVEL, trace to some silt, trace clay Very dense Grey, oxidation staining Moist		7	SS	100/0.1												
94.7			8	SS	100/0.1												
6.8	Silty SAND, trace clay, trace gravel (TILL) Very dense Brown Moist																
	END OF BOREHOLE																
	NOTES:  1. Borehole dry upon completion of drilling, March 9, 2015  2. Borehole caved to a depth of 5.5 m below ground surface (Elev. 95.8 m ) upon completion of drilling, March 9, 2015																

PROJECT <u>11-1184-0143</u>		<b>RECORD OF BOREHOLE No S2</b>		SHEET 1 OF 2		<b>METRIC</b>	
G.W.P. <u>10-20011</u>		LOCATION <u>N 4860328.0 ; E 356527.1</u>		ORIGINATED BY <u>TD</u>			
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>Solid Stem Augers</u>		COMPILED BY <u>PKS</u>			
DATUM <u>Geodetic</u>		DATE <u>March 11, 2015</u>		CHECKED BY <u>LCC</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								<div><div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div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Continued Next Page

+ <sup>3</sup>, × <sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

GTA-MTO 001 T:\PROJECTS\201111-1184-0143 (HWY 401 FROM BROCK RD TO COURTYARD RD)\LOG11-1184-0143.GPJ GAL-GTA.GDT 9/6/16 KD

PROJECT		RECORD OF BOREHOLE No S2				SHEET 2 OF 2		METRIC								
G.W.P. 11-1184-0143		LOCATION N 4860328.0 ; E 356527.1				ORIGINATED BY TD										
DIST Central HWY 401		BOREHOLE TYPE Solid Stem Augers				COMPILED BY PKS										
DATUM Geodetic		DATE March 11, 2015				CHECKED BY LCC										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---															
84.7			13	SS	100/0.13	85										
15.7	END OF BOREHOLE  NOTES:  1. Water level measured in borehole at a depth of 7.6 m below ground surface (Elev. 92.8 m) upon completion of drilling, March 11, 2015  2. Borehole caved to a depth of 8.2 m below ground surface (Elev. 92.2 m) upon completion of drilling, March 11, 2015  3. Water level in piezometer measured as follows:  Date      Depth (m)      Elev. (m) Jun 7/16    6.8                93.6															

GTA-MTO 001 T:\PROJECTS\201111-1184-0143 (HWY 401 FROM BROCK RD TO COURTYARD RD)\LOG11-1184-0143.GPJ GAL-GTA.GDT 9/6/16 KD



# **APPENDIX B**

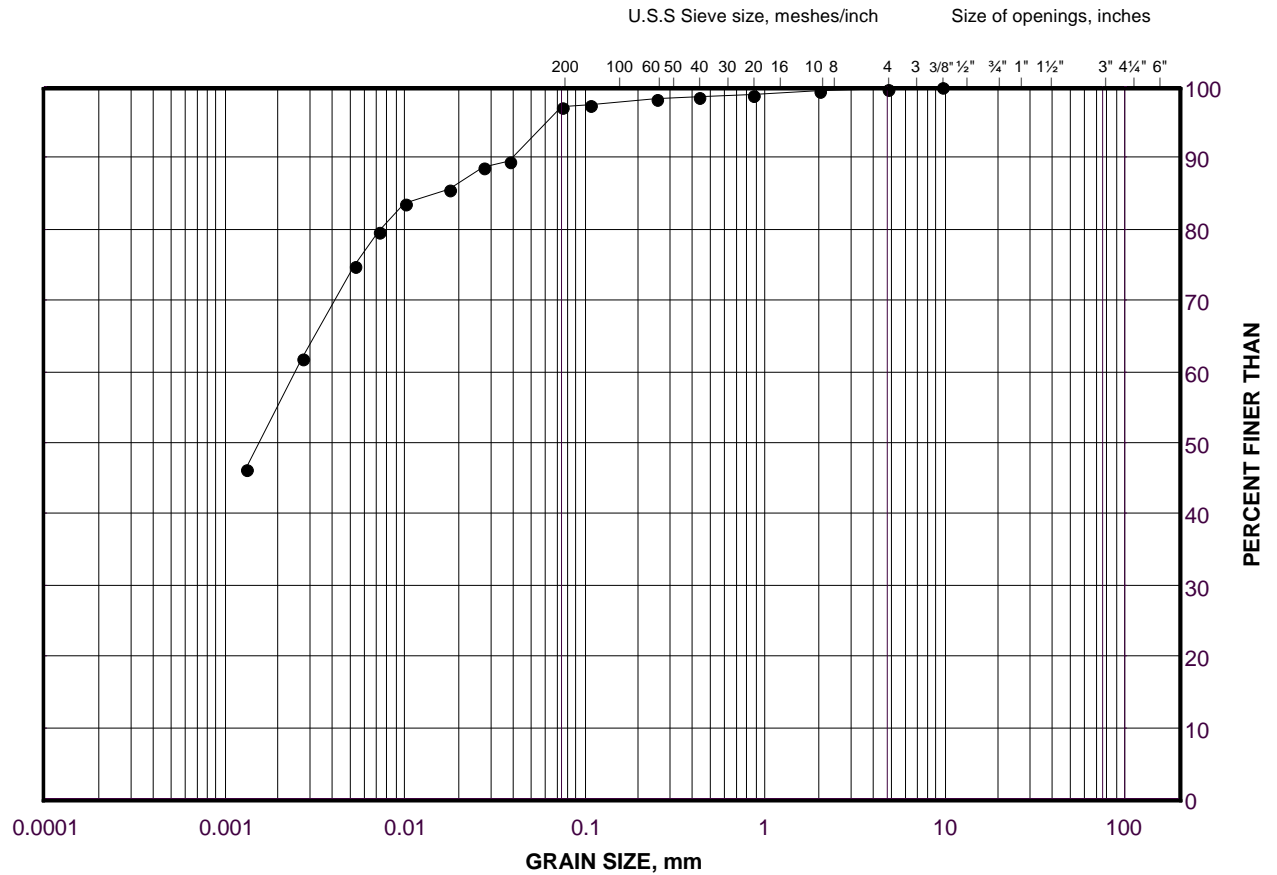
## **Geotechnical Laboratory Test Results**



# GRAIN SIZE DISTRIBUTION

Silty Clay to Clay

FIGURE B1



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

## LEGEND

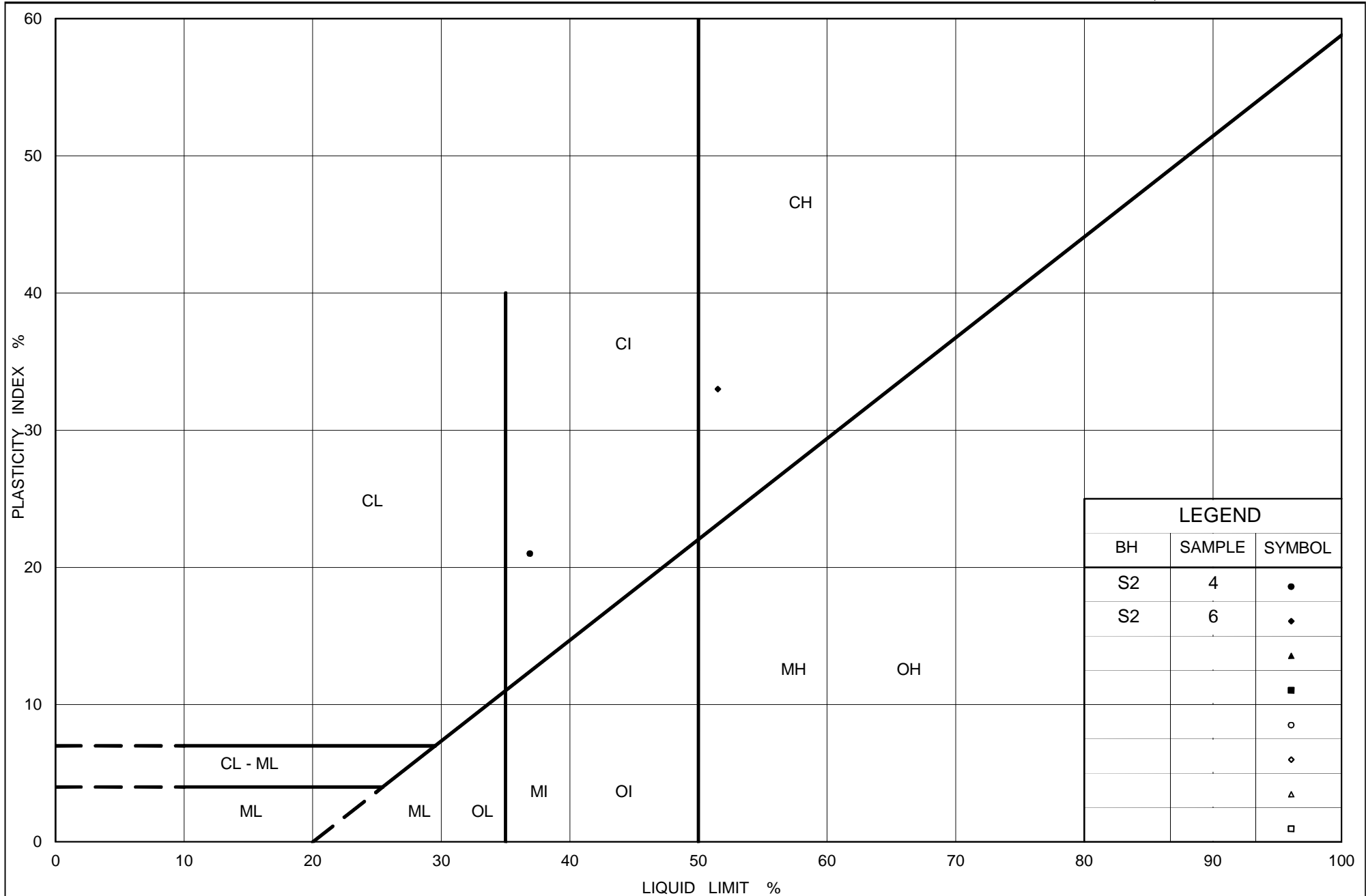
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	S2	6	95.6

Project Number: 11-1184-0143

Checked By: NK

**Golder Associates**

Date: 06-Jun-16



Ministry of Transportation

Ontario

# PLASTICITY CHART Silty Clay to Clay

Figure No. B2

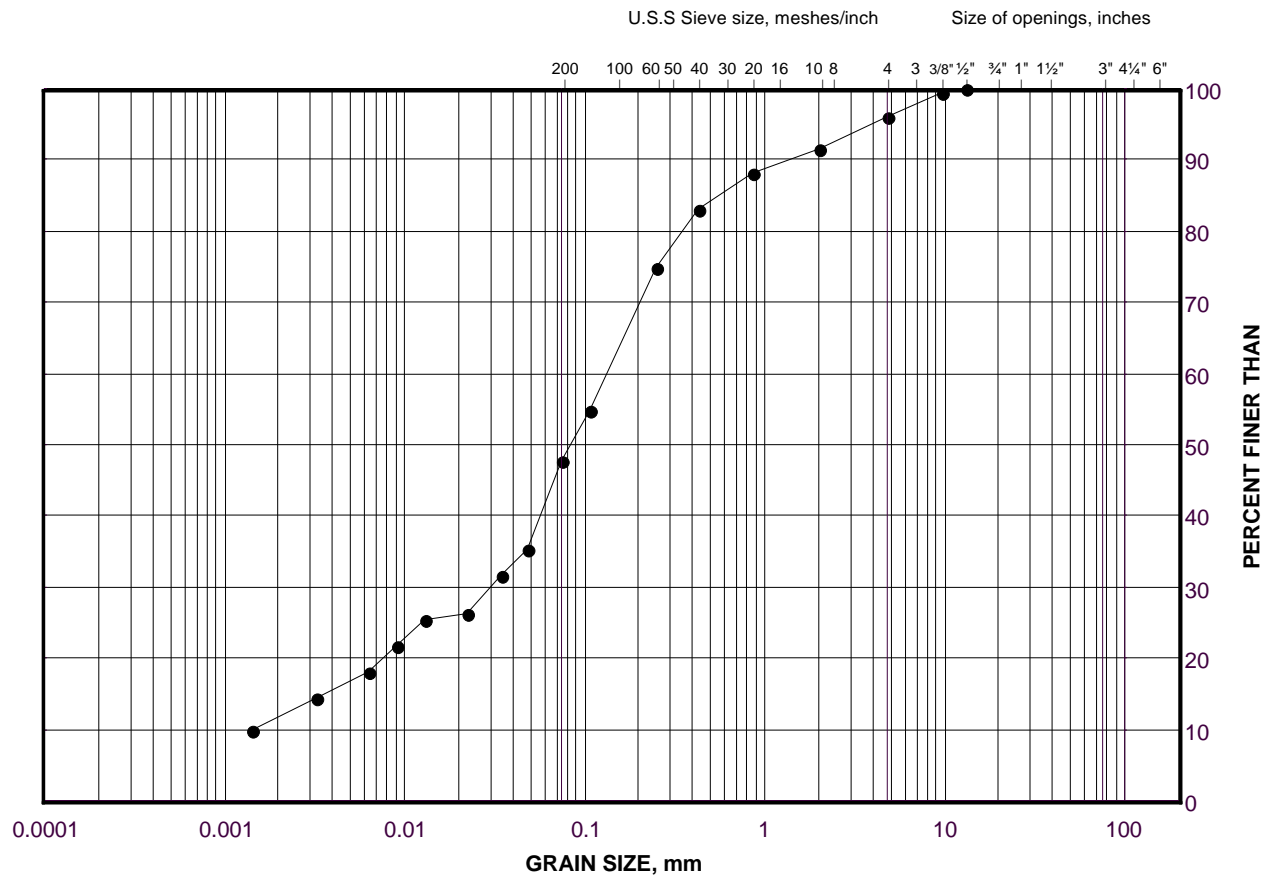
Project No. 11-1184-0143

Checked By: NK

# GRAIN SIZE DISTRIBUTION

Upper Silt and Sand Till

FIGURE B3



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	S1	3	99.5

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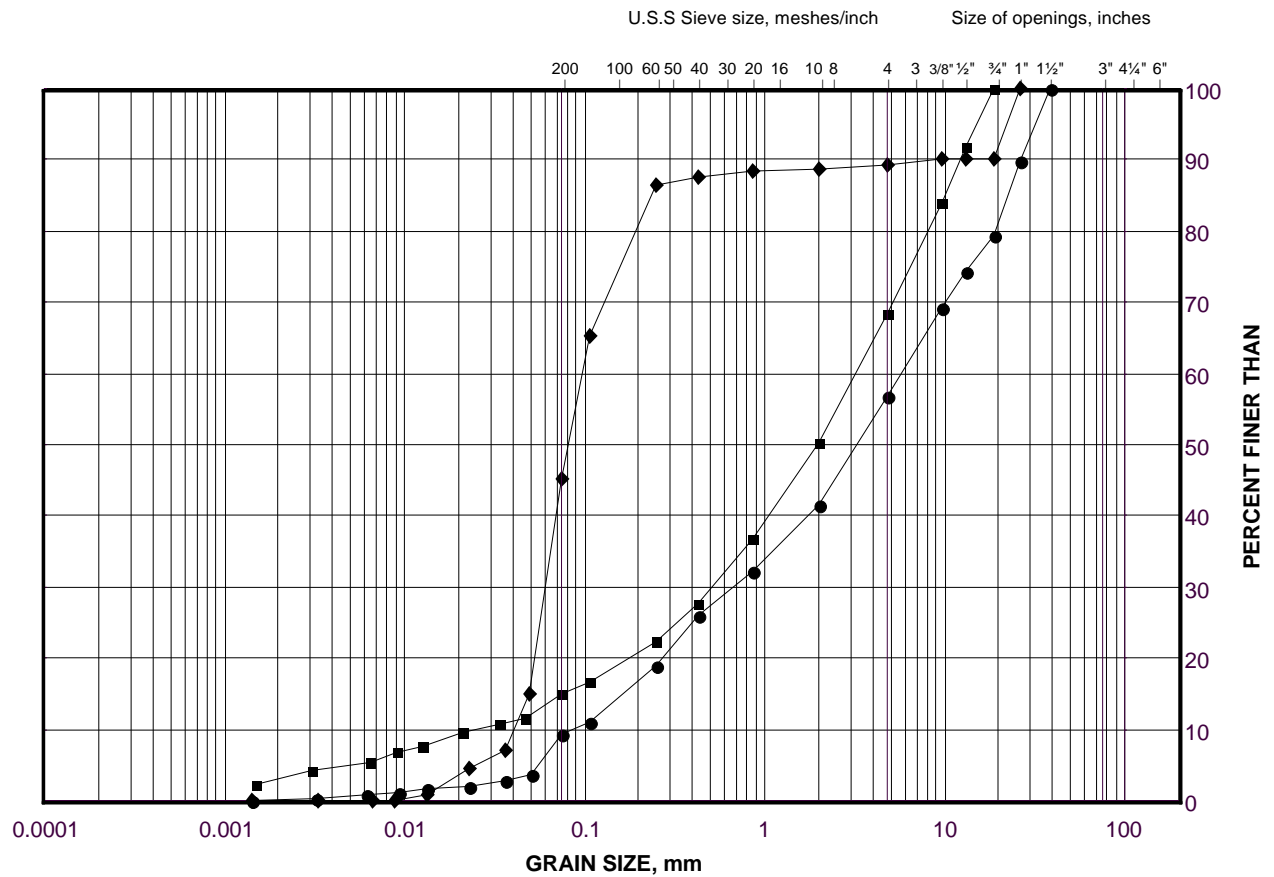
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Date: 06-Jun-16

# GRAIN SIZE DISTRIBUTION

Silt and Sand to Sand and Gravel

FIGURE B4



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

## LEGEND

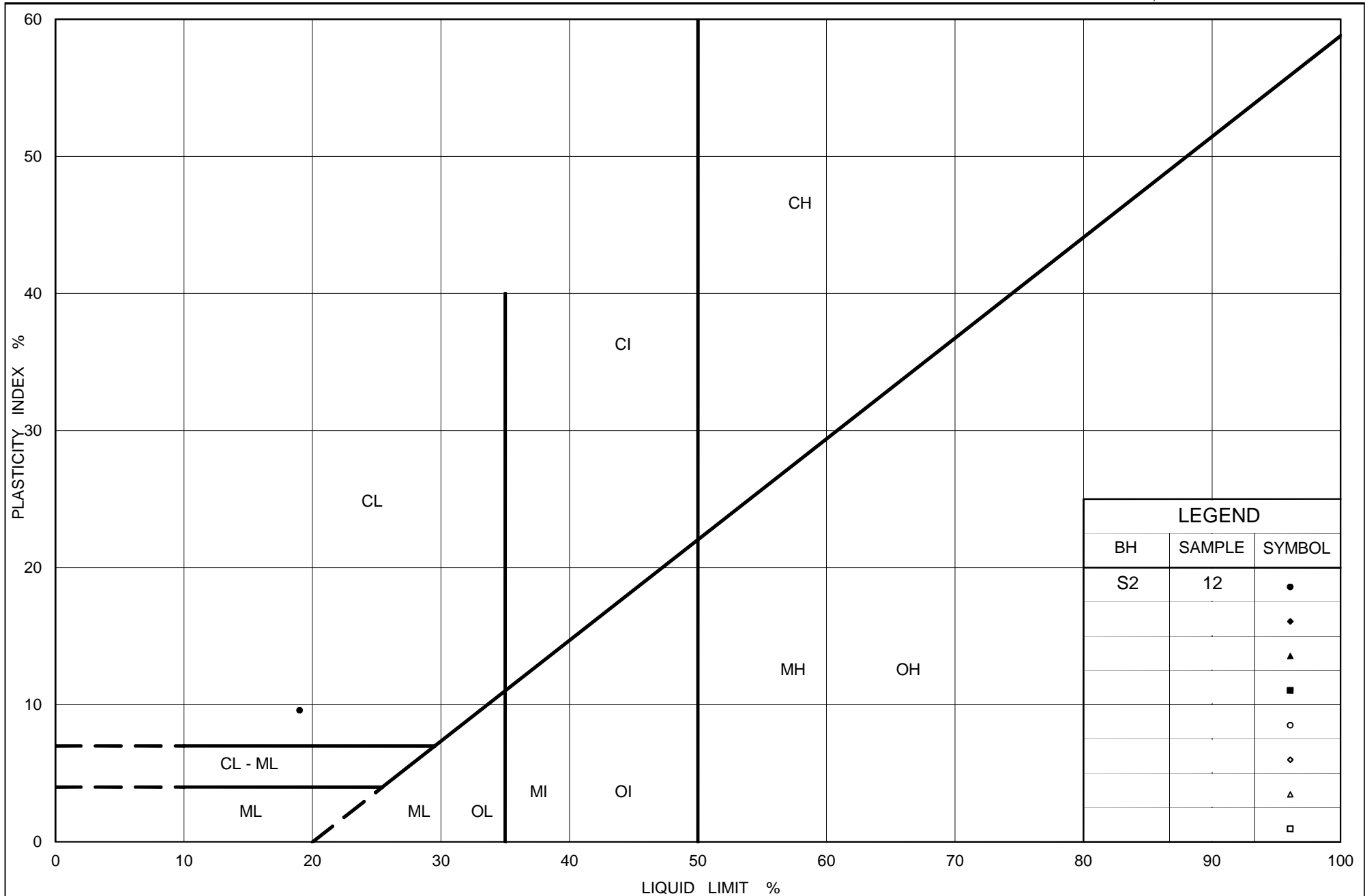
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S2	10	89.5
■	S1	7	95.0
◆	S2	8	92.5

Project Number: 11-1184-0143

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**Golder Associates**

Date: 06-Jun-16



Ministry of Transportation

Ontario

# PLASTICITY CHART

## Lower Clayey Silt Till

Figure No. B5

Project No. 11-1184-0143

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