October 2012

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

CP Rail Overhead Structure
Highway 401 Widening from East of Credit River to Trafalgar Road, Regional Municipalities of Peel and Halton
W.O. 07-20021

Submitted to:
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REPORT

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
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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 East of the Credit River in the Regional Municipality of Peel to Trafalgar Road (approximately 9.7 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 CP 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-0015 dated February 2010, and in Section 5.8 of the *Technical Proposal* for this assignment.

2.0 SITE DESCRIPTION

The existing CP Rail tracks and Highway 401 overhead structure are located approximately 1 km east of Mississauga Road in the City of Mississauga, in the Regional Municipality of Halton, Ontario. The existing single-span overhead structure was built in 1957 and rehabilitated in approximately 2000. It is 12.5 m long and 37.2 m wide, with abutments and associated wingwalls 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 172 m to 173 m. The CP Rail tracks have been constructed near the original ground surface, with the rail grade below Highway 401 at about Elevation 173 m.

Highway 401 has been constructed on embankment fill that is up to about 9 m to 10 m in height, with the pavement surface at about Elevation 181 m to 182 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 September 2011, during which time two boreholes (Boreholes 11-301 and 11-302) were advanced using a track-mounted CME-55 drill rig, supplied and operated by Geo-Environmental Drilling Inc. of Milton, Ontario. The borehole locations are shown on Drawing 1: Borehole 11-301 was advanced in the southeast quadrant and Borehole 11-302 was advanced in the northeast quadrant. Both boreholes were located near the toes of the Highway 401 embankments where the ground surface is close to the CP Rail grade.

Boreholes 11-301 and 11-302 were drilled using 108 mm inner diameter hollow stem augers and terminated at depths of 11.3 m and 11.2 m due to auger refusal. 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 driven by an automatic hammer 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 standpipe piezometer was installed in Borehole 11-302 to permit monitoring of the groundwater level. The 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 installation and measured water level(s) are indicated on the borehole records contained in Appendix A. The remaining borehole (Borehole 11-301) was backfilled with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended).

The field work was supervised on a full-time basis by members of Golder's staff who located the boreholes in the field, contacted public utility companies to locate the existing underground services and cleared the borehole locations, 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 ASTM and/or MTO LS procedures.

The as-drilled borehole locations and ground surface elevations were surveyed by Callon Dietz and provided to Golder. 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)
11-301	4,829,979.9	285,821.9	172.2	11.3
11-302	4,830,051.0	285,794.6	172.5	11.2

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 401 is located in the Peel Plain close to the border of the South Slope 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 overburden within the majority of the Peel Plain area is underlain by shale bedrock of the Georgian Bay Formation which contains limestone interlayers.

The South Slope region slopes gradually downward towards Lake Ontario. The overburden immediately below ground surface within the South Slope generally consists of clayey silt till and silty clay till and at depth consists of alternating deposits of dense lacustrine sands and silts and overconsolidated lacustrine clays and clay tills overlying the bedrock.

4.2 Subsurface Conditions

As part of the subsurface investigation, two boreholes were advanced at the CP 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 B4 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 topsoil overlying a thin surficial deposit of clayey silt, which is underlain by a deposit of clayey silt till. The clayey silt till is underlain by a deposit of silty sand and gravel till which is in turn underlain by shale bedrock. A more detailed description of the soil deposits encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

Approximately 100 mm of topsoil was encountered immediately below the existing ground surface in Boreholes 11-301 and 11-302, which were advanced near the toe of the Highway 401 embankment in the southeast and northeast quadrants of the structure site.

4.2.2 Surficial Clayey Silt

A surficial deposit of clayey silt was encountered below the topsoil in both boreholes, with its surface at approximately Elevation 172.1 m in Borehole 11-301 and at approximately Elevation 172.4 m in Borehole 11-302. The deposit is approximately 0.6 m in both boreholes.

This deposit consists of clayey silt with sand, containing trace gravel; rootlets were observed within the sample recovered from Borehole 11-301. The measured water contents on two selected samples of this deposit are 6 per cent and 27 per cent. The result of a grain size distribution test completed on one selected sample of the clayey silt deposit is shown on Figure B1 in Appendix B.

The deposit has a firm to very stiff consistency based on measured Standard Penetration Test (SPT) "N" values of 6 blows and 29 blows per 0.3 m of penetration.

4.2.3 Clayey Silt Till

A 9.3 m to 9.5 m thick deposit of clayey silt till was encountered below the surficial clayey silt deposit in both boreholes, extending to about Elevation 162.3 m.

This till deposit consists of clayey silt with sand, containing trace to some gravel. Cobbles and/or boulders are anticipated to be encountered within this deposit based on evidence of hard drilling (including grinding augers as noted on the borehole records). The results of grain size distribution tests completed on two selected samples of the clayey silt till are shown on Figure B2 in Appendix B. Atterberg limits testing was conducted on four selected samples of the clayey silt till, and measured plastic limits of 12 per cent to 14 per cent, liquid limits of 17 per cent to 23 per cent, and plasticity indices of 5 per cent to 9 per cent. These test results, which are plotted on a plasticity chart on Figure B3 in Appendix B, confirm that the till deposit is comprised of clayey silt of low plasticity. The water contents measured on selected samples of the clayey silt till range from approximately 7 per cent to 15 per cent.

The measured SPT "N" values within the clayey silt till deposit range from 11 blows to greater than 100 blows per 0.3 m of penetration, suggesting a stiff to hard consistency. The lowest SPT "N" values of 11 blows to 15 blows per 0.3 m of penetration were measured in the upper 0.8 m to 1.5 m of the clayey silt till deposit.

4.2.4 Silty Sand and Gravel Till

A 1.0 m to 1.4 m thick layer of silty sand and gravel till was encountered below the clayey silt till deposit in both boreholes. The surface of the silty sand and gravel till was encountered at a depth of about 9.9 m (Elevation 162.3 m) in Borehole 11-301 and at a depth of about 10.2 m (Elevation 162.3 m) in Borehole 11-302. The base of this layer is at approximately Elevation 160.9 m and 161.3 m in Boreholes 11-301 and 11-302, respectively.

The silty sand and gravel till contains trace clay. The result of a grain size distribution test completed on a selected sample of the silty sand and gravel till is shown on Figure B4 in Appendix B. The measured water content on two selected samples of this deposit were approximately 6 per cent and 7 per cent.

The measured SPT "N" values within the silty sand and gravel till range from 32 blows to greater than 100 blows per 0.3 m of penetration, indicating a dense to very dense relative density.

4.2.5 Shale Bedrock

Shale bedrock has been inferred below the till deposits in both boreholes, based on auger refusal in both boreholes and a small amount of grey shale bedrock recovered from the split-spoon sampler in Borehole 11-302. The inferred bedrock surface is at approximately Elevation 160.9 m and 161.3 m in Boreholes 11-301 and 11-302, respectively. This is approximately 12 m below the CP rail grade and about 20 m to 21 m below the Highway 401 grade.

4.3 Groundwater Conditions

Details of the groundwater conditions observed in the open boreholes at the time of drilling are summarized on the borehole records contained in Appendix A. A standpipe piezometer was installed in Borehole 11-302 within the silty sand and gravel till and the lower portion of the clayey silt till to monitor the groundwater level at the site. The water level measured in the piezometer is summarized in the following table, and shown on the borehole record in Appendix A.

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level	Groundwater Elevation	Date
11-302	172.5	1.7 m	170.8 m	November 2, 2011

The groundwater level should be expected to fluctuate seasonally and should be expected to rise during wet periods of the year.

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Billy Murphy and Mr. Mehdi Mostakhdemi, M.Sc., M.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
CP RAIL OVERHEAD STRUCTURE
HIGHWAY 401 WIDENING FROM EAST OF CREDIT RIVER TO TRAFALGAR ROAD, REGIONAL MUNICIPALITIES OF PEEL AND HALTON
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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed widening of the existing Highway 401-CP 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 east of Credit River 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 CP Rail overhead structure will require widening to the north and south; full replacement of the existing structure is also under consideration.

The existing CP Rail overhead is a single-span structure, with the existing abutments and wingwalls supported on spread footings. Based on the *General Arrangement* drawing for the 1957 construction, the east and west abutment footings were to be founded at approximately Elevation 170.1 m. This founding elevation is about 2.5 m to 2.9 m below the CP 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 CP 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. Shallow foundations would allow for the construction of semi-integral abutments for a new structure and/or if the existing structure can be modified to be compatible with the widening. It is expected that excavation will be required to a depth of about 2.5 m to 2.9 m below rail grade in the widening area either to match the existing footings, or in the case of a replacement structure to extend below the stiff upper portion of the clayey silt till deposit to be supported on very stiff to hard clayey silt till. Temporary excavation support would likely be required along the north and south sides of Highway 401 as well as along the CP 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 potential for approximately 25 mm of settlement (the majority associated with the clayey silt deposit and the stiff upper portion of the clayey silt till deposit) under the widened embankment loading; this could be mitigated by subexcavating a portion of the near-surface soils. "Perched" footings are also not structurally compatible with the existing structure configuration, although they could be appropriate if separate structures (for example, as may be possible for a core-collector system) are adopted or if a full replacement is adopted.

- **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 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 wingwalls/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 for this site but would require the use of temporary or permanent liners given the potential risks and difficulties associated with the water-bearing silty sand and gravel deposit through which the caissons would be constructed. Due to these risks and potential construction difficulties, caissons are not considered to be a preferred foundation system for this structure site and therefore are not discussed in detail in subsequent sections of this report. However, the relative advantages and disadvantages of caisson foundations are summarized in Table 1.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments for the north and south widening or replacement on shallow foundations, due to the relatively shallow depth to competent soil strata for support of foundations. If a full structure replacement is adopted, the use of driven steel H-pile foundations in an integral abutment configuration is considered to be an acceptable alternative from a foundations perspective.

6.3 Shallow Foundations

6.3.1 Founding Elevations

For support of widened or new abutments and associated wing walls or retaining walls, strip or spread footings should be founded below any fill and the stiff portion of the surficial clayey silt or clayey silt till deposits, on the very stiff to hard 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 170.1 m) and structurally 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 preliminary recommendations for founding 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 widening and/or replacement structure is established.

Foundation Element	Borehole No.	Founding Stratum	Strip/Spread Footing Founding Elevation (m)
South widening or southern portion of replacement	11-301	Very stiff to hard clayey silt till	Below 170.5 m
North widening or northern portion of replacement	11-302	Very stiff to hard clayey silt till	Below 170.5 m

Alternatively for a full replacement, subexcavation can be carried out to the elevation identified 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 Standards. 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 footing subgrade should be inspected by a Quality Verification Engineer (QVE) following excavation, in accordance with Provincial Standards, 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 (as discussed further in Section 6.6.3) 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/Reaction

For preliminary design, 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 a factored geotechnical resistance at Ultimate Limit States (ULS) of 450 kPa, and a geotechnical reaction at Serviceability Limit States (SLS, for 25 mm of settlement) of 350 kPa. These values assume a footing width of between 3 m and 4 m.

The geotechnical resistances/reactions 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/reaction 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 very dense silty sand and gravel till (with SPT “N” values of greater than 100 blows per 0.3 m of penetration) or the shale bedrock. 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 at the south widening/replacement, or nominal penetration of approximately 300 mm into the shale bedrock at the north widening/replacement:

Foundation Element	Borehole No.	End-Bearing Stratum	Estimated Design Pile Tip Elevation
South widening or south portion of replacement structure	11-301	Shale bedrock	160.9 m
North widening or north portion of replacement structure	11-302	Shale bedrock	161.0 m

The pile caps should be constructed at a minimum depth of 1.2 m for frost protection purposes, per Provincial Standards.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and/or 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 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.

6.4.2 Axial Geotechnical Resistance/Reaction

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 reaction at SLS (for 15 mm of settlement) may be taken as follows for preliminary design:

Foundation Element	Borehole No.	End-Bearing Stratum	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS
South widening or south portion of replacement structure	11-301	Shale bedrock	1,600 kN	1,400 kN
North widening or north portion of replacement structure	11-302	Shale bedrock	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.).

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 Highway 401 approach embankments. Consideration may also be given to stripping firm portions of the surficial clayey silt deposit, where these are present. 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.

Additional fill for construction of the embankment widening could consist of clean earth fill or granular fill. The embankment fill for the Highway 401 widening should be placed and compacted in accordance with MTO's Special Provisions. 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).*

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.

6.5.2 Approach Embankment Stability

Preliminary 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 CP Rail overhead site, considering the design requirements and the available field and laboratory testing data.

The preliminary stability analyses were completed for an approximately 10 m high embankment widening, based on the subsurface conditions as encountered in Boreholes 11-301 and 11-302. The following parameters have been used in the preliminary analyses, based on field and laboratory test data as well as accepted correlations:

Soil Conditions	Bulk Unit Weight (kN/m^3)	Effective Friction Angle	Undrained Shear Strength (kPa)
Embankment fill	21	32-35°	-
Firm to very stiff surficial clayey silt	20	30°	*
Very stiff to hard clayey silt till	21	35°	-
Dense to very dense silty sand and gravel till	21	34°	-

* Assuming weaker (firm) portions of the surficial clayey silt deposit are stripped from the embankment widening area.

The preliminary stability 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 1.3 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 the detail design investigations.

6.5.3 Approach Embankment Settlement

Based on the study completed to date, the existing approximately 10 m high Highway 401 embankment will require widening by approximately 20 m on both the north and south sides in this area.

Preliminary 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 Deposit	Bulk Unit Weight (kN/m^3)	Elastic Modulus (MPa)
Embankment fill	21	-
Firm to very stiff surficial clayey silt, assuming that weaker (firm) portions of deposit are stripped within the widening footprint	20	25
Stiff clayey silt till	21	25
Very stiff to hard clayey silt till	21	75
Dense to very dense silty sand and gravel till	21	75

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 30 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 and the upper portion of the till deposit 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 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.

6.6.1 Excavation and Temporary Protection Systems

The foundation excavations for spread footings are expected to extend to a depth of about 2.5 m to 2.9 m below the CP Rail grade, through the existing firm to stiff clayey silt and the stiff portion of the clayey silt till deposit, into the very stiff to hard clayey silt till deposit. The excavations for pile caps could be maintained higher within the native soils (or may even be perched within the embankment fill), but would still require excavation through the firm to very stiff clayey silt and potentially into the stiff portion of the 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 and the firm to stiff portions of the native soils 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 CP Rail tracks, to facilitate the removal of the existing wingwalls/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 CP 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 CP Rail tracks as part of the detail design, and having this design pre-approved by CP Rail in advance of tendering the contract, to expedite the construction schedule.

6.6.2 Groundwater Control

Groundwater seepage is anticipated from “perched” water within cohesionless fill or surficial deposits on top of the cohesive deposits, as well as from cohesionless lenses or interlayers within the clayey silt till deposit (if present). 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, it is expected that a Permit to Take Water (PTTW) would 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. The frequency of occurrence of cobbles and boulders should be identified during future investigations as part 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 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 and, therefore, vibration monitoring for the existing overhead structure is not expected to be required during construction at this site. However, there are several commercial and industrial buildings in the vicinity of the site and the requirements for monitoring of vibrations during construction, should be evaluated during the detail design stage. Vibration monitoring on the rail tracks may also be required by CP Rail. If warranted, an NSSP should be included in the Contract Documents at the detail design stage to develop a vibration monitoring plan that would include appropriate review and alert levels for vibrations for the existing buildings and the rail tracks.

6.7 Recommendations for Further Work in Detail Design

Additional boreholes will be required within each of the foundation widening/replacement 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 thickness and properties of any surficial deposits and the upper portion of the clayey silt till to confirm the founding elevation for spread footings within each foundation element, and to assess groundwater control requirements associated with seepage from surficial cohesionless soil deposits or lenses/interlayers in the till, if present.
  - Assessment of the depth and properties of the “100 blow soil” and/or bedrock to confirm pile tip elevations, if deep foundations are selected as the preferred option.
  - Observation of the presence and frequency 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 or temporary protection systems.

- Assessment of vibration thresholds for the CP Rail tracks and the nearby commercial/industrial buildings, and if warranted development of an NSSP for a vibration monitoring plan.
- For consideration: Development of a preliminary design for the temporary protection system along the CP Rail tracks, and pre-approval by CP Rail in advance of tendering the contract, to expedite the construction schedule if required.

- Approach embankments and adjacent high fill embankments:
  - Assessment of the depth and extent of stripping of topsoil/organics and weak surficial soils within the footprint of the widened approach embankments.
  - Further assessment of the thickness and consolidation/elastic compression properties of the soils within the footprint of the widened approach embankments, to confirm the settlement estimates.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Billy Murphy and Mr. Mehdi Mostakhdemi, M.Sc., M.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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n:\active\2010\1111\10-1111-0040 urs - highway 401 widening - halton peel\6 - reports\3 - cp rail overhead\10-1111-0040-3 rpt 12oct cp rail overhead.docx

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Ontario Provincial Standard Specifications (OPSS)

OPSS 501	Construction Specification for Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 572	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 3000.100	Foundation Piles – Steel H-Pile Driving Shoe
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario

Construction Design Estimating and Documentation (CDED) Special Provisions (SP)

SP 105S21	Amendment to OPSS 501 – Construction Specification for Compacting
SP 206S03	Earth Excavation and Grading

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 new or 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	• Excavation to a depth of approximately 2.5 m to 2.9 m below CP Rail grade; 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 about $600/m^3 for a concrete unit 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 considered feasible at this site due to incompatibility with existing structure; if full replacement is adopted there would be approximately 30 mm of settlement of the widened portion of the embankment	• Abutment pile caps could be maintained higher than footings founded on till deposit, reducing excavation depth and associated temporary protection system requirements	• Not compatible with existing “closed” structure configuration; longer span with abutment foreslopes would be required • Potential for greater total and differential settlement than for spread footings supported directly on the very stiff to hard till deposit	• Conventional excavation and construction techniques	• Not assessed as this option not recommended at this site

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel H-piles driven to found in very dense till or shale bedrock	• Feasible for support of widened or new 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 CP 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	• Potential for encountering obstructions (cobbles and/or boulders) during pile driving that could result in piles “hanging up” and lower geotechnical resistances • Potential for noise and/or vibration impacts on nearby buildings and rail tracks	• Conventional construction methods for H-pile foundations	• Lower relative cost compared with caisson option • Estimated unit cost is approximately $250/linear metre 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 or new 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 CP 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 • Potential for noise and/or vibration impacts on nearby commercial buildings or rail tracks	• 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 widened or new 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 CP Rail • Higher capacity than for steel piles, so reduced number of deep foundation elements compared to steel H-piles	• Risk of ground disturbance in water-bearing silty sand and gravel till; temporary or permanent liners would be required; likely not possible to inspect caisson base • Precludes use of integral abutments	• Risk of ground disturbance in water-bearing silty sand and gravel till deposit	• Higher cost compared with shallow foundations or driven piles

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

CONT No.
WO No. 07–20021

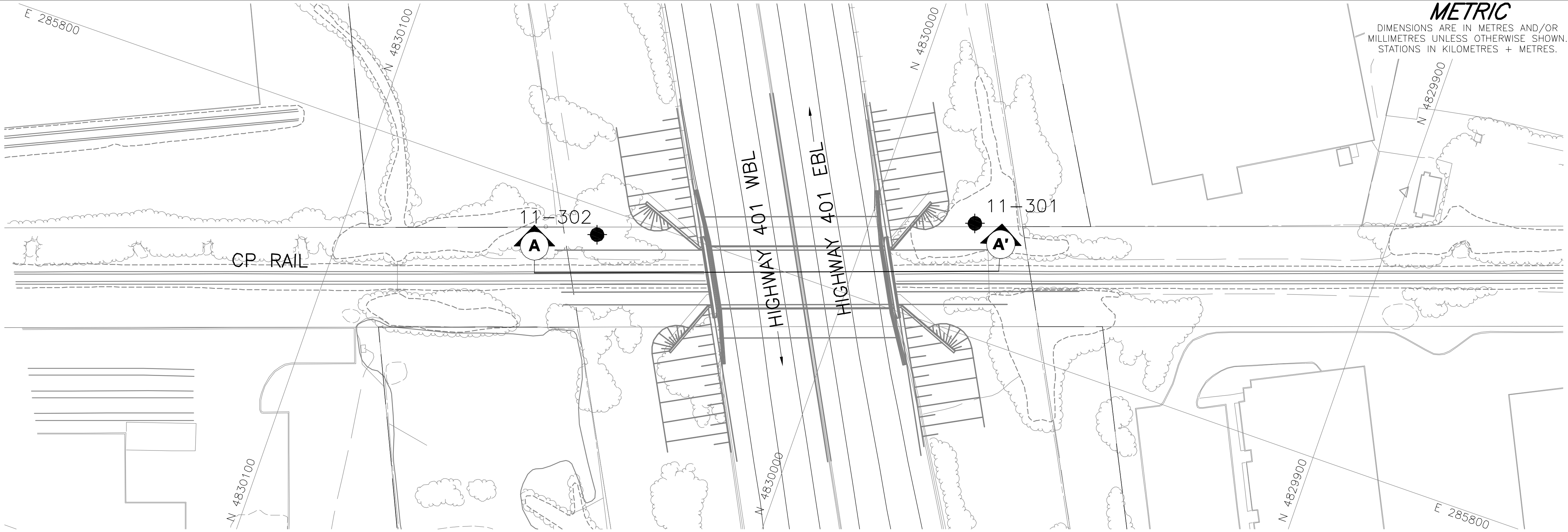
SHEET

CP RAIL OVERHEAD

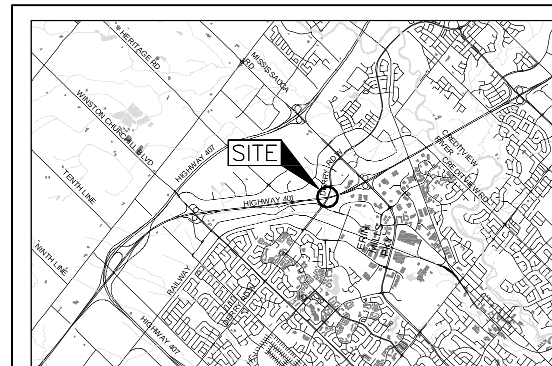
HIGHWAY 401 IMPROVEMENTS

BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN
SCALE
10 0 10 20 m

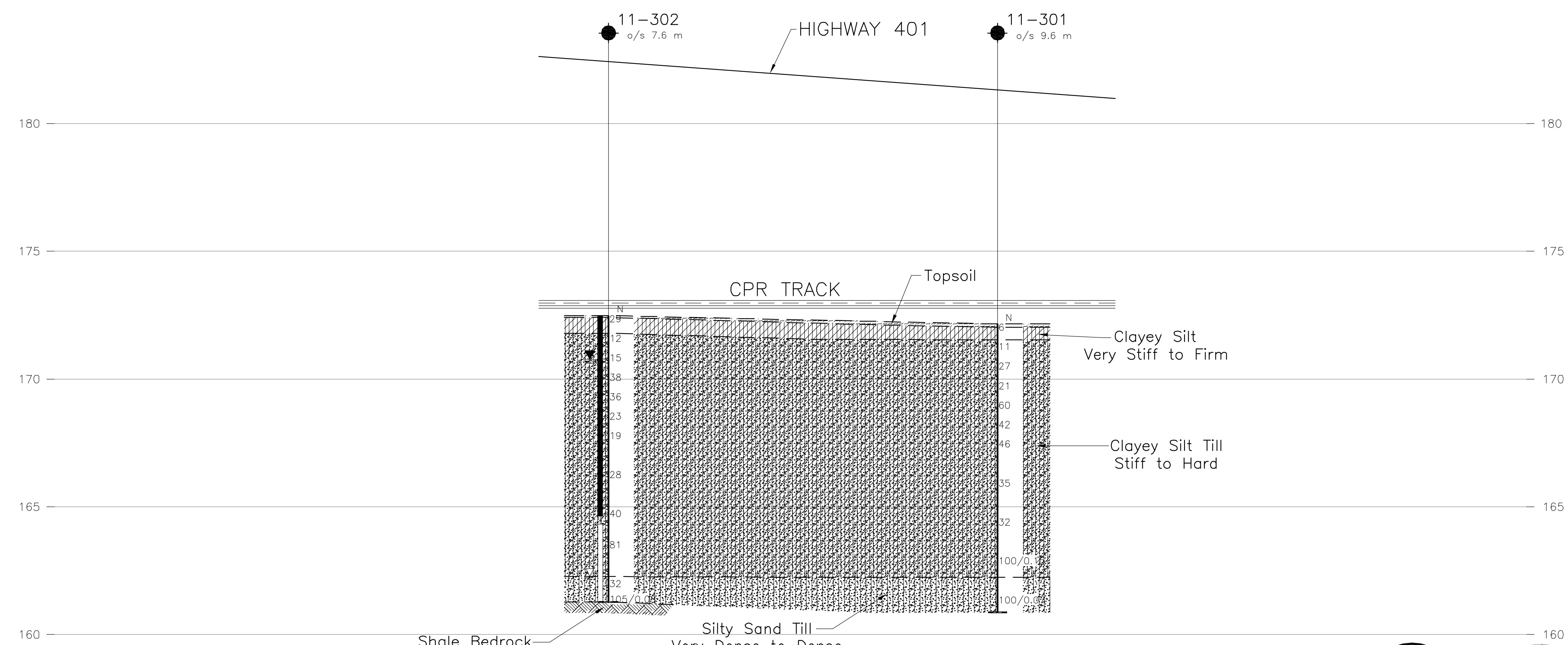


KEY PLAN
SCALE
1.5 0 1.5 3 km

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer, measured on November 2, 2011
- WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
11–301	172.2	4829979.9	285821.9
11–302	172.5	4830051.0	285794.6



PROFILE A–A'
A–A'
HORIZONTAL SCALE
10 0 10 20 m
2 0 2 4 m
VERITCAL SCALE

NOTES

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

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 nos. ACAD–X–base1_to_Trafalgar.dwg, ACAD–Aerials_MTO ROW_ Property Boundaries..dwg and CPR_Overpass_GA.dwg , received August 17, 2011, August 29, 2011 and August 16, 2011

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Oct. 23, 2012 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER T. J. GARDE Oct. 23, 2012 PROVINCE OF ONTARIO

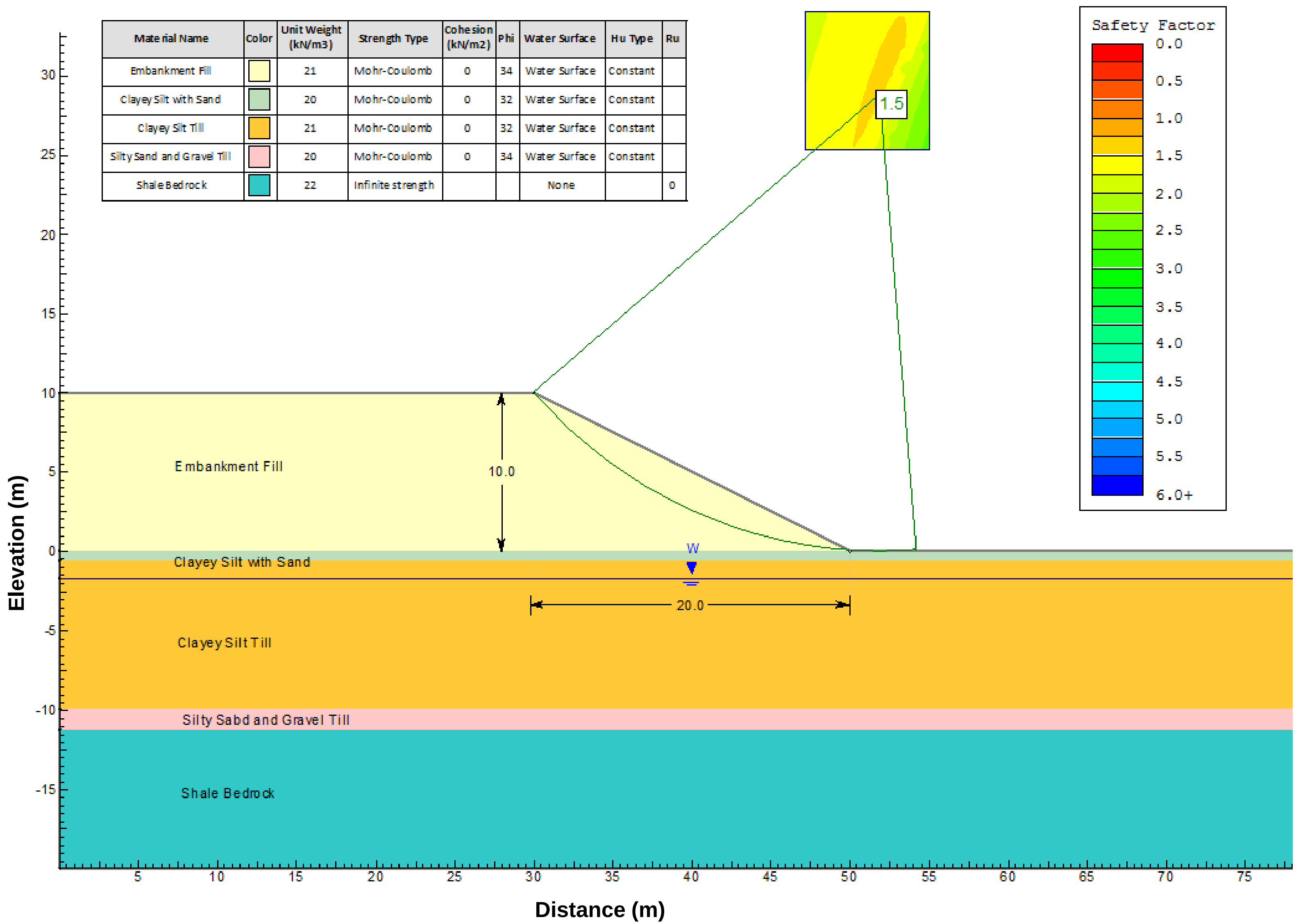
NO.	DATE	BY	REVISION

Geocres No. 30M12–347			
HWY. 401	PROJECT NO. 10–1111–0040		DIST.
SUBM'D. MM	CHKD. LCC	DATE: 10/23/2012	SITE:
DRAWN: JFC	CHKD. MM	APPD. LCC	DWG. 1

PLOT DATE: October 23, 2012
FILENAME: T:\Projects\2010\10-1111-0040 (URS, Halton Peel)\-EA- (CPR Overhead)\1011110040EA001.dwg

Static Global Stability – CP Overhead Approach Embankments

Figure 1



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kN/m2)	Phi	Water Surface	Hu Type	Ru
Embankment Fill		21	Mohr-Coulomb	0	34	Water Surface	Constant	
Clayey Silt with Sand		20	Mohr-Coulomb	0	32	Water Surface	Constant	
Clayey Silt Till		21	Mohr-Coulomb	0	32	Water Surface	Constant	
Silty Sand and Gravel Till		20	Mohr-Coulomb	0	34	Water Surface	Constant	
Shale Bedrock		22	Infinite strength			None		0

Date: February 2012

Project No: 10-1111-0040

Analysis By: BM **Reviewed By:** LCC

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APPENDIX A

Borehole Records

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

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

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

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

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils
Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

Percent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (cohesionless) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

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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

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 ($\sigma' = \sigma$ - u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	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_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{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 δ
c′	effective cohesion
c_u, s_u	undrained shear strength (ϕ = 0 analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

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

2 shear strength = (compressive strength)/2

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RECORD OF BOREHOLE No 11-301

SHEET 1 OF 1 **METRIC**

PROJECT 10-1111-0040
G.W.P. 07-20021 LOCATION N 4829979.9 ;E 285821.9 ORIGINATED BY SB/BM
DIST Central HWY 401 BOREHOLE TYPE Track-Mounted CME55, 108 mm I.D. Hollow Stem Augers COMPILED BY BM/MM
DATUM NAD83 DATE September 7, 2011 CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%) w	GR	SA	SI	CL
172.2	GROUND SURFACE									
0.0	TOPSOIL				172					
0.1	CLAYEY SILT with sand, containing rootlets, Firm, Brown, Moist	1	SS	6		~27				
171.6 / 0.6	CLAYEY SILT with sand, trace to some gravel, containing cobbles or boulders (TILL), Stiff to hard, Grey to brown to brown, Moist	2	SS	11	171					
		3	SS	27	170	~15				
		4	SS	21						
	Auger grinding between 2.9 m and 3.1 m	5	SS	60	169	~7	11	31	46	12
	Auger grinding between 3.7 m and 3.8 m	6	SS	42	168					
		7	SS	46	167	~8				
		8	SS	35	166					
		9	SS	32	165 / 164	~10	5	45	35	15
	Auger grinding between 8.2 m and 9.1 m	10	SS	100/0.15	163					
162.3 / 9.9	Silty SAND and gravel, trace clay (TILL), Very dense, Grey, Moist	11	SS	100/0.07	162	~7				
160.9 / 11.3	END OF BOREHOLE DUE TO AUGER REFUSAL				161					

NOTES:

1. Borehole was open to depth of 6.7 m (Elevation 165.5 m) upon completion of drilling.

2. Borehole was dry to a depth of 6.7 m upon completion of drilling.

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 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 (%)

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

Golder Associates

RECORD OF BOREHOLE No 11-302

SHEET 1 OF 1 **METRIC**

PROJECT 10-1111-0040 | G.W.P. 07-20021 | DIST Central HWY 401 | DATUM NAD83

LOCATION N 4830051.0 ;E 285794.6

BOREHOLE TYPE Track-Mounted CME55, 108 mm I.D. Hollow Stem Augers

DATE September 21, 2011

ORIGINATED BY AM | COMPILED BY BM/MM | CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
172.5	GROUND SURFACE					
0.0 0.1	TOPSOIL					
	CLAYEY SILT with sand, containing rootlets, Very stiff, Brown, Moist	1	SS	29	o	3 59 20 18
171.8 0.7	CLAYEY SILT, trace to some sand, trace gravel (TILL), Stiff to hard, Brown, Moist	2	SS	12		
		3	SS	15		
		4	SS	38	o ├─┤	
		5	SS	36		
		6	SS	23	o	
168.0 4.5	CLAYEY SILT with sand, trace to some gravel, containing cobbles or boulders (TILL), Very stiff to hard, Brown to grey, Moist	7	SS	19		
		8	SS	28	o ├─┤	
	Auger grinding between 7.6 m and 8.8 m	9	SS	40		
		10	SS	81		
	Auger grinding between 9.8 m and 10.4 m					
162.3 10.2	Silty SAND and gravel, trace clay (TILL), Dense, Grey, Wet	11	SS	32	o	37 39 20 4
161.3 11.2	Weathered SHALE	12A 12B	SS	105/0.08		
	END OF BOREHOLE DUE TO AUGER REFUSAL					

Elevation scale: 172, 171, 170, 169, 168, 167, 166, 165, 164, 163, 162

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m^3

NOTES:

1. Groundwater was encountered at a depth of 10.2 m (Elevation 162.3 m) in open borehole upon completion of drilling.

2. Water level in monitoring well measured as follows:

Date	Depth	Elevation
11/02/11	1.7	170.8

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

GTA-MTO 001 1011110040.GPJ GAL-MISS.GDT 7/9/12 DD

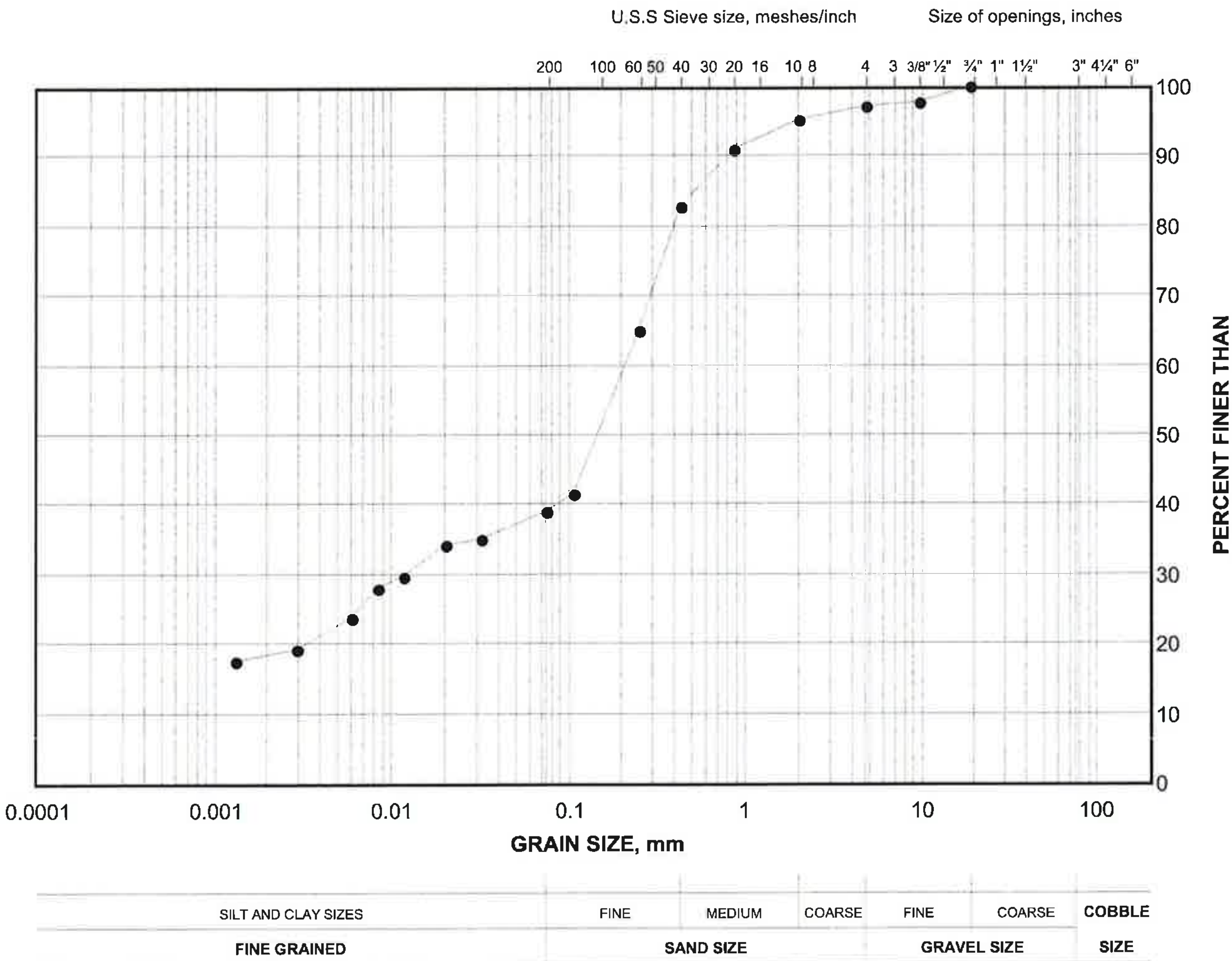
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE B1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	11-302	1	172.2

Project Number: 10-1111-0040

Checked By: *[signature]*

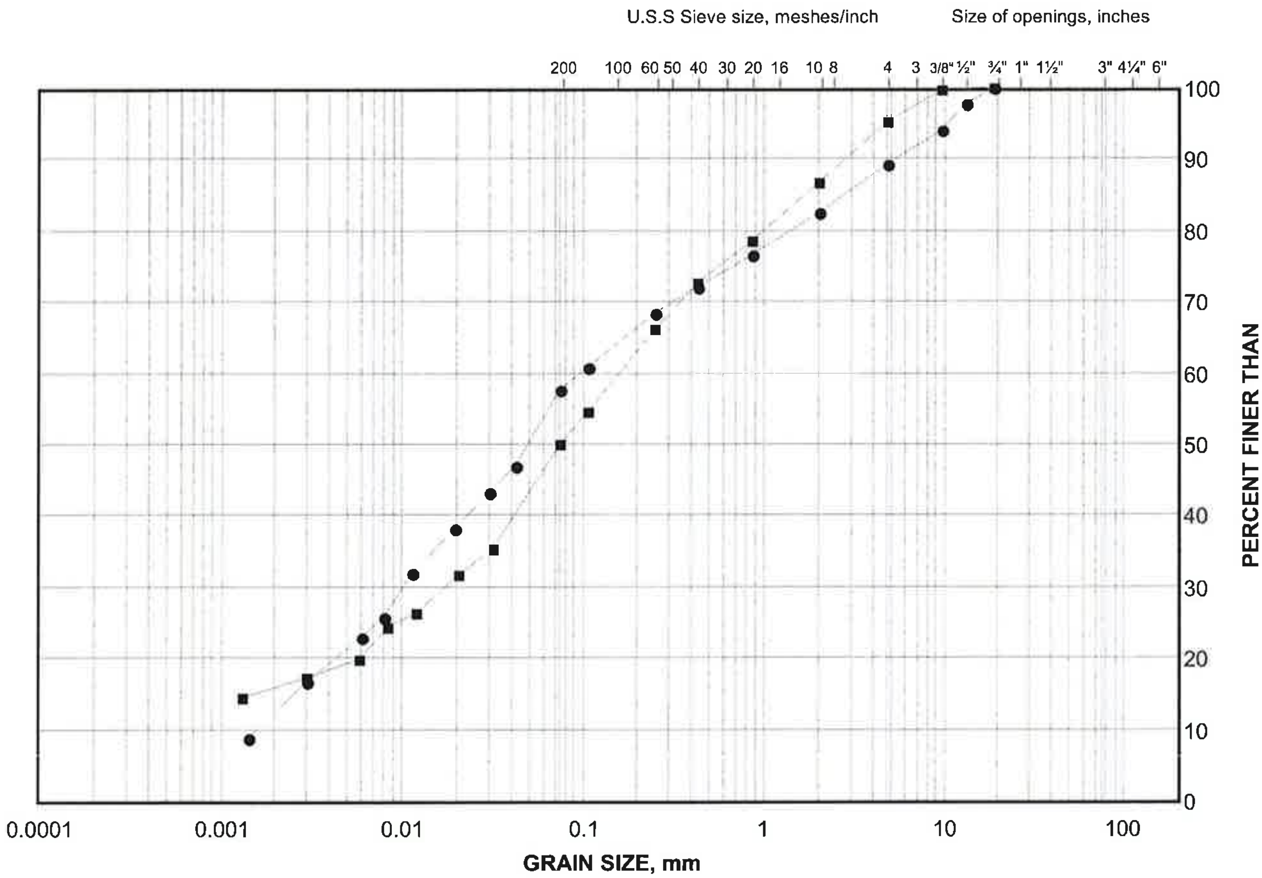
Golder Associates

Date: 02-Feb-12

GRAIN SIZE DISTRIBUTION

Clayey Silt Till

FIGURE B2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	11-301	5	168.8
■	11-301	9	164.5

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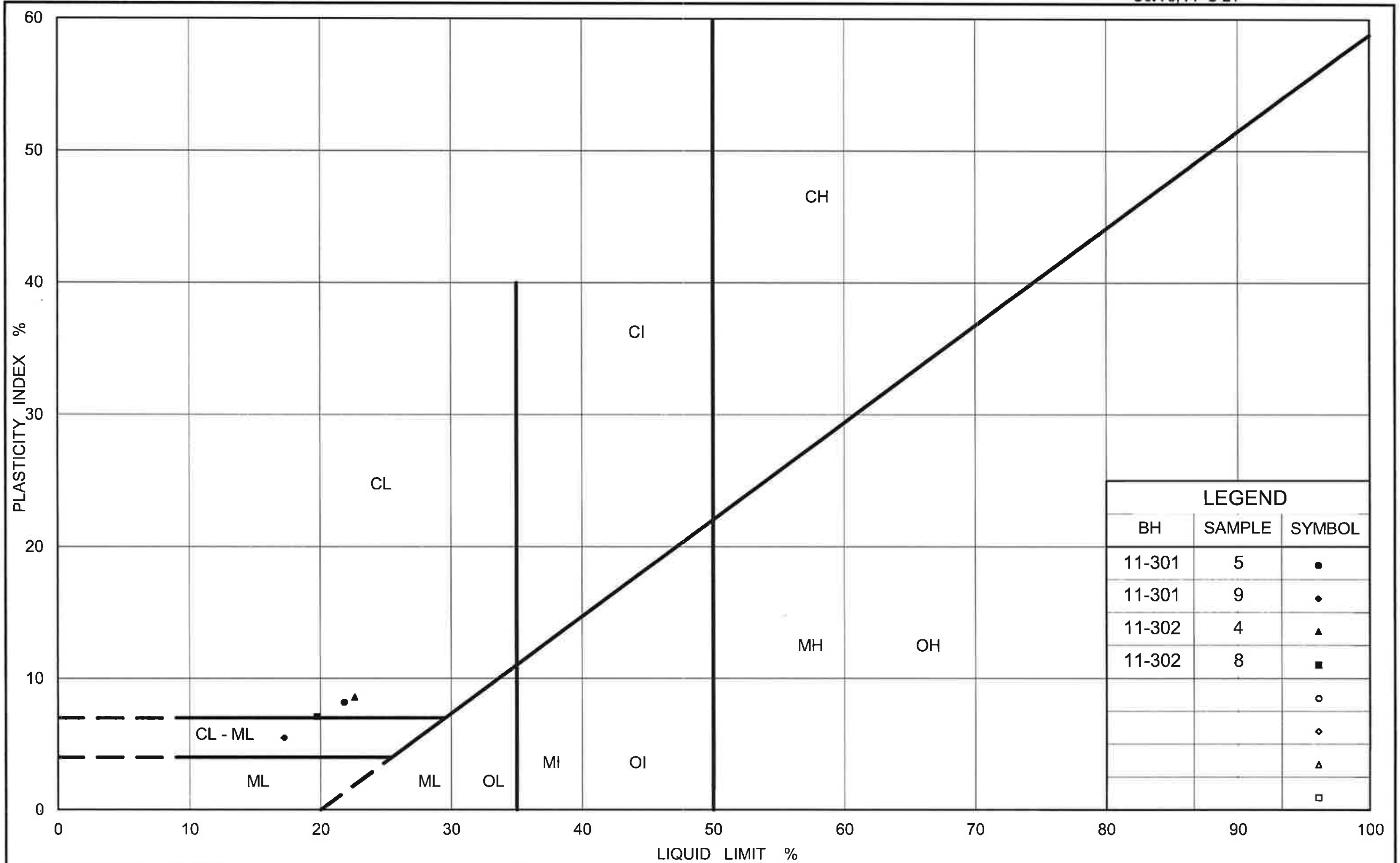
Date: 02-Feb-12

Oct 75, FF-S-21



Ministry of Transportation Ontario	**PLASTICITY CHART** **Clayey Silt Till**

Figure No. B3

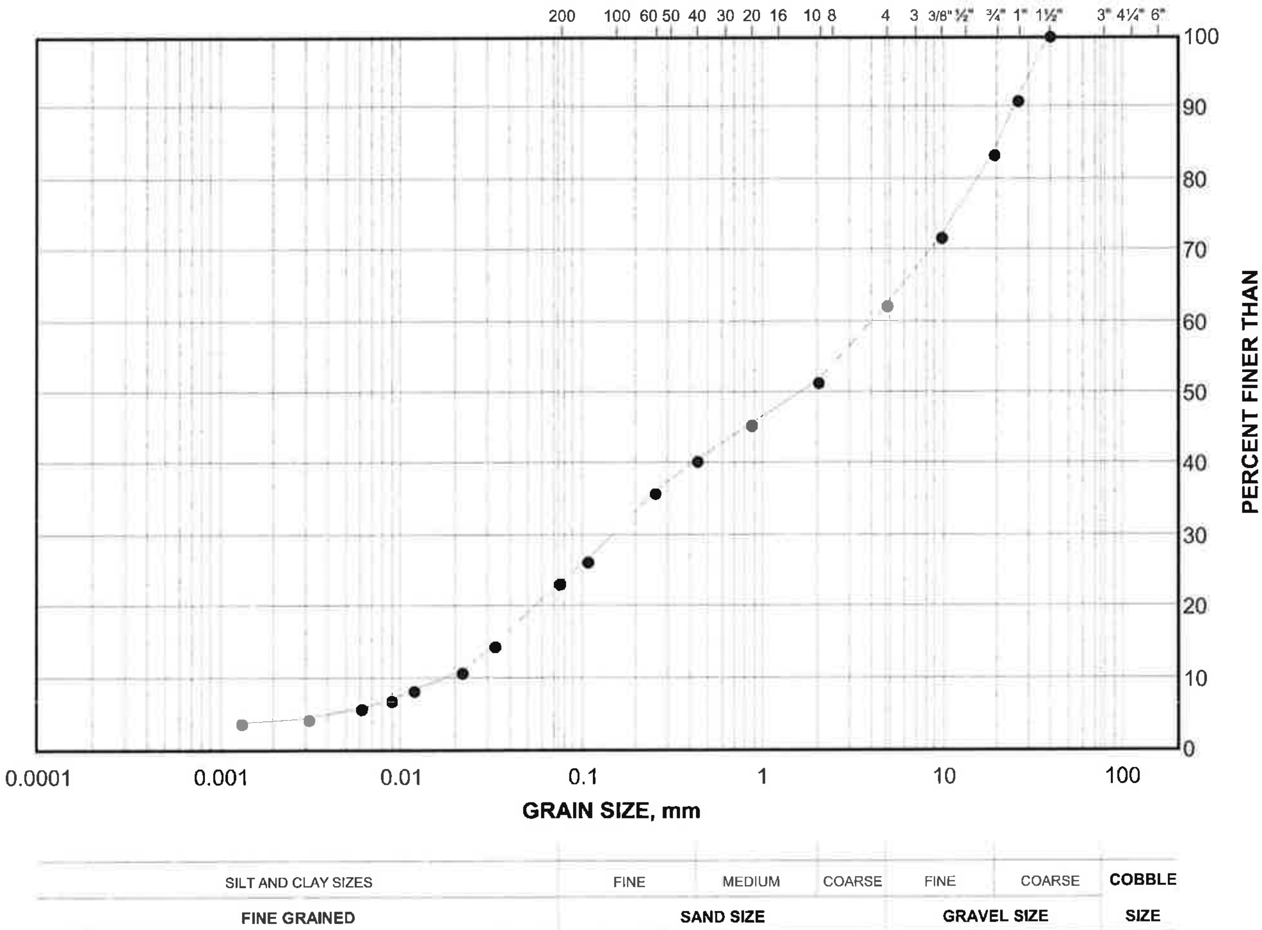
Project No. 10-1111-0040

Checked By:

GRAIN SIZE DISTRIBUTION

Silty Sand and Gravel Till

FIGURE B4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	11-302	11	162.0

Project Number: 10-1111-0040

Checked By: ______

Golder Associates

Date: 02-Feb-12

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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