

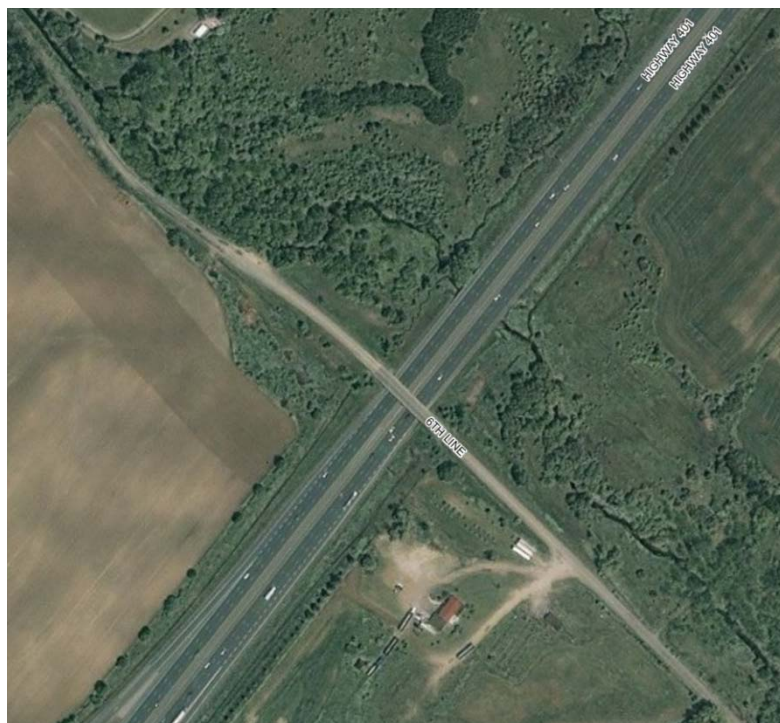


October 2011

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

6th Line Underpass Highway 401 Widening from Trafalgar Road to Regional Road 25, Halton Region W.O. 07-20024

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REPORT





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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
6th LINE UNDERPASS
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 replacement of the existing 6th Line underpass structure.

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 6th Line underpass is located at the intersection of Highway 401 and 6th Line, approximately 7 km east of Regional Road 25 and 1.5 km west of Trafalgar Road, in the Regional Municipality of Halton, Ontario.

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 m. The ground surface declines eastward toward the East Branch of Oakville Creek, which is located about 100 m to the east of 6th Line. A culvert is present on the north side of Highway 401, and runs under 6th Line in an east-west direction.

Highway 401 has been constructed on embankment fill that is approximately 1.5 m to 2.5 m high, with its grade at approximately Elevation 192.5 m. The local road (6th Line) has been constructed on embankment fill that is up to approximately 7 m to 8 m in height, with the pavement grade at about Elevation 199 m at the structure site. The 6th Line abutment foreslopes and 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 December 2010 and February 2011, during which time two boreholes (Boreholes 10-501 and 10-502) were advanced using a truck-mounted and track-mounted CME-75 and CME-55 drill rigs, supplied and operated by Geo-Environmental Drilling Inc. of Milton, Ontario. Borehole 10-501 was advanced through the 6th Line embankment south of Highway 401, and Borehole 10-502 was advanced near the toe of the 6th Line embankment adjacent to the culvert in the northeast quadrant.

Boreholes 10-501 and 10-502 were drilled using 70 mm and 108 mm inner diameter hollow stem augers to depths of 26.2 m and 19.5 m, respectively. Soil samples were obtained at 0.75 m, 1.5 m and 3 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. All 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 borehole locations were measured in the field by Golder personnel relative to site features and using a hand-held global positioning system (GPS) 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 the 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)
10-501	4,824,640.7	277,338.8	197.8	26.2
10-502	4,824,724.8	277,303.2	191.0	19.5

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, located 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 the 6th Line underpass structure. 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 B9 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 deposit of clayey silt till that contains granular till interlayers, underlain by deposits of clayey silt, sand to sandy silt, and sand and silt to sandy silt till, 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 50 mm of topsoil was encountered immediately below the existing ground surface in Borehole 10-502, which was advanced at the toe of the 6th Line embankment adjacent to the culvert.

4.2.2 Embankment Fill

Approximately 0.6 m of fill was encountered immediately below the topsoil layer in Borehole 10-502, which was advanced near the east toe of the 6th Line embankment north of Highway 401. This fill extends to about Elevation 190.4 m, and consists of clayey silt to silty clay containing trace to some sand and trace gravel as well as small amounts of organic matter. This fill has a stiff consistency, based on a measured Standard Penetration Test (SPT) "N" value of 10 blows per 0.3 m of penetration.

Approximately 100 mm of asphalt and 7.1 m of fill associated with the existing 6th Line embankment was encountered in Borehole 10-501, extending to approximately Elevation 190.6 m. This fill consists of the following:

- An upper layer, about 1.2 m thick, of silty sand containing trace to some gravel and trace clay. One SPT "N" value of 13 blows per 0.3 m of penetration was measured in this layer, indicating that the silty sand fill has a compact relative density.
- A lower layer, about 5.9 m thick, of clayey silt to silty clay containing trace to some sand and trace gravel. The result of a grain size distribution test carried out on one selected sample of the cohesive fill is shown on Figure B1 in Appendix B. Atterberg limits testing was carried out on one selected sample of the cohesive fill and measured a plastic limit of 17 per cent, a liquid limit of 35 per cent, and a plasticity index of 18 per cent. These test results, which are plotted on a plasticity chart on Figure B2 in Appendix B, confirm that the fill consists primarily of low to medium plasticity clayey silt to silty clay. The measured SPT "N" values within the cohesive fill range from 4 blows to 13 blows per 0.3 m of penetration, suggesting a firm to stiff consistency.



4.2.3 Clayey Silt Till with Interlayers

A clayey silt till deposit was encountered at a depth of 7.2 m (Elevation 190.6 m) in Borehole 10-501 and at a depth of 0.6 m (Elevation 190.4 m) in Borehole 10-502. The thickness of the till is 9.1 m and 8.1 m in Boreholes 10-501 and 10-502, respectively, with the base of the till deposit encountered at approximately Elevation 181.5 m and 182.3 m.

The deposit consists of clayey silt with sand to trace sand, containing trace to some gravel. Interlayers or lenses of silty sand and gravel till and sand and silt till, about 0.4 m and 1.6 m thick, were encountered at Elevation 188.4 m and 186.5 m in Boreholes 10-501 and 10-502, respectively. The results of grain size distribution tests completed on two selected samples of the clayey silt till are shown on Figure B3 in Appendix B.

Atterberg limits testing was carried out on four selected samples of the cohesive till and measured plastic limits ranging from 12 per cent to 16 per cent, liquid limits ranging from 23 per cent to 27 per cent, and plasticity indices ranging from 10 per cent to 12 per cent. These test results, which are plotted on a plasticity chart on Figure B4 in Appendix B, confirm that the cohesive till consists of low plasticity clayey silt. Atterberg limits testing was also completed on one sample of the sand and silt till and measured a plastic limit of 12 per cent, a liquid limit of 14 per cent and a corresponding plasticity index of 2 per cent. This test result, which is plotted on a plasticity chart on Figure B5 in Appendix B, confirms that this portion of the till deposit is slightly plastic to non-plastic.

The measured SPT "N" values within the clayey silt till range from 10 blows to 152 blows per 0.3 m of penetration, suggesting a stiff to hard consistency. Within the silty sand and gravel to sand and silt till, SPT "N" values of 15 blows and 61 blows per 0.3 m of penetration were measured, indicating a compact to very dense relative density.

4.2.4 Clayey Silt

A 1.5 m to 4.0 m thick deposit of clayey silt was encountered below the clayey silt till in both boreholes. The deposit extended from approximately Elevation 181.5 m to 177.5 m in Borehole 10-501 on the south side of Highway 401, and from approximately Elevation 182.3 m to 180.8 m in Borehole 10-502 on the north side of Highway 401.

The clayey silt deposit contains trace sand. The result of a grain size distribution test carried out on one sample of this deposit is shown on Figure B6 in Appendix B. Atterberg limits testing was carried out on two selected samples of the clayey silt and measured plastic limits of 17 per cent, liquid limits of 25 per cent and 28 per cent, and plasticity indices of 8 and 11 per cent. These test results, which are plotted on a plasticity chart on Figure B7 in Appendix B, confirm that the deposit is comprised of low plasticity clayey silt.

The measured SPT "N" values within the clayey silt range from 27 blows to 53 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.



4.2.5 Sand to Sandy Silt

A 2.6 m to 4.6 m thick deposit of sand to sandy silt was encountered at a depth of 20.3 m (Elevation 177.5 m) in Borehole 10-501 and 10.2 m (Elevation 180.8 m) in Borehole 10-502. The base of the deposit extended to approximately Elevation 174.9 m in Borehole 10-501 on the south side of Highway 401, and to approximately Elevation 176.2 m in Borehole 10-502 on the north side of the Highway.

The deposit varies in composition from sand containing trace silt and clay, to sandy silt containing trace clay. The results of grain size distribution tests carried out on two samples of this deposit are shown on Figure B8 in Appendix B.

The SPT “N” values measured within this deposit range from 6 blows to 30 blows per 0.3 m of penetration, indicating a loose to compact relative density.

4.2.6 Sand and Silt Till to Sandy Silt Till

A deposit of sand and silt till to sandy silt till was encountered underlying the sand to sandy silt, with its surface encountered at Elevation 174.9 m and 176.2 m in Boreholes 10-501 and 10-502 on the south and north sides of Highway 401, respectively. The deposit is about 2.3 m thick as encountered in Borehole 10-501, where it was fully penetrated. Borehole 10-502 was terminated within this deposit after penetrating it for 4.7 m.

This till deposit varies in composition from sand and silt to sandy silt containing some gravel and trace clay. The results of grain size distribution tests carried out on two selected samples of this till deposit are shown on Figure B9 in Appendix B.

The SPT “N” values measured within the sand and silt to sandy silt till deposit range from 24 blows per 0.3 m of penetration to 105 blows per 0.15 m of penetration, indicating a compact to very dense relative density.

4.2.7 Shale Bedrock

Shale bedrock was encountered at approximately Elevation 172.6 m (at a depth of about 25.2 m below the 6th Line grade) and penetrated for approximately 1.0 m in Borehole 10-501. One measured SPT “N” value of 110 blows per 0.10 m of penetration was measured in the bedrock in this borehole.

Borehole 10-502 was terminated at Elevation 171.5 m (at a depth of about 19.5 m) due to auger refusal on the inferred bedrock surface.

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 till interlayers within the clayey silt till, the sand to sandy silt deposit, and the sand and silt to sandy silt till deposit were observed to be water-bearing during the drilling operations. The water



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pressure associated with the lower soil deposits was observed to be artesian with respect to the natural ground surface at the site (i.e., above Elevation 191.0 m).

It should be noted that 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. Billy Murphy and Ms. Nikol Kochmanová, P.Eng., 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
6th LINE UNDERPASS
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 replacement of the existing Highway 401-6th Line underpass. 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 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, it is understood that the future widening of this section of Highway 401 could consist of three additional lanes in both the eastbound and westbound directions. The existing 6th Line underpass will require replacement to accommodate the highway widening, and it is understood that a two-span replacement structure is proposed to be constructed along the existing 6th Line alignment. Further assessment of the removal of the existing foundations and their impact on the replacement structure will be required at detail design, once the geometry and span of the replacement structure is finalized.

Following construction of the replacement structure, the finished grade for 6th Line will be at approximately Elevation 200 m at the north abutment and Elevation 199 m at the south abutment, such that the total height of the approach embankments is approximately 9 m high relative to the natural ground surface. This will require a grade raise on the order of 1 m above the existing 6th Line grade.

Based on the proposed underpass geometry and the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments and centre pier for the new 6th Line underpass. 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 new abutments, associated wing walls/retaining walls, and centre pier at this site, provided they are founded below the stiff portion of the till deposit, on very stiff to hard clayey silt till. This would require excavation to a depth of approximately 1.5 m to 3.5 m relative to the original ground surface, or about 3 m to 5 m relative to the existing Highway 401 grade, and about 9.5 m to 11.5 m relative to the 6th Line grade. Temporary excavation support would be required at the centre pier and would likely be required at the abutment locations if such significant excavations are adopted.



- **Footings “perched” on a compacted granular pad in the approach embankments:** Although this option would be advantageous in minimizing the depth of excavation, “perched” footings are not preferred for support of the new abutments due to the presence of stiff clayey silt till (as encountered in Borehole 10-501), and the firm to stiff consistency of the existing embankment fill materials.
- **Driven steel H-piles:** Driven steel H-piles are suitable and feasible for support of the abutments (and would permit integral abutment design), as well as associated wing walls/retaining walls and the centre pier at this site. However, there is a minor risk associated with penetrating through or the piles “hanging up” on cobbles or boulders within the glacial soils, and for variable pile lengths (although further investigation is required in this regard at the detail design stage).
- **Driven steel pipe (tube) piles:** Steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutments as well as associated wing walls/retaining walls and the centre pier at this site. However, pipe piles are considered to have a slightly 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 suitable option at this site given the potential risks and difficulties associated with the artesian groundwater pressures in the sand to sandy silt and lower sand and silt to sandy silt till deposits.

The following sections provide recommendations for both spread footing foundations and driven steel H-pile or steel tube pile foundations to support the proposed replacement structure. However, from a foundations perspective, deep foundations are the preferred alternative at this site, given the depth of excavation that would be required for shallow foundations relative to the Highway 401 and 6th Line grades.

6.3 Shallow Foundations

6.3.1 Founding Elevations

For support of the new abutments, associated wing walls/retaining walls and centre pier, strip or spread footings should be founded below any fill, softened surficial soils, and the stiff portion of the till deposit, on the very stiff to hard clayey silt till. The following maximum (highest) founding elevations are recommended for preliminary design of footings founded on very stiff to hard clayey silt till.

Foundation Element	Maximum (Highest) Founding Elevation	Approximate Excavation Depth
North abutment	189.5 m	1.5 m – relative to original ground 3.0 m – relative to Highway 401 grade 9.5 m – relative to 6 th Line grade
Centre pier	187.5 m	5.0 m – relative to Highway 401
South abutment	187.5 m	3.5 m – relative to original ground 5.0 m – relative to Highway 401 grade 11.5 m – relative to 6 th Line grade



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 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, soft to stiff clayey silt till 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 surficial soils), at or below the preliminary 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.

Founding Stratum	Footing Width	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS*
North abutment	3 m	500 kPa	450 kPa
	4 m	600 kPa	400 kPa
Centre pier and south abutment	3 m	600 kPa	450 kPa
	4 m	650 kPa	400 kPa
All locations (Following subexcavation and replacement with compacted granular fill)	3 m	700 kPa	350 kPa
	4 m	750 kPa	350 kPa

* For 25 mm of settlement

The geotechnical resistances should be reviewed if the selected footing width or founding elevation differ 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 and centre pier locations.



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

6.4.1 Founding Elevations

The abutments, associated wing walls/retaining walls and centre pier 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 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 abutment	170.5 m
Centre pier	171.5 m
South abutment	171.5 m

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

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the soil 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 OPSP 3001.100 Type II (*Steel Tube Pile Driving Shoe*).

The groundwater pressure associated with the sand to sandy silt and lower cohesionless till deposits was observed to be 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 310x110 piles driven to the estimated tip elevations provided in Section 6.4.1, the factored axial resistance at ULS may be taken as 1,600 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 6th Line embankment widening. The depth and extent of stripping should be assessed during detail design when additional subsurface information will be available for the approach embankment areas.

The new fill 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 6th Line embankment 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 6th Line 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 must 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 embankment 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 embankments on this project, considering the design requirements and the available field and laboratory testing data.

The stability analyses were completed for a 9 m high embankment based on the subsurface conditions as encountered in Boreholes 10-501 and 10-502. The following parameters have been used in the analyses, based on field and laboratory test data as well as accepted correlations:



Soil Deposit	Bulk Unit Weight (kN/m ³)	Effective Friction Angle	Undrained Shear Strength (kPa)
Embankment fill	21	32-35°	-
Very stiff to hard clayey silt till, including compact to dense interlayers	21	32°	-
Very stiff to hard clayey silt	20	32°	-
Loose to compact sand to sandy silt	20	32°	-
Compact to very dense sand and gravel to sandy silt till	21	35°	-

The analysis results indicate that a 9 m high embankment with side slopes no steeper than 2H:1V will have a factor of safety of greater than 1.3 against global instability, assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials. An example static global stability result is provided on Figure 1. This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed based on the actual subsoil conditions encountered within the proposed approach embankment footprints during detail design.

6.5.3 Approach Embankment Settlement

The new 6th Line underpass is proposed to be constructed along the same alignment as the existing structure, with a grade raise of approximately 1 m adjacent to the new abutment locations, and some minor widening of the existing embankments to accommodate the grade raise.

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

Soil Deposit	Bulk Unit Weight (kN/m ³)	Elastic Modulus (MPa)
Embankment fill	21	-
Stiff clayey silt till	21	25
Very stiff to hard clayey silt till, including compact to dense interlayers	21	80
Very stiff to hard clayey silt	20	60
Loose to compact sand to sandy silt	20	30
Compact to very dense sand and silt to sandy silt till	21	90



Assuming an approximately 1 m grade raise and approximately 2 m of widening on each side of the existing embankment to accommodate the grade raise, the settlement of the foundation soils under the 6th Line approach embankments is estimated to be approximately 10 mm to 20 mm. This settlement is expected to occur relatively quickly during and immediately following construction of the widened/raised approach embankments based on the nature of the soils at the site. This estimated magnitude of settlement should be reassessed based on the subsurface conditions under the widened approach embankments as determined during the detail design.

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 widening, 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 about 1.5 m to 3.5 m below the natural ground surface, or about 3 m to 5 m below the Highway 401 grade and 9.5 m to 11.5 m below the 6th Line grade, through the existing firm to stiff fill and the stiff portion of the clayey silt till, into very stiff to hard clayey silt till. The excavations for pile caps could be maintained higher within the 6th Line embankment or Highway 401 embankment fill.

Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. Existing fill and interlayers within the till should be classified as Type 3 soil, according to the OHSA, while the stiff to hard clayey silt till would be classified as a Type 2 material. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V.

At this preliminary stage, it is anticipated that temporary protection systems will be required along Highway 401 to facilitate construction of the centre pier foundations. Depending whether 6th Line is closed to traffic during construction or remains open, temporary protection systems may be required at the abutment locations. These temporary protection systems should be designed and constructed in accordance with OPSS 539 (*Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539.



It is considered that either a driven, interlocking steel sheetpile system or a soldier pile and lagging system would be suitable for the temporary protection system(s) at this site, based on the subsurface soil and groundwater conditions.

6.6.2 Groundwater Control

Groundwater seepage is anticipated from cohesionless soil interlayers within the till deposit (where these are present), and from groundwater “perched” on top of the till deposit within existing granular fill. For the potential depth of excavation associated with spread footings or pile caps, the seepage volume 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. Based on these small seepage volumes, a Permit to Take Water (PTTW) should not be required for the groundwater control systems at this site.

As discussed in Section 6.4, the groundwater pressure associated with the sand to sandy silt and lower cohesionless till deposits was observed to be artesian relative to the natural ground surface, although further investigation and measurement will be required at the detail design stage to determine the stabilized groundwater level. Depending on the piezometric pressure relative to the pile cap level and the finished grade, specialized construction techniques may be required to mitigate the potential upward flow of water and fine soil particles along the pile shaft. Such measures could include construction of a filter blanket with subdrains or an impermeable plug under the pile cap to prevent loss of fine soil particles along the pile-soil interface.

6.6.3 Subgrade Protection

The clayey silt till (and any interlayers, if present) 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. 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 is not expected to be required during construction at this site.

6.7 Recommendations for Further Work During Detail Design

Additional boreholes will be required within each of the foundation elements and within the 6th Line approach embankment areas during the future detail design stage of investigation, to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided herein, as follows:

- Abutments and piers:
 - Assessment of the strength and deformation properties of the upper portion of the clayey silt till to confirm the founding elevation, bearing resistance and settlement for shallow foundations as well as for retained soil systems, if applicable.
 - 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.
 - Confirmation of the groundwater levels associated with the clayey silt till deposit and the deeper aquifer at this site. Depending on the piezometric pressure associated with the deeper aquifer, it may be necessary to assess the potential for base heave in foundation excavations, and/or to include measures for mitigation of water seepage and loss of fine soil particles along the pile-soil interface.
 - Observation of the presence of cobbles and/or boulders within the soil deposits, to assess the need for an NSSP to warn the contractor of the presence of such obstructions as they may affect excavations and the installation of driven steel H-pile or steel pipe pile foundations.
- Approach embankments:
 - Assessment of the depth and extent of stripping of topsoil/organics and fill materials within the footprint of the widened 6th Line approach embankments.
 - More detailed assessment of the stability of the widened/raised 6th Line embankment as well as for retained soil systems or retaining walls, if applicable, once the final grades are established and based on additional borehole information within the approach embankment areas.
 - Further assessment of the estimated magnitude of settlement under the new approach embankments.



PRELIMINARY FOUNDATION REPORT - 6TH LINE UNDERPASS

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Ms. Nikol Kochmanová, P.Eng., 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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BMM/NK/LCC/TJG/jl

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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 |



PRELIMINARY FOUNDATION REPORT - 6TH LINE UNDERPASS

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 abutments, wing walls/retaining walls and centre pier 	<ul style="list-style-type: none"> Limited groundwater control as excavation will be within relatively impermeable clayey silt till deposit Allows for semi-integral abutments 	<ul style="list-style-type: none"> Significant excavations to extend through existing 6th Line or Highway 401 embankment fill and stiff portion of clayey silt till (where present); would require temporary protection systems 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 construction /maintenance costs may be higher due to non- integral abutment configurations Estimated cost is about \$95,000 per abutment and \$50,000 for the centre pier (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 fill	<ul style="list-style-type: none"> Not recommended for support of new abutments 	<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 	<ul style="list-style-type: none"> Moderate to high risk of potential settlement due to firm to stiff embankment fill and stiff portion of clayey silt till underlying perched abutment footings; settlement would be differential with respect to centre pier Precludes use of integral abutments; potentially greater maintenance required at abutments 	<ul style="list-style-type: none"> Conventional excavation and construction techniques 	<ul style="list-style-type: none"> This option not recommended



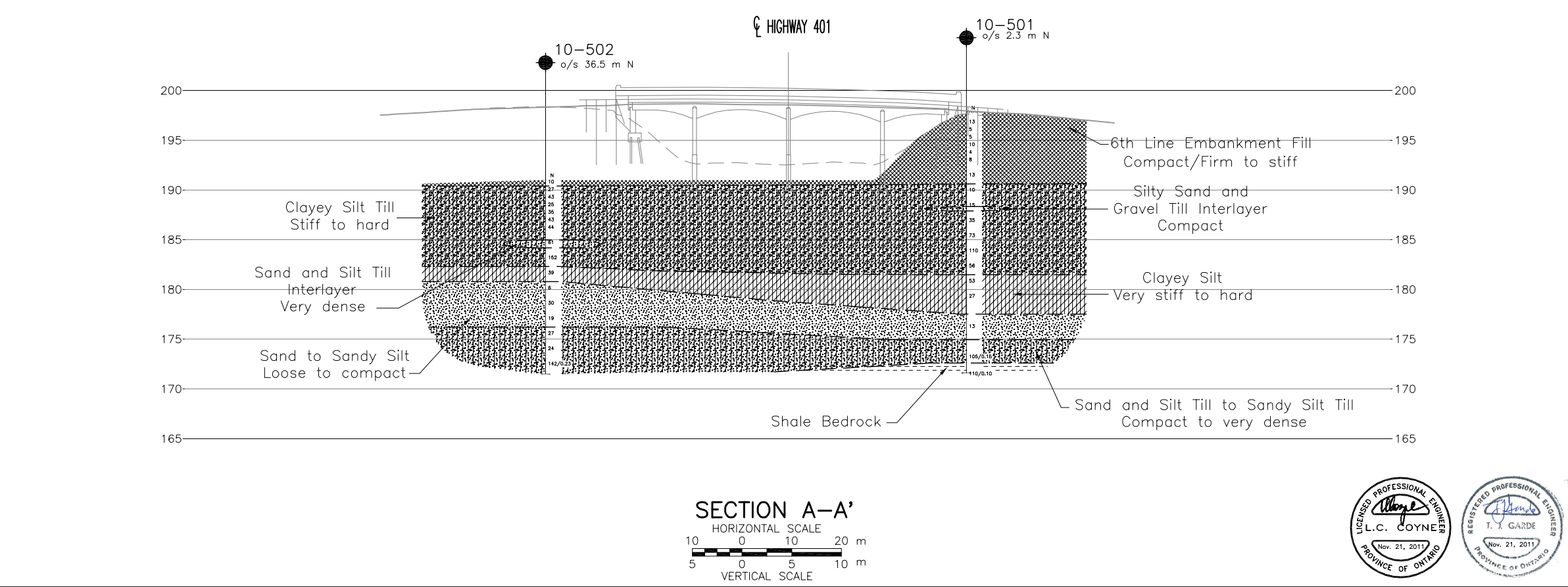
PRELIMINARY FOUNDATION REPORT - 6TH LINE UNDERPASS

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel H-piles driven to found in very dense lower deposit / bedrock	<ul style="list-style-type: none"> Feasible for support of abutments, wing walls/retaining walls and centre pier 	<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 integral abutment construction Would minimize differential settlement between foundation elements 	<ul style="list-style-type: none"> Minor risk of encountering obstructions (cobbles and/or boulders) during pile driving that 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 per abutment and \$135,000 for the centre pier, based on an estimated \$250/m length for pile installation and \$600/m³ for pile cap construction
Steel pipe (tube) piles (closed-end, concrete filled) driven to found in very dense lower deposit/ bedrock	<ul style="list-style-type: none"> Feasible for support of abutments, wing walls/retaining walls and centre pier 	<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 foundation elements 	<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 - 6TH LINE UNDERPASS

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Caissons founded in shale bedrock	<ul style="list-style-type: none">• Feasible but not recommended for support of abutments, wing walls/retaining walls and centre pier	<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• Higher capacity than for steel H-piles, so reduced number of deep foundation elements compared to steel H-piles	<ul style="list-style-type: none">• Artesian groundwater pressures create potential for loss of ground in sand to sandy silt and lower till deposits• 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 soil disturbance and loss of ground during construction	<ul style="list-style-type: none">• Higher cost compared with shallow foundations or steel H-piles

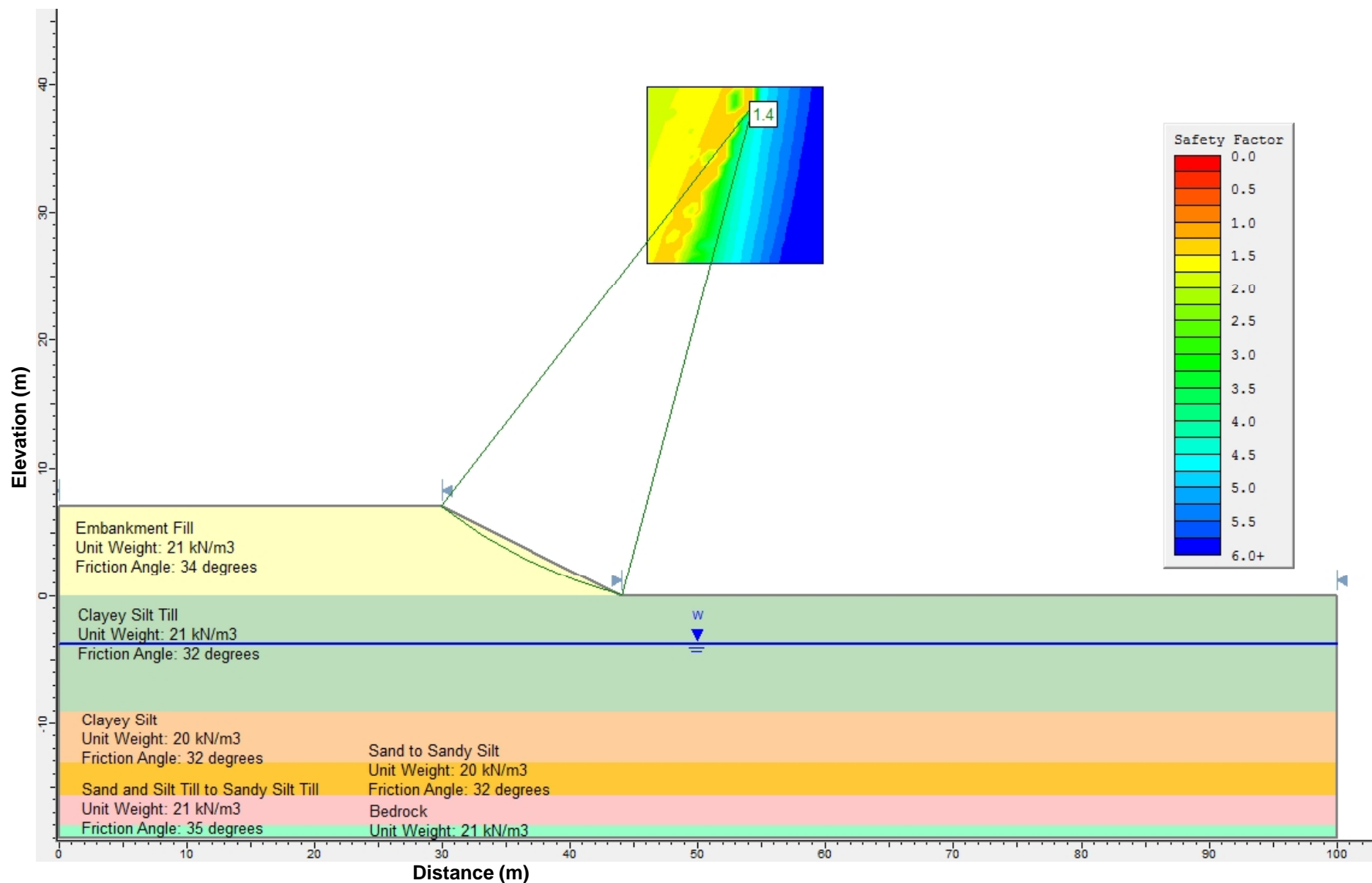


NO.	DATE	BY	REVISION	
Geocres No. 30M12-329				
HWY. 401		PROJECT NO. 09-1111-6036		DIST.
SUBM'D. NK	CHKD. LCC	DATE: 10/21/2011		SITE:
DRAWN: JFC/CD	CHKD. NK	APPD. LCC		DWG. 1



Static Global Stability – 6th Line Underpass Approach Embankment

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

1 OF 2 **METRIC**

CHECKED BY LCC

Continued Next Page

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

MIS-MTO 001 09-1111-6036.GPJ GAL-MISS.GDT 6/16/11 CD

PROJECT 09-1111-6036			RECORD OF BOREHOLE No 10-501			2 OF 2 METRIC																
W.O. 07-20024			LOCATION N 4824640.7 ; E 277338.8			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 17, 2011			CHECKED BY LCC																
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL			
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L 10 20 30			kN/m ³						
181.5	CLAYEY SILT with sand, trace to some gravel (TILL) Hard Brown Moist		13	SS	56		182															
16.3	CLAYEY SILT Very stiff to hard Grey Moist		14	SS	53		181															
							180															
			15	SS	27		179															
							178															
177.5	SAND, trace silt, trace clay Compact Grey Wet						177															
			16	SS	13		176															
							175															
174.9	Sandy SILT, some gravel, trace clay (TILL) Very dense Brown Moist						174															
			17	SS	105/0.15		173															
172.6	SHALE (BEDROCK) Reddish brown and grey						172															
25.2																						
171.6	END OF BOREHOLE AUGER REFUSAL ON INFERRED BEDROCK		18	SS	110/0.10																	
26.2	NOTES: 1. Water level in open borehole noted at a depth of 11.0 m (Elev. 186.8 m) upon completion of drilling.																					

PROJECT 09-1111-6036		RECORD OF BOREHOLE No 10-502		1 OF 2 METRIC	
W.O. 07-20024		LOCATION N 4824724.8 ; E 277303.2		ORIGINATED BY MS	
DIST Central HWY 401		BOREHOLE TYPE Track-Mounted CME 55, 108 mm Internal Diameter Hollow Stem Augers		COMPILED BY CS	
DATUM Geodetic		DATE December 13-14, 2010		CHECKED BY LCC	

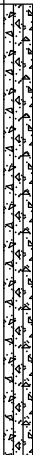
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)			
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED						
191.0	GROUND SURFACE													
0.0	TOPSOIL		1	SS	10									
190.4	Clayey silt, trace gravel, trace sand, containing rootlets (FILL)													
0.6	Stiff Brown Moist		2	SS	27		190							
	CLAYEY SILT with sand, trace to some gravel (TILL)													
	Very stiff to hard Brown Moist		3	SS	43		189			○	├───┤			
	Containing sand pockets below 2.2 m													
			4	SS	25									
							188							
			5	SS	35					○	├───┤		9 33 40 18	
	Augers grinding at 3.8 m.													
			6	SS	43		187							
			7	SS	44		186							
185.4														
5.6	SAND and SILT, trace gravel and clay (TILL)													
	Very dense Brown Moist		8	SS	61		185			○	H			
183.8							184							
7.2	CLAYEY SILT with sand, trace to some gravel (TILL)													
	Hard Brown Moist		9	SS	152		183							
182.3														
8.7	CLAYEY SILT, trace sand													
	Hard Grey Moist		10	SS	39		182			├───┤			0 1 65 34	
180.8							181							
10.2	Sandy SILT, trace clay													
	Loose to compact Brown Wet		11	SS	6		180							
							179							
			12	SS	30					○			0 28 64 8	
							178							
			13	SS	19		177							
176.2														
14.8														

Continued Next Page

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

MIS-MTO 001 09-1111-6036.GPJ GAL-MISS.GDT 6/16/11 CD

PROJECT <u>09-1111-6036</u>		RECORD OF BOREHOLE No 10-502		2 OF 2 METRIC	
W.O. <u>07-20024</u>		LOCATION <u>N 4824724.8 ;E 277303.2</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 13-14, 2010</u>		CHECKED BY <u>LCC</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	W _p	W	W _L					
	--- CONTINUED FROM PREVIOUS PAGE ---																			
	SAND and SILT, some gravel, trace clay (TILL) Compact to very dense Brown Wet		14	SS	27															
					15	SS	24													
			16	SS	142/0.23															
171.5 19.5	END OF BOREHOLE AUGER REFUSAL ON POSSIBLE BEDROCK																			
	NOTES: 1. Artesian groundwater pressures observed at a depth of 13.7 m (Elev. 177.3 m) after leaving borehole overnight. Groundwater continued to flow from borehole as borehole was drilled deeper. 2. Borehole abandoned by grouting with cement-bentonite grout.																			



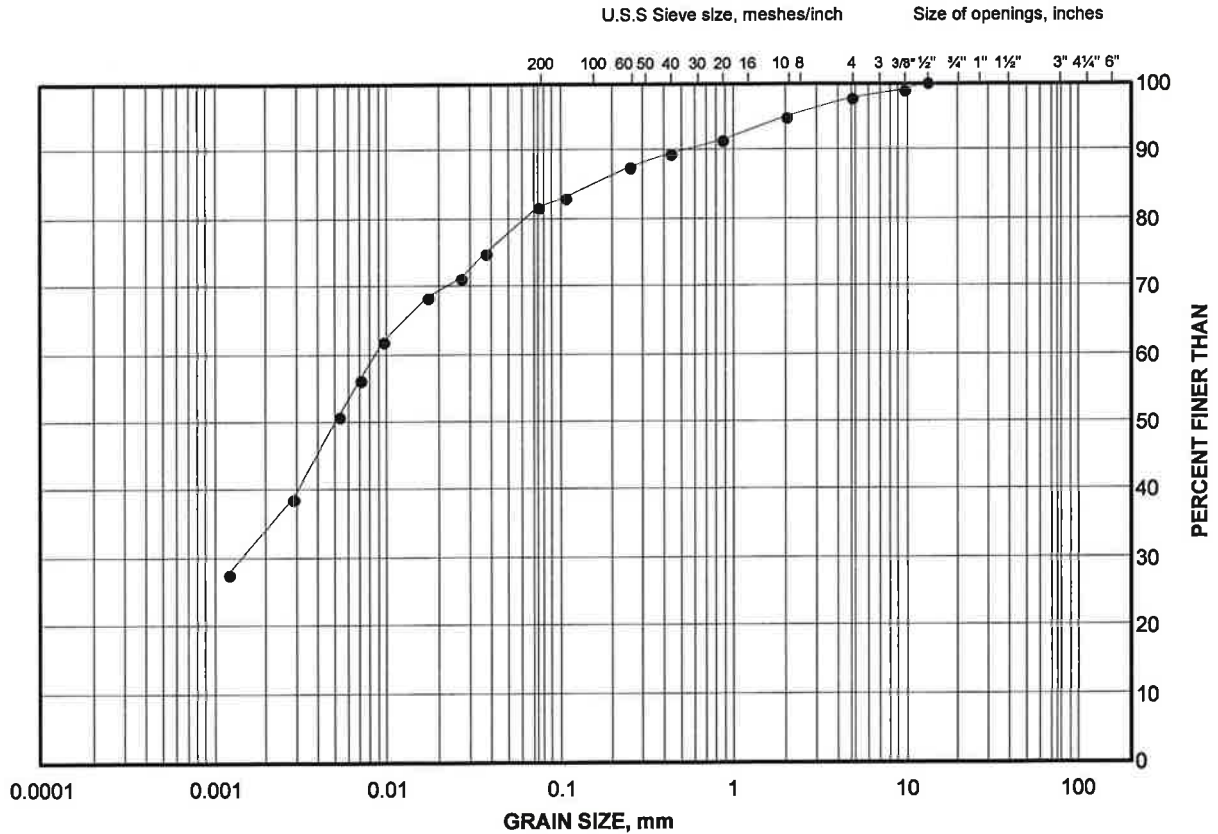
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay Fill

FIGURE B1



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

LEGEND

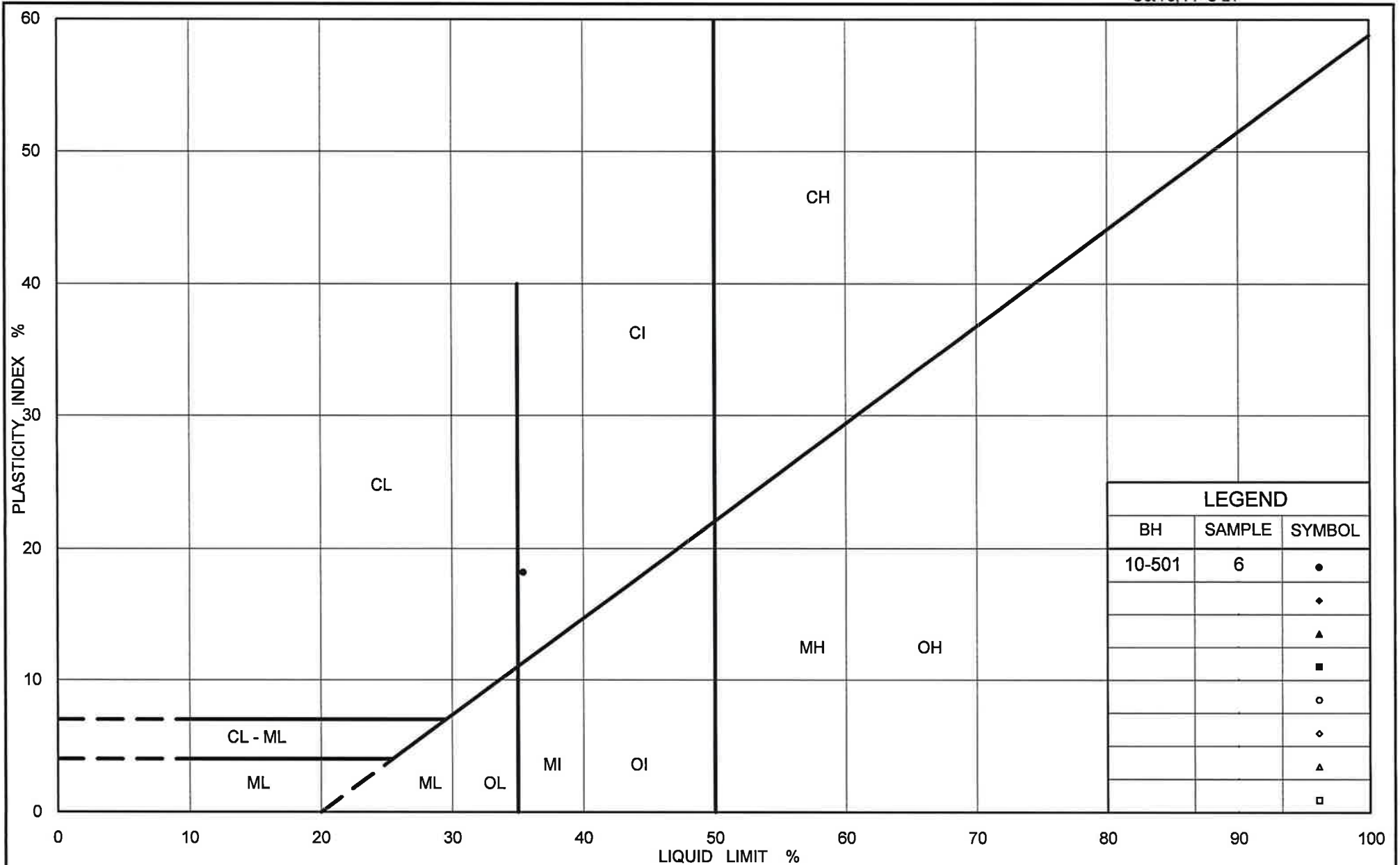
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	10-501	4	194.5

Project Number: 09-1111-6036

Checked By: *Wayne*

Golder Associates

Date: 08-Apr-11



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PLASTICITY CHART Clayey Silt to Silty Clay Fill

Ontario

Figure No. B2

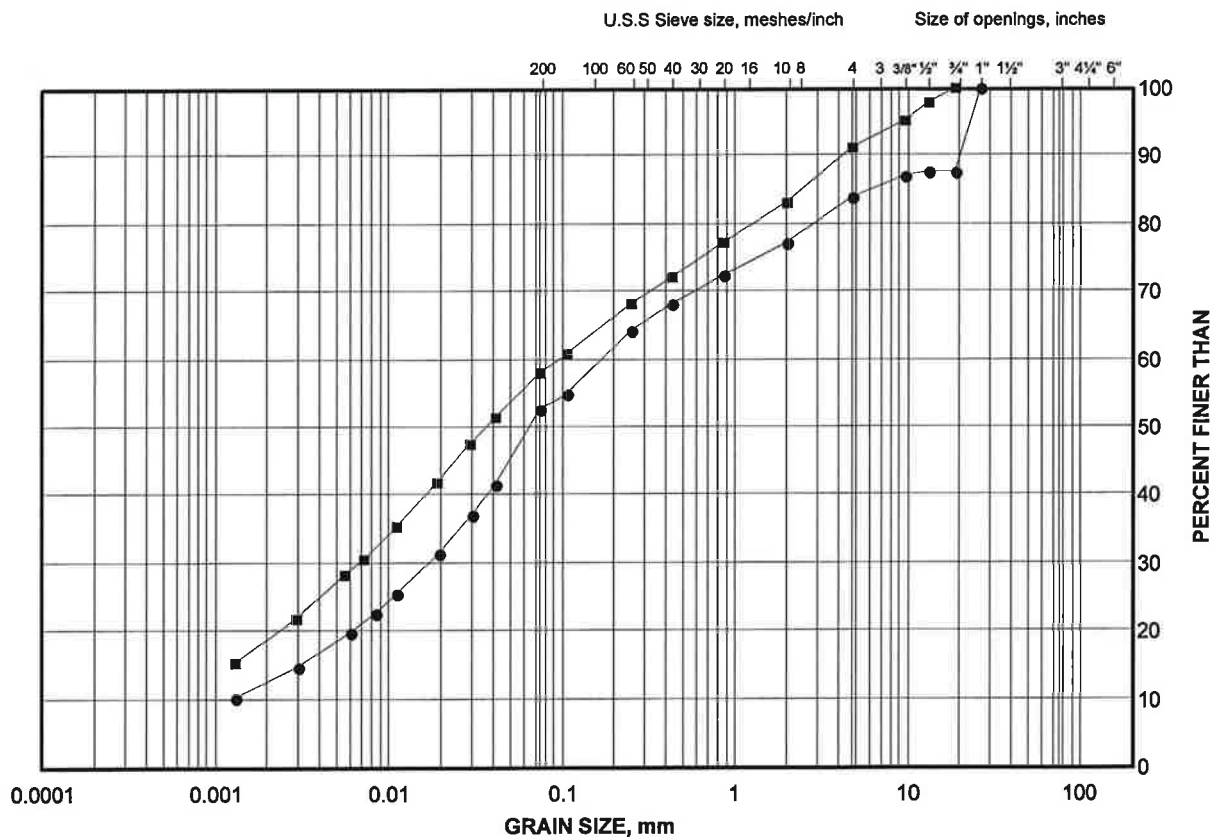
Project No. 09-1111-6036

Checked By: *[Signature]*

GRAIN SIZE DISTRIBUTION

Clayey Silt Till

FIGURE B3



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

LEGEND

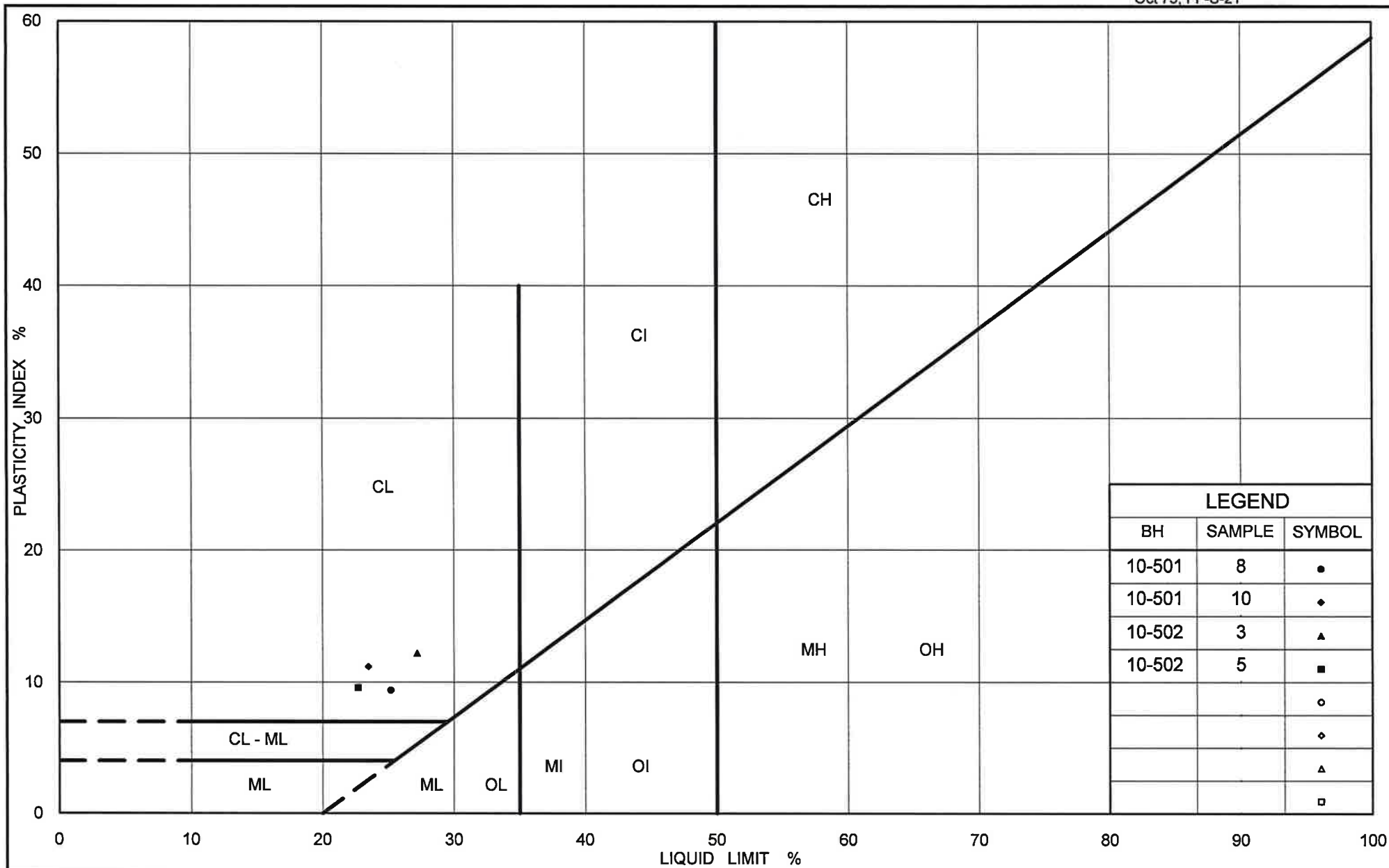
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-501	11	185.3
■	10-502	5	187.7

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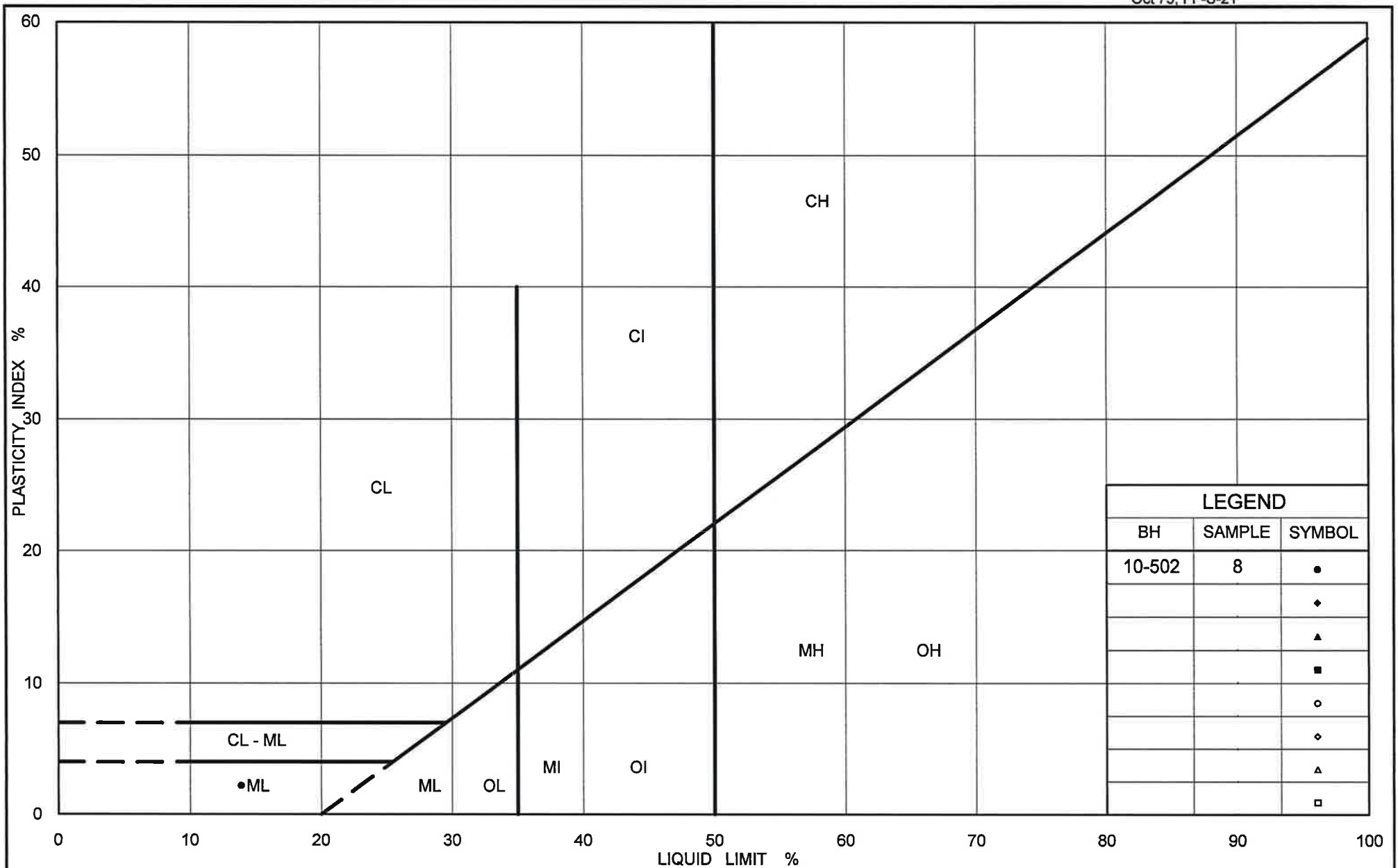
Ontario

PLASTICITY CHART **Clayey Silt Till**

Figure No. B4

Project No. 09-1111-6036

Checked By: *Wayne*



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PLASTICITY CHART Sand and Silt Till Interlayer

Ontario

Figure No. B5

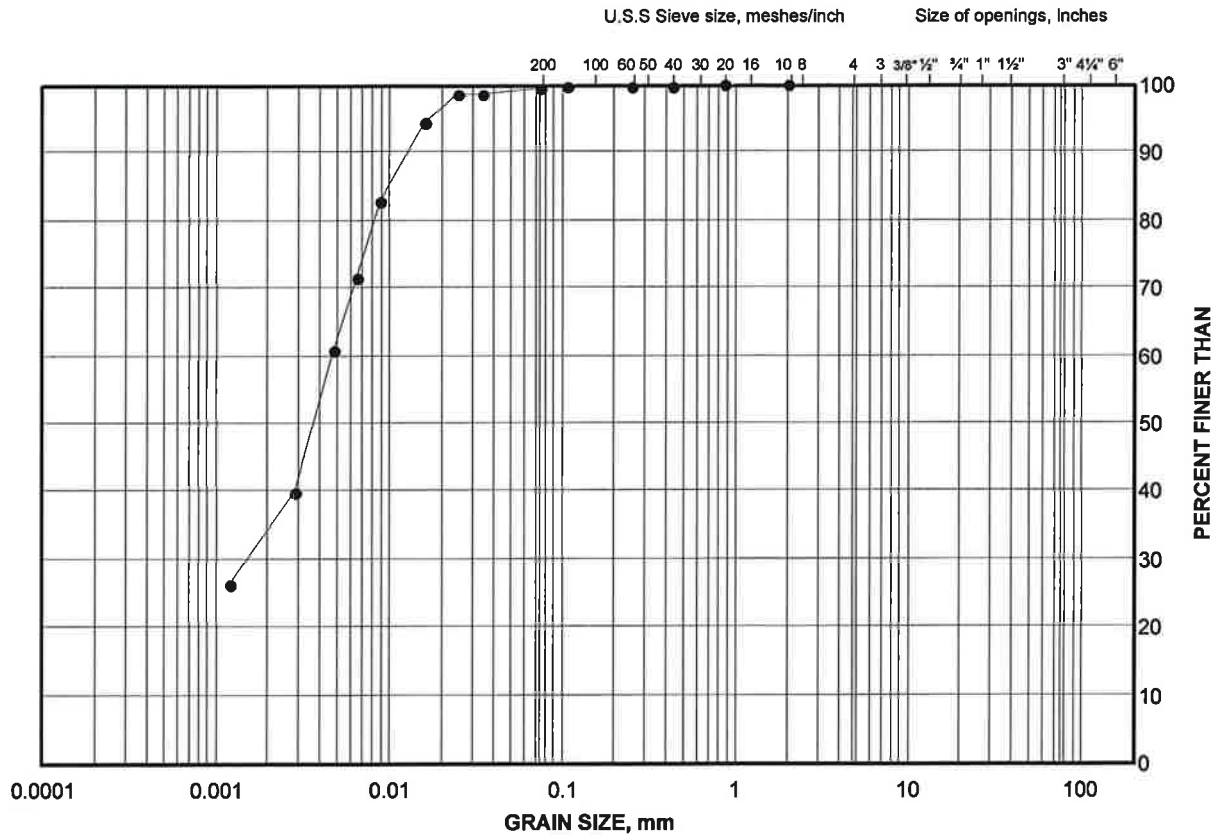
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GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE B6



LEGEND

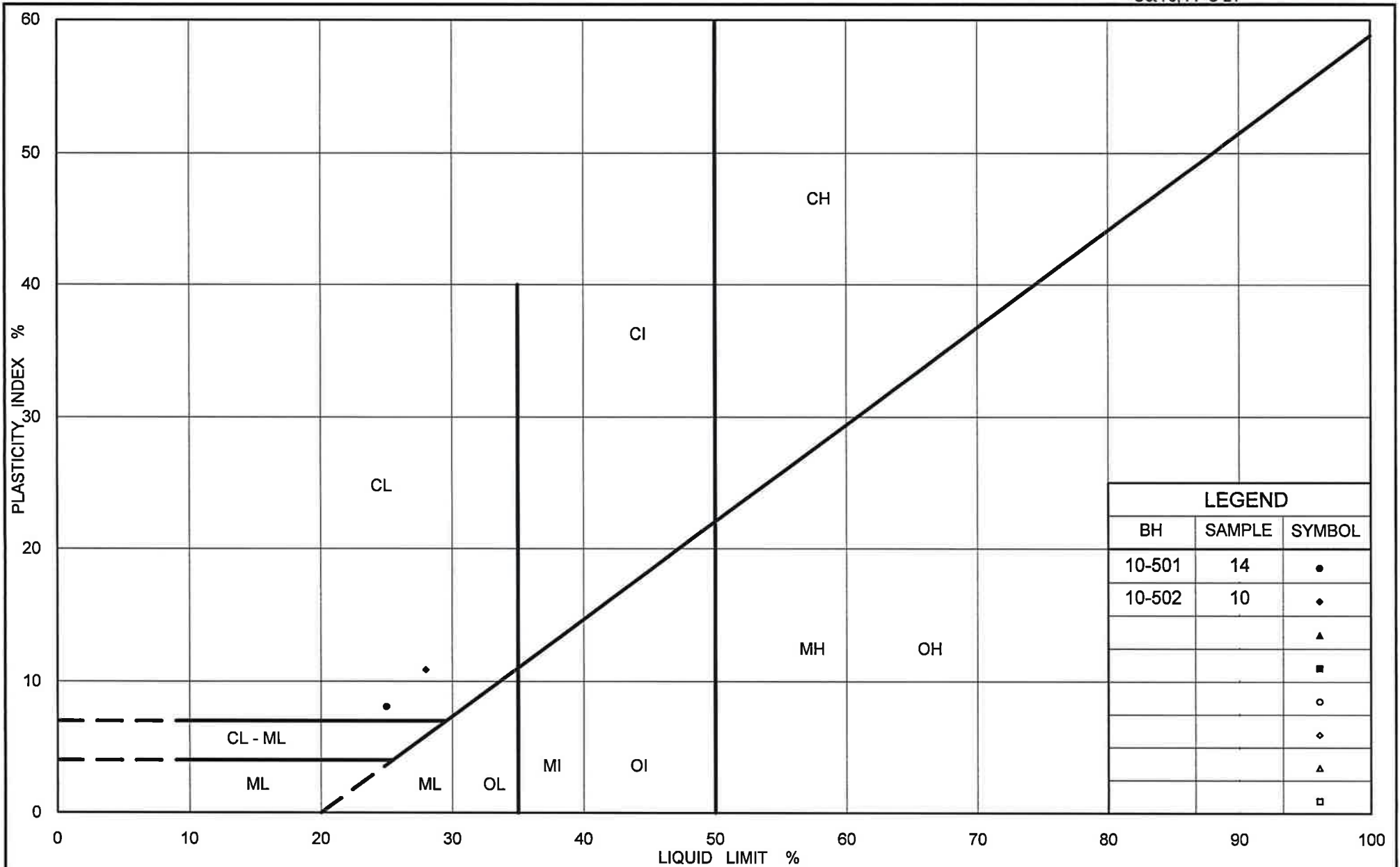
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	10-502	10	181.6

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PLASTICITY CHART Clayey Silt

Figure No. B7

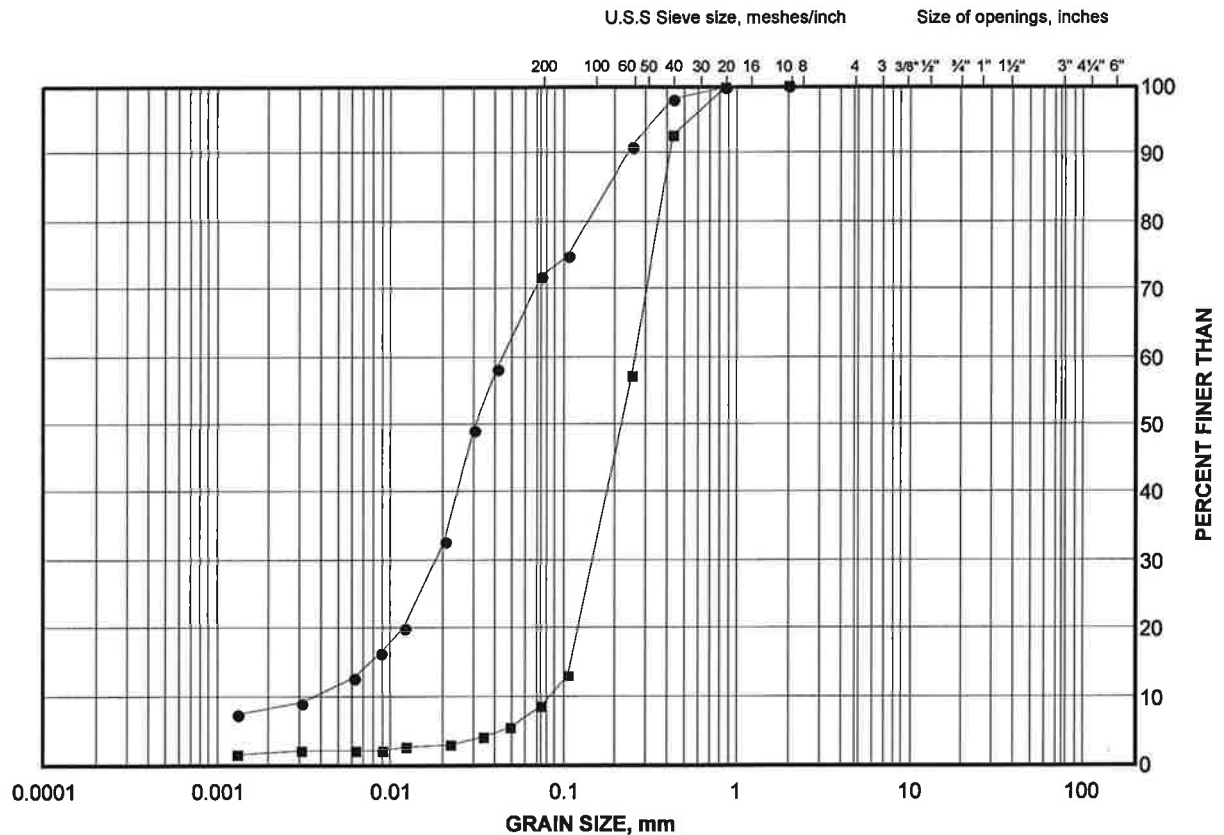
Project No. 09-1111-6036

Checked By: *Woyce*

GRAIN SIZE DISTRIBUTION

Sand to Sandy Silt

FIGURE B8



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-502	12	178.5
■	10-501	16	177.7

Project Number: 09-1111-6036

Checked By: *Moyle*

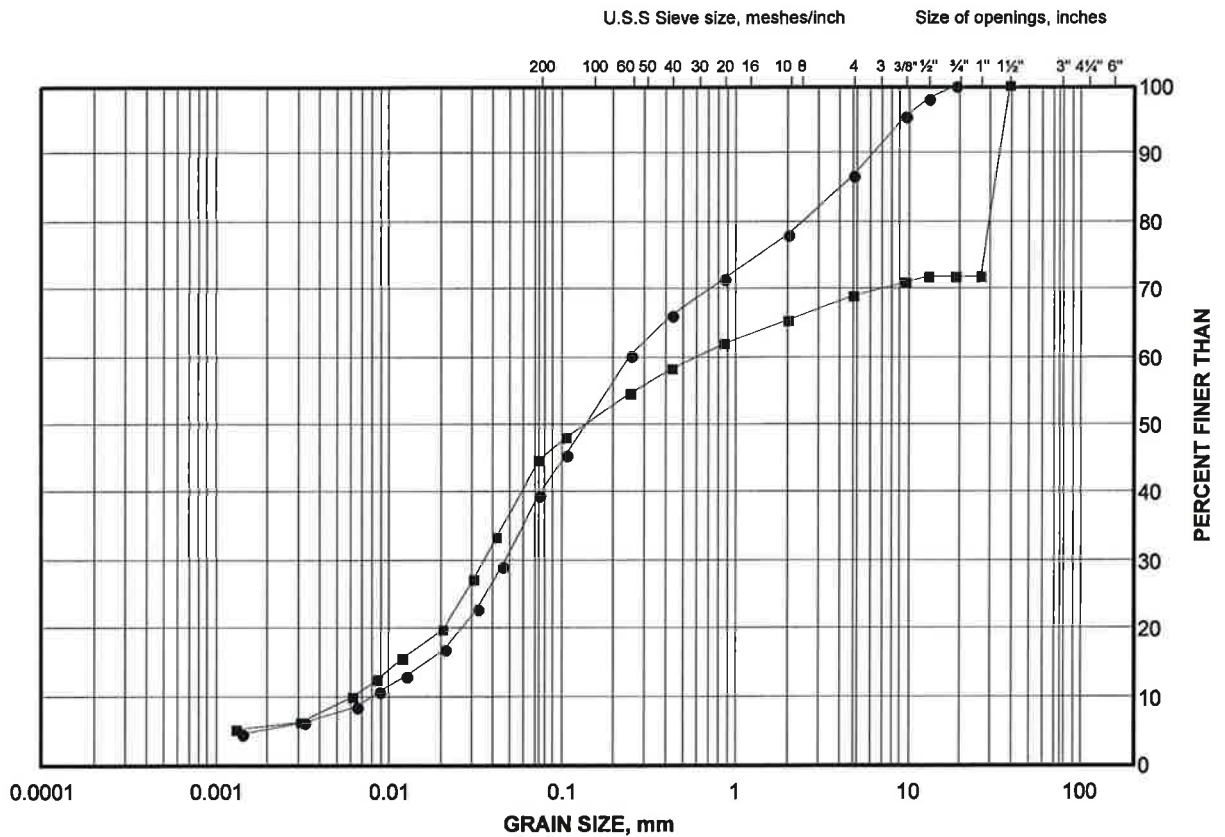
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GRAIN SIZE DISTRIBUTION

Sand and Silt Till to Sandy Silt Till

FIGURE B9



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-502	15	173.9
■	10-501	17	176.2

Project Number: 09-111-6036

Checked By: *Mayer*

Golder Associates

Date: 08-Apr-11

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