



May 2012

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

**East Oakville Creek (Sixteen Mile Creek East
Tributary) Bridge on Trafalgar Road,
Structure Site No. 10-082
Highway 401 Widening from Trafalgar Road to
Regional Road 25, Halton Region
W.O. 07-20024**

Submitted to:
URS Canada Inc.
75 Commerce Valley Drive East
Markham, Ontario
L3T 7N9



REPORT

GEOCREs No. 30M12-333

Report Number: 09-1111-6036-8

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(SIXTEEN MILE CREEK EAST TRIBUTARY) BRIDGE**

PART A

**FOUNDATION INVESTIGATION REPORT
EAST OAKVILLE CREEK (SIXTEEN MILE CREEK EAST TRIBUTARY)
BRIDGE ON TRAFALGAR ROAD
HIGHWAY 401 WIDENING FROM TRAFALGAR ROAD
TO REGIONAL ROAD 25, HALTON REGION
W.O. 07-20024**



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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the future widening of Highway 401 from Regional Road 25 to Trafalgar Road (approximately 9 km) in the Regional Municipality of Halton, Ontario.

This report addresses the results of the subsurface investigation carried out for the proposed widening of the existing East Oakville Creek (Sixteen Mile Creek East Tributary) bridge on Trafalgar Road south of Highway 401.

The scope of work for the foundation engineering services for this structure site have been completed in general conformance with the Terms of Reference outlined in MTO's Request for Proposal (RFP) for Assignment No. 2008-E-0027 dated April 2009, and in a change request letter dated December 7, 2009.

2.0 SITE DESCRIPTION

The East Oakville Creek (Sixteen Mile Creek East Tributary) bridge is located on Trafalgar Road, approximately 400 m south of Highway 401 in the Regional Municipality of Halton, Ontario. The existing structure consists of a 28 m long single-span rigid-frame bridge (which was constructed in 1995-1996 to replace a previous two-lane structure), supported on spread footings.

In general, the terrain in this area is relatively flat, with the natural ground surface in the immediate vicinity of the structure site at about Elevation 194.5 m to 196 m. The "low flow" creek channel is at approximately Elevation 194.0 m, with a typical water level at about Elevation 194.2 m.

Trafalgar Road has been constructed in embankment fill that is approximately 2 m to 3 m in height, with the pavement grade at about Elevation 197.2 m at the bridge. The Trafalgar Road embankment side slopes are oriented at approximately 2 horizontal to 1 vertical (2H:1V).

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out in February 2011, during which time two boreholes (Boreholes 10-801 and 10-802) were advanced using a track-mounted Mini-Mole drill rig, supplied and operated by Kodiak Drilling Inc. of Oakville, Ontario. The borehole locations are shown on Drawing 1: both boreholes were advanced near the west toe of the existing Trafalgar Road embankment, with Borehole 10-801 in the southwest quadrant, and Borehole 10-802 in the northwest quadrant of the structure site.

Boreholes 10-801 and 10-802 were advanced using solid stem augers to depths of 17.8 m and 15.3 m, respectively. Soil samples were obtained continuously in the upper 4.5 m to 5 m of both boreholes, and at 1.5 m intervals of depth in the remainder of the boreholes, using a 50 mm outside diameter split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedure.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations. Both boreholes were backfilled with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended by Ontario Regulation 372).



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The field work was supervised on a full-time basis by a member of Golder's staff who located the boreholes in the field, directed the drilling, sampling, in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further examination and laboratory testing. Index and classification tests consisting of water content determinations, Atterberg limits testing and grain size distribution analyses were carried out on selected soil samples. The laboratory testing was conducted in accordance with MTO LS standards.

The borehole locations were measured by Golder personnel relative to site features and using a hand-held global positioning system (GPS) with a horizontal accuracy of 0.3 m. Using these field measurements, the ground surface elevations were determined from the digital terrain model (DTM) for this project. The borehole locations (referenced to MTM NAD83 coordinate system) and ground surface elevations (referenced to geodetic datum) are summarized in the following table and are shown on Drawing 1.

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
10-801	4,825,432.1	278,457.3	196.5	17.8
10-802	4,825,452.8	278,440.1	195.0	15.3

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 401 is located within the Peel Plain physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984).

The Peel Plain physiographic region covers the central portions of the Regional Municipalities of York, Peel and Halton. The general topography of this region consists of level to gently rolling terrain, sloping gradually southward toward Lake Ontario. A surficial till sheet, which generally follows the surface topography, is present throughout much of this area. The till, which is mapped in this area as the Halton Till, typically consists of clayey silt to silty clay, with occasional sand to silt zones. Shallow, localized deposits of loose sand and silt and/or soft clay can overlie this uppermost till sheet, and these represent relatively recent deposits, formed in small glacial meltwater ponds scattered throughout the Peel Plain and concentrated near river valleys. The recent sand, silt and clay and uppermost till deposits in this area overlie and are interbedded with stratified deposits of sand, silt and clay. The study area, in the western portion of the Peel Plain, is underlain by red shale of the Queenston Formation.

4.2 Subsurface Conditions

As part of the subsurface investigation, two boreholes were advanced at the structure location. The borehole locations, ground surface elevations and interpreted stratigraphic conditions are shown on Drawing 1. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are shown on the borehole records in Appendix A. The results of geotechnical laboratory



testing are also presented on Figures B1 to B6 in Appendix B. The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic sections on Drawing 1 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoil conditions encountered at this site consist of fill materials or a deposit of loose surficial silty sand overlying an upper deposit of very stiff to hard/very dense clayey silt to sand and silt till, which is underlain by a deposit of dense to very dense sand and gravel to silty sand, which is in turn underlain by a lower deposit of hard/very dense clayey silt to sand and silt till. A more detailed description of the soil deposits encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

Approximately 100 mm to 200 mm of topsoil was encountered immediately below the ground surface in both boreholes.

4.2.2 Fill

Approximately 1.6 m of fill was encountered below the topsoil in Borehole 10-801, which was advanced west of the Trafalgar Road embankment in the southwest quadrant of the structure site. The base of the fill was encountered at approximately Elevation 194.7 m.

The fill consists of an upper layer, 0.4 m in thickness, of silty sand containing trace clay and trace gravel, as well as small quantities of organics, and a lower layer, 1.2 m in thickness, of clayey silt containing trace sand and trace gravel. An Atterberg limits test was completed on one selected sample of the cohesive fill and measured a plastic limit of 17 per cent, a liquid limit of 24 per cent, and a plasticity index of about 7 per cent. This test result, which is plotted on a plasticity chart on Figure B1 in Appendix B, confirms that the cohesive portion of the fill is a clayey silt of low plasticity.

One Standard Penetration Test (SPT) "N" value of 10 blows was measured in the silty sand fill, indicating that this portion of the fill has a compact relative density, and SPT "N" values of 16 blows and 50 blows per 0.3 m of penetration were measured in the clayey silt fill, suggesting that this portion of the fill has a very stiff to hard consistency.

4.2.3 Surficial Silty Sand

A 0.5 m thick surficial deposit of silty sand was encountered immediately below the topsoil in Borehole 10-802, which was advanced west of the Trafalgar Road embankment toe in the northwest quadrant of the structure site. The base of this surficial deposit was encountered at approximately Elevation 194.4 m.

The deposit consists of silty sand containing trace clay and trace gravel. One SPT "N" value of 6 blows per 0.3 m of penetration was measured, indicating that this surficial deposit has a loose relative density.



4.2.4 Upper Clayey Silt Till to Sand and Silt Till

An upper till deposit was encountered below the fill in Borehole 10-801 and below the surficial silty sand in Borehole 10-802, with the deposit surface at Elevation 194.7 m and 194.4 m in these two boreholes, respectively. The deposit is 6.9 m thick in Borehole 10-801 in the southwest quadrant of the structure site, and 4.9 m thick in Borehole 10-802 in the northwest quadrant of the structure site, with the deposit base at approximately Elevation 187.8 m and 189.5 m, respectively.

The till deposit is comprised predominantly of clayey silt with sand, containing trace gravel, although the till grades to sand and silt containing trace clay and trace gravel below Elevation 191.0 m in Borehole 10-801. Cobbles were noted or inferred (due to bouncing of the split-spoon sampler) within the upper till deposit in Borehole 10-801. The results of grain size distribution tests completed on two samples of the upper till are presented on Figure B2 in Appendix B. Atterberg limits tests were completed on six samples the upper till and measured plastic limits of about 12 per cent to 14 per cent, liquid limits of about 15 per cent to 23 per cent, and plasticity indices of about 3 per cent to 9 per cent. These test results, which are plotted on a plasticity chart on Figure B3 in Appendix B, confirm that the deposit consists predominantly of low plasticity clayey silt, but that the lower portion of the deposit grades to a non-plastic or slightly plastic material.

The measured SPT “N” values within the clayey silt till range from 25 blows to 84 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency. One measured SPT “N” value of 101 blows for 0.15 m of penetration was measured in the sand and silt portion of the till, indicating a very dense relative density.

4.2.5 Sand and Gravel to Silty Sand

A deposit of sand and gravel to silty sand was encountered below the upper till, with its surface at Elevation 187.8 m and 189.5 m in Boreholes 10-801 and 10-802, respectively. The deposit is about 5.4 m thick in Borehole 10-801 in the southwest quadrant, and about 6.2 m thick in Borehole 10-802 in the northwest quadrant, extending to approximately Elevation 182.4 m and 183.3 m, respectively.

This deposit varies in composition from sand and gravel containing some silt and trace clay, to sand containing trace to some silt, trace gravel and trace clay, to silty sand containing trace clay. The results of grain size distribution tests on two selected samples of this deposit are shown on Figure B4 in Appendix B.

The measured SPT “N” values in this deposit range from 37 blows to 109 blows per 0.3 m of penetration, indicating a dense to very dense relative density.

4.2.6 Lower Clayey Silt Till to Sand and Silt Till

A lower till deposit was encountered below the sand and gravel to silty sand deposit, with its surface at Elevation 182.4 m and 183.3 m in Boreholes 10-801 and 10-802, respectively. Borehole 10-801 in the southwest quadrant was terminated after penetrating this deposit for a thickness of 3.7 m, encountering auger refusal at Elevation 178.7 m. Borehole 10-802 in the northwest quadrant was terminated within the till after penetrating it for a thickness of 3.6 m, to Elevation 179.7 m.



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The lower till deposit varies in composition from clayey silt with sand, containing trace to some gravel, to sand and silt containing trace to some clay and trace to some gravel. The results of grain size distribution tests on two selected samples of the till are shown on Figure B5 in Appendix B. Atterberg limits tests were completed on two selected samples of the lower till and measured plastic limits of about 11 per cent and 12 per cent, liquid limits of about 15 per cent and 17 per cent, and plasticity indices of about 3 per cent and 5 per cent. These test results, which are plotted on a plasticity chart on Figure B6 in Appendix B, confirm that the lower till varies in composition from low plasticity clayey silt to slightly plastic or non-plastic soil.

The measured SPT "N" values in the lower till deposit are greater than 100 blows per 0.3 m of penetration, suggesting a hard consistency or very dense relative density.

4.3 Groundwater Conditions



Details of the water levels observed in the open boreholes at the time of drilling are summarized on the borehole records in Appendix A. During drilling of both boreholes, high water pressure was observed in the sand and gravel to silty sand deposit, and at completion of drilling the water level in Boreholes 10-801 and 10-802 was at a depth of 1.4 m (Elevation 195.1 m) and at ground surface (Elevation 195.0 m), respectively.

The groundwater level is expected to fluctuate seasonally and is expected to rise during wet periods of the year.

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Gilberto Alexandre, and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Principal with Golder. Mr. Ty Garde, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

GOLDER ASSOCIATES LTD.



Lisa C. Coyne, P.Eng.
Senior Geotechnical Engineer, Principal



Ty Garde, P.Eng.
Principal, Designated MTO Foundations Contact

GA/LCC/TJG/jl

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**PRELIMINARY FOUNDATION REPORT - EAST OAKVILLE CREEK
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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
EAST OAKVILLE CREEK (SIXTEEN MILE CREEK EAST TRIBUTARY)
BRIDGE ON TRAFALGAR ROAD
HIGHWAY 401 WIDENING FROM TRAFALGAR ROAD
TO REGIONAL ROAD 25, HALTON REGION
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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed widening of the existing East Oakville Creek (Sixteen Mile Creek East Tributary) bridge on Trafalgar Road. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. Further investigation and analysis will be required during detail design.

Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Foundation Options

As part of the future widening of Highway 401 from Regional Road 25 to Trafalgar Road in the Regional Municipality of Halton, the Trafalgar Road underpass at Highway 401 is to be replaced to the west of the existing underpass structure. As a result of the westward shift of the Trafalgar Road alignment, the existing East Oakville Creek (Sixteen Mile Creek East Tributary) bridge, which is located on Trafalgar Road approximately 400 m south of Highway 401, may require widening by approximately 2.2 m westward.

The existing East Oakville Creek (Sixteen Mile Creek East Tributary) bridge consists of a 28 m long single-span structure with a clear span of 16 m, with the existing abutments supported on spread footings. Based on the General Arrangement drawing for the existing bridge (prepared by Totten Sims Hubicki Associates with revisions dated September 1994 and marked "as constructed" in March 1997), the footings are about 3.5 m wide and are founded at approximately Elevation 191.4 m. This founding elevation is about 2.6 m below the bottom of the creek channel, and about 3.6 m below the natural ground surface at this site.

Based on the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments for the potential 2.2 m westward widening of the existing East Oakville Creek (Sixteen Mile Creek East Tributary) bridge. 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 approximate costs is provided in Table 1 following the text of this report.

- **Spread footings founded on the very stiff to hard upper clayey silt till:** Spread footings founded on the very stiff to hard upper clayey silt till are feasible for the proposed westward widening of the existing abutments, as well as associated wing walls or retaining walls. Temporary protection systems will be required along the west side of Trafalgar Road adjacent to the abutment widening area, and potentially along East Oakville Creek (Sixteen Mile Creek East Tributary), to permit excavation to the required depth. There is potential for up to about 15 mm of differential settlement of the widened portion of the structure



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relative to the existing bridge; however, it is understood that the structural connection can be designed and constructed to accommodate the resulting stresses.

- **Footings “perched” on a compacted granular pad in the existing approach embankments:** Although this option would be advantageous in minimizing the depth of excavation as compared with footings founded on native soil, “perched” footings could not be constructed in line with the existing abutments without impacting one or both of the channel width or the scour protection for the footings. Therefore, this option is not recommended for the westward widening.
- **Driven steel H-piles:** Driven steel H-piles are feasible for support of the abutment widening, and would permit integral abutment design if the existing structure could be modified to semi-integral abutments and appropriate structural connections developed to allow a pile-supported widening to be compatible with the existing spread footing-supported abutments. Driven steel H-piles are also feasible for support of the associated wingwalls/retaining walls. The principle advantage of deep foundations would be to minimize differential settlement between the widened portion and the existing bridge. There is a minor risk associated with the piles meeting refusal on or being damaged or deflected by cobbles or boulders within the glacial soils, as well as a minor to moderate risk for the potential for upward flow of groundwater and fine soil particles along the pile shafts due to the high groundwater pressure at this site. However, the risk associated with upward flow along the piles can be mitigated, as discussed further in Section 6.4.1 and in Section 6.7.
- **Driven steel pipe (tube) piles:** Steel pipe (tube) piles could also be considered as a deep foundation option for support of the abutment widening and any associated wing walls/retaining walls, and these would offer the same advantage as steel H-piles in terms of minimizing differential settlement of the widening relative to the existing structure. However, pipe piles are considered to have a slightly higher (minor to moderate) risk as compared with H-piles for meeting refusal on or being damaged or deflected away from their vertical or battered orientation due to the presence of cobbles and/or boulders within the glacially-derived soils at this site. In addition, as for steel H-piles, mitigation may be required against the potential upward flow of groundwater and fine soil particles along the pile shafts.
- **Caissons:** Caissons are feasible for support of the potential bridge widening. However, this option is considered to pose higher risks than driven pile foundations in terms of the potential for ground disturbance or loss in the water-bearing sand and gravel to silty sand deposit, although this could be mitigated with the use of temporary or permanent liners. Caisson foundations may in fact present an advantage over spread footings at this site, as the narrow widening could be supported directly on a single caisson, potentially eliminating the need for the relatively deep excavation that would be required to construct spread footings to match the existing footing elevation.

The following sections provide recommendations for spread footing foundations, driven steel H-pile or pipe pile foundations, and caisson foundations to support the potential bridge widening. Based on the subsoil conditions at the site and the above considerations, and considering the satisfactory performance of the existing spread footing-supported structure, the preferred option from a geotechnical/foundations perspective is to support the westward widening of the abutments on spread footings founded on the very stiff to hard upper clayey silt till deposit. The use of a single caisson or caissons to directly support the bridge deck widening is considered to be an acceptable alternative from a foundations perspective, provided that temporary or permanent liners are used to control the ground and groundwater in the sand and gravel to silty sand and the sand and silt till deposits.



6.3 Shallow Foundations

6.3.1 Founding Elevations

For support of the widened abutments and associated wing walls or retaining walls, spread footings should be founded below the fill and the loose surficial silty sand, on the very stiff to hard clayey silt till deposit, at a minimum depth of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration as per Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*).

Based on the results from Boreholes 10-801 and 10-802 and considering frost protection requirements relative to the channel base at Elevation 194.0 m, spread footings for the widening could be founded as high as Elevation 192.8 m. It is noted, however, that such a founding level would be approximately 1.4 m higher than the existing abutment footing founding level (at Elevation 191.4 m). It would be difficult to connect the widened footing to the existing footing in this case, the widened abutment footings would increase the loading on the existing, lower footings, and the widened footings may not have adequate cover for scour protection. Therefore, it is recommended that spread footings for the widening be constructed to match the existing footing founding elevation, as summarized in the table below.

Foundation Element	Maximum (Highest) Founding Elevation	Approximate Excavation Depth
South Abutment	191.4 m	3.6 m – relative to original ground 5.8 m – relative to Trafalgar Road grade
North Abutment	191.4 m	3.6 m – relative to original ground 5.8 m – relative to Trafalgar Road grade

The footing subgrade should be inspected by a Quality Verification Engineer (QVE) following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to check that all existing fill, loose or very soft to firm surficial soils, or other unsuitable material have been removed. The founding soils will be susceptible to disturbance and should be protected with a concrete working slab (100 mm thick concrete slab with a compressive strength of 20 MPa) if the concrete for the footing is not placed within four hours of the inspection and approval of the subgrade.

Scour protection should be provided to the widened footings, similar to that provided for the existing structure, which appears to be performing adequately. This could consist of a 600 mm thickness of rock protection placed on top of non-woven geotextile, over and in front of the widened footings.

6.3.2 Geotechnical Resistance/Reaction

Strip or spread footings placed on the properly prepared, very stiff to hard clayey silt till, at or below the design elevations given in the preceding section, should be designed based on the factored geotechnical resistances at Ultimate Limit States (ULS) and geotechnical reactions at Serviceability Limit States (SLS) given below.



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Foundation Element	Footing Width	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS*
South Abutment	3 m	550 kPa	450 kPa
	4 m	600 kPa	400 kPa
North Abutment	3 m	550 kPa	450 kPa
	4 m	600 kPa	400 kPa

* For 25 mm of settlement

The preliminary geotechnical resistances should be reviewed if the selected footing width or founding elevation differs from those given above. In addition, these preliminary geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its *Commentary*.

The preliminary geotechnical resistance values provided above will have to be re-evaluated and modified as necessary during detail design, based on future additional subsurface investigation at the proposed widened abutment locations.

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

6.4.1 Founding Elevations

The widened abutments and any associated wing walls/retaining walls may be supported on steel H-piles or steel pipe (tube) piles driven to found within the hard/very dense lower till deposit. The following pile tip elevations may be used for preliminary design purposes, assuming about 2 m to 2.5 m of penetration into till soils having SPT “N” values of greater than 100 blows per 0.3 m of penetration:

Foundation Element	Estimated Design Pile Tip Elevation
North Abutment	181.0 m
South Abutment	179.5 m

The pile caps should be constructed at a minimum depth of 1.2 m for frost protection purposes, per OPSP 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*).

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 soil deposit. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a slightly higher risk of “hanging up” or being deflected away from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip with driving shoes or flange plates to reduce the potential for damage to the piles during driving, in accordance with OPSS 903 (*Deep Foundations*). In very dense/hard and boulder soils, as are anticipated at this site, driving



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shoes (such as Titus Standard “H” Bearing Pile Points) are preferred over flange plates. If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001-100 Type II (*Steel Tube Pile Driving Shoe*).

The groundwater pressure associated with the sand and gravel to silty sand deposit was observed to be at ground surface, although further investigation and measurement will be required at the detail design stage to confirm the piezometric pressure. Depending on the piezometric pressure relative to the pile cap level and the finished grade, specialized construction techniques may be required to mitigate the potential upward flow of water along the pile shafts. Such measures could include construction of a filter blanket with subdrains to prevent loss of fine soil particles along the pile-soil interface.

6.4.2 Axial Geotechnical Resistance/Reaction

For HP 310x110 piles driven to the estimated tip elevations provided in Section 6.4.1, the factored axial geotechnical resistance at ULS may be taken as 1,600 kN, and the axial geotechnical reaction at SLS (for 25 mm of settlement) may be taken as 1,400 kN. Similar axial resistances may be used for 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.).

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 Standard Structural Drawing SS-103-11) during the final stages of driving to achieve the appropriate ultimate capacity.

The preliminary geotechnical resistances provided above will have to be re-evaluated and modified as necessary during detail design in consideration of the additional subsurface investigation that will be carried out at the widened foundation elements.

6.5 Caisson Foundations

Caissons are feasible for support of the widened abutments and any associated retaining walls at this site. Indeed, the use of a single caisson to directly support the 2.2 m bridge deck widening without a below-grade pile cap is considered to be advantageous over completing excavation to depths of 3.6 m to 5.8 m below the natural ground surface and Trafalgar Road grade, respectively.

It is noted that there is a high risk that running or flowing of the water-bearing sand and gravel to silty sand deposit could occur during or after drilling, and basal heave could occur where the water-bearing sand and silt till is present at the caisson base. If caisson foundations are adopted, a liner would be required to support the soils during construction and permit inspection and cleaning of the caisson base.



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6.5.1 Founding Elevations

The widened abutments could be supported on caissons founded within the hard/very dense soils (with SPT “N” values of greater than 100 blows per 0.3 m of penetration) that were encountered below approximately Elevation 183 m on the north side of Highway 401, and below about Elevation 181.5 m on the south side of Highway 401. For preliminary design, the following caisson base elevations may be assumed based on the borehole results:

Foundation Element	Founding Stratum	Estimated Design Caisson Elevation
North Abutment Widening	Hard/very dense clayey silt till to sand and silt till	181.0 m
South Abutment Widening	Hard clayey silt till	180.0 m

6.5.2 Axial Geotechnical Resistance

The following factored axial geotechnical resistances at ULS and the axial geotechnical reactions at SLS (for 25 mm of settlement) may be used for preliminary design purposes:

Caisson Diameter	Factored Axial Geotechnical Resistance at ULS	Geotechnical Reaction at SLS
1.2	3,500 kN	2,800 kN
1.5	5,000 kN	4,000 kN

The preliminary geotechnical resistances provided above will have to be re-evaluated and modified as necessary during detail design in consideration of the additional subsurface investigation that will be carried out at the widened foundation elements.

6.6 Approach Embankments

6.6.1 Subgrade Preparation and Embankment Construction

It is recommended that all topsoil/organic material or existing surficial fill materials be stripped from the footprint of the proposed westward widening of the existing Trafalgar Road approach embankments. The depth and extent of stripping should be assessed during detail design when additional subsurface information will be available within the widened approach embankment areas.

The embankment fill for Trafalgar Road widening should be placed and compacted in accordance with MTO's Special Provision SP206S03 (*Earth Excavation and Grading*) and SP105S21 (*Compacting*). Benching of the existing Trafalgar Road embankment side slopes should be carried out to “key in” the new fill materials for the widening, in accordance with OPSD 208.010 (*Benching of Earth Slopes*).



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Additional fill for construction of the embankment widening could consist of clean earth fill or granular fill. From a geotechnical/foundations perspective, both earth and granular fill will provide good compatibility with the existing embankment fill materials – both those fill materials remaining in-place in the existing embankment side slope, and any existing embankment fill that is re-used for the widening after being cut from the benches.

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS 572 (*Seed and Cover*).

6.6.2 Approach Embankment Stability

Slope stability analyses have been performed for the proposed Trafalgar Road embankment widening using the commercially available program SLIDE, produced by Rocscience Inc., to check that a minimum factor of safety of 1.3 is achieved for the proposed embankment heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed approach embankment widening at the 16 Mile Creek bridge site, considering the design requirements and the available field and laboratory testing data.

The stability analyses were completed for an approximately 3 m high embankment widening with a width of less than 5 m, based on the subsurface conditions as encountered in Boreholes 10-801 and 10-802. The following parameters have been used in the analyses, based on field and laboratory test data as well as accepted correlations:

Soil Deposit	Bulk Unit Weight (kN/m ³)	Effective Friction Angle	Undrained Shear Strength (kPa)
Embankment fill	21	32-35°	-
Loose surficial silty sand	19	28°	-
Very stiff to hard/very dense upper clayey silt to sand and silt till	21	32°	-
Dense to very dense sand and gravel to silty sand	20	32°	-
Hard/very dense lower clayey silt till	21	32°	-

The analysis results indicate that an approximately 3 m high embankment widening with side slopes oriented no steeper than 2H:1V will have a factor of safety of greater than 1.3 against global instability, assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials. An example static global stability result is provided on Figure 1. This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed based on the subsoil conditions encountered within the proposed approach embankment footprints during detail design.



6.6.3 Approach Embankment Settlement

Based on the study completed to date, the existing East Oakville Creek (Sixteen Mile Creek East Tributary) bridge approach embankments will require widening by approximately 2.2 m immediately adjacent to the north abutment of the bridge, up to approximately 5 m at a distance of 20 m away from the north abutment.

Settlement analyses for the anticipated soil conditions below the widened approach embankments were carried out using both hand calculations and the commercially available computer program *Settle-3D* from Rocscience, using estimated elastic deformation moduli as given in the table below, based on correlations with the SPT “N” values and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Deposit	Bulk Unit Weight (kN/m ³)	Elastic Modulus (MPa)
Embankment fill	21	-
Very stiff to hard upper clayey silt till/Very dense sand and silt till	21	80 MPa
Very dense sand and gravel	21	100 MPa
Dense sand	20	70 MPa
Very dense silty sand	20	60 MPa
Hard/very dense lower till	21	100 MPa

Based on this preliminary assessment, the settlement of the foundation soils under the widened 3 m high approach embankments is estimated to be approximately 15 mm, decreasing to less than 5 mm near the existing highway shoulder and under the widened embankment toe. This settlement is expected to occur relatively quickly during and immediately following construction of the widened approach embankments based on the nature of the soils at the site. This estimated magnitude of settlement should be reassessed based on the soil and groundwater conditions under the widened approach embankments as determined during the detail design, with particular emphasis on the thickness and properties of any surficial soil deposits within the embankment widening footprint.

The above preliminary estimates do not include compression of the fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of fill compression may range from 0.5 to 1 per cent of the height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur essentially during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time.



6.7 Construction Considerations

The following subsections identify future construction issues that should be considered at this stage as they may impact the planning and preliminary design. Where applicable, Non-Standard Special Provisions (NSSP) should be developed during detail design for incorporation in the Contract Documents.

6.7.1 Excavation and Temporary Protection Systems

The foundation excavations for spread footings would extend to a depth of approximately 2 m to 2.5 m below the natural ground surface and about 4.5 m below the Trafalgar Road grade, through existing fill and the loose surficial silty sand, into the very stiff to hard clayey silt till deposit. The excavations for pile caps could be maintained higher within the native soils or embankment fill, but would still require excavation through the existing fill and potentially into the very stiff to hard clayey silt till.

Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill and surficial silty sand would be classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V.

At this preliminary stage, it is anticipated that temporary protection systems would be required along the west side of Trafalgar Road adjacent to the proposed abutment widening areas, as well as in front of the proposed abutment widening areas parallel to East Oakville Creek (Sixteen Mile Creek East Tributary). The temporary excavation support system should be designed and constructed in accordance with OPSS 539 (*Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539.

It is considered that either a driven, interlocking sheetpile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support at this site, based on the subsurface soil and groundwater conditions. An interlocking sheetpile system would contribute to both ground and groundwater control, which may be beneficial along the creek due to the potential for the foundation excavations to intersect alluvial soils. For a soldier pile and lagging system, it would be necessary to control seepage or include measures to mitigate loss of soil particles from alluvial soil deposits through the lagging boards.

6.7.2 Groundwater Control

Groundwater seepage is anticipated from granular soil lenses within the clayey silt till, or from the base of existing granular fill where groundwater may be “perched” on top of the underlying cohesive soils. The seepage volume from these sources is expected to be relatively small, such that the water inflow can be handled by pumping from filtered sumps placed at the base of the excavations. There is a risk that foundation excavations for the abutment widening areas will intersect water-bearing alluvial soils or fill materials associated with the creek meander channel, contributing to higher groundwater inflows into the excavation. Further investigation will be required during detail design to assess the potential presence of such soils within the foundation excavation areas; if appropriate, groundwater inflows could be minimized with the use of an interlocking steel sheetpile system.



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As discussed in Section 6.4, the groundwater pressure associated with the sand and gravel to silty sand aquifer is relatively high (at or near the natural ground surface), although further investigation and measurement will be required at the detail design stage to determine the stabilized groundwater level. Depending on the piezometric pressure relative to the pile cap level and the finished grade, specialized construction techniques may be required to mitigate the potential upward flow of water and fine soil particles along the pile shaft. Such measures could include construction of a filter blanket with subdrains under the pile cap to prevent loss of fine soil particles along the pile-soil interface.

As discussed in Section 6.5, if caisson foundations are adopted, running or flowing of water-bearing cohesionless soil strata could occur during or after drilling of the caissons, and basal heave could occur where water-bearing cohesionless soils are present at/near the caisson base. If caisson foundations are adopted, temporary or permanent caisson liners would be required to support the soils during construction and permit inspection and cleaning of the caisson base. It is recommended that an NSSP be included in the Contract Documents to warn the contractor of these conditions and the need to control the ground and groundwater during caisson construction.

6.7.3 Subgrade Protection

The clayey silt till soils that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the General Arrangement drawing and/or with an NSSP, which can be developed during the detail design stage.

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 the next stage of investigation in support of the detail design. If conditions warrant, an NSSP should be included in the Contract Documents developed during the detail design stage to identify to the contractor the possible presence of cobbles and/or boulders within the overburden soils.

6.7.5 Vibration Monitoring During Pile Installation

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations induced by conventional construction activities (such as pile driving) will reach this threshold level. Therefore, vibration monitoring is not expected to be required during construction at this site.



6.8 Recommendations for Further Work in Detail Design

Additional shallow boreholes are recommended within the footprint of the north and south abutment widening areas, any associated wing walls/retaining walls, and the approach embankment widening areas during the future detail design stage of investigation, to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided in this report, as follows:

- **Abutments and retaining walls:**
 - Assessment of the variability of any existing fill and alluvial or near-surface soils to confirm the founding elevation for spread footings within each abutment widening area, and to assess groundwater control requirements associated with seepage, particularly if alluvial soil deposits are present.
 - Confirmation of the groundwater levels associated with the aquifer at this site. Depending on the piezometric pressure, it may be necessary to include measures for mitigation of water seepage and loss of fine soil particles along the pile-soil interface.
 - Observation of the presence of cobbles and/or boulders within the soil deposits, to assess the need for an NSSP to warn the contractor of the presence of such obstructions as they may affect excavations and the installation of driven pile foundations (if adopted) or temporary protection systems.
- **Approach embankments:**
 - Assessment of the depth and extent of stripping of topsoil/organics and fill materials within the footprint of the widened approach embankments.
 - Further assessment of the thickness and consolidation/elastic compression properties of any surficial soils within the footprint of the widened approach embankments, to confirm the stability analyses and settlement estimates.



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7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Gilberto Alexandre, and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Principal with Golder, with technical input from Mr. Murty Devata, P.Eng., a specialist geotechnical/foundations consultant at Golder. Mr. Ty Garde, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

GOLDER ASSOCIATES LTD.



Lisa C. Coyne, P.Eng.
Senior Geotechnical Engineer, Principal



Ty Garde, P.Eng.
Principal, Designated MTO Foundations Contact

GA/LCC/TJG/jl

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east oakville creek bridge on trafilgar road.docx



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Ontario Provincial Standard Specifications (OPSS)

- | | |
|-----------|--|
| OPSS 539 | Construction Specification for Temporary Protection Systems |
| OPSS 572 | Construction Specification for Seed and Cover |
| OPSS 902 | Construction Specification for Excavating and Backfilling Structures |
| OPSS 903 | Construction Specification for Deep Foundations |
| OPSS 1010 | Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material |

Ontario Provincial Standard Drawings (OPSD)

- | | |
|---------------|--|
| OPSD 3000.100 | Foundation Piles – Steel H-Pile Driving Shoe |
| OPSD 3090.101 | Foundation Frost Penetration Depths for Southern Ontario |

Construction Design Estimating and Documentation (CDED) Special Provisions (SP)

- | | |
|----------|---|
| SP105S21 | Amendment to OPSS 501 – Construction Specification for Compacting |
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PRELIMINARY FOUNDATION REPORT - EAST OAKVILLE CREEK (SIXTEEN MILE CREEK EAST TRIBUTARY) BRIDGE

TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages/Risks	Constructability	Estimated Costs
Spread footings founded on the very stiff to hard upper clayey silt till	<ul style="list-style-type: none">• Feasible for support of widened abutments and associated wing walls/retaining walls	<ul style="list-style-type: none">• Existing structure supported on shallow foundations, and has performed well• Relatively minor groundwater seepage anticipated from clayey silt till deposit• Allows for semi-integral abutments	<ul style="list-style-type: none">• Excavations, to a depth of about 2.2 m below original grade and 4.4 m below Trafalgar Road grade, would require temporary protection systems• Risk of encountering water-bearing alluvial soils within foundation excavation, increasing ground and groundwater control requirements, although this could be mitigated by using interlocking sheetpiles for the temporary protection systems• Potential for up to about 15 mm of differential settlement between existing bridge and widened portion• Precludes use of integral abutments; potentially greater maintenance required at abutments• Lower geotechnical resistances as compared with deep foundations	<ul style="list-style-type: none">• Conventional excavation and construction techniques	<ul style="list-style-type: none">• Less expensive than deep foundations although bridge maintenance costs may be higher due to non-integral abutment configuration• Estimated cost is \$12,000 per abutment widening, based on an estimated \$600/m³ for construction of shallow foundations, excluding deeper excavation and temporary protection system



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Foundation Option	Feasibility	Advantages	Disadvantages/Risks	Constructability	Estimated Costs
Spread/strip footings perched on compacted granular pad in approach embankment fill	<ul style="list-style-type: none"> Not recommended for support of widened abutments 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than footings founded on till deposit, reducing excavation depth and protection system requirements 	<ul style="list-style-type: none"> Susceptible to scour Potential for differential settlement between the existing bridge and the widening is greater than for shallow foundations on till or steel H-piles due to settlement in existing fill and near-surface till soils under the widened approach embankment loading 	<ul style="list-style-type: none"> Conventional excavation and construction techniques 	<ul style="list-style-type: none"> This option not recommended
Steel H-piles driven to found in the "100-blow" lower clayey silt to sand and silt till	<ul style="list-style-type: none"> Feasible for support of widened abutments and associated wing walls/retaining walls 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than footings founded on till deposit, reducing depth of excavation and temporary protection system requirements adjacent to Trafalgar Road and the creek Limited groundwater control required Allows for integral abutment construction Would minimize differential settlement between existing bridge and westward widening 	<ul style="list-style-type: none"> Minor potential for encountering obstructions (cobbles and/or boulders) during pile driving; this could result in piles "hanging up" and lower geotechnical resistances Due to high groundwater level at site, potential risk of upward flow of water and fine soil particles along pile shaft; mitigation measures may be required 	<ul style="list-style-type: none"> Conventional construction methods for H-pile foundations 	<ul style="list-style-type: none"> Lower relative cost compared with caisson option Estimated cost is approximately \$18,000 per abutment widening, based on an estimated \$250/m length for pile installation and \$600/m³ for pile cap construction



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Foundation Option	Feasibility	Advantages	Disadvantages/Risks	Constructability	Estimated Costs
Steel pipe (tube) piles driven to found in the "100-blow" lower clayey silt to sand and silt till	<ul style="list-style-type: none"> Feasible for support of widened abutments and associated wing walls/retaining walls 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than footings founded on till deposit, reducing depth of excavation and temporary protection system requirements adjacent to Trafalgar Road and the creek Limited groundwater control required Allows for semi-integral abutment configuration Would minimize differential settlement between existing bridge and westward widening 	<ul style="list-style-type: none"> Minor to moderate risk (slightly greater risk than for steel H-piles) of encountering obstructions such as cobbles and/or boulders during driving; this could result in piles "hanging up" and lower geotechnical resistances Due to high groundwater level at site, potential risk of upward flow of water and fine soil particles along pile shaft; mitigation measures may be required 	<ul style="list-style-type: none"> Conventional construction methods 	<ul style="list-style-type: none"> Costs for steel pipe (tube) piles slightly higher than for steel H-piles
Caissons founded in the "100-blow" lower clayey silt to sand and silt till	<ul style="list-style-type: none"> Feasible for support of widened abutments and wing walls/retaining walls 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than footings founded on till deposit, reducing depth of excavation and temporary protection system requirements; it may also be possible to eliminate the pile cap entirely and support the widened structure directly on a single caisson or caissons Higher capacity than for steel H-piles, so reduced number of deep foundation elements compared to steel H-piles 	<ul style="list-style-type: none"> Water-bearing cohesionless deposits could contribute to loss of ground; temporary or permanent liners would be required; likely not possible to inspect caisson base Precludes use of integral abutments 	<ul style="list-style-type: none"> Risk of loss of ground through aquifer and in sand and silt till where encountered at or near base of caissons, requiring temporary or permanent liners 	<ul style="list-style-type: none"> Higher cost compared with shallow foundations or steel H-piles



SHEET



LEGEND

BOREHOLE CO-ORDINATES

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 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.

The complete Preliminary Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

Base plans provided in digital format by URS. (Drawing File X-Align_401.dwg, received March 10, 2011, X-contour.dwg, received August, 2010, and X-Base, received September, 2010).

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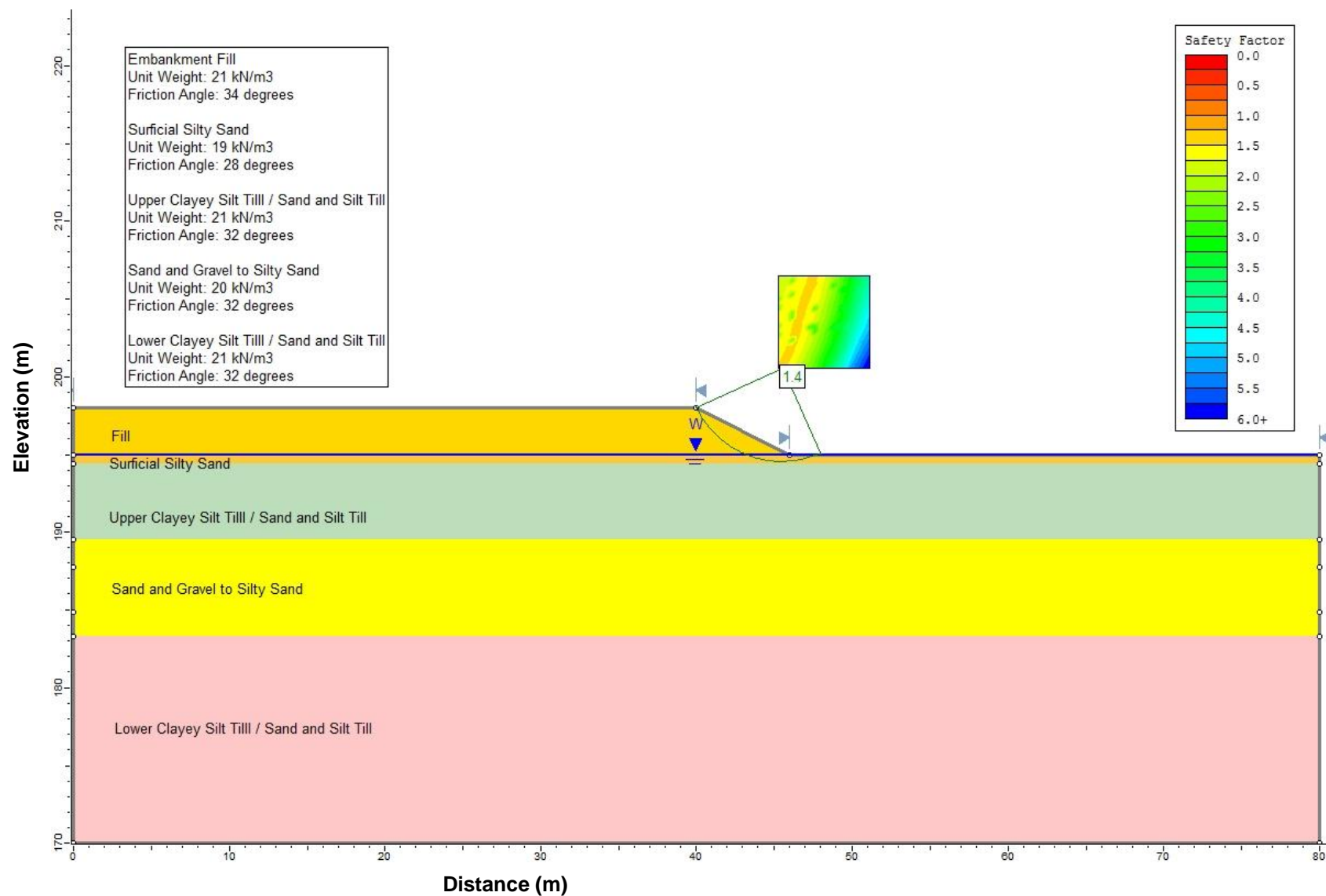
HWY. 401		PROJECT NO. 09-1111-6036		DIST.
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DRAWN: JFC/CD	CHKD. GA	APPD. LCC	DWG. 1	





Static Global Stability – East Oakville Creek (Sixteen Mile Creek East Tributary) Bridge Approach Embankments

Figure 1





**PRELIMINARY FOUNDATION REPORT - EAST OAKVILLE
CREEK (SIXTEEN MILE CREEK EAST TRIBUTARY) BRIDGE**

APPENDIX A

Borehole Records



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
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
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 cm² 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) 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

	kPa	C_u, S_u	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_p	plastic limit
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, 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 ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Percent 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 (cohesionless) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



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
ν	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

(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 = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
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 ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$

PROJECT <u>09-1111-6036</u>		RECORD OF BOREHOLE No 10-801		1 OF 2 METRIC	
W.O. <u>07-20024</u>		LOCATION <u>N 4825432.1 ; E 278457.3</u>		ORIGINATED BY <u>AM</u>	
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>Mini Mole, Solid Stem Augers</u>		COMPILED BY <u>SB</u>	
DATUM <u>Geodetic</u>		DATE <u>February 25, 2011</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 γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
196.5	GROUND SURFACE																
0.0	TOPSOIL																
0.2	Silty sand, trace clay, trace gravel, containing organics (FILL)		1	SS	10		196										
195.9	Compact Brown Moist		2	SS	16												
0.6	Clayey silt, trace sand, trace gravel (FILL) Very stiff to hard Brown Moist		3	SS	50		195										
194.7	CLAYEY SILT with sand, trace gravel, containing cobbles (TILL) Hard Brown Moist	4	SS	72	194												
1.8		5	SS	84													
		6	SS	61	193												
		7	SS	55													
		8	SS	64	192												
191.0								191									
5.5	SAND and SILT, trace clay trace gravel, containing cobbles (TILL) Very dense Brown Moist		9	SS	101/15												
			10	SS	*		189										
187.8								188									
8.7	SAND, trace to some silt, trace gravel, trace clay Dense Grey Wet		11	SS	37	187											
			12	SS	43	186											
184.8							185										
11.7	Silty SAND, trace clay Very dense Grey Wet		13	SS	55	184											
							183										
182.4			14	SS	89		182										
14.1	CLAYEY SILT with sand, trace to some gravel (TILL) Hard Reddish brown Moist																

Continued Next Page

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

MIS-MTO 001 09-1111-6036.GPJ GAL-MISS.GDT 29/6/11 CD

PROJECT 09-1111-6036			RECORD OF BOREHOLE No 10-801				2 OF 2 METRIC									
W.O. 07-20024		LOCATION N 4825432.1 ; E 278457.3				ORIGINATED BY AM										
DIST Central HWY 401		BOREHOLE TYPE Mini Mole, Solid Stem Augers				COMPILED BY SB										
DATUM Geodetic		DATE February 25, 2011				CHECKED BY LCC										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---															
	CLAYEY SILT with sand, trace to some gravel (TILL) Hard Reddish brown Moist		15	SS	114		181									
			16	SS	110/10		180									
			17	AS			179									
178.7 17.8	END OF BOREHOLE AUGER REFUSAL NOTES: * Split-spoon sampler bouncing. 1. High water pressure encountered at a depth of 8.7 m (Elev. 187.8 m) during drilling. Water level in open borehole rose to 1.4 m below ground surface (Elev. 195.1 m).															

PROJECT 09-1111-6036		RECORD OF BOREHOLE No 10-802		1 OF 2 METRIC	
W.O. 07-20024		LOCATION N 4825452.8 ; E 278440.1		ORIGINATED BY AM	
DIST Central HWY 401		BOREHOLE TYPE Mini Mole, Solid Stem Augers		COMPILED BY SB	
DATUM Geodetic		DATE February 24, 2011		CHECKED BY LCC	


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × REMOULDED				W _p	W	W _L					
195.0	GROUND SURFACE							20	40	60	80	100								
0.0	TOPSOIL																			
194.4	Silty SAND, trace clay, trace gravel Loose Brown Moist		1	SS	6															
0.6	CLAYEY SILT with sand, trace gravel (TILL) Very stiff to hard Brown Moist		2	SS	27															
			3	SS	25															
			4	SS	59															
			5	SS	29															
			6	SS	48															
			7	SS	64															
	Becoming grey at a depth of 4.2 m		8	SS	36															
189.5																				
5.5	SAND and GRAVEL, some silt, trace clay Very dense Brown Moist		9	SS	109															
187.8																				
7.2	SAND, trace to some silt, trace clay, trace gravel Dense to very dense Grey wet		10	SS	40															
			11	SS	59															
184.9																				
10.1	Silty SAND, trace clay Very dense Grey Wet		12	SS	92															
183.3																				
11.7	CLAYEY SILT with sand, to SAND and SILT, trace to some clay, trace to some gravel (TILL) Hard/ Very dense Reddish brown Moist		13	SS	122															
			14	SS	110/ 15															
		</																		

Continued Next Page

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

MIS-MTO 001 09-1111-6036.GPJ GAL-MISS.GDT 29/6/11 CD

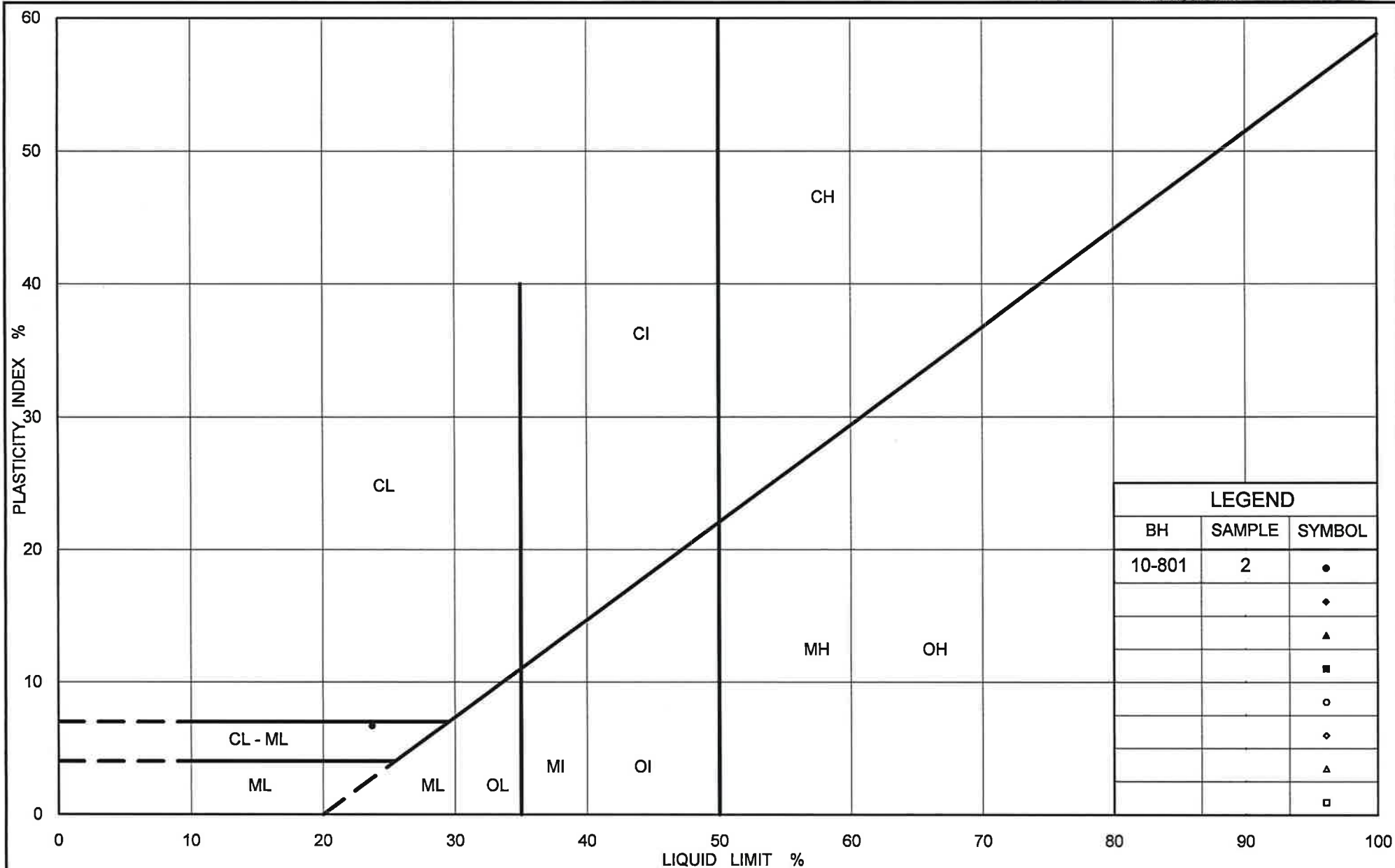
PROJECT <u>09-1111-6036</u>		RECORD OF BOREHOLE No 10-802				2 OF 2 METRIC	
W.O. <u>07-20024</u>		LOCATION <u>N 4825452.8 ; E 278440.1</u>				ORIGINATED BY <u>AM</u>	
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>Mini Mole, Solid Stem Augers</u>				COMPILED BY <u>SB</u>	
DATUM <u>Geodetic</u>		DATE <u>February 24, 2011</u>				CHECKED BY <u>LCC</u>	

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W		
	--- CONTINUED FROM PREVIOUS PAGE ---															
179.7 15.3	END OF BOREHOLE NOTE: 1. High water pressure encountered at a depth of 5.5 m (Elev. 189.5 m). Water level in open borehole rose to the ground surface (Elev. 195.0 m).		15	SS	110/10											



APPENDIX B

Laboratory Test Results



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PLASTICITY CHART Clayey Silt Fill

Figure No. B1

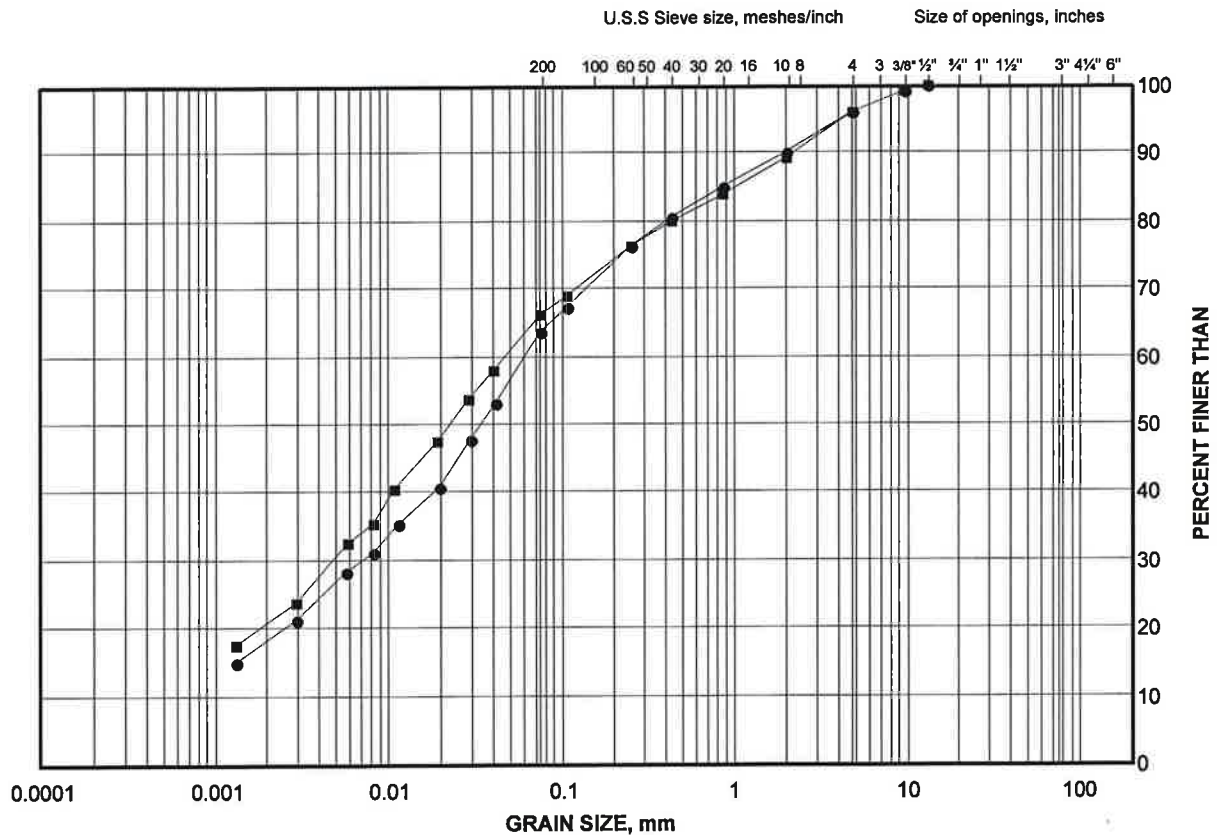
Project No. 09-1111-6036

Checked By: *Wazir*

GRAIN SIZE DISTRIBUTION TEST RESULTS

Upper Clayey Silt Till

FIGURE B2



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

LEGEND

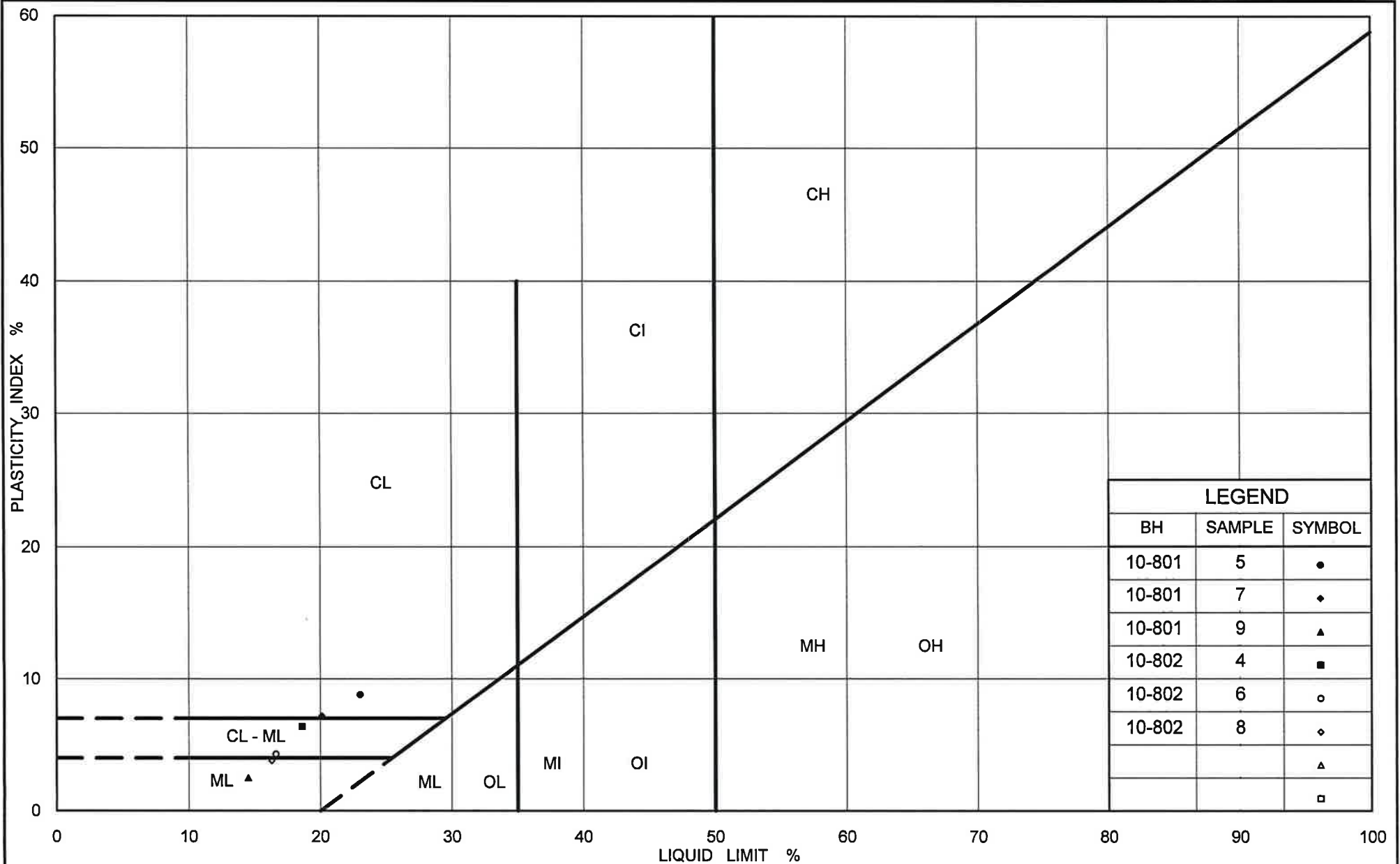
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-802	4	192.8
■	10-801	5	193.7

Project Number: 09-1111-6036

Checked By: *Wayne*

Golder Associates

Date: 24-Jun-11



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PLASTICITY CHART Upper Clayey Silt Till to Sand and Silt Till

Figure No. B3

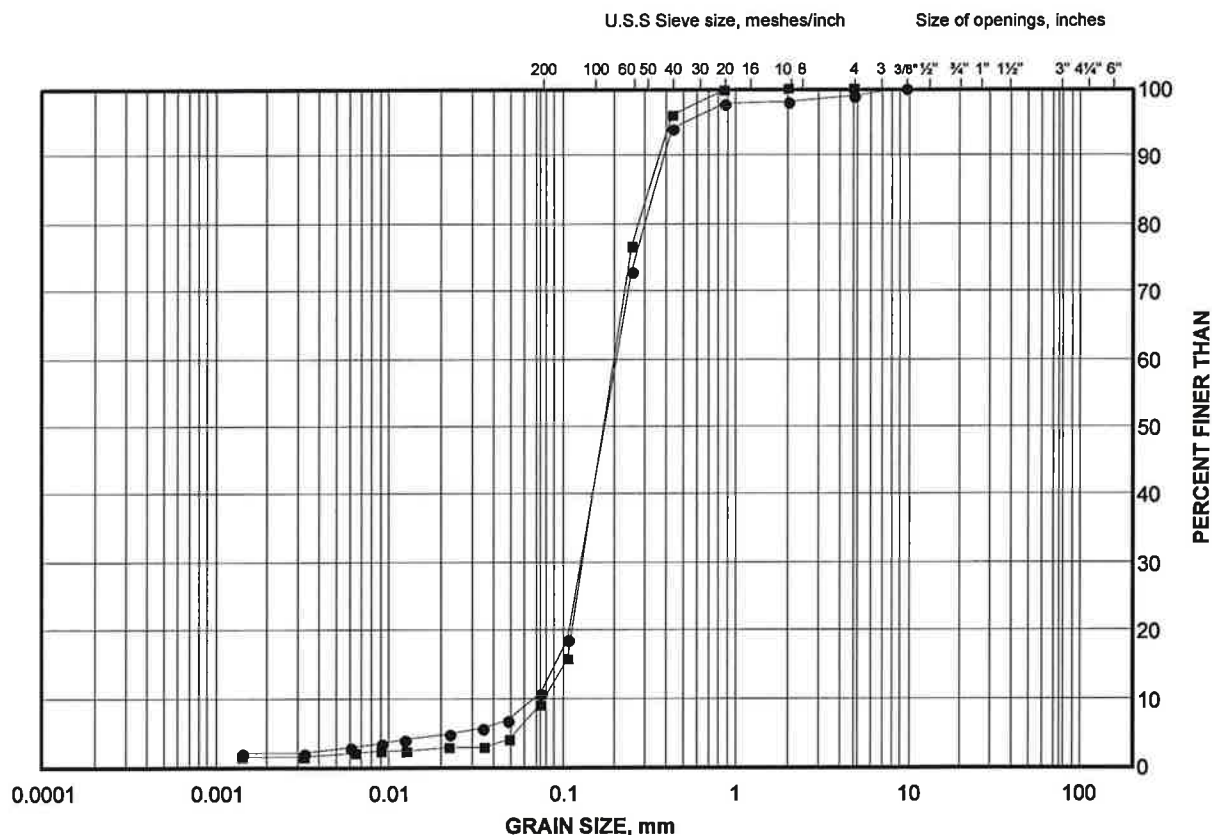
Project No. 09-1111-6036

Checked By: *Wagner*

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand

FIGURE B4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-802	10	187.1
■	10-801	12	185.5

Project Number: 09-1111-6036

Checked By: *Wayne*

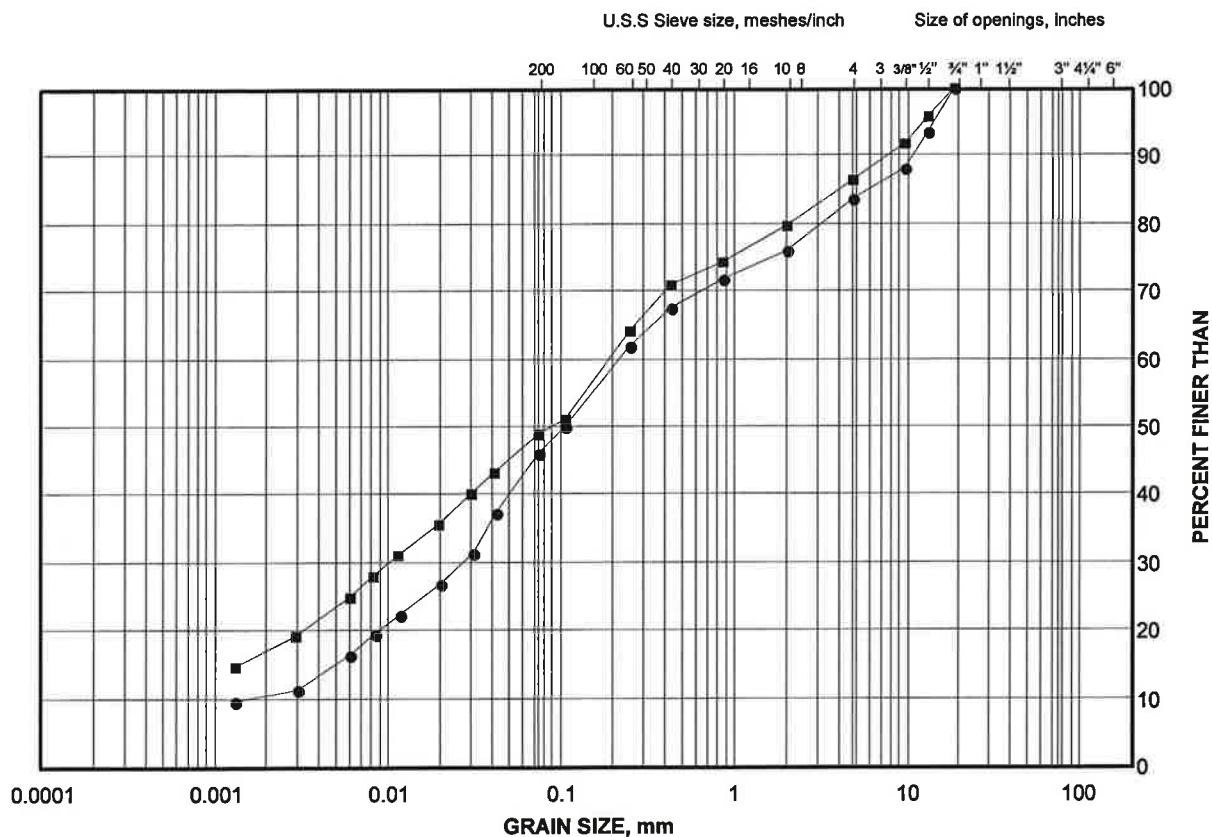
Golder Associates

Date: 24-Jun-11

GRAIN SIZE DISTRIBUTION TEST RESULTS

Lower Clayey Silt Till to Sand and Silt Till

FIGURE B5



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

LEGEND

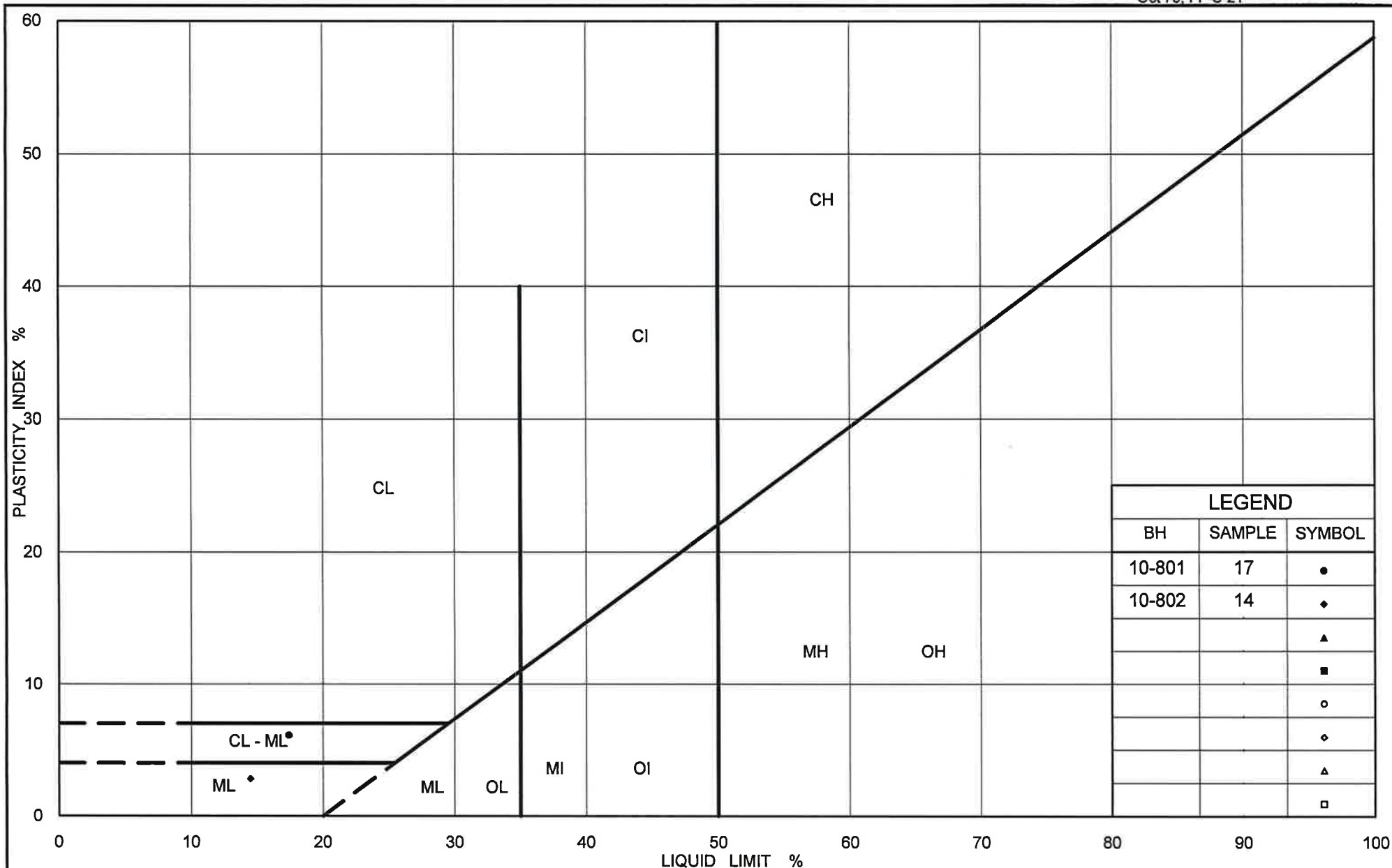
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-802	14	181.2
■	10-801	17	179.7

Project Number: 09-1111-6036

Checked By: *[Signature]*

Golder Associates

Date: 24-Jun-11



Ministry of Transportation

Ontario

PLASTICITY CHART Lower Clayey Silt Till to Sand and Silt Till

Figure No. B6

Project No. 09-1111-6036

Checked By: *Woye*

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
2390 Argentia Road
Mississauga, Ontario, L5N 5Z7
Canada
T: +1 (905) 567 4444

