October 2011

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

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

Submitted to:
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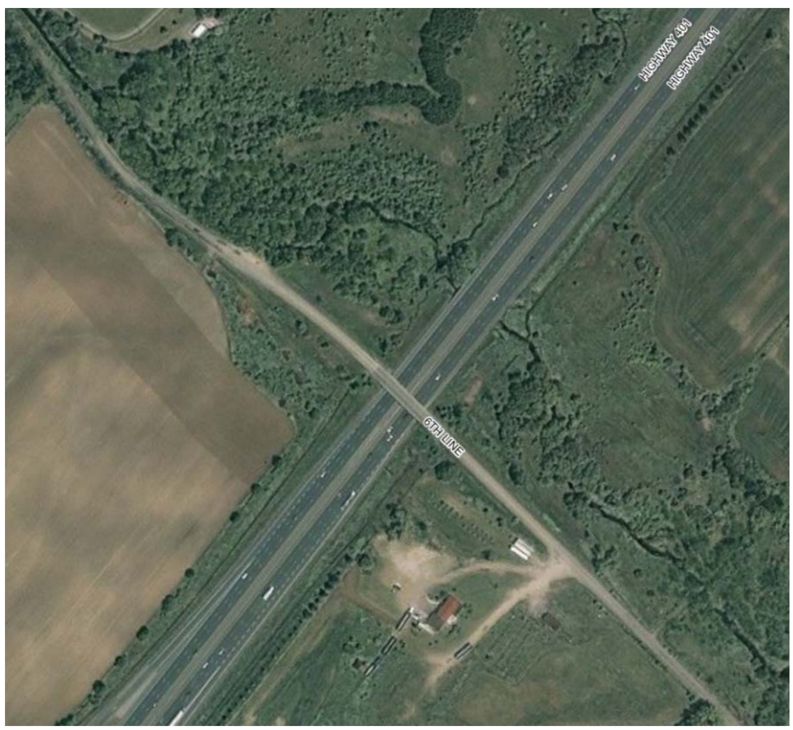

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REPORT

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

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 and/or replacement of the existing Oakville Creek East Branch (Middle East Sixteen Mile Creek) 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 East Branch (Middle East Sixteen Mile Creek) bridge is located approximately 90 m east of the Highway 401-6th Line underpass – about 7 km east of Regional Road 25 and 1.5 km west of Trafalgar Road – in the Regional Municipality of Halton, Ontario. The existing structure consists of a 30.5 m long single-span bridge, with a clear span of 15.2 m. The bridge was built in 1959, with the last rehabilitation occurring in 1977 which included a 5.5 m north widening and a 2.5 m south widening.

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 190 m to 191.5 m, rising toward the west and east away from the creek valley. The creek meander channel was straightened during the 1959 construction, and was constructed with its channel base at approximately Elevation 188.9 m, similar to the original natural channel. The typical water depth is about 0.4 m (Elevation 189.3 m). It is understood that the high water level at this creek site is at approximately Elevation 191.4 m.

Highway 401 has been constructed in embankment fill that is approximately 3 m to 3.5 m in height, with the pavement grade at approximately Elevation 193.4 m to 193.6 m, rising eastward. 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 2010 and February 2011, during which time two boreholes (Boreholes 10-601 and 10-602) were advanced using a truck-mounted CME-75 drill rig supplied and operated by Geo-Environmental Drilling Inc. of Milton, Ontario. The borehole locations are shown on Drawing 1: both boreholes were advanced through the highway shoulder, with Borehole 10-601 located in the southeast quadrant and Borehole 10-602 located in the northwest quadrant of the structure site.

Boreholes 10-601 and 10-602 were drilled using 70 mm inner diameter hollow stem augers to depths of 12.8 m and 23.5 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.

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).

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 co-ordinate 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)
10-601	4,824,741.7	277,396.7	193.5	12.8
10-602	4,824,729.9	277,343.2	192.8	23.5

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 current subsurface investigation, two boreholes were advanced at the vicinity of the Oakville Creek East Branch (Middle East Sixteen Mile Creek) 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 borehole records contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B7 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 relatively thin surficial deposit of soft to firm clayey silt, which is underlain by a deposit of very stiff to hard clayey silt till. The clayey silt till is underlain by a deposit of compact silty sand to sand and silt, which is in turn underlain by a deposit of very dense silt till. A more detailed description of the soil deposits encountered in the boreholes is provided in the following sections.

4.2.1 Embankment Fill

Both boreholes were advanced through the Highway 401 embankment, and encountered about 200 mm of asphalt immediately below the highway surface. In Borehole 10-601 in the southeast quadrant of the structure site, approximately 300 mm of concrete was encountered below the asphalt.

Approximately 2.5 m to 2.8 m of embankment fill was encountered below the asphalt and concrete in Boreholes 10-601 and 10-602, with the base of the fill encountered at about Elevations 190.5 m and 189.8 m, respectively. This fill consists of the following:

- An upper layer, approximately 0.5 m to 0.9 m thick, of sand and gravel containing some silt and trace clay. This sand and gravel fill has a compact relative density, based on one measured Standard Penetration Test (SPT) “N” value of 15 blows per 0.3 m of penetration.
- A lower layer, about 1.6 m to 2.3 m thick, of clayey silt containing some sand and trace to some gravel, as well as trace quantities of organic material. Atterberg limits testing was carried out on two selected samples of the cohesive fill and measured plastic limits of 17 per cent and 20 per cent, liquid limits of 32 per cent and 33 per cent, and plasticity indices of 15 per cent and 13 per cent, respectively. These results, which are plotted on a plasticity chart on Figure B1 in Appendix B, confirm that the cohesive portion of the fill consists of clayey silt of low plasticity. The SPT “N” values measured within the clayey silt fill range from 4 blows to 17 blows per 0.3 m of penetration, suggesting a firm to very stiff consistency.

4.2.2 Surficial Clayey Silt

A 0.7 m to 2.6 m thick surficial deposit of brown to grey clayey silt was encountered below the embankment fill in both boreholes. The surface of this deposit was encountered at Elevation 190.5 m and 189.8 m, and its base was encountered at Elevation 187.9 m and 189.1 m in the boreholes in the southeast and northwest quadrants of the site, respectively.

This surficial deposit consists of clayey silt with sand to some sand, containing trace gravel as well as small quantities of organic matter. A thin interlayer or lens of sand and gravel was encountered near the base of the deposit (Elevation 188.6 m) in Borehole 10-601. The results of grain size distribution tests completed on two selected samples of the surficial clayey silt are shown on Figure B2 in Appendix B. An Atterberg limits test was completed on one selected sample of this deposit and measured a plastic limit of 14 per cent, a liquid limit of 23 per cent, and a plasticity index of 9 per cent. This test result, which is plotted on a plasticity chart on Figure B3 in Appendix B, indicates that the surficial deposit consists of clayey silt of low plasticity.

The measured SPT "N" values within the surficial clayey silt deposit range from 2 blows to 7 blows per 0.3 m of penetration, suggesting a soft to firm consistency.

4.2.3 Clayey Silt Till

A deposit of clayey silt till was encountered below the surficial clayey silt in both boreholes. The surface of the till was encountered at Elevation 187.9 m and 189.1 m in Boreholes 10-601 and 10-602, respectively. The deposit was fully penetrated in Borehole 10-602 in the northwest quadrant, where it is about 10.3 m thick with the deposit base at approximately Elevation 179.5 m. Borehole 10-601 was terminated in this till deposit after penetrating it for a thickness of 7.2 m (to Elevation 180.7 m).

This till deposit consists of clayey silt with sand to trace sand, containing trace to some gravel. A 0.9 m thick interlayer of sand and gravel containing trace silt and clay was encountered within the clayey silt till deposit in Borehole 10-602, between approximately Elevation 182.0 m and 181.1 m. The results of grain size distribution tests carried out on two selected samples of the clayey silt till are shown on Figure B4 in Appendix B. Atterberg limits testing was conducted on four selected samples of this deposit and measured plastic limits of 13 per cent to 18 per cent, liquid limits of 19 per cent to 30 per cent, and plasticity indices of 6 per cent to 15 per cent. These results, which are plotted on a plasticity chart on Figure B5 in Appendix B, confirm that the deposit consists of clayey silt of low plasticity.

The measured SPT "N" values within the clayey silt till range from 23 blows to 114 blows per 0.3 m of penetration suggesting a very stiff to hard consistency. One SPT "N" value of 59 blows per 0.3 m of penetration was measured in the sand and gravel interlayer/lens, indicating that this material has a very dense relative density.

4.2.4 Silty Sand to Sand and Silt

A 6.0 m thick deposit of silty sand to sand and silt was encountered below the clayey silt till in Borehole 10-602. The surface of this deposit was encountered at Elevation 179.5 m and its base was encountered at Elevation 173.5 m (at a depth of 13.3 m to 19.3 m below the Highway 401 surface.

The deposit varies in composition from silty sand to sand and silt containing trace to some gravel. The result of a grain size distribution test conducted on one selected sample of this deposit is shown on Figure B6 in Appendix B.

The measured SPT "N" values within the silty sand to sand and silt deposit range from 6 blows to 23 blows per 0.3 m of penetration; however, these values are considered to have been affected by sample disturbance due to groundwater inflow to the borehole. This deposit is considered to have at least a compact relative density.

4.2.5 Silt Till

A deposit of silt till was encountered below the silty sand to sand and silt deposit in Borehole 10-602, with its surface at approximately Elevation 173.5 m (at a depth of about 19.3 m). Borehole 10-602 terminated within the till deposit after penetrating it for a thickness of 4.2 m, to Elevation 169.3 m.

This till deposit consists of silt containing some sand, trace to some gravel and trace clay. The result of a grain size distribution test conducted on one selected sample of the silt till is shown on Figure B7 in Appendix B.

The measured SPT "N" values within the silt till ranged from 101 blows per 0.3 m of penetration to 110 blows per 0.25 m of penetration, indicating a 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 contained in Appendix A.

In general, the cohesionless interlayers within the clayey silt till deposit and the lower silty sand to sand and silt deposit were observed to be water-bearing during the drilling operations. The water pressure associated with the lower silty sand to sand and silt deposit was observed to be artesian with respect to the Highway 401 grade (i.e., above approximately Elevation 193.5 m), although this water pressure subsided as the borehole was extended.

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

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Matt Soderman, 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 DESIGN REPORT
OAKVILLE CREEK EAST BRANCH (MIDDLE EAST SIXTEEN MILE CREEK) BRIDGE
HIGHWAY 401 WIDENING FROM TRAFALGAR ROAD TO REGIONAL ROAD 25, HALTON REGION
W.O. 07-20024

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 East Branch (Middle East Sixteen Mile Creek) bridge. 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

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 of Highway 401. The existing Oakville Creek East Branch (Middle East Sixteen Mile Creek) bridge will require widening to the north and south; replacement of the existing structure may also be considered.

The existing structure consists of a 30.5 m long single-span bridge, with a clear span of 15.2 m. The bridge was built in 1959, with the last rehabilitation occurring in 1977 which included a 5.5 m north widening and a 2.5 m south widening. Based on the *Elevation and Site Plan* drawing for this structure, prepared by the Department of Highways Ontario in January 1958, the existing abutments are supported on 3.0 m wide spread footings founded at approximately Elevation 187.3 m (614.4 ft.). This founding elevation is about 2.6 m below the bottom of the creek channel, about 3 m to 4 m below the natural ground surface, and about 6 m below the Highway 401 grade 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 north and south widening and/or replacement of the Oakville Creek East Branch (Middle East Sixteen Mile Creek) 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 or retaining walls at this site, provided they are founded below the soft to firm surficial clayey silt deposit on the very stiff to hard clayey silt till. This would require excavation to a depth of approximately 3 m to 4 m relative to the natural ground surface at the site; temporary excavation support would likely be required both adjacent to Highway 401 and along the creek channel. There is potential for up to approximately 10 mm to 15 mm of differential settlement between the existing structure and the new widened portions if shallow foundations are adopted for the widening.

- **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 soft to firm surficial clayey silt 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, differential settlement between the existing structures (which are founded on spread footings) 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 "hanging up" on cobbles or boulders within the glacial till soils, and for variable pile lengths (although further investigation is required in this regard at the detail design stage); driving shoes are recommended to protect piles during driving in the anticipated hard/very dense ground conditions.
- **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, and would minimize differential settlement between the existing spread footing-supported core structure and the widened portions. 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 silty sand to sand and silt deposit.

The following sections provide recommendations for spread footing foundations and driven steel H-pile or steel pipe pile foundations to support the proposed bridge widening. 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 abutments and associated wing walls or retaining walls, strip or spread footings should be founded below the fill, soft to firm clayey silt soils, and any loose alluvial soils on the very stiff to hard clayey silt till deposit. The following table provides the recommended maximum (highest) founding elevations for preliminary design of footings founded on the clayey silt till deposit, based on the results from Boreholes 10-601 and 10-602 and the anticipated depth of the former creek meander channel (prior to straightening during construction of the existing bridge).

If spread footings are used for a widening, it is likely that they will be constructed to match the existing foundation elevation (approximately Elevation 187.3 m) and structurally connected. In the case of a full structure replacement, it may be feasible to found the footings for the north portion of the replacement and widening slightly higher than the existing foundations, consistent with the elevations provided in the following table. Further assessment of the impact of the existing foundations and their removal will be required at detail design, once the geometry and span of the replacement structure is established.

These recommended founding levels should also be assessed relative to the potential scour depth for the hydraulic conditions in the creek, to ensure adequate protection against scour.

Foundation Element	Maximum (Highest) Founding Elevation	Approximate Excavation Depth
North Widening or North Portion of Replacement	188.0 m (or lower to match existing)	2.0 m – relative to original ground
South Widening or South Portion of Replacement	187.3 m	3.2 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 soft to firm clayey silt 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.

Foundation Element	Footing Width	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS*
North Widening or North Portion of Replacement	3 m	500 kPa	450 kPa
	4 m	600 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 Steel 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 within the hard/very dense till soils, where the measured Standard Penetration Test "N" values exceed 100 blows per 0.3 m of penetration. The following pile tip elevations may be used for preliminary design purposes, assuming about 2 m of penetration into the hard/very dense till deposit. Further investigation will be required during detail design to assess the variability in the surface elevation for the 100-blow soil to refine these pile tip elevations.

Foundation Element	Borehole No.	Surface Elevation of "100-Blow" Soil	Estimated Design Pile Tip Elevation
North Widening or North Portion of Replacement	10-602	173.0 m	171.0 m
South Widening or South Portion of Replacement	10-601	184.5 m	182.5 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 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 silty sand to sand and silt deposit was observed to be artesian relative to the Highway 401 grade, 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 310x110 piles driven to the estimated tip elevations provided in Section 6.4.1, the factored axial resistance at ULS may be taken as 1,400 kN, and the axial geotechnical resistance at SLS (for 25 mm of settlement) may be taken as 1,200 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).*

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 3.5 m high embankment widening based on the subsurface conditions as encountered in Boreholes 10-601 and 10-602. 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°	-
Soft to firm surficial clayey silt	20	-	30 kPa
Very stiff to hard clayey silt till including sand and gravel interlayer	21	32°	-
Compact silty sand to sand and silt	20	30°	-
Very dense silt till	21	34°	-

The analysis results indicate that an approximately 3.5 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 3 m to 3.5 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/m^3)	Elastic Modulus (MPa)	P_c' (kPa)	C_c	C_r
Embankment fill	21	-	-	-	-
Soft to firm surficial clayey silt	20	5	130	0.11	0.011
Very stiff to hard clayey silt till	21	70 (North Side) 115 (South Side)	-	-	-
Compact silty sand to sand and silt	20	30	-	-	-
Very dense silt till	21	125	-	-	-

NOTES: P_c' = preconsolidation pressure
C_c = compression index
C_r = recompression index

Based on this preliminary assessment, the settlement of the foundation soils under the widened 3 m to 3.5 m high Highway 401 approach embankments is estimated to be approximately 20 mm to 35 mm. Approximately one-third to one-half of this settlement is expected to occur relatively quickly during and immediately following construction of the widened approach embankments. However, settlement of the surficial clayey silt deposit will be time-dependent and could take a few to several months, depending on the thickness and consolidation properties of the surficial deposit. Although the anticipated magnitude of settlement is not overly large, it is recommended that the approach embankment widening area be preloaded or the final paving in this area be delayed until the majority of settlement is complete. The estimated magnitude and rate of settlement should be reassessed based on the soil and groundwater conditions under the widened approach embankments as determined during detail design, with particular emphasis on the thickness and properties of the surficial clayey silt deposit throughout the embankment widening areas.

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 into the Contract Documents.

6.6.1 Excavation and Temporary Protection Systems

The foundation excavations for spread footings would extend to a depth of about 2.0 m to 3.2 m below the natural ground surface at the site, through existing fill, the soft to firm surficial clayey silt, and into the very stiff to hard 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 soft to firm 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 would 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 through the lagging boards from any water-bearing alluvial soil deposits.

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.

There is a risk that foundation excavations in the abutment widening areas 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. Where alluvial soils are present, groundwater inflows could be minimized with the use of an interlocking steel sheetpile system.

For deep foundations, due to the anticipated artesian groundwater conditions associated with the lower silty sand to sand and silt aquifer, it is recommended that a sand filter, possibly in combination with a geotextile, be placed beneath the pile caps to prevent the migration of fines that may be transported along the piles during and after construction.

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 Preloading in Approach Embankment Widening Areas

As discussed in Section 6.5, some time-dependent consolidation settlement is anticipated within the soft to firm surficial clayey silt deposit at this site, and preloading of the approach embankment widening area may be considered to minimize differential settlement between the existing and widened embankment areas and behind the new abutments following final paving. Further assessment of the distribution, thickness and consolidation properties of this surficial deposit will be required during detail design to confirm the need for and length of preloading.

6.6.6 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.

6.7 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:

- 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 within each of the abutment widening areas.
  - 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 silty sand to sand and silt deposit. Due to the high (slightly artesian) groundwater levels, the use of the vibrating wire piezometers or pressure transducers may be appropriate.
- Approach 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, particularly the surficial clayey silt deposit, to confirm the stability analyses and settlement estimates.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Matt Soderman, E.I.T., 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 consultant 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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MAS/LCC/TJG/mas

n:\active\2009\1111\09-1111-6036 urs - hwy 401 - halton region\5 - reports\6 - oakville creek east branch bridge\09-1111-6036 rpt06 11oct oakville creek east branch bridge.docx

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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

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	• Feasible for support of widened abutments and associated wing walls/retaining walls	• Existing structure supported on shallow foundations, and has performed well • Relatively minor groundwater seepage anticipated from cohesive till deposit • Allows for semi-integral abutments	• Significant excavations (to a depth of up to 3 m below the original ground surface) to extend below firm to stiff surficial clayey silt soils; would require temporary excavation support • Moderate risk for up to about 15 mm of differential settlement between existing bridge and widening areas • Precludes use of integral abutments; potentially greater maintenance required at abutments • Lower geotechnical resistances as compared with deep foundations	• Conventional excavation and construction techniques	• Less expensive than deep foundations although bridge maintenance costs may be higher due to non-integral abutment configuration • Estimated cost is $84,000 per foundation area (i.e., southeast abutment, southwest abutment, etc.) based on an estimated $600/m^3 for construction of shallow foundations, excluding deeper excavation and temporary protection system
Spread/strip footings perched on compacted granular pad in approach embankment fill	• Not recommended for support of widened abutments	• Abutment pile caps could be maintained higher than footings founded on till deposit, significantly reducing subexcavation depth and associated temporary protection system requirements	• 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	• Conventional excavation and construction techniques	• This option not recommended

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel H-piles driven to found within "100-blow" clayey silt till or silt till	• Feasible for support of widened abutments and associated wing walls/retaining walls	• 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 if existing structure can be modified to accommodate, or if replacement is adopted • Would minimize differential settlement between existing bridge and widened portions of structure	• Minor risk for encountering obstructions (cobbles and/or boulders) during pile driving that could result in piles "hanging up" and lower geotechnical resistances • Artesian groundwater pressures at site will require further investigation and may require special construction techniques to mitigate the potential upward flow of water and fine soil particles along the pile shafts	• Conventional construction methods	• Lower relative cost compared with caisson option • Estimated cost is approximately \$125,000 for each of the northwest and northeast abutment widening areas, and \$70,000 for each of the southwest and southeast abutment widening areas, based on an estimated \$250/m length for pile installation and \$600/m^3 for pile cap construction

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel pipe piles (closed-end and concrete filled) driven to found within “100-blow” clayey silt till or silt till	• Feasible for support of widened abutments and associated wing walls/retaining walls	• 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	• 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 • Artesian groundwater pressures at site will require further investigation and may require special construction techniques to mitigate the potential upward flow of water and fine soil particles along the pile shafts	• Conventional construction methods	• Costs for steel pipe (tube) piles slightly higher than for steel H-piles
Caissons founded in “100-blow” clayey silt till or silt till deposit	• Feasible but not recommended for support of abutments and wing walls/ retaining walls	• 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	• High groundwater pressures create potential for disturbance or loss of ground in lower silty sand to sand and silt deposit • Temporary or permanent liners would be required; likely not possible to inspect caisson base • Precludes use of integral abutments	• Risk of loss of ground near base of caissons	• Higher cost compared with shallow foundations or steel H-piles

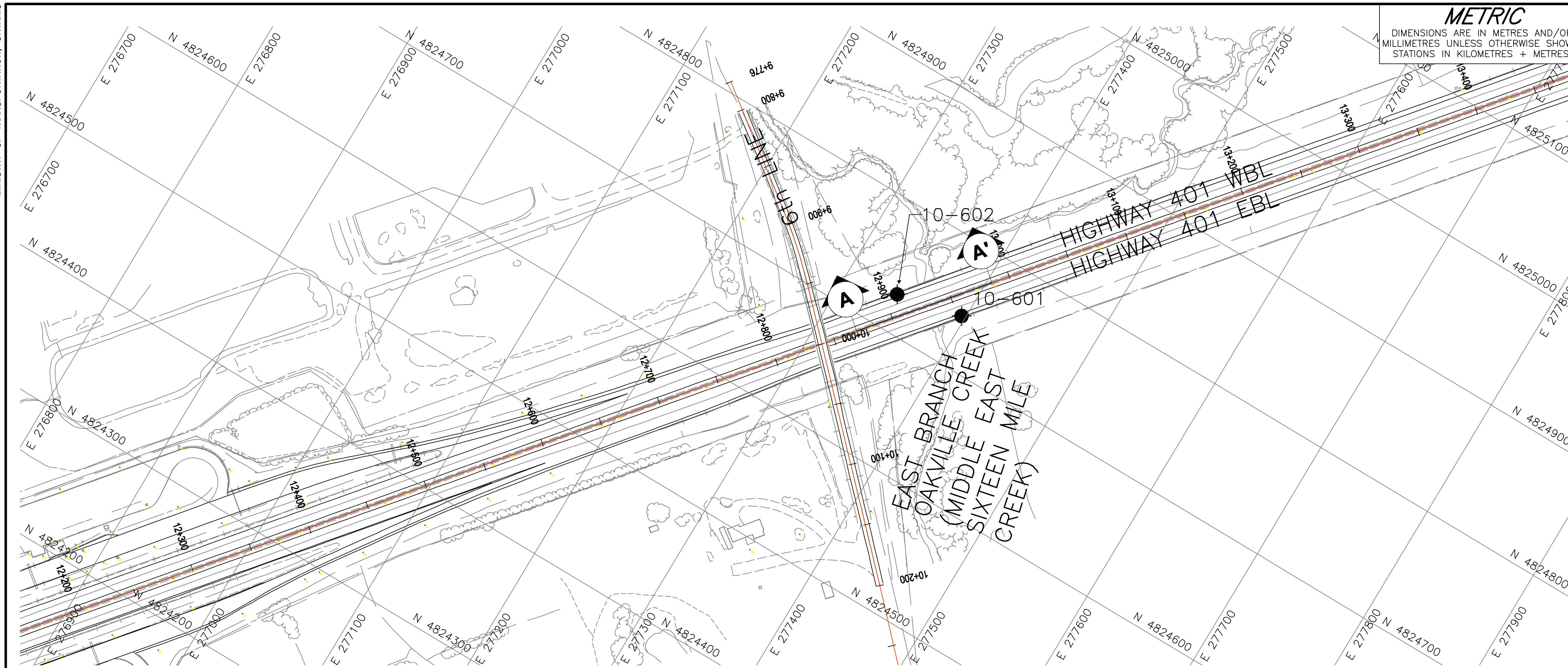
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WO No. 07–20024

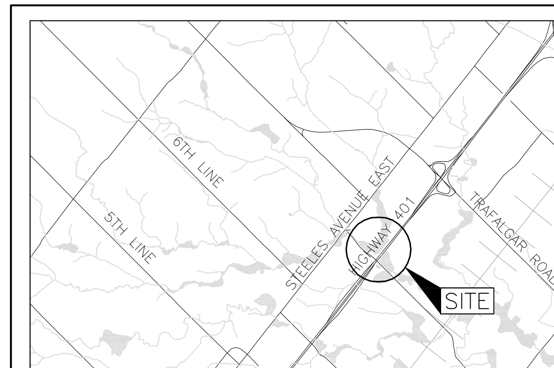
SHEET

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

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN
SCALE
40 0 40 80 m



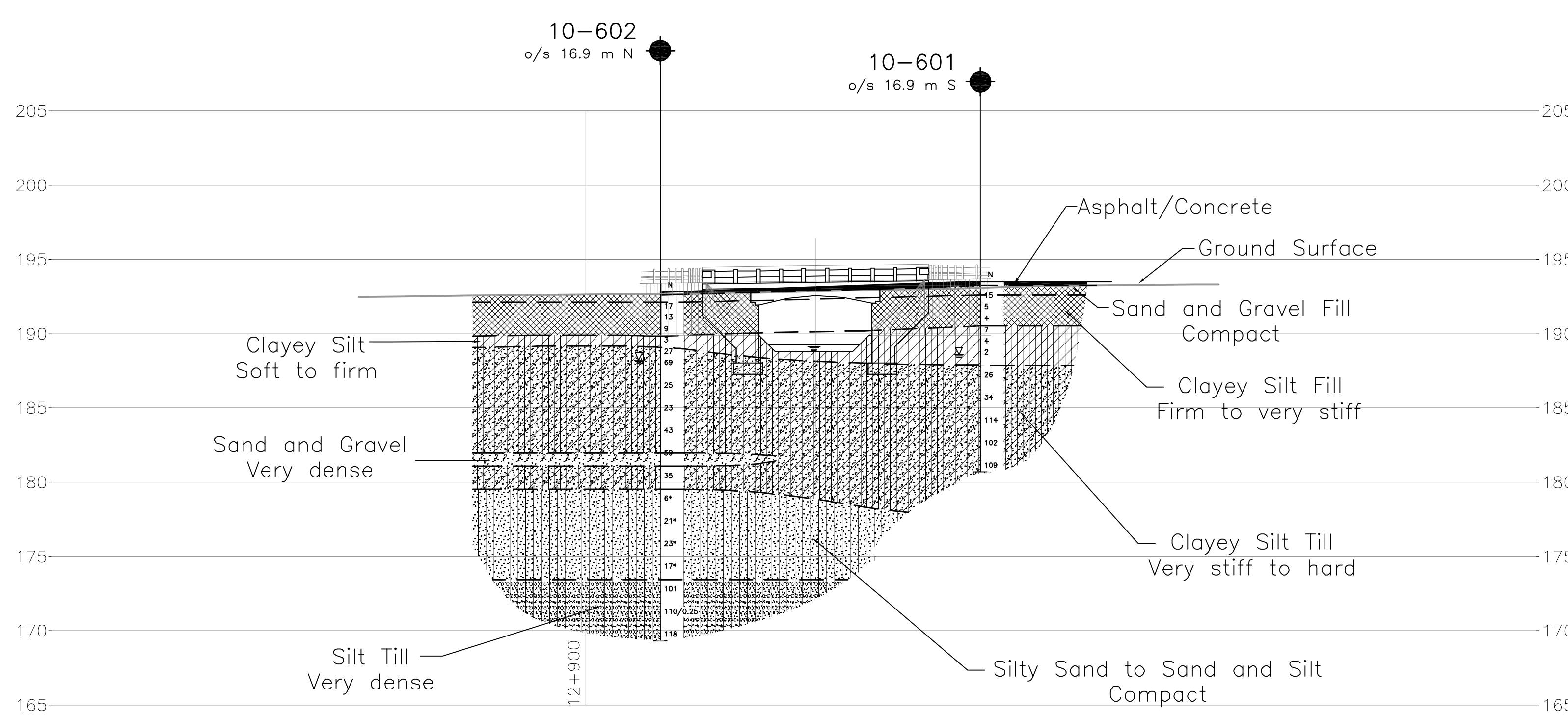
KEY PLAN
SCALE
1 0 1 2 km

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

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
10–601	193.5	4824741.7	277396.7
10–602	192.8	4824729.9	277343.2



SECTION A–A'
HORIZONTAL SCALE
10 0 10 20 m
5 0 5 10 m
VERTICAL SCALE

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 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.

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).

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Nov. 21, 2011 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER T. J. GARDE Nov. 21, 2011 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

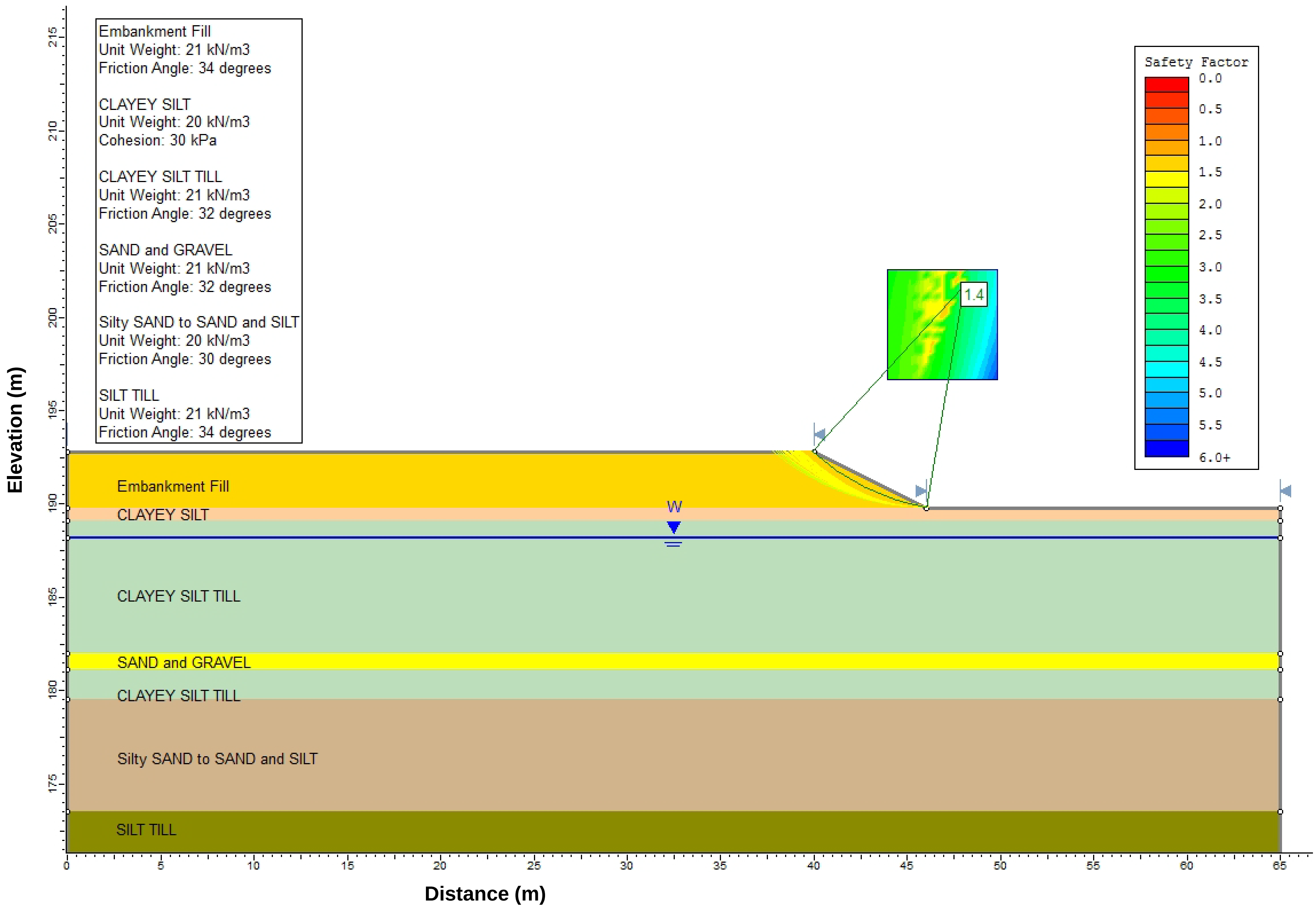
Geocres No. 30M12–330

HWY. 401	PROJECT NO. 09–1111–6036		DIST.
SUBM'D. MS	CHKD. NK	DATE: 10/21/2011	SITE:
DRAWN: JFC/CD	CHKD. MS	APPD. LCC	DWG. 1

PLOT DATE: October 21, 2011
FILENAME: T:\Projects\2009\09-1111-6036 (URS, HWY 401 Halton, MTO)\-EA-(Stamped Drawings)\0911116036EA006.dwg

Static Global Stability – East Branch Oakville Creek Embankment Widening

Figure 1



Date: June 2011

Project No: 09-1111-6036

Analysis By: MAS **Reviewed By:** LCC

Golder Associates

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, 60º cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

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

Piezo-Cone Penetration Test (CPT)
A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 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 Relative Density	N Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils
Consistency

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

IV. SOIL TESTS

w	water content
w_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
in 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
υ	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

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

(a) Index Properties (continued)

w	water content
w_l	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

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

Notes: 1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (compressive strength)/2

Golder Associates

RECORD OF BOREHOLE No 10-601

 METRIC

PROJECT 09-1111-6036

W.O. 07-20024 | LOCATION N 4824741.7 ;E 277396.7 | ORIGINATED BY MS

DIST Central HWY 401 | BOREHOLE TYPE Truck-Mounted CME-75, 70 mm Internal Diameter Hollow Stem Augers | COMPILED BY CS

DATUM Geodetic | DATE November 28, 2010 | CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
193.5	GROUND SURFACE					
0.0	ASPHALT					
193.0	CONCRETE					
0.5	Sand and gravel, some silt, trace clay (FILL) Compact Brown Moist	1	SS	15		
192.1 / 1.4	Clayey silt, some sand, some gravel, containing organics (FILL) Firm Dark brown Moist	2	SS	5		
		3	SS	4		
190.5 / 3.0	CLAYEY SILT, some sand, trace gravel, containing organics Firm to soft Brown becoming grey at 3.8 m Moist becoming wet at 3.8 m	4	SS	7		1 14 68 17
		5	SS	4		
	Layer of sand and gravel at 4.9 m	6	SS	2	∇	
187.9 / 5.6	CLAYEY SILT with sand, trace gravel (TILL) Very stiff to hard Brown to grey Wet	7	SS	26		2 30 43 25
		8	SS	34		
		9	SS	114		
		10	SS	102		
		11	SS	109		
180.7 / 12.8	END OF BOREHOLE					

NOTES:

1. Water level in open borehole at a depth of 4.9 m (Elev. 188.6 m) upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

ELEVATION SCALE: 193 192 191 190 189 188 187 186 185 184 183 182

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 10-602

 METRIC

PROJECT 09-1111-6036

W.O. 07-20024 LOCATION N 4824729.9 ;E 277343.2 ORIGINATED BY AM

DIST Central HWY 401 BOREHOLE TYPE Truck-Mounted CME-75, 70 mm Internal Diameter Hollow Stem Augers COMPILED BY SB

DATUM Geodetic DATE February 15, 2011 CHECKED BY LCC

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
192.8	GROUND SURFACE
0.0	ASPHALT
0.2	Sand and gravel, some silt (FILL)
192.1 / 0.7	Clayey silt, some sand, trace gravel (FILL) Stiff to very stiff Grey to brown Moist
189.8 / 3.0	CLAYEY SILT with sand, trace gravel, containing organics Soft Brown Moist
189.1 / 3.7	CLAYEY SILT, trace to some sand, trace to some gravel (TILL) Very stiff to hard Brown Moist
182.0 / 10.8	SAND and GRAVEL, trace silt, trace clay Very dense Brown Wet
181.1 / 11.7	CLAYEY SILT, trace sand and gravel (TILL) Hard Grey Moist
179.5 / 13.3	Silty SAND to SAND and SILT, trace to some gravel Compact Brown Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	17	
2	SS	13	
3	SS	9	
4	SS	3	1 42 45 12
5	SS	27	
6	SS	69	
7	SS	25	1 15 40 44
8	SS	23	
9	SS	43	
10	SS	59	
11	SS	35	
12	SS	6*	

GROUND WATER CONDITIONS ∇ (at approx. elev. 188.3)

ELEVATION SCALE: 192, 191, 190, 189, 188, 187, 186, 185, 184, 183, 182, 181, 180, 179, 178

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 10-602

 METRIC

PROJECT 09-1111-6036
W.O. 07-20024 LOCATION N 4824729.9 ;E 277343.2 ORIGINATED BY AM
DIST Central HWY 401 BOREHOLE TYPE Truck-Mounted CME-75, 70 mm Internal Diameter Hollow Stem Augers COMPILED BY SB
DATUM Geodetic DATE February 15, 2011 CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	Silty SAND to SAND and SILT, trace to some gravel Compact Brown Wet	13	SS	21*	177	○	0	44	54	2
		14	SS	23*	176					
		15	SS	17*	175, 174					
173.5 / 19.3	SILT, some sand, trace to some gravel, trace clay (TILL) Very dense Brown Moist	16	SS	101	173	○	21	19	52	8
		17	SS	110/0.25	172, 171					
		18	SS	118	170					
169.3 / 23.5	END OF BOREHOLE									

NOTES:

*SPT "N" value considered to be affected by sample disturbance due to groundwater inflow to borehole.

1. Artesian groundwater pressure encountered at a depth of 15.2 m to 15.8 m (Elev. 177.6 to 177.0 m). Groundwater pressure subsided when borehole drilled deeper.

2. Water level in open borehole noted at a depth of 4.6 m (Elev. 188.2 m) upon completion of drilling.

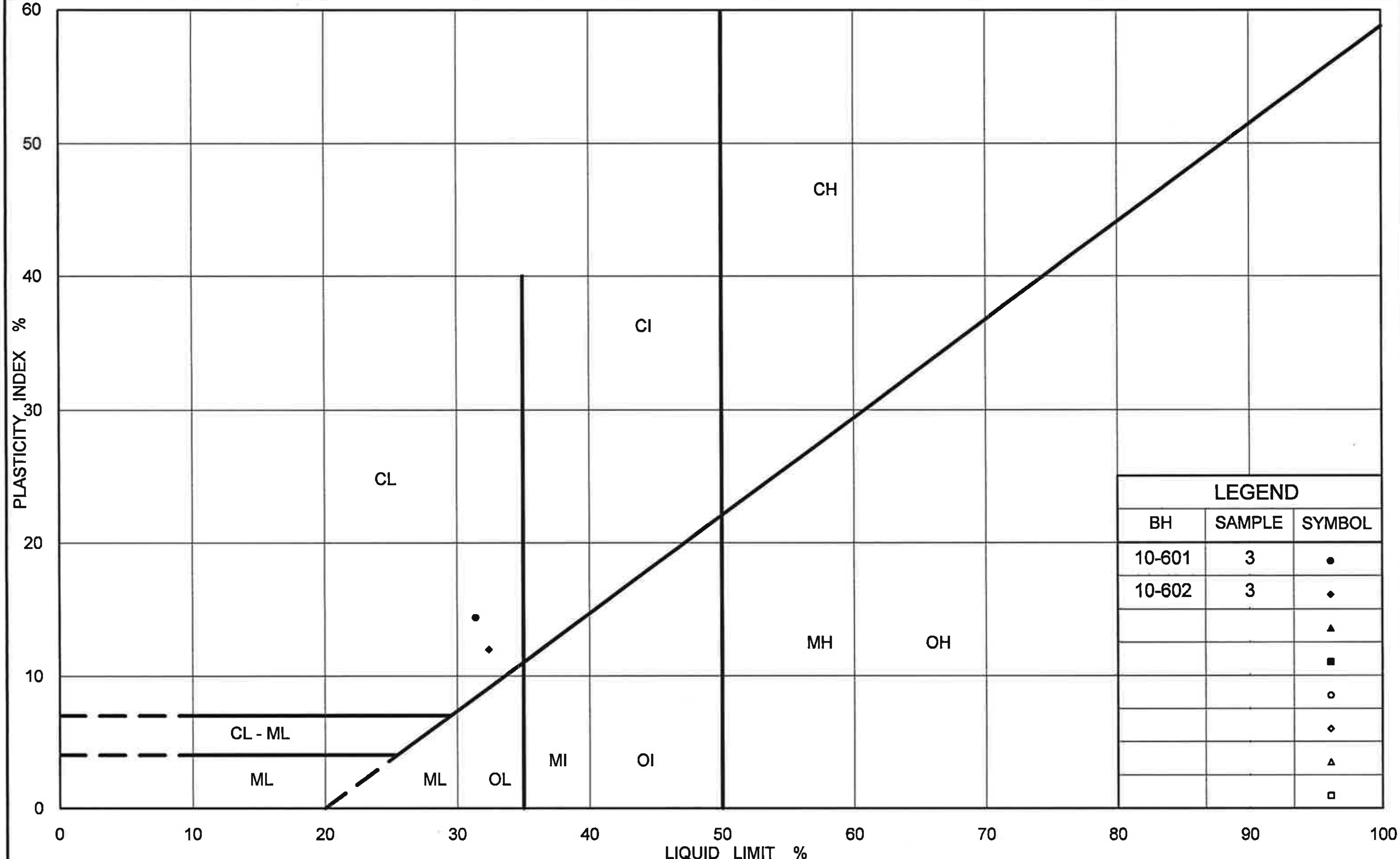
Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

APPENDIX B

Laboratory Test Results

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
10-601	3	●
10-602	3	◆
		▲
		■
		○
		◇
		△
		□

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PLASTICITY CHART
Clayey Silt Fill

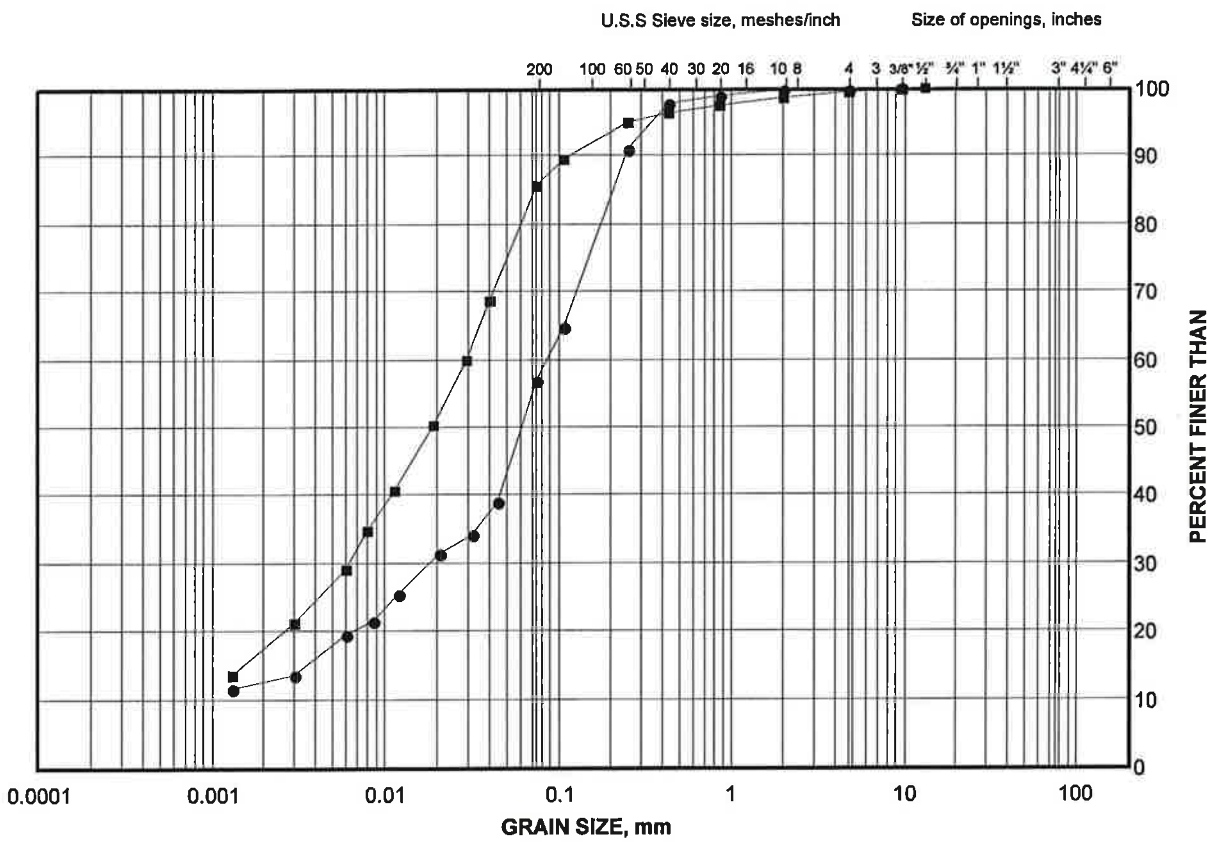
Figure No. B1

Project No. 09-1111-6036

Checked By:

GRAIN SIZE DISTRIBUTION TEST RESULTS
Surficial Clayey Silt

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)
●	10-602	4	189.5
■	10-601	4	190.2

Project Number: 09-1111-6036

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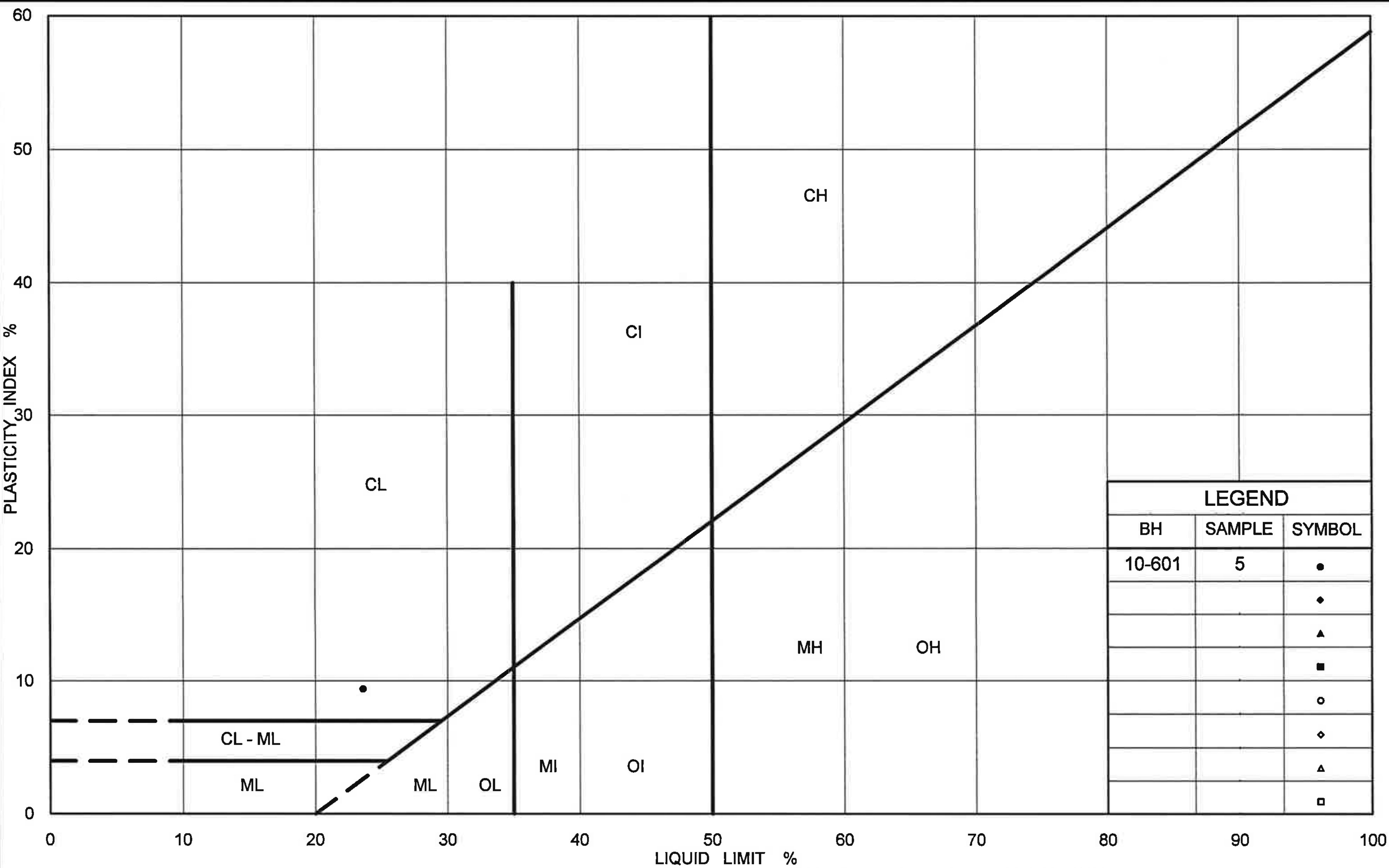
Date: 17-Jun-11

Oct 75, FF-S-21



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PLASTICITY CHART
Surficial Clayey Silt

Figure No. B3

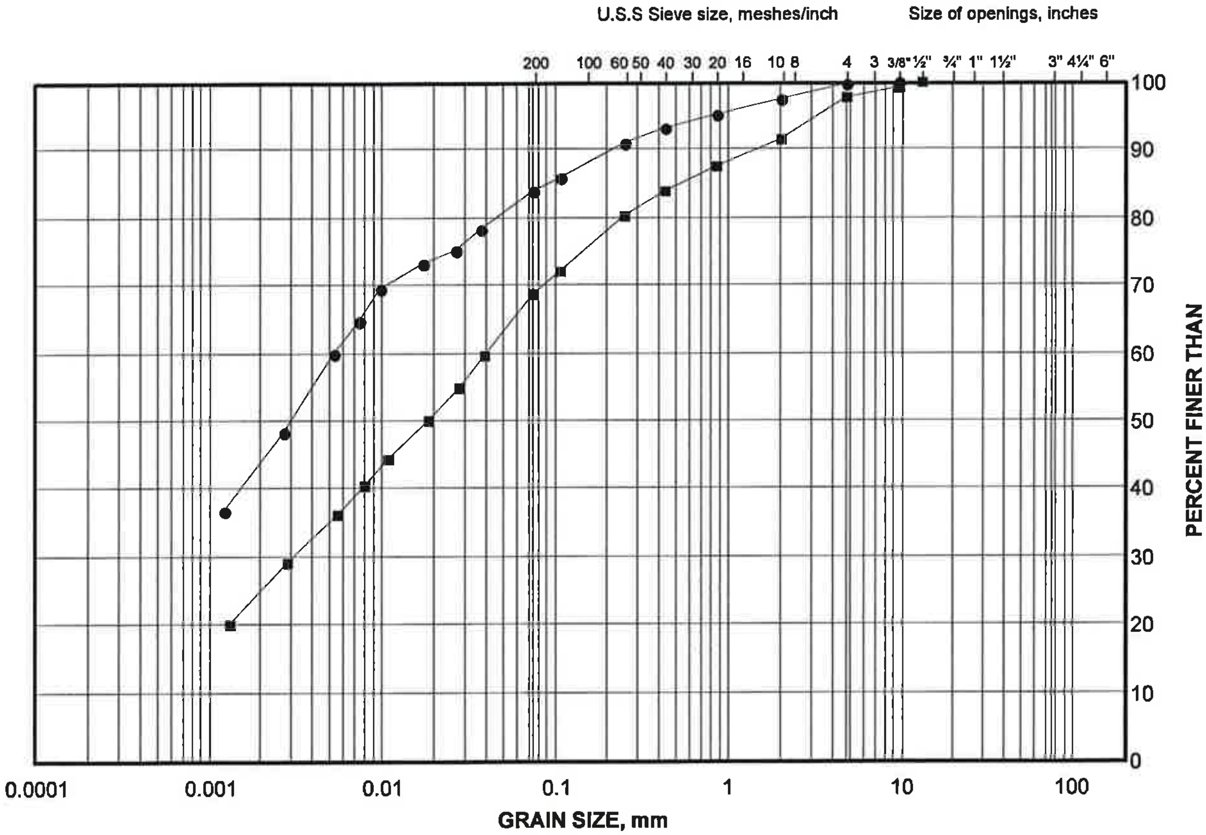
Project No. 09-1111-6036

Checked By: [signature]

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt Till

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)
●	10-602	7	186.4
■	10-601	7	187.1

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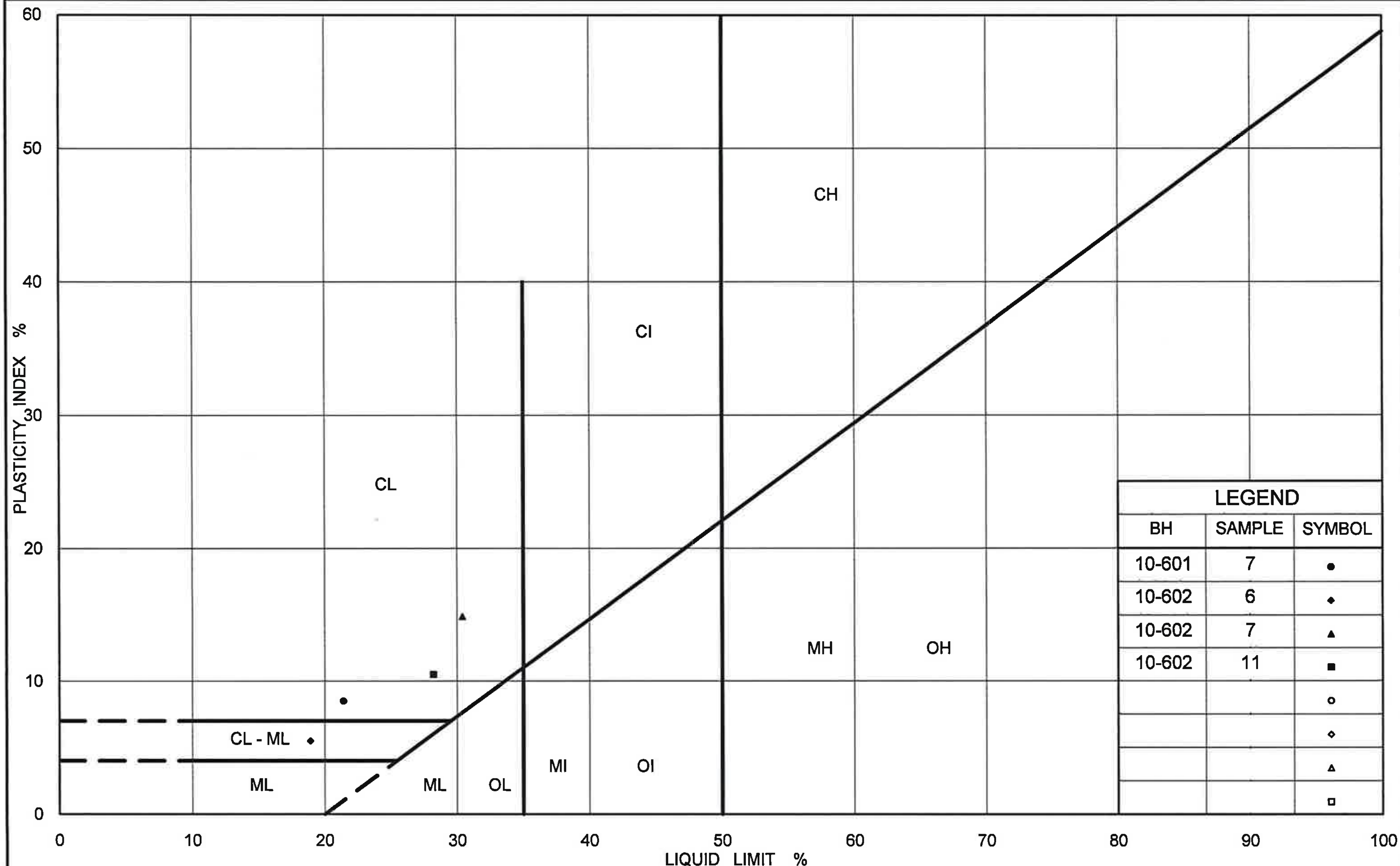

LEGEND

BH	SAMPLE	SYMBOL
10-601	7	●
10-602	6	◆
10-602	7	▲
10-602	11	■
		○
		◇
		△
		□

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PLASTICITY CHART
Clayey Silt Till

Figure No. B5

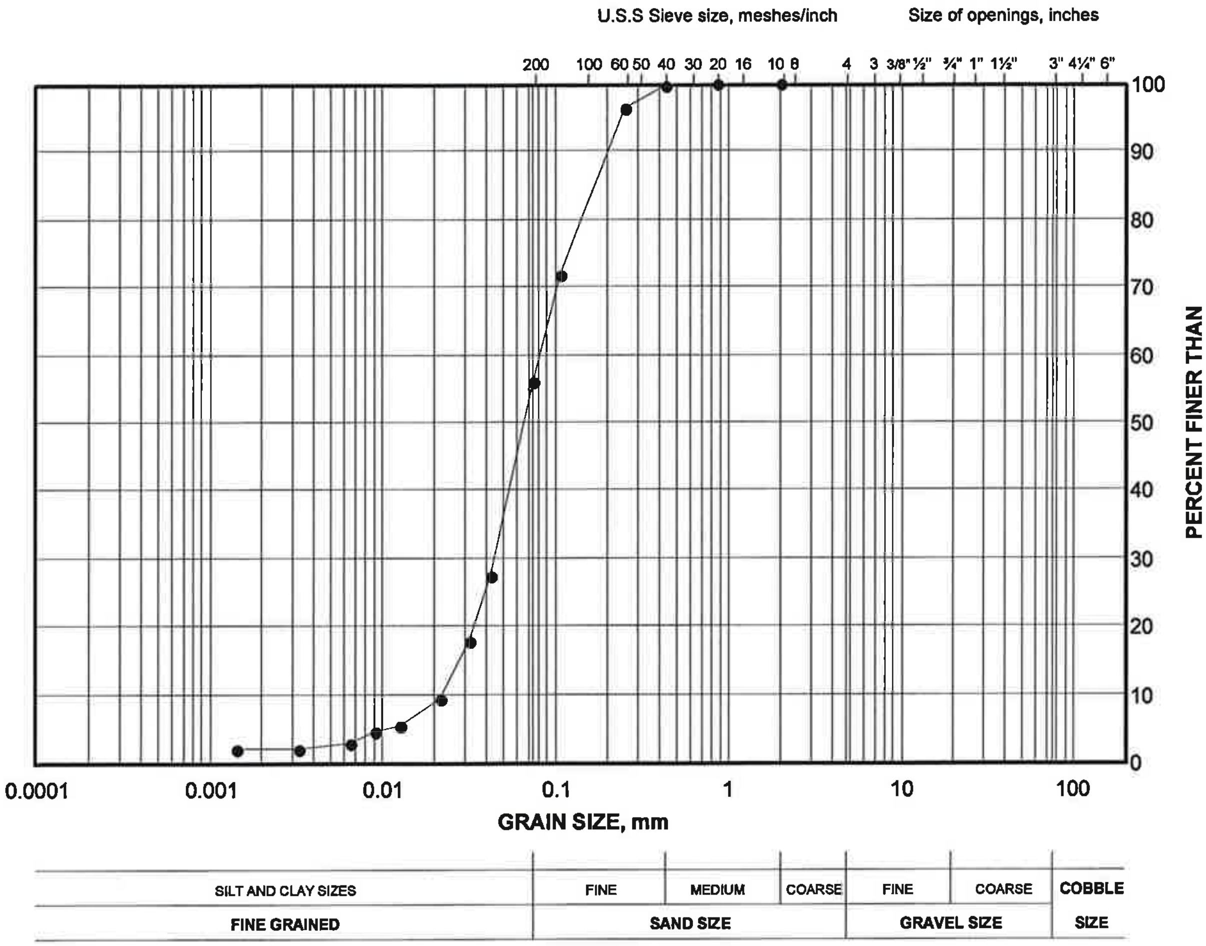
Project No. 09-1111-6036

Checked By: [signature]

GRAIN SIZE DISTRIBUTION TEST RESULT

Silty Sand to Sand and Silt

FIGURE B6



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-602	13	177.3

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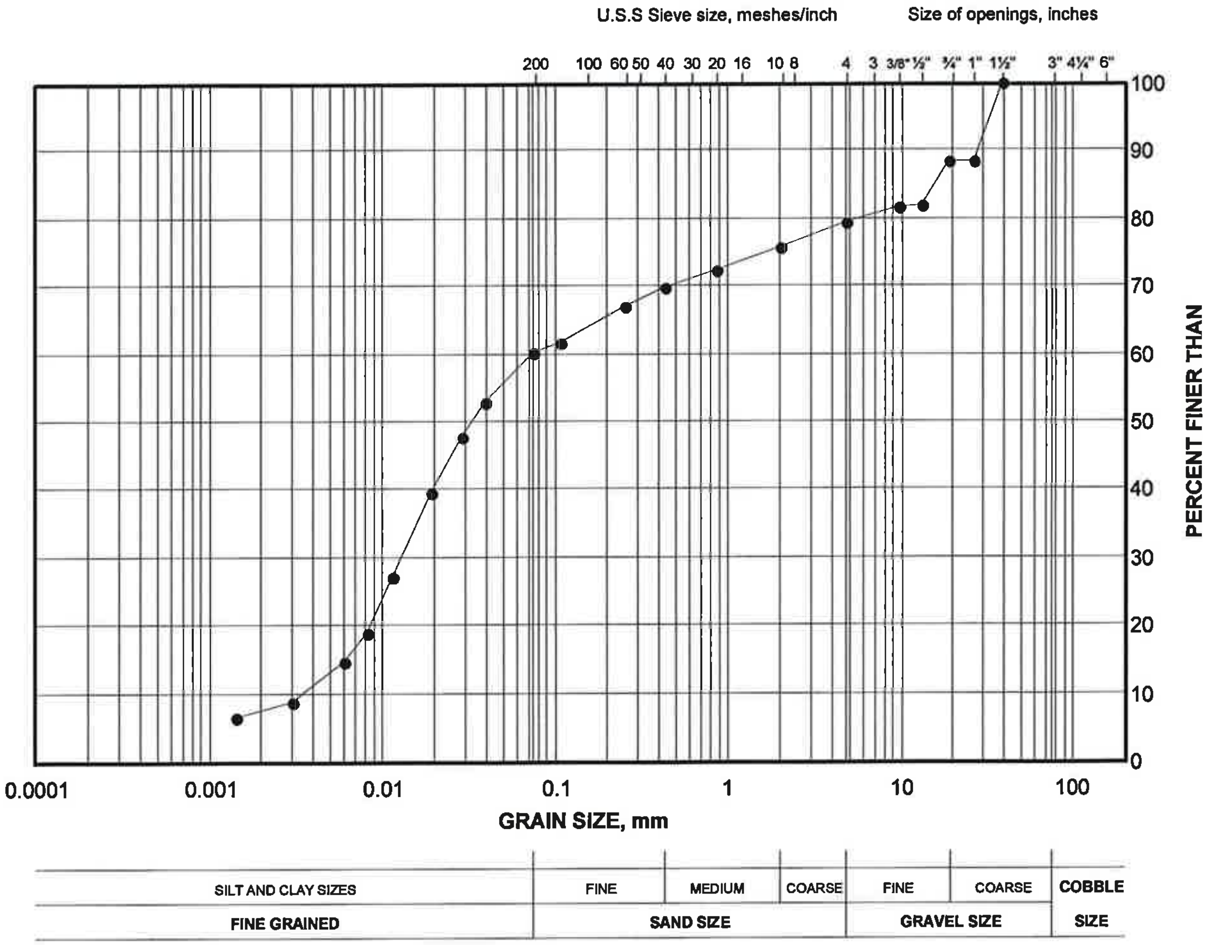
Date: 17-Jun-11

GRAIN SIZE DISTRIBUTION TEST RESULT

Silt Till

FIGURE B7



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-602	16	172.7

Project Number: 09-1111-6036

Checked By:

Golder Associates

Date: 17-Jun-11

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