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REPORT ON

**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**

Submitted to:

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GEOCRE NO.: 31D-427

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June 2007

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PART A

**FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by National Capital Engineering Limited (NCE) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out preliminary foundation investigation associated with the 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 P31-8197, dated August 2003, that forms part of the Consultant's Agreement (P.O. Number 4005-A-000268). This report addresses the preliminary foundation investigation carried out for the removal of the existing CNR Overhead structure and replacement with a Recreational Trail Opening structure (Site No. 26-35) as part of the Highway 7 improvement project. Preliminary foundation investigation and design services are required for a total of four structures (i.e. Jackson Creek Bridge, Recreational Trail Opening structure and two structural culvert sites) in four separate reports for this project. The scope of foundations work for this project was expanded to include several swamp areas and a high fill area as outlined in Golder's letter titled "Proposal for Addendum Foundation Investigation", dated April 6, 2006 and approved by MTO on April 8, 2006. The work was carried out in accordance with the Quality Control Plan for this project dated April 20, 2006.

The purpose of this investigation is to establish the subsurface conditions at the proposed Recreational Trail Opening structure site by borehole drilling, in situ testing and laboratory testing on selected samples. A plan drawing of the proposed new Highway 7 re-alignment and recreational trail culvert was provided to Golder by NCE in July 2006.

The preliminary investigation was supplemented with information from the following previous reports and drawings:

- Report titled "Foundation Investigation for a Bridge over the CNR Bridge Crossing at Fowlers Corner, Highway 133, Near Peterborough, Ontario", dated September 30, 1957, prepared by Racey, MacCallum and Associates Limited, GEOCRE. No 31D-227.
- Design Drawings for Smith Twp. C.N.R. Overhead at Fowler's Corners titled "General Layout, Dwg No. 211-35-1-A", "Preliminary Plan, Dwg. No. 211-35-P1-A", "Footing Plan, Dwg. No. 211-35-2-A", and "Structure Reinforcement, Dwg. No. 211-35-3-A", dated May 1958 to July 1958, prepared by Department of Highways, Ontario, W.P. 942-57.
- Preliminary Design Drawing titled "Smith Twp. C.N.R. Overhead, Site No. 26-035, General Arrangement, Alternative 5, New Concrete Structure", dated January 2005, provided by NCE Limited.

2.0 SITE DESCRIPTION

The general site is located on 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 that is no longer in operation and 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 overhead structure. The ground surface east and west of the highway embankment generally ranges from about Elevation 266 m to 269 m. The existing Highway 7 overpass bridge deck is as high as Elevation 278.2 m and embankment side-slopes are inclined at about 2.5 horizontal to 1 vertical (2.5H:1V). The existing highway embankment side-slopes are grass covered and no visible signs of movement / instability were observed at the time of our investigation. The existing structure is a three-span reinforced concrete bridge shown on the design drawings to be supported on steel H-pile foundations driven to refusal. It is understood that the existing overpass 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

The 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 current 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 up to 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 that follow the text of this report.

The field work was observed on a full-time basis by a member of Golder's engineering technical 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.

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

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, ranging in thickness from about 0.5 m to 2.5 m. "Wave-washed" drumlins, with exposed bouldery surfaces, are also present near the Simcoe Lowlands immediately south and east of Lake Simcoe. Localized deposits of silt, clay and peat are found in the low-lying areas between drumlins.

4.2 Subsoil Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of in situ and laboratory testing are

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.

shown on the Record of Borehole sheets and in Appendix A following the text of this report. A copy of the relevant Record of Borehole sheets from the 1957 investigation are included in Appendix B and the approximate locations and interpreted stratigraphic profile included on Drawing 1.

The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy based on the results of the boreholes at the site is shown on Drawing 1.

In general, the subsoils at the proposed structure site consist of a surficial deposit of sand fill, underlain by a deposit of clayey silt. A layer of fibrous peat and clayey silt containing organics was encountered between the fill deposit and clayey silt deposit in borehole 06-13. The clayey silt deposit is underlain by a silty sand to sandy silt layer, which was underlain by a silty sand till to clayey silt till deposit. Limestone bedrock was reported below the till deposit in borehole 57-3, drilled as part of the previous 1957 investigation. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

A surficial layer of topsoil, about 0.6 m thick, was encountered in borehole 57-3 during the 1957 investigation. It is not known whether this topsoil was stripped prior to the construction of the existing highway embankment.

A Standard Penetration Test (SPT) “N” value recorded within the topsoil was 6 blows per 0.3 m of penetration indicating a loose relative density.

4.2.2 Fill

A surficial layer of fill, about 1.5 m thick, was encountered at boreholes 06-13 and 06-14 which were located on the existing recreational trail embankment (previous CNR rail embankment) and near the toe of the existing highway embankment, respectively. The fill consists of predominantly sand and contains trace silt and gravel.

Standard Penetration Test (SPT) “N” values recorded within the sand fill were 3 and 10 blows per 0.3 m of penetration, indicating the fill has a very loose to compact relative density.

4.2.3 Fibrous Peat / Clayey Silt containing organics

A layer of fibrous peat and clayey silt containing organics was encountered below the fill deposit in borehole 06-13. The peat and clayey silt containing organics layer was encountered at a depth of 1.5 m (Elevation 266.9 m) and was about 0.8 m thick.

An SPT “N” value recorded within the peat / clayey silt containing organics layer was 0 blows (i.e. weight of hammer) per 0.3 m of penetration indicating the peat / clayey silt layer has a very soft consistency.

The natural water content measured on a sample of the clayey silt containing organics layer was 32 per cent.

4.2.4 Clayey Silt

Underlying the topsoil and fill, a clayey silt deposit was encountered in boreholes 06-13 and 06-14. This brown and grey clayey silt deposit generally contains trace to some sand and trace gravel. The top of the clayey silt was encountered at a depth of 2.3 m and 1.5 m (Elevation 266.1 m and 266.8 m) and was about 1.6 m and 2.0 m thick in boreholes 06-13 and 06-14, respectively.

The SPT “N” values recorded within the clayey silt ranged from 4 blows to 11 blows per 0.3 m of penetration indicating the clayey silt has a soft to stiff consistency.

The natural water content measured on select samples of the clayey silt were 11 per cent and 20 per cent.

The results of Atterberg Limits testing carried out on two samples of the clayey deposit are illustrated on the plasticity chart on Figure A1 in Appendix A. The test results are summarized below and indicate the clayey silt is of low plasticity.

<i>Borehole</i>	<i>Sample</i>	<i>Elevation (m)</i>	<i>Liquid Limit (%)</i>	<i>Plastic Limit (%)</i>	<i>Plasticity Index (%)</i>
06-13	4	264.8 – 265.4	15	9	6
06-14	2	266.2 – 266.8	25	17	8

4.2.5 Silty Sand to Sandy Silt

A cohesionless soil layer consisting of silty sand to sandy silt was encountered below the clayey silt deposit at boreholes 06-13 and 06-14, and at ground surface and beneath the topsoil in

boreholes 57-1 and 57-3, respectively. The silty sand to sandy silt soil typically contained trace to some gravel.

In boreholes 06-13 and 06-14, the top of the silty sand layer was encountered at a depth of about 4.3 m (Elevation 264.1 m) and 3.1 m (Elevation 265.3 m), respectively, and was 1.5 m thick. SPT “N” values recorded within this silty sand layer were 8 and 10 blows per 0.3 m of penetration indicating a loose to compact relative density.

Two water contents measured on samples of the silty sand in boreholes 06-13 and 06-14 were 11 and 21 percent.

In boreholes 57-1 and 57-3 (located west of the proposed new highway alignment), the sandy layer was encountered at ground surface and at a depth of 0.6 m, respectively. The sandy silt to silty sand layer was about 5.8 m and 4.3 m thick in boreholes 57-1 and 57-3, respectively. The silty sand to sandy silt was described as containing some clay and boulders in borehole 57-3. In borehole 57-1, a clayey interlayer was present from about 1.7 m to 3.4 m depth (Elevation 265.1 m to 266.8 m) and the sandy deposit contained boulders at depth. SPT “N” values recorded within the silty sand to sandy silt deposit in boreholes 57-1 and 57-3 ranged between about 18 and 47 blows per 0.3 m of penetration indicating that the silty sand to sandy silt at this location is compact to dense.

4.2.6 Silty Sand Till

A silty sand glacial till deposit was encountered below the silty sand to sandy silt in boreholes 06-13, 06-14 and 57-1. The till consisted of predominantly silty sand and contained variable amounts of gravel, clay, cobbles and boulders. The top of the silty sand till deposit was encountered at depths ranging between about 4.6 m and 5.8 m (Elevation 262.6 m to 263.7 m). The silty sand till was 1.5 m thick in borehole 06-14 and was penetrated for 3.5 m and 1.8 m in boreholes 06-13 and 57-1 which were terminated within this deposit at depths of 9.3 m (Elevation 259.1 m) and 7.6 m (Elevation 260.9 m), respectively.

SPT “N” values recorded within the silty sand till range from about 55 blows per 0.3 m of penetration to 150 blows per 0.15 m of penetration, indicating the silty sand till deposit is very dense.

The natural water contents measured on two select samples of the silty sand till were 7 and 9 percent. Grain size distribution curves for two samples of the silty sand till deposit are shown on Figure A2 in Appendix A.

4.2.7 Clayey Silt Till

Underlying the silty sand to sandy silt layer and the silty sand till, a deposit of clayey silt till was encountered in boreholes 06-14 and 57-3. The clayey silt till contained variable amounts of sand, gravel, cobbles and boulders. The top of the clayey silt till deposit was encountered at a depth of 6.1 m (Elevation 262.2 m) and 4.9 m (Elevation 263.5 m) in boreholes 06-14 and 57-3, respectively. The clayey silt till penetrated 3.2 m and 4.3 m in boreholes 06-14 and 57-3 which were terminated within this deposit at depths of 9.3 m (Elevation 259.0 m) and 9.1 m (Elevation 259.3 m), respectively.

SPT “N” values recorded within the clayey silt till ranged from about 70 blows to 150 blows per 0.15 m of penetration, indicating a hard clayey silt till deposit.

The natural water content measured on a sample of the clayey silt till was 7 per cent. A grain size distribution curve for a sample of the clayey silt till deposit is shown on Figure A3 in Appendix A.

4.2.8 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 per cent.

4.2.9 Groundwater Conditions

During the current investigation, the water levels were noted within the open boreholes at the time of the drilling operations and piezometers were installed in both boreholes 06-13 and 06-14. The piezometers were sealed into the upper portion of the till deposit, below the clayey silt deposit. 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. There were no recorded water levels or soil moisture information on the previous record of boreholes 57-1 and 57-3.

<i>Borehole</i>	<i>Installation</i>	<i>Ground Surface Elevation (m)</i>	<i>Depth to Water Level(m)</i>	<i>Water Level Elevation (m)</i>	<i>Date</i>
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
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

Although no water level was encountered during drilling in July 2006, the cohesionless fill materials and sandy soils should be expected to be water-bearing during wet periods of the year, and "perched" water above the clayey silt and till deposits should be expected. Although not evident within the current boreholes, artesian conditions are known to exist in the area and may be present at certain times of the year within the confined sandy layer between the till and the clayey silt deposit. It should be noted that groundwater levels at the site are expected to fluctuate seasonally and are expected to rise during wet periods of the year.

5.0 CLOSURE

The field technician supervising the drilling program was Mr. Suresh Bainey. This report was prepared by Ms. Shannon Palmer, EIT, and Mr. Kevin J. Bentley, P.Eng., a 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.

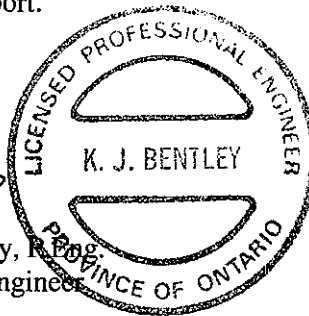
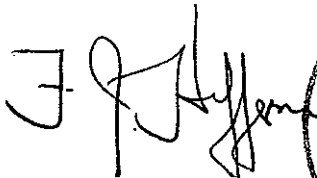
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PART B

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RECREATIONAL TRAIL CULVERT
HIGHWAY 7 FROM FOWLERS CORNERS
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6.0 PRELIMINARY ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the preliminary design of the proposed recreational trail culvert structure 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 current preliminary and the 1957 subsurface investigations at the 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 preliminary design of the proposed culvert structure foundations. Where comments are made on construction they are provided in order to highlight those aspects which could affect the preliminary design of the project and for which, ultimately, provision will have to be made at the detail investigation design stage of the project and as the contract documents are prepared.

Further borehole drilling will be required during the detail design phase of the project, when the details of the proposed recreational trail culvert structure are finalized. At that time, further investigation of the thickness and composition of the surficial fill, underlying peat / organics and clayey silt will be required to confirm founding elevations and sub-excavation / backfill requirements. For the anticipated high fill embankment to be constructed as part of the recreational trail culvert structure, additional borehole drilling is recommended to confirm slope stability, magnitude and time rate of settlement under the proposed embankment loading, and to further assess settlement mitigation options and develop the necessary operational constraints and/or special provisions for the contract.

6.1 General

As part of the highway operational improvement plan, it is understood that Highway 7 is to be re-aligned around the existing CNR overhead structure and eventually 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. Based on conversations with the designer, the existing CNR overhead bridge structure is to be removed / demolished and a new recreational trail culvert constructed in order to accommodate recreational uses. The proposed new culvert is located about 35 m east of the existing structure along the proposed new Highway 7 alignment at about Station 26+410 (see Drawing 1).

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 the Preliminary General Arrangement (GA) drawing provided by NCE, the proposed recreational trail culvert is to consist of a concrete box culvert approximately 5.7 m wide x 4.7 m high x 20.5 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 (see Drawing 1). The proposed new embankment is shown on the GA drawing to be about 6.2 m high above existing grade and sloped at about 2.5H:1V. 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 re-directed 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 clayey silt deposit. A thin layer of peat and clayey silt containing organics was encountered between the fill and clayey silt deposit in borehole 06-13. Underlying the clayey silt deposit, a layer of silty sand was present, underlain by a deposit of silty sand till to clayey silt till. Water levels were measured at depths of 8.7 m and 7.3 m below ground surface upon completion of the drilling operations in borehole 06-13 and 06-14. However, piezometers installed at boreholes 06-13 and 06-14, which were sealed within the upper portion of the silty till deposit, measured water levels up to 1.7 m (Elevation 261.2 m) and 1.6 m (Elevation 266.5 m) below ground surface on July 31, 2006. It should be noted that the water levels in this area will fluctuate on a seasonal basis.

Based on the subsurface information obtained, the lower portion of the clayey silt deposit (located below the surficial topsoil, fill, peat, and very soft to soft clayey silt containing organics) is firm to 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. Consideration was given to using an open footing culvert and deep foundations; however, these options were not considered practical given the relatively competent shallow

subsoil conditions. As indicated in Table 1, from a foundations perspective, the preferred culvert structure type at this location is a concrete box culvert.

Based on discussions with NCE, it is understood that the preferred culvert will be constructed as a cast in place structure with longitudinal reinforcement. A precast 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. As a result, the preferred alternative (taking into consideration structural and foundations considerations) would be a cast in place concrete box culvert.

Some subexcavation of any topsoil, fill, peat, and any very soft to soft clayey silt or highly organic soils and replacement with engineered fill soils will be required below the proposed culvert and embankment footprints.

6.3 Preferred Alternative – Concrete Box Culvert

6.3.1 Geotechnical Resistance

The approximate invert elevation, recommended level of subexcavation, and the founding soil type for a box culvert is presented below.

Approximate Culvert Station	Structure	Relevant Boreholes	Approximate Invert Elevation (m)	Recommended Subexcavation Level for Proposed Culvert (m)	Founding Soil Type
26+410	Concrete Box Culvert	06-13, 06-14	268.4	266.0 to 266.5	Firm to Stiff Clayey Silt

The above recommended subexcavation levels indicates the estimated minimum and maximum target elevations required to reach the appropriate founding subgrade soil. The actual founding level will depend on the final location of the culvert, thickness of the bottom of the culvert, design invert level and frost protection requirements, the depth of the granular bedding required under the culvert (see Section 6.7.3), and the results of additional boreholes / investigation during detailed design.

Assuming the sub-excavation / 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 preliminary design of the box culvert is given below.

Approximate Culvert Station	Structure	Total Proposed Culvert Width (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Resistance at SLS (kPa) for 25 mm settlement
26+410	Concrete Box Culvert	5.7	250	175

The geotechnical resistance values above assume the culvert is founded on granular bedding soils supported on approved engineered fill and/or undisturbed native clayey silt which for design may be assumed to extend to the subexcavation levels indicated above. Water levels were measured to be as high as 1.6 m below ground surface (Elevation 266.7 m) within the lower cohesionless silty sand and silty sand till deposit (located within about 1 m of the proposed founding elevation). As a result, it is recommended that subexcavation / founding depths be kept as high as possible within the firm to stiff clayey silt deposit to reduce the risk of unstable founding soils and to reduce dewatering efforts.

A factored geotechnical resistance at ULS of 250 kPa and a geotechnical resistance at SLS of 175 kPa may also be used for any spread footings for the retaining walls (wing walls) or for an open footing culvert founded within the firm to stiff clayey silt (i.e. founded between Elevation 266.0 to 266.5 m).

6.3.2 Resistance to Lateral Loads

Resistance to lateral forces (i.e. sliding resistance) between the base of the concrete culvert foundation and the underlying soils should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For the concrete box culvert option, assuming the culvert is cast in place concrete and is placed on compacted granular bedding (Granular 'A'), a coefficient of friction value ($\tan \delta$) of 0.58 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.

6.3.3 Frost Protection

The frost penetration depth in the area of the proposed culvert is estimated to be approximately 1.5 m. Any shallow foundations should be provided with a minimum of 1.5 m of soil cover or equivalent thermal insulation for frost protection.

6.4 Lateral Earth Pressures for Preliminary Design

The lateral earth pressures acting on the new structure any associated wing walls / retaining 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 3101.150 and 3121.150.
- 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* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with Special Provision 105S10. 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 1.5 m behind the back of the wall stem (Case I in Figure C6.9.1(l)(i) of the *Commentary to the CHBDC*) or within the wedge-shaped zone 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 II in Figure C6.9.1(l)(ii) of the *Commentary to the CHBDC*).
- For Case I, 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:

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

	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 wall support and superstructure allow lateral yielding of the stem, 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 6 m, a 10 to 20 per cent 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 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 I and Case II) 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 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/m³) (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).

6.5 Settlement

Referring to Drawing 1, it is anticipated that the new culvert structure invert elevation will match the existing recreational trail ground surface which is at about Elevation 268.4 m. Based on the preliminary general arrangement drawing, it is understood that the proposed re-aligned Highway 7 embankment will be up to about 6.2 m high (approximate Elevation 274.6 m) with 2.5H: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. Assuming the proposed embankment and culvert footprint will be stripped of topsoil, peat, fills and any very soft to soft clayey soils or soils containing significant organics (i.e. up to about 2.3 m in borehole 06-13) and replaced with approved engineered fill, the total thickness of the new embankment fill is anticipated to be up to about 8.5 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 150 kPa.

A preliminary settlement analysis was performed using the commercially available program UNISETTLE (Version 3.2) produced by Unisoft Limited. The soil parameters used for the analysis were based on the laboratory and in situ test data collected during the current preliminary investigation.

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 firm to stiff 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 clayey silt deposits from nearby sites as part of the overall Highway 7 foundations investigation. Based on the subsoil information collected, the firm to stiff 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 clayey silt deposit.

The following summarizes the simplified stratigraphy, unit weights and deformation parameters employed in the settlement analysis:

<i>Soil</i>	<i>Thickness (m)</i>	<i>Bulk Unit Weight (kN/m³)</i>	<i>Deformation Properties*</i>
Granular Engineered Fill (backfill) and Bedding	2	21	$E' = 15 \text{ MPa}$
Firm to Stiff Clayey Silt	2	20	$m_v = 1.1 \times 10^{-4} \text{ kPa}^{-1}$
Loose to Compact Silty Sand	1.5	19	$E' = 9 \text{ MPa}$
Very Dense Silty Sand Till to Hard Clayey Silt Till	4	21	$E' = 50 \text{ MPa}$

* E' = Elastic Modulus and m_v = coefficient of volume change

Although the net loading due to the placement of the new culvert itself on the foundation soils is expected to be relatively small, the structural design of the culvert sections should consider resistance to the bending moment anticipated to occur along the centreline of the culvert due to the non-uniform embankment geometry, past and present loading conditions (i.e. accommodate vertical settlements and horizontal strains). Some areas have been partially preloaded by the existing highway embankment. 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; however, this statement should be re-visited after the detailed design investigation.

The predicted maximum total settlement of the foundation soils at the culvert location is estimated to be about 50 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 30 mm of time dependent settlement of the cohesive soil layer.

Based on an estimated coefficient of consolidation (c_v) of about $4 \times 10^{-2} \text{ cm}^2/\text{s}$ and assuming two-way drainage of the approximately 2 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.

Considering portions of the proposed embankment and culvert footprint have been preloaded by the existing highway embankment, the estimated consolidation settlement of 30 mm is anticipated to be differential within the culvert footprint. As a result, preloading after subgrade preparation (i.e. subexcavation and backfill) may be considered. The approach embankments adjacent to the culvert location should be constructed and allowed to settle at the proposed culvert location for at least 1 month 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 or cambering the culvert with articulating joints. 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 clayey silt deposit, full subexcavation of the clayey deposit is not recommended. The use of light weight fill is likely too expensive and surcharging not necessary. Cambering of the culvert using articulated joints will require specialized construction and may not be cost effective. 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 box culvert; as a result, consideration could be given to also preloading the culvert footprint with about 3 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; however, the practical benefits of preloading this area should be assessed during detailed design.

6.6 Stability

Static slope stability analyses for the approximate 6.2 m high embankment configuration (assuming 2H:1V side-slopes) were carried out using the following parameters, derived from field and laboratory testing and accepted correlations, using the commercially available program SLOPE/W, produced by Geo-Slope International Ltd.

<i>Soil</i>	<i>Bulk Unit Weight</i>	<i>Effective Friction Angle</i>	<i>Undrained Shear Strength</i>
Granular embankment fill, engineered backfill and bedding materials	21 kN/m ³	32°	–
Firm to stiff clayey silt	20 kN/m ³	28°	50 kPa
Loose to compact silty sand	19 kN/m ³	30°	–
Very dense silty sand to hard clayey silt till	21 kN/m ³	39°	–

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.

6.7 Preliminary Considerations for Culvert / Embankment Construction

6.7.1 Subgrade Preparation and Excavation

Prior to the placement of any foundations, engineered fill (i.e. backfill), bedding, or new embankment construction, all existing fills, topsoil, peat and clayey silt containing excessive organics, and softened or loosened soils should be stripped from below the proposed culvert and embankment footprint and wasted/reused accordingly. All subgrade soils should be inspected and/or proofrolled prior to placement of any foundations or engineered fill (i.e. backfill and embankment fill).

Based on the preliminary boreholes, it is anticipated that excavations up to about 2.3 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 firm to stiff clayey silt deposit. Based on the peat and clayey silt containing organics encountered below the fill in borehole 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). It is not known whether the peat / organics soils were removed as part of the existing highway embankment construction, as such, detail design investigation should investigate the subsoil conditions below the proposed high fill embankment footprint limits which will encompass portions of the east side of the existing highway embankment.

For the culvert construction, the subexcavated area and engineered fill placement should extend from about 1 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 works must 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.

6.7.2 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 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.

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.

6.7.3 Bedding and Backfill / Embankment Construction

For the concrete box culvert option being considered, the firm to stiff clayey silt located at and below the founding elevation provided in Section 6.3 is considered suitable for the support of the bedding and/or backfill for the proposed box culvert. Stripping of any existing fills, topsoil, peat, very soft or loosened and highly organic soils will be required which, based on borehole 06-13, could be up to about 2.3 m below existing ground surface.

For the concrete box culvert option, the bedding, leveling pad, and backfill requirements for the culvert should be in general accordance with OPSS 422 and OPSD 803.01 for precast concrete rigid frame culverts. It is recommended that the box culvert should be provided with at least 300 mm of OPSS Granular 'A' for bedding purposes and partial frost protection. The bedding should be placed in lifts not exceeding 150 mm in loose thickness, and compacted to at least 98 per cent of the Standard Proctor maximum dry density. A minimum 75 mm thick uncompacted leveling pad of Granular 'A' or fine aggregate (OPSS 1002) should be provided.

As previously mentioned, subexcavation of unsuitable soils of up to about 2.3 m 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). If Granular 'B' Type 2 is used, the placement of a geotextile filter between the bottom of the backfill and native soils may be required depending on the actual in situ 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 1.5 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 SP105S10.

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.

7.0 CLOSURE

This report was prepared by Ms. Shannon Palmer, EIT. and Mr. Kevin J. Bentley, P.Eng., a 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.

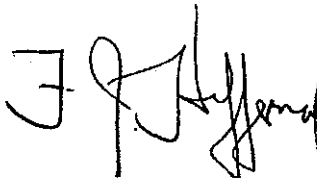
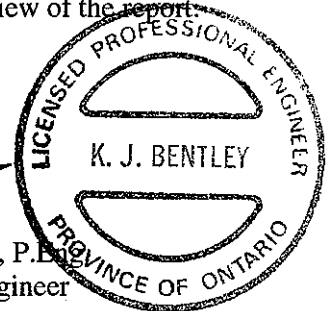
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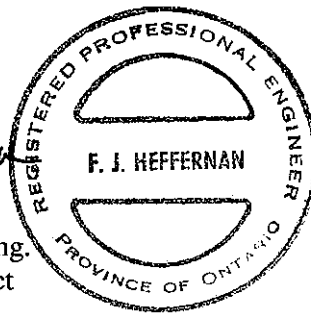
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TABLE 1
EVALUATION OF CULVERT FOUNDATION/CONSTRUCTION ALTERNATIVES
HIGHWAY 7 RECREATIONAL TRAIL CULVERT (SITE NO. 26-35)
G.W.P. 73-99-00

<i>Option</i>	<i>Rank/ Option</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Concrete 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"> • Some water seepage is anticipated during subexcavation and backfill; • Some post-construction settlement is anticipated due to embankment loading 	<ul style="list-style-type: none"> • Low Costs relative to deeper foundation options. 	<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 and embankment will experience some long-term settlement; however, this can be reduced by preloading embankment and the culvert footprint.
Open Footing culvert	2	<ul style="list-style-type: none"> • Shallow sub-excavation depth; • Foundations will be located below frost depth. 	<ul style="list-style-type: none"> • Dewatering required (i.e. more extensive than box culvert option) 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"> • Higher costs than 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 and embankment will experience some long-term settlement; however, this can be reduced by preloading embankment and surcharging 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

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

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

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.)

Consistency

	<u>kPa</u>	<u>psf</u>
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

(b) Cohesive Soils

c_u, s_u

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
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 x acceleration due to gravity)

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

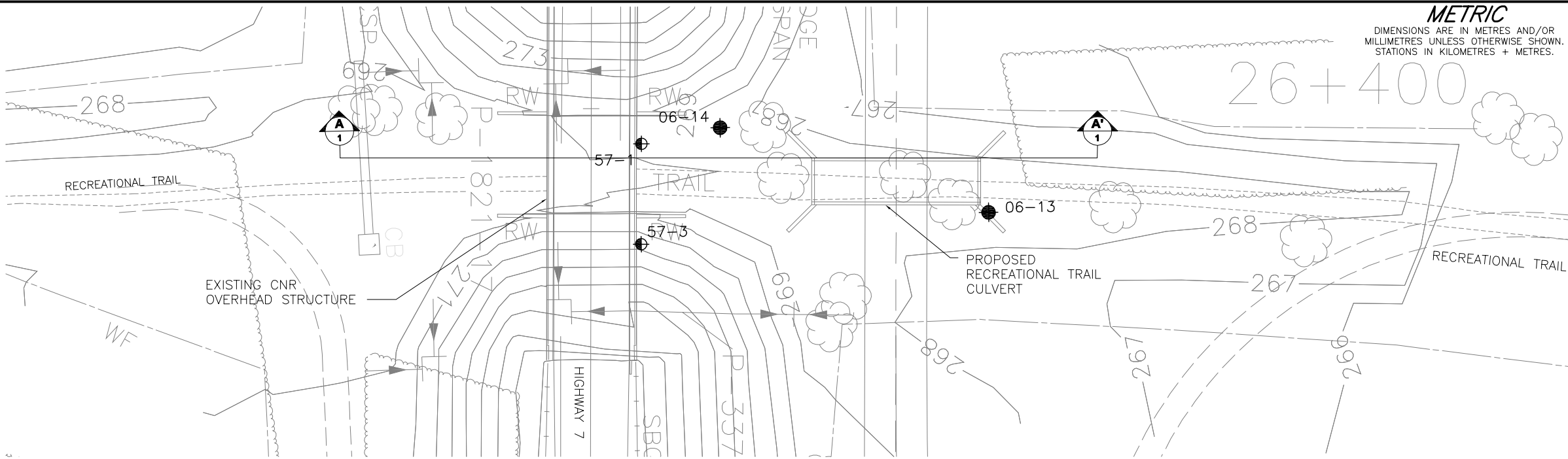
- Notes:** 1 $\tau = c' + \sigma' \tan \phi'$
2 Shear strength = (Compressive strength)/2

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		264	
			4	SS	9		263	
264.1							262	
4.3	SILTY SAND, some gravel Compact Brown Wet		5	SS	10		261	
262.6							260	
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			
			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.								

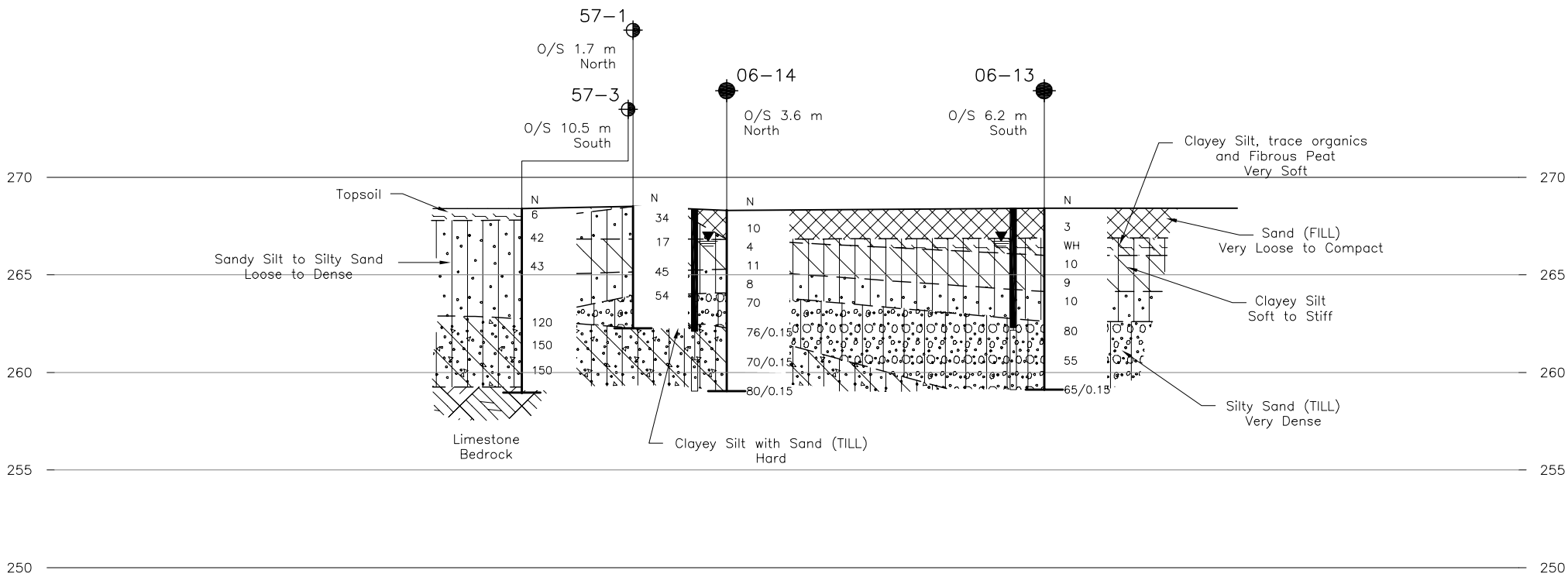
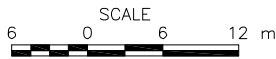
MIS-MTO 001 04-1111-024.GPJ GAL-MISS.GDT 6/12/07 DD

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			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W _p W W _L			γ	GR SA SI CL
268.3	GROUND SURFACE							20 40 60 80 100									
0.0	Sand, trace gravel and silt (FILL) Loose to compact Brown Moist		1	SS	10		268										
266.8	CLAYEY SILT, some sand, trace gravel Soft to stiff Brown and grey Moist to wet		2	SS	4		267										
1.5			3	SS	11		266										
265.3																	
3.1	SILTY SAND, trace to some gravel and clay Loose Brown and grey Moist to wet		4	SS	8		265										
263.7							264										
4.6	SILTY SAND, some gravel contains cobbles and boulders (TILL) Very dense Brown and grey Moist		5	SS	70		263										29 32 32 7
262.2																	
6.1	CLAYEY SILT, with sand, trace gravel, contains cobbles and boulders (TILL) Hard Grey Moist		6	SS	76/0.15		262										
							261										
			7	SS	70/0.15		260										4 47 32 17
259.0																	
9.3	End of Borehole		8	SS	80/0.15												
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.																	

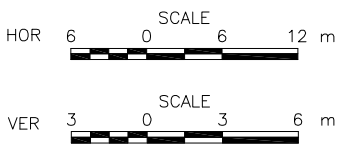
MIS-MTO 001 04-1111-024.GPJ GAL-MISS.GDT 6/12/07 DD



PLAN



SECTION A-A'



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

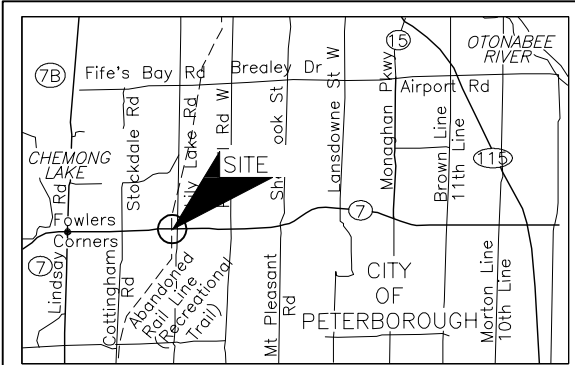
CONT No.
WP No. 73-99-00

HIGHWAY 7
RECREATIONAL TRAIL CULVERT
BOREHOLE LOCATIONS &
SOIL STRATA

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND			
	Borehole - Current Investigation		
	Approximate Borehole Location - Previous Investigation by Racey, MacCallum and Associates (1957)		
	Seal		
	Piezometer		
	N Standard Penetration Test Value		
	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)		
	WL in piezometer, measured on Aug 18, 2006		
No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
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

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 Contracts Documents.

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

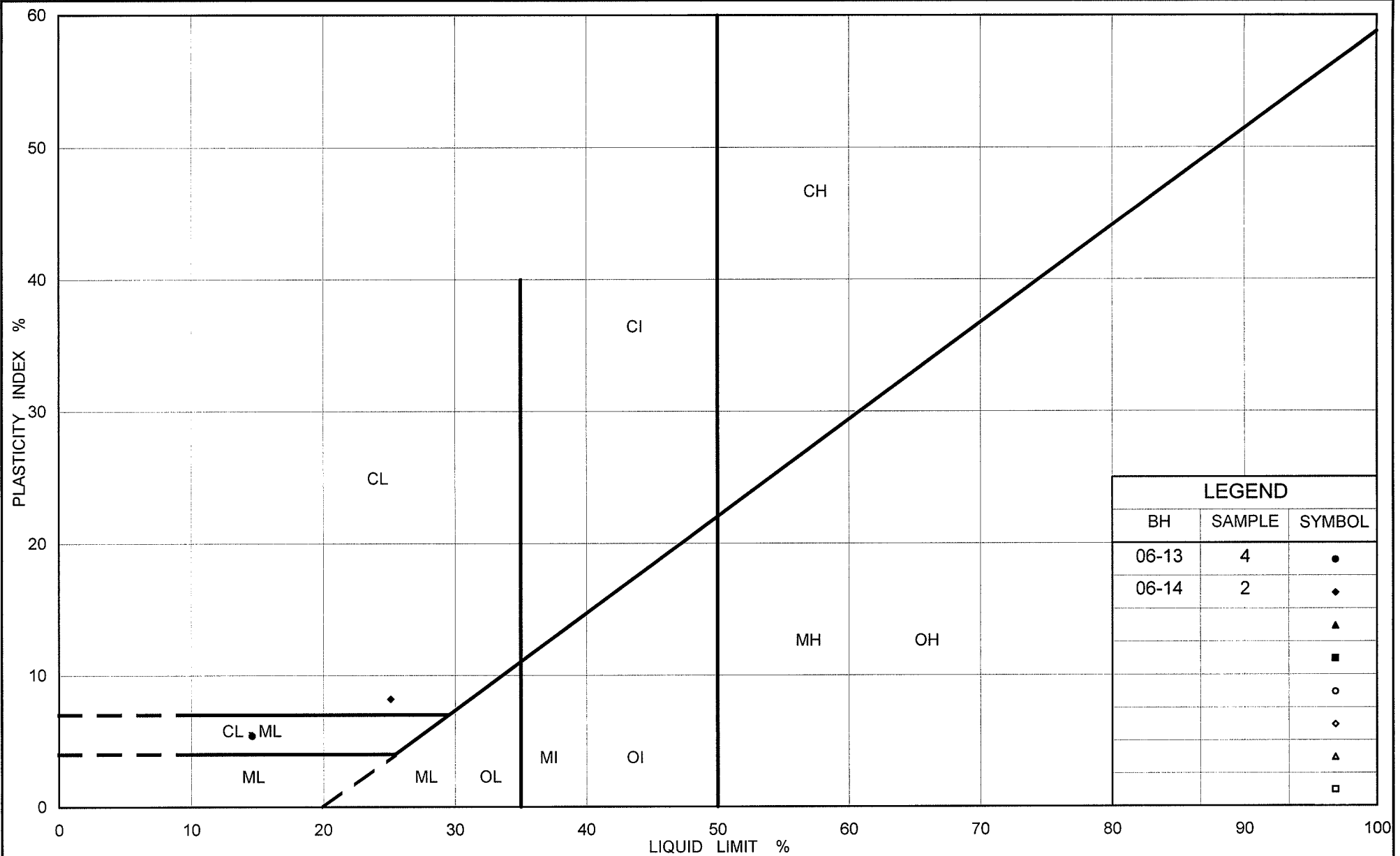
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

REFERENCE

Base plans provided in digital format by NCE, drawing file no. 2004-002 recommended plan.dwg, received July 31, 2006.

NO.	DATE	BY	REVISION
Geocres No. 31D-427			
HWY. 7	PROJECT NO. 04-1111-024B		DIST.
SUBM'D. SLP	CHKD. KJB	DATE: FEB 2007	SITE: 26-35
DRAWN: MSM	CHKD. KJB	APPD.	DWG. 1

APPENDIX A
LABORATORY TEST DATA



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt

FIG No. A1

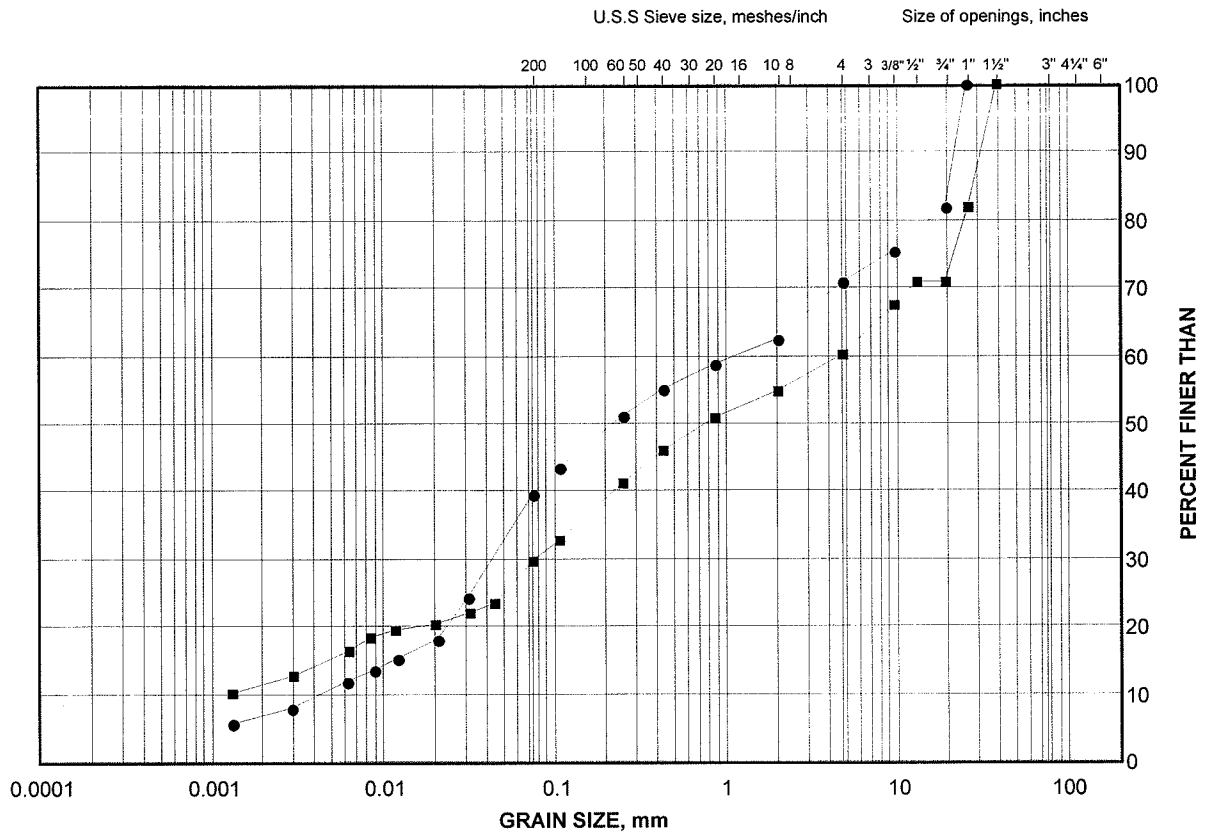
Project No. 04-1111-024B

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

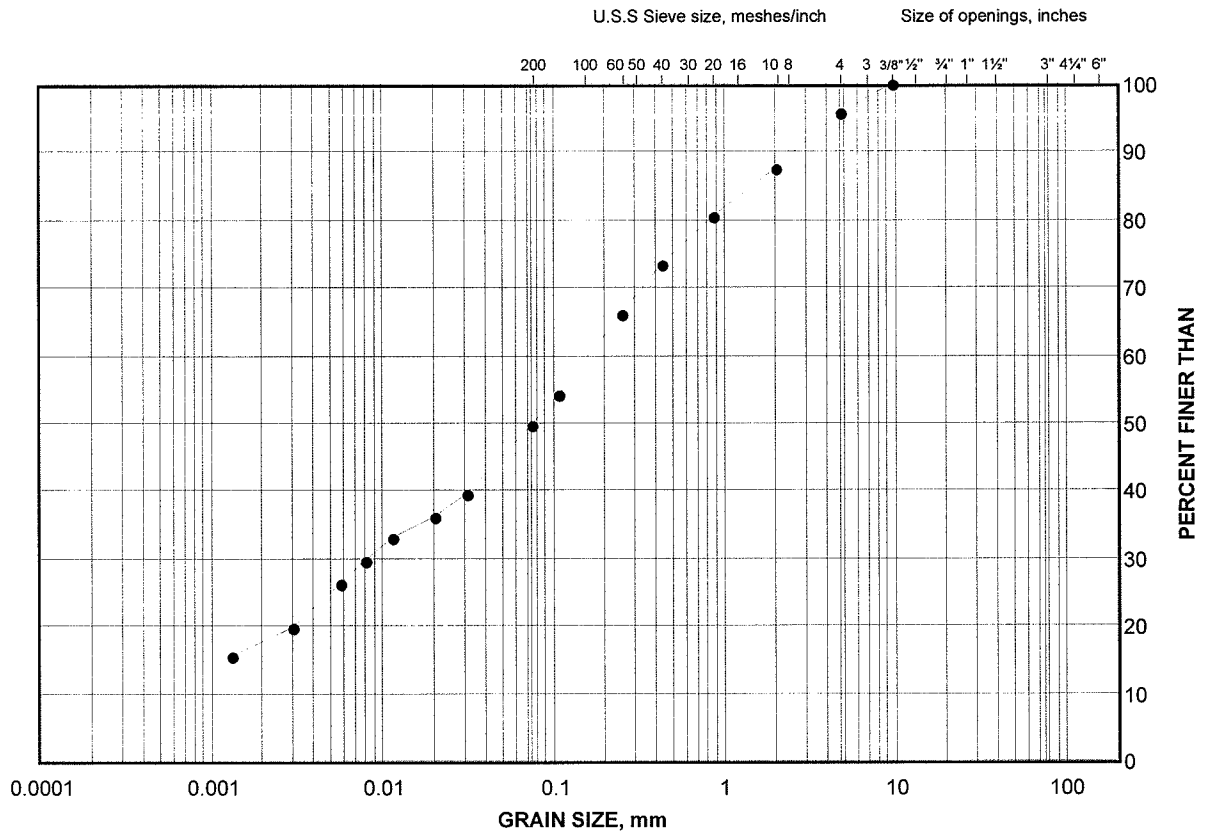
Golder Associates

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

APPENDIX B

**SUBSURFACE INFORMATION FROM
PREVIOUS 1957 INVESTIGATION**

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

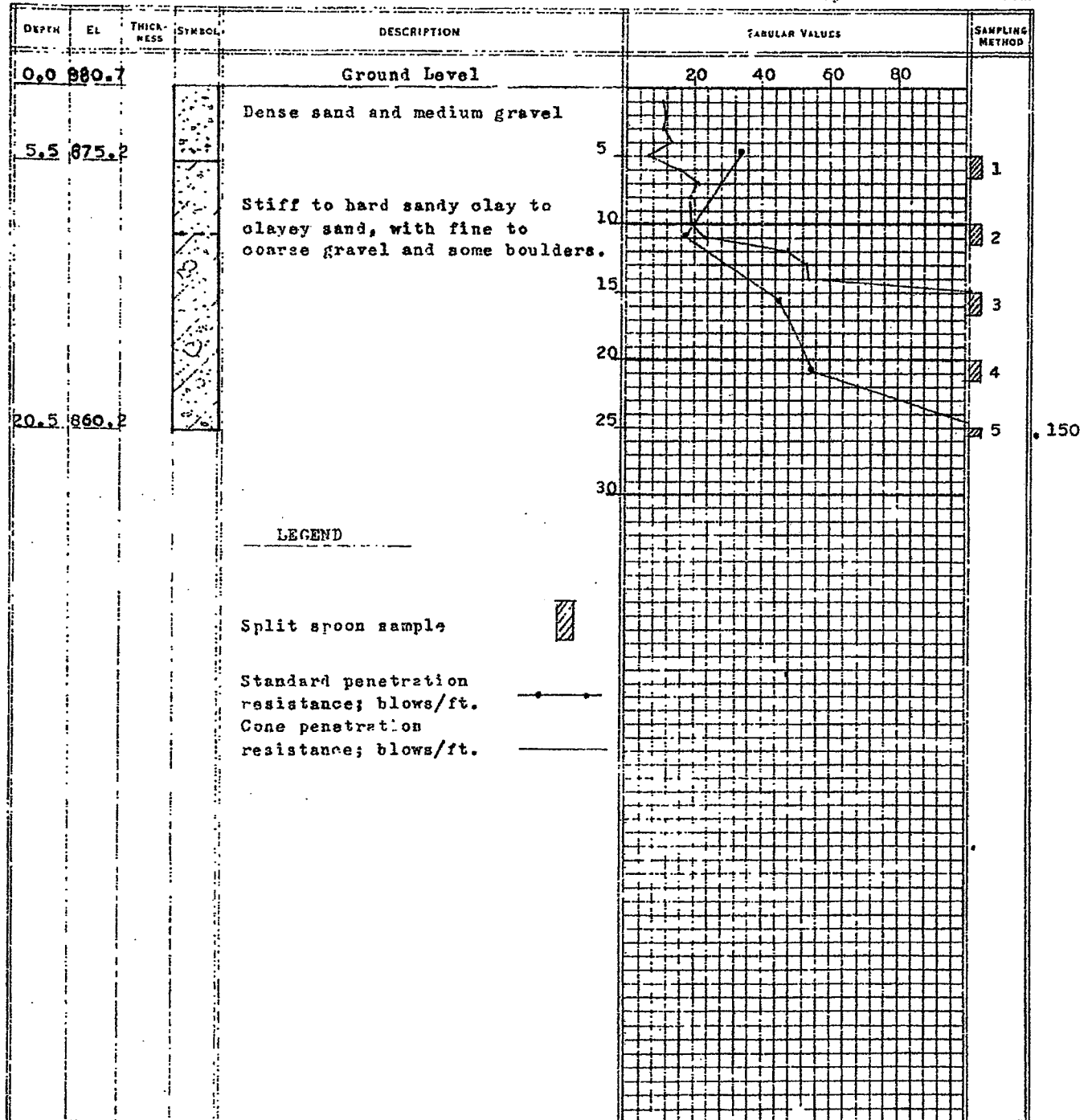
J.S.

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

