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REPORT



PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Oakville Creek West Branch (Sixteen Mile Creek Middle Branch) Bridge Highway 401 Widening from Trafalgar Road to Regional Road 25, Halton Region W.O. 07-20024

Submitted to:
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BRANCH (SIXTEEN MILE CREEK MIDDLE BRANCH) BRIDGE**

PART A

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



PRELIMINARY FOUNDATION REPORT - OAKVILLE CREEK WEST BRANCH (SIXTEEN MILE CREEK MIDDLE BRANCH) BRIDGE

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 or replacement of the existing Oakville Creek West Branch (Sixteen Mile Creek Middle Branch) bridge.

The terms of reference and scope of work for the foundation engineering services are outlined in MTO's Request for Proposal (RFP) for Assignment No. 2008-E-0027 dated April 2009, and in Section 5.8 of the *Technical Proposal* for this assignment.

2.0 SITE DESCRIPTION

The Oakville Creek West Branch (Sixteen Mile Creek Middle Branch) bridge is located approximately 50 m to 75 m east of the Highway 401-5th Line overpass – about 5 km east of Regional Road 25 and 3 km west of Trafalgar Road – in the Regional Municipality of Halton, Ontario. The existing structure consists of a 40 m long, 24.5 m wide single span bridge, built in 1958, that is 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 193 m to 195 m. The creek channel is at approximately Elevation 191.5 m, with a typical water depth of about 0.7 m (Elevation 192.2 m). It is understood that the high water level at this creek site is at approximately Elevation 194.0 m.

Highway 401 has been constructed in embankment fill that is approximately 5.5 m to 7 m in height, with the pavement grade at about Elevation 201.1 m to 201.5 m at the structure site. The Highway 401 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 November and December 2010, during which time two boreholes (Boreholes 10-401 and 10-402) were advanced using track-mounted and truck-mounted CME-75 drill rigs, respectively, supplied and operated by Geo-Environmental Drilling Inc. of Milton, Ontario. The borehole locations are shown on Drawing 1: Borehole 10-401 was advanced adjacent to the southwest abutment at the highway embankment toe, and Borehole 10-402 was advanced in the northeast quadrant of the structure site through the Highway 401 embankment.

Boreholes 10-401 and 10-402 were drilled using 108 mm and 70 mm inside diameter hollow stem augers to depths of 18.3 m and 20.7 m, respectively. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth in the boreholes, using a 50 mm outside diameter split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedure.



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The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations and two standpipe piezometers (one shallow and one deep) were installed in Borehole 10-401 to permit monitoring of the groundwater levels at the site; the piezometer installation details are shown on the borehole record contained in Appendix A. The piezometers consisted of 50 mm diameter PVC pipe, with a slotted screen sealed within a sand filter pack at selected depth intervals within the borehole. Above the sand filter packs, the annulus surrounding the piezometer pipes was backfilled to the ground surface with bentonite. Borehole 10-402 was backfilled with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended by Ontario Regulation 372).

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, and 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 geotechnical laboratory testing was completed according to applicable MTO LS procedures.

The borehole locations were measured in the field by Golder personnel relative to site features and using a hand-held global position system (GPS) unit with a horizontal accuracy of 0.3 m. The ground surface elevation at each borehole was determined from the digital terrain model for the site. 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)
10-401	4,823,614.1	276,498.5	193.2	18.3
10-402	4,823,662.1	276,469.5	201.2	20.7

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 in the vicinity of Oakville Creek West Branch (Sixteen Mile Creek Middle Branch) bridge. 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 given on the Record of Borehole sheets contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B4 contained 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 the site consist of embankment fill overlying a thin surficial deposit of silty clay and an extensive deposit of stiff to hard clayey silt till. The till is underlain by a deposit of sand and gravel, in turn underlain by shale bedrock. 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 of topsoil was encountered immediately below the existing ground surface in Borehole 10-401, which is located beside the southwest abutment near the highway embankment toe.

4.2.2 Fill

Approximately 0.5 m of asphalt and concrete and 5.1 m of fill associated with the existing Highway 401 embankment was encountered in Borehole 10-402, extending to approximately Elevation 195.6 m.

The fill consists of sand and gravel containing some silt and trace clay. The result of a grain size distribution test completed on one selected sample of the fill is shown on Figure B1 in Appendix B.

The Standard Penetration Test (SPT) “N” values measured within the fill range from 22 blows to 45 blows per 0.3 m of penetration, indicating that the sand and gravel fill has a compact to dense relative density.

4.2.3 Surficial Silty Clay

A 1.4 m thick surficial deposit of brown silty clay was encountered immediately below the topsoil in Borehole 10-401, extending to Elevation 191.8 m. The deposit consists of silty clay with sand, containing trace gravel, as well as small quantities of rootlets and organic matter.

The measured SPT “N” values within the silty clay are 5 blows and 12 blows per 0.3 m of penetration, suggesting a firm to stiff consistency.



4.2.4 Clayey Silt Till Including Sand and Silt Till Interlayer

An 11.9 m to 15.1 m thick till deposit was encountered below the surficial silty clay in Borehole 10-401 in the southwest quadrant of the structure site, and below the embankment fill in Borehole 10-402 in the northeast quadrant of the structure site. The surface of the deposit was encountered at Elevation 191.8 m and 195.6 m in Boreholes 10-401 and 10-402, respectively, and its base was encountered at Elevation 179.9 m and 180.5 m.

The till is comprised predominantly of clayey silt with sand to some sand, containing trace to some gravel. In Borehole 10-401, a 1.5 m thick zone or interlayer of less plastic sand and silt till was encountered within the deposit, between approximately Elevation 183.0 m and 181.5 m (at a depth of about 10.2 m to 11.7 m). The results of grain size distribution tests completed on four selected samples of the clayey silt till and one sample from the sand and silt till zone are shown on Figure B2 in Appendix B.

Atterberg limits testing was completed on eight selected samples of the till deposit and measured plastic limits ranging from 12 per cent to 16 per cent, liquid limits ranging from 18 per cent to 25 per cent, and plasticity indices ranging from 5 per cent to 10 per cent. These results, which are plotted on a plasticity chart on Figure B3 in Appendix B, indicate that the tested portions of the deposit consist of clayey silt of low plasticity.

The measured SPT “N” values within this deposit range from 13 blows to greater than 100 blows per 0.3 m of penetration, suggesting that the clayey silt till has a stiff to hard (but generally very stiff to hard) consistency. One SPT “N” value of 39 blows per 0.3 m of penetration was measured in the sand and silt till layer, indicating that this portion of the till deposit has a dense relative density.

4.2.5 Sand and Gravel

A 2.6 m thick deposit of sand and gravel was encountered below the till in Borehole 10-401 in the southwest quadrant of the structure site, between approximately Elevation 179.9 m and 177.3 m.

The sand and gravel deposit contains trace to some silt and trace clay. The result of a grain size distribution test completed on one selected sample of this deposit is shown on Figure B4 in Appendix B.

The measured SPT “N” values are 12 blows per 0.3 m of penetration and 100 blows per 0.05 m of penetration, indicating a compact to very dense relative density.

4.2.6 Shale Bedrock

Red-brown shale was encountered below the sand and gravel deposit in Borehole 10-401, and was inferred below the clayey silt till deposit in Borehole 10-402, as summarized below. The bedrock surface is approximately 11 m to 14 m below the base of the creek channel, and about 21 m to 24 m below the Highway 401 grade at this site.



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Borehole No.	Approximate Bedrock Surface Elevation	Comments
10-401	177.3 m	Shale bedrock was penetrated for a thickness of 2.4 m by augering and split-spoon sampling, with measured SPT "N" values of 100 blows per 0.05 m to 0.08 m of penetration.
10-402	180.5 m	Inferred based on auger refusal (smooth grinding).

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 contained in Appendix A. In general, the cohesionless (sand and silt) portion of the till deposit and the lower sand and gravel deposit were observed to be water-bearing during the drilling operations, and the clayey silt till samples were also observed to be wet.

Two standpipe piezometers were installed in Borehole 10-401 to monitor the groundwater level(s) at the site: one shallow piezometer within the clayey silt till, and one deeper piezometer installed at the interface of the sand and gravel and shale bedrock. The water levels measured in the piezometers are summarized in the following table:

Date	Shallow Piezometer		Deep Piezometer	
	Depth	Elevation	Depth	Elevation
December 15, 2010	1.3 m	191.9 m	0.3 m	192.9 m
February 3, 2011	1.3 m	191.9 m	0.2 m	193.0 m
April 21, 2011	0.7 m	192.5 m	0.1 m above ground surface	193.3 m

The groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.



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5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Alex Mayot, 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.

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PART B

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
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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 and/or replacement of the existing Oakville Creek West Branch (Sixteen Mile Creek Middle Branch) bridge. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this subsurface investigation. The discussion and preliminary 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 construction aspects 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

Based on the planning study completed to date for the widening of Highway 401 from Regional Road 25 to Trafalgar Road in the Regional Municipality of Halton, it is understood that the future widening could consist of three additional lanes in both the eastbound and westbound directions on Highway 401. The existing Oakville Creek West Branch (Sixteen Mile Creek Middle Branch) bridge will require widening to the north and south; full replacement of the existing bridge is also under consideration.

The existing bridge consists of a 40 m long by 24.5 m wide single-span structure, with the existing abutments supported on spread footings. Based on the *General Layout* drawing for this structure, prepared by M.M. Dillon & Co. Ltd. Consulting Engineers in 1958, the footings are about 3 m wide and are founded at approximately Elevation 189.0 m (620.0 ft.). This founding elevation is about 2.5 m below the bottom of the creek channel, and more than 4 m below the natural ground surface at the 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 north and south widening and/or replacement of the Oakville Creek West Branch (Sixteen Mile Creek Middle Branch) 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 relative costs is provided in Table 1 following the text of this report.

- **Strip or spread footings founded on the very stiff to hard clayey silt till:** Strip or spread footings are feasible for support of the widened abutments and associated wing walls/retaining walls, provided they are founded below the firm to stiff surficial silty clay and the stiff portion of the till deposit, on very stiff to hard clayey silt till. This would require excavation to a depth of approximately 2.5 m to 3 m relative to the original ground surface; temporary excavation support would likely be required both adjacent to Highway 401 and adjacent to the creek channel. There is potential for up to approximately 15 mm of differential settlement between the existing structure and the new widened portions.



- **Footings “perched” on a compacted granular pad in the widened Highway 401 approach embankments:** Although this option would be advantageous in minimizing the depth of excavation, “perched” footings are not recommended for support of the north and south bridge widening due to the potential settlement associated with the firm to stiff surficial silty clay and the stiff portion of the clayey silt till deposit under the widened highway approach embankment loading.
- **Driven steel H-piles:** Driven steel H-piles are feasible and suitable for support of the abutment widening or for a full structure replacement. In the case of widening, the differential settlement between the existing spread footing-supported core structure and the widening would be negligible. Steel H-pile foundations would also allow for the construction of integral abutments, if the existing structure can be modified to be compatible with integral abutments for the widening, or if a full structure replacement is adopted. There is a minor risk associated with the piles penetrating through or “hanging up” on cobbles or boulders within the glacial soils; the use of driving shoes is recommended due to the hard/very dense nature of the soils and the potential presence of cobbles and boulders within the soil deposits.
- **Driven steel pipe (tube) piles:** Steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutment widening and any associated wing walls/retaining walls. However, pipe piles are considered to have a higher risk than H-piles for “hanging up” or being 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.
- **Caissons:** Caissons are not considered to be a practical option at this site given the potential risks and difficulties associated with the high groundwater pressures in the lower sand and gravel deposit.

The following sections provide recommendations for spread footing foundations, driven steel H-pile and steel pipe pile foundations to support the proposed bridge widening or replacement. However, based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments for the north and south widening on driven steel H-pile or pipe (tube) pile foundations, to minimize differential settlement between the existing and new structures, and to minimize the depth of excavation as compared with a spread footing option.

6.3 Shallow Foundations

6.3.1 Founding Elevations

For support of widened or new abutments and associated wing walls or retaining walls, strip or spread footings should be founded below the fill and any soft to stiff silty clay or clayey silt till soils, on the very stiff to hard clayey silt till deposit. The table on the following pages summarizes the maximum (highest) founding elevations recommended for preliminary design of footings founded on the clayey silt till deposit, based on the results from Boreholes 10-401 and 10-402.

If spread footings are used for support of structure widening only, it is likely that the footings will be constructed to match the existing foundation elevation (approximately Elevation 189.0 m) and structurally connected to the existing footings. In the case of a full structure replacement, it may be feasible to found the footings



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approximately 1.5 m to 3.5 m higher than the existing foundations, consistent with the recommendations provided in the following table.

Further assessment is recommended following additional investigation during the detail design stage to confirm the near-surface soil conditions and founding elevations. In addition, these recommended founding levels should be assessed relative to the potential scour depth for the hydraulic conditions in the creek, and deepened if necessary to provide adequate protection against scour. Finally, further assessment of the impact of the existing foundations and their removal (if full structure replacement is adopted) will be required at detail design, once the geometry and span of the replacement structure is established.

Foundation Element	Maximum (Highest) Founding Elevation	Approximate Excavation Depth
North Widening or North Portion of Replacement	192.5 m (or lower to match existing)	3.0 m – relative to original ground
South Widening or South Portion of Replacement	190.5 m (or lower to match existing)	2.7 m – relative to original ground

Alternatively, subexcavation can be carried out to the elevations identified in the table above, then backfilled with compacted Ontario Provincial Standard Specification (OPSS) 1010 Granular A or Granular B Type II fill prior to construction of the footings at a higher elevation. In this case, the founding elevation for the new footings should be a minimum of 1.2 m below the lowest surrounding grade, to provide adequate protection against frost penetration, per Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*). The compacted granular pad should extend at least 1 m beyond the front and back edge of the new abutment footings, then outward and downward at 1H:1V. The granular fill should be placed in accordance with OPSS 501 (*Compacting*).

The footing subgrade should be inspected by the Quality Verification Engineer following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to check that all existing fill, firm to stiff cohesive soils or other unsuitable material have been removed. The founding soils will be susceptible to disturbance. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a 100 mm thick concrete working slab (of 20 MPa compressive strength concrete) be placed on the prepared subgrade within four hours of its inspection and approval.

6.3.2 Geotechnical Resistance

Strip or spread footings placed on the properly prepared, very stiff to hard clayey silt till (or on compacted granular fill following subexcavation of the firm to stiff cohesive soils), 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 resistances at Serviceability Limit States (SLS) given below.



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Foundation Element	Footing Width	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS*
North Widening or North Portion of Replacement	3 m	600 kPa	450 kPa
	4 m	650 kPa	400 kPa
South Widening or South Portion of Replacement	3 m	600 kPa	450 kPa
	4 m	650 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 abutment widening locations.

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

6.4.1 Founding Elevations

The widened or new abutments and any associated wing walls/retaining walls may be supported on steel H-piles or steel pipe (tube) piles driven to found on or within the shale bedrock. The following pile tip elevations may be used for preliminary design purposes, assuming about 0.5 m to 1 m of penetration into the shale to account for weathering. Further investigation will be required during detail design to assess the degree of weathering of the upper portion of the shale bedrock and refine these pile tip elevations.

Foundation Element	Estimated Design Pile Tip Elevation
North Widening or North Portion of Replacement	180.0 m
South Widening or South Portion of Replacement	176.5 m

The pile caps should be constructed at a minimum depth of 1.2 m for frost protection purposes, per OPSD 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 deposits. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a 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



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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 bouldery soils, as are anticipated at this site, driving 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 lower sand and gravel deposit was observed to be slightly artesian relative to the natural 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 or an impermeable plug to prevent loss of fine soil particles along the pile-soil interface.

6.4.2 Axial Geotechnical Resistance

For HP 310x310 piles driven to the estimated tip elevations provided in Section 6.4.1, the factored axial resistance at ULS may be taken as 1,600 kN, and the axial geotechnical resistance at SLS (for 25 mm of settlement) may be taken as 1,400 kN. Similar axial resistances may be used in the design for 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 at the foundation elements.

6.5 Approach Embankments

6.5.1 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 widened Highway 401 embankments. The depth and extent of stripping should be assessed during detail design when additional subsurface information will be available for the widened approach embankment areas.

The embankment fill for the Highway 401 widening should be placed and compacted in accordance with MTO's Special Provision 206S03 (*Earth Excavation and Grading*) and 105S10 (*Amendment to OPSS 501 – Construction Specification for Compaction*). Benching of the existing Highway 401 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 Highway 401 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.

In accordance with MTO's standard practice, a minimum 2 m wide bench should be provided where the embankment side slopes are equal to or greater than 8 m in height, such that the uninterrupted slope height does not exceed 8 m. 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 (*Construction Specification for Seed and Cover*).

6.5.2 Approach Embankment Stability

Slope stability analyses have been performed for the proposed Highway 401 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 northward and southward embankment widening on this project, considering the design requirements and the available field and laboratory testing data.

The stability analyses were completed for an approximately 7 m high embankment widening based on the subsurface conditions as encountered in Boreholes 10-401 and 10-402. The following parameters have been used in the analyses, based on field and laboratory test data as well as accepted correlations:

Soil Conditions	Bulk Unit Weight (kN/m ³)	Effective Friction Angle	Undrained Shear Strength (kPa)
Embankment fill	21	32-35°	-
Firm to stiff surficial silty clay	20	28°	-
Very stiff to hard clayey silt till	21	32°	-
Dense to very dense sand and gravel	21	34°	-

The analysis results indicate that an approximately 7 m high embankment widening with side slopes oriented no steeper than 2H:1V will have a factor of safety of at least 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 widened Highway 401 embankment footprints during detail design.



6.5.3 Approach Embankment Settlement

Based on the study completed to date, the existing 5.5 m to 7 m high Highway 401 embankment will require widening by approximately 20 m on both the north and south sides in this area.

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 Conditions	Bulk Unit Weight (kN/m³)	Elastic Modulus (MPa)
Embankment fill	21	-
Firm to stiff silty clay	20	15 MPa
Very stiff to hard clayey silt till	21	60 MPa
Dense to very dense sand and gravel	21	75 MPa

Based on this preliminary assessment, the settlement of the foundation soils under the widened 5.5 m to 7 m high approach embankments is estimated to be approximately 25 mm to 40 mm. 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 new 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.



6.6 Construction Considerations

The following subsections identify future construction considerations 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.

6.6.1 Excavation and Temporary Protection Systems

The foundation excavations for spread footings would extend to a depth of 2.5 m to 3 m below the natural ground surface at the site, through existing fill, the firm to stiff surficial silty clay and stiff clayey silt till; there is also potential to encounter water-bearing alluvial soils associated with the former meander channel of the creek. The excavations for abutment pile caps could be maintained higher than those for spread footings.

Where space permits at the abutment widening locations, 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 firm to stiff cohesive soils are 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 roadway protection would likely be required along the north and south sides of Highway 401 to facilitate the removal of the existing wing walls/retaining walls adjacent to the abutments and excavation to foundation level for the abutment widening, as well as along the creek depending on the depth of excavation. 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, provided that the existing bridge, as well as any adjacent utilities, can tolerate this magnitude of deformation.

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 and in areas where the foundation excavations intersect alluvial soils associated with the former creek meander channel. For a soldier pile and lagging system, it would be necessary to control seepage or include measures to mitigate loss of soil particles from the surficial silty sand deposit through the lagging boards.

6.6.2 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.



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There is a risk that foundation excavations for the abutment widening will intersect water-bearing alluvial soils or fill materials associated with the former 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.

6.6.3 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.

6.6.4 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.

6.6.5 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 for the existing bridge structure is not expected to be required during construction at this site.

6.7 Recommendations for Further Work in Detail Design

Additional boreholes will be required within each of the abutment widening areas 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:



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■ Abutments:

- Assessment of the variability and distribution of the surficial silty clay and “softened” clayey silt till as well as the strength and deformation properties of the upper portion of the clayey silt till, to confirm the founding elevation and bearing resistances for spread footings within each widened abutment area.
- Assessment of the presence of alluvial soils (from the former meander channel prior to straightening of the creek during construction of the existing structure) within or near the footprint of foundation excavations, to assess potential for groundwater seepage and subexcavation requirements.
- Confirmation of the tip elevation for driven steel H-piles or steel pipe (tube) piles, based on bedrock coring to assess the weathering characteristics of the shale bedrock.
- 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, the installation of driven steel H-pile or steel pipe pile foundations, and the installation of temporary protection systems.
- Confirmation of the groundwater elevation associated with the lower sand and gravel deposit/bedrock. Due to the high (slightly artesian) groundwater levels, the use of the vibrating wire piezometers or pressure transducers may be appropriate.

■ Approach embankments and adjacent high fill embankments:

- Assessment of the depth and extent of stripping of topsoil/organics and fill materials within the footprint of the widened Highway 401 approach embankments.
- Further assessment of the thickness, strength and consolidation/elastic compression properties of the surficial soils within the footprint of the widened approach embankments, to confirm the stability analyses and settlement estimates.
- Additional boreholes are recommended to the east of the approach embankments if the widened Highway 401 embankments will be greater than 4.5 m in height, to assess the need for any stripping/subexcavation, as well as to complete stability and settlement analyses.




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7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Alex Mayot, EIT, 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 senior geotechnical/foundations specialist with Golder. Mr. Ty Garde, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

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AM/LCC/TJG/am/lcc

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REFERENCES

- Bowles, J.E., 1984. *Physical and Geotechnical Properties of Soils*, Second Edition. McGraw Hill Book Company, New York.
- Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual*, 4th 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*, 3rd 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.
- Ontario Geological Society, 1991. *Geology of Ontario*. Special Volume 4, Part 1. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.
- Peck, R.B., Hanson, W.E., and Thornburn, T.H., 1974. *Foundation Engineering*, Second Edition, John Wiley and Sons, New York.

Ontario Provincial Standard Specifications (OPSS)

- | | |
|-----------|--|
| OPSS 501 | Construction Specification for Compacting |
| 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 |



PRELIMINARY FOUNDATION REPORT - OAKVILLE CREEK WEST BRANCH (SIXTEEN MILE CREEK MIDDLE BRANCH) BRIDGE

TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Spread/strip footings on very stiff to hard 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 cohesive till deposit Allows for semi-integral abutments 	<ul style="list-style-type: none"> Significant excavations (to a depth of 2.5 m to 3 m below the original ground surface) to extend below firm to stiff soils; would require temporary excavation support Moderate potential for up to about 15 mm of differential settlement between existing bridge and widening 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 \$100,000 per foundation area (i.e., southeast abutment, southwest abutment, etc.) based on an estimated \$600/m³ for construction of shallow foundations, excluding deeper excavation and temporary protection system
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, significantly reducing subexcavation depth and associated temporary protection system requirements 	<ul style="list-style-type: none"> 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 the firm to stiff soils under the approach embankment loading 	<ul style="list-style-type: none"> Conventional excavation and construction techniques 	<ul style="list-style-type: none"> This option not recommended



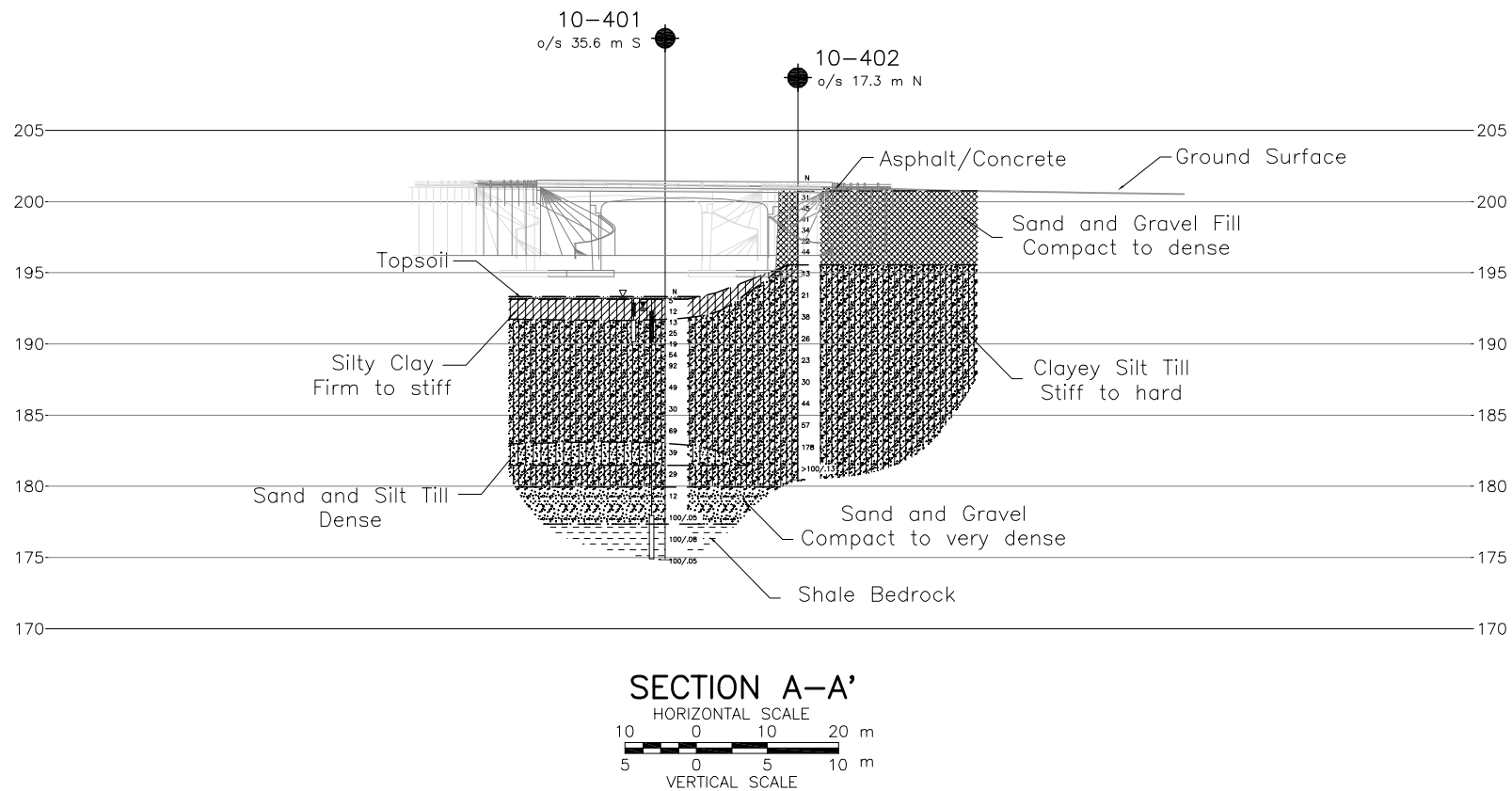
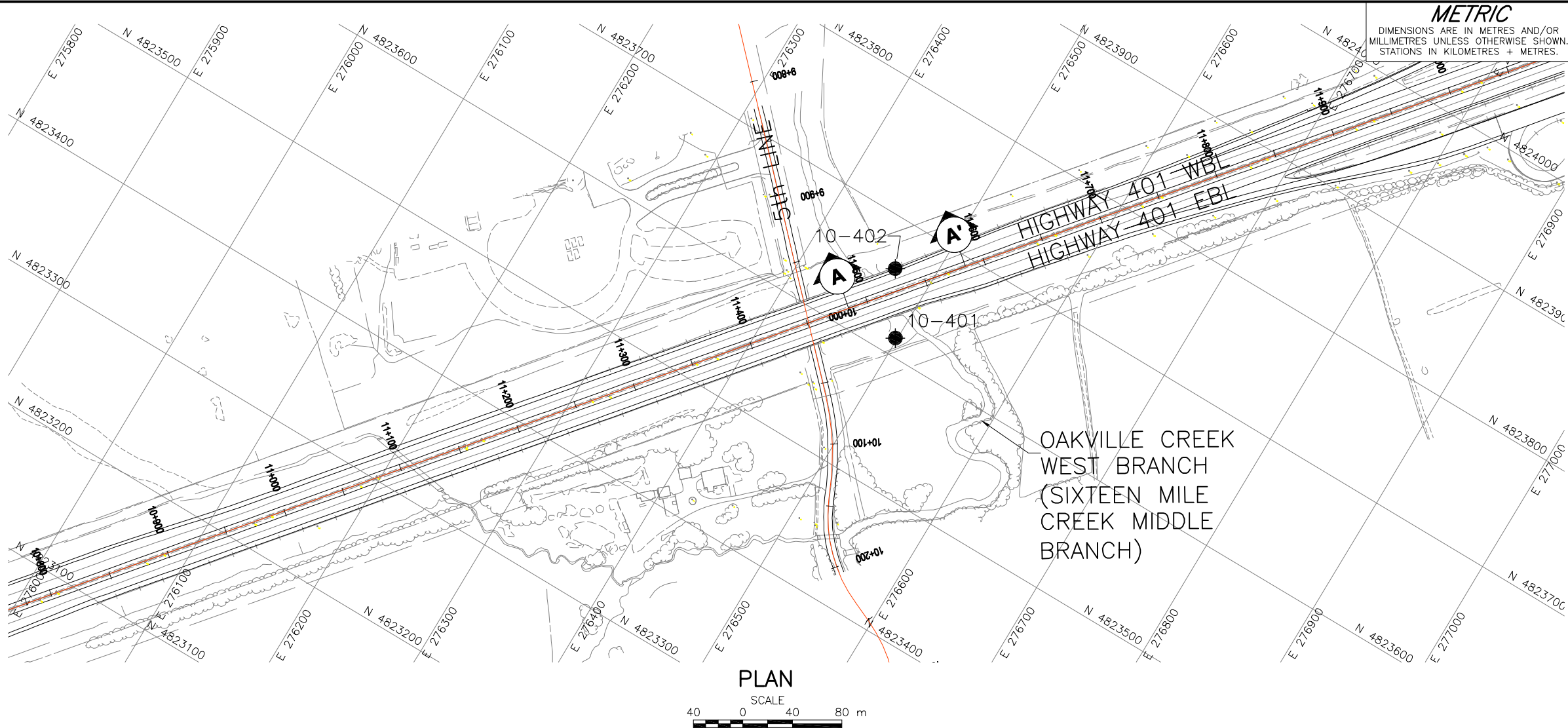
PRELIMINARY FOUNDATION REPORT - OAKVILLE CREEK WEST BRANCH (SIXTEEN MILE CREEK MIDDLE BRANCH) BRIDGE

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel H-piles driven to found on/in shale bedrock	<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 excavation support requirements Limited groundwater control required Allows for integral abutment construction Would minimize differential settlement between existing bridge and widened portions of structure 	<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 	<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 \$125,000 each for the northwest and northeast abutment areas, and \$140,000 each for the southwest and southeast abutment areas, based on an estimated \$250/m length for pile installation and \$600/m³ for pile cap construction
Steel pipe piles (closed-end and concrete filled) driven to found on/in shale bedrock	<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 Limited groundwater control required Allows for semi-integral abutment configuration Would minimize differential settlement between existing bridge and widened portions of structure 	<ul style="list-style-type: none"> Slightly greater risk than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving; this could result in piles "hanging up" and lower geotechnical resistances 	<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



PRELIMINARY FOUNDATION REPORT - OAKVILLE CREEK WEST BRANCH (SIXTEEN MILE CREEK MIDDLE BRANCH) BRIDGE

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Caissons founded on/in shale bedrock	<ul style="list-style-type: none">• Feasible but not recommended for support of 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 excavation support requirements• Higher capacity than for driven piles, so reduced number of deep foundation elements compared to driven piles	<ul style="list-style-type: none">• High groundwater pressures create potential for disturbance or loss of ground in lower sand and gravel deposit• 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 near base of caissons	<ul style="list-style-type: none">• Higher cost compared with shallow foundations or steel H-piles

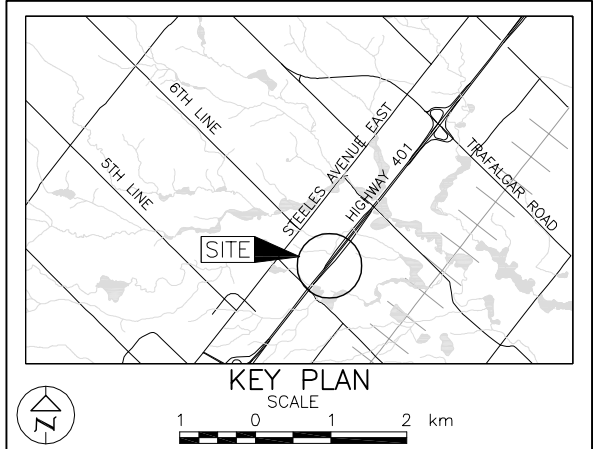


CONT No.
WO No. 07-20024

OAKVILLE CREEK WEST BRANCH (SIXTEEN
MILE CREEK MIDDLE BRANCH) BRIDGE
HIGHWAY 401 WIDENING
BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND			
	Borehole - Current Investigation		
N	Standard Penetration Test Value		
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)		
	WL upon completion of drilling		
	WL in piezometer, measured on April 21, 2011		

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
10-401	193.2	4823614.1	276498.5
10-402	201.2	4823662.1	276469.5

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 design configuration as shown elsewhere in the Preliminary Design Project.

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.

REFERENCE

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.

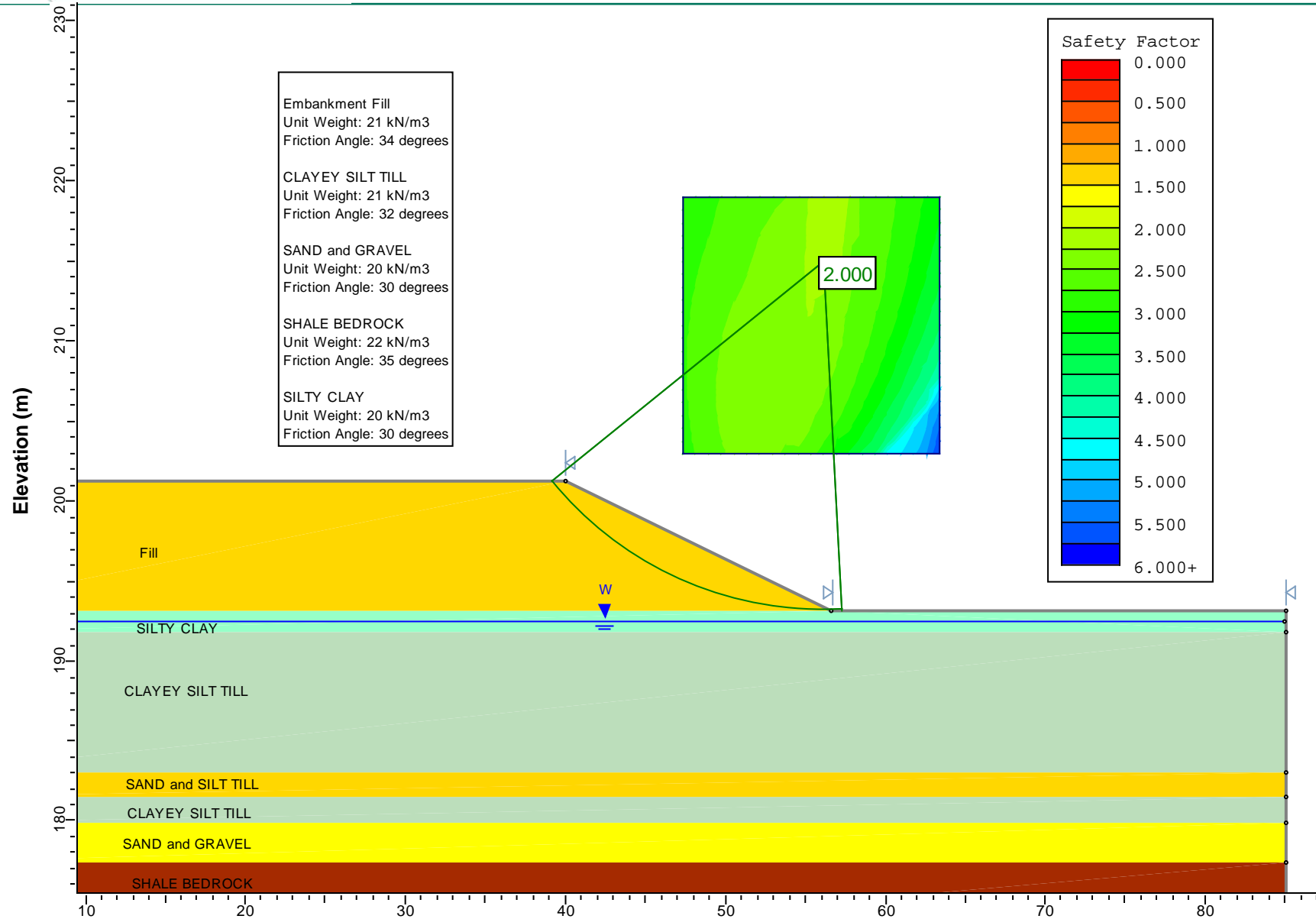


NO.	DATE	BY	REVISION
Geocres No. 30M12-328			
HWY. 401	PROJECT NO. 09-1111-6036		DIST.
SUBM'D. AM	CHKD. LCC	DATE: 10/21/2011	SITE:
DRAWN: JFC/CD	CHKD. AM	APPD. LCC	DWG. 1



Static Global Stability – West Branch Oakville Creek Embankment Widening

Figure 1





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

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

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.)

(b) Cohesive Soils Consistency

	Cu, Su	psf
	kPa	
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

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.

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.



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
F	factor of safety
V	volume
W	weight

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 ($\sigma' = \sigma - \mu$)
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
μ	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	liquid limit
w_p	plastic limit
I_p	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_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

T_p, T_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 $\tau = c' + \sigma' \tan \phi'$
2 shear strength = (compressive strength)/2

PROJECT <u>09-1111-6036</u>		RECORD OF BOREHOLE No 10-401		1 OF 2 METRIC	
W.O. <u>07-20024</u>		LOCATION <u>N 4823614.1 ; E 276498.5</u>		ORIGINATED BY <u>MS</u>	
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>Track-Mounted CME 55, 108 mm Internal Diameter Hollow Stem Augers</u>		COMPILED BY <u>CS</u>	
DATUM <u>Geodetic</u>		DATE <u>December 15, 2010</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		WATER CONTENT (%)				
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
193.2	GROUND SURFACE													
0.9	TOPSOIL													
	SILTY CLAY with sand, trace gravel, containing rootlets, and organics Firm to stiff Brown Moist		1	SS	5									
			2	SS	12									
191.8														
1.5	CLAYEY SILT with sand to some sand, trace to some gravel (TILL) Stiff to hard Brown to grey Moist becoming wet below a depth of 2.3 m		3	SS	13									
			4	SS	25									
			5	SS	19									
			6	SS	54									
			7	SS	92									
			8	SS	49									
			9	SS	30									
			10	SS	69									
183.0														
10.2	SAND and SILT, some gravel, trace clay (TILL) Dense Brown Wet		11	SS	39									
181.5														
11.7	CLAYEY SILT, some sand and gravel (TILL) Very stiff Brown Wet		12	SS	29									
179.9														
13.3	SAND and GRAVEL, trace to some silt, trace clay Compact to very dense Brown Wet		13	SS	12									

Continued Next Page

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

MIS-MTO-001 09-1111-6036.GPJ GAL-MISS.GDT 6/20/11 CD

PROJECT		RECORD OF BOREHOLE No 10-401				2 OF 2 METRIC						
W.O.		LOCATION				ORIGINATED BY						
DIST		BOREHOLE TYPE				COMPILED BY						
DATUM		DATE				CHECKED BY						
09-1111-6036		N 4823614.1 ; E 276498.5				MS						
07-20024		Track-Mounted CME 55, 108 mm Internal Diameter Hollow Stem Augers				CS						
Central HWY 401		December 15, 2010				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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE								
--- CONTINUED FROM PREVIOUS PAGE ---												
177.3	SAND and GRAVEL, trace to some silt, trace clay Compact to very dense Brown Wet		14	SS	100/05							31 55 11 3
15.9	SHALE (BEDROCK) Reddish-brown		15	SS	100/08							
174.9			16	SS	100/05							
18.3	END OF BOREHOLE											
NOTES: 1. Shallow piezometer installed in separate borehole 2. Water level in piezometers measured as follows: Deep piezometer Date Depth (m) Elev. (m) Dec. 15, 2010 0.3 192.9 Feb. 3, 2011 0.2 193.0 Apr. 21, 2011 0.1 a.g.s.* 193.3 *Above ground surface Shallow piezometer Date Depth (m) Elev. (m) Dec. 15, 2010 1.3 191.9 Feb. 3, 2011 1.3 191.9 Apr. 21, 2011 0.7 192.5												

PROJECT 09-1111-6036		RECORD OF BOREHOLE No 10-402		1 OF 2 METRIC	
W.O. 07-20024		LOCATION N 4823662.1 ; E 276469.5		ORIGINATED BY MS	
DIST Central HWY 401		BOREHOLE TYPE Truck-Mounted CME-75, 70 mm Internal Diameter Hollow Stem Power Auger		COMPILED BY CS	
DATUM Geodetic		DATE November 29, 2010		CHECKED BY LCC	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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
								20	40	60	80	100	w _p	w	w _L					
201.2	GROUND SURFACE																			
0.0	ASPHALT																			
200.7	CONCRETE																			
0.5	Sand and gravel, some silt, trace clay (FILL) Compact to dense Brown Moist		1	SS	31															
			2	SS	45															
			3	SS	41															
			4	SS	34															
			5	SS	22															
			6	SS	44															
195.6																				
5.6	CLAYEY SILT with sand, trace to some gravel (TILL) Stiff to hard Brown to grey Wet		7	SS	13															
			8	SS	21															
			9	SS	38															
			10	SS	26															
			11	SS	23															
			12	SS	30															

Continued Next Page

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

PROJECT <u>09-1111-6036</u>		RECORD OF BOREHOLE No 10-402		2 OF 2 METRIC	
W.O. <u>07-20024</u>		LOCATION <u>N 4823662.1 ; E 276469.5</u>		ORIGINATED BY <u>MS</u>	
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>Truck-Mounted CME-75, 70 mm Internal Diameter Hollow Stem Power Auger</u>		COMPILED BY <u>CS</u>	
DATUM <u>Geodetic</u>		DATE <u>November 29, 2010</u>		CHECKED BY <u>LCC</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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	20	40	60	80					
	--- CONTINUED FROM PREVIOUS PAGE ---																			
180.5 20.7	CLAYEY SILT with sand, trace to some gravel (TILL) Stiff to hard Brown to grey Wet		13	SS	44															
			14	SS	57															
			15	SS	178															
			16	SS	100/.13															

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



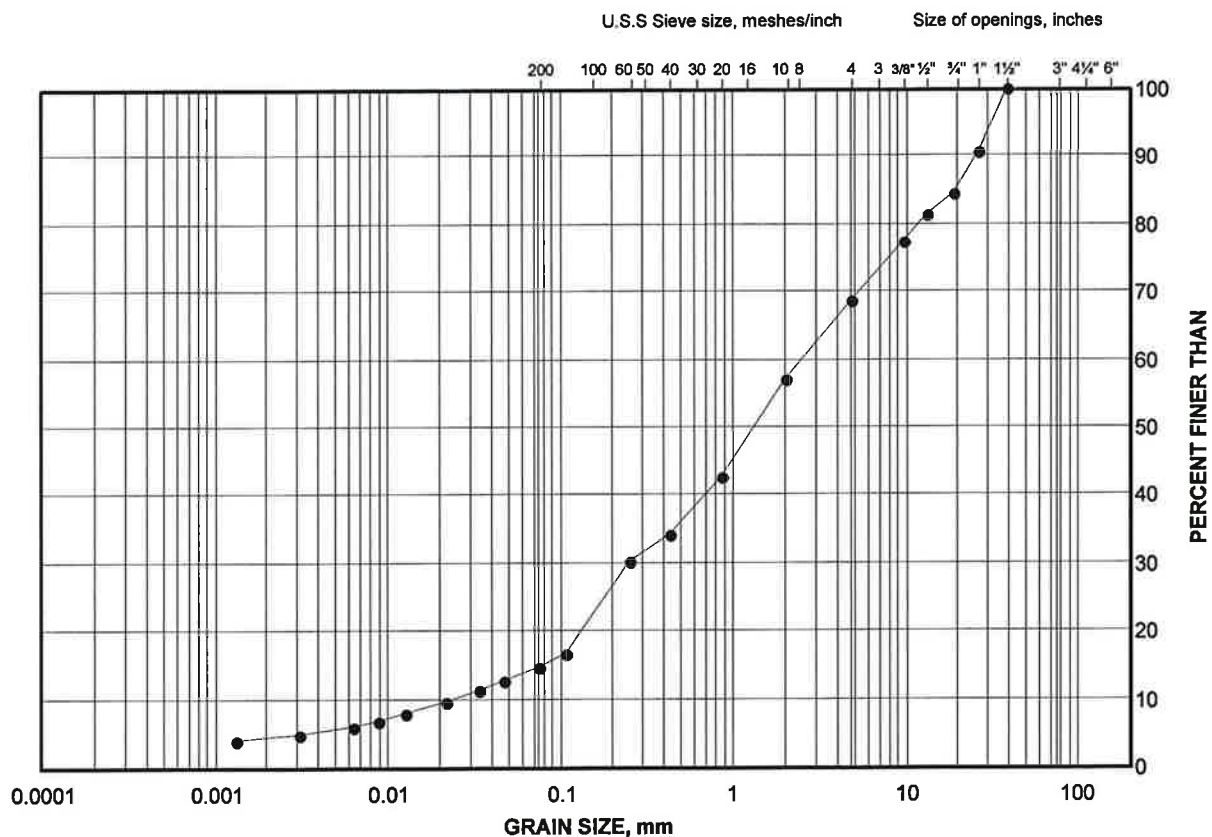
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION TEST RESULT

Sand and Gravel Fill

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)
•	10-402	4	197.8

Project Number: 09-1111-6036

Checked By: *[Signature]*

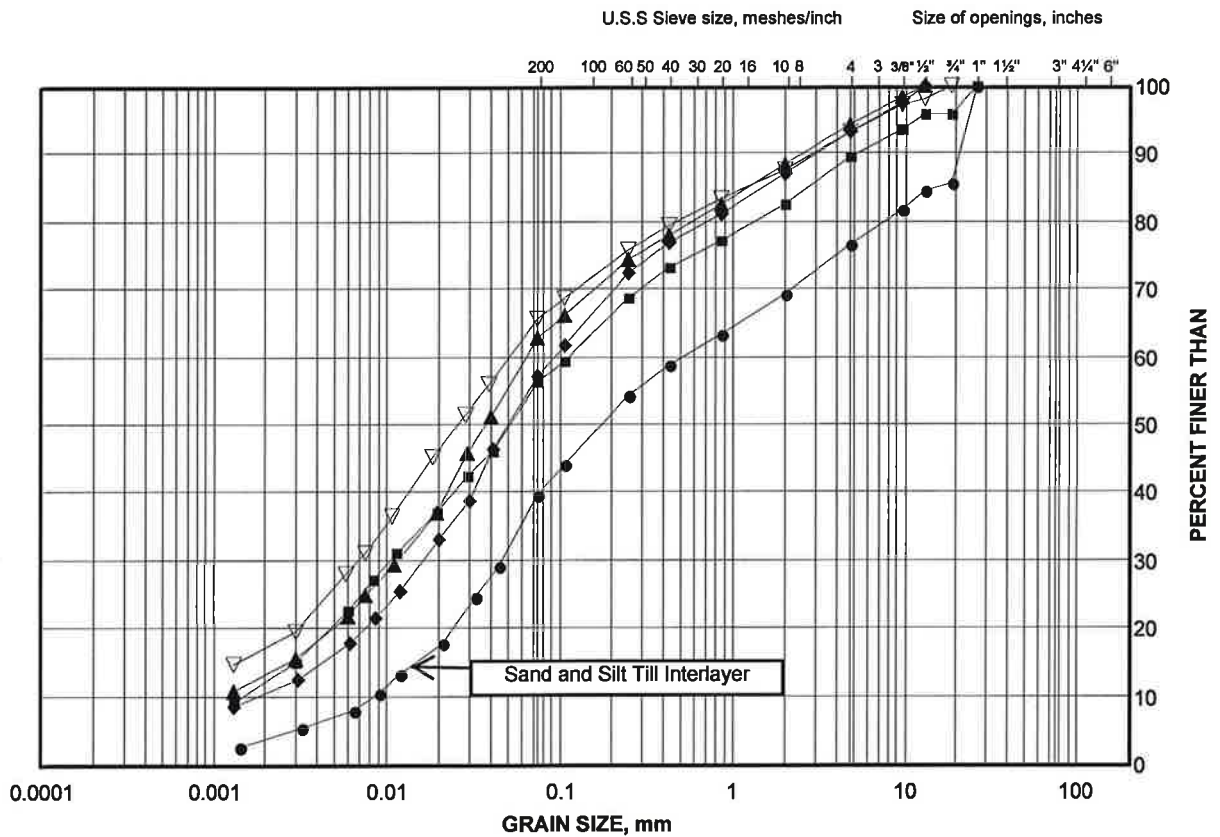
Golder Associates

Date: 17-Jun-11

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt Till

FIGURE B2



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

LEGEND

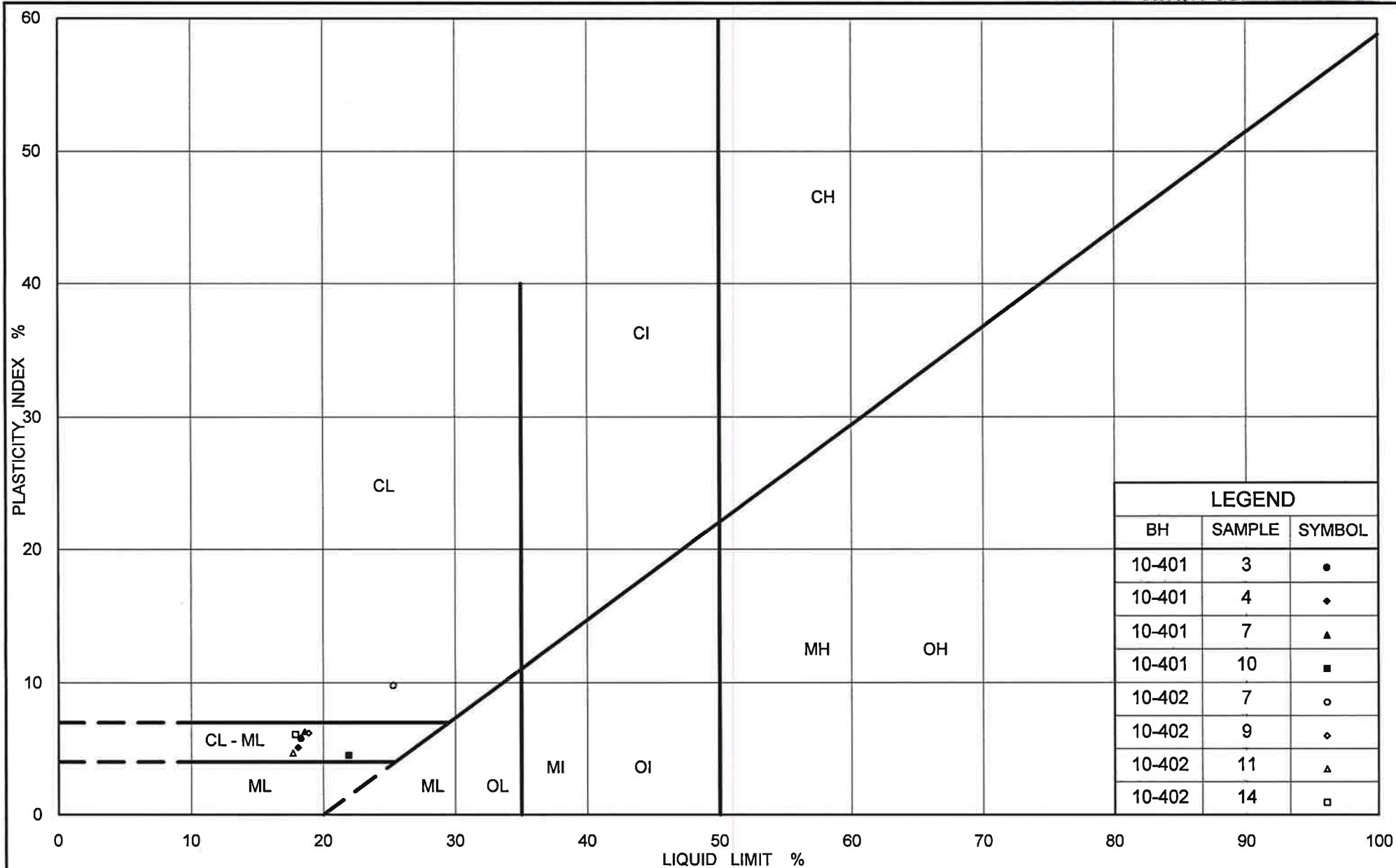
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-401	11	182.2
■	10-402	14	184.1
◆	10-402	15	182.6
▲	10-401	4	190.6
▽	10-402	7	194.8

Project Number: 09-1111-6036

Checked By: *Wagye*

Golder Associates

Date: 17-Jun-11



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt Till

Figure No. B3

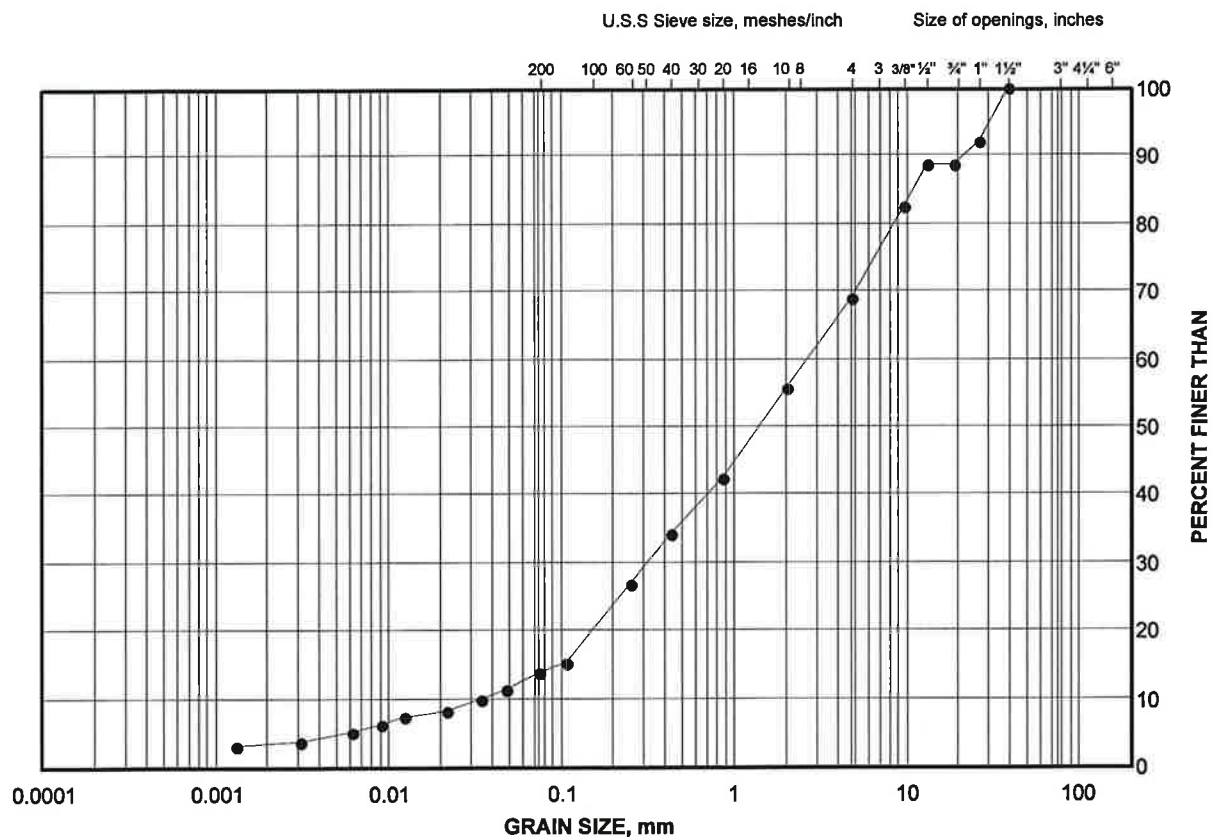
Project No. 09-1111-6036

Checked By: *Wayne*

GRAIN SIZE DISTRIBUTION TEST RESULT

Sand and Gravel

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)
•	10-401	14A	177.9

Project Number: 09-1111-6036

Checked By: *[Signature]*

Golder Associates

Date: 17-Jun-11

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.

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