



June 2012

REPORT ON

**Foundation Investigation and Design Report
Recreational Trail Culvert
Structure No. 26-035
Highway 7 from Fowlers Corners
Southerly to County Road 28
Peterborough, Ontario
W.P. 245-00-01**

Submitted to:

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REPORT



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FOUNDATION INVESTIGATION AND DESIGN REPORT RECREATIONAL TRAIL CULVERT

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FOUNDATION INVESTIGATION AND DESIGN REPORT RECREATIONAL TRAIL CULVERT

**PART A
FOUNDATION INVESTIGATION REPORT
DETAILED DESIGN
RECREATIONAL TRAIL CULVERT, STRUCTURE No. 26-035
HIGHWAY 7 FROM FOWLERS CORNERS
SOUTHERLY TO COUNTY ROAD 15
PETERBOROUGH, ONTARIO
G.W.P. 245-00-01**



FOUNDATION INVESTIGATION AND DESIGN REPORT RECREATIONAL TRAIL CULVERT

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by D.M. Wills Limited on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with proposed highway operational improvements and future four laning of Highway 7 from Fowlers Corners Southerly to County Road 28 in Peterborough, Ontario.

The terms of reference for the scope of work are outlined in Golder's proposal P0-1121-0007, dated February 2010, that forms part of the consultant's agreement (GWP 4053-06-00/245-00-01/345-01-01). Detailed foundation investigation and design services are required for a total of four structures (i.e., Jackson Creek Bridge, Trans-Canada Recreational Trail Culvert, and two structural culvert sites) and one high fill section in five separate reports for this project. This report addresses the detailed foundation investigation carried out for the proposed Trans-Canada Recreational Trail Culvert at about Station 26+414 along the proposed realignment of Highway 7 as part of the Highway 7 improvement project.

The purpose of this investigation is to establish the subsurface conditions at the proposed culvert (Structure No. 26-035) by borehole drilling, in situ testing and laboratory testing on selected samples.

The current investigation was supplemented with information from two previous investigations at or near this site, as follows:

- Racey, MacCallum and Associates report titled "Foundation Investigation for a Bridge over the CNR Bridge Crossing at Fowlers Corner, Highway 133, near Peterborough, Ontario", dated September 20, 1957, Geocres No. 31D-227.
- Golder's report titled "Foundation Investigation and Design, Preliminary Design, Recreational Trail Culvert, Highway 7 from Fowlers Corners, Southerly to County Road 28, Peterborough, Ontario, G.W.P. 73-99-00, Site No. 26-35", dated June 2007, Geocres No. 31D-427.



2.0 SITE DESCRIPTION

The general site is located east of the existing Highway 7, approximately 350 m north of the existing intersection of Highway 7 and Lily Lake Road in Peterborough, Ontario (see key plan on Drawing 1). The existing highway in this area has two lanes, one lane each for northbound and southbound traffic. There is an existing structure that carries Highway 7 over an abandoned CNR railway line. This abandon line has been converted into a recreational / pedestrian walking trail.

The site generally consists of flat terrain comprised of fields, grassy areas and sparse vegetation east and west of the existing highway embankment and the overhead structure. The ground surface east and west of the highway embankment generally ranges from about Elevation 266 to 269 m. The existing deck of the Highway 7 overhead structure is at about Elevation 278.2 m, therefore the approach embankments are about 9.2 to 12.2 m in height and have side-slopes inclined at about 2.5 horizontal to 1 vertical (2.5H:1V). The existing highway embankment side-slopes are grass covered with no visible signs of instability which was observed at the time of our investigation. The existing structure is a three-span reinforced concrete bridge shown on the design drawings and is supported on steel H-pile foundations driven to refusal. It is understood that the existing overhead structure is to be removed once the new recreational trail structure has been constructed and the new highway re-alignment is complete.



3.0 INVESTIGATION PROCEDURES

3.1 Previous Foundation Investigation

The preliminary field work for this structure investigation was carried on July 10 and 11, 2006, during which time two (2) boreholes (designated as 06-13 and 06-14) were advanced at the approximate locations shown in plan on Drawing 1.

The field investigation was carried out using a track-mounted CME-55 drill rig supplied and operated by Eastern Drilling Investigation Limited of Courtice, Ontario. The boreholes were advanced using 107 mm outside diameter (O.D.) solid stem augers. Soils samples were obtained at intervals ranging from 0.75 m to 1.5 m in depth, using a 50 mm outside diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures.

The boreholes were sampled to depths of 9.3 m below the existing ground surface. The groundwater conditions in the open boreholes were observed throughout the drilling operations and piezometers were installed at both boreholes to monitor the groundwater level at the site. The piezometers consisted of 50 mm diameter PVC pipe, with a slotted screen sealed at a select depth within the borehole. The boreholes and annulus surrounding the piezometer pipe were backfilled to the surface with bentonite pellets in accordance with Ontario Regulation (O.Reg.) 903 amended to O.Reg. 128/03 of the Ontario Water Resources Act. The piezometer installation details and water level readings are described on the Record of Borehole sheets in Appendix B.

The field work was observed on a full time basis by a member of Golder's engineering staff who located the boreholes in the field, arranged for clearance of underground services, monitored the drilling, sampling, and in situ testing operations, logged the boreholes, and examined and cared for the soil samples. The soil samples were identified in the field, placed in appropriate containers and transported to Golder's laboratory in Mississauga where the samples underwent further detailed visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on select samples.

The borehole locations were identified in the field by Golder relative to on-site features. Upon completion of drilling operations, the borehole locations (i.e., MTM NAD83 northing and easting coordinates) and ground surface elevations (reference to geodetic datum) were surveyed by a licensed surveyor (i.e., Transenco Limited) and are summarized below and on Drawing 1.

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-13	4907829.8	390031.5	268.4
06-14	4907829.5	389997.4	268.3

3.2 Current Foundation Investigation

The field work for the current structure investigation was carried on September 8 to 10, 2010, during which time three (3) boreholes (designated as 10-1 to 10-3) were advanced at the locations shown in plan on Drawing 1. Relevant to this structure is borehole 10-8, which was put down east of the structure as part of the high fill embankment investigation.



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The field investigation was carried out using a track-mounted Diedrich D50 drill rig supplied and operated by Walker Drilling Limited of Utopia, Ontario. The boreholes were advanced using 107 mm outside diameter (O.D.) solid stem augers. Soils samples were obtained at intervals ranging from 0.75 m to 1.5 m in depth, using a 50 mm outside diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures.

The boreholes at the proposed culvert were sampled to depths of 10.1 to 10.4 m below the existing ground surface. A piezometer was installed at borehole 10-2 to monitor the groundwater level at the site. The piezometer consisted of 50 mm diameter PVC pipe, with a slotted screen sealed at a select depth within the borehole. The borehole and annulus surrounding the piezometer pipe were backfilled to the surface with bentonite pellets in accordance with Ontario Regulation (O.Reg.) 903 amended to O.Reg. 128/03 of the Ontario Water Resources Act. The piezometer installation details and water level readings are described on the Record of Borehole sheets in Appendix A.

The field work was observed on a full time basis by a member of Golder's engineering staff who located the boreholes in the field, arranged for clearance of underground services, monitored the drilling, sampling, and in situ testing operations, logged the boreholes, and examined and cared for the soil samples. The soil samples were identified in the field, placed in appropriate containers and transported to Golder's laboratory in Ottawa where the samples underwent further detailed visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classification testing (water content and grain size distribution) was carried out on select samples.

The borehole locations were staked in the field by Golder using a Trimble R8 GPS survey unit. Upon completion of drilling operations, the borehole locations (i.e., MTM NAD83 northing and easting coordinates) and ground surface elevations (referenced to geodetic datum) were surveyed by Golder using a Trimble R8 GPS survey unit and are summarized below and on Drawing 1.

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
10-1	4907839.4	390028.7	268.0
10-2	4907824.3	390022.5	268.5
10-3	4907832.2	390016.0	267.9
10-8	4907832.7	390053.2	268.1



4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario*¹, the study area for this assignment lies within the physiographic region known as the Peterborough Drumlin Field.

The surficial soils in the Peterborough Drumlin Field consist of drumlinized till. Toward the southwestern portion of this physiographic region, near the Oak Ridges Moraine, the till is typically sandy. Some of the drumlins in this area have shallow coverings of silt and fine sand, between about 0.5 m and 2.5 m in thickness. "Wave-washed" drumlins, with exposed bouldery surfaces, are also present near the Simcoe Lowlands immediately south and east of Lake Simcoe.

4.2 Subsoil Conditions

The following sections discuss the findings of the borehole investigation carried out at the proposed location of the Trans-Canada recreational trail culvert, as shown on Drawing 1. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of the in situ and laboratory testing are given on the Record of Borehole Sheets and on the figures in both Appendices A and B. The stratigraphic boundaries shown on the borehole records 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.

The subsoils at the proposed structure location generally consist of granular fill overlying a thin discontinuous deposit of silty clay and clayey silt, which in turn is underlain by a sand to silty sand layer, overlying glacial till. Surficial topsoil was encountered over much of the fill layer and in the eastern portions of the site peat and organics were also encountered. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil and Fill

A surficial layer of topsoil was encountered in the boreholes 10-1, 10-2, 10-3, 10-8 and 57-3. The thickness of this topsoil varied from 100 to 300 mm in the more recent boreholes.

Fill was encountered in all the boreholes, with the exception of the boreholes 57-1 and 57-3. This fill was encountered below the topsoil in boreholes 10-1, 10-2, 10-3 and 10-8 and at the ground surface in boreholes 06-13 and 06-14. This fill extended to depths varying from 0.8 to 2.1 m below the existing ground surface (Elevations 267.2 to 266.0 m, respectively). The fill generally consisted of silty sand with varying amounts of gravel and cobbles.

A second layer of possible fill was encountered in boreholes 10-1 and 10-3, due the composition of this material and the surrounding soils it is difficult to determine if this material is native or imported. This possible fill generally consists of silty sand with varying amounts of gravel. This layer extends to 1.7 and 1.5 m (Elevation 266.3 and 266.4 m) below the existing ground surface, respectively.

¹ Chapman, L.J. and D.F. Putnam. *The Physiography of Southern Ontario*, Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,000.



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The Standard Penetration Test (SPT) “N” values recorded within the fill material ranged from 3 to 12 blows per 0.3 m of penetration, indicating that the fill is a very loose to compact relative density.

The laboratory natural water content measured on select samples of the fill varied from 9 to 15 percent. Grain size distribution tests performed on select samples of the fill are shown on Figures A1 and A2 in Appendix A.

4.2.2 Peat and Organic Soils

Fibrous peat and soils containing organics were encountered below the fill deposit in boreholes 06-13 and 10-8. Peat and clayey silt containing organics were encountered in borehole 06-13 at a depth of 1.5 m (Elevation 266.9 m) and was about 0.8 m thick. Silty sand containing organics was encountered in borehole 10-8 at a depth of 2.1 m (Elevation 266.0 m) and was about 0.4 m thick.

In borehole 06-13, an SPT “N” value recorded within the peat and clayey silt containing organics layer was 0 blows (i.e., weight of hammer) per 0.3 m of penetration indicating the peat and clayey silt layer has a very soft consistency. In borehole 10-8, the SPT “N” value in the silty sand was 12 blows per 0.3 m of penetration indicating a compact relative density.

The natural water content measured on two samples of the organic soil was 32 and 45 percent.

4.2.3 Silty Clay to Clayey Silt

A thin layer of silty clay and clayey silt was encountered below the fill, peat, clayey silt/silty sand with organics in boreholes 10-1, 10-3, 10-8, 06-13 and 06-14. The cohesive layer contained isolated sand and gravel seams. The top of the clayey silt to silty clay deposit was encountered at depths of about 1.5 m to 3.2 m (Elevation 266.4 to 264.9 m) and the thickness of the deposit ranged from about 0.2 to 2.0 m.

The SPT “N” values recorded within the silty clay deposit ranged between 4 to 27 blows per 0.3 m of penetration, indicating a firm to very stiff consistency.

The natural water content measured on select samples of the silty clay layer ranged from 11 to 24 percent. The results of Atterberg Limits testing carried out on these samples of silty clay deposit are illustrated on the plasticity chart on Figure A5 in Appendix A and Figure A1 in Appendix B. The test results are summarized below and indicate the silty clay is of low to medium plasticity.

Borehole	Sample	Elevation (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
10-1	2	266.5 – 265.9	40	21	19
06-13	4	264.8 – 265.4	15	9	6
06-14	2	266.2 – 266.8	25	17	8



4.2.4 Sand to Silty Sand

Underlying the silty clay and clayey silt in boreholes 10-1, 10-3, 10-8, 06-13 and 06-14 and underlying the fill and topsoil in boreholes 10-2, 57-1 and 57-3, a layer of native silty sand to fine to medium sand with trace amounts of gravel and clay was encountered in all the boreholes. The thickness of this layer ranges from 1.5 to 3.2 m. This layer extends down to elevations ranging from Elevation 261.5 m in the east at borehole 10-8 to Elevation 265.3 m in the west at boreholes 57-1 and 57-3.

SPT “N” values measured within the silty sand to silty sand and gravel layer range from 8 blows per 0.3 m of penetration to greater than 100 blows per 0.3 m of penetration, indicating a loose to very dense relative density, with an average SPT ‘N’ value of 29 blows per 0.3 m of penetration indicating generally a compact relative density.

The natural water content measured on select samples of the native sand layer ranged from 11 to 21 percent. Figure A3 shows the results of a grain size distributions performed on a selected sample within this sand layer.

4.2.5 Glacial Till

Underlying the native sand to silty sand layer, a glacial till deposit was encountered in all the boreholes. The glacial till deposit as recovered in the sampler was principally silty sand and contained some clay and gravel. The drilling encountered some cobbles and boulders while penetrating this layer. The top of this layer was encountered at depths varying from about 3.1 m in the west at boreholes 57-1 and 57-3 to a depth of about 6.6 m in borehole 10-8 in the east. In boreholes 10-1, 10-2 and 10-3, near the centre of the proposed structure, the top of the till was encountered at a depth of about 4 metres. The depth of the till deposit extends beyond the termination depths of the boreholes.

SPT “N” values measured within the silty sand till deposit ranged from 35 blows per 0.3 m of penetration to greater than 100 blows per 0.3 m of penetration at depth with an average SPT ‘N’ value of 85 blows per 0.3 m of penetration. The SPT “N” values indicate that the glacial till has a dense to very dense relative density.

The natural water content measured on samples of the till ranged between about 5 or 7 percent. Grain size distribution curves for selected samples of the till deposit are shown on Figure A4 in Appendix A and in Figures A2 and A3 in Appendix B.

4.2.6 Bedrock

Bedrock was reported to have been encountered at a depth of 9.1 m below ground surface (Elevation 259.3 m) in borehole 57-3, which was cored about 0.3 m into the rock during the 1957 investigation. The bedrock was described as a grey limestone; however, the Total Core Recovery was only about 17 percent (50 of 305 mm). Therefore, this core sample retrieved could possibly be a cobble or boulder within the overlying till and not actual bedrock, since the glacial till in all the surrounding boreholes extended beyond this depth.

4.2.7 Groundwater Conditions

Piezometers were installed in boreholes 10-2, 06-13 and 06-14. The piezometers were sealed into the glacial till deposits below the sand to silty sand layer. Details of the piezometer installations are shown on the Record of Borehole Sheets following the text of this report. The water levels measured in the piezometers and open boreholes upon completion of drilling are summarized below.



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Borehole	Installation	Ground Surface Elevation (m)	Depth to Water Level (m)	Water Level Elevation (m)	Date
10-2	Piezometer	268.5	2.1	266.4	September 24, 2010
			2.1	266.4	October 6, 2010
			2.0	266.5	October 16, 2010
			2.0	266.5	December 13, 2010
			1.6	266.9	April 26, 2011
06-13	Piezometer	268.4	8.7	259.7	July 10, 2006
			7.2	261.2	July 11, 2006
			1.7	266.7	July 31, 2006
			2.1	266.3	August 18, 2006
			1.8	266.6	April 26, 2011
06-14	Piezometer	268.3	7.3	261.0	July 11, 2006
			1.6	266.7	July 31, 2006
			1.8	266.5	August 18, 2006
			1.2	267.1	April 26, 2011

The groundwater level measured in the piezometers both during this investigation and in the previous investigation was approximately at Elevation 267.0 m. Groundwater levels at the site of this structure are expected to fluctuate seasonally and a measured range of 1.4 metres from summer to spring was observed.



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5.0 CLOSURE

The field technician supervising the drilling program was Mr. Harold Cameron. This report was prepared by Mr. Bruce D. Goddard, P.Eng., a senior geotechnical engineer. Mr. Fintan J. Heffernan, Golder's Designated MTO Contact for this project, conducted a technical and independent quality control review of the report.

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**PART B
FOUNDATION DESIGN REPORT
DETAILED DESIGN
RECREATIONAL TRAIL CULVERT, STRUCTURE No. 26-035
HIGHWAY 7 FROM FOWLERS CORNERS
SOUTHERLY TO COUNTY ROAD 15
PETERBOROUGH, ONTARIO
G.W.P. 245-00-01**



6.0 ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the detailed design of the proposed culvert as part of the proposed highway operational improvement plan. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the detailed as well as the preliminary subsurface investigation at this site. The interpretation and recommendations provided are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out design of the proposed culvert foundations. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project.

6.1 General

As part of the highway operational improvement plan, it is understood that Highway 7 is to be relocated to the east of the current alignment and eventually will be widened to four lanes in the future. Based on the results of inspection of the existing bridge structure (performed by Harmer Podolak Engineering Consultants Inc.), we understand that the existing bridge requires significant rehabilitation and maintenance. As a result, considering the railway has been abandoned, the more feasible long-term option is to construct a smaller culvert structure with an opening designed to accommodate the passage of pedestrians and recreational vehicles below Highway 7 along the existing recreational trail. It is also understood that the existing CNR overhead bridge structure is to be removed / demolished. The proposed new culvert will be located along the new Highway 7 alignment at about Station 26+410 (see Drawing 1) and the midpoint of the new culvert is to be located 43.5 m east of the centerline of the existing Highway 7.

The existing ground surface at the proposed culvert site is generally flat and is covered with grass and small shrubs. The existing recreational trail (i.e., the previous railway track embankment) is slightly elevated and runs in an east-west direction and separates two low-lying farm fields located north and south of the trail. The existing CNR structure (i.e., a three span concrete bridge) is located directly west of the proposed new structure site. The existing ground surface at the site varies between about Elevation 267 m and 269 m. Based on the existing topography and original CNR bridge design drawings provided to us, the existing CNR bridge deck is at about Elevation 278.2 m, resulting in an existing embankment height of about 11 m with 2.5H:1V side-slopes.

Based on a draft General Arrangement (GA) drawing provided by D.M. Wills on May 17, 2012, the proposed recreational trail culvert is to consist of a cast in place open footing culvert approximately 5.0 m wide x 5.0 m high x 25 m long. Retaining walls (i.e., wing walls) are proposed to be constructed at both ends of the proposed culvert which are flared at an angle of about 45 degrees and are about 6 m in length. The proposed new realigned embankment is shown on the GA drawing to be at elevation 274.67, about 6 m high above existing grade. The culvert and wing walls should be designed to withstand the maximum anticipated overburden, lateral pressures and live loads. The effects of frost should also be considered in the structural design if founding levels, groundwater levels, or frost susceptible soils are located within the depth of frost penetration.

It is understood that after the new culvert structure and Highway 7 re-alignment are complete, traffic will be redirected to the new highway alignment and the existing structure and embankments removed. We understand the recreational trail is to be temporarily diverted around the site during construction.



6.2 Culvert Foundation Options

The shallow subsoils at the proposed culvert site consist of a surficial layer of topsoil and sand fill (from the railway and highway embankment construction) underlain by a silty clay / clayey silt deposit. A thin layer of peat and soils (clayey silt and silty sand) containing organics was encountered between the fill and silty clay / clayey silt deposit in boreholes 10-8 and 06-13 along the eastern portion of the proposed culvert. Underlying the silty clay / clayey silt deposit, a layer of silty sand was encountered, underlain by a glacial deposit of silty sand till. Water levels were measured at depths of 2 m below ground surface upon completion of the drilling operations. However, piezometers installed at boreholes 10-2, 06-13 and 06-14, which were sealed within the upper portion of the till deposit, measured water levels ranging from 1.2 m (Elevation 267.1 m) to 1.8 m (Elevation 266.6 m) below ground surface during spring conditions (April 2011). It should be noted that the water levels in this area will fluctuate on a seasonal basis.

Based on the subsurface information obtained, silty clay / clayey silt deposit (located below the surficial topsoil, fill, peat, and soils containing organics) is stiff to very stiff and considered suitable for the support of the proposed culvert foundation or engineered fill soils supporting the foundation. Several foundation options were considered and the advantages, disadvantages, relative costs and risks associated with each option are summarized in Table 1. As indicated in Table 1, from a foundations perspective, the preferred culvert structure type at this location is a concrete box culvert, with an open footing culvert also being suitable.

It is understood that the preferred culvert from a structural perspective will be a cast in place structure with longitudinal reinforcement. Due to the size of the culvert structure, precast segments of this culvert would not be easily transported or lifted into place, therefore a precast structure is not preferable. In addition, a precast closed box structure would consist of multiple segments and joints across the length; the joints would need to be sealed and wrapped with geotextile and there is the risk of future soil and/or water migration through these joints. The interior of the precast culvert would also require an interior lining to promote water runoff (prevent seepage through the joints) and to minimize trip hazards or “bumps” at the joint locations.

Some subexcavation of any topsoil, fill, peat, soils containing organics and very soft to soft clayey silt will be needed, especially in the eastern portions of the proposed culvert. These soils will require replacement with engineered fill soils below the proposed culvert and embankment footprints.

6.2.1 Geotechnical Resistance

6.2.1.1 Closed Bottom Box Culvert

Based on Table 1 and the subsurface conditions, a closed box culvert is a preferred foundation option from a foundations perspective. The approximate invert elevation, recommended level of subexcavation, and the founding soil type for a box culvert is presented below.

Approximate Culvert Station	Relevant Boreholes	Approximate Trail Elevation (m)	Recommended Subexcavation Level for Proposed Culvert (m)	Founding Soils
26+410	06-13, 06-14, 10-1, 10-2, 10-3, 10-8	268.6	266.0 to 266.5	Stiff to Very Stiff Clayey Silt to Silty Clay or Compact Silty Sand



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The above recommended subexcavation level indicates the estimated target elevation required to reach the appropriate founding subgrade soil at about elevation 266.5 metres. The actual founding level will also depend on frost protection requirements and the depth of the granular bedding and/or engineered fill required under the culvert (see Section 6.6.3). If a precast culvert is pursued, then construction of this culvert should be in accordance with OPSS 422. If a cast in place structure is pursued, the construction of this culvert should be in accordance with OPS 902.

Assuming the founding levels noted above, the factored geotechnical resistance at Ultimate Limit States (ULS) and the unfactored geotechnical resistance at Serviceability Limit States (SLS) to be used for detailed design of the box culvert is given below.

Approximate Culvert Station	Foundation System	Total Proposed Culvert Width (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Resistance at SLS (kPa) for 25 mm Settlement
26+410	Closed Bottom Box Culvert	5.5	300	200

The geotechnical resistance values assume the culvert is founded on a granular bedding of 300 mm of Granular "A" approved engineering fill over on undisturbed competent native soils consisting of either stiff to very stiff clayey silt to silty clay or compact silty sand which for design may be assumed to extend to the subexcavation levels indicated above. It should be noted that, depending on the water levels at the time of construction, dewatering may be required to limit the potential for loosening, and/or disturbance of the founding soils at the founding level. It is recommended that the water levels be checked before excavation, i.e., in the piezometers installed in boreholes 06-13, 06-14 and 10-2 to determine whether the groundwater levels are at or above elevation 266.5 metres.

A construction joint should be provided between the wing walls and the box culvert to account for possible differential settlement.

6.2.1.2 Open Footing Culvert

Based on Table 1 and the subsurface conditions, an open footing culvert is considered as an alternative to the closed box culvert, but is also an acceptable choice of a foundation system. The approximate trail bed elevation, recommended footing founding level (i.e., depth of subexcavation), and the founding soil type at the footing level for the proposed open footing culvert replacement / extension are presented below.

Approximate Culvert Station	Relevant Boreholes	Approximate Trail Surface Elevation (m)	Recommended Culvert Footing Founding Elevation (m)	Founding Soils
26+410	06-13, 06-14, 10-1, 10-2, 10-3, 10-8	268.6	266.5	Stiff to Very Stiff Clayey Silt to Silty Clay or Compact Silty Sand



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The above recommended founding level indicates the estimated target elevation required to reach the appropriate founding subgrade soil(s). The actual founding level will depend on the final location of the culvert, the design invert level, frost protection requirements and onsite conditions.

Assuming the founding level noted above, the factored geotechnical resistance at Ultimate Limit States (ULS) and the unfactored geotechnical resistance at Serviceability Limit States (SLS) to be used for detailed design of the open footing culvert is given below.

Approximate Culvert Station	Foundation System	Footing Widths (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Resistance at SLS (kPa) for 25 mm Settlement
26+410	Open Footing Culvert	0.8	215	150
		1.2	240	170
		1.6	275	190
		2.0	280	200

The geotechnical resistance values assume the culvert is founded on the undisturbed competent native soils consisting of either stiff to very stiff clayey silt to silty clay or compact silty sand. It should be noted that, depending on design founding elevation and the water levels at the time of construction, dewatering may be required to limit the potential for loosening, and/or disturbance of the founding soils at the founding level. It is recommended that the water levels be checked before excavation, i.e., in the piezometers installed in boreholes 06-13, 06-14 and 10-2 to determine whether the groundwater levels are at or above elevation 266.5 metres. As a result, excavation for the construction of the open footings should be in accordance with OPSS 902.

6.2.1.3 Head and Wing Wall Foundations

The culvert may require head walls at either end and/or wing walls that extend outward from the head wall. The SLS bearing resistance expressed as a gross bearing resistance (i.e., not net bearing pressure) and the gross factored bearing resistance at ultimate limit states (ULS) are presented in the following table. For design using these gross bearing resistances, the entire weight of the soil and footings overlying the subgrade should be included in the calculation of the loads on the founding soils. In addition, the unit weights should be the full bulk unit weights (i.e., not the net unit weights relative to the excavated material nor the buoyant unit weights).

Approximate Culvert Station	Recommended Culvert Footing Founding Elevation (m)	Footing Widths (m)	Gross Factored Bearing Resistance at ULS (kPa)	Gross Bearing Resistance at SLS (kPa) for 25 mm Settlement
26+410	266.5	1.5	255	180
		2.0	275	200
		2.5	275	200



The Ultimate Limit States (ULS) gross factored bearing resistance presented above should be adjusted to consider the load eccentricity and inclination in accordance with Section 6.7.2 of the CHBDC:

$$q_r = q_N \cdot s_{cq} \cdot i_{cq}$$

Where:

- q_r = Design ULS factored bearing resistance, kilopascals;
- q_N = ULS factored bearing resistance from the above table;
- s_{cq} = Shape factor = $1 + 0.19 \cdot B'/L'$;
- B' = Effective width of footing, metres – see CHBDC Figure 6.7.2;
- L' = Effective length of footing, metres – see CHBDC Figure 6.7.2;
- i_{cq} = Inclination factor = $(1 - \delta^*/90^\circ)$; and,
- δ^* = Angle of resultant force with respect to vertical, degrees.

6.2.1.4 Temporary Wing Wall Foundations

Based on conversations with the designers the extension of the culvert to the east will have a temporary wing wall in the future widening section. We understand that a gabion wall is being considered as one option to treat the temporary east end of the culvert. A gabion wall system typically consists of stacking of stone filled baskets. This wall system is considered a gravity wall, which gathers its stability from the weight of the retaining wall and from the enlarged base. This wall system does not have a separate foundation, but does require a suitable bearing surface. Typically the first course of baskets needs to be embedded into the foundation soils. Therefore, the first course of baskets should be placed no higher than Elevation 267.5 m. At this foundation level the existing native soils could contain organic materials and are considered unsuitable for foundation soils of this wall system, therefore these native organic soils will need to be subexcavated and replaced with the same bedding materials used for the culvert, which are described in section 6.6.3 of this report.

For design of this gabion wall, the factored geotechnical resistance at ULS may be taken as 250 kPa, assuming that the armour stone wall has a minimum base width of 2.0 m. The geotechnical resistance at SLS, for 25 mm of settlement, may be taken as 200 kPa.

The design and construction of the gabion wall adjacent to the culvert extension area should incorporate placement of a suitable non-woven geotextile fabric immediately below and behind the gabions, to minimize the potential for migration of fine soil particles into the voids between the stones and will result in a loss of ground between the wall as well as maintain a free draining wall system. The backfill on the wall should consist of a Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' Type II, but with less than 5 percent passing the No. 200 sieve, should be used as backfill behind the walls. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 and 3121.150.

The global stability of the gabion wall up to 2.5 m in height has been assessed using the commercially available program SLOPE/W (Version 7.17), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. The ranges of parameters used for the foundation soils in the analyses are summarized below.



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Soil Type	Bulk Density, γ (kN/m^3)	Apparent Cohesion, kPa	Internal Friction Angle, ϕ'
New Roadway Fill	22	0	32°
New Retaining Wall Backfill	21	0	32°
Native Clayey Silt to Silty Clay	19	100	0°
Native Till	21	0	34°

The results of the global stability analyses indicate that a factor of safety of 1.5 or greater is achieved for a gabion wall up to 2.5 m in height with a base of 2.0 m wide at this culvert location. The design of the armour stone wall should be designed by the supplier and checked for internal stability.

RSS walls are also being considered for the temporary wing walls at the east end of the culvert. The RSS retaining walls are to be designed for medium performance and appearance in accordance with MTO Special Provision (SP) 599S22 and the Non-Standard Special Provision for the design and construction of RSS walls dated September 2005.

In conventional RSS wall construction, the retained soil mass is constructed using Granular A material that is placed and compacted in lifts with reinforcing strips placed at regular intervals within the soil mass. A typical RSS wall has a front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall. This footing, and the reinforced soil mass, should be founded below any topsoil, loose fill or unsuitable native soils. Based on the borehole 10-8 advanced on the east end of the culvert structure, the facing footings would be constructed over the compact silty sand at about elevation 266. The upper 0.4 m in borehole 10-8 contained organic matter which on inspection may have to be sub-excavated and replaced with Granular A or Granular B Type II if below the water level existing at that time.

It has been assumed that each RSS wall will act as a unit and utilize the full width of the reinforced soil mass, which has been taken to be 0.8 times the height of the wall, with a facing footing width of at least 0.5 m. With the footing founded on a Granular pad of 0.3 m minimum thickness over the compact silty sand the design can be based on a factored geotechnical resistance at ULS of 250 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 200 kPa. The recommended elevation for the bottom of the footings is 266.3 which should be at or above the groundwater level in the summer period.

No grade raises and only a minor change in embankment geometry have been proposed over the RSS walls. Therefore, it is anticipated that the settlement performance for the RSS walls and facing panels that are founded on the footings following subexcavation to Elevation 266 and backfilling with Granular fill will be within 25 mm. Some minor differential settlements along the retaining wall segments should be expected, although the differential settlements along the wall are expected to be less than 1 per cent of the wall length; therefore, precast concrete panels or block facings could be used.

The resistance to lateral forces / sliding resistance between the compacted fill of the reinforced soil mass (assumed to be Granular A) and the compact silt sand subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \phi'$, between the compacted granular fills of the RSS wall and the properly prepared subgrade may be taken as 0.55. This represents an unfactored value; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.



The internal stability of the mechanically-reinforced soil walls should be verified by the RSS supplier/designer. The external (global) stability of a typical RSS retaining wall height of maximum 4 m has been assessed assuming a reinforcement length of 0.8 times the wall height. The walls were found to be stable based on achieving minimum Factors of Safety of over 1.5 for sliding and over 2.0 for eccentricity/overturning and bearing under static conditions.

6.2.2 Resistance to Lateral Loads

Resistance to lateral forces (i.e., sliding resistance) between the base of the concrete culvert foundation and the undisturbed native materials should be calculated in accordance with Section 6.7.5 of the CHBDC. For the concrete box option, assuming the culvert is precast concrete and is placed on compacted granular bedding, a coefficient of friction value ($\tan \delta$) of 0.55 can be used for design. The coefficient of friction value for this option can be increased to 0.58 if cast-in-place concrete is placed on compacted granular bedding.

For the open footing culvert option, assuming the footings are cast-in-place and founded on undisturbed clayey silt to silty clay, a coefficient of friction value ($\tan \delta$) of 0.45 can be used for design. In accordance with the CHBDC, a factor of 0.8 is to be applied to the coefficient of friction value when calculating the horizontal resistance.

6.2.3 Frost Protection

The design frost penetration depth in the area of the proposed culvert is 1.6 m. All shallow foundations should be provided with a minimum of 1.6 m of soil cover or equivalent thermal insulation for frost protection.

6.3 Lateral Earth Pressures

The lateral earth pressures acting on the new structure and any associated foundation walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls.

The following recommendations are made concerning the design of the walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' Type II, but with less than 5 percent passing the No. 200 sieve, should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 803.010.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the culvert / wall stem, in accordance with CHBDC Figure 6.6. Compaction equipment should be used in accordance with OPSS 501. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.6 m behind the back of the wall stem (Case (a) in Figure C6.20 of the Commentary to the CHBDC) or within the wedge-shaped zone



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defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case (b) in Figure C6.20 of the Commentary to the CHBDC).

- For Case (a), the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used, assuming the use of Select Subgrade Material (SSM) for the new portions of the approach embankments.

Material	SSM
Soil unit weight	20 kN/m ³
Coefficients of static lateral earth pressure	
Active, K_a	0.33
At rest, K_o	0.50

- For Case (b), the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

Material	Granular 'A'	Granular 'B' (Type II)
Soil unit weight	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43

- If the culvert structure allows lateral yielding of the culvert walls, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (such as typically the case for a rigid concrete box culvert), at-rest earth pressures should be assumed for geotechnical design.
- Seismic loading will result in increased lateral earth pressures acting on the culvert wall and any retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to Table A3.1.1 of the CHBDC, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio (A) for Peterborough is 0.05. Based on experience, for the overburden soils at the site and embankment heights of up to about 2.5 m, a 10 to 20 percent amplification of the ground motion may occur, resulting in an increase in the ground surface acceleration from 0.05g to between 0.055g and 0.06g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.06.
- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e., $k_h = 0.03$). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e., $k_h = 0.09$). The seismic active earth pressure coefficients are also dependent on the vertical component of the earthquake



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acceleration, k_v . Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2.3k_h$, $k_v = 0$, and $k_v = -2/3$.

- The following seismic active earth pressure coefficients (K_{AE}) for the two cases (Case (a) and Case (b)) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

Seismic Active Pressure Coefficients, K_{AE}

	Case I (SSM)	Case II	
		Granular A	Granular B Type II
Yielding wall	0.32	0.26	0.26
Non-yielding wall	0.37	0.30	0.30

Note: These CHBDC seismic K_{AE} values include the effect of wall friction ($\delta=\phi'/2$) and are less than the static values of K_a and K_o (i.e., yielding and non-yielding wall) reported above for the very low zonal acceleration ratio for this site.

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.06. This corresponds to displacements of up to 15 mm at this site.
- The earthquake-induced dynamic active lateral pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$p = K \gamma' d + (K_{AE} - K) \gamma' H$$

Where:

- p is the total (static plus seismic) pressure distribution (kPa);
- K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
- K_{AE} is the seismic active earth pressure coefficient;
- γ' is the effective unit weight of the soil (kN/m^3) (given above for fill materials);
- d is the depth below the top of the wall (m); and,
- H is the height of the wall above the toe (m).



6.4 Settlement

It is anticipated that the elevation of the floor of the new culvert structure elevation will match the existing recreational trail ground surface which is at about Elevation 268.6 m. Based on the preliminary general arrangement drawing provided by D.M. Wills, it is understood that the proposed re-aligned Highway 7 embankment will be up to about 5.9 m high (approximate Elevation 274.5 m) with assumed 2.0H:1V side-slopes.

At the proposed culvert location, which will allow for pedestrian passage below Highway 7 along the recreational trail, some settlement is anticipated due to the loading imposed on the foundation subsoils from the new embankment loading. The proposed embankment and culvert footprint should be stripped of topsoil, peat, unsuitable fills and any localized areas of very soft to soft clayey soils and soils containing significant organics. It is anticipated that stripping and subexcavation will be more significant in the eastern portions of the culvert (i.e., up to about 2.5 m). The subexcavation should be replaced with approved engineered fill, the total thickness of the new embankment fill is anticipated to be up to about 8.4 m. The total net loading on the foundation soils (after stripping, backfilling and embankment fill construction) adjacent to the culvert location from the proposed embankment is estimated to be about 175 kPa, assuming the embankment material has a unit weight of about 21 kN/m³.

A settlement analysis was performed using the commercially available program SETTLE 3D (Version 2.011) produced by Rocscience Inc. The soil parameters used for the analysis were based on the laboratory and in situ test data collected during the current and preliminary investigations.

The immediate compression of the lower silty sand and glacial tills below the clayey silt deposit were modelled using elastic moduli (see Chapter 6, "Commentary to the CHBDC, 2001"). The time dependant, consolidation settlement of the stiff to very stiff silty clay / clayey silt deposit was modelled using parameters derived from correlations with laboratory test data (i.e., moisture content and Atterberg Limits test results). The parameters were consistent with the results from laboratory consolidation tests performed on samples of similar silty clay / clayey silt deposits from nearby sites as part of the overall Highway 7 foundations investigation. Based on the subsoil information collected, the stiff to very stiff silty clay / clayey silt deposit is considered to be over-consolidated and the estimated net loading due to the embankment is not anticipated to exceed the preconsolidation pressure of the silty clay / clayey silt deposit.

The following summarizes the simplified stratigraphy, unit weights and deformation parameters for the soil units (see Chapter 6, "Commentary to the CHBDC, 2001") employed in the settlement analysis:

Soil	Thickness (m)	Bulk Unit Weight (kN/m ³)	Deformation Properties
Clayey Silt to Silty Clay	1.5	19	See table below
Silty Sand	2	20	$E_s' = 10$ MPa
Till	5	22	$E_s' = 150$ MPa

The following summarizes the deformation parameters for the silty clay / clayey silt deposit employed in the settlement analysis:



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Soil	Modulus of Elasticity E_s'	Initial Void Ratio e_o	Compression Index C_c	Recompression Index C_r	Apparent Preconsolidation Pressure σ'_p (kPa)	Coefficient of Consolidation C_v (cm ² /sec)
Silty Clay/ Clayey Silt	60 MPa	1.0	0.35	0.02	350	2.2×10^{-3}

Although the net loading due to the placement of the new culvert itself on the foundation soils is expected to be relatively small, the removal of the existing fill soils below and beyond the new culvert footprint will help to reduce the potential for differential settlement within the new culvert footprint.

The predicted maximum total settlement of the foundation soils at the culvert location is estimated to be about 45 mm due to the loading imposed by the new embankment fill. The total is estimated to be comprised of about 20 mm of immediate settlement due to compression of the cohesionless soils and about 25 mm of time dependent settlement of the cohesive soil layer.

Based on an estimated coefficient of consolidation (c_v) and assuming two-way drainage of the approximately 1.5 m thick clayey silt deposit, it is estimated that about 90% of the consolidation settlement will be complete within about 1 month.

Settlement of the new granular embankment fill itself is expected to occur rapidly (i.e., during or shortly after construction) and be less than 25 mm if placed and compacted properly.

Due to the varying thickness of the underlying silty clay / clayey silt, preloading after subgrade preparation (i.e., subexcavation and backfill) may be considered to reduce the amount of differential settlement and consolidation settlement after the culvert and roadway embankment have been constructed. The approach embankments adjacent to the culvert location should be constructed and allowed to settle at the proposed culvert location for at least 2 months prior to culvert installation. A camber could be incorporated into the design of the culvert to manage the expected settlements after placement of fill above the culvert and to prevent collection of any surface water within the culvert. Other options (other than preloading) to mitigate the impacts of consolidation settlement could include surcharging, use of light weight fill, full subexcavation of the silty clay / clayey silt deposit. Given the relatively high water level (i.e., dewatering efforts) and potential for disturbance to the water-bearing silty sand layer located directly below the silty clay / clayey silt deposit, full subexcavation of the silty clay / clayey silt deposit is not recommended. The use of light weight fill is likely too expensive and not necessary. Surcharging would be a cost effective option. Based on the preliminary general arrangement drawing, the proposed Highway 7 road surface will be located about 1.5 m above the top of the culvert; as a result, consideration could be given to preloading the culvert footprint with about 2 m of soil (i.e., above the proposed invert level) during the embankment preload period to further reduce the potential for settlements at the culvert location after final construction. Consideration should be given to delaying the final paving above the culvert to allow most of the consolidation settlement to take place prior to the final lift of pavement structure.

6.5 Stability

Static slope stability analyses for the approximate 5.8 m high embankment configuration (assuming 2H:1V side-slopes) localized at the proposed culvert locations were carried out using the following parameters, derived



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from field and laboratory testing and accepted correlations, using the commercially available program SLOPE/W, produced by Geo-Slope International Ltd.

Soil	Bulk Unit Weight	Effective Friction Angle	Undrained Shear Strength
Proposed Embankment Fill, Engineered Backfill and Bedding Materials	21 kN/m ³	32°	–
Existing Granular Fill	20 kN/m ³	32°	–
Silty Clay / Clayey Silt	19 kN/m ³	28°	100 kPa
Silty Sand	20 kN/m ³	30°	–
Glacial Till	22 kN/m ³	43°	–

Both undrained and effective stress analyses were carried out assuming appropriate subgrade preparation (i.e., stripping of topsoil, peat, the very soft to soft clayey silt soils containing organics or any loosened soils) and proper placement and compaction of embankment engineered fill soils. Based on the results of the analyses, a factor of safety of greater than 1.3 was calculated against global slope instability. The global stability should be checked after the detail design investigation is complete.

6.6 Considerations for Culvert Construction

6.6.1 Subgrade Preparation and Excavation

Prior to the placement of any foundations, engineered fill, bedding, or new embankment construction, all surficial peat, topsoil, organics, and softened or loosened soils should be stripped from below the proposed culvert and embankment widening footprint and wasted/reused for landscaping. All subgrade soils should be inspected and proofrolled prior to placement of foundations or engineered fill and embankment fill should be placed in accordance with OPSS 902.

Based on the boreholes from the previous and current investigations, it is anticipated that excavations up to about 2.5 m below the existing ground surface will be required at the proposed culvert location to remove the sand fill, topsoil, peat, very soft to soft clayey silt, and any soils containing excessive organics to expose the native stiff to very stiff silty clay / clayey silt deposit. Based on the peat and soils containing organics encountered below the fill in boreholes 10-8 and 06-13, it is likely that the surficial organic soils were not stripped prior to construction of the railway embankment (i.e., the existing recreational trail embankment).

For the culvert construction, the subexcavated area and engineered fill placement should extend from about 2 m beyond the outside edge of the proposed culvert outward and downward at 1 horizontal to 1 vertical (1H:1V). Engineered fill (i.e., backfill) should be placed and compacted as described in Section 6.7.3.

Excavation work should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities, and follow the guidelines outlined in OPSS 902.



It is noted that the soils in which the excavations will be formed are susceptible to disturbance due to groundwater seepage and construction traffic. Groundwater and surface water control will be required.

6.6.2 Groundwater and Surface Water Control

Some water seepage into the subexcavation could occur as a result of surface water and subexcavation extending below the groundwater table. It is expected that the quantity of seepage could be handled using a system of sumps and pumps. The severity of the groundwater conditions is dependent upon many factors including the season during which construction occurs. In general, pumping using properly filtered sumps, and/or filtered drains placed along the base of the excavation should provide sufficient groundwater control during foundation works. Ditches to divert perched water and/or storm water flows around the construction area will also be required to help permit construction and compaction of backfill / engineered fill in the dry. The silty clay / clayey silt that is exposed at the native subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. Surface water should be directed away from the excavation area, to prevent ponding of water that could result in disturbance and weakening of the subgrade soils. For this reason, backfill should be placed and compacted immediately after subexcavation to protect the subgrade soils. A Non-Standard Special Provision (NSSP) for dewatering and groundwater control is within Appendix C of this report.

Special care should be taken to not over-excavate the clayey silt deposit which would result in exposing the underlying water-bearing silty sand layer. If the silty sand layer is exposed, more extensive dewatering efforts will be required in order to lower the water table to a minimum of 0.5 m below the base of the excavation level in order to limit the potential for disturbance of the native sandy soils and allow placement and compaction of engineered fill in the dry.

6.6.3 Bedding and Backfill / Embankment Construction

For the concrete culvert being considered, the silty clay / clayey silt soils located within the founding elevations provided in Section 6.2.1 are considered suitable for the support of the bedding for the proposed culvert replacement. Stripping of any existing fills, topsoil, peat, very loose and highly organic soils will be required.

For the box culvert option, the bedding, levelling pad and backfill requirements for the culvert replacement should be in accordance with OPSS 422 for precast concrete rigid frame culverts. The box culvert should be provided with at least 300 millimetres of OPSS Granular 'A', or Granular 'B' Type II (OPSS 1010) material if in wet ground conditions, for bedding purposes and partial frost protection. If Granular 'B' Type II is used, the placement of a geotextile filter between the bottom of the bedding and native soils is required. The bedding should be placed in lifts not exceeding 150 mm in loose thickness, and compacted to at least 95 per cent of the Standard Proctor maximum dry density. In addition, for closed box culverts, a minimum 75 mm thick uncompacted levelling pad of Granular 'A' or fine aggregate (OPSS 1002) should be provided.

Frost treatment for the culvert structure should follow the guidelines provided in OPSD 803.010 for box and open footing culverts. In order to reduce the potential for frost damage to the culvert (i.e., differential heave and settlement), the combined thickness of backfill and bedding should be at least 1.6 m (i.e., equal to the depth of frost penetration).

As previously mentioned, subexcavation of unsuitable soils of up to about 2.5 m locally is anticipated. As a result, the subgrade will require placement of engineered fill (i.e., backfill) to raise grades to the bedding level. This can be achieved by placing additional lifts of the properly placed and compacted bedding material



(i.e., OPSS Granular 'A' or Granular 'B' Type II if in wet ground conditions). In order to reduce the potential for frost damage to the culvert (i.e., differential heave and settlement), the combined thickness of backfill and bedding should be at least 1.6 m (i.e., equal to the depth of frost penetration) and the subgrade below the backfill and bedding soils should promote drainage away from the culvert footprint.

Placement of granular fill for the new embankment should consist of OPSS Select Subgrade Material (SSM) and be carried out in accordance with Special Provision SP206S03 and compacted in accordance with OPSS 501.

Depending on the culvert base thickness and actual subexcavation depth during construction, the subgrade may require placement of engineered fill to raise grades to the bedding level. This can be achieved by placing additional lifts of the properly placed and compacted bedding material (i.e., Granular 'A' or Granular 'B' Type II).

Compaction equipment should be used in accordance with OPSS 902. Inspection and field density testing should be carried out by qualified geotechnical personnel during all engineered fill placement operations to ensure that appropriate materials are used, and that adequate levels of compaction have been achieved.

6.6.4 Corrosion Potential

Two samples of soil from boreholes 10-01 and 10-03 were submitted to Exova Accutest Laboratories Ltd. for chemical analysis related to potential corrosion of exposed buried steel and concrete elements (corrosion and sulphate attack). The results of this testing are provided in Appendix A. The results indicate that concrete made with Type GU Portland cement should be acceptable for substructures. The result of the pH, chlorides and resistivity testing indicate a mild potential for corrosion of exposed ferrous metal. These results should be considered when selecting protective coatings for any buried steel objects.




FOUNDATION INVESTIGATION AND DESIGN REPORT RECREATIONAL TRAIL CULVERT


7.0 CLOSURE

This report was prepared by Mr. Bruce Goddard, P.Eng., a senior geotechnical engineer. Mr. Fintan J. Heffernan, P.Eng., and a Designated MTO Contact with Golder, reviewed the technical aspects and conducted a quality control review of the report.

GOLDER ASSOCIATES LTD.


Bruce D. Goddard, P.Eng.
Senior Geotechnical Engineer




Fintan J. Heffernan, P.Eng.
Designated MTO Contact



BDG/FJH/bg

n:\active\2010\1121 - geotechnical\10-1121-0007 mto hwy 7 peterborough\report 4 - former cnr 26-035\10-1121-0007-4 v2 final mto foundation report recreational trail crossing june 2012.docx



FOUNDATION INVESTIGATION AND DESIGN REPORT RECREATIONAL TRAIL CULVERT

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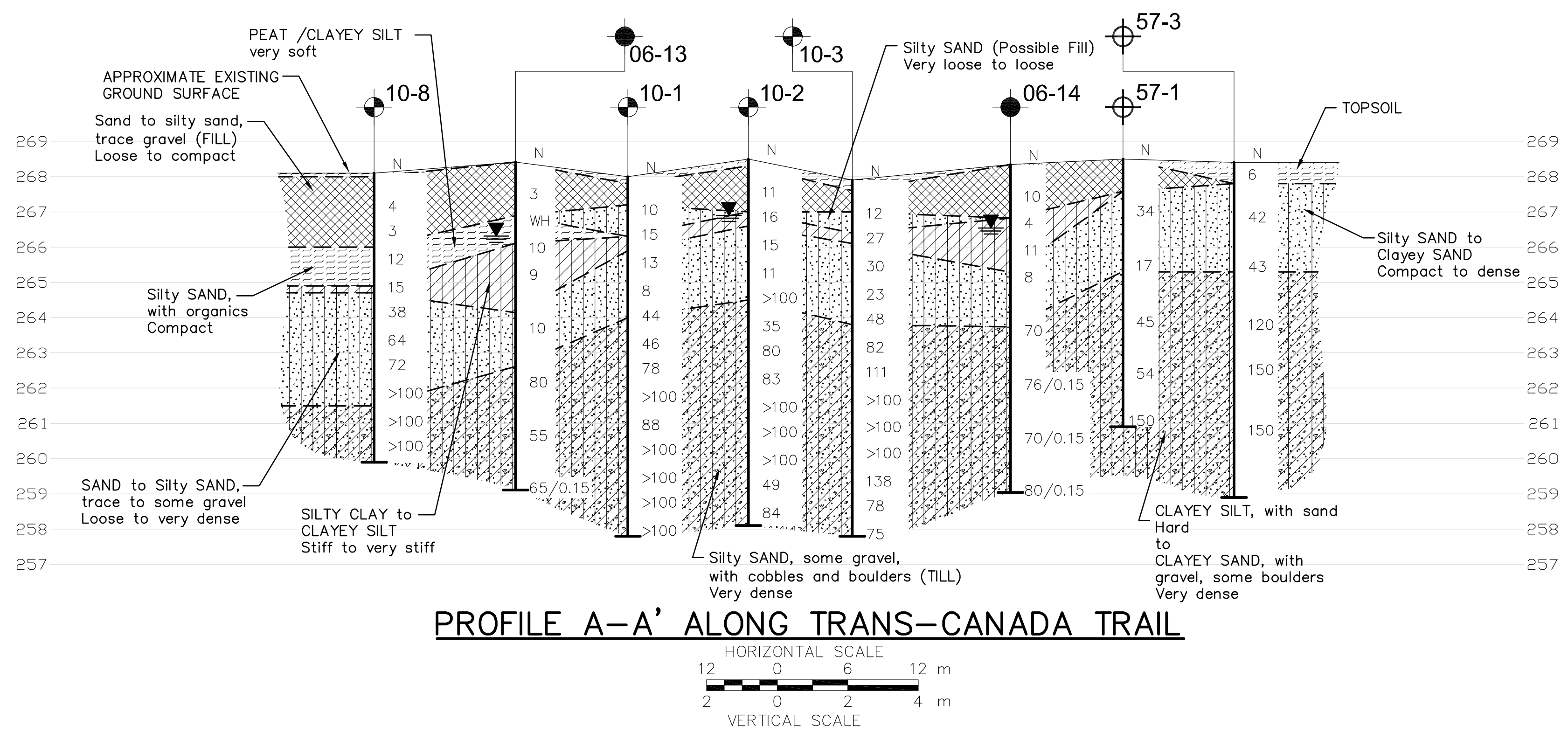
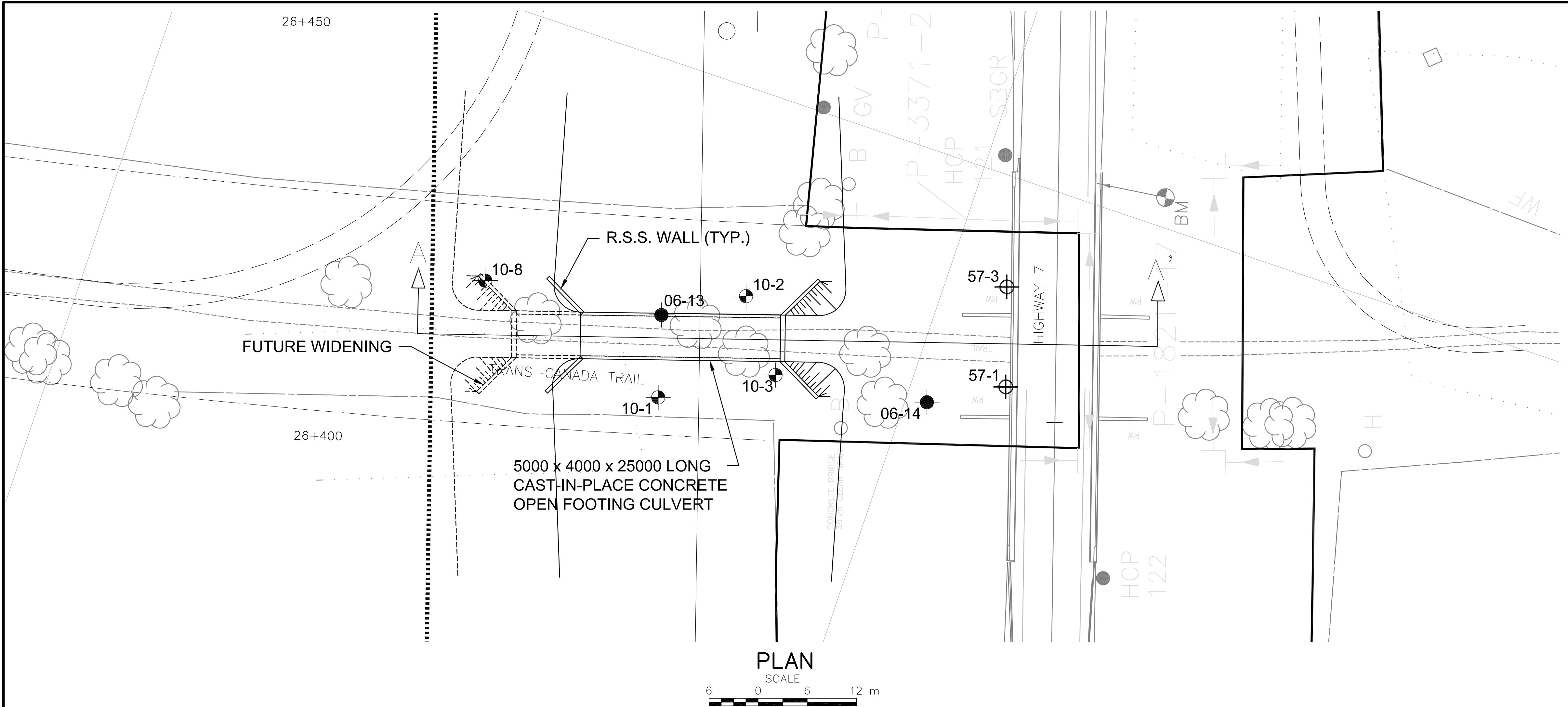


FOUNDATION INVESTIGATION AND DESIGN REPORT RECREATIONAL TRAIL CULVERT

Table 1
Evaluation of Culvert Foundation Alternatives
Highway 7 Recreational Trail Culvert (Site No. 26-35)
WP 245-00-01

Option	Rank/ Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Box Culvert	1	<ul style="list-style-type: none"> ■ Routine excavation and construction procedure ■ Shallow subexcavation depth ■ Wide base reduces bearing pressure; thus, reducing potential for total and differential settlement 	<ul style="list-style-type: none"> ■ Depending on the groundwater elevation at the time of construction, dewatering may be needed during subexcavation and backfill ■ Some minor post-construction settlement is anticipated due to embankment loading 	<ul style="list-style-type: none"> ■ Similar costs to Option No. 2 to due to quantity of subexcavation and engineered fill 	<ul style="list-style-type: none"> ■ Subexcavation and replacement with non-frost susceptible engineered fill will reduce potential for frost action and differential settlement ■ Culvert will experience some long-term settlement; however, this can be reduced by preloading and surcharging the culvert footprint.
Open Footing Culvert	1	<ul style="list-style-type: none"> ■ Routine excavation and construction procedure ■ Shallow sub-excavation depth ■ Foundations will be located below frost depth. 	<ul style="list-style-type: none"> ■ Depending on the groundwater elevation at the time of construction, dewatering may be required in order to place footings on competent founding soils and place concrete in the dry ■ Some post-construction settlement is anticipated due to embankment loading ■ Reduced geotechnical resistance than wider box culvert 	<ul style="list-style-type: none"> ■ Similar costs to Option No. 1 due to extra care required to maintain competent subgrade and place concrete footings in the dry 	<ul style="list-style-type: none"> ■ Risk of disturbing / loosening subgrade soils during construction due to underlying silty sand layer located below groundwater table ■ Culvert will experience some long-term settlement; however, this can be reduced by preloading and surcharging the culvert footprint
Deep Foundations (i.e., Piles or Caissons)	NP	<ul style="list-style-type: none"> ■ Reduced post-construction settlement of the culvert structure 	<ul style="list-style-type: none"> ■ Differential settlement between culvert and embankments will cause a "hard point" at the road surface 	<ul style="list-style-type: none"> ■ Much higher costs than Option No. 1 and No. 2 	

NP – not feasible or not practical



CONT No.
WP No. 245-00-01

SHEET

Golder Associates

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA

KEY PLAN
SCALE 2 0 2 4 km

LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation by Golder (Geocres No. 31D-428)
- Borehole - Previous Investigation by Racey, MacCallum and Associates (1957)
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Seal
- Piezometer
- WL in piezometer

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
10-1	268.0	4907839.4	390028.7
10-2	268.5	4907824.3	390022.5
10-3	267.9	4907832.2	390016.0
10-8	268.1	4907832.7	390053.2
06-13	268.4	4907829.8	390031.5
06-14	268.3	4907829.5	389997.4
57-1	268.5	4907824.6	389988.9
57-3	268.4	4907813.0	389992.7

REFERENCE

Base plans provided in digital format by D.M. WILLS (Drawing File No. "73-99-00 PR-2 Property Request Part B.dwg", received July 13, 2010).

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 final 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.

NO.	DATE	BY	REVISION

Geocres No. 31D-528

HWY. 7	PROJECT NO. 10-1121-0007	DIST. 43
SUBM'D. BDG	CHKD. BDG	DATE: MAY 2011
DRAWN: JM	CHKD. BDG	APPD. FJH
		DWG. 1

METRIC

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



APPENDIX A

**List of Abbreviations and Symbols
Record of Borehole Sheets and Detailed Laboratory Test Results
Present Investigation**

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	III.	SOIL DESCRIPTION
AS	Auger sample	(a)	Cohesionless Soils
BS	Block sample	Density Index (Relative Density)	N Blows/300 mm <u>Or Blows/ft.</u>
CS	Chunk sample	Very loose	0 to 4
DO	Drive open	Loose	4 to 10
DS	Denison type sample	Compact	10 to 30
FS	Foil sample	Dense	30 to 50
RC	Rock core	Very dense	over 50
SC	Soil core	(b)	Cohesive Soils
ST	Slotted tube	Consistency	C _u or S _u
TO	Thin-walled, open		
TP	Thin-walled, piston		
WS	Wash sample		
DT	Dual Tube 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.) DD- Diamond Drilling		
Dynamic Penetration Resistance; N_d:	The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive Uncased a 50 mm (2 in.) diameter, 60° cone attached to “A” size drill rods for a distance of 300 mm (12 in.).	Kpa	Psf
PH:	Sampler advanced by hydraulic pressure	Very soft	0 to 12
PM:	Sampler advanced by manual pressure	Soft	12 to 25
WH:	Sampler advanced by static weight of hammer	Firm	25 to 50
WR:	Sampler advanced by weight of sampler and rod	Stiff	50 to 100
		Very stiff	100 to 200
		Hard	Over 200
Peizo-Cone Penetration Test (CPT):	An electronic cone penetrometer with a 60° conical tip and a projected end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q _t), porewater pressure (PWP) and friction along a sleeve are recorded Electronically at 25 mm penetration intervals.	IV.	SOIL TESTS
		w	water content
		w _p	plastic limited
		w _l	liquid limit
		C	consolidaiton (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 test (LV-laboratory vane test)
		γ	unit weight

Note:

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

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma'$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1 \sigma_2 \sigma_3$	principal stresses (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 ($D_R = p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is p . Unit weight symbol is γ where $\gamma = pg$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (cont'd.)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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


Notes: 1. $\tau = c' + \sigma' \tan \phi'$
2. Shear strength $= (\text{Compressive strength})/2$

Continued Next Page

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



○ 3% STRAIN AT FAILURE

MIS-MTO 001 1011210007-245-00-01.GPJ GAL-MISS.GDT 06/27/11 JM

PROJECT <u>10-1121-0007</u>		RECORD OF BOREHOLE No 10-1				2 OF 2 METRIC																	
G.W.P. <u>245-00-01</u>		LOCATION <u>N 4907839.4 ; E 390028.7</u>				ORIGINATED BY <u>HEC</u>																	
DIST <u>43</u> HWY <u>7</u>		BOREHOLE TYPE <u>Power Auger, 105mm Diam. Solid Stem</u>				COMPILED BY <u>JM</u>																	
DATUM <u>Geodetic</u>		DATE <u>Sept. 9-10, 2010</u>				CHECKED BY <u>NRL</u>																	
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W			W _L						
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div>																
257.8 10.2	End of Borehole		13	SS	>100																		

MIS-MTO 001 1011210007-245-00-01.GPJ GAL-MISS,GDT 06/27/11 JM

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

PROJECT <u>10-1121-0007</u>		RECORD OF BOREHOLE No 10-2				2 OF 2 METRIC											
G.W.P. <u>245-00-01</u>		LOCATION <u>N 4907824.3 ; E 390022.5</u>				ORIGINATED BY <u>HEC</u>											
DIST <u>43</u> HWY <u>7</u>		BOREHOLE TYPE <u>Power Auger, 105mm Diam. Solid Stem</u>				COMPILED BY <u>JM</u>											
DATUM <u>Geodetic</u>		DATE <u>Sept. 9, 2010</u>				CHECKED BY <u>NRL</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W			W _L
	--- CONTINUED FROM PREVIOUS PAGE ---																
258.1 10.4	End of Borehole Note: 1. Water level in well screen at 2.1 m depth (Elev. 266.4 m) below ground surface on Sept. 24, 2010. 2. Water level in well screen at 2.0 m depth (Elev. 266.5 m) below ground surface on Dec. 13, 2010. 3. Water level in well screen at 1.6 m depth (Elev. 266.9 m) below ground surface on Apr. 26, 2011.		13	SS	84												22 48 22 8

MIS-MTO 001 1011210007-245-00-01.GPJ GAL-MISS,GDT 06/27/11 JM

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



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

MIS-MTO 001 1011210007-245-00-01.GPJ GAL-MISS.GDT 06/27/11 JM

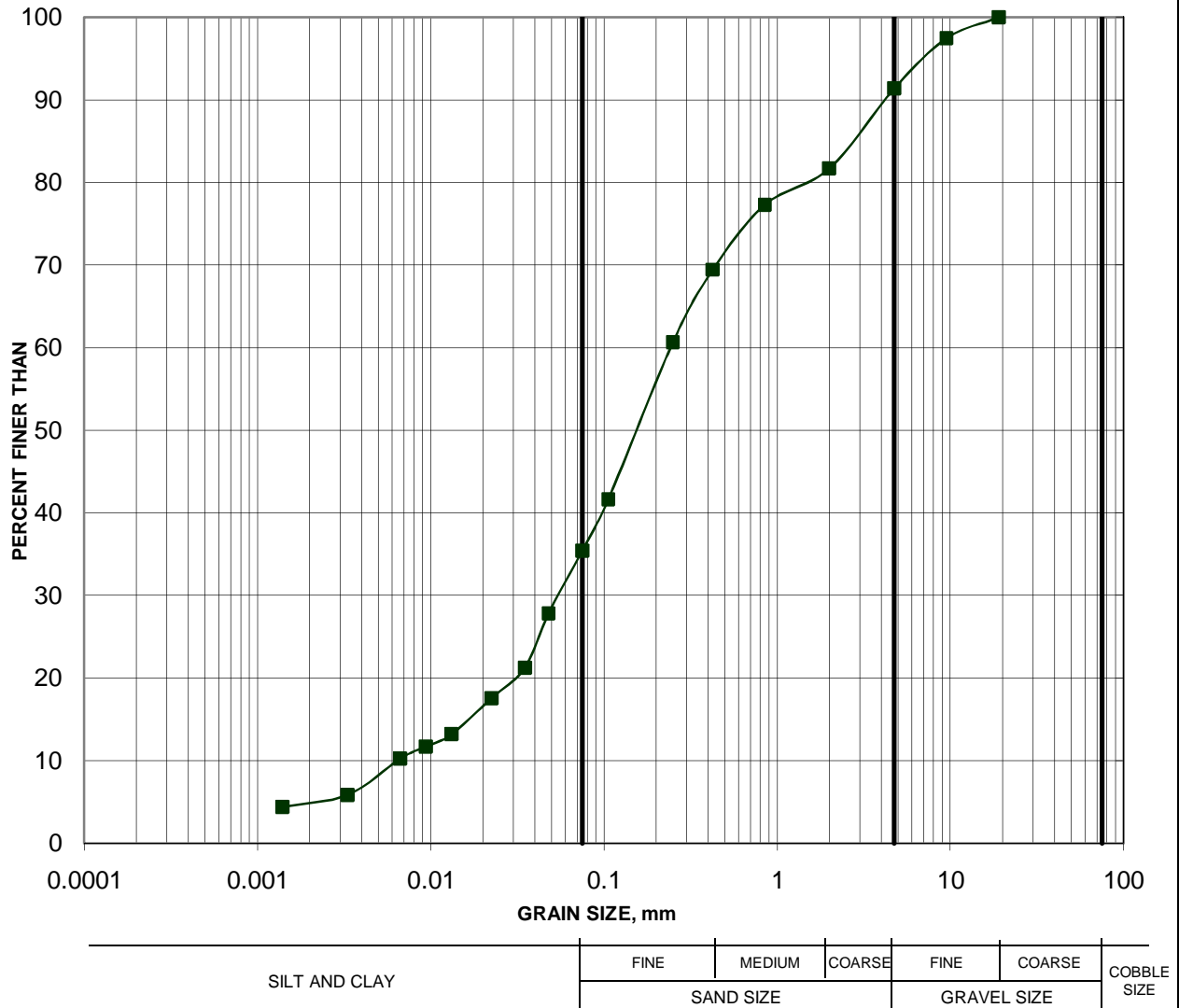
PROJECT 10-1121-0007			RECORD OF BOREHOLE No 10-8			1 OF 1 METRIC												
G.W.P. 245-00-01			LOCATION N 4907832.7 ; E 390053.2			ORIGINATED BY HEC												
DIST 43 HWY 7			BOREHOLE TYPE Power Auger, 105mm Diam. Solid Stem			COMPILED BY JM												
DATUM Geodetic			DATE Sept. 8, 2010			CHECKED BY NRL												
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× REMOULDED	W _p	W	W _L	25 50 75		
268.1	0.0	GROUND SURFACE						268										
	0.1	Sand, with organic matter (TOPSOIL) Dark brown to black Sand, trace gravel (FILL) Grey-brown																
267.2	0.9	Silty sand (FILL) Very loose Light brown to grey-brown Moist to wet		1	SS	4		267										
				2	SS	3												
266.0	2.1	Silty SAND, with organic matter Compact Dark brown to black Wet						266										
265.6	2.5	Silty SAND Compact Grey-brown to brown Wet		3	SS	12												
264.9								265										
264.7	3.4	SILTY CLAY Very stiff Grey-brown Wet		4	SS	15												
		SAND Dense to very dense Brown Wet		5	SS	38		264										
				6	SS	64		263										
262.5	5.6	SAND, trace gravel Very dense Brown Wet		7	SS	72		262										
				8	SS	>100												
261.5	6.6	SAND, some gravel, trace clay and cobbles (TILL) Very dense Grey-brown to grey Wet		9	SS	>100		261										
				10	SS	>100												
259.9	8.2	End of Borehole						260										

MIS-MTO 001 1011210007-245-00-01.GPJ GAL-MISS.GDT 06/27/11 JM

GRAIN SIZE DISTRIBUTION

FIGURE A1

Silty Sand, trace gravel and clay (FILL)

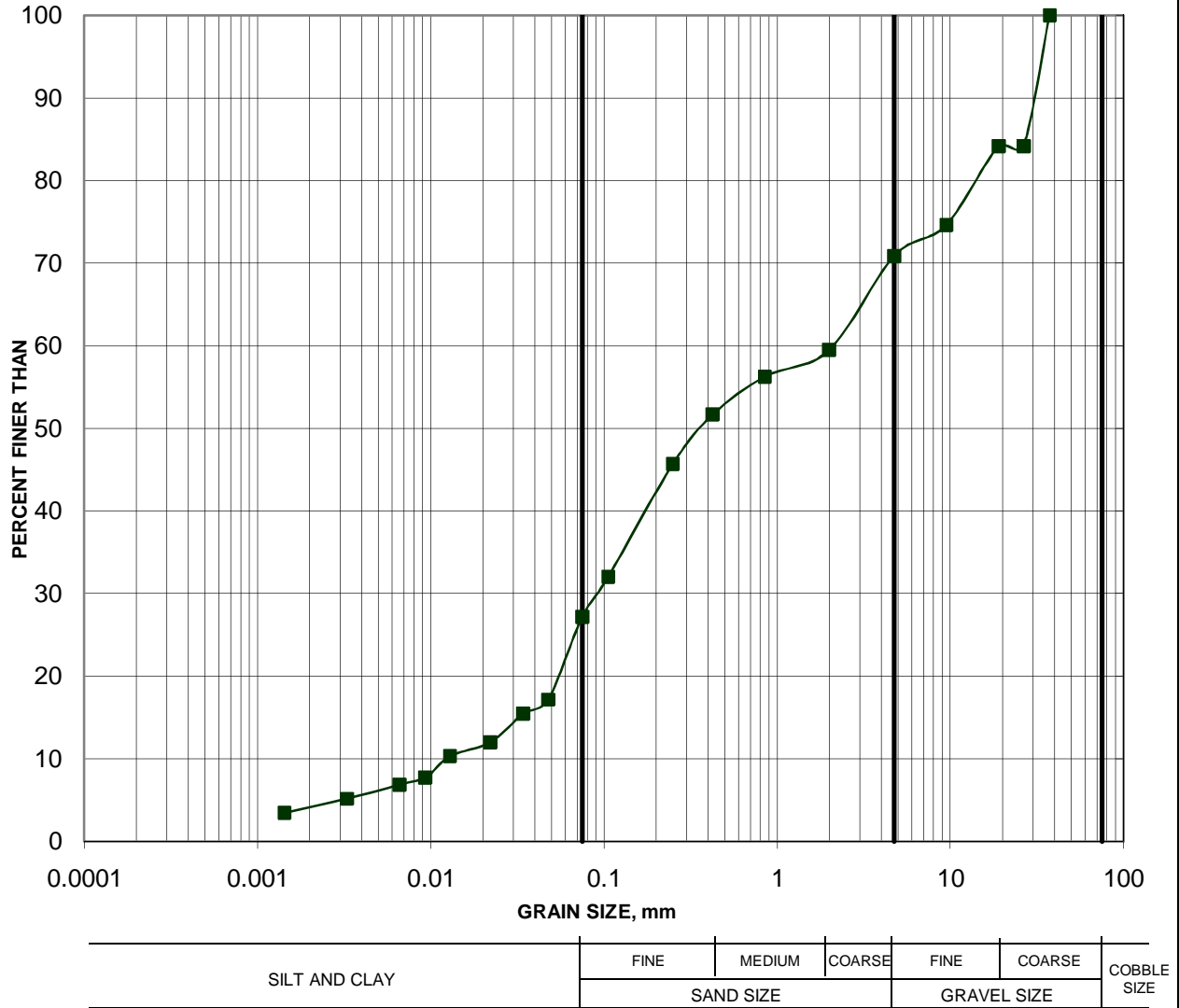


Borehole	Sample	Depth (m)
10-2	1	0.76-1.37

GRAIN SIZE DISTRIBUTION

FIGURE A2

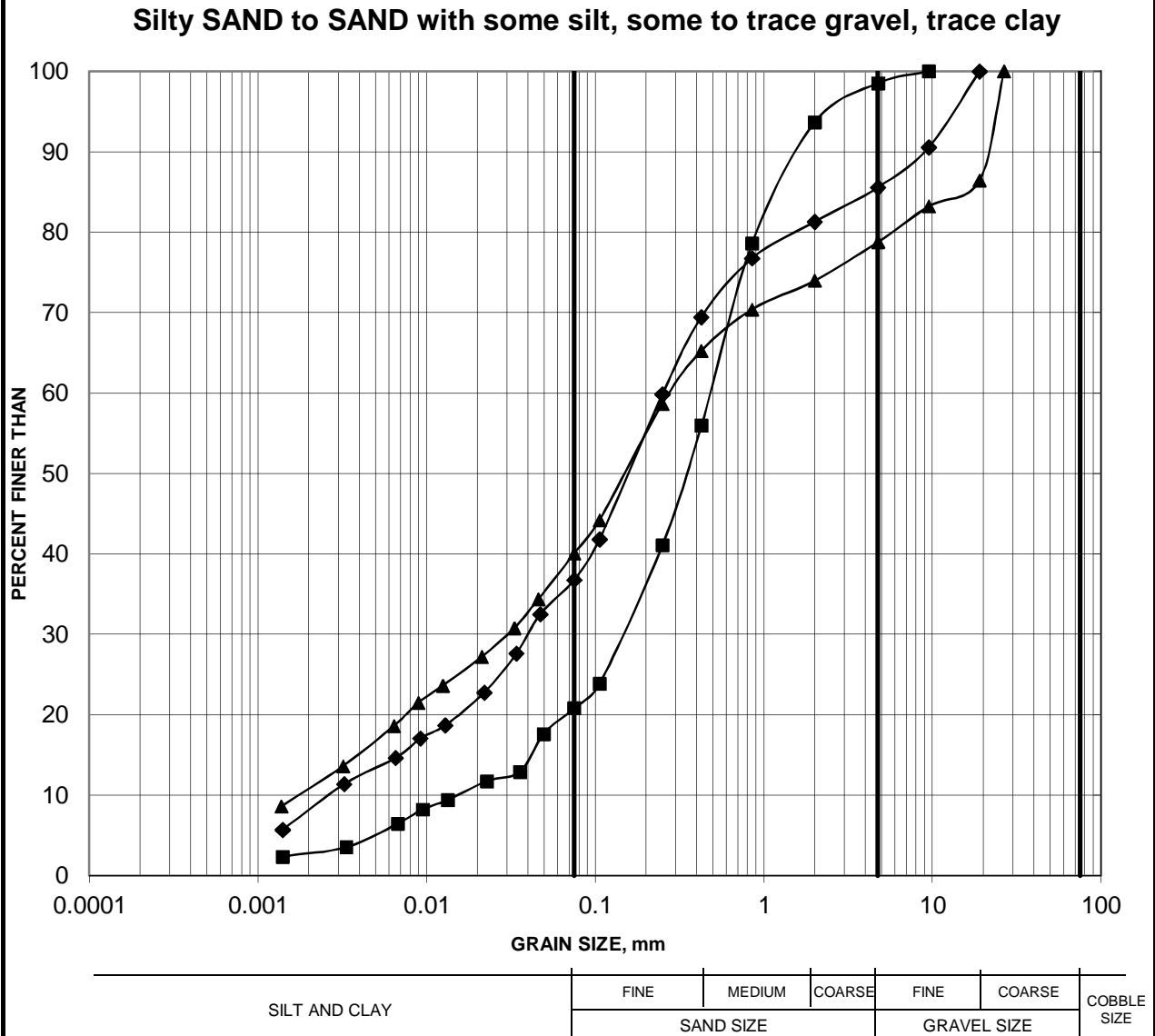
Gravelly Silty SAND, trace clay (Possible Fill)



Borehole	Sample	Depth (m)
10-1	1	0.76-1.37

GRAIN SIZE DISTRIBUTION

FIGURE A3

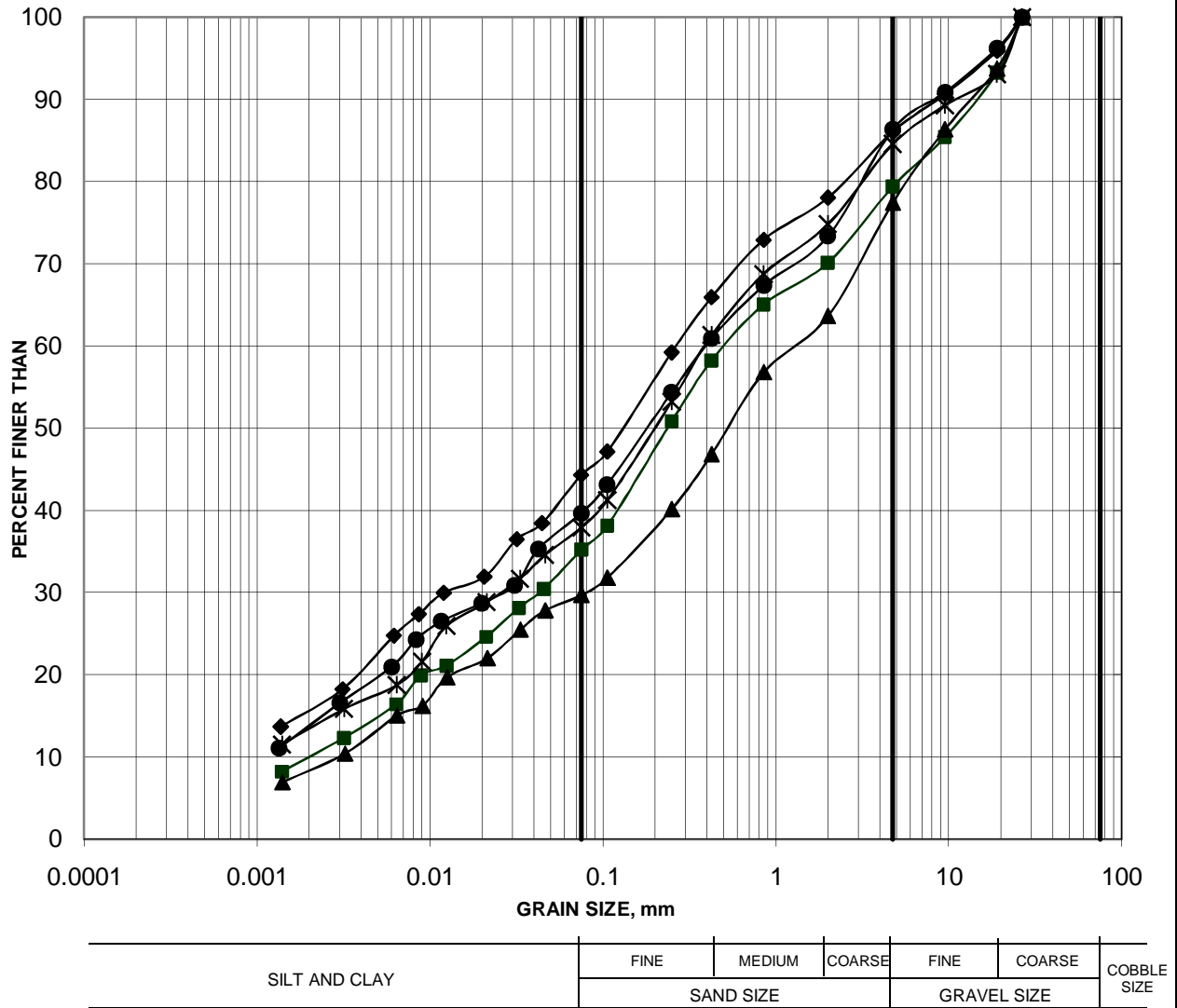


Borehole	Sample	Depth (m)
10-2	2A	1.88-2.13
10-2	4	3.05-3.66
10-3	3	2.29-2.90

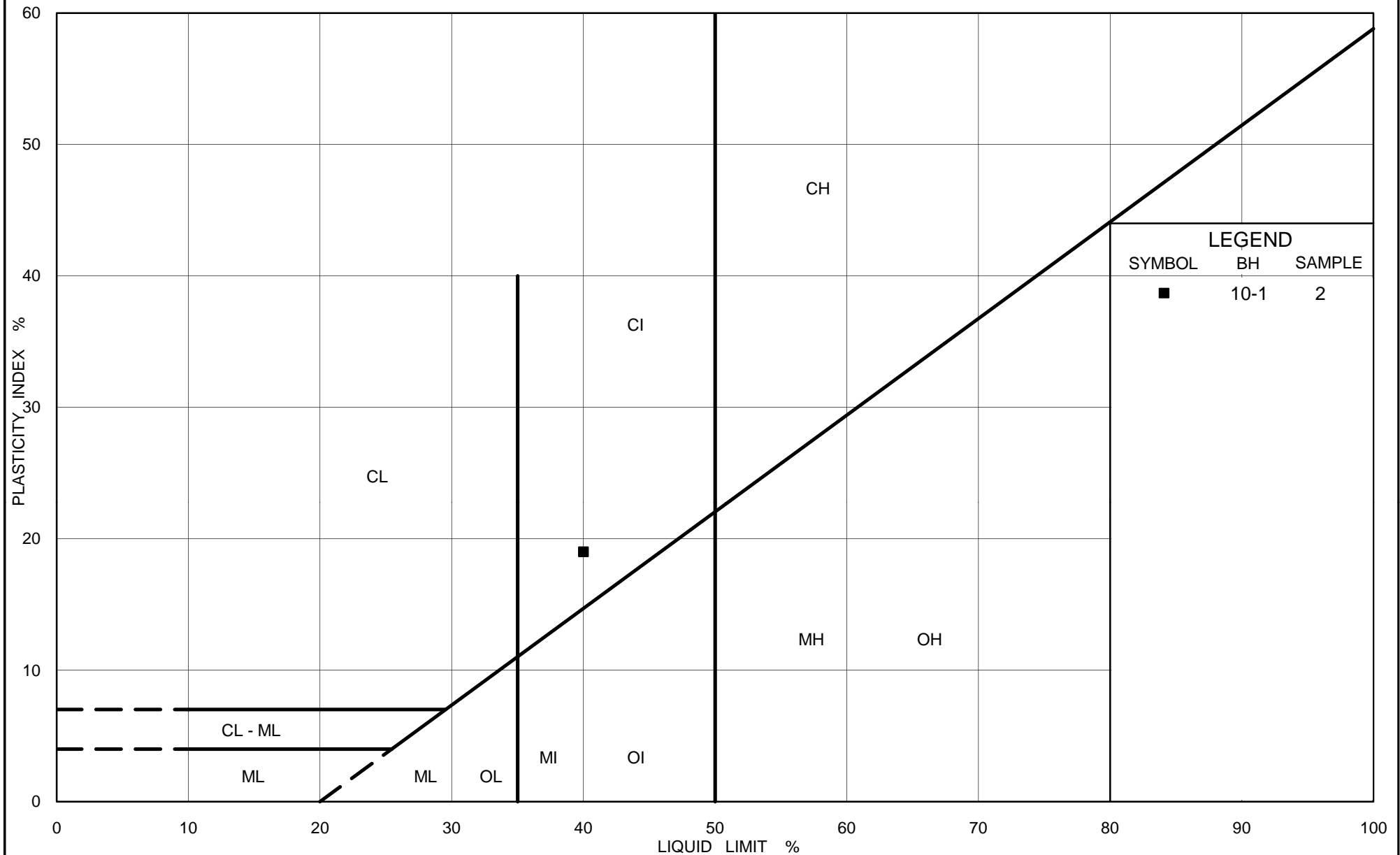
GRAIN SIZE DISTRIBUTION

FIGURE A4

Silty SAND, some gravel and clay (TILL)



Borehole	Sample	Depth (m)
10-1	6	4.57-5.18
10-2	9	6.86-7.47
10-2	13	9.91-10.37
10-3	7	5.34-5.95
10-3	11	8.38-8.69



Ministry of Transportation

Ontario

PLASTICITY CHART

Silty Clay

FIG No. A5

Project No. 10-1121-0007-4

Chk by : CNM

Client: Golder Associates Ltd. (Ottawa)
 32 Steacie Drive
 Kanata, ON
 K2K 2A9

Attention: Mr. Bruce Goddard

Report Number: 1029637
 Date: 2010-12-07
 Date Submitted: 2010-12-03

Project: 10-114-0007-200


Chain of Custody Number: 127476

P.O. Number:
 Matrix: Soil

			LAB ID:	850245	850246	850247	850248	GUIDELINE		
			Sample Date:	2010-12-02	2010-12-02	2010-12-02	2010-12-02			
			Sample ID:	10-1/S-3	10-3/S-2	10-34/S-7	10-37/S-4			
PARAMETER	UNITS	MRL						TYPE	LIMIT	UNITS
Chloride	%	0.002		0.007	0.015	0.029	0.017			
Electrical Conductivity	mS/cm	0.05		0.24	0.41	0.80	0.52			
pH				8.4	8.4	7.7	8.0			
Resistivity	ohm-cm	1		4170	2440	1250	1920			
Sulphate	%	0.01		0.04	0.03	0.07	0.04			

MRL = Method Reporting Limit INC = Incomplete AO = Aesthetic Objective OG = Operational Guideline MAC = Maximum Allowable Concentration IMAC = Interim Maximum Allowable Concentration
 Comment:

APPROVAL:


 Lorna Wilson
 Agriculture Lab Supervisor

Methods references and/or additional QA/QC information available on request.



APPENDIX B

Record of Borehole Sheets and Detailed Laboratory Test Results Previous Investigations



**FOUNDATION INVESTIGATION AND DESIGN REPORT
RECREATIONAL TRAIL CULVERT**

GOLDER ASSOCIATES

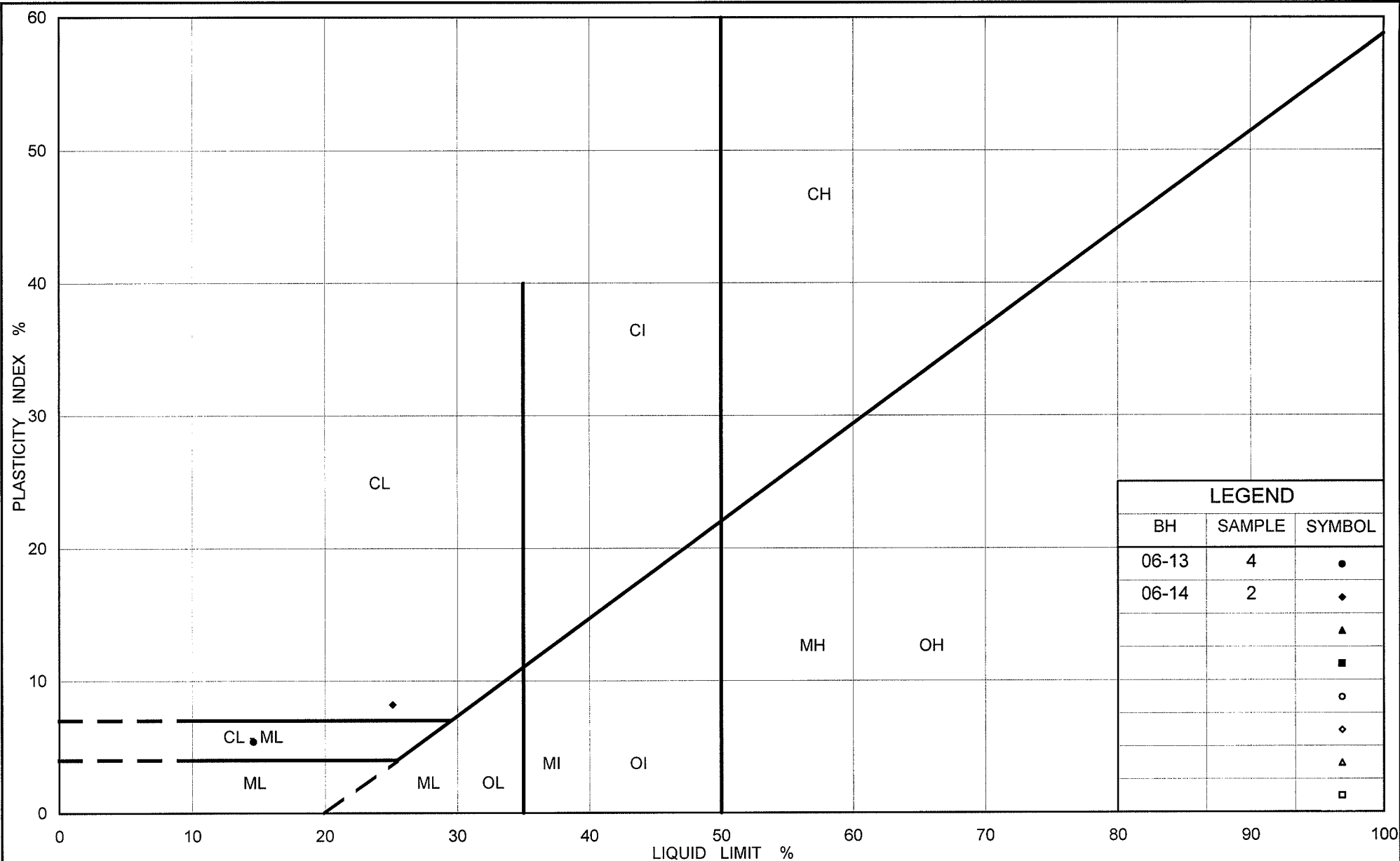
**Foundation Investigation and Design, Preliminary Design,
Recreational Trail Culvert, Highway 7 from Fowlers Corners,
Southerly to County Road 28, Peterborough, Ontario,
G.W.P. 73-99-00, Site No. 26-35", dated June 2007.
Project # 04-1111-0**

PROJECT 04-1111-024B			RECORD OF BOREHOLE No 06-13			1 OF 1 METRIC		
W.P. 73-99-00			LOCATION N 4907829.8 ; E 390031.5			ORIGINATED BY SB		
DIST HWY 7			BOREHOLE TYPE Power Auger, 107 mm O.D. Solid Stem Augers			COMPILED BY DD		
DATUM Geodetic			DATE July 10, 2006			CHECKED BY SLP		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
268.4	GROUND SURFACE							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L WATER CONTENT (%)
0.0	Sand, trace gravel (FILL) Very loose Brown Moist		1	SS	3		268	
266.9							267	
1.5	CLAYEY SILT, trace organics AND FIBROUS PEAT, trace sand Very soft Brown and grey to black Moist		2	SS	WH		266	
266.1							265	
2.3	CLAYEY SILT, trace to some sand and gravel Stiff Brown and grey Moist		3	SS	10			
			4	SS	9		264	
264.1							263	
4.3	SILTY SAND, some gravel Compact Brown Wet		5	SS	10		262	
262.6							261	
5.8	SILTY SAND, trace to some clay and gravel, contains sand and gravel interlayers, cobbles and boulders (TILL) Very dense Grey Moist		6	SS	80		260	
			7	SS	55			
259.1			8	SS	65/0.15			
9.3	End of Borehole							
Notes: 1. Water level measured in piezometer at 8.7 m depth (Elevation 259.7 m) upon completion of installation. 2. Water level measured in piezometer at 7.2 m depth (Elevation 261.2 m) on July 11, 2006. 3. Water level measured in piezometer at 1.7 m depth (Elevation 266.7 m) on July 31, 2006. 4. Water level measured in piezometer at 2.1 m depth (Elevation 266.3 m) on August 18, 2006.								

MIS-MTO 001 04-1111-024.GPJ GAL-MISS.GDT 6/12/07 DD

PROJECT 04-1111-024B			RECORD OF BOREHOLE No 06-14			1 OF 1 METRIC																	
W.P. 73-99-00			LOCATION N 4907829.5 ; E 389997.4			ORIGINATED BY SB																	
DIST _____ HWY 7			BOREHOLE TYPE Power Auger, 107 mm O.D. Solid Stem Augers			COMPILED BY DD																	
DATUM Geodetic			DATE July 11, 2006			CHECKED BY SLP																	
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	20 40 60 80 100	20 40 60 80 100	10 20 30	γ	GR SA SI CL						
268.3	GROUND SURFACE																						
0.0	Sand, trace gravel and silt (FILL) Loose to compact Brown Moist		1	SS	10		268																
266.8							267																
1.5	CLAYEY SILT, some sand, trace gravel Soft to stiff Brown and grey Moist to wet		2	SS	4		266																
			3	SS	11		265																
265.3							264																
3.1	SILTY SAND, trace to some gravel and clay Loose Brown and grey Moist to wet		4	SS	8		263																
263.7							262																
4.6	SILTY SAND, some gravel contains cobbles and boulders (TILL) Very dense Brown and grey Moist		5	SS	70		261																
							260																
262.2							259																
6.1	CLAYEY SILT, with sand, trace gravel, contains cobbles and boulders (TILL) Hard Grey Moist		6	SS	76/0.15																		
			7	SS	70/0.15																		
259.0			8	SS	80/0.15																		
9.3	End of Borehole																						
Notes:																							
1. Water level measured in piezometer at 7.3 m depth (Elevation 261.0 m) on upon completion of installation.																							
2. Water level measured in piezometer at 1.6 m depth (Elevation 266.7 m) on July 31, 2006.																							
3. Water level measured in piezometer at 1.8 m depth (Elevation 266.5 m) on August 18, 2006.																							

MIS-MTO 001 04-1111-024.GPJ GAL-MISS.GDT 6/12/07 DD



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt

FIG No. A1

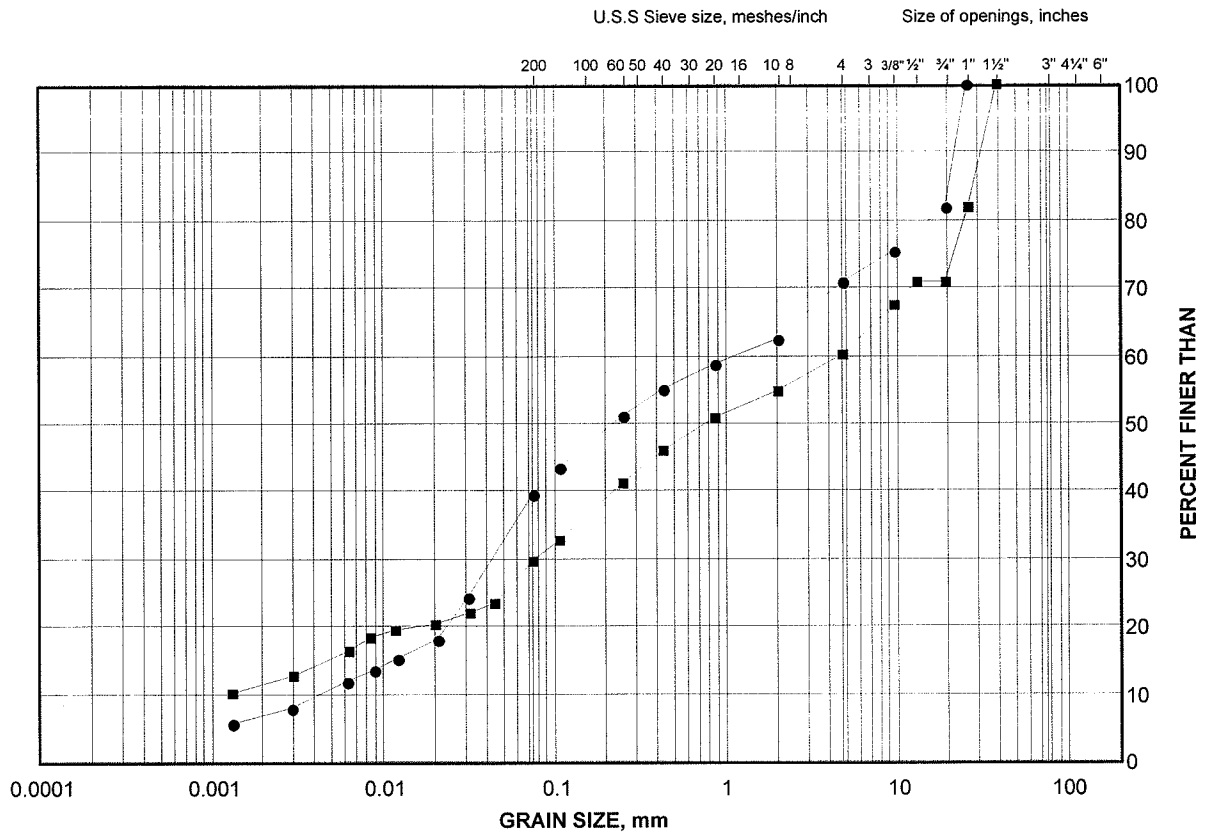
Project No. 04-1111-024B

Checked by: *ASB*

GRAIN SIZE DISTRIBUTION

Silty Sand (Till)

FIGURE A2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	06-14	5	263.4
■	06-13	6	262.0

Project Number: 04-1111-024B

Checked By: KTB

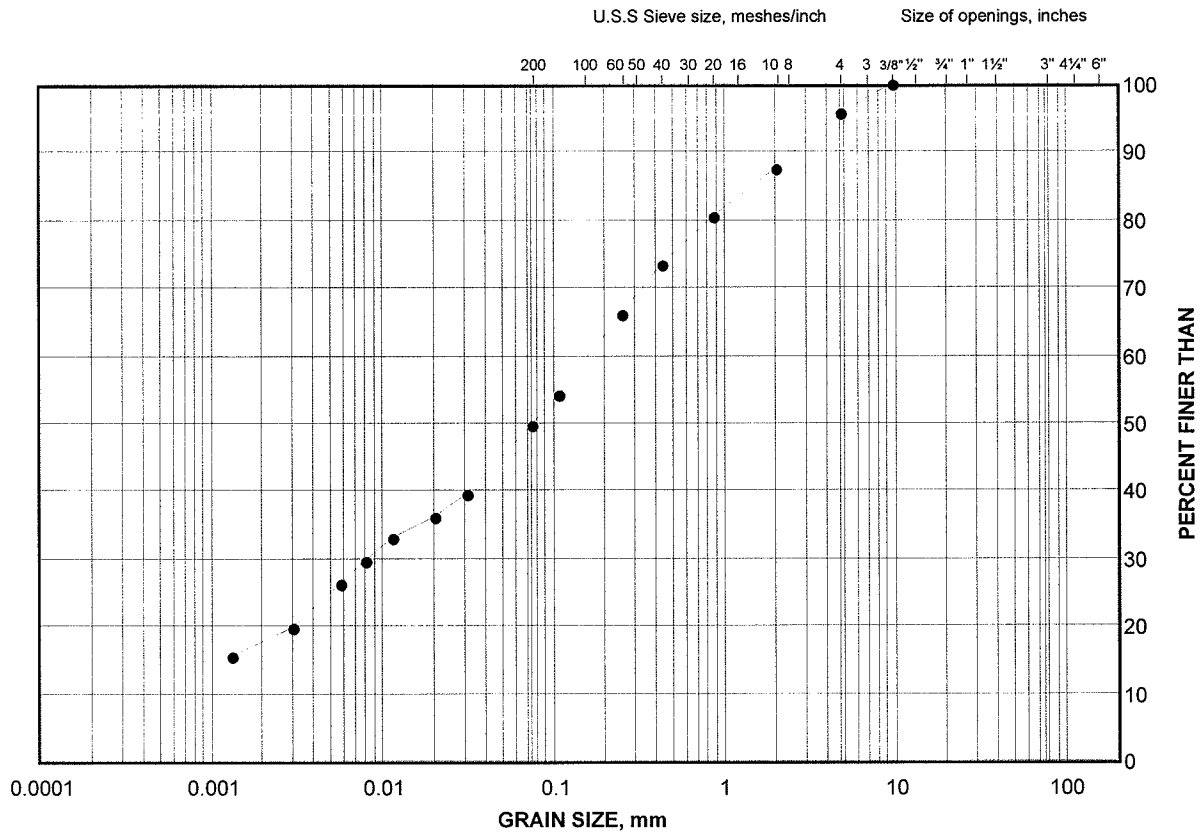
Golder Associates

Date: 19-Feb-07

GRAIN SIZE DISTRIBUTION

Clayey Silt (Till)

FIGURE A3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	06-14	7	260.7

Project Number: 04-1111-024B

Checked By: KTB

Golder Associates

Date: 19-Feb-07



FOUNDATION INVESTIGATION AND DESIGN REPORT RECREATIONAL TRAIL CULVERT

RACEY, MACCALLUM and ASSOCIATES

“Foundation Investigation for a Bridge over the CNR Bridge Crossing at Fowlers Corner, Highway 133, near Peterborough, Ontario”, dated Se

Order No.: S-500/T-852 RACEY, MACCALLUM AND ASSOCIATES

LIMITED

Barley
Driller

Hole Begun _____ Foundation Engineering Division

Hole Ended _____ Engineering Data Sheet for Borehole: 57-1

Helper

Job Name: C.N.R. Overhead

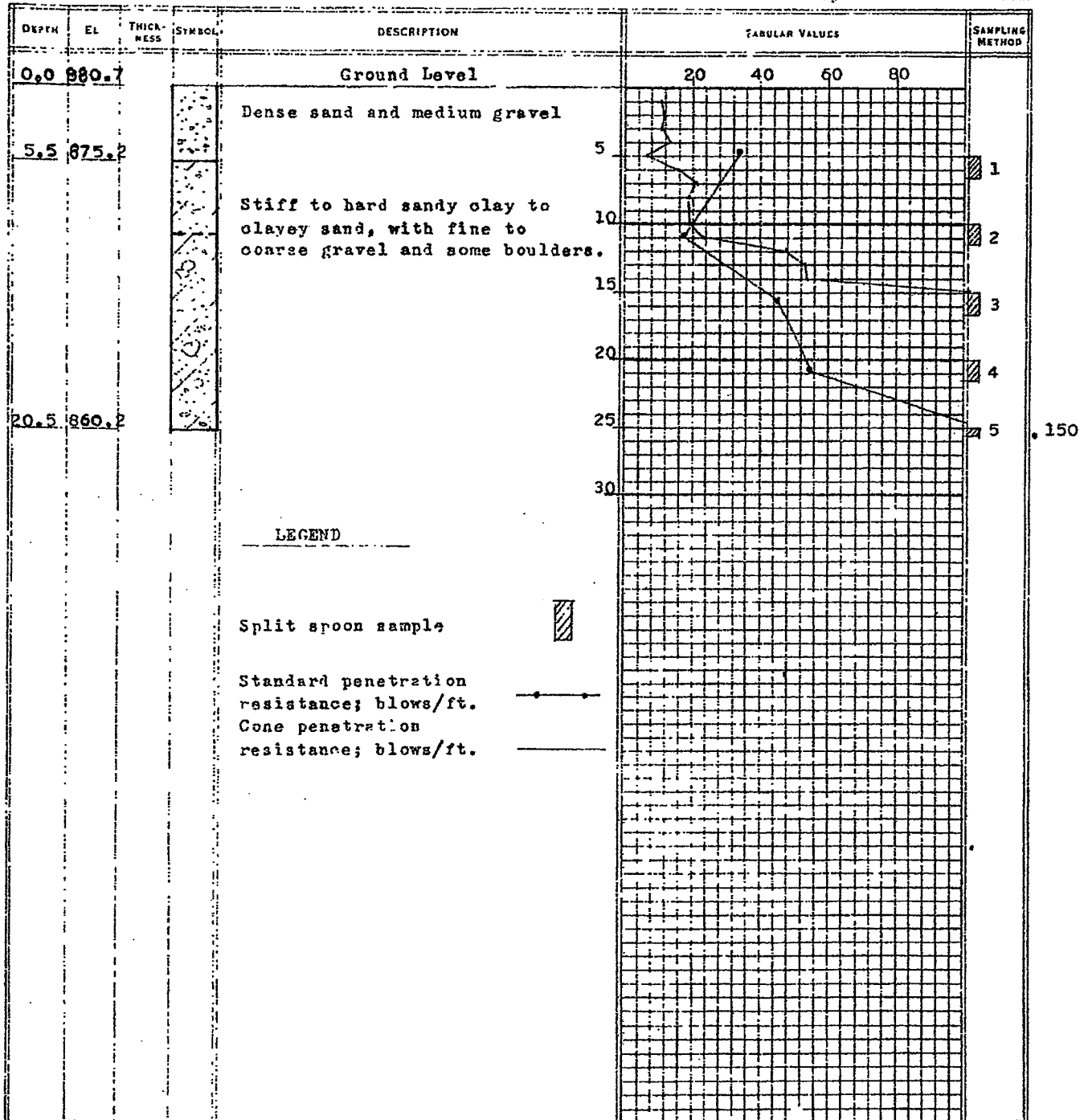
J.S.

Job Located: Highway 133, Peterborough, Ontario

Checked by

Hole Located: See enclosure No. 1

Hole Elevation: 880.7 Datum: M.S.L.

3 9 57
Day Month Year

Order No.: E-500/T-652 RACEY, MACCALLUM AND ASSOCIATES

LIMITED

Vidal

Driller

Hole Begun _____

Foundation Engineering Division

Hole Ended _____

Engineering Data Sheet for Borehole: 57-3

Helper

Job Name: C.N.R. Overhead

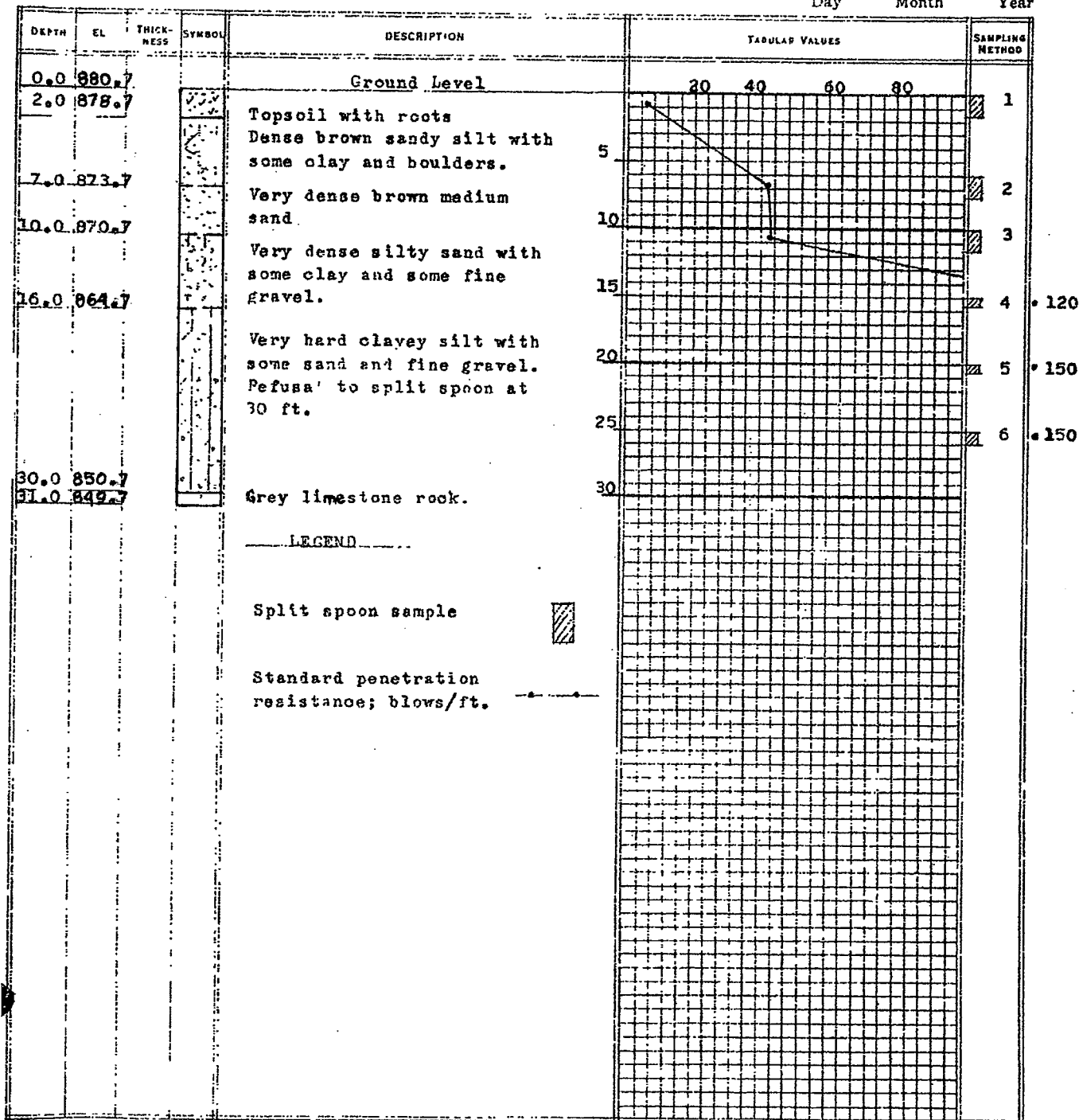
J.S.

Job Located: Highway 133, Peterborough, Ontario

Checked by

Hole Located: See enclosure No. 1

Hole Elevation: 880.7 Datum: M.S.L.

3 9 57
Day Month Year



APPENDIX C

Dewatering Non-Standard Special Provision

GROUNDWATER CONTROL - Item No.

Non-Standard Special Provision

Foundations for the new culvert will require excavations to extend below the groundwater level at the site. Cohesionless (sand to silty sand) soils that are present below the groundwater table will slough, run, boil or cave in to the excavation unless appropriate groundwater control is in place. Depending on where the groundwater level is at with respect to the depth excavation at the time of construction, unwatering may be required during the construction of cast-in-place elements for the culvert to minimize groundwater inflow into the excavation and construction area as well as to ensure construction of the cast-in-place elements is carried out in dry conditions. The Contractor is to design and install an appropriate unwatering system for the culvert site to prevent disturbance of the founding soils and enable construction in dry conditions.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. 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 who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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