October 2012

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Creditview Road Underpass
Highway 401 Improvements from East of the Credit River to Trafalgar Road, Regional Municipalities of Peel and Halton
W.O. 07-20021

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REPORT

Golder Associates

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
CREDITVIEW ROAD UNDERPASS
HIGHWAY 401 WIDENING FROM EAST OF THE CREDIT RIVER TO TRAFALGAR ROAD, REGIONAL MUNICIPALITIES OF PEEL AND HALTON
W.O. 07-20021

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 replacement/realignment of the existing Creditview Road underpass structure.

The terms of reference and scope of work for the foundation engineering services are outlined in MTO's Request for Proposal (RFP) for Assignment No. 2008-E-0015 dated February 2010, and in Section 5.8 of the *Technical Proposal* for this assignment.

2.0 SITE DESCRIPTION

The Creditview Road underpass structure is located at the intersection of Highway 401 and Creditview Road in the City of Mississauga, within the Regional Municipality of Halton, Ontario. The existing underpass consists of a 65 m long by 10 m wide four-span structure, with the existing abutments supported on battered piles and the piers 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 at about Elevation 171 m on the north side of the highway and about Elevation 168.5 m on the south side. The existing Highway 401 grade is at approximately Elevation 169.5 m to 170.5 m.

Creditview Road has been constructed on embankment fill that is up to about 7.5 m in height at the south approach and up to about 5 m in height at the north approach. The pavement grade on Creditview Road is at approximately Elevation 176.1 m at the abutments, up to about Elevation 176.5 m at the structure crown. The abutment foreslopes and embankment side slopes are oriented at approximately 2 horizontal to 1 vertical (2H:1V).

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out in September 2011, at which time two boreholes (Boreholes 11-201 and 11-202) 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: Boreholes 11-201 and 11-202 were advanced in the southeast and northwest quadrants of the interchange through the Creditview Road embankments.

Boreholes 11-201 and 11-202 were drilled using 108 mm inner diameter hollow stem augers through the overburden and then advanced by bedrock coring to depths of 14.2 m and 16.4 m, respectively. Soil samples were obtained at 0.75 m to 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. Samples of the bedrock were obtained using an 'NQ' size rock core barrel.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations, and a standpipe piezometer was installed in Borehole 11-201 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 piezometer installation details and water level readings are indicated on the borehole record contained in Appendix A. The remaining borehole (Borehole 11-202) was backfilled with bentonite pellets upon completion, in accordance with Ontario Regulation 903 (as amended).

The field work was supervised on a full-time basis by a member 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. Strength testing (uniaxial compression and point load index testing) was carried out on selected rock core specimens. The geotechnical laboratory testing was completed according to applicable MTO LS standards.

The location of the boreholes and ground surface elevations were measured in the field by Callon Dietz. The borehole locations (referenced to the MTM NAD83 co-ordinate system) and ground surface elevations (referenced to geodetic datum) are summarized in the following table and are shown on Drawing 1.

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
11-201	4,830,180.6	286,216.4	169.7	14.2
11-202	4,830,227.6	286,104.2	173.4	16.4

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 current subsurface investigation, two boreholes were advanced in the vicinity of the existing Creditview Road underpass structure. The borehole locations, ground surface elevations and interpreted stratigraphic conditions at the site are shown on Drawing 1. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the borehole records contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B9 contained in Appendix B. The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic 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 subsoil conditions encountered at the site consist of embankment fill overlying a deposit of very stiff to firm clayey silt, which is underlain by a deposit of very stiff to hard clayey silt till, which grades to sand and silt till in one of the boreholes. The till is underlain by shale bedrock of the Georgian Bay Formation. A more detailed description of the soil deposits encountered in these boreholes is provided in Sections 4.2.1 to 4.2.6.

In addition, two boreholes and two dynamic cone penetration test holes were advanced at this site as part of a 1957 investigation conducted by the Department of Highways Ontario *("Foundation Report on New Bridge at Highway No. 401 and Road Allowance between Concessions 3 and 4, One Mile South of Meadowvale, W.P. 75-57",* dated September 1957). The boreholes encountered firm to stiff clay, with measured Standard Penetration Test (SPT) "N" values ranging from 9 blows to 11 blows per 0.3 m of penetration underlain by stiff to hard clay till, with measured SPT "N" values ranging from 14 blows to 72 blows per 0.3 m of penetration. The report noted that the presence of boulders made driving the casings very difficult, such that the boreholes were terminated at depths of about 9.1 m (30 ft.) to 11.7 m (39 ft.). The records for these boreholes are included in Appendix C, and their approximate locations are shown on Drawing 1.

4.2.1 Topsoil

Approximately 100 mm of topsoil was encountered immediately below the ground surface in both Boreholes 11-201 and 11-202, which were advanced in the southeast and northwest quadrant of the structure site, respectively.

4.2.2 Fill

Approximately 2.1 m of fill or reworked native soil was encountered immediately below the topsoil layer in Borehole 11-201, which was advanced south of Highway 401. This fill extends to a depth of about 2.2 m (Elevation 167.5 m), and consists of clayey silt with sand containing some gravel, as well as trace quantities of organic matter. This fill has a firm to stiff consistency, based on measured SPT "N" values of 5 blows to 8 blows per 0.3 m of penetration.

Approximately 2.2 m of fill or reworked native soil was encountered immediately below the topsoil layer in Borehole 11-202, which was advanced north of Highway 401. This fill extends to about Elevation 171.1 m. This fill varies in composition from clayey silt with sand containing trace gravel, to sand and gravel containing some silt. The measured SPT "N" values within the clayey silt fill are 13 blows and 15 blows per 0.3 m penetration, suggesting a stiff to very stiff consistency. One SPT "N" value of 64 blows per 0.3 m of penetration was measured in the gravelly sand fill, indicating that this layer has a very dense relative density.

The results of grain size distribution tests completed on one sample of the clayey silt fill and one sample of the gravelly sand fill are shown on Figures B1 and B3 in Appendix B. Atterberg Limits testing was completed on a sample of the clayey silt fill, and measured a plastic limit of 15 per cent, a liquid limit of 22 per cent, and a plasticity index of 7 per cent; these results are plotted on a plasticity chart on Figure B2 in Appendix B. Laboratory testing of selected samples of the clayey silt fill materials measured natural water contents of approximately 9 per cent to 10 percent. The natural water content measured on a selected sample of the gravelly sand fill material is approximately 3 per cent.

4.2.3 Clayey Silt

An approximately 3.5 m to 5.0 m thick deposit of clayey silt containing varying amounts of sand and gravel was encountered below the fill in Boreholes 11-201 and 11-202, extending to about Elevation 164.0 m and 166.1 m, respectively. Based on the geological history of this area and the proximity of this site to the Credit River, this deposit has been interpreted to be either a "Peel pond" deposit on top of the clayey silt till sheet, or a deposit of "softened" till (potentially related to flooding of the Credit River valley during the last period of glacial melting).

The results of grain size distribution tests completed on two selected samples of the clayey silt are shown on Figure B4 in Appendix B. Atterberg limits testing was carried out on two selected samples of the deposit and measured plastic limits of about 14 per cent and 15 per cent, liquid limits of about 24 per cent and 26 per cent, and plasticity indices of 10 per cent and 11 per cent. These test results, which are plotted on a plasticity chart on Figure B5 in Appendix B, confirm that the deposit consists of low plasticity clayey silt. Laboratory testing of selected samples of the clayey silt measured natural water contents ranging from about 13 per cent to 28 per cent.

The measured SPT "N" values within the upper 0.5 m to 1.5 m of the clayey silt deposit range from 14 blows to 18 blows per 0.3 m of penetration, suggesting a very stiff consistency. The measured SPT "N" values within the lower portion of the deposit range from 2 blows to 7 blows per 0.3 m of penetration. In situ vane testing was conducted in this portion of the deposit and measured undrained shear strengths of approximately 52 kPa to 58 kPa. These test results suggest that the lower portion of the clayey silt deposit has a firm to stiff consistency.

4.2.4 Clayey Silt Till

A 4.3 m to 4.5 m thick deposit of clayey silt till was encountered below the clayey silt in both boreholes, extending to a depth of about 10.2 m to 11.6 m (Elevation 159.5 m to 161.8 m).

This till deposit consists of clayey silt with sand to some sand, and with gravel to some gravel. The till deposit contains an interlayer of sand and silt till within Borehole 11-201; this interlayer is described in Section 4.2.5. Cobbles and/or boulders are anticipated to be encountered within the till deposit based on evidence of hard drilling (such as bouncing of the split-spoon sampler in Borehole 11-202 at depth of about 10.9 m).

The results of grain size distribution tests completed on three selected samples of the clayey silt till are shown on Figure B6 in Appendix B. Atterberg limits testing was carried out on two selected samples of the till and measured plastic limits of 12 per cent and 13 per cent, liquid limits of 20 per cent and 21 per cent, and plasticity indices of 6 per cent and 8 per cent. These test results, which are plotted on a plasticity chart on Figure B7 in Appendix B, confirm that the till consists of low plasticity clayey silt till. The natural water contents measured on selected samples of the clayey silt till samples range from 6 per cent to 9 per cent.

The measured SPT "N" values within the clayey silt till range from 15 blows to 32 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

4.2.5 Sand and Silt Till Interlayer in Clayey Silt Till

An approximately 1.8 m thick interlayer of sand and silt till was encountered within the clayey silt till in Borehole 11-201, extending to a depth of about 9.0 m (Elevation 160.7 m).

The sand and silt till contains trace clay and some gravel. Cobbles and/or boulders are anticipated to be encountered within the till deposit based on evidence of hard drilling (such as bouncing of the split-spoon sampler at a depth of about 7.8 m and auger grinding between depths of 7.8 m and 8.2 m in Borehole 11-201 during drilling. The results of one grain size distribution test completed on a selected sample of the sand and silt till are shown on Figure B8 in Appendix B. Laboratory testing of two selected samples of the sand and silt till measured natural water contents of 7 per cent and 8 per cent.

The measured SPT "N" values within the sand and silt till are 55 blows and greater than 100 blows per 0.3 m of penetration, suggesting a very dense relative density.

4.2.6 Shale Bedrock

Bedrock was encountered below the clayey silt till deposit at depths of 10.2 m and 11.6 m (corresponding to Elevations 159.5 m and 161.8 m) in Boreholes 11-201 and 11-202, respectively, on the south and north sides of Highway 401.

Based on the cored bedrock samples, the bedrock generally consists of grey to black shale of the Georgian Bay Formation. The upper 0.7 m to 0.9 m of the bedrock is described as highly weathered, based on being able to penetrate this portion of the bedrock by augering and split-spoon sampling.

Below the weathered portion of the bedrock, the core samples are described as slightly to moderately weathered, laminated, grey, and weak to medium strong, with strong fossiliferous limestone interbeds. The Rock Quality Designation (RQD) measured on the core samples is typically between about 22 per cent and 48 per cent, indicating a rock mass of very poor to poor quality. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of the core samples are typically between 81 per cent and 100 per cent and 41 percent and 76 per cent, respectively.

Point load strength tests were performed on selected core samples. Diametral point load strength index values are shown on the Record of Drillhole Sheets and on Table B1 in Appendix B following the text of this report. The point load index (Is_{50}) results from diametral laboratory tests carried out on three samples of the shale bedrock range from approximately 0.3 MPa to 4.2 MPa, and the Is_{50} results from axial laboratory tests carried out on two samples of the shale bedrock range from approximately 4.8 MPa to 10.0 MPa. These point load test results correspond to estimated unconfined compressive strengths of approximately 7 MPa to 229 MPa, as shown on Table B2 in Appendix B.

An unconfined compressive strength (UCS) test carried out on a sample of the shale bedrock obtained from Borehole 11-201 measured about 32 MPa, as summarised on Table B2 in Appendix B. Photographs of one bedrock core sample before and after UCS testing are shown on Figure B9 in Appendix B.

Based on the laboratory UCS test and point load test results as summarized in Tables B1 and B2 in Appendix B, the estimated intact strength of the shale bedrock is weak to medium strong with very strong limestone interbeds, excluding the upper highly weathered zones.

4.3 Groundwater Conditions

Details of the water conditions observed in the open boreholes at the time of drilling are summarized on the borehole records following the text of this report. Both boreholes were dry and open upon completion of drilling.

A standpipe piezometer was installed in Borehole 11-201 within the lower portion of the clayey silt till deposit to monitor the groundwater level at the site. The water levels measured in the piezometer are summarized in the following table:

Borehole Number	Ground Surface Elevation (m)	Depth to Water Level	Groundwater Elevation	Date
11-201	169.7	4.4 m	165.3 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. Mehdi Mostakhdemi, M.Sc., M.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Ty Garde, P.Eng., a Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
CREDITVIEW ROAD UNDERPASS
HIGHWAY 401 WIDENING FROM EAST OF THE CREDIT RIVER TO TRAFALGAR ROAD, REGIONAL MUNICIPALITIES OF PEEL AND HALTON
W.O. 07-20021

October 2012
Report No. 10-1111-0040-2

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed replacement/realignment of the existing Highway 401-Creditview Road underpass. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. Further investigation and analysis will be required during detail design.

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

6.2 Foundation Options

Based on the planning study completed to date 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 65 m long Creditview Road underpass structure will require replacement. It is understood that the preferred alternative involves replacement with a three-span structure to be constructed immediately east of the existing Creditview Road underpass.

The existing structure consists of a four-span underpass, with the existing abutments supported on vertical and battered piles and the piers supported on spread footings. Based on the *General Plan and Elevation* drawing for the existing structure, dated February 1958, the existing foundation details are summarized as follows:

Foundation Element	Footing or Pile Cap Width	Founding Elevation
South abutment	1.8 m	Pile cap: 173.0 m (567.5 ft.) Pile tip: 166.9 m (547.5 ft.)
South, centre and north piers	2.7 m	168.2 m (552.0 ft.)
North abutment	1.8 m	Pile cap: 173.0 m (567.5 ft.) Pile tip: 166.9 m (547.5 ft.)

With the future widening of Highway 401, the pavement grade is proposed to be maintained at approximately Elevation 169.5 m to 170.5 m at the structure site. The finished grade for the realigned Creditview Road will be approximately Elevation 177 m at north and south abutments. Based on the current natural ground surface in the vicinity of the approach embankments, the north approach embankment will be up to approximately 6 m in height, and the south approach embankment will be up to approximately 9 m in height.

Based on the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments and piers for the new Creditview Road underpass. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 1 following the text of this report.

- **Strip or spread footings founded on the very stiff to firm clayey silt deposit:** Strip or spread footings may be feasible for support of the new abutments and piers at this site, and would permit semi-integral abutment design; however, the preliminary geotechnical resistances associated with the firm to stiff portion of the deposit are likely not sufficiently high to permit design of the replacement structure on foundations supported at relatively shallow depth, and it would be necessary to excavate approximately 3 m to 5 m below the Highway 401 grade. Temporary protection systems would be required along the east side of the existing Creditview Road to facilitate excavation through the existing embankment side slopes, as well as parallel to the Highway 401 lanes for the pier excavations.

- **Footings "perched" on a compacted granular pad in the approach embankment:** Up to about 80 mm of settlement is predicted under the new 6 m to 8.5 m high approach embankments that will be constructed east of the existing road alignment; while about half of this settlement is expected to be completed during and immediately following construction, it is anticipated that there will be some longer-term settlement associated with consolidation of the firm portion of the upper clayey silt deposit. Depending on the foundation option for the piers, there is greater potential for differential settlement between the foundation elements with this option. Therefore, perched abutment footings are not recommended for support of the replacement structure at this site.

- **Driven steel H-piles:** Driven steel H-piles are suitable and feasible for support of new abutments (and would permit integral abutment design), wing walls/retaining walls and piers at this site. There is a relatively minor risk associated with penetrating through or the piles "hanging up" on cobbles or boulders within the glacial soils (although further investigation is required in this regard at the detail design stage).

- **Driven steel pipe (tube) piles:** Steel tube (pipe) piles could also be considered as a deep foundation option for support of new abutments, associated wing walls/retaining walls, and the piers 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 sand and silt till deposit through which 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, wingwalls/retaining walls and piers for the replacement structure on steel pile foundations. The following sections provide recommendations for both shallow and deep foundation options to support the proposed replacement structure.

6.3 Shallow Foundations

6.3.1 Founding Elevations

For support of the new abutments, associated wing walls/retaining walls, and new piers, strip or spread footings should be founded below any fill and ideally below any firm to stiff near-surface soils, on very stiff clayey silt or clayey silt till. The following maximum (highest) founding elevations are recommended for preliminary design of shallow foundations.

Foundation Element	Borehole No.	Maximum (Highest) Founding Elevation	Approximate Excavation Depth
South abutment and south pier	11-201	167.0 m	3 m to 4 m
North abutment and north pier	11-202	166.0 m	4 m to 5 m

The founding elevations given above will require excavation to a depth of 3 m to 5 m below the existing Highway 401 grade. Based on the borehole results on the south side of Highway 401, footings would be founded on very stiff clayey silt, above a zone of firm to stiff clayey silt; on the north side of Highway 401, excavation to a depth of 4 m to 5 m would allow the footings to be founded on the very stiff to hard clayey silt till below the firm to stiff clayey silt. Alternatively, subexcavation can be carried out to the elevations identified in the table above, then backfilled with compacted Ontario Provincial Standard Specification (OPSS) 1010 Granular A or Granular B Type II fill prior to construction of the footings at a higher elevation. In this case, the founding elevation for the footings should be a minimum of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration, in accordance with Provincial Standards. The compacted granular fill should extend at least 1 m beyond the front and back edge of the new footings, then outward and downward at 1H:1V.

The footing subgrade should be inspected by a Quality Verification Engineer following excavation, in accordance with provincial standards to confirm that all existing fill, softened clayey silt soils or other unsuitable material have been removed. The founding soils will be susceptible to disturbance. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a concrete working slab be placed on the prepared subgrade within four hours of its inspection and approval, as discussed further in Section 6.6.3.

6.3.2 Geotechnical Resistance/Reaction

Strip or spread footings placed on the properly prepared subgrade, at or below the preliminary design elevations given in the preceding section, should be designed based on the factored geotechnical resistances at Ultimate Limit States (ULS) and geotechnical reaction at Serviceability Limit States (SLS) given below.

Founding Stratum	Footing Width	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS*
South abutment, south pier and centre pier**	3 m	250 kPa	150 kPa
North abutment and north pier	3 m	400 kPa	300 kPa

* For 25 mm of settlement

** For higher geotechnical resistances, the south abutment and south pier footings would have to be founded below Elevation 164 m

The preliminary geotechnical resistances should be reviewed if the selected footing width or founding elevation differs from those given above. In addition, these preliminary geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its *Commentary.*

The preliminary geotechnical resistance values provided above will have to be re-evaluated and modified as necessary during detail design, based on future additional subsurface investigation at the proposed abutment and pier locations.

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

6.4.1 Founding Elevations

The new abutments, associated wing walls/retaining walls and piers may be supported on steel H-piles or steel pipe (tube) piles driven to found on or in the shale bedrock. The surface elevation for the shale bedrock and the thickness of the highly weathered zone varied in the two boreholes, and further investigation will be required at the detail design stage to confirm these preliminary founding elevations. The following pile tip elevations may be used for preliminary design purposes, assuming penetration through the highly weathered shale bedrock, and termination on or just into the slightly weathered portion of the bedrock:

Foundation Element	Estimated Design Pile Tip Elevation
South abutment and south pier	158.7 m
North abutment and north pier	160.8 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 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 Provincial Standards. In very dense/hard and/or bouldery soils, as may be encountered at this site, driving shoes are preferred over flange plates.

6.4.2 Axial Geotechnical Resistance/Reaction

For preliminary design for HP 310x110 piles driven to the estimated tip elevations provided in Section 6.4.1, the factored axial geotechnical resistance at ULS may be taken as 1,600 kN, and the axial geotechnical reaction at SLS (for approximately 10 mm of settlement) may be taken as 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.). These preliminary geotechnical resistances will have to be re-evaluated and modified as necessary during detail design in consideration of the additional subsurface investigation at the new foundation elements.

The long-term settlement associated with the consolidation of the firm to stiff portion of the clayey silt deposit will induce a downward movement of the soils adjacent to the piles and negative skin friction will develop along portions of the pile shafts embedded within or above the firm to stiff clayey silt layer. For preliminary design purposes, factored downdrag loads of 120 kN for HP 310x110 piles (assuming a negative skin friction factor of 0.25) should be considered in the preliminary design of the piles. The structural capacity of the pile must be sufficient to withstand the combined permanent load plus the downdrag load (if the downdrag loads are greater than the live loads). The magnitude and duration for the settlement and the downdrag loads should be reassessed during detail design, following completion of additional investigation and testing.

Alternatively, the embankment could be constructed to design grade and preloaded for a period of approximately three months (with the duration to be confirmed during detail design). This latter method is preferred, as it would address concerns with differential settlement in the immediate vicinity of the abutment. If there is no preload, the embankment may have to be constructed using lightweight fill to eliminate the differential settlement.

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 approach embankments. The depth and extent of stripping should be assessed during detail design when additional subsurface information will be available for the 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 realigned Creditview Road should be placed and compacted in accordance with Provincial Standards. Benching of the west and east sides of the existing Creditview Road embankment should be carried out to “key in” the new fill materials for the realignment/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 fill 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 widened/new approach embankments 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 realignment/eastward widening on this project, considering the design requirements and the available field and laboratory testing data.

Preliminary stability analyses were completed for a 6 m high north approach embankment and a 9 m high south approach embankment, based on the subsurface conditions as encountered in Boreholes 11-201 and 11-202, respectively. No mid-height bench was included in the preliminary stability analysis for the 8.5 m high slope. The following parameters have been used in the analyses, based on field and laboratory test data as well as accepted correlations:

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle	Undrained Shear Strength (kPa)
Embankment fill	21	32-35°	-
Very stiff upper portion of clayey silt	20	32°	
Firm to stiff lower portion of clayey silt	20	28°	50
Very stiff to hard clayey silt till	21	32-34°	-
Dense to very dense sand and silt till	20	32°	-

The analysis results indicate that a 6 m to 9 m high embankment with side slopes no steeper than 2H:1V will have a factor of safety of at least 1.3 against global instability, assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials. Example static global stability results are provided on Figures 1 and 2. This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed based on the subsoil conditions encountered within the proposed approach embankment footprints during detail design.

6.5.3 Approach Embankment Settlement

The new Creditview Road underpass is proposed to be constructed immediately east of the existing structure. The new approach embankments will essentially be constructed as an eastward widening of the existing Creditview Road embankments.

Preliminary settlement analyses for the anticipated soil conditions below the new/widened approach embankments were carried out using 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, undrained shear strengths, Atterberg limits testing 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)	Preconsolidation Pressure (kPa)	C_c	C_r
Embankment fill	21	-	-	-	-
Very stiff upper portion of clayey silt	20	35	-	-	-
Firm to stiff lower portion of clayey silt	21	-	220	0.14	0.02
Very stiff to hard clayey silt till	21	50	-	-	-
Dense to very dense sand and silt till	20	75	-	-	-

Based on this preliminary assessment, the settlement of the foundation soils under new 6 m to 9 m high approach embankments is estimated to be up to about 80 mm. Approximately 45 mm of this settlement is expected to occur relatively quickly during and immediately following construction of the approach embankments. However, approximately 35 mm of this settlement is associated with longer-term consolidation of the firm to stiff portion of the clayey silt deposit under the new/widened approach embankment loading; it is anticipated that the majority of this settlement would be completed within approximately three months. This estimated magnitude and duration of settlement should be reassessed following additional investigation (including consolidation testing) during detail design.

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

6.6 Construction Considerations

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

6.6.1 Excavation and Temporary Roadway Protection

The foundation excavations for spread footings would extend through any existing fill and potentially through firm to stiff clayey silt, into very stiff clayey silt or clayey silt till. If 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 firm to stiff clayey silt should be classified as Type 3 soil, according to the OHSA, while the very stiff clayey silt/clayey silt till would be classified as a Type 2 material. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V.

At this preliminary stage, it is anticipated that temporary roadway protection will be required along the east side of Creditview Road to maintain traffic on the local road during excavation into the existing side slopes; it is also anticipated that temporary protection systems may be required at the pier locations to facilitate construction of pile caps or the deeper excavations that would be required for spread footings.

6.6.2 Groundwater Control

Groundwater seepage is anticipated from cohesionless soil interlayers within the clayey silt or till deposits (where these are present), and from groundwater “perched” on top of the clayey silt deposit within existing granular fill. 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 excavation. Based on these small seepage volumes, a Permit to Take Water (PTTW) should not be required for the groundwater control system at this site.

6.6.3 Subgrade Protection

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

6.6.4 Obstructions

The soils at this site are glacially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of deep foundations 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 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 underpass structure is not expected to be required during construction. However, there is an industrial building in the vicinity of the structure site, and the requirements for monitoring of vibrations at this building during construction should be evaluated during the detail design stage. 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 building.

6.7 Recommendations for Further Work During Detail Design

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

- Abutments and piers:
  - Assessment of the properties and thickness of the clayey silt deposit to confirm the bearing resistance and founding elevation for shallow foundations, or to confirm downdrag loads for deep foundations at the abutments.
  - Assessment of the bedrock surface elevation and thickness of highly weathered shale to confirm the tip elevation for driven piles.
  - Assessment of the presence of any cohesionless soil lenses or interlayers within the cohesive deposits at the site, which could impact groundwater control requirements for foundation excavations.
  - 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.
  - Assessment of vibration thresholds for the nearby commercial/industrial building, and if warranted development of an NSSP for a vibration monitoring plan.
- Approach embankments:
  - Assessment of the depth and extent of stripping of topsoil/organics and fill materials within the footprint of the new approach embankments.
  - Further assessment of the consolidation characteristics of clayey silt layer and estimated magnitude of settlement under the new approach embankments.
  - Further assessment of preloading requirements.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by 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 Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

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Peck, R.B., Hanson, W.E., and Thornburn, T.H., 1974. *Foundation Engineering*, Second Edition, John Wiley and Sons, New York.

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 OPTIONS
CREDITVIEW ROAD UNDERPASS

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Spread/strip footings on very stiff clayey silt or very stiff clayey silt till	• May be feasible but not recommended for support of abutments, wingwalls/ retaining walls and piers due to depth of excavation required	• Limited groundwater control as excavation will be within relatively impermeable clayey silt till deposit • Allows for semi-integral abutments	• Requires relatively deep excavations to approximately 3 m to 5 m below Highway 401 grade, with associated temporary excavation support • 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 fill	• Not considered feasible at this site due to predicted settlement under new/widened embankment loading	• Abutment pile caps could be maintained higher than footings founded on clayey silt/till deposit, reducing depth of excavation and temporary excavation support requirements adjacent to existing Creditview Road embankment	• Up to about 80 mm of settlement predicted under new/widened embankment loading • Potential for differential settlement between abutments and pier due to settlement of soils under approach embankment loading • Precludes use of integral abutments; potentially greater maintenance required at abutments	• Conventional excavation and construction techniques	• Not assessed as this option is not considered appropriate at this site

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel H-piles driven to found on or in the shale bedrock	• Feasible for support of abutments, wing walls/retaining walls and piers	• Pier pile caps could be maintained higher than footings founded on clayey silt/ till deposit, reducing depth of excavation and temporary excavation support requirements adjacent to existing Creditview Road embankment and Highway 401 • Limited groundwater control required • Allows for integral abutment construction • Higher axial resistance than for shallow foundations	• Potential for encountering obstructions (cobbles and/or boulders) during pile driving; this 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 unit cost is approximately \$250/linear metre for pile installation and \$600/m^3 for pile cap construction
Steel pipe (tube) piles, driven to found on or in shale bedrock	• Feasible for support of new abutments, wing walls/retaining walls, and piers	• Abutment pile caps could be maintained higher than footings founded on clayey silt/till, reducing depth of excavation and temporary protection system requirements adjacent to Creditview Road and Highway 401 • Limited groundwater control required • Allows for semi-integral abutment configuration • Would minimize differential settlement between foundation elements	• 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

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Caissons founded in shale bedrock	• Feasible but not recommended for support of abutments, wing walls/retaining walls and centre pier	• Abutment pile caps could be maintained higher than footings founded on till deposit, reducing depth of excavation and temporary excavation support requirements adjacent to existing Creditview Road embankment • Higher capacity than for steel H-piles or pipe piles, so reduced number of deep foundation elements compared to steel piles	• Potential for loss of ground in water-bearing sand and silt till deposit • Temporary or permanent liners would be required; likely not possible to inspect caisson base • Precludes use of integral abutments	• Conventional construction methods with temporary liners required	• Higher cost compared with shallow foundations or steel H-piles

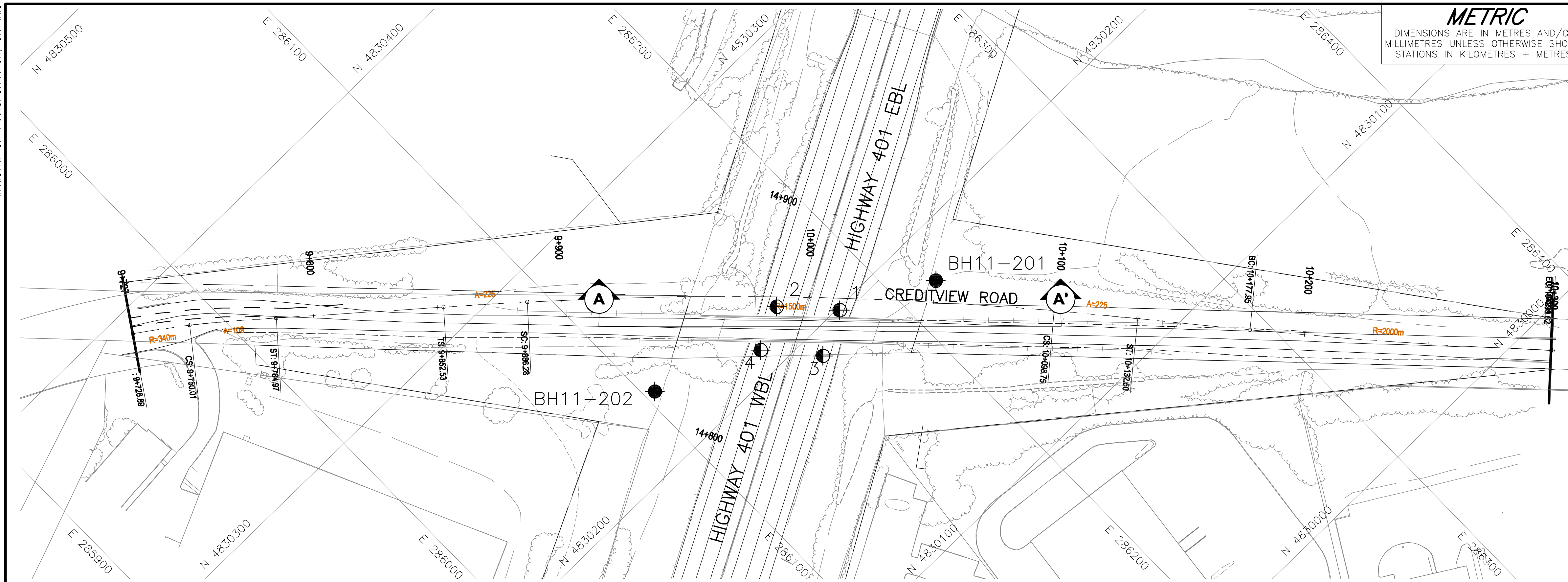
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WO No. 07–20021

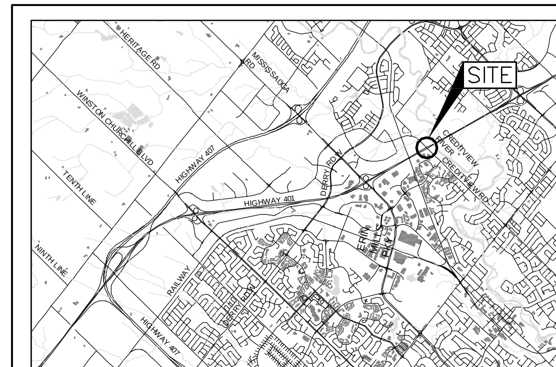
SHEET

CREDITVIEW ROAD UNDERPASS
HIGHWAY 401 IMPROVEMENTS
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN
SCALE
20 0 20 40 m



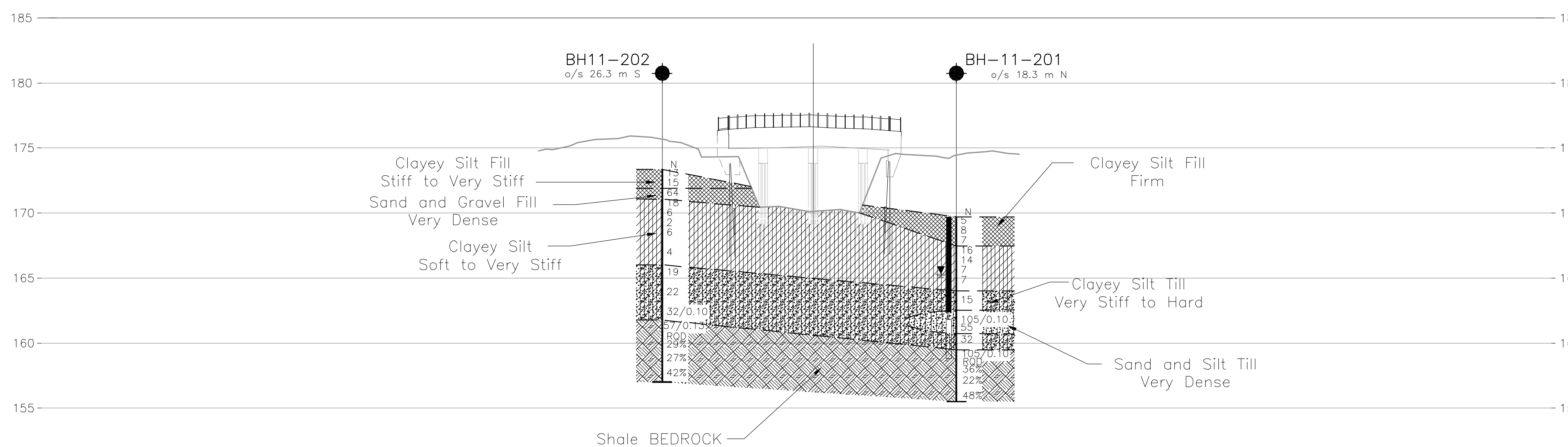
KEY PLAN
SCALE
1.5 0 1.5 3 km

LEGEND

●	Borehole – Current Investigation
⊕	Borehole – Previous Investigation
	Seal
	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▼	WL in piezometer, measured on November 2, 2011
▽	WL upon completion of drilling

BOREHOLE CO–ORDINATES

No.	ELEVATION	NORTHING	EASTING
BH11–201	169.7	4830180.6	286216.4
BH11–202	173.4	4830227.6	286104.2



PROFILE A–A'
HORIZONTAL SCALE
20 0 20 40 m
4 0 4 8 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 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 nos. ACAD–X–base1_to_Trafalgar.dwg, ACAD–Aerials_MTO ROW_ Property Boundaries..dwg and Creditview_Realign_Alt2_East.dwg , received August 17, 2011, August 29, 2011 and August 16, 2011

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

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

NO.	DATE	BY	REVISION

Geocres No. 30M12–345			
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

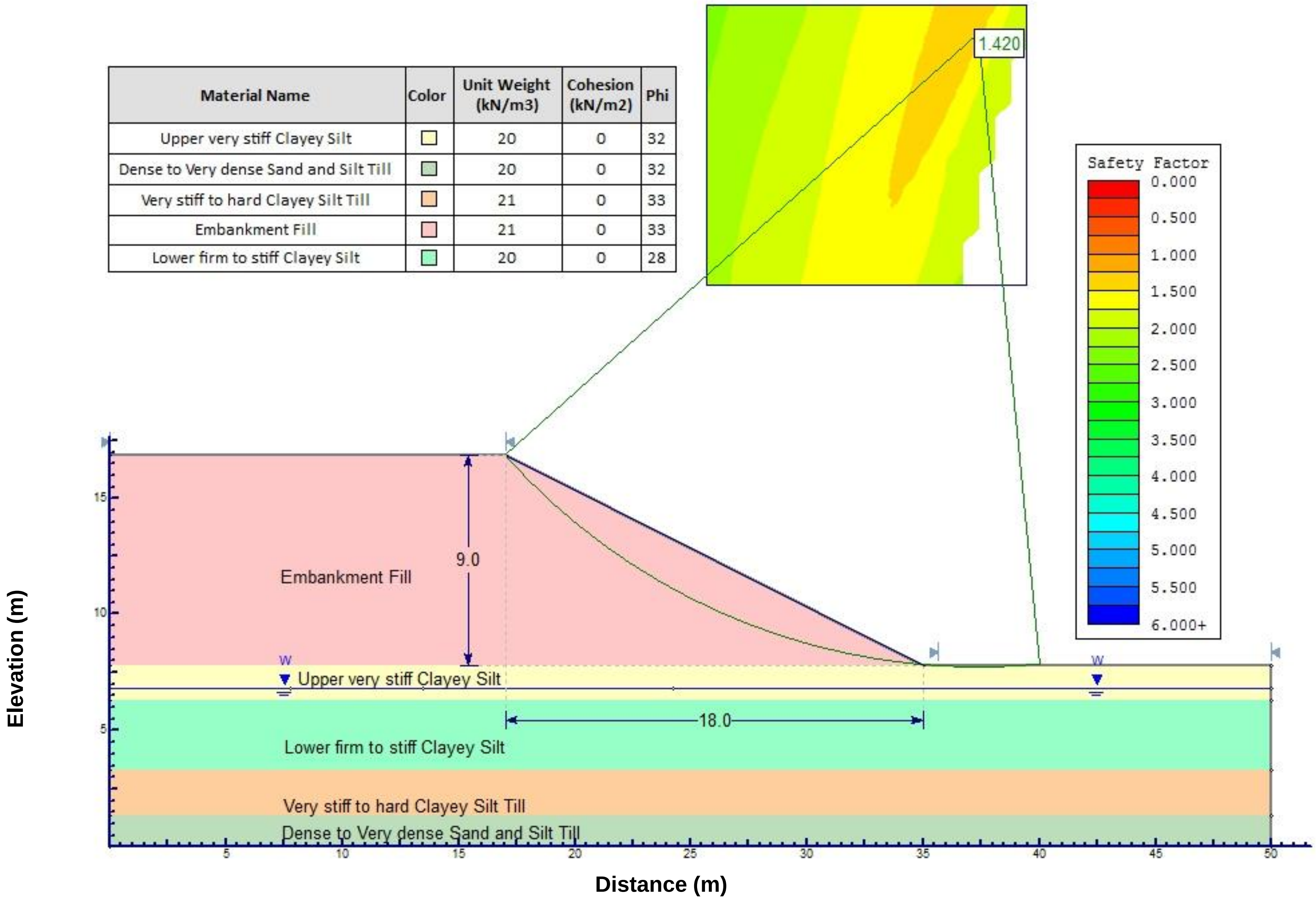
MINISTRY OF TRANSPORTATION, ONTARIO

PLOT DATE: October 23, 2012
FILENAME: T:\Projects\2010\10-1111-0040 (UCS, Halton Peel)\-BB- (Creditview)\1011110040BB001.dwg

Static Global Stability – Creditview Road South Approach Embankment

Figure 1

Material Name	Color	Unit Weight (kN/m3)	Cohesion (kN/m2)	Phi
Upper very stiff Clayey Silt		20	0	32
Dense to Very dense Sand and Silt Till		20	0	32
Very stiff to hard Clayey Silt Till		21	0	33
Embankment Fill		21	0	33
Lower firm to stiff Clayey Silt		20	0	28



Date: December 2011

Project No: 10-1111-0040

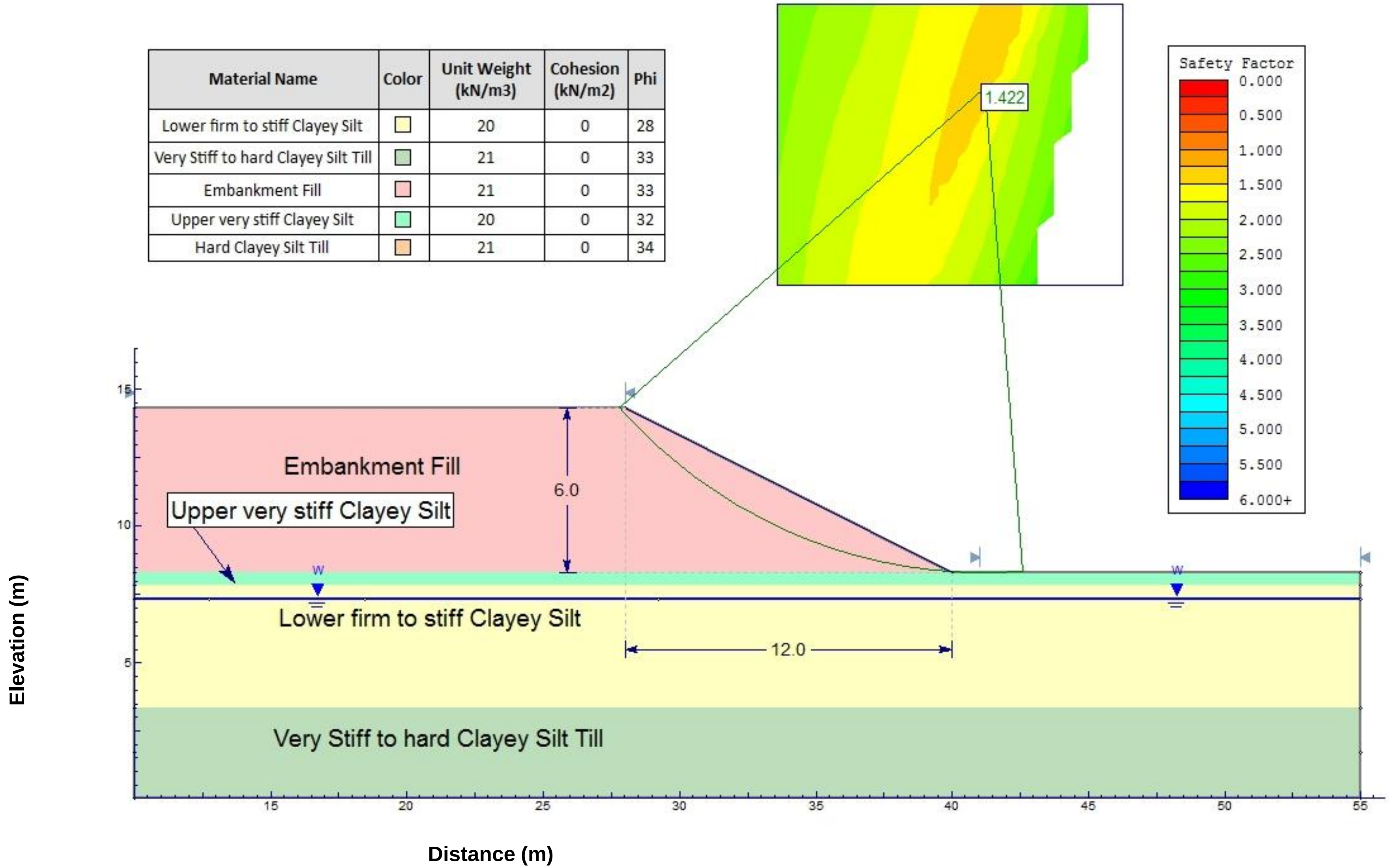
Analysis By: MM **Reviewed By:** LCC

Golder Associates

Static Global Stability – Creditview Road North Approach Embankment

Figure 2

Material Name	Color	Unit Weight (kN/m3)	Cohesion (kN/m2)	Phi
Lower firm to stiff Clayey Silt		20	0	28
Very Stiff to hard Clayey Silt Till		21	0	33
Embankment Fill		21	0	33
Upper very stiff Clayey Silt		20	0	32
Hard Clayey Silt Till		21	0	34



Date: December 2011
Project No: 10-1111-0040

Analysis By: MM **Reviewed By:** LCC

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	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.

V. MINOR SOIL CONSTITUENTS

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

LIST OF SYMBOLS

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

I. GENERAL

π	3.1416
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 $= \sigma'_p / \sigma'_{vo}$

(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 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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)
The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)
The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)
The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index
A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90^{o} angle is horizontal.

Description and Notes
An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

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RECORD OF BOREHOLE No 11-201

SHEET 1 OF 3 **METRIC**

PROJECT 10-1111-0040
G.W.P. 07-20021
DIST Central HWY 401
DATUM NAD83, Geodetic
LOCATION N 4830180.6 ;E 286216.4
BOREHOLE TYPE Track-Mounted CME55, 108 mm I.D. Hollow Stem Augers
DATE September 12, 2011
ORIGINATED BY AM
COMPILED BY MM
CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
169.7	GROUND SURFACE						
0.0 / 0.1	TOPSOIL						
	Clayey silt with sand, some gravel, containing rootlets (FILL) Firm Brown Moist	1	SS	5	169		
		2	SS	8			14 30 42 14
		3	SS	7	168		
167.5 / 2.2	CLAYEY SILT with sand to some sand, trace to some gravel Very stiff to firm Brown Moist Becoming grey below a depth of 3.1 m	4	SS	16	167		
		5	SS	14	166		11 26 43 20
		6	SS	7			
		7	SS	7	165		
164.0 / 5.7	CLAYEY SILT with sand to some sand, trace to some gravel (TILL) Very stiff Grey Moist	8	SS	15	164, 163		
162.5 / 7.2	SAND and SILT, trace to some clay, some gravel, containing cobbles or boulders (TILL) Dense to very dense Grey Moist Split spoon bouncing at a depth of 7.9 m Auger grinding between 7.8 m and 8.2 m	9	SS	105/0.10	162		19 37 37 7
		10	SS	55	161		
160.7 / 9.0	CLAYEY SILT, some sand and gravel (TILL) Hard Grey Moist	11	SS	32	160		13 14 55 18
159.5 / 10.2	Shale (BEDROCK) Highly weathered Grey to black Moist Split spoon bouncing at a depth of 10.9 m	12	SS	105/0.10	159		
158.8 / 10.9	Shale (BEDROCK) Bedrock cored between 10.9 m and 14.2 m For bedrock coring details, refer to Record of Drillhole 11-201	1	RC	REC 95%	158		RQD = 36%
		2	RC	REC 94%	157		RQD = 22%
		3	RC	REC 81%	156		RQD = 48%
155.5 / 14.2	END OF BOREHOLE						

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 11-201

SHEET 2 OF 3 **METRIC**

PROJECT 10-1111-0040			
G.W.P. 07-20021	LOCATION N 4830180.6 ;E 286216.4		ORIGINATED BY AM
DIST Central HWY 401	BOREHOLE TYPE Track-Mounted CME55, 108 mm I.D. Hollow Stem Augers		COMPILED BY MM
DATUM NAD83, Geodetic	DATE September 12, 2011		CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O 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
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	NOTES: 1. Monitoring well was dry and open upon completion of drilling. 2. Water level in monitoring well measured as follows: Date Depth (m) Elev. (m) 11/02/11 4.4 165.3										

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

PROJECT: 10-1111-0040

RECORD OF DRILLHOLE: 11-201

LOCATION: N 4830180.6 ;E 286216.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 12, 2011

DRILL RIG: Track-Mounted CME 55

DRILLING CONTRACTOR: Geo-Environmental Drilling Inc.

DATUM: NAD83, Geodetic

JN - Joint, FLT - Fault, SH - Shear, VN - Vein, CJ - Conjugate
BD - Bedding, FO - Foliation, CO - Contact, OR - Orthogonal, CL - Cleavage
PL - Planar, CU - Curved, UN - Undulating, ST - Stepped, IR - Irregular
PO - Polished, K - Slickensided, SM - Smooth, RO - Rough, VR - Very Rough
MB - Mechanical Break, BR - Broken Rock
NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH / COLOUR % RETURN	RECOVERY TOTAL CORE % (80 60 40 20)	RECOVERY SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER 0.3 m (5 10 15 20)	B Angle (0 90 180 270)	DIP w.r.t. CORE AXIS (0 30 60 90)	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3})	Diametral Point Load Index (MPa) (2 4 6)	RMC -Q' AVG.	NOTES
		Continued from Record of Borehole BH11-201		158.79																
11–14		SHALE (BEDROCK) with fossiliferous limestone beds Slightly to moderately weathered Laminated Grey Weak to medium strong		10.91	1											3				
					2											3				(Axial)
				155.50 14.20	3											3				
		END OF DRILLHOLE																		

Depth scale: 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

GTA-RCK 018 1011110040.GPJ GAL-MISS.GDT 10/23/12 DD

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: AM

CHECKED: LCC

RECORD OF BOREHOLE No 11-202

SHEET 1 OF 3 **METRIC**

PROJECT: 10-1111-0040			
G.W.P.: 07-20021	LOCATION: N 4830227.6 ;E 286104.2	ORIGINATED BY: AM	
DIST: Central	HWY: 401	BOREHOLE TYPE: Track-Mounted CME55, 108 mm I.D. Hollow Stem Augers	COMPILED BY: MM
DATUM: NAD83, Geodetic	DATE: September 9, 2011	CHECKED BY: LCC	

ELEV / DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT / LIMITS (%)	SHEAR STRENGTH (kPa)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
173.4	GROUND SURFACE						
0.0 / 0.1	TOPSOIL						
	Clayey silt with sand, trace gravel (FILL) Stiff to very stiff Brown Moist	1	SS	13	w ≈ 9		
		2	SS	15			
172.0 / 1.5	Sand and gravel, some silt, trace clay (FILL) Very dense Brown Moist	3	SS	64	w ≈ 3		38 45 14 3
171.1 / 2.3	CLAYEY SILT with sand to some sand, trace to some gravel Soft to very stiff Brown Moist	4	SS	18			
		5	SS	6	w ≈ 28		
	Wet at a depth of 3.8 m	6	SS	2			
	Grey at a depth of 4.6 m	7	SS	6	wP ≈ 14, w ≈ 17, wL ≈ 24		8 21 41 30
						Field vane ≈ 52, 58	
		8	SS	4			
						Field vane ≈ 52	
166.1 / 7.3	CLAYEY SILT with sand, some gravel (TILL) Very stiff Grey Wet	9	SS	19	w ≈ 9, wP ≈ 12, wL ≈ 21		13 32 39 16
		10	SS	22			
163.5 / 9.9	CLAYEY SILT with sand and gravel, containing cobbles or boulders (TILL) Hard Grey Wet Split spoon bouncing at a depth of 10.9 m	11	SS	32/0.10	w ≈ 6, wP ≈ 13, wL ≈ 19		40 26 27 7
161.8 / 11.6	Shale (BEDROCK) Highly weathered Black Wet Split spoon bouncing at a depth of 12.5 m	12	SS	57/0.13			
160.9 / 12.5	Shale (BEDROCK) Bedrock cored between 12.5 m and 16.4 m. For bedrock coring details, refer to Record of Drillhole 11-202	1	RC	REC 83%			RQD = 29%
		2	RC	REC 100%			RQD = 27%
		3	RC				RQD = 42%

Continued Next Page

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No 11-202

METRIC

PROJECT 10-1111-0040

G.W.P. 07-20021

LOCATION N 4830227.6 ;E 286104.2

ORIGINATED BY AM

DIST Central HWY 401

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

COMPILED BY MM

DATUM NAD83, Geodetic

DATE September 9, 2011

CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O 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
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Shale (BEDROCK) Bedrock cored between 12.5 m and 16.4 m For bedrock coring details, refer to Record of Drillhole 11-202		3	RC	REC 97%		158				RQD = 42%
157.1 16.4	END OF BOREHOLE NOTE: 1. Borehole dry on completion of overburden drilling.										

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

PROJECT: 10-1111-0040

RECORD OF DRILLHOLE: 11-202

LOCATION: N 4830227.6 ;E 286104.2

DRILLING DATE: September 9, 2011

DATUM: NAD83, Geodetic

DRILL RIG: Track-Mounted CME 55

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Geo-Environmental Drilling Inc.

JN - Joint, FLT - Fault, SH - Shear, VN - Vein, CJ - Conjugate
BD - Bedding, FO - Foliation, CO - Contact, OR - Orthogonal, CL - Cleavage
PL - Planar, CU - Curved, UN - Undulating, ST - Stepped, IR - Irregular
PO - Polished, K - Slickensided, SM - Smooth, RO - Rough, VR - Very Rough
MB - Mechanical Break, BR - Broken Rock
NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	Jn	Diametral Point Load Index (MPa)	NOTES
	Continued from Record of Borehole BH11-202	160.90				
13	SHALE (BEDROCK) with fossiliferous limestone beds Slightly to moderately weathered Laminated Grey Weak to medium strong	12.46	1	3		
14			2	3	●	
15			3	3	●	9.97 MPa (Axial)
16		157.01			●	
	END OF DRILLHOLE	16.35				

Depth scale marks: 17, 18, 19, 20, 21, 22

DEPTH SCALE
1 : 50

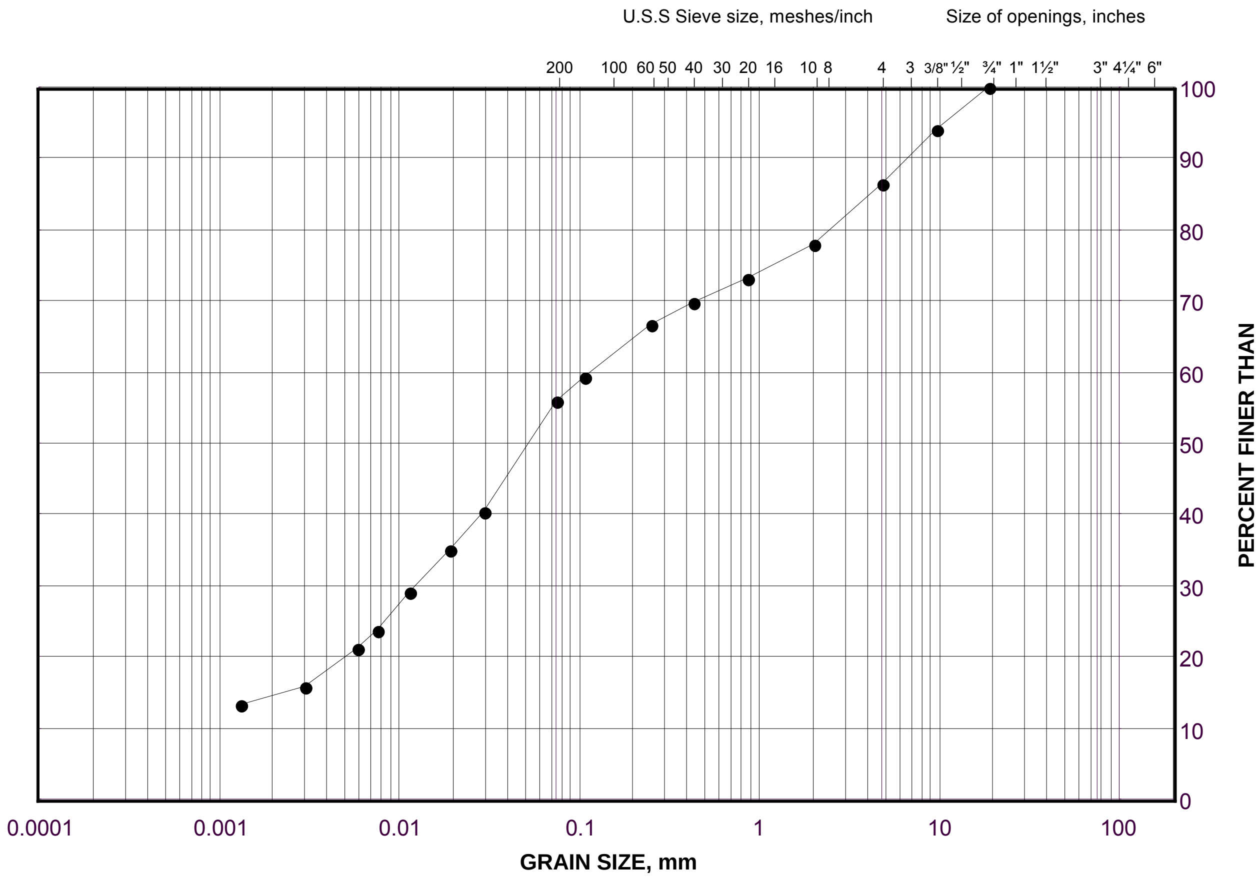
Golder Associates

LOGGED: AM
CHECKED: LCC

APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION Clayey Silt Fill	FIGURE B1



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

LEGEND

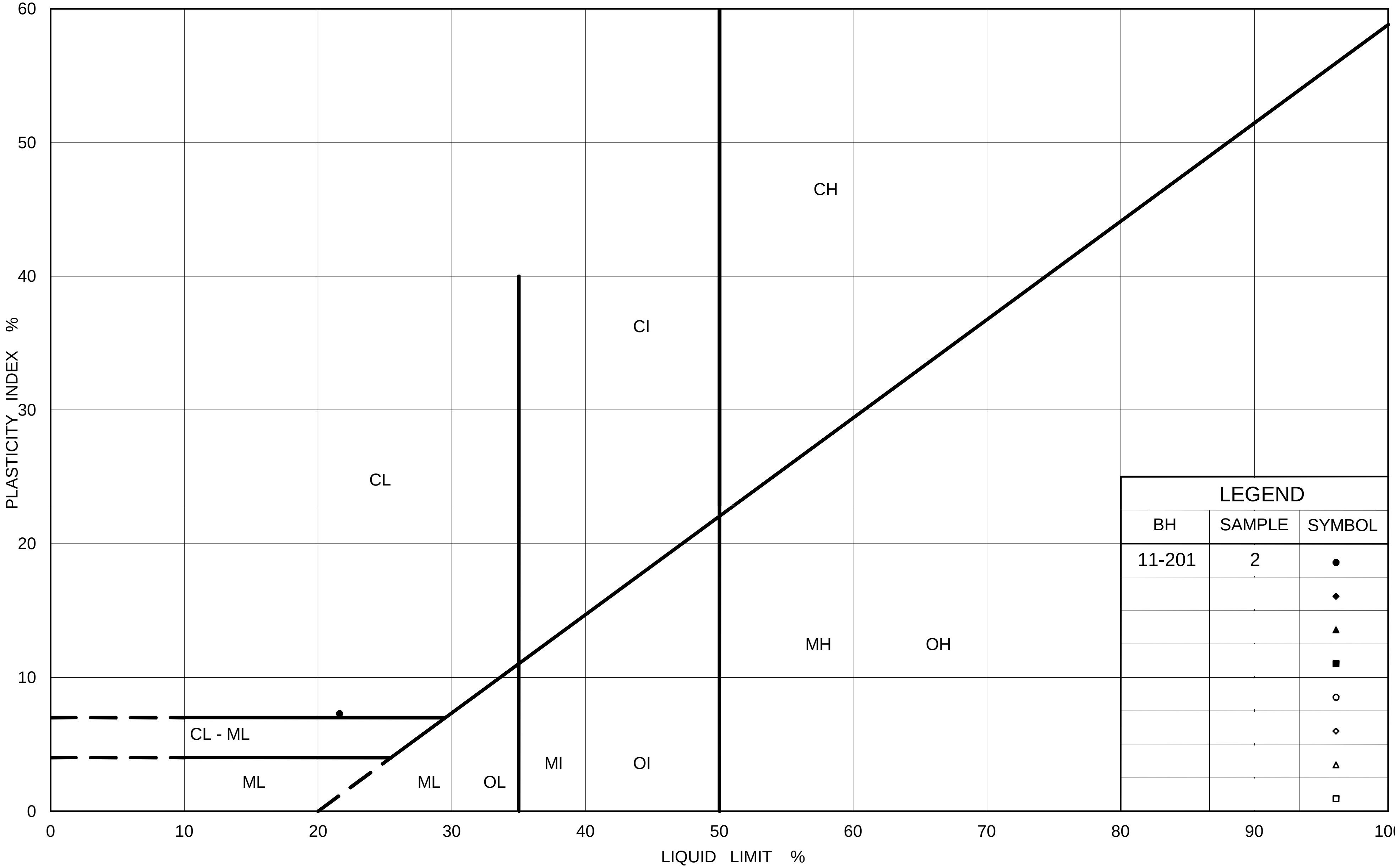
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	11-201	2	168.6

Project Number: 10-1111-0040-2

Checked By: MM

Golder Associates

Date: 21-Feb-12



LEGEND

BH	SAMPLE	SYMBOL
11-201	2	●
		◆
		▲
		■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt Fill

Figure No. B2

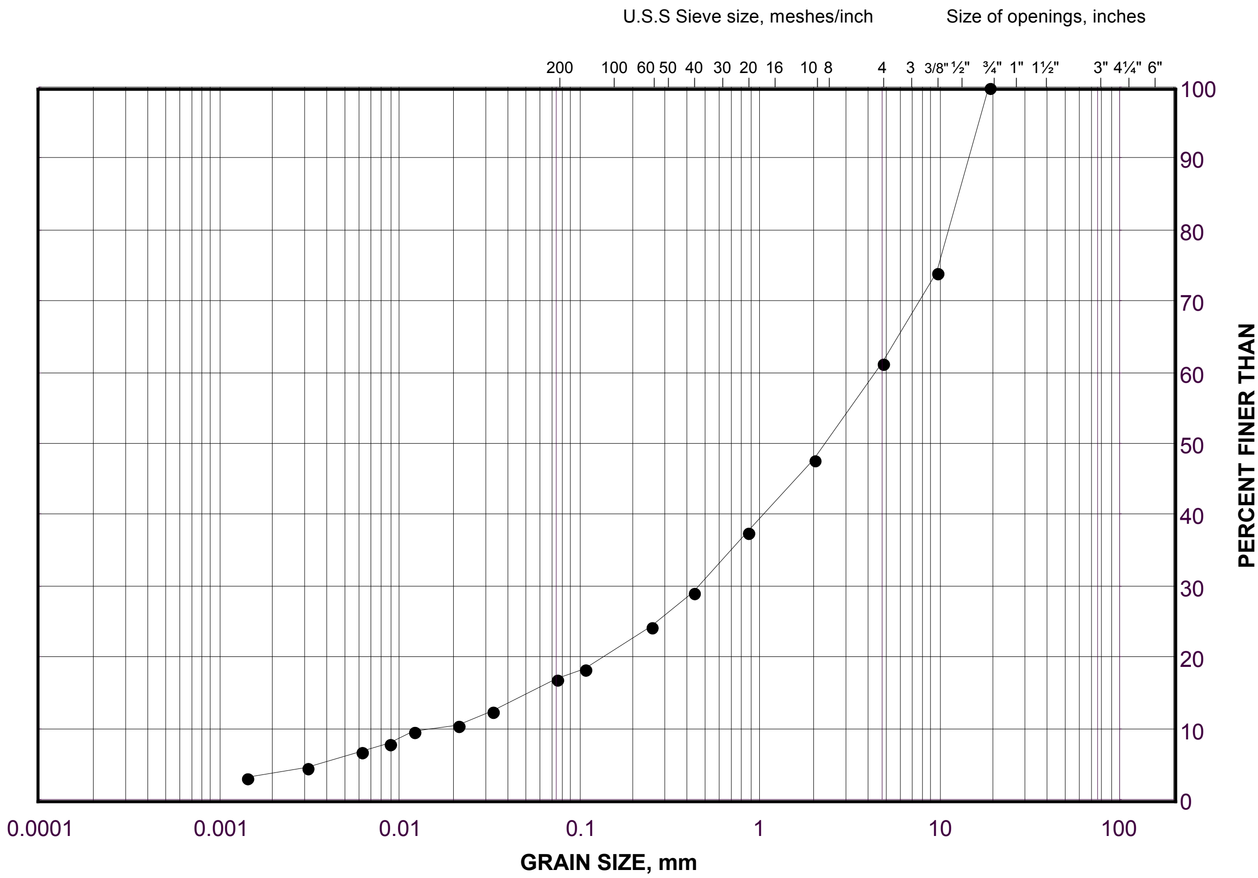
Project No. 10-1111-0040-2

Checked By: MM

GRAIN SIZE DISTRIBUTION

Gravelly Sand

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)
●	11-202	3	171.6

Project Number: 10-1111-0040-2

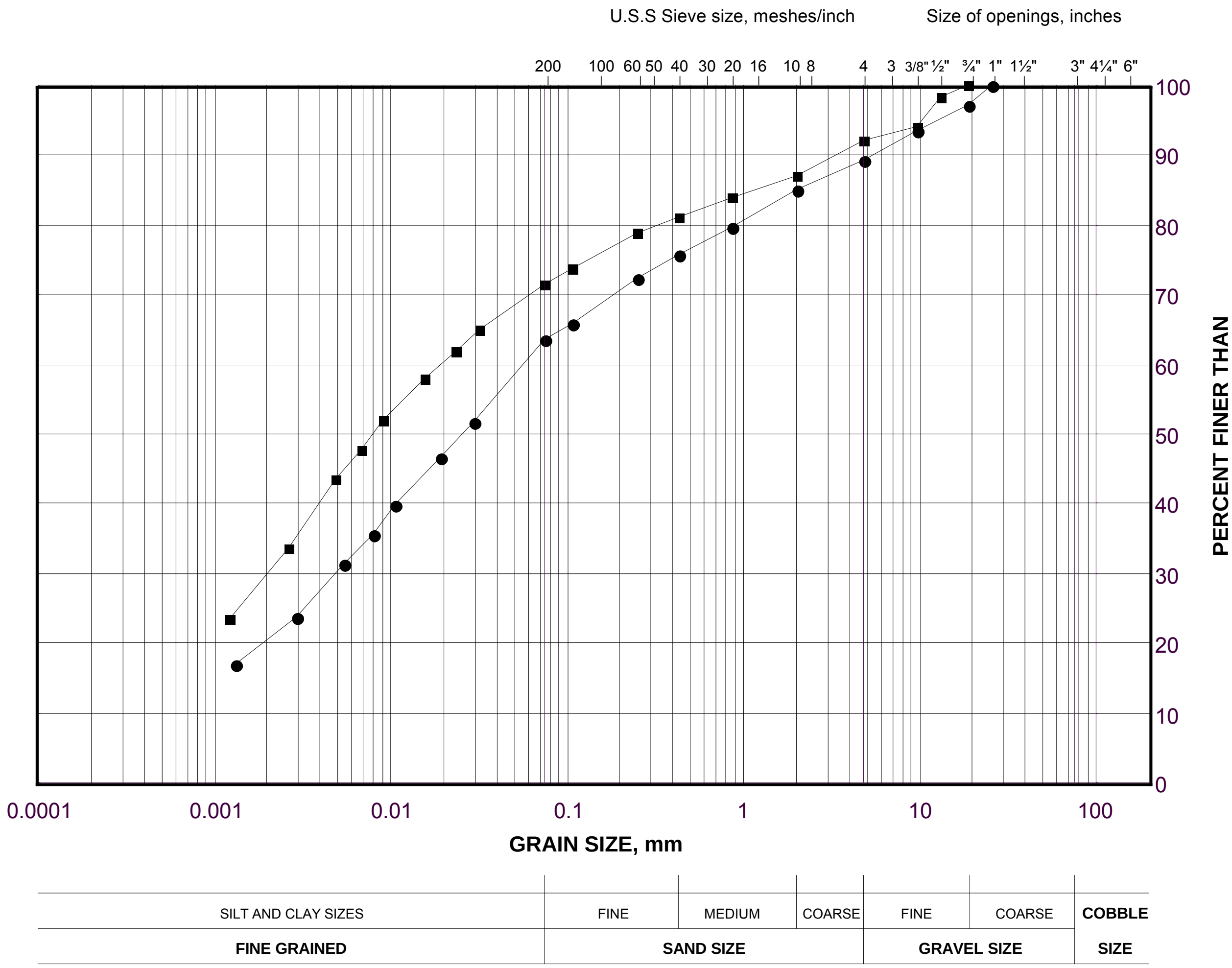
Checked By: MM

Golder Associates

Date: 21-Feb-12

GRAIN SIZE DISTRIBUTION
Clayey Silt

FIGURE B4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	11-201	5	166.4
■	11-202	7	168.5

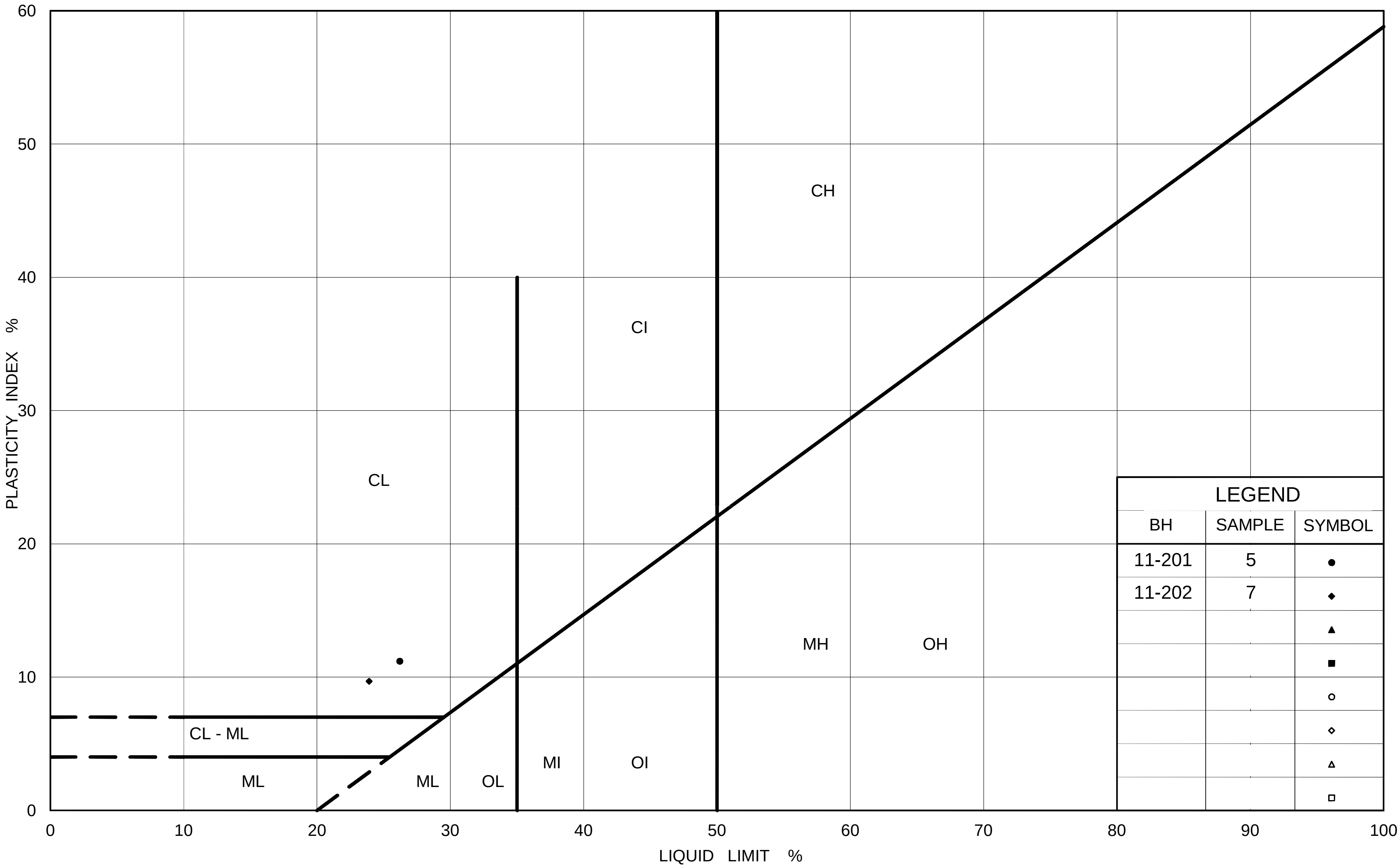
Project Number: 10-1111-0040-2

Checked By: MM

Golder Associates

Date: 21-Feb-12

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt

Figure No. B5

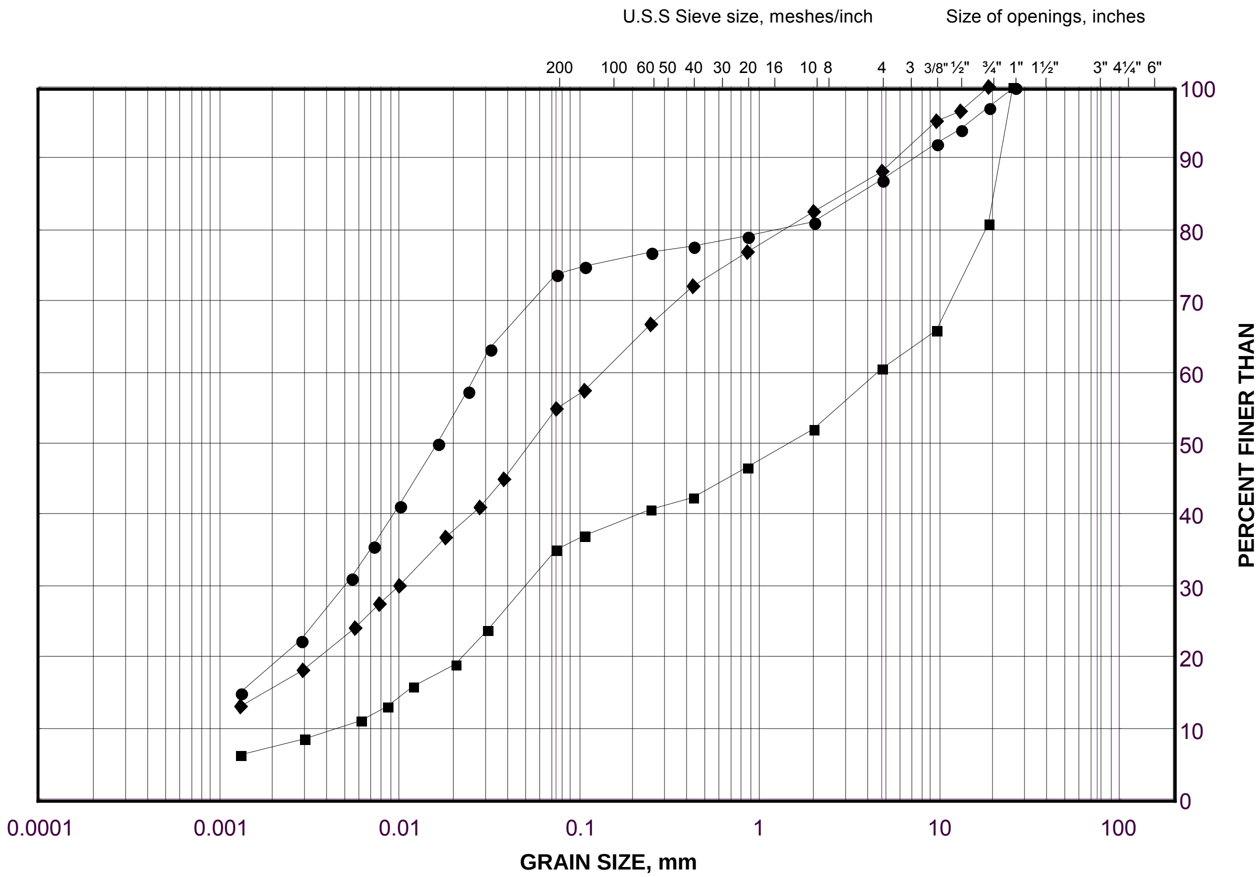
Project No. 10-1111-0040-2

Checked By: MM

GRAIN SIZE DISTRIBUTION

Clayey Silt Till

FIGURE B6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	11-201	11	160.2
■	11-202	11	162.4
◆	11-202	9	165.5

Project Number: 10-1111-0040-2

Checked By: MM

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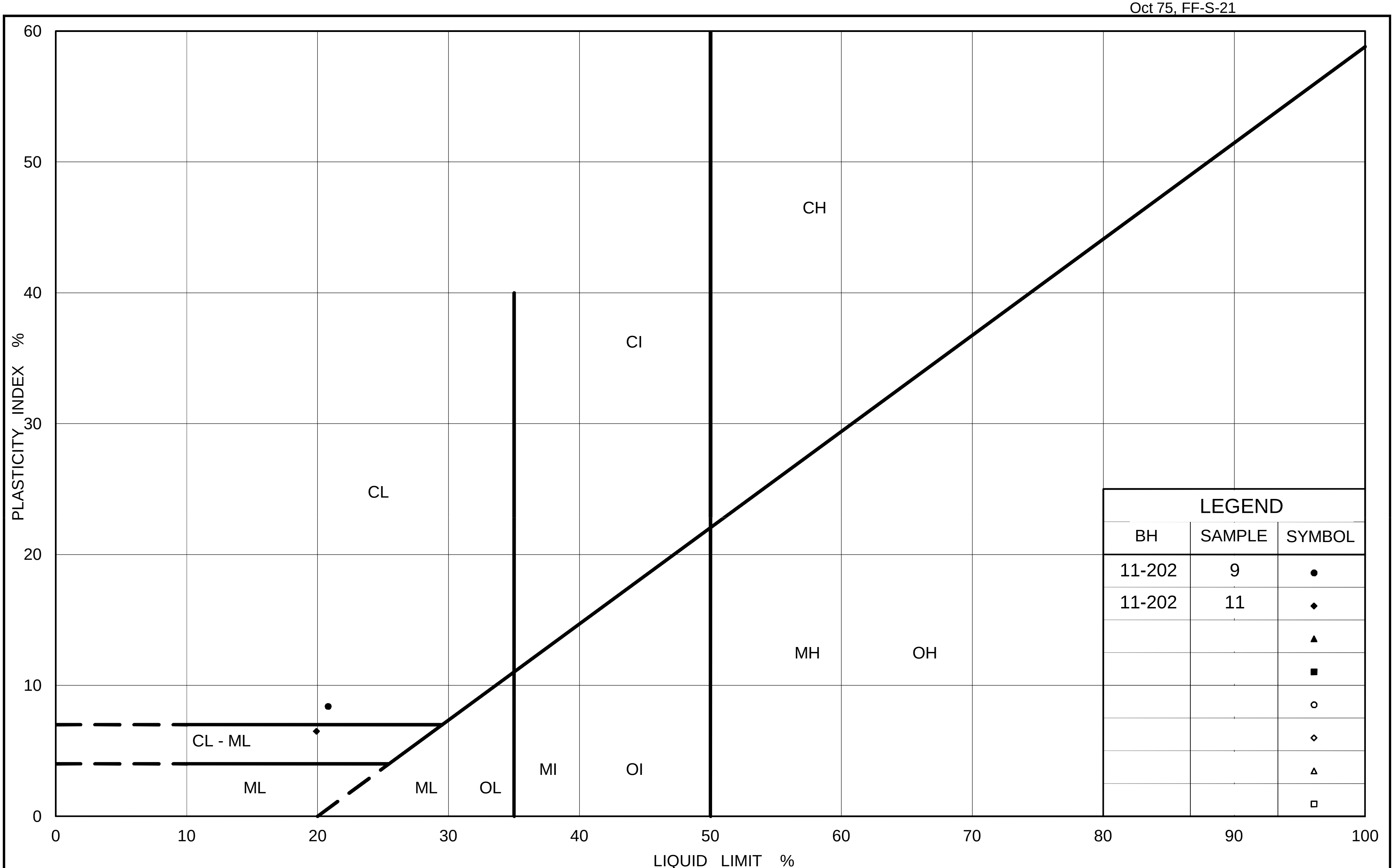
Date: 21-Feb-12



LEGEND		
BH	SAMPLE	SYMBOL
11-202	9	●
11-202	11	◆

Ministry of Transportation

Ontario

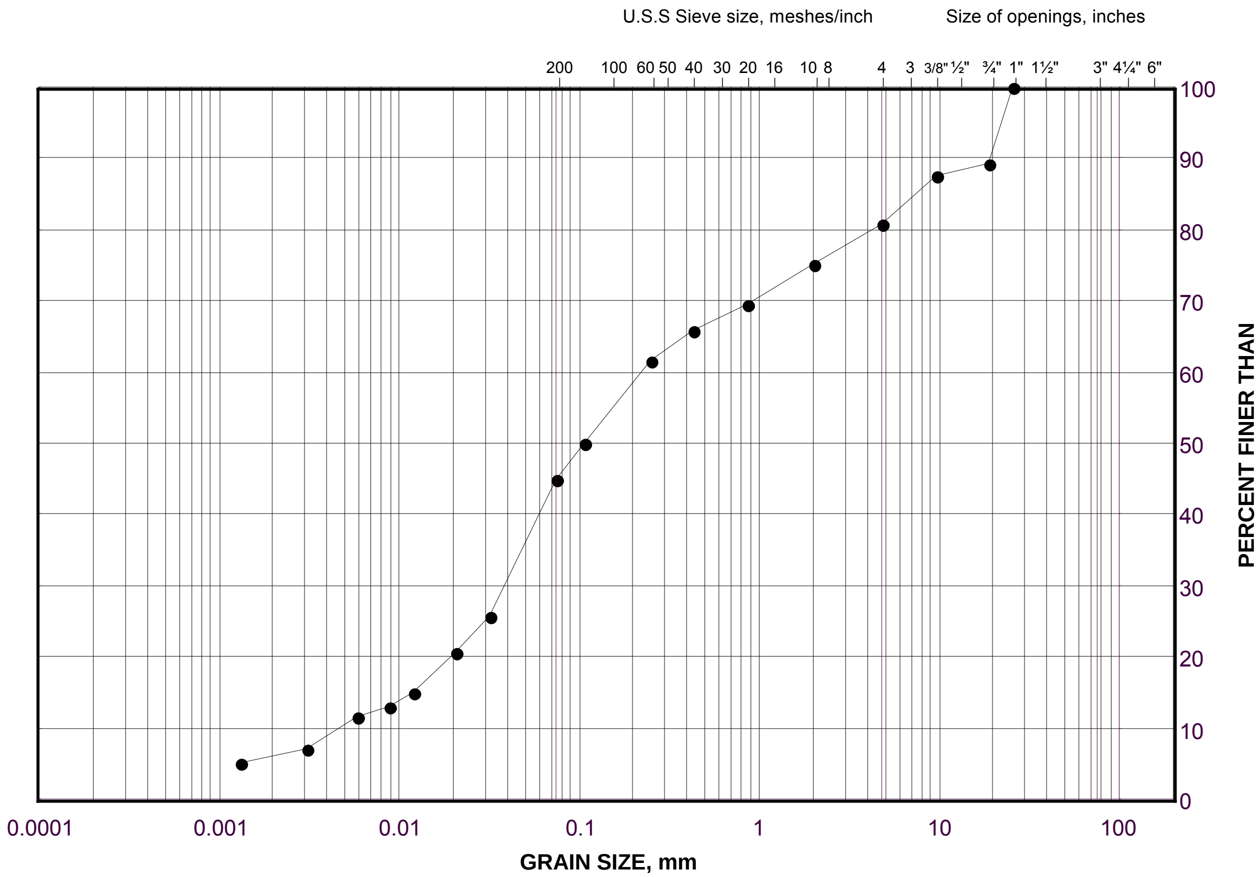
PLASTICITY CHART
Clayey Silt Till

Figure No. B7

Project No. 10-1111-0040-2

Checked By: MM

GRAIN SIZE DISTRIBUTION Sand and Silt Till	FIGURE B8



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	11-201	9	162.0

Project Number: 10-1111-0040-2

Checked By: MM

Golder Associates

Date: 21-Feb-12

UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE B9

BEFORE COMPRESSION

AFTER COMPRESSION

Date 10/19/2011
Project 10-1111-0040-2

Golder Associates

Drawn Frank
Chkd. MM

FORM PRODUCED JUNE 1986
Form G.A.-D-4 (imperial)

TABLE B1 - UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	10-1111-0040-2	SAMPLE NUMBER	-
BOREHOLE NUMBER	11-201	SAMPLE DEPTH, m	14.0-14.2

TEST CONDITIONS

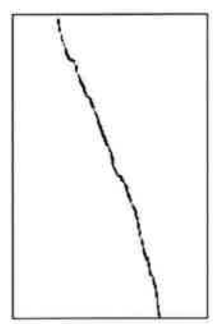
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.32

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.96	WATER CONTENT, (specimen) %	0.26
SAMPLE DIAMETER, cm	4.72	UNIT WEIGHT, kN/m^3	25.94
SAMPLE AREA, cm^2	17.50	DRY UNIT WT., kN/m^3	25.87
SAMPLE VOLUME, cm^3	191.87	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	507.74	VOID RATIO	0.02
DRY WEIGHT, g	506.42		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	31.8

REMARKS:

DATE: 10/19/2011

Checked By: MM

Golder Associates

TABLE B2 - POINT LOAD TEST ON ROCK SAMPLES

PROJECT NO. 10-1111-0040-2

TITLE URS / Hwy 401 Widening / Halton Peel

DATE October, 2011

Borehole Number	Sample Depth (m)	Test Type	Core Length (mm)	Core [2] Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	**Is (50mm)** (MPa)	**Approx. [1] UCS** (MPa)
11-202	14.51-14.58	D	38.39	41.23	-	8,320.00	7.89	-	4.640	4.254	89
11-201	11-41-11.48	D	33.06	46.90	-	740.00	0.70	-	0.319	0.310	7
11-202	14.88-14.95	A	15.38	47.28	30.43	12,180.00	11.55	12.472	-	9.974	229
11-201	12.17-12.24	A	14.79	46.82	29.69	5,700.00	5.40	6.129	-	4.848	111
11-202	15.83-15.90	D	40.65	41.29	-	7,560.00	7.17	-	4.204	3.857	81

[1] Is_{50} x C (actual value will have to be confirmed by UCS testing), from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

[2] Actual distance between point load cones at time of failure.

jll

Checked By: MM

APPENDIX C

Records of Boreholes from Previous Investigation (GEOCRES No. 30M12-035)

c.c. Foundation Section.

Mr. A.M. Toye.
Bridge Engineer.
Materials & Research.

September 30th, 1957.

Re: Foundation Report - W.P. 75-57. W.J.F. 57-12.

Highway #401 and road allowance between Concessions 3 & 4, one mile south of Meadowvale.

Attached herewith are two copies of the above mentioned Foundation Report. The sub-soil consists of a shallow layer of clay over till. If the spread footing is placed on this till layer, the bearing value of this layer at about elevation 552 would be competent to support approximately two tons per square foot. This bearing value may be increased if the footings are placed at a lower elevation.

This structure has already been given for design to Mr. Lee, consultant at Kingston, and we advised him on August 19th that the bearing value of 2 tons per square foot would be satisfactory at elevation 552.

F.C. Brownridge.
Materials & Research Engineer.

per: A. Rutka

A. RUTKA.
Principal Soils Engineer.

c.c. Mr. A. Toye.
Mr. H. Tregaskes.
Mr. D.G. Ramsay.
Mr. J.F. Wilkes.
Foundation Section.
File.

Foundation Report

on

Underpass Bridge at Highway 401
crossing road allowance between
concessions 111 & 1V, one mile
south of Meadowvale, Township of
Toronto.

Plan No.: F-3522-9

Station: 491+56.67

<u>Distribution.</u>

Mr. A. Toye
Bridge Engineer (2)

Mr. H. Tregaskes
Construction Engineer (1)

Mr. D. G. Ramsay
Design Engineer (1)

Mr. J. B. Wilkes
District Engineer, Toronto (1)

Foundation Section (1)

File (1)

W. P. 75-57

W. J. F-57-12

Introduction.

A subsoil investigation was carried out to determine the bearing values of the layers for supporting the foundations of the proposed structure.

The site is located at about one mile south of Meadowvale, where the new Highway 401 crosses the road allowance between Concessions 111 & 1V in the township of Toronto, County of Peel, (profile no. F-3522-8, station 491/56.67).

The work started on May 13, 1957 and was interrupted on May 17, 1957, due to unsettled status of the property. However, after the arrangements to enter the property were made, the work was resumed and was completed on August 14, 1957.

Procedure.

The investigation was carried out by a skid mounted coredrill machine. In the course of investigations 2 boreholes with dynamic cone penetration and 2 separate penetration tests were made.

The location of the boreholes are shown in Drawing F-57-12A and their elevations on log sheets under Appendix I.

Subsoil findings and Analysis.

The terrain is till plain with a veneer of presumably lacustrine deposit. The subsoil explorations revealed the following stratigraphy.

Under the topsoil down to elevation about 553 ft. the layer is brown, loamy clay. Below this elevation the soil is gravelly grey clay till. The borings were stopped at elevations 521 ft. (borehole no. 2), and 531 ft. (borehole no. 3) where dense boulders were encountered.

The samples extracted from the boreholes were tested in the laboratory. The test results show that the soil in the top brown clay layer (above elevation 553 ft.) has higher moisture content (24%) compared to 12%

moisture content of the soil below this elevation.

The standard penetration results, from the ground level down to 20 ft. depth, show an average of 25 blows per foot penetration. Due to gravelly nature of the soil the attempts for unconfined compression tests did not yield reliable results.

Conclusions and Recommendations.

From the above discussion it will follow that:

1. The terrain is till plain. The subsoil is brown loamy clay at the top, changing to grey gravelly clay till below elevation 553 ft.
2. The nature of the stratification and of the soil is convenient for spread footing type foundations. However, if due consideration were given to relatively soft consistency recorded in the upper 10 ft. from the ground level, it would seem advisable to place the footings not higher than elevation about 552 ft. At this elevation the layer can provide a bearing value of 2 T.s.f. and at elevation 550 ft. 2.5 T.s.f. For higher bearing values it would be safer to place the footings below this elevation 550 ft.
3. The approach fills to the structure do not present any stability problem.

V. Korlu

Foundation Engineer

VK/JY.

APPENDIX I

REV. 56)

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

RIG 54-2 OPERATION PENETRATION JOB E-57-12 W.P. 75-57 BORING 1 STA.

BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT Ac

ER HAMMER WT. 250 LBS. DROP 23 INCHES COMPILED BY H.S. CHECKED BY A.L. DATE BORING 13

ABBREVIATIONS

VANE SHEAR TEST
ICAL ANALYSIS
NED COMPRESSION
CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW
WL - WATER LEVEL IN CASING
WT. - WATER TABLE IN SOIL
K - PERMIABILITY
C - CONSOLIDATION
CA. - CASING
γ - UNIT WEIGHT

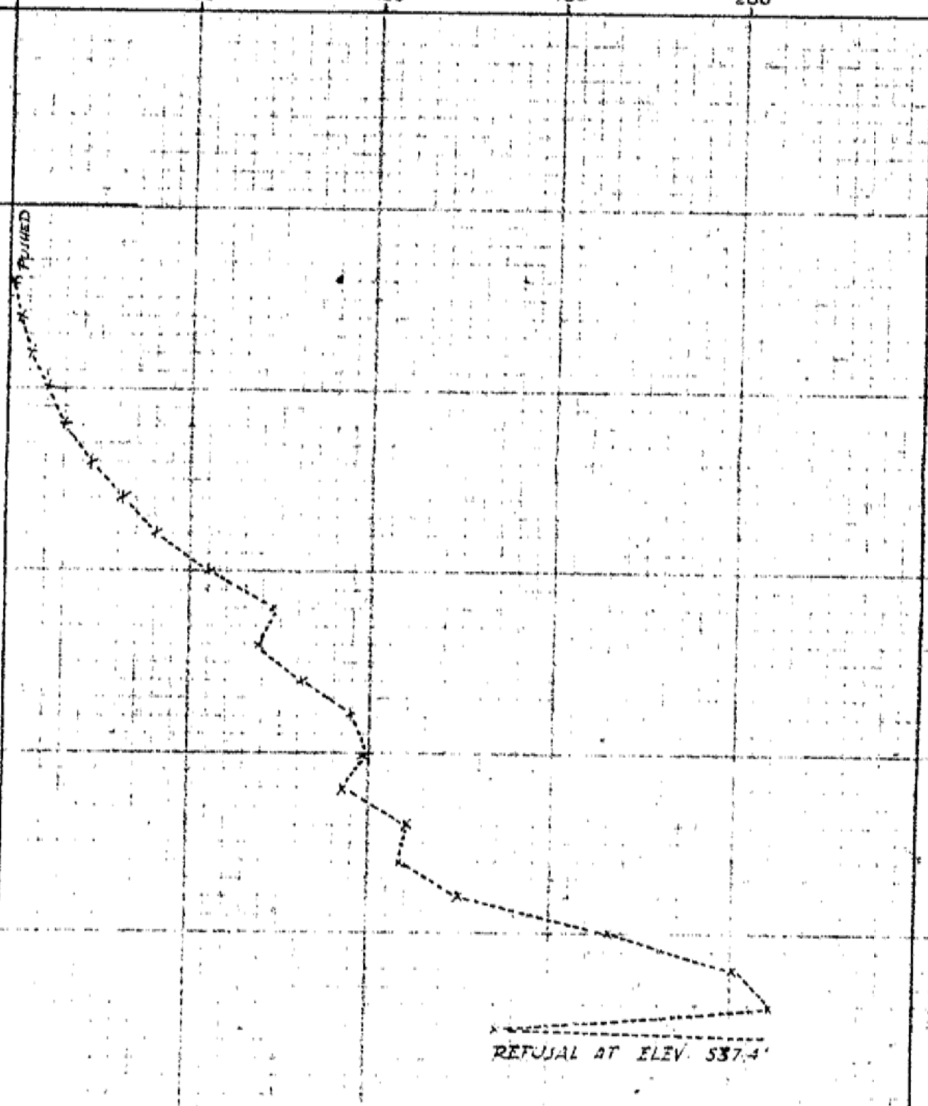
SAMPLE TYPES

C.S. - CHUNK
D.O. - DRIVE OPEN
D.F. - DRIVE FOOT VALVE
T.O. - THIN WALLED OPEN
S.S. - SLEEVE SAMPLE
P.S. - PISTON SAMPLE
W.S. - WASHED SAMPLE
R.C. - ROCK CORE

SAMPLE

L PROFILE

WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE	WATER CONTENT W % — O - NAT. □ - P.W. △ - L.W. / PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS. PER BLOW) D. CONE PEN. X-----X-----X STAND. PEN. •——•——• 50 100 150 200	CASING BLOWS (ACTUAL)	OTHER TESTS	CONDITION	TYPE	N
	GROUND LEVEL		560						
			555						
			550						
			545						
			540						
			535						



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG 54-2 OPERATION BORE & PENET'N JOB F 57-12 W.P. 75-57 BORING 2 STA. 491+37 (38' RT.)

CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT AUG. 1957

SAMPLER HAMMER WT. 250 LBS. DROP 23 INCHES COMPILED BY H.S. CHECKED BY A.L. DATE BORING 14 MAY 1957

ABBREVIATIONS

V - INSITU VANE SHEAR TEST; Q - TRIAXIAL QUICK; K - PERMIABILITY; M - MECHANICAL ANALYSIS; S - TRIAXIAL SLOW; C - CONSOLIDATION; U - UNCONFINED COMPRESSION; WL - WATER LEVEL IN CASING; CA. - CASING; Q_C TRIAXIAL CONSOLIDATED QUICK; W.T. - WATER TABLE IN SOIL; γ - UNIT WEIGHT

SAMPLE TYPES

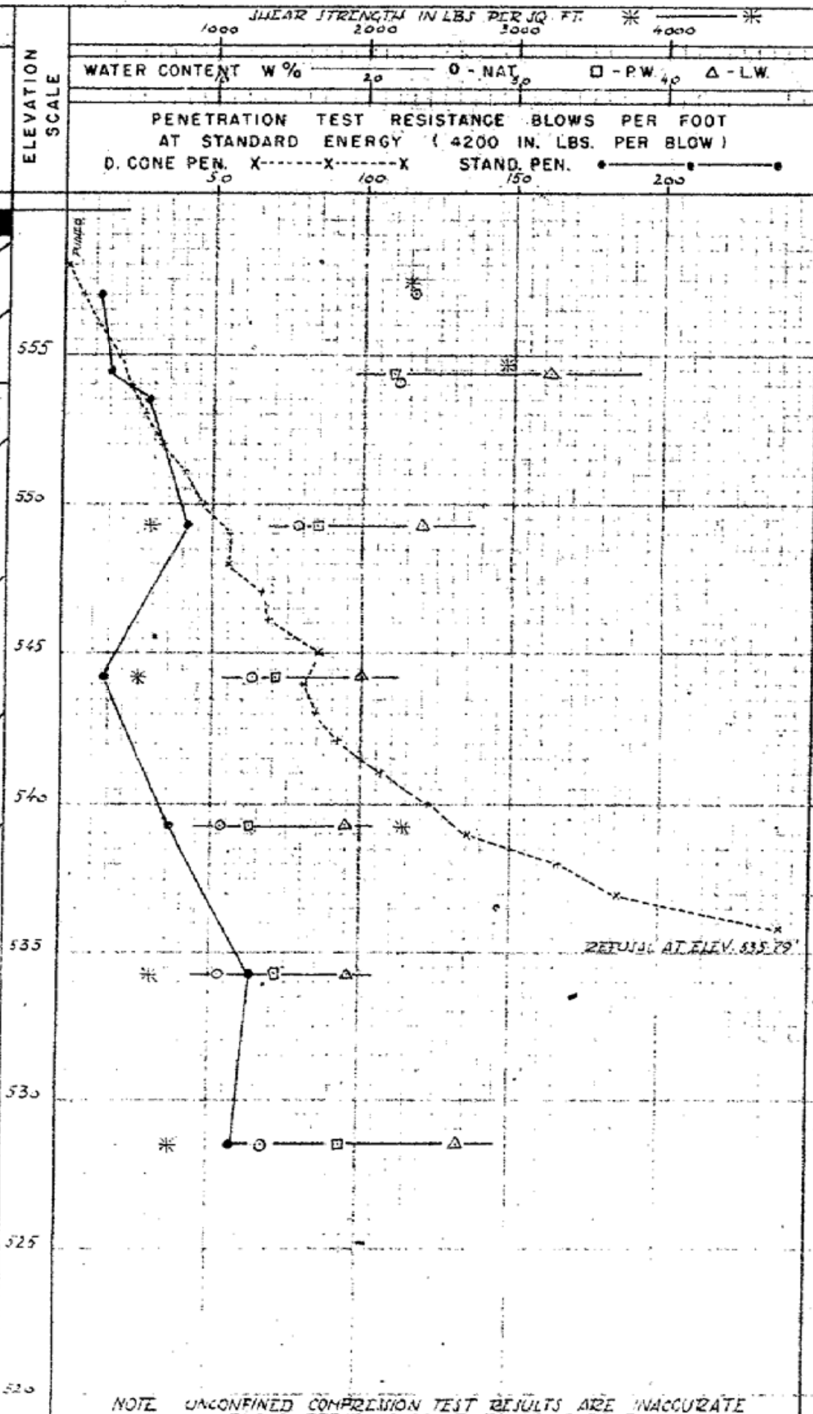
C.S. - CHUNK; S.S. - SLEEVE SAMPLE; D.O. - DRIVE OPEN; P.S. - PISTON SAMPLE; D.F. - DRIVE FOOT VALVE; W.S. - WASHED SAMPLE; T.O. - THIN WALLED OPEN; R.C. - ROCK CORE

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SOIL PROFILE

ELEVATION / DEPTH	WATER CONDITIONS	DESCRIPTION
559.79'		GROUND LEVEL
558.79' / 1.0		TOPSOIL
553.79' / 6.0'	W.L. @ ELEV. 550.99 on MAY 17/57 RECORDED FROM BORING	BROWN CLAY
		GREY CLAY TILL
		BOULDERS
521.29' / 38.5'		END OF BOREHOLE



NOTE UNCONFINED COMPRESSION TEST RESULTS ARE INACCURATE DUE TO PRESENCE OF GRAVEL OR PEBBLES

Casing blows (actual): 2, 5, 4, 8, 14, 12, 21, 32, 39, 53, 50, 70, 70, 72, 80, 47, 47, 55, 73, 79, 70, 105, 122, 134, 162, 92, 300, 187, 295, 150, 180, 88, 323, 106, 300; END OF CASING @ 525.29'

SAMPLES

OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV. %
					559.79
					557.79
γ 129.5 p.c.f.	GOOD	T.O.	1	11	75
					554.79
γ 126.5 p.c.f.	GOOD	T.O.	2	15 29	79
					549.79'
γ 131.5 p.c.f.	FAIR	S.S. 2"	3	41	67
W.S.					
					544.79'
γ 133.0 p.c.f.	GOOD	S.S. 2"	4	14	61
					539.79'
γ 148.0 p.c.f.	GOOD	S.S. 2"	5	36	72
					534.79'
γ 140.0 p.c.f.	DISTURBED	D.O.	6	63	72
					528.79'
γ 123.5 p.c.f.	DISTURBED	D.O.	7	56	83
		DRILLED			
					521.95

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

RIG 54-2 OPERATION BORE & PENET'N JOB F-57-12 W.P. 75-57 BORING 3 STA. 491

BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT AUG

ER HAMMER WT. 250 LBS. DROP 22 INCHES COMPILED BY H.S. CHECKED BY A.L. DATE BORING 13 AUG

ABBREVIATIONS

VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY

ICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION

NED COMPRESSION WL - WATER LEVEL IN CASING CA. - CASING

CONSOLIDATED QUICK W.T. - WATER TABLE IN SOIL γ - UNIT WEIGHT

SAMPLE TYPES

C.S. - CHUNK S.S. - SLEEVE SAMPLE

D.O. - DRIVE OPEN P.S. - PISTON SAMPLE

D.F. - DRIVE FOOT VALVE W.S. - WASHED SAMPLE

T.O. - THIN WALLED OPEN R.C. - ROCK CORE

SAMPLE CO

IL PROFILE

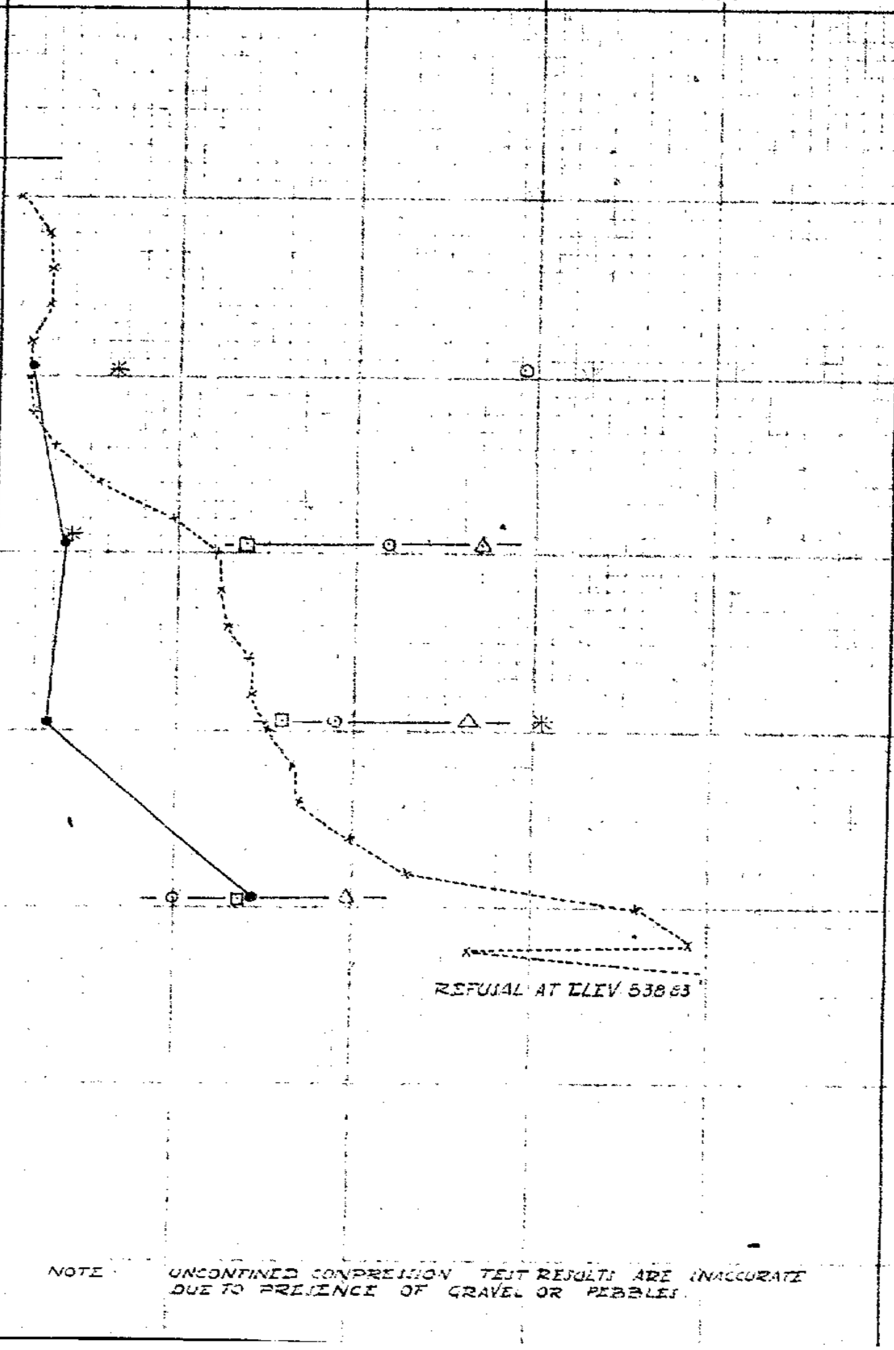
WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE
	GROUND LEVEL		
	TOPSOIL		560
	BROWN CLAY		555
	GREY CLAY TILL		550, 545, 540
	BOULDERS		535
	END OF BOREHOLE		530

SHEAR STRENGTH IN LBS PER SQ. FT. 1000 2000 3000 4000

WATER CONTENT W % O - NAT. □ - P.W. Δ - L.W. 10 20 30 40

PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS. PER BLOW)

D. CONE PEN. X-------X-------X STAND. PEN. •——•——• 50 100 150 200



NOTE UNCONFINED COMPRESSION TEST RESULTS ARE INACCURATE DUE TO PRESENCE OF GRAVEL OR PEBBLES

CASING BLOWS (ACTUAL): 6, 12, 16, 13, 9, 5, 11, 17, 27, 40, 32, 36, 41, 37, 39, 38, 41, 58, 76, 106, 64, 95, 200/182

END OF CASING @ 537.63

SAMPLE

OTHER TESTS	CONDITION	TYPE	NO.
γ = 112.8 p.c.f.		S.S. 2"	1
γ = 123.5 p.c.f.		S.S. 2"	2
γ = 124.3 p.c.f.		S.S. 2"	3
γ = 139.6 p.c.f.		S.S. 2"	4
ATTEMPT OF D.O. SAMPLER RE			
	DRILLED		

EV. 56)

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW

OFFICE REPORT ON SOIL EXPLORATION

G 54·2 OPERATION PENETRATION JOB F-57-12 W.P. 75-57 BORING 4 STA. 4

BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT AUG

HAMMER WT. 250 LBS. DROP 22 INCHES COMPILED BY H.J. CHECKED BY A.L. DATE BORING 14 A

ABBREVIATIONS

NE SHEAR TEST
AL ANALYSIS
ED COMPRESSION
CONSOLIDATED QUICK

Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

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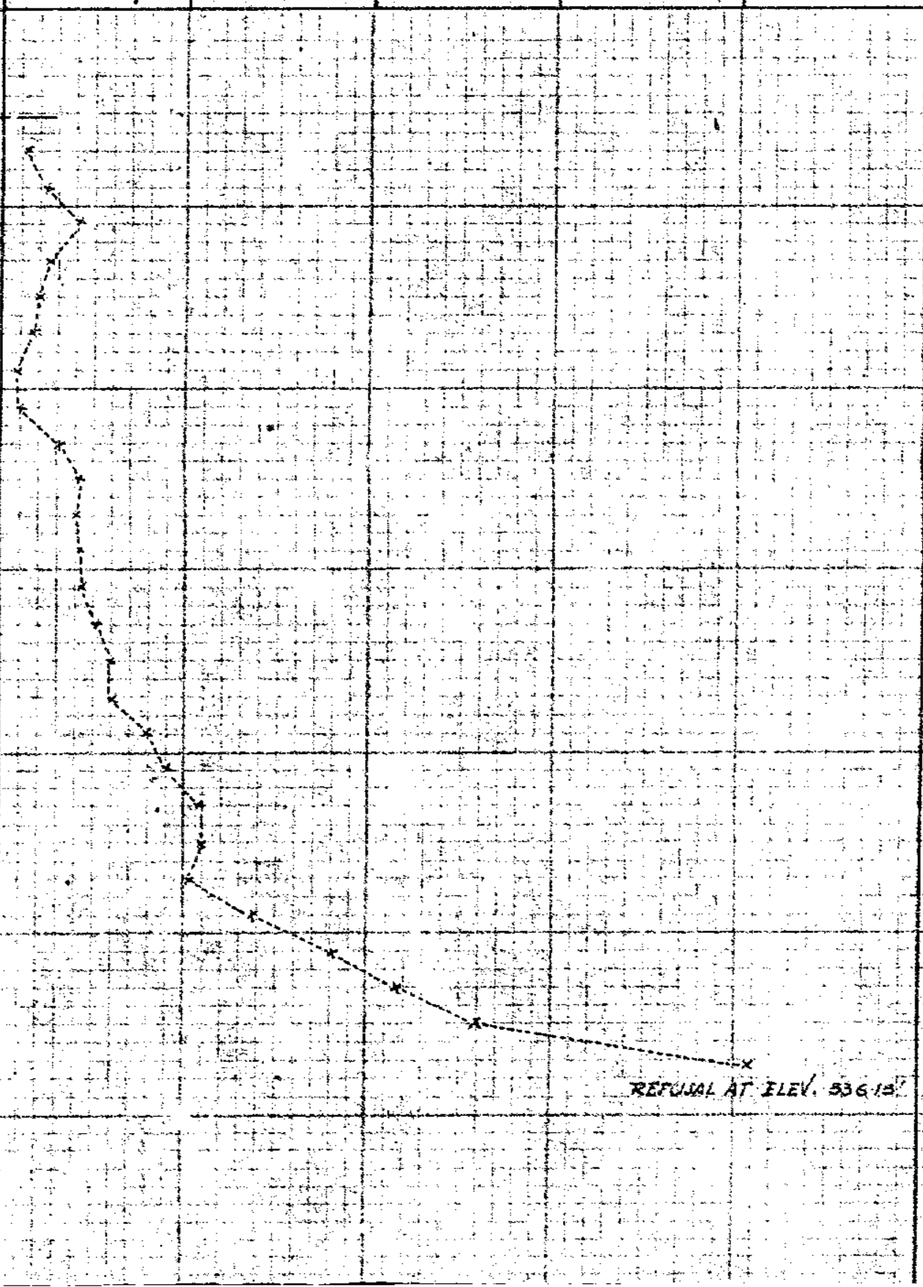
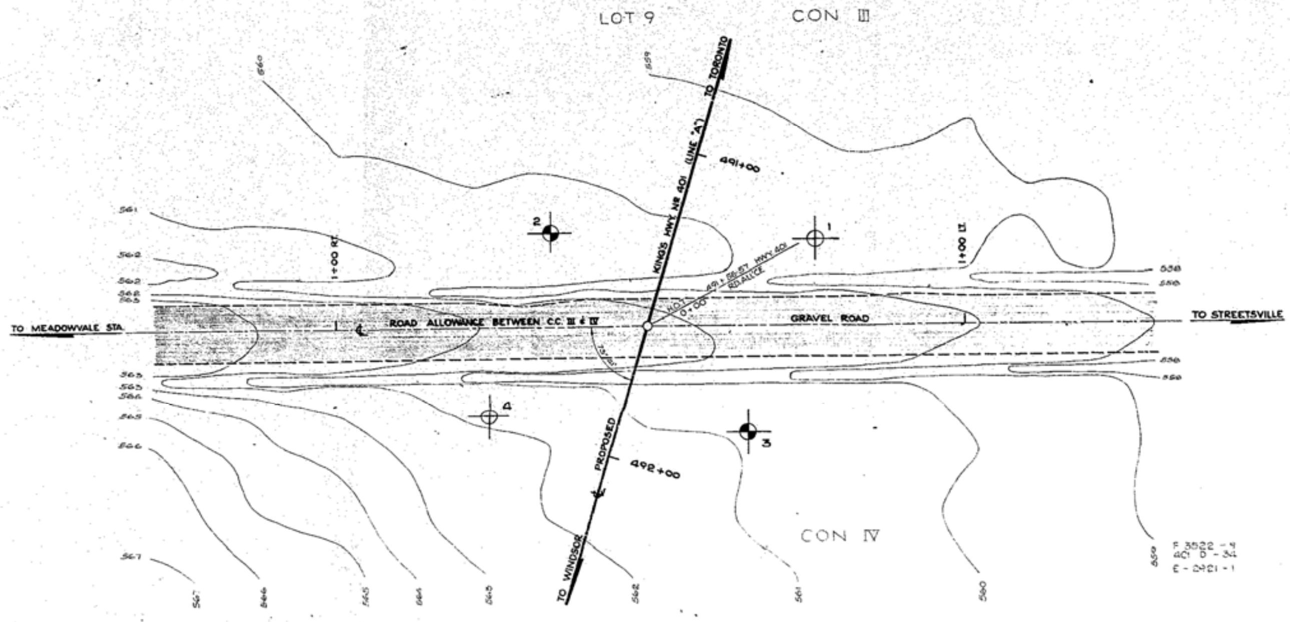
S.S. - SLEEVE SAMPLE
P.S. - PISTON SAMPLE
W.S. - WASHED SAMPLE
R.C. - ROCK CORE

SAMPLE C

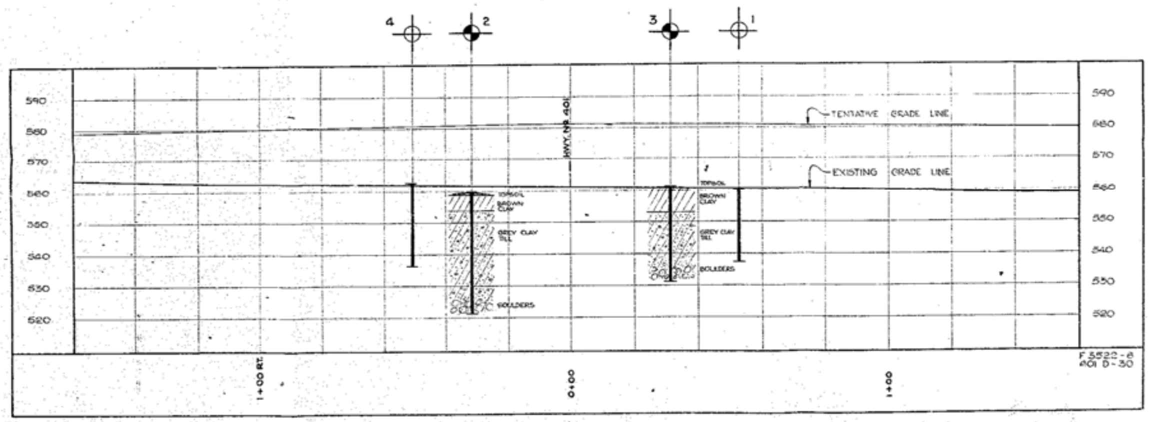
PROFILE							SAMPL		
WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE	WATER CONTENT W % — O - NAT. □ - P.W. Δ - L.W. PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS. PER BLOW) D. CONE PEN. X-------X-------X STAND. PEN. •———•———• 50 100 150 200	CASING BLOWS (ACTUAL)	OTHER TESTS	CONDITION	TYPE	NO
	GROUND LEVEL		560 555 550 545 540 535						



#57-F-12
W.P.#75-57
Hwy.#401
Underpass Bridge
Con.#3 & #4
1 mile S. of
Meadowvale



PLAN SCALE 1 IN = 20 FT



PROFILE SCALE HOR VER 1 IN = 20 FT

LEGEND

BORE HOLES

PENETRATION HOLE

BORE & PENETRATION HOLE

HOLE NO.	ELEVATION	STATION	DISTANCE FROM ℄
1	560.09'	491 +16'	43' LT
2	559.79'	491 +37'	38' RT
3	561.13'	491 +81'	40' LT
4	562.45'	491 +98'	40' RT

— NOTE —

THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BORE HOLE LOCATIONS. BETWEEN BORE HOLES THE BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE AND MAY BE SUBJECT TO CONSIDERABLE ERROR.

DEPARTMENT OF HIGHWAYS:-ONTARIO-
MATERIALS & RESEARCH SECTION - DOWNSVIEW

GRAVEL ROAD PROPOSED CROSSING 1/2 MILE S.E. OF MEADOWVALE STA.

THE KING'S HIGHWAY No. 401 (LINE 'A') DIV. No. 4
CO. PEEL
TWP. TORONTO LOT 9 CON. III & IV

POSITION & ELEVATION OF HOLES

APPROVED

ENGINEER CHIEF ENGINEER

DESIGN		CHECK		CONTRACT NUMBERS	W.P. 75-57
DRAWING	R.E.F.	CHECK			
TRACING		CHECK		LOADING	DRAWING NUMBER F-57-12 A
SEPTEMBER 5, 1957					

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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