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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

5th Line Overpass Highway 401 Widening from Trafalgar Road to Regional Road 25, Halton Region W.O. 07-20024

Submitted to:
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REPORT

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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
5th LINE OVERPASS
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 5th Line overpass 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 5th Line overpass is located at the intersection of Highway 401 and 5th Line, approximately 5 km east of Regional Road 25 and 3 km west of Trafalgar Road in the Regional Municipality of Halton, Ontario. The existing structure consists of a 10.4 m long, single-span overpass, with the existing abutments supported on spread footings.

In general, the terrain in this area is relatively flat, with the natural ground surface in the immediate vicinity of the structure site at about Elevation 194 m. The ground surface declines eastward toward the West Branch of Oakville Creek, which is located about 50 m to 75 m to the east of 5th Line.

The local road (5th Line) has been constructed near the original ground surface, with its grade below Highway 401 at approximately Elevation 195 m. Highway 401 has been constructed in embankment fill that is up to approximately 7.5 m in height, with the pavement grade at about Elevation 201.5 m at the structure site. The Highway 401 embankment side slopes are oriented at approximately 2 horizontal to 1 vertical (2H:1V).

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out on November and December 2010, during which time two boreholes (Boreholes 10-301 and 10-302) 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: Borehole 10-301 was advanced in the southeast quadrant from the 5th Line grade, adjacent to Highway 401 embankment toe, and Borehole 10-302 was advanced in the northwest quadrant through the Highway 401 embankment.

Boreholes 10-301 and 10-302 were drilled using 70 mm inner diameter hollow stem augers to depths of 16.8 m and 23.1 m, respectively. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth in the boreholes, using a 50 mm outside diameter split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedure.



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The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations. 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 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)
10-301	4,823,573.2	276,462.9	194.2	16.8
10-302	4,823,592.3	276,409.9	201.0	23.1

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 in the vicinity of the 5th Line overpass. 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 B6 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 thin surficial deposits of clayey silt and silty sand, overlying a deposit of clayey silt till to sand and silt till. A lower deposit of silt to sand and gravel was encountered below the till in one of the boreholes, while in the second borehole the clayey silt till is 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 200 mm of topsoil was encountered immediately below the ground surface in Borehole 10-301, which is located beside 5th Line near the toe of the Highway 401 embankment.

4.2.2 Fill

Approximately 0.7 m of fill was encountered immediately below the topsoil layer in Borehole 10-301, which was advanced near the south toe of the Highway 401 embankment immediately east of 5th Line. This fill extends to about Elevation 193.3 m, and consists of clayey silt containing trace sand and gravel as well as small amounts of organic matter (rootlets). This fill has a stiff consistency, based on one measured Standard Penetration Test (SPT) "N" value of 13 blows per 0.3 m of penetration.

Approximately 0.4 m of asphalt, 0.2 m of concrete, and 8.1 m of fill associated with the existing Highway 401 embankment was encountered in Borehole 10-302, extending to approximately Elevation 192.3 m. This fill consists of the following:

- An upper layer, about 1.5 m thick, of silty sand containing some gravel and trace clay. This fill has a loose to compact relative density based on measured SPT "N" values of 7 blows and 13 blows per 0.3 m of penetration.
- A lower layer, about 6.6 m thick, of clayey silt with sand containing trace to some gravel. The result of a grain size distribution test completed on one selected sample of this fill is shown on Figure B1 in Appendix B. Atterberg limits testing was conducted on two selected samples of the cohesive fill and measured plastic limits of 15 and 18 per cent, liquid limits of 23 and 35 per cent, and plasticity indices of 8 and 17 per cent; these test results, which are plotted on a plasticity chart on Figure B2 in Appendix B, confirm that the cohesive fill consists of clayey silt of low plasticity. The measured SPT "N" values within this fill range from 3 blows to 14 blows per 0.3 m penetration, suggesting a soft to stiff consistency.



4.2.3 Surficial Clayey Silt

A 0.6 m to 1.5 m thick deposit of clayey silt was encountered immediately below the fill in both boreholes, with its base at Elevation 190.8 m to 192.8 m.

This surficial deposit consists of clayey silt containing some sand and trace gravel, as well as rootlets and organic matter; sand seams were observed in one of the recovered samples. Atterberg limits testing was carried out on one selected sample of the deposit and measured a plastic limit of 21 per cent, a liquid limit of 30 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, confirms that the deposit consists of clayey silt of low plasticity.

The measured SPT “N” values within the surficial clayey silt deposit are 12 blows and 14 blows per 0.3 m of penetration, suggesting a stiff consistency.

4.2.4 Surficial Silty Sand

A 0.5 m thick layer of silty sand was encountered immediately below the surficial clayey silt deposit in Borehole 10-301. The base of the silty sand layer is at Elevation 192.2 m as encountered in this borehole. The silty sand deposit contains trace gravel, pockets or lenses of clayey silt, and small quantities of organic material.

One SPT “N” value of 8 blows per 0.3 m of penetration was measured in this deposit, indicating a loose relative density.

4.2.5 Clayey Silt Till to Sand and Silt Till

An 11.3 m to 12.2 m thick till deposit was encountered below the surficial clayey silt and silty sand layers in both boreholes. The base of the till deposit was encountered in the boreholes between Elevation 178.6 m and 180.9 m.

The till is comprised predominantly of clayey silt with sand to some sand, containing trace to some gravel. In Borehole 10-301, the lower 3.8 m of the till grades to sand and silt containing some gravel and trace clay. The till deposit also contains an interlayer of clayey silt, which was encountered in one of the boreholes; this interlayer is discussed in the following section (Section 4.2.6).

The results of grain size distribution tests conducted on three selected samples of the till deposit – two of the clayey silt till, and one of the sand and silt till – are shown on Figure B4 in Appendix B. Atterberg limits testing was completed on four selected samples of the cohesive portion of the till and measured plastic limits of 12 to 17 per cent, liquid limits of 18 to 28 per cent, and plasticity indices of 6 to 11 per cent. These test results, which are plotted on a plasticity chart on Figure B5 in Appendix B, confirm that the cohesive portion of the till deposit is comprised of clayey silt of low plasticity.

The measured SPT “N” values within the till deposit range from 29 blows to 108 blows per 0.3 m penetration, indicating that the clayey silt till has a very stiff to hard consistency and the sand and silt portion of the till deposit has a dense to very dense relative density.



4.2.6 Clayey Silt Interlayer in Till Deposit

A 1.5 m thick interlayer of clayey silt was encountered within the clayey silt till in Borehole 10-302, between approximately Elevation 184.7 m and 183.2 m.

This interlayer consists of clayey silt containing trace sand; the result of the grain size distribution test completed on the recovered sample of this layer is plotted on Figure B4 in Appendix B, which also illustrates the grain size distribution test results for the remainder of the till deposit. Atterberg limits testing was completed on the recovered sample and measured a plastic limit of 18 per cent, a liquid limit of 28 per cent, and a plasticity index of 10 per cent. This result is plotted on a plasticity chart on Figure B5 in Appendix B, along with the Atterberg limits test results for selected clayey silt till samples; the result confirms that the interlayer consists of clayey silt of low plasticity.

The measured SPT “N” value within the clayey silt was 29 blows per 0.3 m of penetration, suggesting a very stiff consistency.

4.2.7 Lower Silt to Sand and Gravel

A lower cohesionless deposit was encountered below the clayey silt till to sand and silt till deposit in Borehole 10-301. The surface of this deposit was encountered at approximately Elevation 180.9 m, and the borehole was terminated in this deposit after penetrating it for 3.5 m (to Elevation 177.4 m).

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

The measured SPT “N” values in this deposit range from 117 blows per 0.3 m of penetration to 137 blows per 0.23 m of penetration, indicating a very dense relative density.

4.2.8 Shale Bedrock

Red shale bedrock was encountered below the till deposit in Borehole 10-302, and was inferred below the silt to sand and gravel deposit in Borehole 10-301, as summarized below. The bedrock surface is approximately 16.5 m to 17.5 m below the 5th Line grade and 22.5 m to 23.5 m below the Highway 401 grade.

Borehole No.	Approximate Bedrock Surface Elevation	Comments
10-301	178.6 m	Small amount of shale bedrock recovered using split-spoon sampler, with a measured SPT “N” value of 100 blows per 0.08 m of penetration.
10-302	177.4 m	Inferred based on split-spoon sampler refusal.



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4.3 Groundwater Conditions

Details of the water levels observed in the open boreholes at the time of drilling are summarized on borehole records contained in Appendix A.

In general, the cohesionless (sand and silt) portion of the till deposit and the lower silt to sand and gravel deposit were observed to be water-bearing during the drilling operations, and wet embankment fill soils were noted in Borehole 10-302. Based on the observed water levels, measured water contents and observation of changes in colour/weathering in the boreholes, the groundwater level in the till deposit is expected to be at approximately Elevation 190 m, although higher groundwater pressures may be associated with the lower silt to sand and gravel deposit, and "perched" groundwater may be encountered in surficial soil deposits. 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 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
5th LINE OVERPASS
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 Highway 401-5th Line overpass. 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 for the widening of Highway 401 from Regional Road 25 to Trafalgar Road in the Regional Municipality of Halton, it is understood that the future widening could consist of three additional lanes in both the eastbound and westbound directions on Highway 401. The existing 5th Line overpass will require widening to the north and south; full replacement of the existing structure is also under consideration.

The existing structure consists of a single-span overpass, with the existing abutments supported on spread footings. Based on the *General Layout* drawing for this structure, dated July 1957, the existing footing details are summarized below. This founding elevation is about 3.4 m below the original ground surface at the site, and about 4.6 m below the 5th Line grade.

Foundation Element	Footing Width	Founding Elevation
West abutment	2.4 m (8.0 ft.)	190.5 m (625.0 ft.)
East abutment	2.4 m (8.0 ft.)	190.5 m (625.0 ft.)

Based on the subsurface conditions, both shallow and deep foundation options have been considered for the north and south widening and/or replacement of the 5th Line overpass. 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:** Although the surficial clayey silt and silty sand deposits at the site are not suitable for support of shallow foundations, strip or spread footing are feasible for support of the widened abutments provided that they are founded below the surficial deposits on the very stiff to hard clayey silt till deposit (similar to the existing foundations). This would



require excavation to a depth of approximately 3.5 m to 4.5 m relative to the original ground surface and local road grade. Temporary excavation support 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 the foundation level, and along 5th Line if traffic is to be maintained on the local road during construction. 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.

- **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 preferred for support of the north and south widening, in part due to the presence of soft to stiff soils within the existing embankment fill (which would underlie a portion of the perched footings), and in part due to the potential settlement of the stiff/compact surficial soil layers under the widened 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 widenings 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. The use of driving shoes is recommended due to the hard/very dense nature of the soils and the potential presence of cobbles and boulders in the glacially derived soils.
- **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 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 feasible, but are not considered to be a practical option at this site given the potential risks and difficulties associated with the relatively high groundwater pressures in the lower silt to sand and gravel deposit.

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 steel H-pile foundations. Strip or spread footings are also feasible, although they present greater challenges and risks as compared with steel H-pile foundations, they are considered to be an acceptable alternative.

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 and any loose or soft to stiff surficial soils, on the very stiff to hard clayey silt till deposit. The following table provides the maximum (highest) founding elevations recommended for preliminary design of footings founded on the clayey silt till deposit. If spread footings are used for a widening, it is likely that they will be constructed to match the existing foundation elevation (approximately Elevation 190.5 m) and structurally



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connected. In the case of a full structure replacement, it may be feasible to found the footings 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.

Foundation Element	Borehole No.	Founding Stratum	Strip/Spread Footing Founding Elevation
North Widening or North Portion of Replacement	10-302	Very stiff to hard clayey silt till	Below 190.5 m
South Widening or South Portion of Replacement	10-301	Very stiff to hard clayey silt till	Below 192.0 m (or lower to match existing)

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, loose or soft to stiff surficial 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, 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 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 in the following table.



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Foundation Element	Footing Width	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS*
North Widening or North Portion of Replacement Structure	2.4 m (match existing)	600 kPa	500 kPa
	3 m	600 kPa	450 kPa
	4 m	650 kPa	400 kPa
South Widening or South Portion of Replacement Structure	2.4 m (match existing)	600 kPa	500 kPa
	3 m	600 kPa	450 kPa
	4 m	650 kPa	400 kPa
Replacement Structure (Following subexcavation and replacement with compacted granular fill)	3 m	600 kPa	450 kPa
	4 m	650 kPa	400 kPa

* For 25 mm of settlement

The 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 soils (with SPT "N" values of greater than 100 blows per 0.3 m of penetration) that were encountered below approximately Elevation 181 m. The following pile tip elevations may be used for preliminary design purposes, assuming about 2 m of penetration into the "100-blow" soil deposit.

Foundation Element	End-Bearing Stratum	Estimated Design Pile Tip Elevation
North Widening or North Portion of Replacement Structure	Hard clayey silt till	179.0 m
South Widening or South Portion of Replacement Structure	Very dense sand and gravel	179.0 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/or bouldery soils, as may be encountered 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*).

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 and the axial geotechnical resistance at SLS (for 25 mm of settlement) may be taken as follows for preliminary design.

Foundation Element	End-Bearing Stratum	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS*
North Widening – East and West Abutments	Hard clayey silt till	1,600 kN	1,400 kN
South Widening – East and West Abutments	Very dense sand and gravel	1,600 kN	1,400 kN

Similar axial resistances may be used in the design of closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.).

Pile installation should be in accordance with OPSS 903 (*Deep Foundations*). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO Standard Structural Drawing SS-103-11) during the final stages of driving to achieve the appropriate ultimate capacity.

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



6.5 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 approach 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 OPSS 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 7 m high embankment widening based on the subsurface conditions as encountered in Boreholes 10-301 and 10-302. 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°	-
Stiff surficial clayey silt	20	28°	-
Loose surficial silty sand	19	29°	-
Very stiff to hard clayey silt till	21	32°	-



PRELIMINARY FOUNDATION REPORT - 5TH LINE OVERPASS WIDENING

Soil Conditions	Bulk Unit Weight (kN/m ³)	Effective Friction Angle	Undrained Shear Strength (kPa)
Dense to very dense sand and silt till	21	34°	-
Very dense silt to sand and gravel	21	34°	

The analysis results indicate that an approximately 7 m high embankment widening with side slopes oriented no steeper than 2H:1V will have a factor of safety of greater than 1.3 against global instability, assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials. An example static global stability result is provided on Figure 1. This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed based on the subsoil conditions encountered within the proposed approach embankment footprints during detail design.

6.5.3 Approach Embankment Settlement

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

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

Soil Conditions	Bulk Unit Weight (kN/m ³)	Elastic Modulus (MPa)
Embankment fill	21	-
Stiff clayey silt	20	20 MPa
Loose silty sand	19	15 MPa
Very stiff to hard clayey silt till	21	75 MPa
Dense to very dense sand and silt till	21	75 MPa
Very dense silt	20	75 MPa
Very dense sand and gravel	20	100 MPa

Based on this preliminary assessment, the settlement of the foundation soils under the widened 7 m high approach embankments is estimated to be approximately 30 mm to 40 mm. This settlement is expected to occur relatively quickly during and immediately following construction of the widened approach embankments based on the nature of the soils at the site. This estimated magnitude of settlement should be reassessed based on the soil and groundwater conditions under the new approach embankments as determined during the detail design, with particular emphasis on the thickness and properties of any surficial soil deposits within the embankment widening footprint.



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

6.6 Construction Considerations

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

6.6.1 Excavation and Temporary Roadway Protection

The foundation excavations for spread footings would extend to a depth of about 3.5 m to 4.5 m below the original ground surface and local road grade, through the existing soft to very stiff clayey silt fill, firm to stiff surficial clayey silt, loose surficial silty sand, and into the very stiff to hard clayey silt till deposit. The excavations for pile caps could be maintained higher within the native soils or embankment fill, but would still require excavation through the existing fill and into the surficial clayey silt and silty sand soils.

Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill and surficial silty sand would be classified as Type 3 soils, 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 5th Line if traffic is to be maintained on the local road during construction. The temporary excavation support system should be designed and constructed in accordance with OPSS 539 (*Construction Specification for 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 adjacent Highway 401 structures, 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, where applicable, groundwater control. For a soldier pile and lagging system, it would be necessary to control seepage or include measures to mitigate loss of soil particles from the surficial silty sand deposit through the lagging boards.



6.6.2 Groundwater Control

Groundwater seepage is anticipated from the surficial silty sand deposit, as well as potentially from groundwater “perched” on top of the cohesive soils 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 system at this site.

6.6.3 Subgrade Protection

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

6.6.4 Obstructions

The soils at this site are glacially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of deep foundations or protection systems. Further observation is recommended in the next stage of investigation in support of the detail design. If conditions warrant, an NSSP should be included in the Contract Documents developed during the detail design stage to identify to the contractor the possible presence of cobbles and/or boulders within the overburden soils.

6.6.5 Vibration Monitoring During Pile Installation

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

6.7 Recommendations for Further Work in Detail Design

Additional boreholes will be required within each of the abutment widening areas and the approach embankment widening areas during the future detail design stage of investigation, to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided in this report, as follows:

- Abutments:
 - Assessment of the variability of the surficial clayey silt and silty sand soils to confirm the founding elevation for spread footings within each widened abutment area, and to assess groundwater control requirements associated with seepage from the surficial cohesionless soil deposits.



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

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 S. Devata, P.Eng., a senior geotechnical/foundations consultant at Golder. Mr. Ty Garde, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

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NK/LCC/TJG/nk

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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 - 5TH LINE OVERPASS WIDENING

TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Spread/strip footings on very stiff to hard clayey silt till	<ul style="list-style-type: none"> Feasible for support of widened abutments and associated wing walls/retaining walls 	<ul style="list-style-type: none"> Existing structure supported on shallow foundations, and has performed well Relatively minor groundwater seepage anticipated Allows for semi-integral abutments Lower vibration impacts on existing structures than for driven steel H-pile installation 	<ul style="list-style-type: none"> Significant excavations (to a depth of 3.5 m to 4.5 m below the original ground surface and local road grade) to extend below surficial clayey silt and silty sand soils; would require temporary excavation support Moderate potential for up to about 15 mm of differential settlement between existing overpass and widening Precludes use of integral abutments; potentially greater maintenance required at abutments Lower geotechnical resistances as compared with deep foundations 	<ul style="list-style-type: none"> Conventional excavation and construction techniques 	<ul style="list-style-type: none"> Less expensive than deep foundations although bridge maintenance costs may be higher due to non-integral abutment configuration Estimated cost is \$68,000 per foundation area (i.e., southeast abutment, southwest abutment, etc.) based on an estimated \$600/m³ for construction of shallow foundations, excluding deeper excavation and temporary protection system
Spread/strip footings perched on compacted granular pad in approach embankment fill	<ul style="list-style-type: none"> Not recommended for support of widened abutments 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than footings founded on till deposit, significantly reducing subexcavation depth and associated temporary protection system requirements 	<ul style="list-style-type: none"> Potential for differential settlement between the existing overpass and the widening is greater than for shallow foundations on till or steel H-piles due to settlement in the stiff surficial clayey silt and loose surficial silty sand under the approach embankment loading 	<ul style="list-style-type: none"> Conventional excavation and construction techniques 	<ul style="list-style-type: none"> This option not recommended



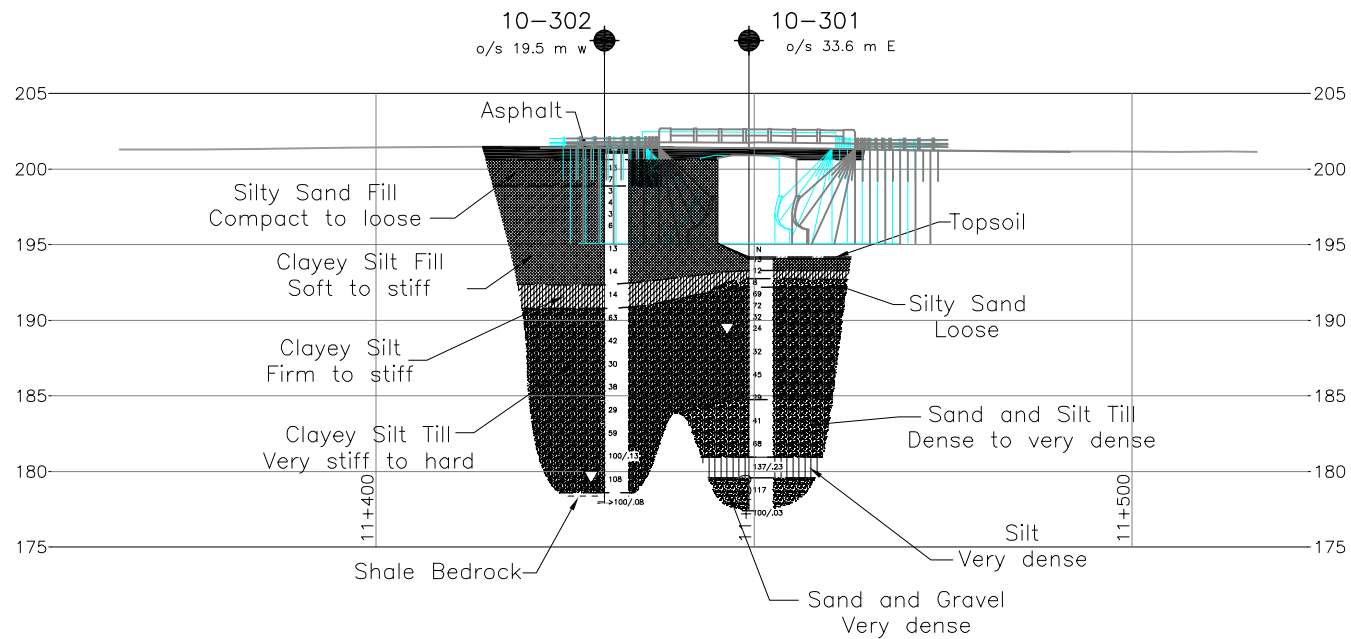
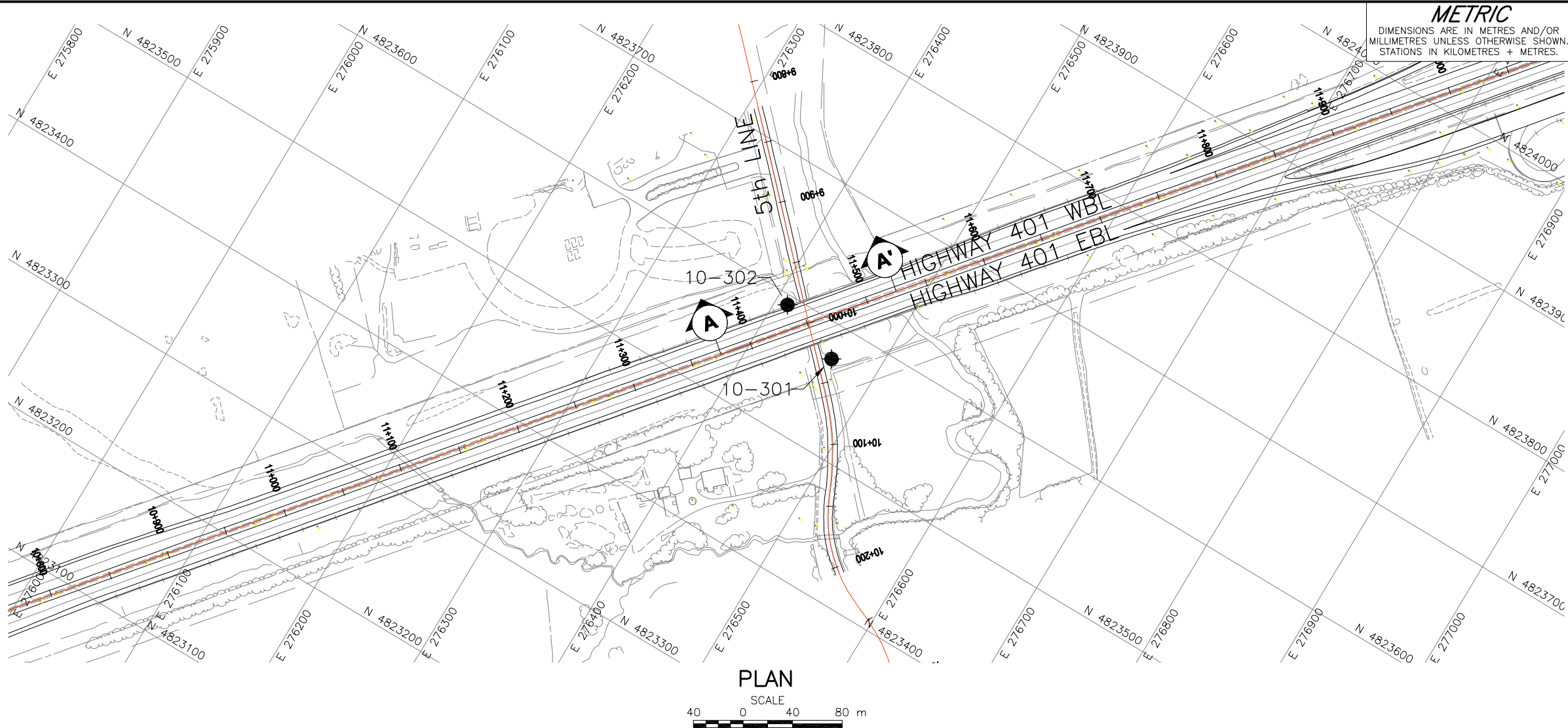
PRELIMINARY FOUNDATION REPORT - 5TH LINE OVERPASS WIDENING

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel H-piles driven to found in hard/very dense lower deposit	<ul style="list-style-type: none">• Feasible for support of widened or new abutments and associated wing walls/retaining walls	<ul style="list-style-type: none">• Abutment pile caps could be maintained higher than footings founded on till deposit, reducing depth of excavation and temporary excavation support requirements adjacent to Highway 401 and 5th Line• 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 overpass and widened portions of structure	<ul style="list-style-type: none">• Minor potential for encountering obstructions (cobbles and/or boulders) during pile driving; this could result in piles “hanging up” and lower geotechnical resistances	<ul style="list-style-type: none">• Conventional construction methods for H-pile foundations	<ul style="list-style-type: none">• Lower relative cost compared with caisson option• Estimated cost is approximately \$91,500 per foundation area (i.e., southeast abutment, southwest abutment, etc., based on an estimated \$250/m length for pile installation and \$600/m³ for pile cap construction

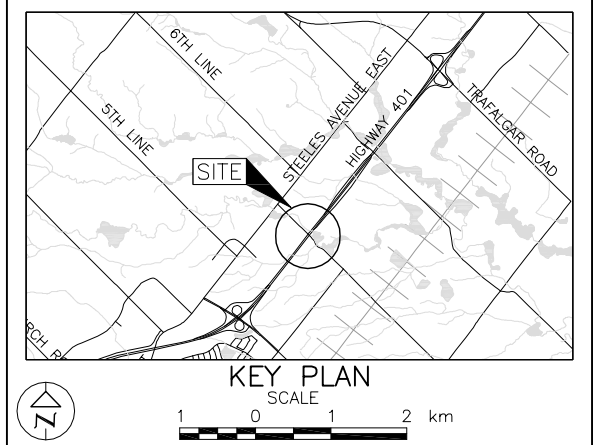


PRELIMINARY FOUNDATION REPORT - 5TH LINE OVERPASS WIDENING

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel pipe (tube) piles, driven to found in hard/very dense lower deposit	<ul style="list-style-type: none"> Feasible for support of widened or new abutments and associated wing walls/ retaining walls 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than footings founded on dense to very dense soils, reducing depth of excavation and temporary protection system requirements adjacent to Highway 401 and 5th Line Limited groundwater control required Allows for semi-integral abutment configuration if existing structure can be modified, or if replacement is adopted 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
Caissons founded in very dense lower deposit	<ul style="list-style-type: none"> Feasible but not recommended for support of abutments and wing walls/ retaining walls 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than footings founded on till deposit, reducing depth of excavation and temporary excavation support requirements adjacent to Highway 401 and 5th Line 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"> High groundwater pressures create potential for loss of ground or base heave in lower silt to sand and gravel deposit Temporary or permanent liners would be required; likely not possible to inspect caisson base Precludes use of integral abutments 	<ul style="list-style-type: none"> Risk of loosening or heaving soils at base of caissons 	<ul style="list-style-type: none"> Higher cost compared with shallow foundations or steel H-piles

CONT No.
WO No. 07-200245th LINE OVERPASS
HIGHWAY 401 WIDENING
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

**Golder Associates Ltd.**
MISSISSAUGA, ONTARIO, CANADA**LEGEND**

- Borehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
10-301	194.2	4823573.2	276462.9
10-302	201.0	4823592.3	276409.9

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).

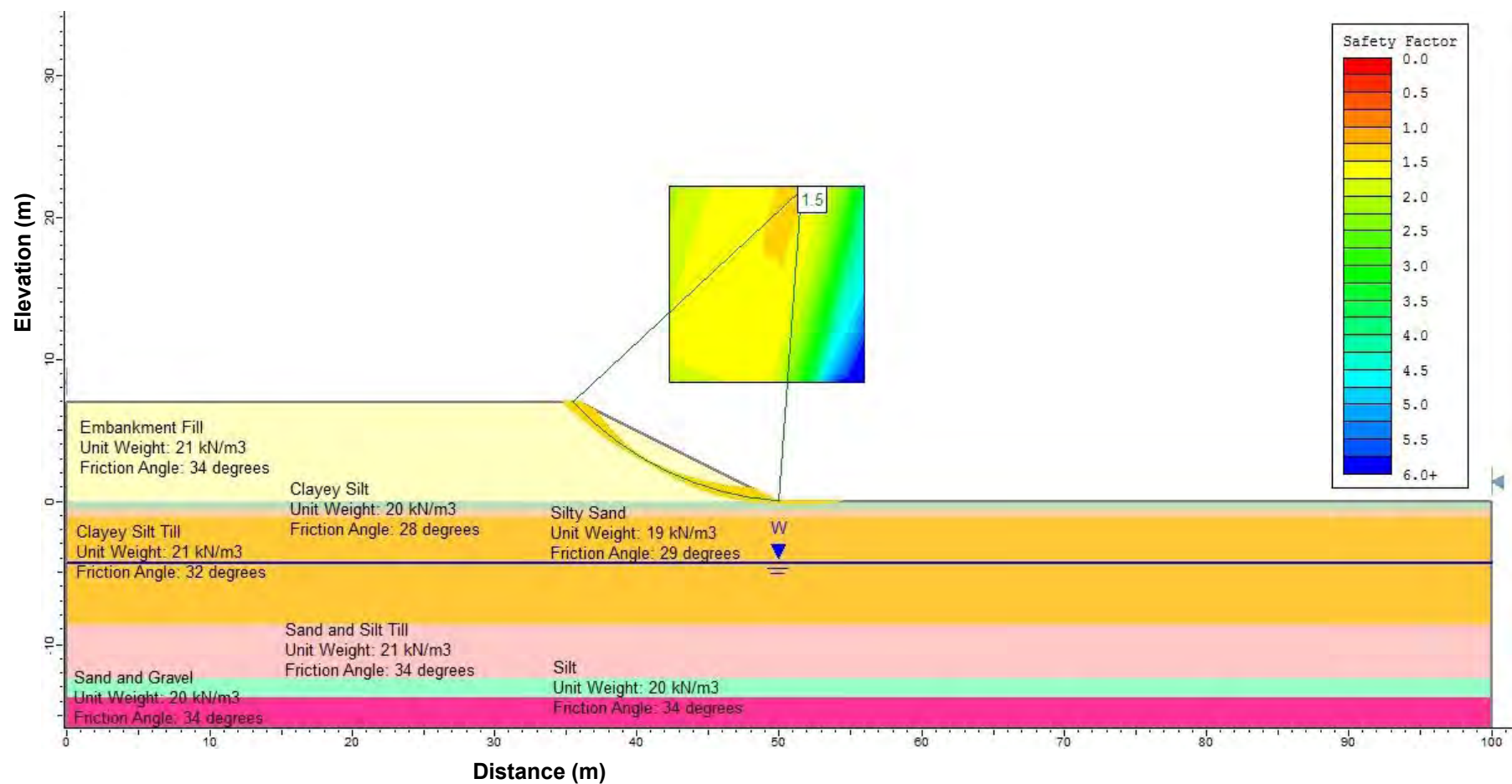


NO.	DATE	BY	REVISION
Geocres No.	30M12-327		
HWY. 401		PROJECT NO. 09-1111-6036	DIST.
SUBM'D. NK	CHKD. LCC	DATE: 10/21/2011	SITE:
DRAWN: JFC/CD	CHKD. NK	APPD. LCC	DWG. 1



Static Global Stability – 5th 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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d :

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

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

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils Consistency

	C_u, S_u	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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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



LIST OF SYMBOLS

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

I. GENERAL

π	3.1416
$\ln x$,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - \mu$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
μ	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

T_p, T_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

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

Notes: 1 $\tau = c' + \sigma' \tan \phi'$
2 shear strength = (compressive strength)/2


PROJECT 09-1111-6036		RECORD OF BOREHOLE No 10-301		1 OF 2		METRIC	
W.O. 07-20024		LOCATION N 4823573.2 ;E 276462.9		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 26, 2010		CHECKED BY		LCC	

[illegible]

Continued Next Page

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

MIS-MTO 001 09-1111-6036.GPJ GAL-MISS.GDT 4/29/11 CD

PROJECT 09-1111-6036		RECORD OF BOREHOLE No 10-301				2 OF 2 METRIC											
W.O. 07-20024		LOCATION N 4823573.2 ; E 276462.9				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 26, 2010				CHECKED BY LCC											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100						
	SAND and GRAVEL, some silt, trace clay Very dense Brown Wet		14	SS	117												39 43 13 5
177.4	END OF BOREHOLE		15	SS	100.08												
16.8	SPLIT-SPOON SAMPLER REFUSAL ON POSSIBLE BEDROCK NOTE: 1. Water level in hollow stem augers at a depth of 5.2 m (Elev. 189.0 m) upon completion of drilling.																

RECORD OF BOREHOLE No 10-302

1 OF 2 **METRIC**

PROJECT 09-1111-6036

W.O. 07-20024

LOCATION N 4823592.3 ; E 276409.9

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 December 1, 2010

CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)									
201.0	GROUND SURFACE							20	40	60	80	100	W _P	W	W _L	GR	SA	SI	CL
0.0	ASPHALT																		
200.6	CONCRETE																		
0.6	Silty sand, some gravel, trace clay (FILL) Compact to loose Brown Moist		1	SS	13		200												
			2	SS	7		199												
198.9																			
2.1	Clayey silt with sand, trace to some gravel (FILL) Soft to stiff Brown Moist becoming wet below 3.8 m		3	SS	3		198												
			4	SS	4		197												
			5	SS	3		196												
			6	SS	6		195												
							194												
			7	SS	13		193												
			8	SS	14		192												
192.3																			
8.7	CLAYEY SILT, some sand, containing sand seams, and containing rootlets and organics Stiff Dark brown Wet		9	SS	14		191												
190.8							190												
10.2	CLAYEY SILT with sand to some sand, trace to some gravel (TILL) Hard Brown Wet		10	SS	63		189												
							188												
			11	SS	42		187												
			12	SS	30														

Continued Next Page

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

MIS-MTO.001 09-1111-6036.GPJ GAL-MISS.GDT 5/19/11 CD

PROJECT 09-1111-6036		RECORD OF BOREHOLE No 10-302		2 OF 2 METRIC	
W.O. 07-20024		LOCATION N 4823592.3 ; E 276409.9		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 December 1, 2010		CHECKED BY LCC	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
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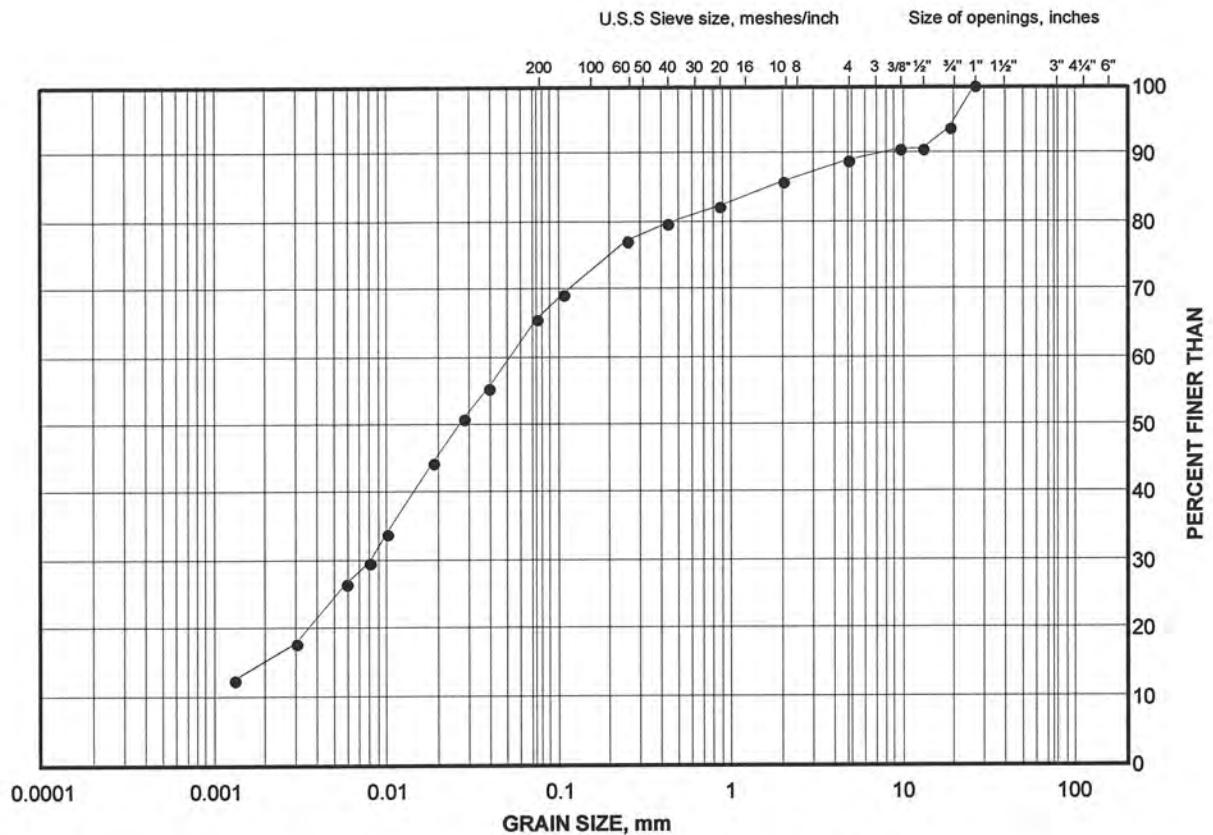
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION TEST RESULT

Clayey Silt Fill

FIGURE B1



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

LEGEND

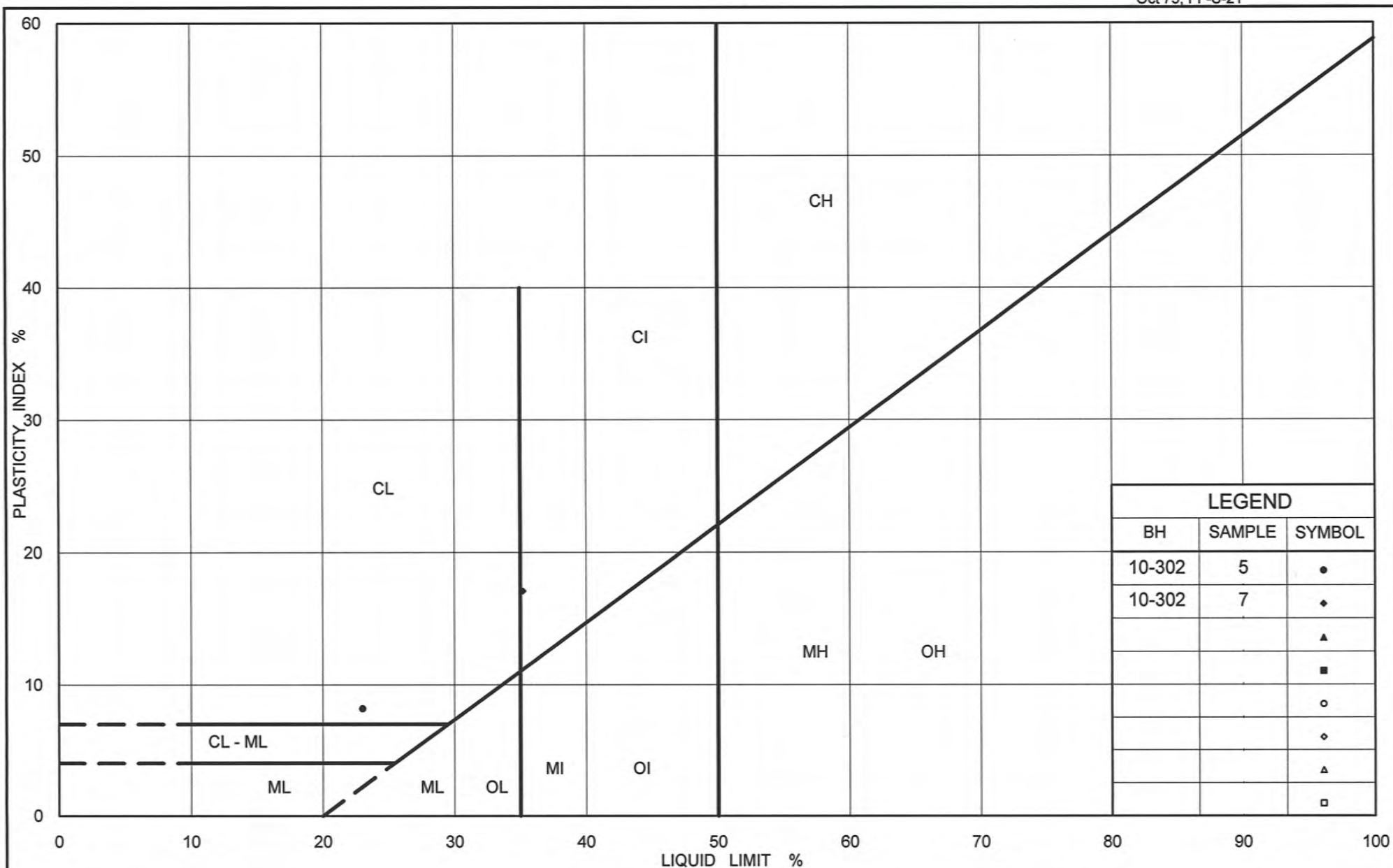
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	10-302	5	196.9

Project Number: 09-1111-6036

Checked By: *[Signature]*

Golder Associates

Date: 28-Apr-11



Ministry of Transportation

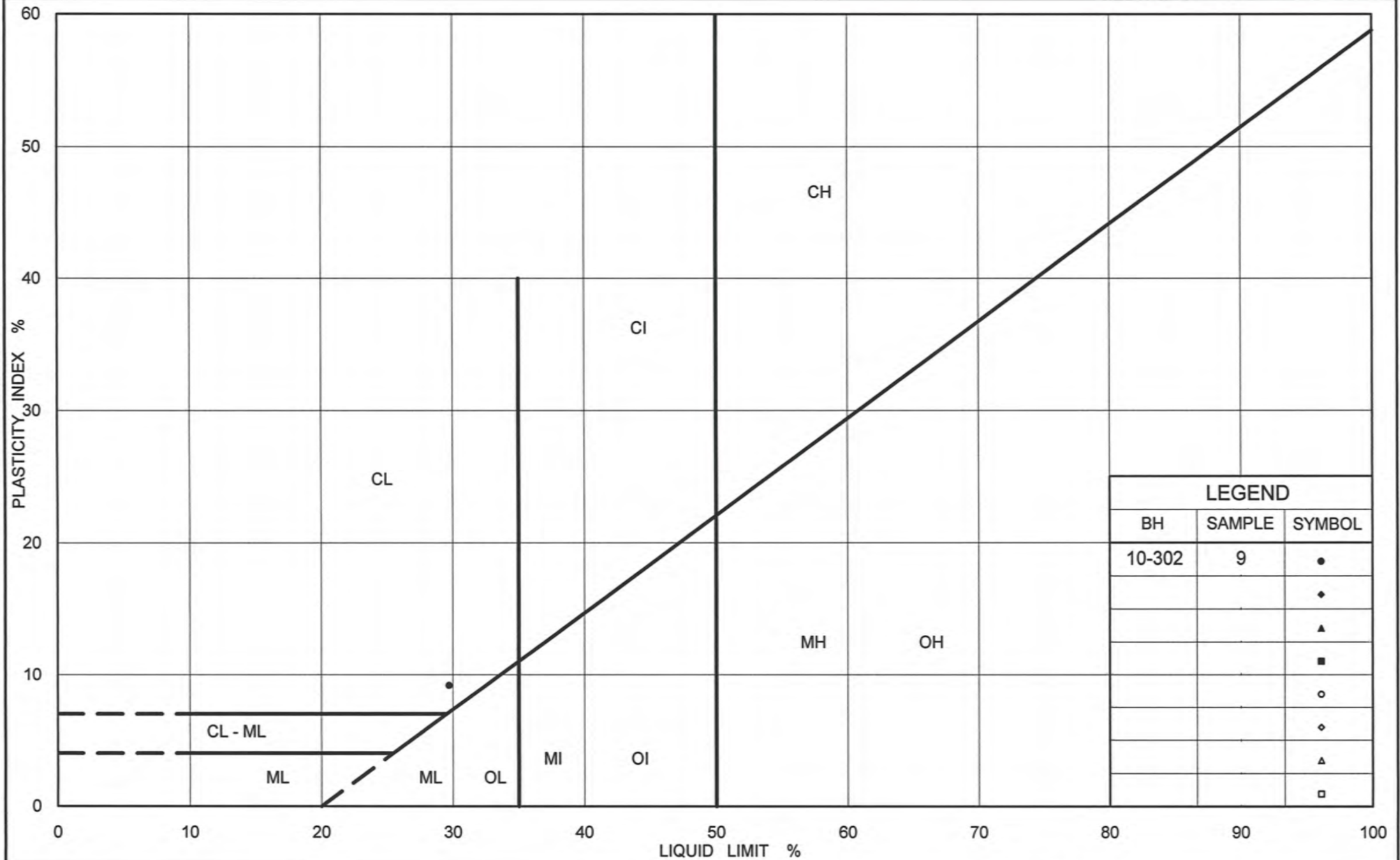
Ontario

PLASTICITY CHART Clayey Silt Fill

Figure No. B2

Project No. 09-1111-6036

Checked By: *Wayne*



Ministry of Transportation

Ontario

PLASTICITY CHART Surficial Clayey Silt

Figure No. B3

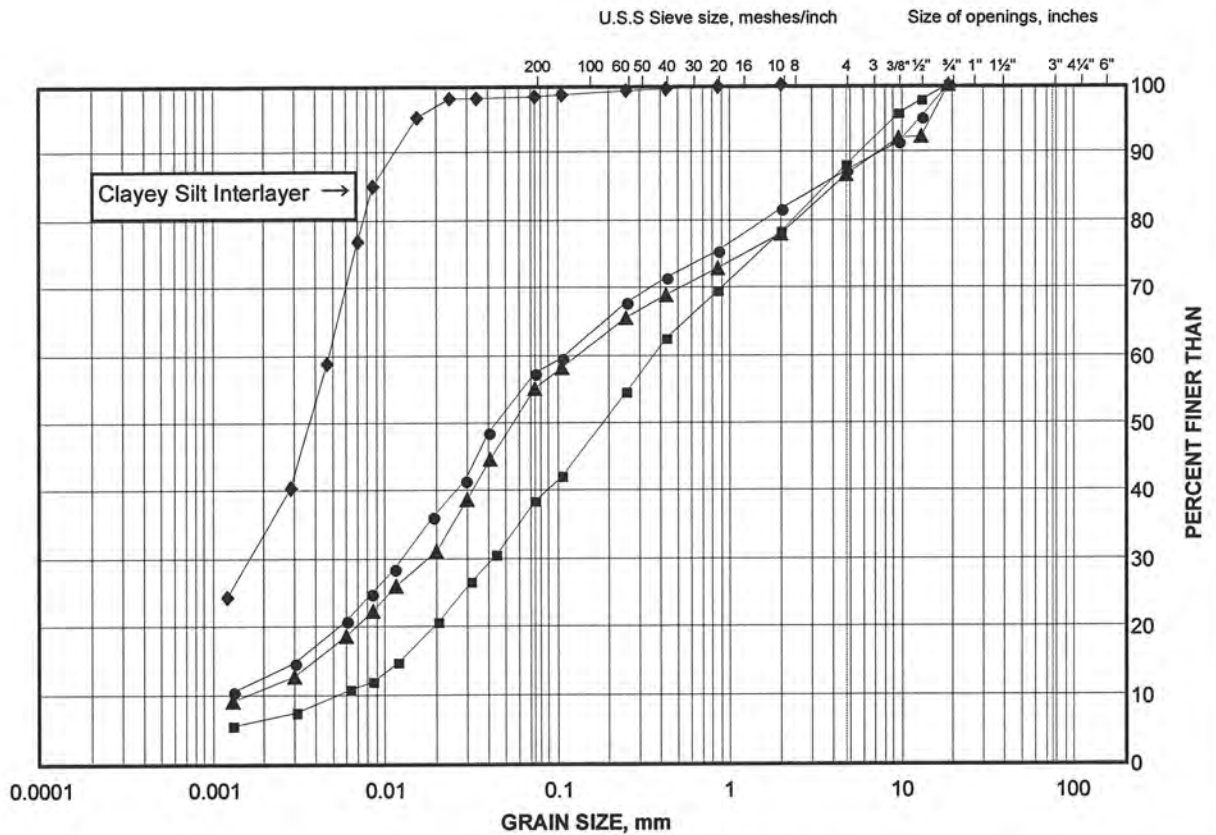
Project No. 09-1111-6036

Checked By: *Woye*

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt Till to Sand and Silt Till, Including Interlayer

FIGURE B4



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

LEGEND

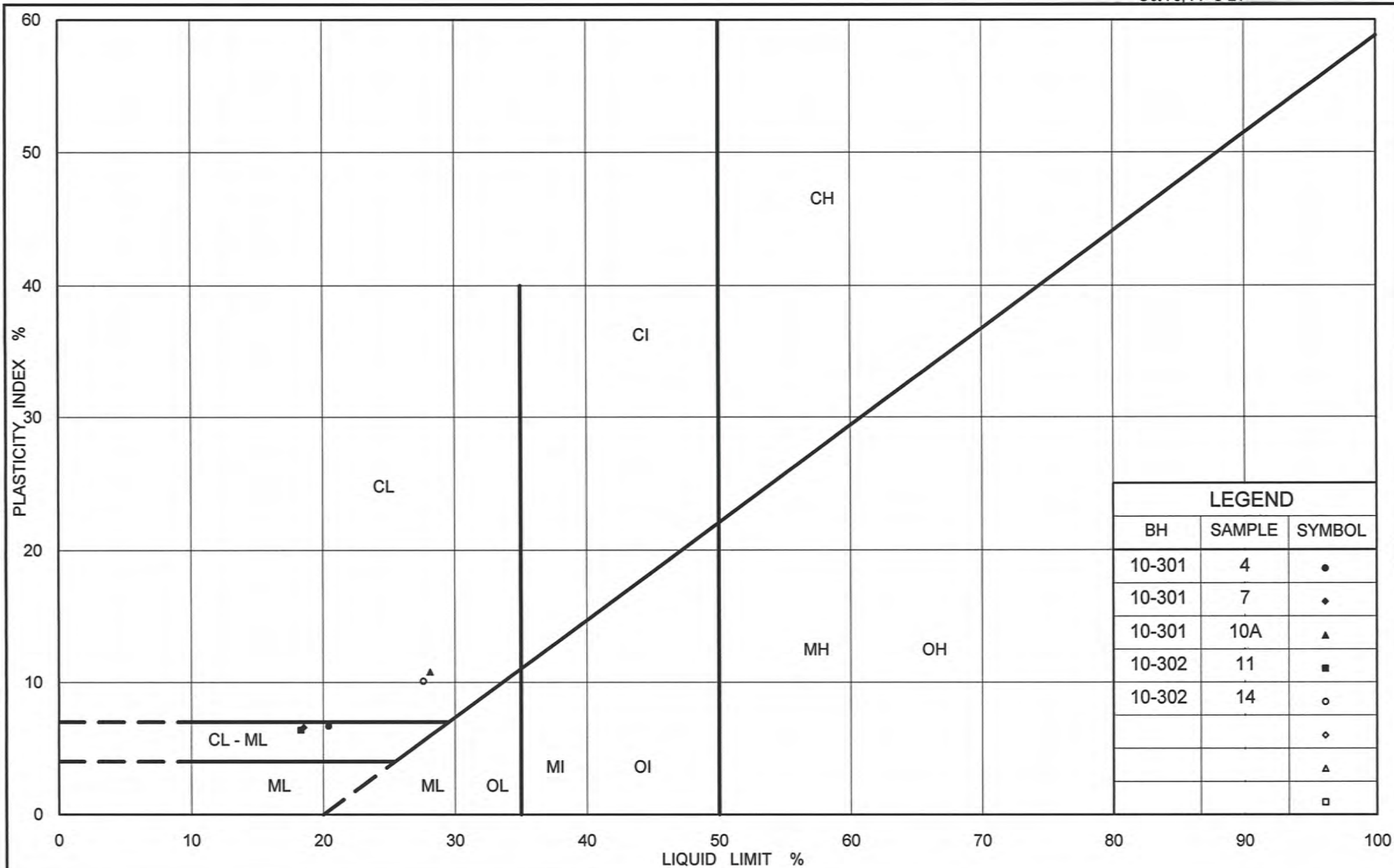
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-302	11	188.5
■	10-301	12	181.7
◆	10-302	14	183.9
▲	10-301	5	190.9

Project Number: 09-1111-6036

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Date: 28-Apr-11



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PLASTICITY CHART Clayey Silt Till Including Interlayer

Figure No. B5

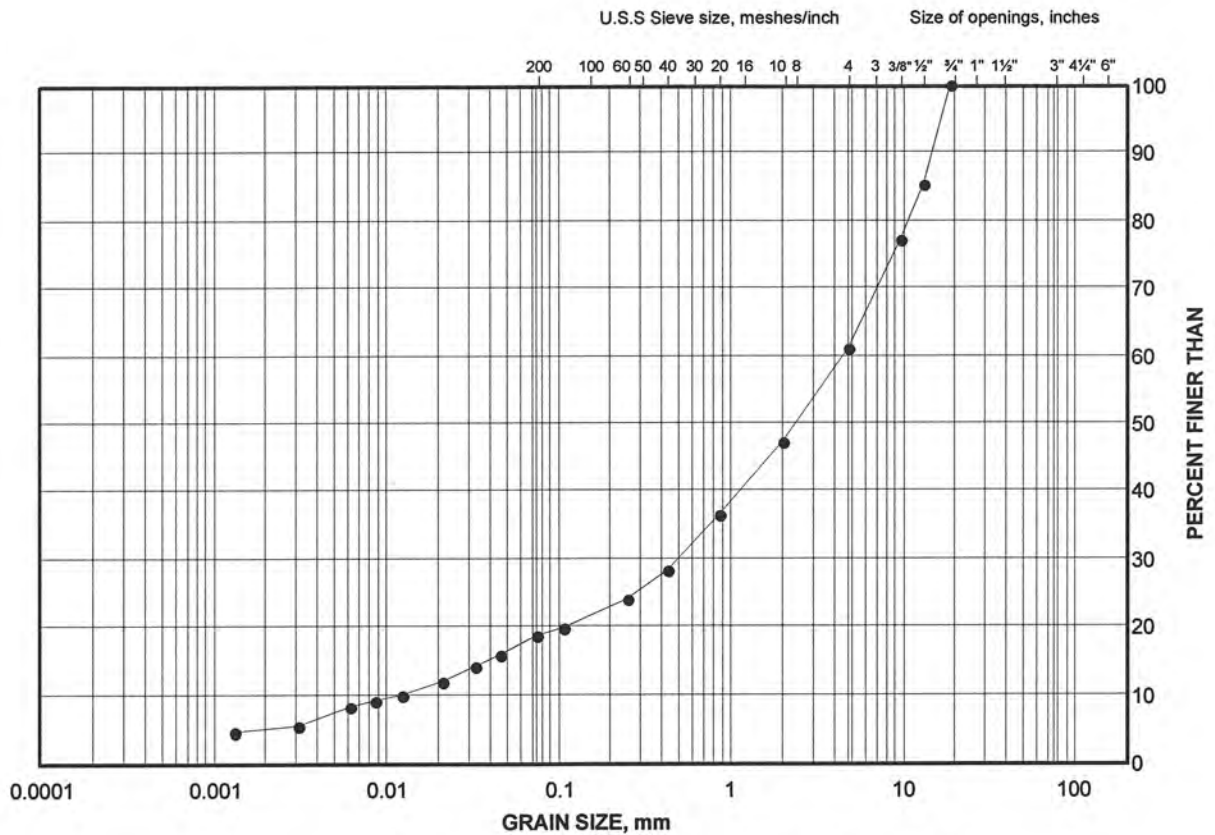
Project No. 09-1111-6036

Checked By: *Woye*

GRAIN SIZE DISTRIBUTION TEST RESULT

Sand and Gravel

FIGURE B6



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	10-301	14	178.8

Project Number: 09-1111-6036

Checked By: *Woyce*

Golder Associates

Date: 28-Apr-11

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

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