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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

CN Rail Overhead Structure
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
CN RAIL OVERHEAD STRUCTURE
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TO REGIONAL ROAD 25, HALTON REGION
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1.0 INTRODUCTION

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

This report addresses the results of the subsurface investigation carried out for the proposed widening and/or replacement of the existing CN Rail overhead 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 existing CN Rail tracks and Highway 401 overhead structure are located approximately 850 m east of Regional Road 25 in the Town of Milton in the Regional Municipality of Halton, Ontario. The existing single-span overhead structure is 13.7 m long and 36.4 m wide, with abutments supported on spread footings. The overhead structure was built in 1958, and was rehabilitated and widened northward, southward and into the centre median in 1981.

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 207 m to 210 m. The CN Rail tracks have been constructed near the original ground surface, with the rail grade at about Elevation 210.2 m to 210.5 m (rising northward) below Highway 401.

Highway 401 has been constructed on embankment fill that is up to about 8 m to 9 m in height, with the road surface at about Elevation 218.5 m. The highway 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 June and December 2010, during which time two boreholes (Boreholes 10-201 and 10-202) were advanced using track-mounted and truck-mounted CME-75 drill rigs, respectively, supplied and operated by Geo-Environmental Drilling Inc. of Milton, Ontario. The borehole locations are shown on Drawing 1: Borehole 10-201 was advanced in the southeast quadrant from near CN Rail grade, adjacent to the Highway 401 embankment toe, and Borehole 10-202 was advanced in the northwest quadrant through the Highway 401 embankment.

Boreholes 10-201 and 10-202 were drilled using 57 mm and 70 mm inner diameter hollow stem augers to depths of 24.5 m and 20.4 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.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations and one shallow and one deep standpipe piezometer were installed in Borehole 10-201 to permit monitoring of the groundwater level. Each piezometer consists of a 50 mm diameter PVC pipe, with a slotted screen sealed within a sand filter pack at a selected depth interval within the borehole. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was backfilled to the ground surface with bentonite pellets. The details of the piezometer installations and water level readings are indicated on the borehole records contained in Appendix A. Borehole 10-202 was backfilled with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended by Ontario Regulation 372).

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

The borehole locations were measured in the field by Golder personnel relative to site features and using a hand-held global 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)
10-201	4,821,414.6	272,812.6	211.0	24.5
10-202	4,821,434.5	272,740.1	218.5	20.4

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

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

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

4.2 Subsurface Conditions

As part of the subsurface investigation, two boreholes were advanced in the vicinity of the CN Rail overhead structure site. 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 section 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 subsurface conditions encountered at the site consist of the Highway 401 embankment fill overlying a deposit of clayey silt till that transitions to sand and silt till with depth. The till deposit is underlain by a deposit of sand to silty sand, which is in turn underlain by a lower deposit of clayey silt till or residual soil. A more detailed description of the soil deposits encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

Approximately 100 mm of topsoil was encountered immediately below the existing ground surface in Borehole 10-201, which is located near the toe of the Highway 401 embankment in the southeast quadrant of the structure site.

4.2.2 Embankment Fill

Approximately 1.4 m of fill was encountered immediately below the topsoil layer in Borehole 10-201, which was advanced near the south toe of the Highway 401 embankment east of the CN Rail tracks. This fill extends to about Elevation 209.5 m, and consists of clayey silt with sand, containing trace gravel as well as small amounts of rootlets and organic material. This fill has a firm to stiff consistency, based on measured Standard Penetration Test (SPT) “N” values of 5 blows and 8 blows per 0.3 m of penetration.

Approximately 0.3 m of asphalt and 11.4 m of fill associated with the existing Highway 401 embankment was encountered in Borehole 10-202, extending to approximately Elevation 206.8 m. The embankment fill in this borehole consists of the following:

- An upper layer, about 7.3 m thick, of sand and gravel containing trace to some silt and trace clay. This fill has a loose to compact relative density, based on measured SPT “N” values of 6 blows to 16 blows per 0.3 m of penetration.
- A 1.9 m thick layer of clayey silt with sand, containing trace gravel as well as rootlets. A grain size distribution test was completed on one sample of this fill, and the result is plotted on Figure B1 in Appendix B. An Atterberg limits test was completed on one sample of this fill and measured a plastic limit of 16 percent, a liquid limit of 26 percent, and a plasticity index of 10 percent; this result, which is plotted on a plasticity chart on Figure B2 in Appendix B, confirms that the tested material is a clayey silt of low plasticity. This layer has a very stiff consistency based on measured SPT “N” values of 15 blows and 17 blows per 0.3 m of penetration.

- A 0.9 m thick lower layer of sand and gravel fill containing trace silt and clay. This layer has a compact relative density, based on one measured SPT "N" value of 15 blows per 0.3 m of penetration.
- A 1.3 m thick lower layer of clayey silt fill containing some sand and some gravel. This layer has a stiff consistency, based on one measured SPT "N" value of 12 blows per 0.3 m of penetration.

4.2.3 Clayey Silt Till

A deposit of clayey silt till was encountered below the fill in both boreholes, with its surface at approximately Elevation 209.5 m in Borehole 10-201 in the southeast quadrant of the structure site, and at approximately Elevation 206.8 m in Borehole 10-202 in the northwest quadrant of the structure site. The deposit extends to approximately Elevation 197.7 m and is about 11.8 m thick in Borehole 10-201, where it was fully penetrated. Borehole 10-202 was terminated within the clayey silt till deposit at Elevation 198.1 m, after penetrating it for approximately 8.7 m.

This till deposit consists of clayey silt with sand, containing trace gravel. 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 conducted on six selected samples of the clayey silt till, and measured plastic limits between 11 per cent and 14 per cent, liquid limits between 17 per cent and 26 per cent, and plasticity indices between 4 per cent and 12 per cent. These test results, which are plotted on a plasticity chart on Figure B4 in Appendix B, confirm that the till deposit consists of clayey silt of low plasticity.

The measured SPT "N" values within the clayey silt till range from 25 blows to greater than 100 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

4.2.4 Sand and Silt Till

In Borehole 10-201, a layer of sand and silt till was encountered below the upper clayey silt till deposit. The surface of the sand and silt till was encountered at a depth of about 13.3 m (Elevation 197.7 m) in this borehole. The layer is about 1.3 m thick, with its base at approximately Elevation 196.4 m in this borehole.

The sand and silt till contains trace clay and gravel. The result of a grain size distribution test completed on the recovered sample is shown on Figure B5 in Appendix B.

One SPT "N" value of 42 blows per 0.3 m of penetration was measured in the sand and silt till deposit, indicating a dense relative density.

4.2.5 Sand to Silty Sand

A deposit of sand to silty sand was encountered below the sand and silt till in Borehole 10-201. The surface of this deposit was encountered at a depth of 14.6 m (Elevation 196.4 m), and it is about 9.6 m thick with its base at approximately Elevation 186.8 m.

The deposit varies in composition from sand containing trace to some silt, trace clay and gravel, to silty sand containing some gravel and trace clay. The results of grain size distribution tests completed on two selected samples of this deposit are shown on Figure B6 in Appendix B.

The measured SPT "N" values in this deposit are between 19 blows and 101 blows per 0.3 m of penetration, indicating a compact to very dense relative density.

4.2.6 Lower Clayey Silt Till/Residual Soil

A lower clayey silt deposit was encountered below the sand to silty sand deposit in Borehole 10-201. The top of this deposit was encountered at a depth of approximately 24.2 m (Elevation 186.8 m), and it was penetrated for a thickness of 0.3 m.

This lower deposit consists of clayey silt containing trace sand. It has been interpreted as a till or residual soil deposit.

One SPT "N" value of 100 blows per 0.08 m of penetration was recorded within this lower clayey silt till/residual soil deposit, suggesting a hard consistency.

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 sand to silty sand deposit was observed to be water-bearing during the drilling operations, and the lower portion of the embankment fill materials encountered in Borehole 10-202 were observed to be wet (between approximately Elevation 209.1 m and 206.8 m).

Two standpipe piezometers were installed in Borehole 10-201 to monitor the groundwater levels at the site: one shallow piezometer at the fill-clayey silt till interface, and one deeper piezometer installed within the sand to silty sand deposit. The water levels measured within the open boreholes upon completion of drilling and in the piezometers are summarized in the table below:

Borehole Number	Ground Surface Elevation (m)	Piezometer Installation	Depth to Water Level (m)	Depth to Water Elevation (m)	Date
10-201	211.0	Shallow	0.6 0.4	210.4 210.6	February 3, 2011 Apr 21, 2011
		Deep	2.1 2.1	208.9 208.9	February 3, 2011 April 21, 2011

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

5.0 CLOSURE

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

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
CN RAIL OVERHEAD STRUCTURE
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-CN Rail overhead structure. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. Further investigation and analysis will be required during detail design.

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

6.2 Foundation Options

Based on the planning study completed to date for the widening of Highway 401 from Regional Road 25 to Trafalgar Road, 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 CN Rail overhead structure will require widening to the north and south; full replacement of the existing structure is also under consideration.

The existing CN Rail overhead structure is single-span, with the existing abutments supported on spread footings. Based on the *General Arrangement* drawing for the 1981 structure widening, the top of the footings for the widening were to be constructed at Elevation 209.1 m (686.0 ft.), with the east and west abutment footings founded at approximately Elevation 207.6 m. This founding elevation is about 2.6 m to 2.9 m below the CN Rail grade at this site.

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 existing CN Rail overhead structure. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and approximate costs is provided in Table 1 following the text of this report.

- **Strip or spread footings founded on the very stiff to hard clayey silt till:** Strip footings are feasible for support of the new widened abutments and any associated retaining walls, or for support of a replacement structure. It is expected that excavation will be required to a depth of about 3 m below rail grade in the south widening area, and about 4.5 m below rail grade (to extend below existing stiff clayey silt fill) in the north widening area. Temporary excavation support would likely be required along the north and south sides of Highway 401 as well as along the CN Rail tracks to facilitate the removal of the existing wing walls/retaining walls adjacent to the abutments and excavation to the founding level. There is potential for up to approximately 15 mm of differential settlement between the existing structure and the new widened portions.

- **Footings "perched" on a compacted granular pad in the widened Highway 401 approach embankments:** Although this option would be advantageous in minimizing the depth of excavation, "perched" footings are not recommended for support of the north and south widening due to the consistency/relative density of the existing Highway 401 embankment fill (as encountered in Borehole 10-202), which would underlie a portion of the perched footings.
- **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 founded within the "100-blow" clayey silt till (north side) or "100-blow" clayey silt till/residual soil (south side) are feasible for support of the abutment widening. However, this option is considered to have a higher risk due to the potential difficulties associated with the relatively high groundwater pressure in the sand to silty sand deposit: there is a risk of base disturbance for caissons founded in the clayey silt till above the aquifer, and a risk of loosening or heaving of the silty sand to sand soils for caissons that extend into or through this deposit. Temporary or permanent liners would be required.

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 associated with deeper excavation 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, or for a full structure replacement, 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 very stiff to hard clayey silt till. If spread footings are used for a widening, it is likely that they will be constructed to match the existing foundation elevation (approximately Elevation 207.6 m) and structurally connected. It is noted that for widening at the north abutment, subexcavation would be required below this level to remove stiff till soils and special precautions will be required to prevent undermining and protect the existing foundations during subexcavation. The subexcavation could be backfilled with compacted granular fill to enable shallow foundations for the widening to be constructed to match the existing founding level. 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	Boreholes	Founding Stratum	Strip/Spread Footing Founding Elevation (m)
North Widening or Northern Portion of Replacement	10-202	Very stiff to hard clayey silt till	Below 206.0 m
South Widening or Southern Portion of Replacement	10-201	Very stiff to hard clayey silt till	Below 208.5 m (or lower to match existing)

Alternatively for the north widening, subexcavation can be carried out to the elevation 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, as 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 a Quality Verification Engineer (QVE) 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 and should be protected with a concrete working slab (100 mm thick concrete slab with a compressive strength of 20 MPa) if the concrete for the footing is not placed within four hours of the inspection and approval of the subgrade.

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 existing fill), at or below the design elevations given in the preceding section, should be designed based on the factored geotechnical resistances at Ultimate Limit States (ULS) and geotechnical resistances at Serviceability Limit States (SLS) given below.

Foundation Element	Footing Width	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS*
North Widening or North Portion of Replacement Structure	3 m	650 kPa	450 kPa
	4 m	700 kPa	400 kPa
North Widening or North Portion of Replacement Structure (Following subexcavation and replacement with compacted granular fill)	3 m	600 kPa	450 kPa
	4 m	650 kPa	400 kPa
South Widening or South Portion of Replacement Structure	3 m	600 kPa	450 kPa
	4 m	650 kPa	400 kPa

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 201.5 m on the north side of Highway 401, and below about Elevation 189.5 m on the south side of Highway 401. The following pile tip elevations may be used for preliminary design purposes, assuming about 1.5 m to 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	200.0 m
South Widening or South Portion of Replacement Structure	Very dense silty sand	188.0 m

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

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the soil deposits. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of "hanging up" or being deflected away from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip with driving shoes or flange plates to reduce the potential for damage to the piles during driving, in accordance with OPSS 903 *(Deep Foundations).* In very dense/hard and/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 OPSD 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 or North Portion of Replacement Structure	Hard clayey silt till	1,400 kN	1,200 kN
South Widening or South Portion of Replacement Structure	Very dense silty sand	1,400 kN	1,200 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 Caissons

Caisson foundations are feasible for support of abutments and retaining walls associated with the northward and southward widening of the existing CN Rail overhead structure.

Running or flowing of water-bearing cohesionless soil strata could occur during or after drilling of the caissons, and basal heave could occur where water-bearing cohesionless soils are present at/near the caisson base. If caisson foundations are adopted, a liner would be required to support the soils during construction and permit inspection and cleaning of the caisson base.

Construction experience in similar soil conditions has demonstrated that temporary liners can be difficult to withdraw, owing to the length of the liners and the hard/very dense nature of the 100-blow material. Such difficulties can result in "necking" of the caisson, although this can be controlled by tremie-pumping the concrete into the caisson and ensuring that the base of the liner always remains below the surface of the pumped concrete during withdrawal. Alternatively, permanent liners could be considered for the construction of the caissons in these soil conditions.

6.5.1 Founding Elevations

The widened abutments and associated retaining walls could be supported on caissons founded within the hard/very dense soils (with SPT "N" values of greater than 100 blows per 0.3 m of penetration) that were encountered below approximately Elevation 201.5 m on the north side of Highway 401, and below about Elevation 189.5 m on the south side of Highway 401. For preliminary design, the following caisson base elevations may be assumed based on the borehole results:

Foundation Element	Founding Stratum	Estimated Design Caisson Elevation
North Widening or North Portion of Replacement Structure	Hard clayey silt till	198.5 m
South Widening or South Portion of Replacement Structure	Very dense silty sand	187.5 m

6.5.2 Axial Geotechnical Resistance

The caissons will derive the majority of their capacity from base resistance, although some shaft friction has also been taken into account based on "socketting" approximately 1.5 m into the "100-blow" soil. Using the preliminary design elevations given above, and assuming that the caisson excavations are inspected prior to pouring concrete, the factored axial geotechnical resistance at ULS and the axial resistance at SLS (for 25 mm of settlement) may be taken as follows for preliminary design purposes:

Caisson Diameter	Factored Axial Geotechnical Resistance at ULS	Geotechnical Resistance at SLS
1.2	3,800 kN	3,100 kN
1.5	5,900 kN	5,000 kN

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

6.6 Approach Embankments

6.6.1 Subgrade Preparation and Embankment Construction

It is recommended that all topsoil/organic material or existing surficial fill materials be stripped from the footprint of the proposed widened Highway 401 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 OPSD 208.010 *(Benching of Earth Slopes).*

Additional fill for construction of the embankment widening could consist of clean earth fill or granular fill. From a geotechnical/foundations perspective, both earth and granular fill will provide good compatibility with the existing Highway 401 embankment fill materials – both those fill materials remaining in-place in the existing embankment side slope, and any existing embankment fill that is re-used for the widening after being cut from the benches.

In accordance with MTO's standard practice, a minimum 2 m wide bench should be provided where the embankment side slopes are equal to or greater than 8 m in height, such that the uninterrupted slope height does not exceed 8 m. To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS 572 *(Construction Specification for Seed and Cover).*

6.6.2 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 at the CN Rail overhead site, considering the design requirements and the available field and laboratory testing data.

The stability analyses were completed for an approximately 10 m high embankment widening, based on the subsurface conditions as encountered in Boreholes 10-201 and 10-202. 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°	-
Very stiff to hard clayey silt till	21	32°	-
Dense sand and silt till	20	34°	-
Compact to dense sand	19	34°	-
Very dense silty sand	20	34°	-

The analysis results indicate that an approximately 10 m high embankment widening with side slopes oriented no steeper than 2H:1V will have a factor of safety of 1.4 or better against global instability, assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials. An example static global stability result is provided on Figure 1. This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed based on the subsoil conditions encountered within the proposed approach embankment footprints during detail design.

6.6.3 Approach Embankment Settlement

Based on the study completed to date, the existing 9 m to 10 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	-
Very stiff to hard clayey silt till	21	75 MPa
Dense sand and silt till	20	60 MPa
Compact to dense sand	19	50 MPa
Very dense silty sand	20	75 MPa

Based on this preliminary assessment, the settlement of the foundation soils under the widened 9 m to 10 m high approach embankments is estimated to be a maximum of approximately 25 mm, decreasing to less than 10 mm near the existing highway shoulder and under the new widened embankment toe. This settlement is expected to occur relatively quickly during and immediately following construction of the widened approach embankments based on the nature of the soils at the site. This estimated magnitude of settlement should be reassessed based on the soil and groundwater conditions under the 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.

6.7 Construction Considerations

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

6.7.1 Excavation and Temporary Protection Systems

The foundation excavations for spread footings are expected to extend to a depth of about 1.5 m to 4.5 m below the CN Rail grade, through the existing firm to very stiff clayey silt fill 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 potentially into the very stiff to hard clayey silt till.

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

At this preliminary stage, it is anticipated that temporary roadway protection would likely be required both along the north and south sides of Highway 401 and parallel to the CN Rail tracks, to facilitate the removal of the existing wing walls/retaining walls adjacent to the abutments and excavation to foundation level for the widened abutments and associated wing walls or retaining walls. 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 the following performance levels as specified in OPSS 539:

- Performance Level 2 for protection systems along Highway 401; and
- Performance Level 1b for protection systems along the CN Rail tracks.

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.

Depending on the construction schedule that will be required, consideration could be given to developing a conceptual design for the temporary protection system along the CN Rail tracks as part of the detail design, and having this design pre-approved by CN Rail in advance of tendering the contract, to expedite the construction schedule.

6.7.2 Groundwater Control

Groundwater seepage is anticipated from the fill ("perched" conditions on top of the underlying cohesive till deposit) and from cohesionless soil lenses or layers in the clayey silt till deposit. 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.

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

6.7.3 Subgrade Protection

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

6.7.4 Obstructions

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

6.7.5 Vibration Monitoring During Pile or Caisson 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 overhead structure is not expected to be required during construction at this site. Vibration monitoring on the rail tracks may, however, be required by CN Rail.

6.8 Recommendations for Further Work in Detail Design

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

- Abutments and retaining walls:
  - Assessment of the variability of any existing fill and surficial soils to confirm the founding elevation for spread footings within each widened abutment area, and to assess groundwater control requirements associated with seepage from surficial cohesionless fill materials or soil deposits if present.
  - 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 any 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 Mr. Gilberto Alexandre, and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Principal with Golder, with technical input from Mr. Murty Devata, P.Eng., a 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.

GOLDER ASSOCIATES LTD.

for Gilberto Alexandre
Geotechnical Group

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

LICENSED PROFESSIONAL ENGINEER
21 Oct/11
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REGISTERED PROFESSIONAL ENGINEER
T. J. GARDE
Oct 21/11
PROVINCE OF ONTARIO

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

GA/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

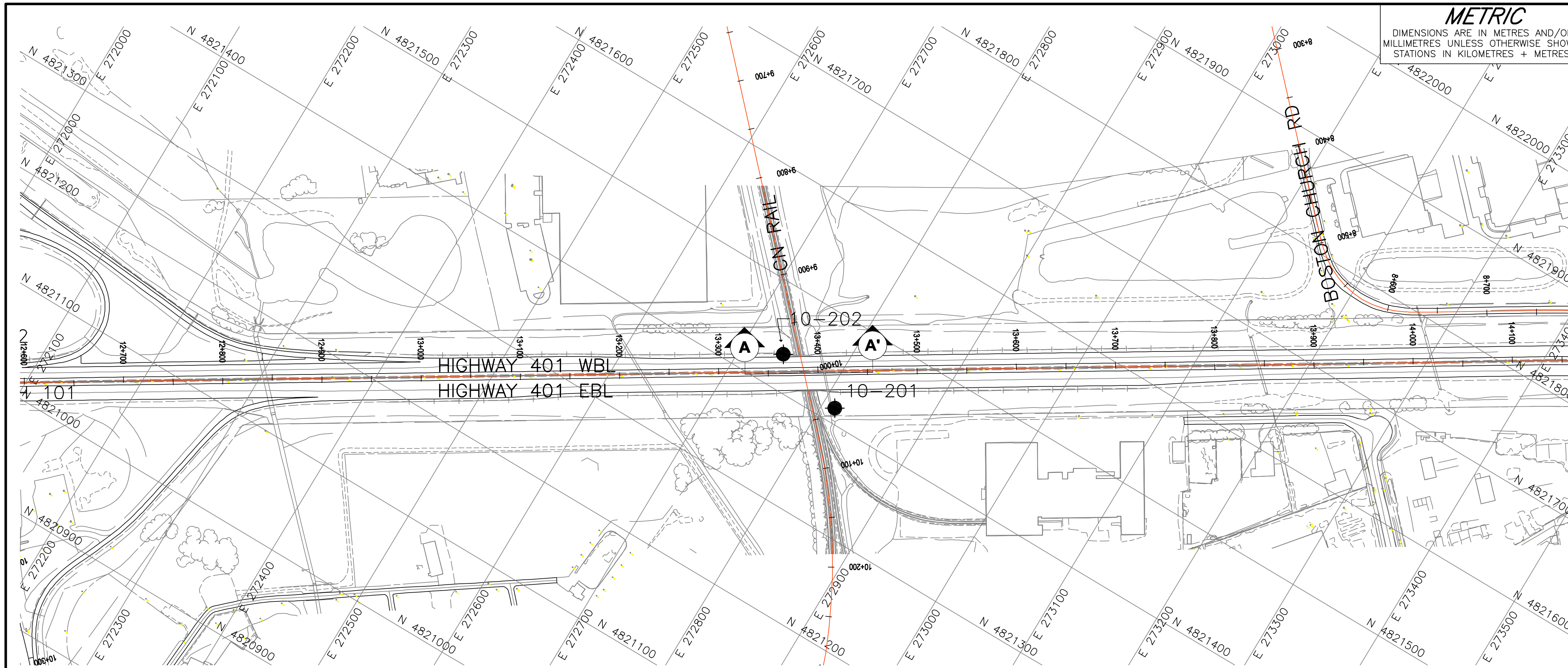
OPSD 3090.101 Foundation Frost Penetration Depths for Southern Ontario

TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Spread/strip footings on very stiff to hard clayey silt till	• Feasible for support of widened abutments and associated wing walls/retaining walls	• Existing structure supported on shallow foundations, and has performed well • Relatively minor groundwater seepage anticipated • Allows for semi-integral abutments • Lower vibration impacts on existing structures than for driven steel H-pile installation	• Significant excavations (to a depth of up to 4.5 m below the CN Rail grade) to extend below existing fill for north widening area; would require temporary excavation support • Moderate potential for differential settlement (up to about 15 mm) between existing overhead structure and widening • Precludes use of integral abutments; potentially greater maintenance required at abutments • Lower geotechnical resistances as compared with deep foundations	• Conventional excavation and construction techniques	• Less expensive than deep foundations although bridge maintenance costs may be higher due to non-integral abutment configuration • Estimated cost is \$45,000 per foundation element (i.e., southeast abutment, southwest abutment, etc.), based on an estimated \$600/m^3 for construction of shallow foundations, excluding deeper excavation and temporary protection system
Spread/strip footings perched on compacted granular pad in approach embankment fill	• Not recommended for support of widened abutments	• Abutment pile caps could be maintained higher than footings founded on till deposit, significantly reducing subexcavation depth and associated temporary protection system requirements	• Moderate 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 loose/stiff portions of existing fill under the widened approach embankment loading	• Conventional excavation and construction techniques	• This option not recommended

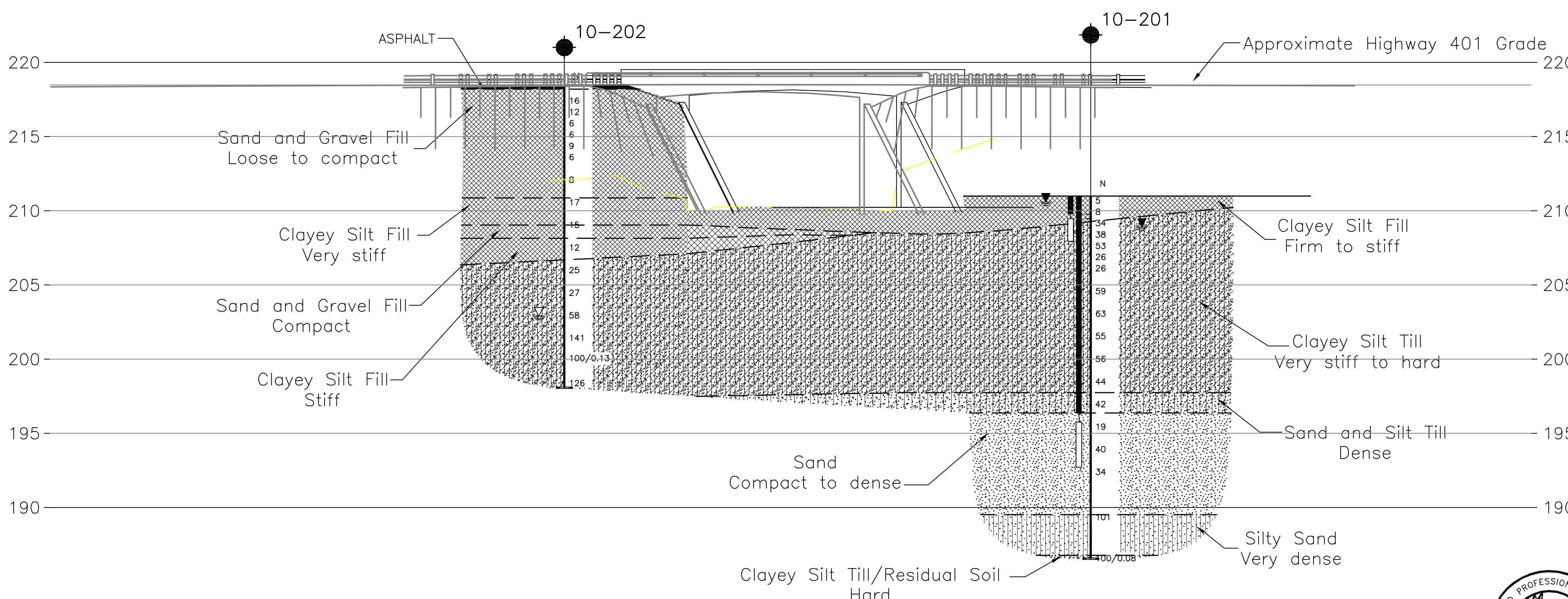
Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel H-piles driven to found in hard/very dense soils	• Feasible for support of widened abutments and associated wing walls/retaining walls	• Abutment pile caps could be maintained higher than footings founded on till deposit, reducing depth of excavation and temporary excavation support requirements adjacent to Highway 401 and CN Rail • 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 overhead structure and widened portions of structure	• Minor potential for encountering obstructions (cobbles and/or boulders) during pile driving that could result in piles “hanging up” and lower geotechnical resistances	• Conventional construction methods for H-pile foundations	• Lower relative cost compared with caisson option • Estimated cost is approximately \$45,000 for each of the northwest and northeast abutment widening, and about \$76,000 for each of the southwest and southeast abutment widenings, based on an estimated \$250/m length for pile installation and \$600/m^3 for pile cap construction

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel pipe (tube) piles, driven to found in hard/very dense soils	• Feasible for support of widened abutments and associated wing walls/ retaining walls	• Abutment pile caps could be maintained higher than footings founded on till deposit, reducing depth of excavation and temporary protection system requirements adjacent to Highway 401 and CN Rail • Limited groundwater control required • Allows for semi-integral abutment configuration • Would minimize differential settlement between foundation elements	• 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	• Conventional construction methods	• Costs for steel pipe (tube) piles slightly higher than for steel H-piles
Caissons founded in hard/very dense soils	• Feasible but not preferred for support of abutments and wing walls/ retaining walls	• Abutment pile caps could be maintained higher than footings founded on till deposit, reducing depth of excavation and temporary excavation support requirements adjacent to Highway 401 and CN Rail • Higher capacity than for steel H-piles, so reduced number of deep foundation elements compared to steel H-piles	• Water-bearing sand and silt till and sand to silty sand deposits could contribute to loss of ground or base disturbance • Temporary or permanent liners would be required; likely not possible to inspect caisson base • Precludes use of integral abutments	• Risk of loosening soils at base of caissons	• Higher cost compared with shallow foundations or steel H-piles



PLAN

SCALE 50 0 50 100 m



SECTION A–A'

SCALE 4 0 4 8 m

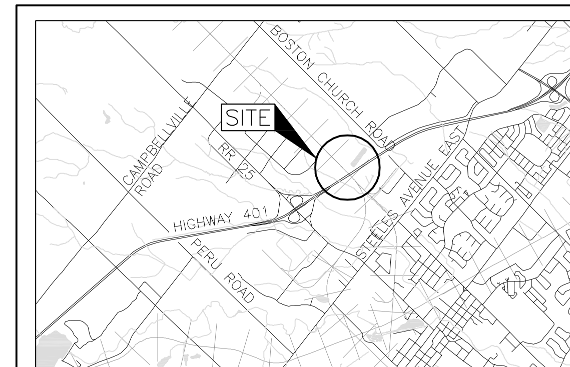
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WO No. 07–20024

SHEET

CN RAIL OVERHEAD
HIGHWAY 401 WIDENING
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE 1 0 1 2 km

LEGEND

- Borehole – Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling
- WL in piezometer, measured on Feb. 03, 2011

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
10–201	211.0	4821414.6	272812.6
10–202	218.5	4821434.5	272740.1

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the design configuration as shown elsewhere in the Preliminary Design Report

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

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

REFERENCE

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

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

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

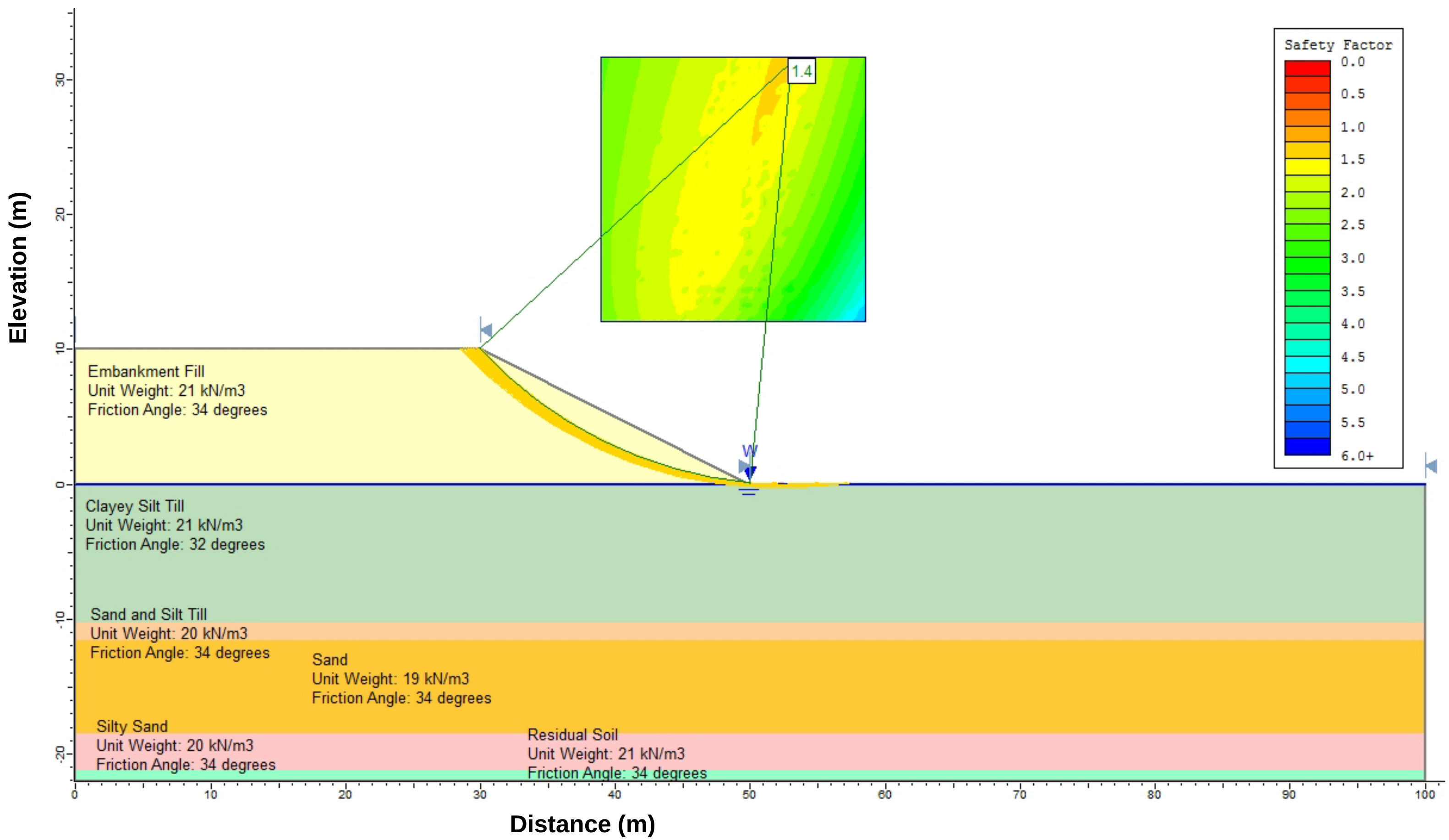
NO.	DATE	BY	REVISION

Geocres No. 30M12–326

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

Static Global Stability – CN Rail Overhead Approach Embankment

Figure 1



Date: April 2011

Project No: 09-1111-6036

Analysis By: BT **Reviewed By:** LCC

APPENDIX A

Borehole Records

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

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

Dynamic Cone Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to “A” size drill rods for a distance of 300 mm (12 in.).

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

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

* 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

Golder Associates

RECORD OF BOREHOLE No 10-201

 METRIC

PROJECT	09-1111-6036		
W.O.	07-20024	LOCATION	N 4821414.6 ;E 272812.6
DIST	Central HWY 401	BOREHOLE TYPE	CME 75 Truck-mount, 57 mm Internal Diameter Hollow Stem Power Augers
DATUM	Geodetic	DATE	June 24, 2010
ORIGINATED BY	SB/MS	COMPILED BY	OK/MS
CHECKED BY	LCC		

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
211.0	GROUND SURFACE
0.0	Topsoil
0.1	Clayey silt with sand, trace gravel, containing rootlets and organics (FILL / Reworked Soil) Firm to stiff Brown Moist
209.5 / 1.5	CLAYEY SILT with sand, trace gravel (TILL) Very stiff to hard Brown to grey Moist
197.7 / 13.3	SAND and SILT, trace clay and gravel (TILL) Dense Brown Moist
196.4 / 14.6	

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	5	
2	SS	8	
3	SS	34	
4	SS	38	
5	SS	53	
6	SS	26	10 34 43 13
7	SS	26	
8	SS	59	
9	SS	63	
10	SS	55	
11	SS	56	
12	SS	44	
13	SS	42	11 34 48 7

ELEVATION SCALE: 210, 209, 208, 207, 206, 205, 204, 203, 202, 201, 200, 199, 198, 197

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

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

RECORD OF BOREHOLE No 10-201

 METRIC

PROJECT 09-1111-6036
W.O. 07-20024
DIST Central HWY 401
DATUM Geodetic
LOCATION N 4821414.6 ;E 272812.6
BOREHOLE TYPE CME 75 Truck-mount, 57 mm Internal Diameter Hollow Stem Power Augers
DATE June 24, 2010
ORIGINATED BY SB/MS
COMPILED BY OK/MS
CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND, trace to some silt, trace clay and gravel; Compact to dense; Brown; Wet	14	SS	19	
		15	SS	40	0 88 10 2
		16	SS	34	
189.5 / 21.5	Silty SAND, some gravel, trace clay; Very dense; Brown; Wet	17	SS	101	23 45 29 3
186.8 / 24.5	CLAYEY SILT, trace sand (TILL/Residual Soil); Hard; Brown; Moist	18	SS	100/0.08	

Elevation scale: 195, 194, 193, 192, 191, 190, 189, 188, 187

END OF BOREHOLE

Notes:

1. Shallow monitoring well was dry upon completion of drilling. Water level in deep monitoring well at ground surface upon completion of drilling due to drilling activities (addition of water to borehole).

2. Water level in piezometer measured as follows:

Shallow Piezometer

Date	Depth (m)	Elev. (m)
Feb. 3, 2011	0.6	210.4
April 21, 2011	0.4	210.6

Deep Piezometer

Date	Depth (m)	Elev. (m)
Feb. 3, 2011	2.1	208.9
April 21, 2011	2.1	208.9

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

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

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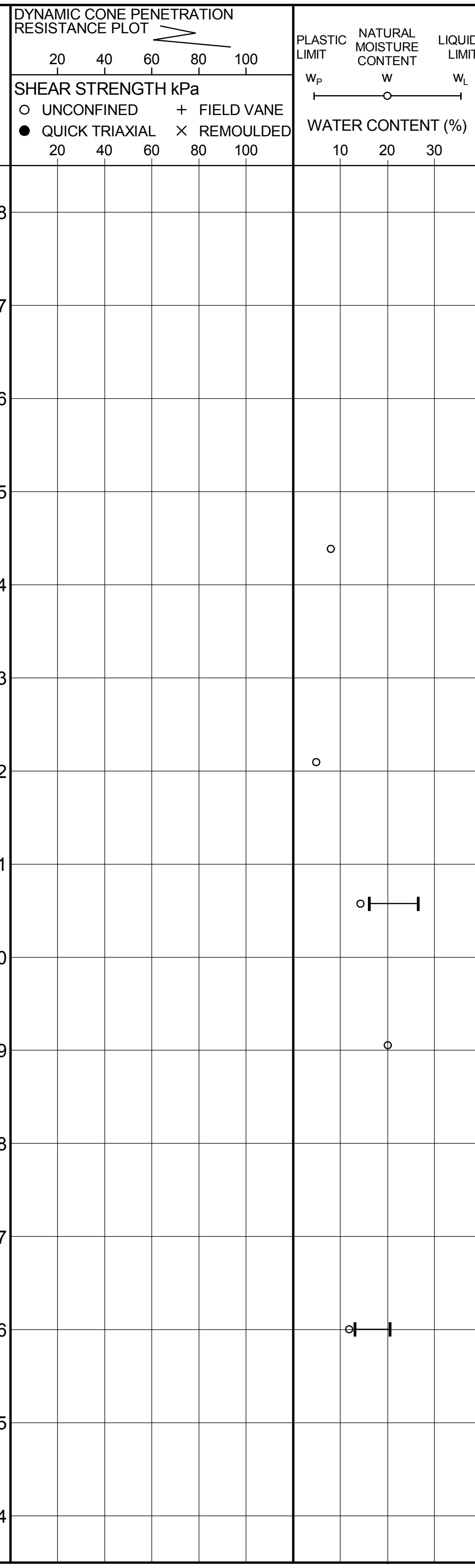
RECORD OF BOREHOLE No 10-202

 METRIC

PROJECT	09-1111-6036		
W.O.	07-20024	LOCATION	N 4821434.5 ;E 272740.1
DIST	Central HWY 401	BOREHOLE TYPE	70 mm I.D. Hollow Stem Power Auger
DATUM	Geodetic	DATE	December 5, 2010
ORIGINATED BY	MS	COMPILED BY	CS
CHECKED BY	LCC		

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
218.5	GROUND SURFACE					
0.0 / 218.2	ASPHALT					
0.3	Sand and gravel, trace to some silt, trace clay (FILL) Loose to compact Brown Moist				218	
		1	SS	16		
		2	SS	12	217	
		3	SS	6	216	
		4	SS	6	215	
		5	SS	9	214	
		6	SS	6	213	
		7	SS	8	212	
210.9 / 7.6	Clayey silt with sand, trace gravel, containing rootlets (FILL) Very stiff Brown Moist				211	
		8	SS	17	210	
209.1 / 9.5	Sand and gravel, trace silt, trace clay (FILL) Compact Grey Wet	9	SS	15	209	3 23 50 24
208.1 / 10.4	Clayey silt, some sand, some gravel (FILL) Stiff Brown Wet				208	
		10	SS	12	207	
206.8 / 11.7	CLAYEY SILT with sand, trace gravel (TILL) Very stiff to hard Brown Wet					
		11	SS	25	206	3 34 46 17
					205	
		12	SS	27	204	



UNIT WEIGHT γ kN/m^3

Continued Next Page

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

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RECORD OF BOREHOLE No 10-202

 METRIC

PROJECT 09-1111-6036

W.O. 07-20024 LOCATION N 4821434.5 ;E 272740.1 ORIGINATED BY MS

DIST Central HWY 401 BOREHOLE TYPE 70 mm I.D. Hollow Stem Power Auger COMPILED BY CS

DATUM Geodetic DATE December 5, 2010 CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
	CLAYEY SILT with sand, trace gravel (TILL) Very stiff to hard Brown Wet	13	SS	58	▽	203
						202
		14	SS	141		201
		15	SS	100/0.13		200
						199
198.1		16	SS	126		
20.4	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 15.6 m (Elev. 202.9 m) upon completion of drilling.					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

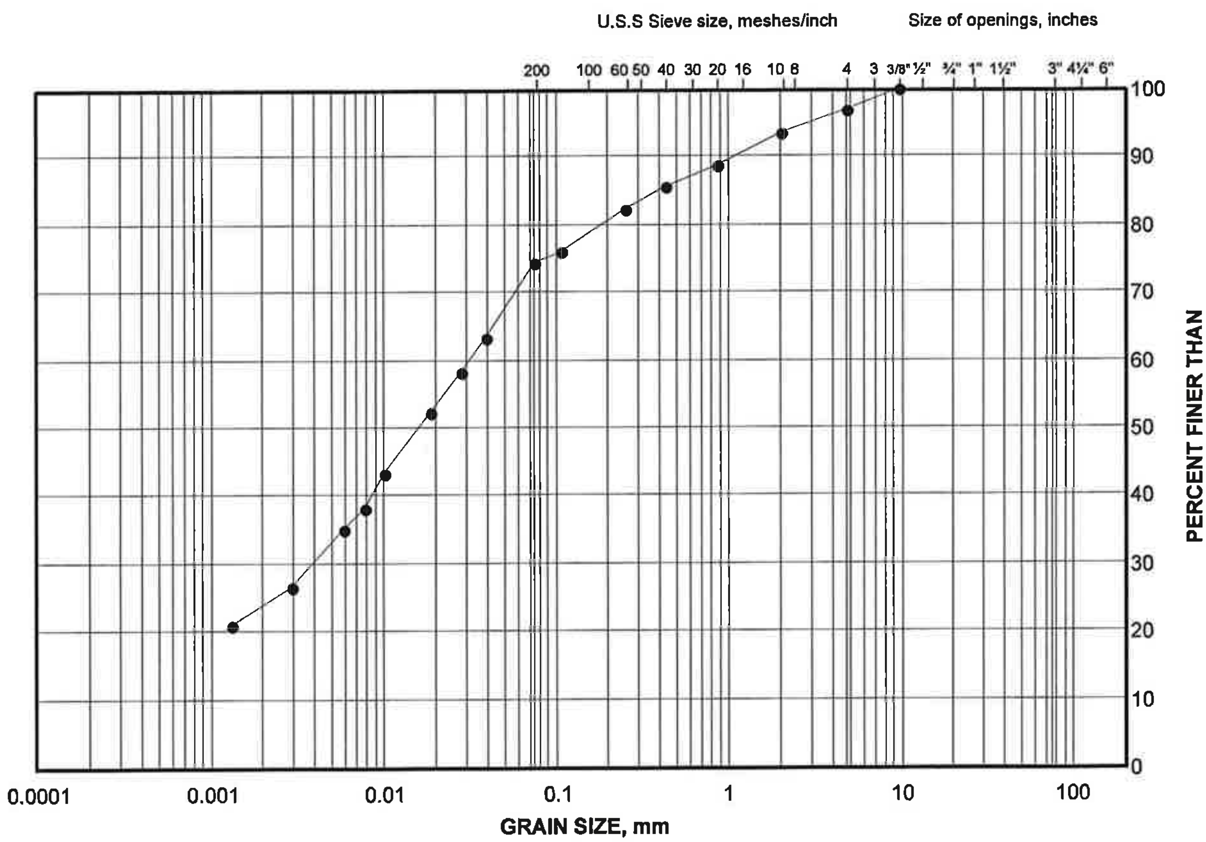
+ ³, × ³: Numbers refer to Sensitivity ○ ³% STRAIN AT FAILURE

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)
●	10-202	9A	209.2

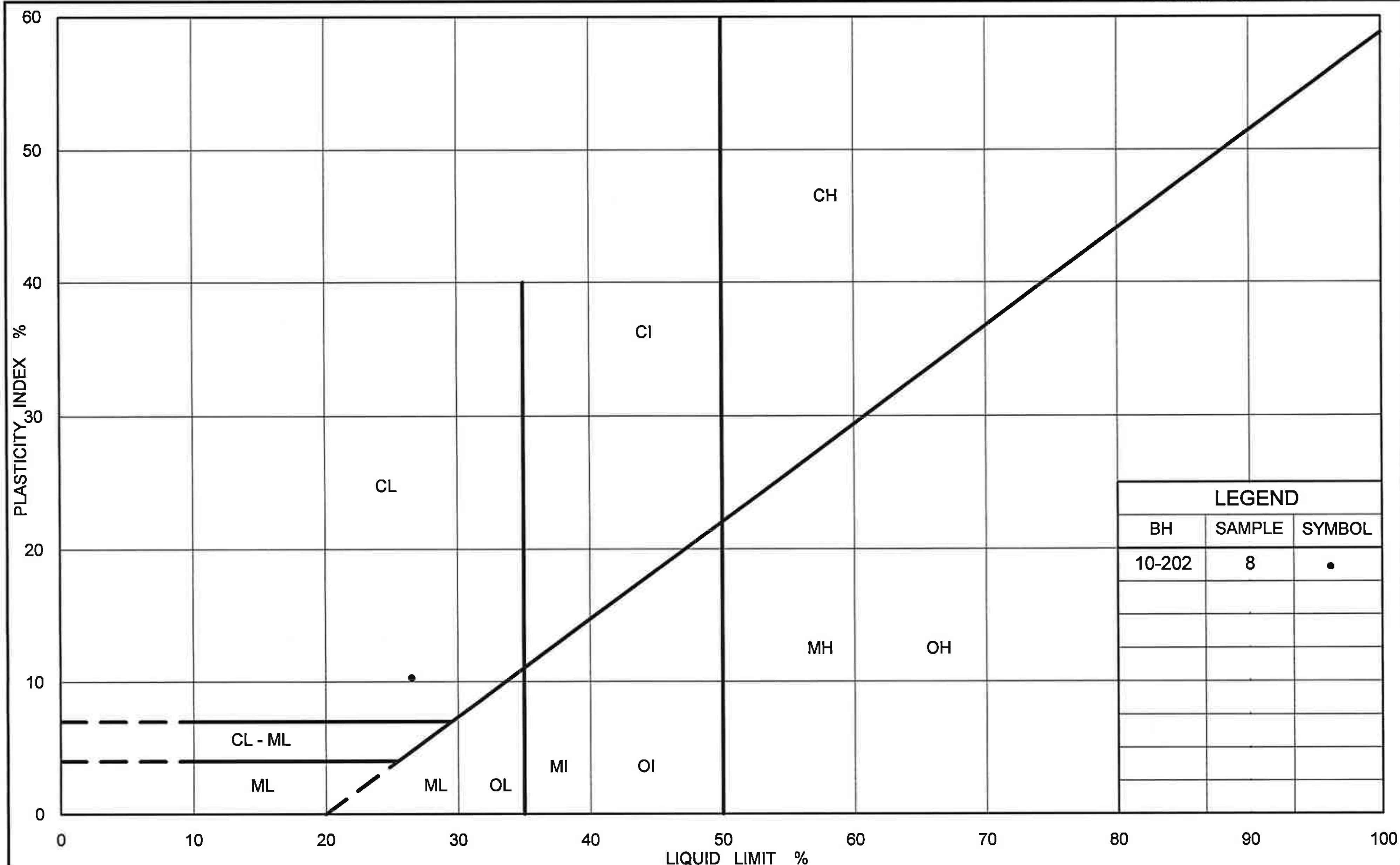
Project Number: 09-1111-6036

Checked By: LCC

Golder Associates

Date: 18-May-11

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
10-202	8	•

Ministry of Transportation
Ontario

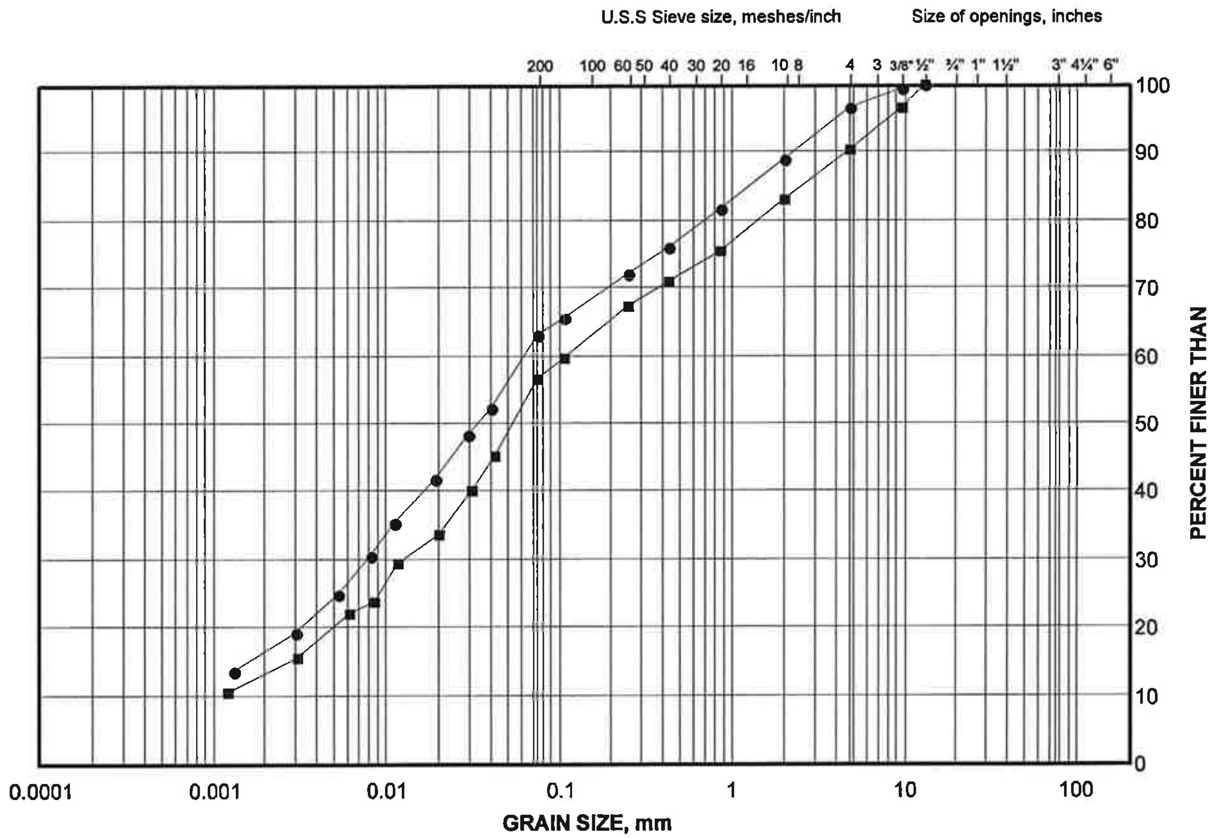
PLASTICITY CHART
Clayey Silt Fill

Figure No. B2
Project No. 09-1111-6036
Checked By: LCC

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt Till

FIGURE B3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-202	11	206.0
■	10-201	6	206.8

Project Number: 09-1111-6036

Checked By: LCC

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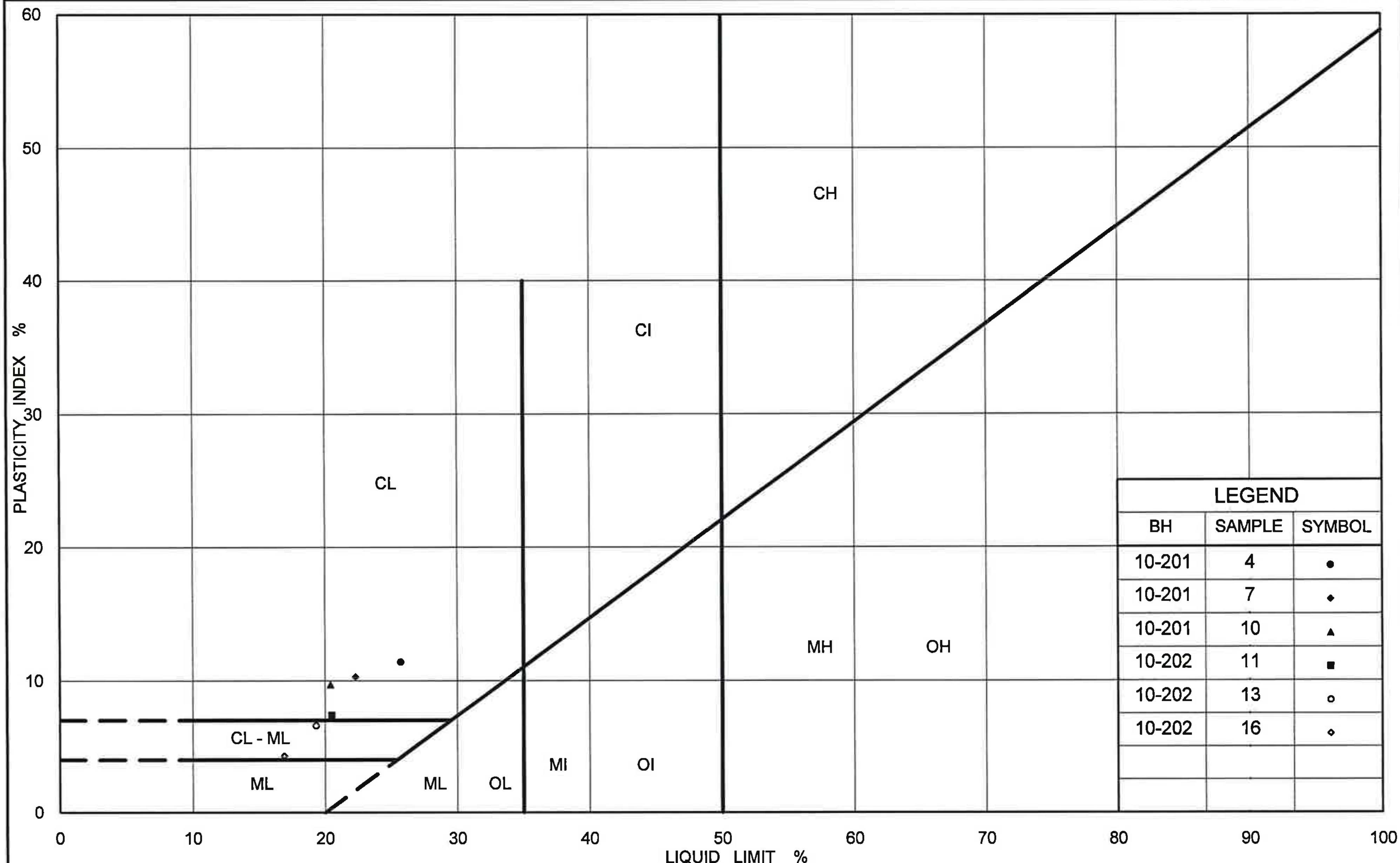
Date: 18-May-11

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt Till

Figure No. B4

Project No. 09-1111-6036

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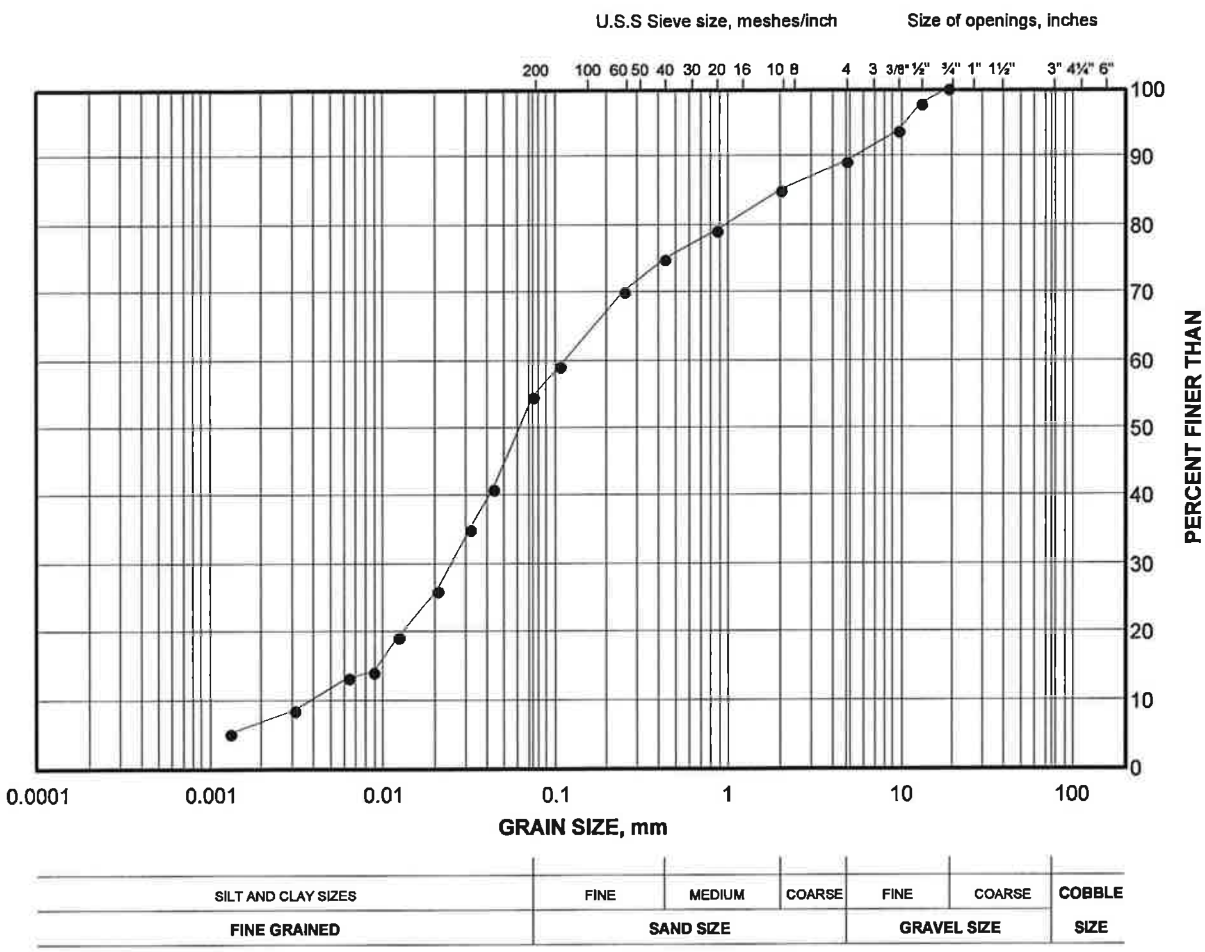
GRAIN SIZE DISTRIBUTION TEST RESULT

Sand and Silt Till

FIGURE B5



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-201	13	197.0

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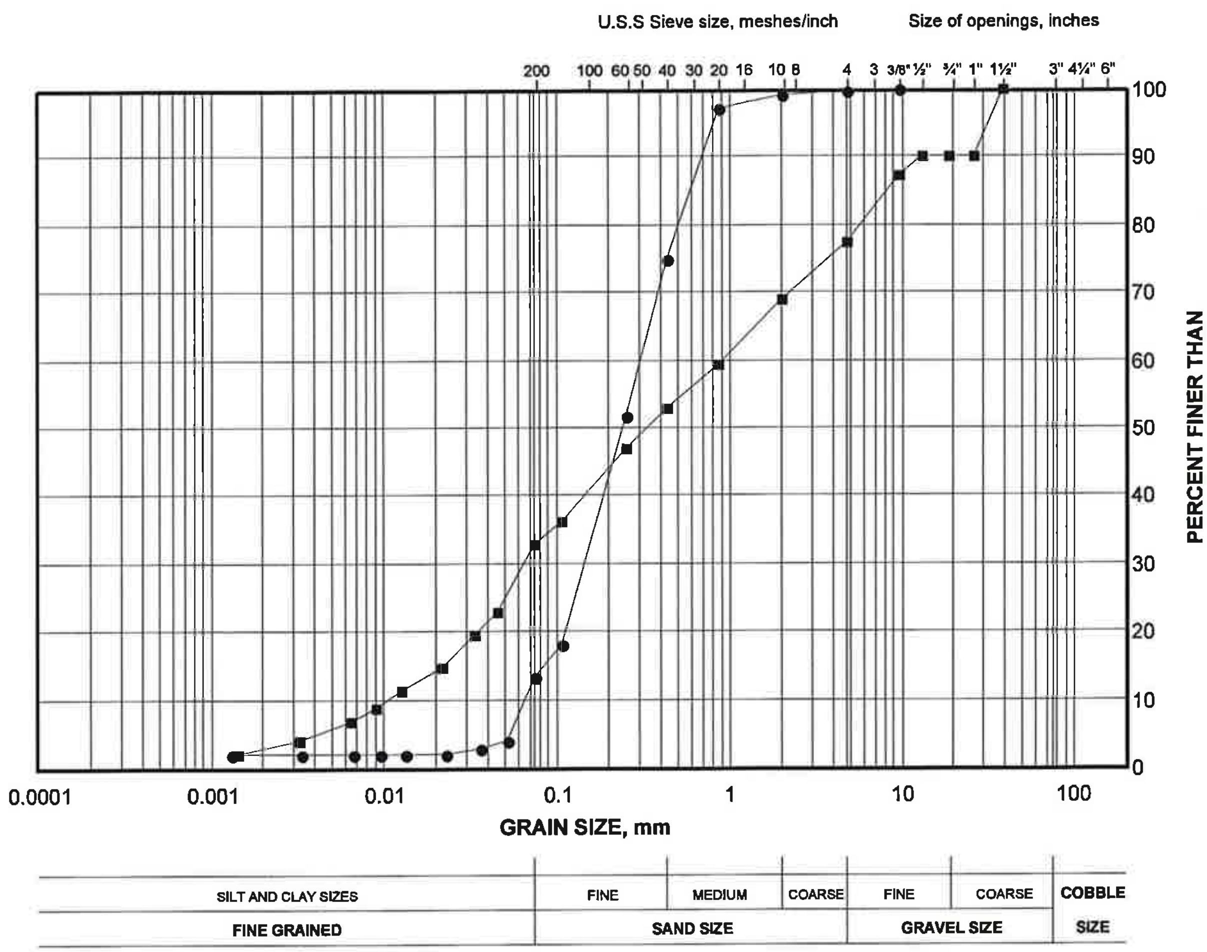
GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand to Silty Sand

FIGURE B6



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-201	15	193.9
■	10-201	17B	189.4

Project Number: 09-1111-6036

Checked By: LCC

Golder Associates

Date: 18-May-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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