FOUNDATION INVESTIGATION
AND DESIGN REPORT
BEAVER CREEK BRIDGE REPLACEMENT
STRUCTURE SITE NO. 11-034
HIGHWAY 62 REHABILITATION FROM 5.3 KM
NORTH OF CLEVELAND ROAD TO 300 M SOUTH OF
COUNTY ROAD 620, BANCROFT, ONTARIO
G.W.P. 66-99-00

Submitted to:

Transenco Limited
69 Cleak Avenue, Box 187
Bancroft, Ontario
K0L 1C0

GEOCRES No. 31C-189

DISTRIBUTION:

2 Copies - GENIVAR, Bancroft, Ontario

3 Copies - Ministry of Transportation, Ontario – Kingston, Ontario

1 Copy - Ministry of Transportation, Ontario, Foundations Section – Downsview, Ontario

2 Copies - Golder Associates Ltd., Mississauga, Ontario

June 2009 07-1111-0044-1

TABLE OF CONTENTS

SECTION **PAGE**

LIST OF TABLES

LIST OF DRAWINGS

LIST OF FIGURES

LIST OF APPENDICES

PART A

FOUNDATION INVESTIGATION REPORT
BEAVER CREEK BRIDGE REPLACEMENT
STRUCTURE SITE NO. 11-034
HIGHWAY 62 REHABILITATION FROM
5.3 KM NORTH OF CLEVELAND ROAD TO
300 M SOUTH OF COUNTY ROAD 620
BANCROFT, ONTARIO
W.P. 66-99-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by GENIVAR on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the rehabilitation of Highway 62, including replacement of the existing Beaver Creek bridge, extending from about 5.3 km north of Cleveland Road to 300 m south of County Road 620, in the Townships of Tudor and Cashel. Foundation engineering services are required for the following components under G.W.P. 66-99-00:

- Replacement of the existing Beaver Creek bridge;
- Replacement or rehabilitation of seven existing pipe culverts; and
- Rehabilitation of embankment areas exhibiting pavement distress.

This report addresses the foundation engineering services for the replacement of the existing Beaver Creek bridge. The current investigation supplements the preliminary foundation investigation completed at the site in 2005-2006 by Jacques Whitford Limited (MTO GEOCRES No. 31C-172: *Preliminary Report, Foundation Investigation and Design, G.W.P. 248-99-00, Highway 62, Beaver Creek Bridge Replacement, Site No. 11-034*, dated August 22, 2006).

The terms of reference for the foundation engineering services are outlined in the MTO's Request for Proposal (RFP), dated April 4, 2007, for this assignment. The work was carried out in accordance with Golder's Quality Control Plan for this project, dated September 7, 2007.

2.0 SITE DESCRIPTION

The existing Beaver Creek bridge is located on Highway 62, approximately 1.2 km north of Weslemkoon Lake Road and 1.0 km south of Phillips Road, in the Township of Tudor. More broadly, the site is located about 35 km south of Bancroft, Ontario. Through this section, Highway 62 is a two-lane road with a rural cross-section.

Beaver Creek flows from the east to the west along a meandering channel; at its widest point at the site, the channel is approximately 35 m in width during non-flood conditions. At the time of the investigation, the water in the creek was estimated to be approximately 2 m to 3 m deep, with the water level at about Elevation 302.4 m. The natural ground surface to the north and south of the river is relatively flat, and just above the river level, at approximately Elevation 302.5 m to 302.6 m. No evidence of active erosion was observed along the river channel in the vicinity of the bridge site at the time of the current investigation.

The existing bridge, which was constructed in 1938, consists of a three-span steel girder structure with a concrete deck, approximately 25 m in total length. The existing structure foundations are supported on timber piles, approximately 10 m in length, that are founded on “possible hardpan” (a term historically applied to a hard, often clayey layer of soil, such as glacial till). The existing approach embankments are about 3 m in height, with side slopes oriented at approximately 2 horizontal to 1 vertical (2H:1V). Based on Golder’s observations at the site as part of the current investigation, the existing embankments appear to be stable and there is no evidence of differential settlement of the highway embankment relative to the bridge, although the maintenance history at this location is not known.

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out between November 6 and 8, 2007 and January 8 and 14, 2008. Three boreholes (Boreholes 07-22 to 07-24) and two seismic piezocone penetration test (SCPT) holes (SCPT 07-23 and SCPT 07-24) were advanced at the locations shown on Drawings 1 and 2. Borehole 07-22 was advanced through the existing north approach embankment, and the remaining boreholes were advanced in the area of the proposed south abutment. The boreholes were advanced using both truck- and track-mounted drill rigs supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario.

The boreholes were advanced to total depths of 17.0 m to 26.8 m below the existing ground surface. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth, increasing to 3 m intervals below a depth of about 15 m, using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. In situ vane shear strength testing was carried out using an MTO 'N'-sized vane, where "soft" clayey soils were encountered. Boreholes 07-22 and 07-23 were advanced into the bedrock using diamond drilling techniques (NQ-size rock coring barrel) for depths of 3.2 m and 3.1 m, respectively.

The water level in the open boreholes was observed throughout the drilling operations, and a standpipe piezometer was installed in Borehole 07-23 to monitor the groundwater level at the site. The screened portion of the standpipe was installed within the overburden soils, slightly above the bedrock surface, at a depth of approximately 16 m. The standpipe consists of a 20 mm diameter rigid PVC pipe with a 0.6 m long slotted screen section, installed within silica sand, and sealed below a minimum 1 m long sections of bentonite backfill. The remaining boreholes were backfilled with bentonite, in places mixed with native soils, and the site conditions restored following completion of work.

Two SCPT holes were advanced within the area of the proposed south abutment. The SCPT is an in situ technique for site characterisation studies. The SCPT consists of a special cone tip equipped with electronic sensing elements to continuously measure tip resistance, sleeve friction, dynamic porewater pressure, temperature, and cone inclination. It is pushed at a constant rate into the ground using a drill rig (ASTM D5778-95). A continuous stratigraphic profile together with engineering properties (such as strength, stress history, density, and shear wave velocity) can be interpreted from the results of the SCPT.

The SCPT work was carried out by ConeTec Investigations Ltd. The SCPT equipment was advanced using the hydraulic ram system on a track-mounted drill rig. The two SCPTs were advanced to refusal, which was encountered at depths of about 13.4 m and 12.1 m, respectively, below the existing ground surface. Seismic Cone Penetration Test records are included with the borehole records following the text of this report, and the complete field data report prepared by ConeTec Investigations Ltd. is provided in Appendix B. Profiles of tip resistance, friction ration, sleeve friction and dynamic pore pressure during pushing are presented together with an interpreted shear wave velocity profile and inferred soil type (stratigraphy).

The conventional borehole drilling and SCPT field work were supervised on a full-time basis by members of Golder's staff who located the boreholes and SCPTs, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil and bedrock samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Ottawa for further examination and laboratory testing. Laboratory testing, including water content determinations, Atterberg Limits testing, and grain size distribution analyses, was carried out on selected soil samples.

The borehole locations were determined by Golder relative to existing site features. The ground surface elevations at the borehole locations were determined by GENIVAR from a digital terrain model, based on the locations provided by Golder. The borehole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to geodetic datum, are summarized in the following table and are shown on Drawings 1 and 2.

Borehole No.	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
05-4	4,964,939.9	215,015.7	302.4
07-22	4,964,925.7	215,005.5	304.6
07-23	4,964,890.5	215,044.0	302.6
07-24	4,964,889.0	215,029.9	302.6
SCPT 07-23	4,964,890.1	215,041.1	302.6
SCPT 07-24	4,964,887.5	215,029.9	302.6

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The study area for this assignment lies within the physiographic region known as the Algonquin Highlands, as delineated by Chapman and Putnam in *The Physiography of Southern Ontario*[1].

The Algonquin Highlands region is characterized by frequent outcrops of granite and other strong Precambrian bedrock. The outcrops can extend as high as 160 m above the surrounding land. The thickness of soils over the bedrock can vary greatly over short distances, with many of the valleys between the bedrock outcrops floored with outwashed sand, silt and gravel. Several areas within this region have deeper deposits of glacial till with few bedrock outcrops.

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, three boreholes and two SCPTs were advanced within the general limits of the foundation elements for the proposed structure. The borehole locations and ground surface elevations are shown on Drawings 1 and 2.

The detailed subsurface soil, bedrock, and groundwater conditions encountered in the boreholes and inferred from the SCPT's, together with the results of laboratory testing carried out on selected samples, are given on the attached Record of Borehole, Drillhole and Seismic Cone Penetration Test sheets following the text of this report; the results of the laboratory testing are provided on Figures 1 to 5. Records for the five boreholes that were advanced at the site as part of the 2005-2006 preliminary investigation (GEOCRES No. 31C-172) are provided in Appendix A. The field investigation report prepared by ConeTec Investigations Ltd. for the seismic cone penetration testing is provided in Appendix B.

The stratigraphic boundaries shown on all 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. Furthermore, subsurface conditions will vary between and beyond the borehole locations.

In summary, the native soils below the existing highway embankment fill consist of a surficial sand to silt deposit overlying a thick layered deposit of silt, sand, and clay, overlying a glacial till deposit. These overburden materials are underlain by bedrock, the surface of which was encountered between Elevations 285.7 m and 281.1 m (at depths of approximately 16.9 m to 23.6

---

[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.

m below the ground surface at the borehole locations). A more detailed description of the subsurface conditions encountered in the boreholes is presented in the following sections.

4.2.1 Embankment Fill

Borehole 07-22 was advanced through the existing Highway 62 embankment at the northeast corner of the existing bridge. Below about 200 mm of asphaltic concrete pavement, the borehole encountered 1.3 m of sand and gravel base and sub-base fill, which contains some silt and cobbles with depth; the result of a grain size distribution test completed on one sample of this granular fill is shown on Figure 1. One SPT "N" value of 15 blows per 0.3 m of penetration was measured within the sub-base, indicating that this fill has a compact relative density.

The sand and gravel fill is underlain by approximately 2.2 m of sandy silt to silty sand fill, containing some gravel, trace clay, and trace organic matter. The measured SPT "N" values range from 3 to 5 blows per 0.3 m of penetration, indicating that this fill has a very loose to loose relative density.

4.2.2 Topsoil/Organic Matter

Approximately 400 mm and 500 mm of topsoil/organic matter was encountered immediately below the ground surface in Boreholes 07-23 and 07-24, respectively, which were advanced near the proposed south abutment.

4.2.3 Surficial Sand to Silt

A surficial deposit of sand to silt was encountered below the embankment fill in Borehole 07-22, below the topsoil/organic matter in Boreholes 07-23 and 07-24, immediately below the ground surface in Boreholes 05-1 and 05-4, and below the base of the Beaver Creek channel in Boreholes 05-2, 05-3 and 06-3. This deposit varies from about 0.8 m to 3.3 m in thickness, with its base encountered between Elevations 298.2 m and 302.0 m.

The deposit ranges in composition from sand containing trace to some silt, to silty sand containing seams of sandy silt and silty clay, to sandy silt containing trace clay, to silt containing some sand and trace clay; trace organic matter was observed in the upper portion of this deposit in Borehole 07-24 and Boreholes 05-1 to 05-4. The results of grain size distribution tests completed on four selected samples of the surficial sand to silt deposit (obtained as part of the current investigation) are shown on Figure 2.

The measured SPT "N" values in the surficial sand to silt deposit range from 1 to 9 blows per 0.3 m of penetration, indicating that this deposit has a very loose to loose relative density.

4.2.4 Layered Silts, Clays, and Sands

The surficial sand to silt deposit is underlain by a layered deposit of silts, clays and sands. The surface of the layered deposit was encountered between Elevations 298.2 m and 302.0 m (at a depth of 0.8 m to 6.4 m below the existing ground surface or creek bed at the borehole locations). This layered deposit was fully penetrated in Boreholes 07-22 to 07-24, which were advanced as part of the current investigation, and in Boreholes 05-2, 05-3 and 06-3, which were advanced as part of the preliminary investigation; Boreholes 05-1 and 05-4 from the preliminary investigation were terminated within this deposit. Where the deposit was fully penetrated, it was found to be between 9.6 m and 12.1 m in thickness, extending to between Elevation 286.6 m and 290.0 m at the base of the deposit.

The deposit consists of interlayers of silt, clayey silt, silty clay, sandy silt, silty sand and sand. The results of grain size distribution tests conducted on eight selected samples from the layered deposit (obtained from the current investigation) are shown on Figures 3A and 3B. Atterberg limits testing was conducted on five samples of the layered deposit (obtained from the current investigation). Two of the samples were determined to be non-plastic silts. The remaining three samples had plastic limits of 17 to 20 per cent, liquid limits of 23 to 35 per cent, and plasticity indices of 6 to 15 per cent; these results confirm that the tested cohesive soils are clayey silt of low plasticity. All of the Atterberg limits test results are plotted on a plasticity chart on Figure 5.

The measured SPT "N" values within the layered deposit range from 0 blows (weight of rods) to 19 blows per 0.3 m of penetration, but are typically between 1 and 10 blows per 0.3 m of penetration, indicating that the layered deposit has a very loose to compact (but typically very loose to loose) relative density. In situ vane testing carried out in the clayey silt to silty clay layers within this deposit measured undrained shear strengths ranging from approximately 40 kPa to 62 kPa, indicating that the clayey silt to silty clay layers have a firm to stiff consistency. Based on the measured remoulded shear strengths of 12 kPa to 20 kPa, corresponding to sensitivities of 2.4 to 4.3, the clayey silt to silty clay is moderately sensitive.

4.2.5 Sand to Sand and Gravel Till

The layered silt, clay and sand deposit is underlain by a deposit of glacial till, which was encountered in Boreholes 07-22 to 07-24 and 05-2, and inferred in Boreholes 05-3 and 06-3; the surface of the till deposit was encountered between Elevations 286.6 m and 290.0 m (at depths of 12.7 m to 18.0 m below the ground surface at the borehole locations) in these boreholes. Boreholes 05-1 and 05-4 were terminated above the till deposit. Where fully penetrated in Boreholes 07-22 to 07-24, the till deposit is about 3.3 m to 5.6 m in thickness.

In Boreholes 07-22 to 07-24, the till deposit varies in composition from sand and gravel containing trace to some silt, to sand containing some silt and trace to some gravel, to silty sand containing trace to some gravel; cobbles were noted within the till deposit based on auger performance and observation of the cuttings. In Boreholes 05-2 and 05-3 from the preliminary investigation, the till deposit is described as a sandy silt containing some gravel and cobbles; however, no grain size distribution tests were completed on this material as part of the preliminary investigation. The results of grain size distribution testing completed on three samples of the till deposit (obtained as part of the current investigation) are shown on Figure 5.

The measured SPT "N" values in the till deposit ranged from 8 to 60 blows per 0.3 m of penetration, indicating that the deposit has a loose to very dense relative density. The higher SPT "N" values may, however, reflect the presence of cobbles and/or boulders in the deposit, rather than the state of packing of the soil matrix.

4.2.7 Dolomite and Marble Bedrock

Dolomite bedrock was encountered below the sand and gravel till deposit in Borehole 07-22 on the north side of the river, and marble bedrock was encountered below the sand and gravel till in Borehole 07-23 on the south side of the river. The bedrock in the area consists predominantly of crystalline limestone and dolomite. However, intrusive igneous deposits and products of contact metamorphism are present in the area; hence, the presence of both sedimentary and metamorphic bedrock at this site.

Bedrock was proved by coring for depths of more than 3 m in both Boreholes 07-22 and 07-23; in Borehole 07-24, also on the south side of the river, the bedrock surface elevation has been inferred based on auger refusal. The following table summarizes the bedrock surface depth and elevation as encountered at the borehole locations.

Borehole No.	Ground Surface Elevation	Depth to Bedrock	Bedrock Surface Elevation
07-22	304.6 m	23.6 m	281.1 m (Cored)
07-23	302.6 m	16.9 m	285.7 m (Cored)
07-24	302.6 m	17.0 m	285.6 m (Refusal)

A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet that precedes the borehole records following the text of this report.

In the north portion of the site, the encountered dolomite bedrock is grey, generally unweathered (fresh), and medium strong to strong. In the south portion of the site, the encountered marble bedrock is white, grey and pink, generally unweathered (fresh), and medium strong to strong. The Rock Quality Designation (RQD) values measured on recovered bedrock core samples ranged from 59 to 92 per cent, indicating fair to excellent quality rock.

4.2.8 Groundwater Conditions

The site soils, including the surficial sand to silt deposit, are water-bearing, with the water level during and immediately following completion of overburden drilling measured at approximately Elevation 302.0 m to 302.4 m (i.e., near the natural ground surface at the site).

A piezometer was installed in Borehole 07-23, sealed into the sand and gravel till deposit that overlies the bedrock; details of the piezometer installation are shown on the borehole record. The measured groundwater level in the piezometer is summarized in the following table; this water level is artesian with respect to the ground surface in the floodplain area, although flowing artesian conditions were not encountered during borehole drilling.

Date	Depth	Elevation
January 11, 2008	0.8 m above ground surface	303.4 m

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

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Troy Skinner, P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder. Mr. Fin Heffernan, P.Eng., the Designated MTO Contact for this project, reviewed the technical and quality aspects of the report.

GOLDER ASSOCIATES LTD.

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

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

TMS/LCC/FJH/tms/lcc
n:\active\2007\1111\07-1111-0044 transenco hwy 62 tudor twp\6 - reports\final reports\bridge\07-1111-0044 rpt01 09jun beaver creek bridge.doc

PART B

FOUNDATION DESIGN REPORT
BEAVER CREEK BRIDGE REPLACEMENT
STRUCTURE SITE NO. 11-034
HIGHWAY 62 REHABILITATION FROM
5.3 KM NORTH OF CLEVELAND ROAD TO
300 M SOUTH OF COUNTY ROAD 620
BANCROFT, ONTARIO
W.P. 66-99-00

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed replacement of the existing bridge that carries Highway 62 over Beaver Creek. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigations 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 design the proposed structure foundations. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents.

Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Bridge Foundation Options

The existing Beaver Creek bridge, which was constructed in 1938, is a three-span steel girder structure with a concrete deck that is approximately 25 m in total length (consisting of end spans of approximately 8.4 m, and a centre span of about 8.2 m). The existing structure foundations are supported on timber piles, approximately 300 mm in diameter and 10 m in length, that are founded on "possible hardpan" (a term historically applied to a hard, often clayey layer of soil, such as glacial till). The existing bridge is to be replaced with a single-span structure, to be located approximately 15 m east of the existing bridge. The existing bridge is to remain in service during construction of the new bridge, and will be removed following completion of the new bridge.

Based on the high groundwater table at the site and the generally loose relative density of the surficial sand to silt deposit and underlying layered deposit, shallow foundations are not recommended for support of the new bridge foundation elements, including any associated retaining walls. The bearing resistance of the near-surface soils is insufficient for support of the abutment loads, and the settlement of the foundations would exceed acceptable levels.

Deep foundations will, therefore, be required for support of the new abutments, and are also recommended for support of any associated retaining walls. It is considered that driven steel H-piles founded on the bedrock represent the most feasible and cost-effective foundation option for support of the new bridge structure, and this foundation type is the preferred alternative from a foundations perspective. Steel H-pile foundations could be adopted with either an integral abutment or conventional abutment configuration, as either configuration would be acceptable at the site from a geotechnical perspective.

As an alternative to driven steel H-pile foundations, caissons founded on or socketted nominally into the bedrock have been considered. However, caissons will be more difficult to construct than steel H-piles, as it will be necessary to use a temporary or permanent liner during construction to minimize running or flowing of the water-bearing soils into the caisson hole; in addition, it will be necessary to socket the caissons at least 0.5 m into the strong bedrock that exists at the site, to eliminate the potential for loss of ground immediately overlying the bedrock (which could occur if a “gap” existed between the bottom of the liner and the uneven bedrock surface).

Geotechnical recommendations for the design of foundations for the bridge abutments are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the foundation options is presented in Table 1 following the text of this report.

6.3 Steel H-Pile Foundations

6.3.1 Founding Elevation

Steel H-piles driven to found on the bedrock surface are recommended for support of the bridge abutments, as well as for any associated retaining walls. For design, the following pile tip elevations may be assumed, based on consideration of the elevation of the bedrock surface as encountered in Boreholes 07-22 to 07-24:

Foundation Element	***Design Pile Tip Elevation***
North Abutment and Retaining Walls	281 m
South Abutment and Retaining Walls	285.5 m

The abutment pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection. Based on the natural ground surface elevation, it is anticipated that the piles for the north and south abutments will be approximately 20 m and 15 m in length, respectively.

In the installation of steel H-piles, consideration must be given to the potential presence of cobbles and boulders within glacial till deposit that overlies the bedrock at this site. It is recommended that all piles be fitted with Titus injector rock bearing points (or equivalent), in accordance with the manufacturer’s specifications, for protection during pile driving and to ensure adequate seating of the piles on the bedrock; this requirement should be noted on the Contract Drawings.

Pile installation should be in accordance with MTO's Special Provision SP903S01. For this site, the piles will be driven to practical refusal on the bedrock. The drawings should incorporate the appropriate note stating that the piles should be equipped with Titus injector points or equivalent, and should be driven to bedrock. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile. All of these factors must be taken into consideration in establishing the driving criteria to ensure that the piles are not overdriven and to avoid damage to the piles. In this regard, for piles driven to refusal on bedrock, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and then to gradually increase the energy over a series of blows to seat the pile.

6.3.2 Axial Geotechnical Resistance

The following factored axial geotechnical resistances at Ultimate Limit States (ULS) may be assumed for design of piles driven to found on the bedrock surface:

Pile Size	Factored Axial Geotechnical Resistance at ULS
HP 310x110	2,000 kN
HP 310x132	2,200 kN
HP 360x132	2,400 kN
HP 360x152	2,750 kN

The above values represent structural limitations for the piles rather than geotechnical limitations. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

As discussed further in Section 6.7.3 (*Approach Embankment Settlement*), it is estimated that up to approximately 50 mm of settlement of the native soils will occur under the new, 3 m to 3.5 m high embankment loading; this settlement will occur relatively rapidly, during and immediately following completion of the embankment construction. Because of the proximity of the abutments to the edge of Beaver Creek, there is not room to preload the abutment areas before pile driving and, therefore, downdrag loads must be taken into account in the design of deep foundations. In addition, as discussed further in Section 6.5.4 (*Summary of Results – Liquefaction and Seismic Settlement*), it is estimated that between 35 mm and 40 mm of ground settlement would be induced by seismic shaking.

In calculating the magnitude of the downdrag force, the methods described in both the Canadian Foundation Engineering Manual as well as the US Transportation Research Board's report, "Design and Construction Manual For Downdrag on Uncoated and Bitumen-Coated Piles" [Briaud and Tucker (1994)] were considered. Considering the larger predicted settlement of the layered silt deposit versus the elastic shortening of the pile, the neutral plane used in the analyses was assumed to be at the base of the layered silt deposit.

Based on the above, the unfactored downdrag load acting on a single HP 310x110 pile (as a result of either the initial embankment construction or seismic settlement) is estimated to be 350 kN. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *CHBDC.*

6.3.3 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. Alternatively, if vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles. It may be assumed that this resistance will be nearly the same for vertical and inclined piles, as indicated in Section C6.8.7.2 of the Commentary to the CHBDC.

The resistance to lateral loading in front of the piles may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction is based on the equations given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (3rd Edition).

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where
k_h is the coefficient of horizontal subgrade reaction (MPa/m);
n_h is the constant of subgrade reaction (MPa/m);
z is the depth (m); and
B is the pile diameter/width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

where
k_h is the coefficient of horizontal subgrade reaction (kPa/m);
s_u is the undrained shear strength of the soil (kPa); and
B is the pile diameter/width (m).

The following values of n_h and s_u may be assumed in the structural analysis.

Soil Unit	n_h	s_u
North abutment:		
Fill above Elevation 302 m	5 MPa/m	-
Very loose to loose surficial sand/silt, Elevation 302 m to 298 m	2 MPa/m	-
Very loose to loose layered sand/silt, Elevation 298 m to 295.5 m	3 MPa/m	-
Firm to stiff clayey silt to silty clay, Elevation 295.5 m to 294 m	-	50 kPa
Very loose to loose layered sand/silt, Elevation 294 m to 286.5 m	3 MPa/m	-
Compact to very dense sand and gravel till, Elevation 286.5 m to 281 m	10 MPa/m	-
Dolomite bedrock below Elevation 281 m		

Soil Unit	n_h	s_u
South abutment:		
Fill above Elevation 302 m	5 MPa/m	-
Very loose to loose sand/silt, Elevation 302 m to 298 m	2 MPa/m	-
Firm to stiff clayey silt to silty clay, Elevation 298 m to 295 m	-	50 kPa
Loose layered sand/silt, Elevation 295 m to 289 m	3 MPa/m	-
Loose to dense silty sand to sand and gravel till, Elevation 289 m to 285.6 m	5 MPa/m	-
Marble bedrock below Elevation 285.6 m		

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R, as follows:

Pile Spacing in direction of Loading (d = Pile Diameter)	*Subgrade Reaction Reduction Factor*
8d	1.00
6d	0.70
4d	0.40
3d	0.25

Reference: Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2. Department of the Navy, Naval Facilities Engineering Command (1982).

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above table.

For establishing the factored *structural* resistance at ULS, the shear force and bending moment distribution in the piles under factored loading can be established using the procedures and parameters given above for evaluating the SLS response of the piles.

The factored *geotechnical* resistance ULS for lateral loading may be calculated using passive earth pressure theory as outlined in Section C6.8.7 of the *Commentary to the CHBDC*. For individual piles in non-cohesive soils (i.e., sands and till) the passive resistance may be assumed to act over the pile shaft to a depth equal to six pile diameters below the underside of the pile cap and may be calculated using the following equations:

Above the water table: $P_p(z) = 3\ K_p\,\gamma\ z$

Below the water table: $P_p(z) = 3\ K_p\,\gamma\ D_w + 3\ K_p\ (z - D_w)\ (\gamma - \gamma_w)$

where $P_p(z)$ is the factored lateral resistance at ULS at depth ‘z’ below ground surface (kN);
γ is the average unit weight of overlying soil, from table below (kN/m^3);
K_p is the coefficient of passive earth pressure, from table below;
D_w is the depth to groundwater table below ground surface (m); and
γ_w is the unit weight of water, taken as 9.8 kN/m^3.

In cohesive soils (i.e., silty clay and clay) the lateral resistance is assumed to vary linearly from a magnitude of $2s_u$ at the surface of the deposit to a magnitude of $9s_u$ at a depth equal to three pile diameters below the underside of the pile cap, where S_u is the undrained shear strength. Below a depth equal to three pile diameters, the lateral resistance is assumed to be constant at $9S_u$.

The following values for γ, K_p and s_u may be assumed to assess the factored geotechnical lateral resistances at ULS:

Soil Unit	γ *(kN/m³)*	K_p	s_u *(kPa)*
North abutment:			
Fill above Elevation 302 m	20	3.0	-
Very loose to loose surficial sand/silt, Elevation 302 m to 298 m	19	2.8	-
Very loose to loose layered sand/silt, Elevation 298 m to 295.5 m	19	2.8	-
Firm to stiff clayey silt to silty clay, Elevation 295.5 m to 294 m	19	-	50
Very loose to loose layered sand/silt, Elevation 294 m to 286.5 m	19	2.8	-
Compact to very dense sand and gravel till, Elevation 286.5 m to 281 m	21	3.7	-
Dolomite bedrock below Elevation 281m			
South abutment:			
Fill above Elevation 302 m	20	3.0	-
Very loose to loose sand/silt, Elevation 302 m to 298 m	19	2.8	-
Firm to stiff clayey silt to silty clay, Elevation 298 m to 295 m	19	-	50
Loose layered sand/silt, Elevation 295 m to 289 m	19	2.8	-
Loose to dense silty sand to sand and gravel till, Elevation 289 m to 285.6 m	21	3.7	-
Marble bedrock below Elevation 285.6 m			

The ULS lateral resistance of a pile group may be estimated as the sum of the individual pile resistances across the face of the pile group, perpendicular to the direction of the applied lateral force.

The ULS resistances obtained using the above parameters represent unfactored values; in accordance with the *CHBDC*, a resistance factor of 0.5 is to be applied in calculating the horizontal resistance.

A maximum lateral resistance of 105 kN at ULS, and a maximum lateral resistance of 35 kN at SLS (for 10 mm of horizontal deflection at pile cap level) is recommended for HP 310 x 110 piles, based on the "Assessed Horizontal Passive Resistance and Geotechnical Reaction at SLS" provided under Clause C6.8.7.1, Table C6.4 of the *Commentary on CHBDC*, using the values provided for piles with a flange width of 310 mm (i.e., HP 310x79 sections).

6.4 Caisson Foundations

As an alternative to steel H-piles, caissons could be considered for support of the new abutments and any associated wing walls/retaining walls. However, caissons will be more difficult to construct at this site than steel H-piles, since it will be necessary to use a temporary or permanent liner during construction to minimize running or flowing of the water-bearing soils into the caisson hole. Additionally, since the bedrock surface is variable and the liner will therefore not seat perfectly on the bedrock surface, it will be difficult to prevent running or flowing of water-bearings soils at the bedrock interface, and these soils will be difficult to clean from the bedrock surface. Therefore, it will be necessary to socket the caissons at least 0.5 m into the bedrock, rather than found on the bedrock surface. The bedrock is typically moderately strong to strong, so the sockets would have to be advanced into the rock by churn drilling or rock coring supplemented by down-hole hammer.

If caisson foundations are adopted for this site, it is recommended that an NSSP be included in the Contract Documents to address the need for control of the ground and groundwater during caisson construction, and to warn the Contractor of the bedrock strength as it may affect the caisson socketting operation; these recommendations are summarized under *Construction Considerations* in Section 6.8.

6.4.1 Founding Elevation

Caissons socketted approximately 0.5 m into the bedrock could be considered for support of the new bridge abutments, as well as for any associated retaining walls. For design, the following caisson base elevations may be assumed, based on consideration of the elevation of the bedrock surface as encountered in Boreholes 07-22 to 07-24:

Foundation Element	***Caisson Base Elevation***
North Abutment and Retaining Walls	280.5 m
South Abutment and Retaining Walls	285 m

The abutment pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection. Based on the natural ground surface elevation, it is anticipated that the caissons for the north and south abutments will be approximately 20 m and 15.5 m in length, respectively.

6.4.2 Axial Geotechnical Resistance

Caissons founded approximately 0.5 m into the bedrock should be designed based on end-bearing resistance, using a factored geotechnical resistance at ULS of 5 MPa. The geotechnical resistance at SLS does not apply to caissons founded on/socketted into the bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

6.4.3 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons, and the reductions due to group effects, may be determined as per Section 6.3.3.

6.5 Seismic Liquefaction Assessment

6.5.1 Background

Seismic liquefaction can occur when earthquake vibrations cause an increase in the pore water pressure within the soil, which reduces the effective stress between the soil particles and the soil's frictional resistance to shearing. This phenomenon, which leads to a temporary reduction in the shear strength of the soil, may cause the following:

- Large lateral movements of even gently sloping ground, referred to as "lateral spreading", which could impact embankment stability;

- Reduced shear resistance (i.e., bearing capacity) of soils that support foundations, as well as reduced resistance to sliding; and

- Reduced shaft resistance for deep foundations, as well as reduced resistance to lateral loading.

In addition, "seismic settlements" may occur once the vibrations and shear stresses have ceased. Seismic settlement is the process whereby the soils stabilize into a denser arrangement after an earthquake, causing potentially large surface settlements. This settlement can result in temporary downdrag forces on deep foundations.

The following soil types/conditions can be prone to experiencing seismic liquefaction:

- Coarse-grained soils (i.e., liquefaction is more probable for sands than for silts);
- Soils having a loose state of packing; and
- Soils located below the groundwater level.

6.5.2 Seismic Magnitude and Peak Ground Acceleration

Halchuk and Adams (2004) have suggested a mean seismic magnitude, M, of 6.1 for 2,400 earthquakes in the Montreal and Ottawa areas, and a mean seismic magnitude, M, of 5.7 in the Toronto area for de-aggregation analyses of the peak ground acceleration. A conservative M equal to 6.2 has been selected for soil liquefaction analyses for this project, as no specific de-aggregation data are available in the area of Bancroft.

According to the most updated earthquake records provided by the NRCan website for the site (latitude 45.05 degrees north and longitude 77.85 degrees west), a value of 0.065 g (g = acceleration due to gravity) applies for the peak ground acceleration, based on a 10 per cent exceedance probability in 50 years. Using a soil amplification factor of 1.37 based on the subsurface conditions at the site, a peak horizontal ground acceleration (PHGA) equal to 0.089g is applicable for the soil liquefaction analysis.

6.5.3 Liquefaction Analysis Methods

For this site, a seismic liquefaction assessment has been carried out consistent with the state of practice outlined by the National Center for Earthquake Engineering Research (NCEER). The assessment of the potential seismic liquefaction hazard at this site involves comparing the cyclic shear stresses applied to the soil by the design earthquake (represented by the cyclic stress ratio, CSR) to the cyclic shear strength offered by the soil (represented as the cyclic resistance ratio, CRR).

The CSR is primarily a function of the effective overburden pressure, the design ground acceleration, and the earthquake magnitude and ground acceleration specific to the site. The values of CSR have been estimated based on the empirical methods described in NCEER 1997.

The CRR is primarily related to the relative density of the soil and its gradation. The relative density of a soil is typically measured using in situ testing techniques such as the Standard Penetration Test (SPT), shear wave velocity and the seismic cone penetration test (SCPT). The analysis methods and results for each of these testing techniques are briefly described in the following sub-sections.

SPT Method

The empirical method based SPT results for cohesionless soils is consistent with that presented in the state of practice outlined by the National Center for Earthquake Engineering Research (NCEER, 1997). The method used to perform soil liquefaction assessment for cohesive soil is still being developed based on recent cases of strong earthquakes around the world (Boulanger and Idriss, 2004). Boulanger and Idriss suggest that fine-grained soils can be expected to exhibit clay-like behaviour if the plasticity index of the soil is greater than 7 per cent. Fine-grained soils having a plasticity index of less than 7 per cent should be considered as likely exhibiting sand-like behaviour (i.e., liquefiable), unless shown otherwise through appropriate in situ and laboratory tests.

According to the methods identified above and the subsurface conditions encountered in Boreholes 07-22 to 07-24, the factor of safety against soil liquefaction is greater than 1.0.

Shear Wave Velocity Method

In the state of practice outlined by NCEER (1997), soil liquefaction analysis methods also include assessment using the results of shear wave velocity testing in the subsurface deposits. Shear wave velocities were measured in SCPT 07-23 and SCPT 07-24 for soils at depths ranging from 2.9 m to 11.9 m.

The factors of safety against soil liquefaction have been analyzed using the measured shear wave velocities and are calculated to range from approximately 3.0 to 7.9 between depths of about 3 m and 12 m in SCPT 07-23 and SCPT 07-24, within the interpreted interlayered sand/silty sand/silt deposit.

According to the methods identified above and the data from SCPT 07-23 and SCPT 07-24, the factor of safety against soil liquefaction is greater than 1.0.

CPT Method

The seismic piezocone penetration resistance profiles completed in SCPT 07-23 and SCPT 07-24 have been used for soil liquefaction analysis. The method used to perform the liquefaction assessment is consistent with that presented in NCEER (1997) and by Robertson and Wride (1998).

Robertson and Wride (1998) developed a chart for determining cyclic resistance ratio ($CRR_{7.5}$) for clean sands (having a fines content less than or equal to 5 per cent), based on SCPT data collected from sites where liquefaction effects were or were not observed following earthquakes. The chart shows calculated CRR plotted as a function of corrected and normalized CPT resistance (q_{c1N}) and separates regions of the plot with data indicative of liquefaction from regions with data indicative of non-liquefaction.

Based on the data from SCPT 07-23 and SCPT 07-24, and the method described above, the factor of safety against soil liquefaction is greater than 1.0 at this site.

6.5.4 Summary of Results – Liquefaction and Seismic Settlement

Based on the analysis results discussed in Section 6.5.3, no soil liquefaction concerns need to be considered for the bridge site for the design earthquake magnitude and peak horizontal ground acceleration.

The ground settlement induced by seismic shaking has been assessed based on SPT “N” values, using methods presented in Tokimatsu and Seed (1984 and 1987). The estimated total seismic settlement below the pile cap level is approximately 35 mm to 40 mm; the potential differential settlement across a foundation element or embankment area will be about 50 to 65 per cent of the total estimated seismic settlement. The accuracy of this estimated seismic settlement is considered to be within 25 to 50 per cent of the actual settlement magnitude.

Based on the estimated seismic settlement, negative skin friction (downdrag loads) should be considered for deep foundation design, as discussed in Section 6.3.2.

6.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls or 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. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the walls. 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 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in accordance with MTO Special Provision SP105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the abutment stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with MTO’s 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 1.8 m behind the back of the wall stem (Case I, Figure C6.20(a) of the *Commentary on CHBDC*) or within a 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, Figure C6.20(b) of the *Commentary on CHBDC).*

- For Case I, the pressures are based on the existing and/or new embankment fill materials, and the following parameters (unfactored) may be used assuming the use of Select Subgrade material for the new embankment construction:

Soil unit weight:	20 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.35
At rest, K_o	0.50

- For Case II, the pressures are based on the granular fill placed behind the abutments, 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, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as follows in accordance with Section C6.9.1 of the *Commentary to CHBDC*:

  - rotation (i.e. ratio of wall movement to wall height) of approximately 0.002 about the base of a vertical wall;
  - horizontal translation of 0.001 times the height of the wall; or
  - a combination of both.

6.6.1 Seismic Considerations

Seismic (earthquake) loading must be considered in the design in accordance with Section 4.6.4 of *CHBDC,* as significant seismic loading will result in increased lateral earth pressures acting on the abutment stem and any associated wing walls/retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the applicable earthquake-induced dynamic earth pressure. The earthquake-induced dynamic 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) can be determined using the following equation:

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

where
- K = either the static active earth pressure coefficient (K_a) or the static at-rest earth pressure coefficient (K_o);
- K_{AE} = the seismic active earth pressure coefficient determined in accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its *Commentary,* as given below;
- γ' = the effective unit weight of the backfill soil (kN/m^3), as given previously;
- d = the investigated depth below the top of the wall (m); and
- H = the total height of the wall above the underside of footing/ pile cap or toe (m).

Using an amplified zonal acceleration ratio of 0.089g for this site, the seismic lateral earth pressure coefficients (K_{AE}) for both yielding and non-yielding walls, considering earth and granular fills, were determined in accordance with Sections 4.6.4 and C4.6.4 of the *CHBDC* and its *Commentary,* and these are presented below for the two backfill cases (Case I and Case II as described above). These seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is essentially flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
	Earth Fill	Granular A	Granular B Type II
Yielding wall [1]	0.33	0.27	0.27
Non-yielding wall	0.40	0.34	0.34

[1] 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 0.089. This corresponds to a displacement of approximately 20 mm to 25 mm at this site.

6.7 Approach Embankment Design and Construction

The realigned Highway 62 approach embankments for the new Beaver Creek bridge structure will be approximately 3 m to 4 m in height relative to the surrounding natural ground surface. It is recommended that any surficial topsoil or organic material be stripped from beneath the footprint of the new embankment prior to fill placement.

The new embankment fill could consist of select subgrade material or rock fill. If integral abutments are adopted, rock fill should not be placed within the active wedge zone. As rock fill contains numerous voids into which finer material can migrate due to water action and/or repeated loading, a filter material is required at the transition between the rock fill and the abutment backfill, or at any transition between rock fill and earth embankment fill. In this regard, Granular B Type II (OPSS 1010) meets the criteria for filtration and drainage and therefore could be used as backfill to the abutment without additional filter requirements (per MTO's directive, "Backfill to Structures Adjacent to Rock Embankment Approaches", dated November 2002).

Embankment fill should be placed and compacted in accordance with MTO's Special Provision SP105S10.

6.7.2 Approach Embankment Stability

The following sections outline the methods and parameters used to assess the static and seismic stability of the approach embankment side slopes, and the results of the stability analyses.

Analysis Methods – Static Stability

Static slope stability analyses were performed for the approach embankment side slopes, assuming an embankment height of approximately 3 m to 3.5 m.

The static slope stability analyses were performed using the commercially available program SLOPE/W produced by Geo-Slope International Ltd., employing the Morgenstern-Price method. For all analyses, the factor of safety of potential failure surfaces was computed to establish the minimum factor of safety. A target factor of safety of 1.3 against deep-seated, global failure that would affect the operation of the highway/bridge is normally used for the design of embankment slopes under static conditions. This factor of safety is considered appropriate for the embankment side slopes at this site, considering the design requirements and the field data available.

Effective stress parameters were employed in the static stability analyses assuming drained conditions for the soils. The effective stress parameters (effective friction angle and cohesion) for these soils were estimated from empirical correlations using the results of in situ Standard Penetration Tests (SPT) and Atterberg limits, in conjunction with engineering judgement considering experience in similar soil conditions. Static global stability was also checked using undrained parameters for the firm to stiff clayey silt to silty clay layer, based on the measured undrained shear strength in this deposit. The soil parameters that have been used in the stability analyses are summarized below.

Material	Bulk Unit Weight (kN/m^3)	Effective Friction Angle	Undrained Shear Strength (kPa)
New earth embankment fill	21	32º	-
New rock embankment fill	18	38º	-
Very loose to loose surficial sand/silt deposit	19	28º	-
Very loose to loose, upper layered sand/silt deposit	19	28º	-
Firm to stiff clayey silt to silty clay	19	28º	45
Loose, lower layered sand/silt deposit	19	28º	-
Loose to dense sand to sand and gravel till	21	35º	-

Analysis Methods – Embankment Stability Under Seismic Conditions

If liquefaction of the subsoils is not anticipated, the stability of the embankment slope may be assessed using conventional pseudo-static methods of slope stability analysis under earthquake-induced peak ground acceleration. A calculated factor of safety of 1.0 is considered appropriate; however, a factor of safety less than 1.0 does not indicate full-scale failure of the embankment slope due to the application of the peak ground acceleration in one direction for a short period of time. In this case, other methods, such as the Newmark sliding block method may be used to assess the magnitude of the ground movement.

Results of Global Stability Analyses

The results of the static slope stability analyses, using the parameters given above, indicate that 3 m to 4 m high approach embankments with side slopes oriented at 2H:1V will have a factor of safety of greater than 1.3 against deep-seated slope instability; the result of a static stability analysis for a 2H:1V embankment side slope is shown on Figure 6. A factor of safety of greater than 1.3 is also obtained for a 3 m to 4 m high rock fill embankment, with side slopes oriented at 1.25H:1V.

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the embankment side slopes will have factors of safety of greater than 1.0 against deep-seated slope instability, using a peak ground acceleration of 0.089g based on the site-specific assessment as discussed in Section 6.5.

The results of the pseudo-static seismic stability analysis do indicate that some shallow sloughing could occur on the embankment side slopes during seismic events. This sloughing would not, however, impair the use of the highway or bridge, and would mainly be a maintenance issue. The potential for sloughing could be reduced by providing well-vegetated side slopes.

Surficial Stability and Erosion Protection

To reduce surface water erosion and sloughing on the embankment side slopes, placement of topsoil and seeding or pegged sod is recommended on earth side slopes. As noted above, well-established vegetation will also improve surficial stability during seismic loading events.

At the time of a structural inspection in June 2004, the slope protection in the four quadrants of the existing Beaver Creek bridge was observed to be in fair to poor condition, with some erosion noted. Provision should be made for erosion protection (suitable non-woven geotextiles and/or rip-rap) on those portions of the embankment side slopes that have the potential to be exposed to flowing water and/or flooding conditions. It is recommended that the erosion protection treatment be extended to at least 0.3 m above the design high water level at the site.

6.7.3 Approach Embankment Settlement

Settlement of the new approach embankments will occur as a result of compression of the new embankment fill itself, as well as elastic compression of the predominantly cohesionless soils that underlie the bridge site.

Settlement of New Embankment Fill

Provided that the new embankment fill material consists of granular fill or clean earth fill, the settlement of the 3 m to 3.5 m high embankment fill itself is expected to be less than about 15 mm to 20 mm. The majority of this settlement is expected to occur during placement and compaction of the fill material, and therefore will not impact the post-construction performance of the new approach embankments.

Where rock fill is used for the construction of the new embankment area, settlement of the rock fill itself will depend on the type of rock, and on the method and sequence of placement and compaction of the rock fill. Post-construction settlement may occur as a result of rearrangement of rock particles under load and breakage of rock particles (i.e. local crushing and degradation). Assuming that the rock fill is not end-dumped into its final position and that it is placed in accordance with the requirements outlined in the Special Provision Amendment to OPSS 206, the settlement of rock fill in embankments up to about 5 m in height is estimated to be about 0.4 per cent of the rock fill height (per "Rockfill in the Foundation Design of Highway Structures" prepared by the MTO Research and Development Branch, dated 1982). Therefore, for the approximately 4 m high approach embankments, the potential settlement of rock fill (if adopted) would be less than 15 mm. The majority of this settlement would occur during the first year following construction.

Settlement of Founding Soils

Elastic compression settlement will occur under the new embankment loading in the very loose to loose surficial sand/silt deposit and the underlying, generally very loose to loose, layered sand/silt/clay deposit. Settlement analyses have been carried out to estimate the total magnitude of settlement that will occur under the new approach embankments, using the commercially-available program UNISETTLE (Version 3.0).

The compression of the founding soils has been modelled by estimating an elastic modulus of deformation based on the SPT "N" values and correlations proposed by Bowles (1984)[2] and Kulhawy and Mayne (1990)[3]. The parameters used in the settlement analyses are presented in the following table:

---

[2] Bowles, J.E. 1984. *Physical and Geotechnical Properties of Soils*, 2nd Edition, Ed. McGraw-Hill Book Company.

[3] Kulhawy, F.H. and P.W. Mayne. 1990. *Manual on Estimating Soil Properties for Foundation Design*. Final Report 1493-6, EL-6800, Electric Power Research Institute, Palo Alto, California.

Soil Unit	*Bulk Unit Weight*	*Elastic Modulus*
Fill for embankment widening	21 kN/m^3	–
Very loose to loose surficial sand/silt and layered upper sand/silt	19 kN/m^3	5 MPa
Firm to stiff clayey silt to silty clay	19 kN/m^3	6 MPa
Loose, layered lower sand/silt	19 kN/m^3	20 MPa
Loose to dense sand to sand and gravel till	21 kN/m^3	15 MPa

Based on the methods of settlement analysis and parameters identified above and the subsurface conditions as encountered in the boreholes, it is estimated that up to about 50 mm of settlement will occur below the new 3 m to 3.5 m high approach embankments. This settlement will occur relatively rapidly during and immediately following (within approximately one to two months) construction of the new approach embankments.

6.8 Construction Considerations

6.8.1 Excavation and Groundwater/Surface Water Control

The pile cap excavations for the new bridge will extend to a depth of at least 1.8 m below the natural ground surface at the site, for frost protection purposes. The excavations will extend into the very loose to loose surficial sand/silt deposit, which is water-bearing with a measured groundwater level near or slightly above the natural ground surface.

Given the proximity of the pile cap excavations to Beaver Creek and the high groundwater table at the site, it is anticipated that a closed cofferdam/shoring system will be required to control the ground and groundwater and facilitate excavation for the pile caps. The cofferdam/shoring system should be designed and constructed in accordance with MTO Special Provision SP105S19, using Performance Level 3 as defined in this SP. If shoring is required to facilitate construction in close proximity to or into the existing Highway 62 embankment side slope, then the system should be designed and constructed to meet Performance Level 2 as defined in the MTO SP.

The design of the cofferdam/shoring system will be the responsibility of the Contractor. The system will have to be designed to resist lateral earth pressures that are controlled by the flexibility of the shoring and its method of support, together with basal heave/instability related to the high groundwater level at the site. Conceptually, it is anticipated that the system will consist of driven steel sheet piling, supplemented with dewatering within the cofferdam to control the excavation base stability. For the anticipated depth of the pile cap excavations, it may be feasible to cantilever the sheet piling (i.e., no additional lateral support); otherwise, it would be necessary to provide lateral support using rakers supported on footings or piles within the excavation.

MTO's SP 105S19 has been modified to include additional aspects to address the cofferdam/shoring system, including groundwater control to control excavation base stability and allow foundation construction in dry conditions. This modified SP is contained in Appendix C, for inclusion in the Contract Documents.

6.8.2 Vibration Monitoring During Pile Driving and Protection System Installation

Based on the condition of the existing bridge, it is recommended that vibration monitoring be carried out during pile driving and installation of protection systems/cofferdams to ensure that vibration levels at the existing structure are maintained below tolerable levels. A maximum peak particle velocity of 100 mm/s is recommended at the existing bridge abutments, though this can be modified in conjunction with the structural engineer and MTO, depending on the existing structure condition.

MTO's Special Provision SP903S01 has been modified to incorporate the vibration monitoring requirement for the installation of driven piles, and MTO's Special Provision SP105S19 has been modified to incorporate the vibration monitoring requirement for the installation of protection systems and cofferdams. These modified SPs are included in Appendix C, for inclusion in the Contract Documents.

6.8.3 Cobbles and Boulders in Overburden Soils

The sand to sand and gravel till deposit that overlies the bedrock contains cobbles and boulders, as noted on the borehole records. Appropriate equipment and construction procedures will be required where cobbles and/or boulders are encountered during the installation of the deep foundations for support of the new abutments and any associated wing walls/retaining walls, or during installation of protection systems or cofferdams.

With respect to pile driving and the installation of protection systems/cofferdams, respectively, MTO's Special Provisions SP903S01 and SP105S19 have been modified to warn the Contractor of the presence of cobbles and boulders in the overburden soils. These modified SPs are included in Appendix C, for inclusion in the Contract Documents.

6.8.4 Ground and Groundwater Control for Caisson Installation

As discussed in Section 6.4, running or flowing of water-bearing cohesionless soil strata could occur during or after drilling of the caissons. If caisson foundations are adopted for support of the new abutments and any associated wing walls/retaining walls, temporary or permanent caisson liners would be required to support the soils during construction. It is recommended that an NSSP be included in the Contract Documents to warn the contractor of these conditions and the need to control the ground and groundwater during caisson construction; an example NSSP is included in Appendix C.

6.8.5 Caisson Socket Formation in Bedrock

If caissons are adopted for support of the new bridge abutments and any associated wing walls/retaining walls, nominal socketting into the bedrock will be required. As discussed in Section 6.4, it is recommended that an NSSP be included in the Contract Documents to warn the contractor that the bedrock at the site is generally medium strong to strong, and will require socket formation using appropriate construction equipment and procedures (coring or churn drilling) to advance the hole. An example NSSP is provided in Appendix C.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder, with input regarding seismic stability and liquefaction from Mr. Sen Hu. This report was reviewed by Mr. Fin Heffernan, P.Eng., the Designated MTO Foundations Contact for this project.

GOLDER ASSOCIATES LTD.

Lisa C. Coyne, P.Eng.
Associate

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

SH/LCC/FJH/lcc
n:\active\2007\1111\07-1111-0044 transenco hwy 62 tudor twp\6 - reports\final reports\bridge\07-1111-0044 rpt01 09jun beaver creek bridge.doc

TABLE 1
COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES
BEAVER CREEK BRIDGE REPLACEMENT, HIGHWAY 62
G.W.P. 66-99-00

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-pile foundations driven to found on bedrock	• Feasible for support of new abutments	• High bearing resistance • Negligible settlement • Allows for integral abutments	• Possibility of encountering cobbles or boulders in the glacial till deposit that overlies the bedrock	• Less expensive than caisson option	• Possibility of piles being driven out of alignment due to cobbles/boulders in glacial till
Caissons socketted nominally (about 0.5 m) into bedrock	• Feasible for support of new abutments	• Very high bearing resistance • Negligible settlement	• Temporary or permanent liners required to minimize disturbance to water-bearing cohesionless soils • Possibility of encountering cobbles or boulders in glacial till deposit that overlies bedrock • Rock socket required to “seat” liner into bedrock to facilitate caisson construction; coring or churn drilling will be required to form rock socket in medium strong to strong bedrock • Does not allow for integral abutment configuration for new structure	• More expensive than steel H-pile option	• Socketting into the medium strong bedrock would be difficult and time-consuming
Spread footings supported on native soils	• Not feasible for support of new bridge abutments (insufficient geotechnical resistance)	• N/A	• N/A	• N/A	• Excessive settlement / foundation failure

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 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 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

S:\FINALDAT\ABBREV\2000\LOFA-D00.DOC

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
v	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

(a) Index Properties (continued)

w	water content
w_1	liquid limit
w_p	plastic limit
l_p	plasticity index = $(w_1 - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p)/I_p$
I_C	consistency index = $(w_1 - w)\,/I_p$
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 = σ'_p/σ'_{vo}

(d) Shear Strength

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

Notes:
1 $\tau = c' + \sigma' \tan \phi'$
2 shear strength = (compressive strength)/2
* density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

S:\FINALDAT\SYMBOLS\2000\SYMB-D00.DOC

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

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

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

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

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
II - Parallel To	
⊥ - Perpendicular To	

Form...G.A.-R-3

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 07-22

METRIC

PROJECT 07-1111-0044

W.P. 66-99-00

LOCATION N 4964925.7; E 215005.5

ORIGINATED BY P.A.H.

DIST Eastern HWY 62

BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger

COMPILED BY S.L.

DATUM Geodetic

DATE Nov. 6-8, 2007

CHECKED BY T.M.S.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
304.6	GROUND SURFACE
0.0	ASPHALTIC CONCRETE
	Sand and gravel (FILL) Grey-brown
0.4	Sand and gravel, some silt, containing cobbles (FILL) Compact Grey Moist
303.1 / 1.5	Sandy silt, some gravel, trace clay (FILL) Very loose Brown Wet
302.3 / 2.3	Silty sand, some gravel, containing trace organic matter (FILL) Loose to very loose Brown Wet
300.9 / 3.7	SAND, trace silt Very loose Grey Wet
298.2 / 6.4	Layered CLAYEY SILT, Sandy SILT and SAND Firm / very loose to loose Grey Wet
295.5 / 9.2	Layered CLAYEY SILT and SILTY CLAY Firm to stiff Grey Wet
293.9 / 10.7	Layered SILT, CLAYEY SILT, Sandy SILT and SAND Loose Grey Wet

Ground water level symbol shown at approx. elev. 302.5.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	A.S.	–	
2	SS	15	33 49 (18)
3	SS	3	
4	SS	5	
5	SS	3	
6	SS	2	
7	SS	2	0 94 (6)
8	SS	3	
9	SS	2	0 19 77 4
10	SS	4	
11	SS	3	
12	SS	4	
13	SS	7	
14	SS	7	0 1 91 8
15	SS	7	
16	SS	7	

ELEVATION SCALE: 304, 303, 302, 301, 300, 299, 298, 297, 296, 295, 294, 293, 292, 291, 290

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 25 50 75

UNIT WEIGHT γ kN/m³

Continued Next Page

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

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

Golder Associates

RECORD OF BOREHOLE No 07-22

METRIC

PROJECT 07-1111-0044
W.P. 66-99-00
LOCATION N 4964925.7; E 215005.5
ORIGINATED BY P.A.H.
DIST Eastern HWY 62
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY S.L.
DATUM Geodetic
DATE Nov. 6-8, 2007
CHECKED BY T.M.S.

— CONTINUED FROM PREVIOUS PAGE —

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
289.4 15.2	Layered Sandy SILT and Silty SAND Very loose to loose Grey Wet	17	SS	4	289	
		18	SS	WR	288, 287	
286.6 18.0	SAND and GRAVEL, trace to some silt, containing cobbles (TILL) Compact to very dense Grey Wet	19	SS	14	286	
		20	SS	60	285	39 44 16 1
		21	SS	43	284, 283, 282	
281.1 23.6	Dolomite, containing quartz layers (BEDROCK) Fresh Grey Medium strong to strong Bedrock cored between 23.6 m and 26.8 m depth. For bedrock coring details refer to Record of Drillhole 07-22.	22	NQ RC	REC 100%	281	RQD = 85%
		23	NQ RC	REC 100%	280	RQD = 92%
		24	NQ RC	REC 100%	279	RQD = 59%
277.9 26.8	End of Borehole Note: Water level in open borehole at 2.2 m depth (Elev. 302.4) upon completion of drilling on Nov. 8, 2007				278	

Column headings also include: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³.

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

PROJECT: 07-1111-0044

RECORD OF DRILLHOLE: 07-22

SHEET 1 OF 1

LOCATION: N 4964925.7; E 215005.5

DRILLING DATE: Nov. 6-8, 2007

DATUM: Geodetic

DRILL RIG: CME 75

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Marathon Drilling Co. Ltd.

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole 07-22		281.00												
24	Rotary Drill NQ Core	Dolomite (BEDROCK), containing quartz layers Fresh Grey Medium strong to strong		23.60	1											
25					2											
26				277.80 26.80	3											
27		End of Drillhole														

28

29

30

31

32

33

34

35

36

37

38

MIS-RCK 001 07111110044-1000-ROCK.GPJ GAL-MISS.GDT 7/3/08

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: P.A.H.
CHECKED: [signature]

Golder Associates

RECORD OF BOREHOLE No 07-23

METRIC

PROJECT 07-1111-0044
W.P. 66-99-00
DIST Eastern HWY 62
DATUM Geodetic
LOCATION N 4964890.5; E 215044.0
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
DATE Jan. 9, 2008
ORIGINATED BY P.A.H.
COMPILED BY J.M.
CHECKED BY T.M.S.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
302.6	GROUND SURFACE				
0.0	ORGANIC MATTER, Dark brown				
302.2 / 0.4	Layered Sandy SILT and SAND, Loose, Grey-brown, Wet	1	SS	6	
301.3 / 1.4	SAND, Very loose, Grey, Wet	2	SS	2	0 97 (3)
300.2 / 2.4	Layered CLAYEY SILT, Sandy SILT and SAND, Very loose to loose, Grey, Wet	3	SS	2	
		4	SS	3	0 23 72 5
		5	SS	5	
298.1 / 4.6	Layered CLAYEY SILT and SILTY CLAY, Firm to stiff, Grey, Wet	6	SS	3	
		7	SS	6	
295.2 / 7.5	Layered SILT, CLAYEY SILT and Sandy SILT, Loose, Grey, Wet	8	SS	7	0 2 94 4
		9	SS	9	
		10	SS	8	0 3 90 7
		11	SS	6	
289.1 / 13.6	SAND and GRAVEL, trace to some silt, containing cobbles (TILL), Compact, Grey, Wet	12	SS	22	

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³; ELEVATION SCALE 302–288; STRAT PLOT; GROUND WATER CONDITIONS.

Continued Next Page

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 07-23

METRIC

PROJECT 07-1111-0044

W.P. 66-99-00 LOCATION N 4964890.5; E 215044.0 ORIGINATED BY P.A.H.

DIST Eastern HWY 62 BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger COMPILED BY J.M.

DATUM Geodetic DATE Jan. 9, 2008 CHECKED BY T.M.S.

— CONTINUED FROM PREVIOUS PAGE —

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND and GRAVEL, trace to some silt, containing cobbles (TILL) Compact Grey Wet	13	SS	30	287	43 44 11 2
					286	
285.7 / 16.9	Marble (BEDROCK) Fresh White, grey and pink Medium strong to strong Bedrock cored between 16.9 m and 20.0 m depth. For bedrock coring details refer to Record of Drillhole 07-23.	14	NQ RC	REC 97%	285	RQD = 65%
					284	
		15	NQ RC	REC 100%	283	RQD = 90%
282.7 / 20.0	End of Borehole Note: Water level in open borehole at 0.8 m above ground surface (Elev. 303.4) upon completion of drilling on Jan. 11, 2008					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

MISS_MTO 071110044-1000.GPJ ON_MOT.GDT 7/3/08

RECORD OF DRILLHOLE: 07-23

PROJECT: 07-1111-0044

SHEET 1 OF 1

LOCATION: N 4964890.5; E 215044.0

DRILLING DATE: Jan. 9, 2008

DATUM: Geodetic

DRILL RIG: CME 75

INCLINATION: -90°   AZIMUTH: ---

DRILLING CONTRACTOR: Marathon Drilling Co. Ltd.

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN FLUSH	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole 07-23		285.74												
17	Rotary Drill NQ Core	Marble (BEDROCK) Fresh White, grey and pink Medium strong to strong		16.90	1											
19					2											
20				282.64 20.00												
		End of Drillhole														

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: P.A.H.

CHECKED: [illegible]

MIS-RCK 001 07111110044-1000-ROCK.GPJ GAL-MISS.GDT 7/3/08

Golder Associates

RECORD OF BOREHOLE No 07-24

METRIC

PROJECT 07-1111-0044			ORIGINATED BY P.A.H.
W.P. 66-99-00	LOCATION N 4964889.0; E 215029.9		COMPILED BY J.M.
DIST Eastern HWY 62	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		CHECKED BY T.M.S.
DATUM Geodetic	DATE Jan. 10, 2008		

Column headings: SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (25, 50, 75); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

ELEV / DEPTH	DESCRIPTION
302.6	GROUND SURFACE
0.0	ORGANIC MATTER Dark brown
302.1	Sandy SILT, trace organic matter Grey brown Wet
301.8 / 0.8	Layered SILT, some sand, trace clay Loose Grey-brown to grey Wet
300.8 / 1.8	SAND, trace to some silt Very loose Grey Wet
299.6 / 3.1	Layered CLAYEY SILT, Sandy SILT and Silty SAND Very loose Grey Wet
297.1 / 5.5	Layered CLAYEY SILT and SILTY CLAY Firm to stiff Grey Wet
295.6 / 7.0	Layered SILT, CLAYEY SILT and Sandy SILT Loose Grey Wet
290.0 / 12.7	Silty SAND to SAND, some silt, trace to some gravel, containing cobbles (TILL) Loose to dense Grey to red-grey Wet

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	A.S.					
2	SS	6	0	17	75	8
3	SS	5				
4	SS	3	0	86	(14)	
5	SS	2				
6	SS	4	0	27	66	7
7	SS	3				
8	SS	4				
9	SS	8	0	2	90	8
10	SS	8				
11	SS	6	0	4	90	6
12	SS	1				
13	SS	8	2	79	17	2

Elevation scale: 302, 301, 300, 299, 298, 297, 296, 295, 294, 293, 292, 291, 290, 289, 288

Continued Next Page

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 07-24

METRIC

PROJECT 07-1111-0044

W.P. 66-99-00 LOCATION N 4964889.0; E 215029.9 ORIGINATED BY P.A.H.

DIST Eastern HWY 62 BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger COMPILED BY J.M.

DATUM Geodetic DATE Jan. 10, 2008 CHECKED BY T.M.S.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —										
	Silty SAND to SAND, some silt, trace to some gravel, containing cobbles (TILL) Loose to dense Grey to red-grey Wet		14	SS	48		287				
							286				
285.6 17.0	End of Borehole Auger Refusal Note: Water level in open borehole at 0.8 m depth (Elev. 301.8) upon completion of drilling on Jan. 10, 2008										

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

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

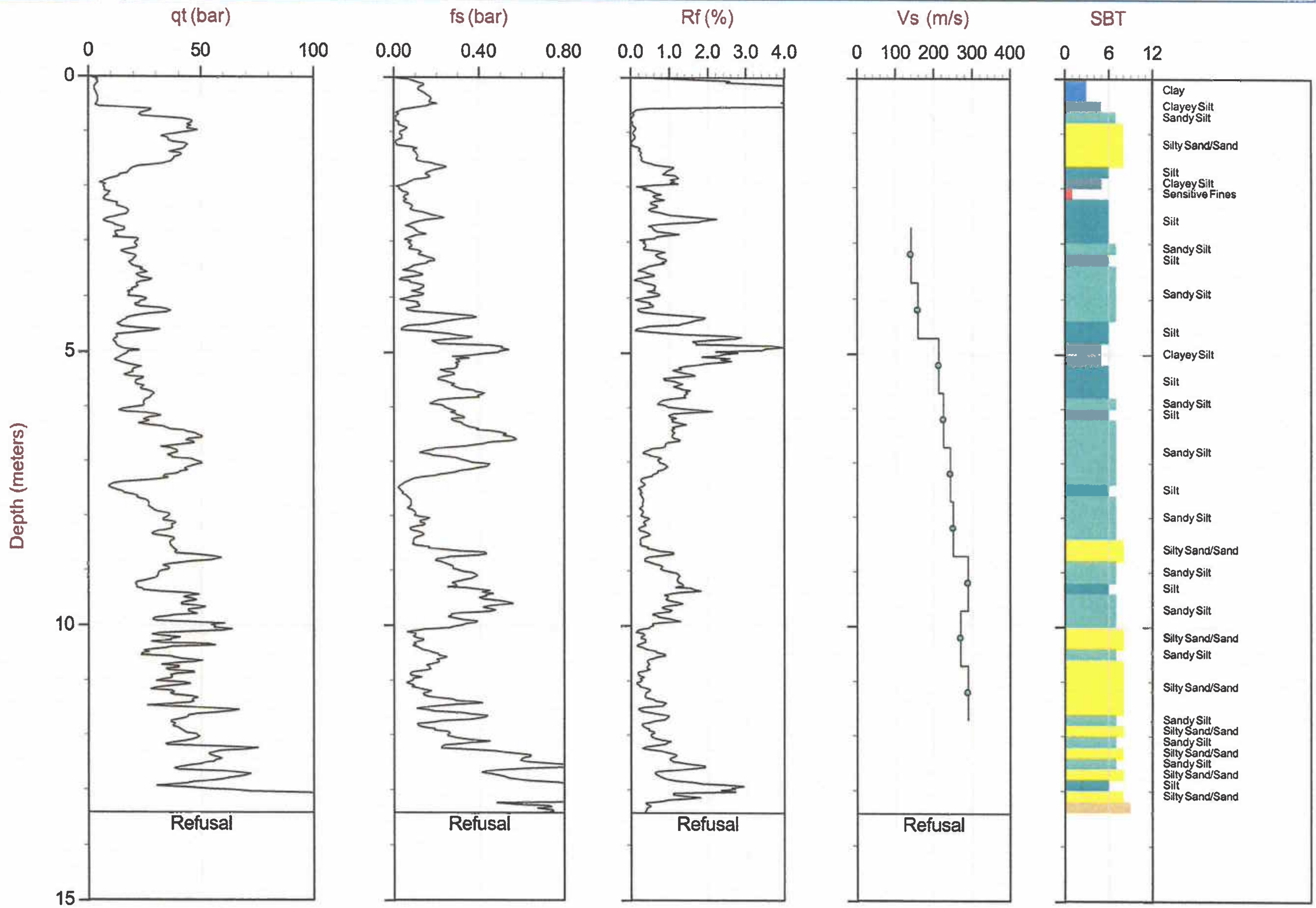
MISS_MTO 07111110044-1000.GPJ ON_MOT.GDT 7/3/08

CONETEC

Golder Associates

Job No: 08-019
Date: 01:14:08 15:14
Site: Beaver Creek, Bancroft, ON

Sounding: SCPT-07-23
Cone: Med. Cap. AD-199



Max Depth: 13.425 m / 44.04 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: 0.200 m

File: 019SCP02.COR
Unit Wt: SBT Chart Soil Zones

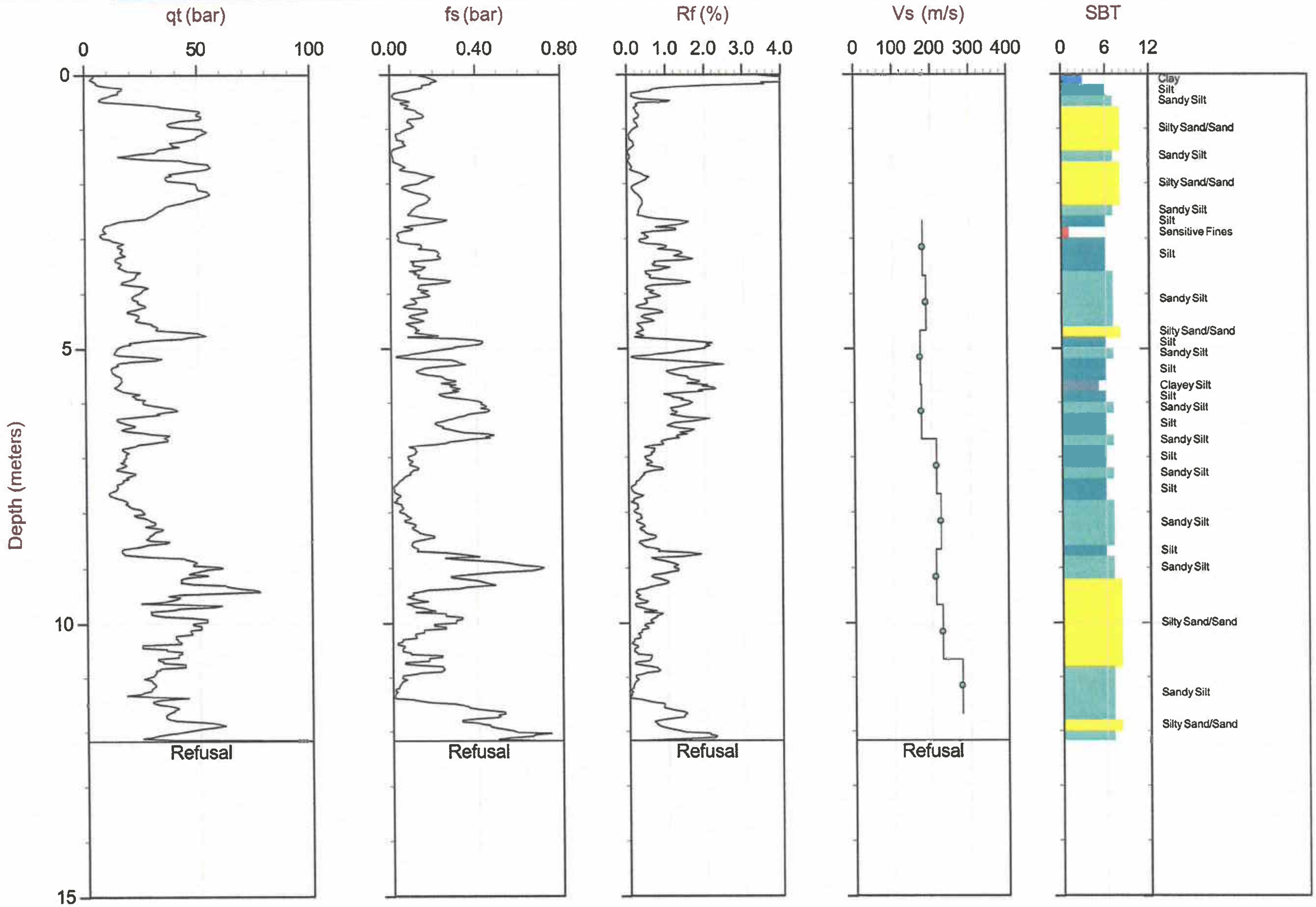
SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

CONETEC

Golder Associates

Job No: 08-019
Date: 01:14:08 12:15
Site: Beaver Creek, Bancroft, ON

Sounding: SCPT-07-24
Cone: Med. Cap. AD-199



Max Depth: 12.175 m / 39.94 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: 0.200 m

File: 019SCP01.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

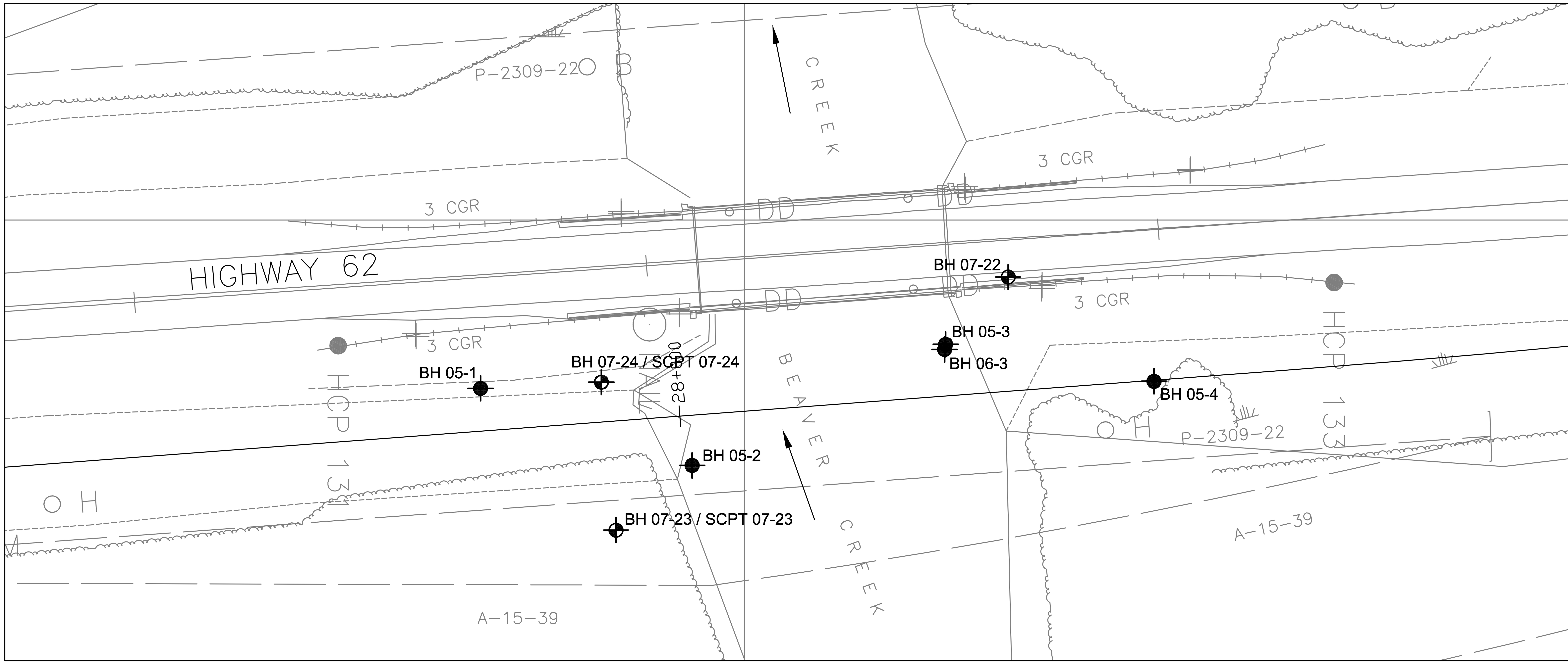
MINISTRY OF TRANSPORTATION, ONTARIO

CONT No. 2009-4726
WP No. 66-99-00

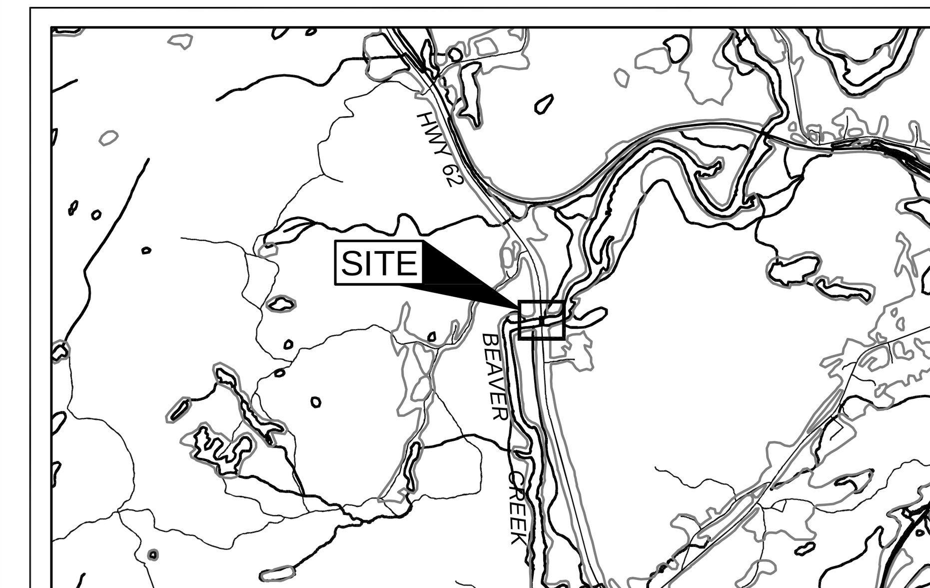
HIGHWAY 62
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET 80

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



PLAN

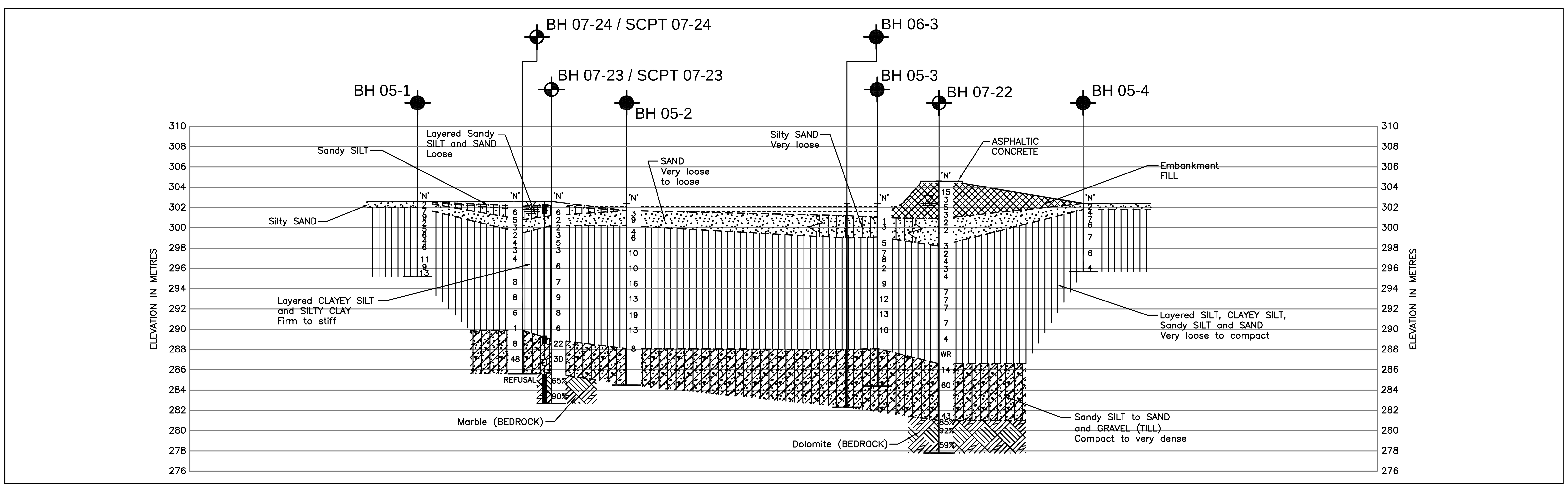


KEY PLAN

LEGEND

- Borehole – Current Golder Associates Ltd. Investigation
- Borehole – 2005–2006 Investigation Geocres No. 31C–172
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation (RQD)
- WL upon completion of drilling
- Seal
- Piezometer

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
07–22	304.6	4964928.8	215020.1
07–23	302.6	4964890.5	215044.0
07–24	302.6	4964889.0	215029.9
06–3	302.4	4964919.5	215012.6
05–1	302.6	4964874.3	215016.4
05–2	302.4	4964894.9	215023.9
05–3	302.4	4964919.6	215012.1
05–4	302.4	4964939.9	215015.7



PROFILE ALONG CL NEW HIGHWAY 62

NOTES

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.

This drawing is for subsurface information only. The proposed structure details are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

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

NO.	DATE	BY	REVISION

Geocres No. –

HWY. 62	PROJECT NO. 07–1111–0044		DIST.
SUBM'D. T.M.S.	CHKD. T.M.S.	DATE: APR. 2008	SITE:
DRAWN: J.M.	CHKD. F.J.H.	APPD. L.C.C.	DWG. 1

PLOT DATE: June 22, 2009

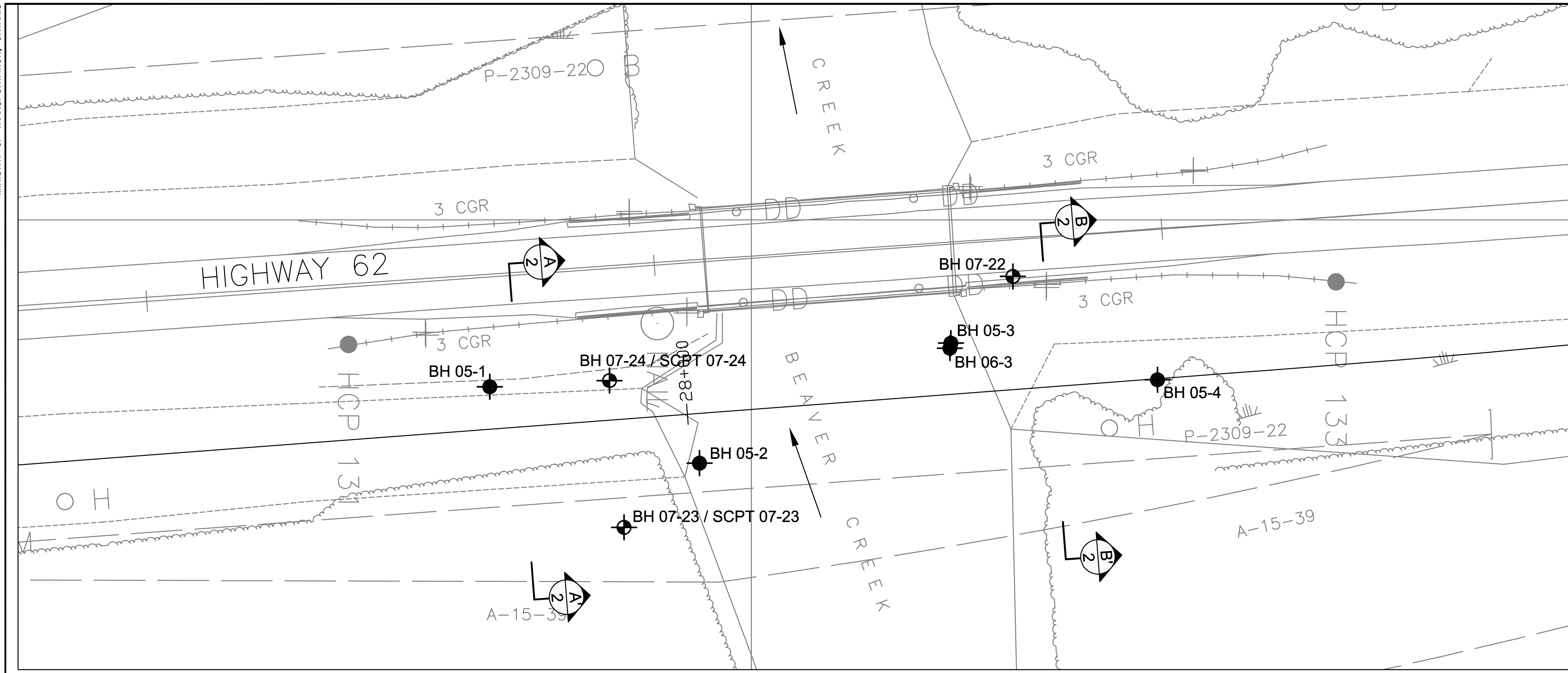
MINISTRY OF TRANSPORTATION, ONTARIO

CONT No. 2009-4726
WP No. 66-99-00

HIGHWAY 62
BOREHOLE LOCATIONS AND SOIL STRATA

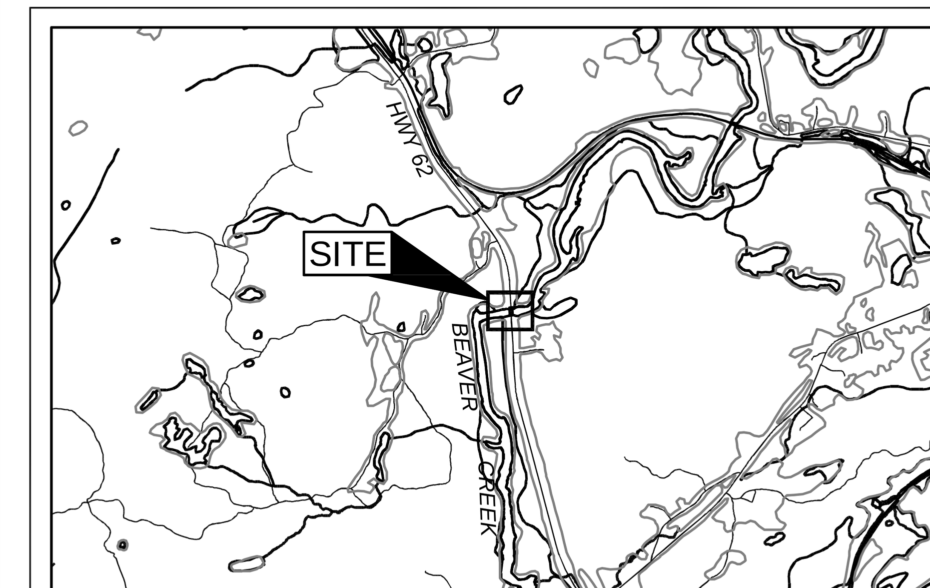
SHEET 81

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



PLAN

SCALE 5 0 5 10 METRES



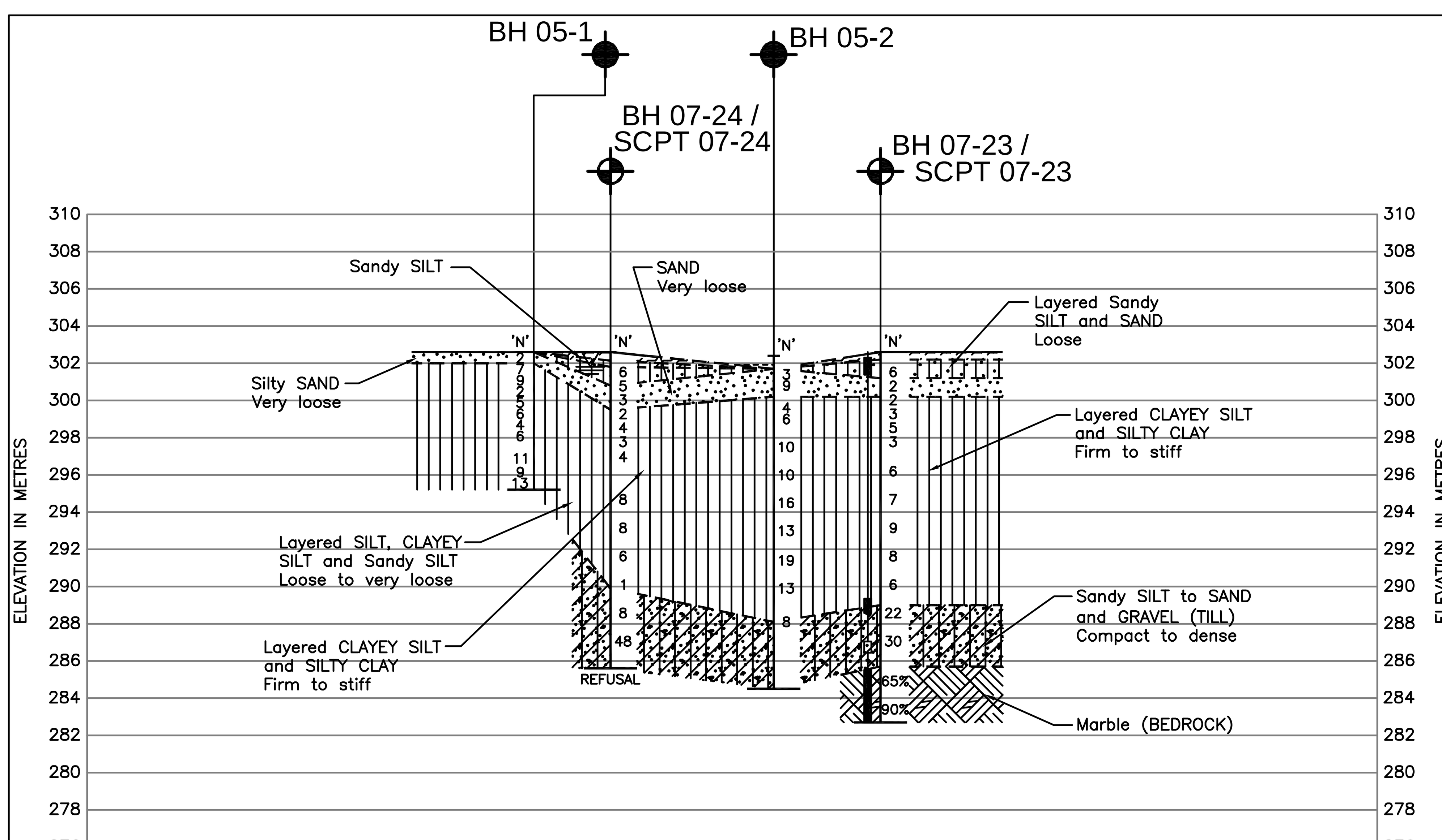
KEY PLAN

SCALE 500 0 500 METRES

LEGEND

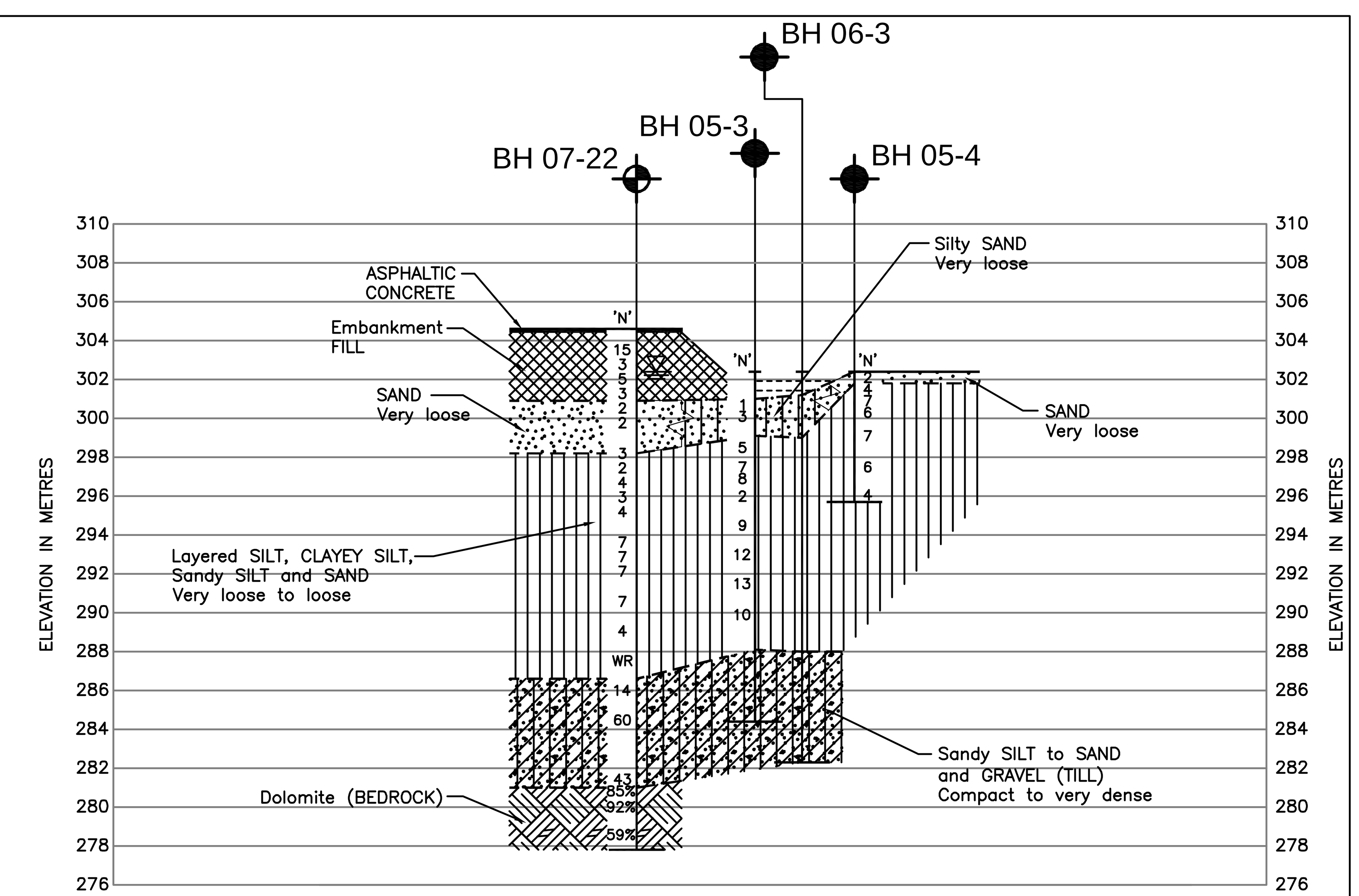
Symbol	Description
	Borehole – Current Golder Associates Ltd. Investigation
	Borehole – 2005–2006 Investigation Geocres No. 31C–172
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
	WL upon completion of drilling
	Location of cross–section
	Seal
	Piezometer

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
07–22	304.6	4964928.8	215020.1
07–23	302.6	4964890.5	215044.0
07–24	302.6	4964889.0	215029.9
06–3	302.4	4964919.5	215012.6
05–1	302.6	4964874.3	215016.4
05–2	302.4	4964894.9	215023.9
05–3	302.4	4964919.6	215012.1
05–4	302.4	4964939.9	215015.7



SECTION A-A'

SCALE 5 0 5 10 METRES



SECTION B-B'

SCALE 5 0 5 10 METRES

NOTES

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.

This drawing is for subsurface information only. The proposed structure details are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

METRIC

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

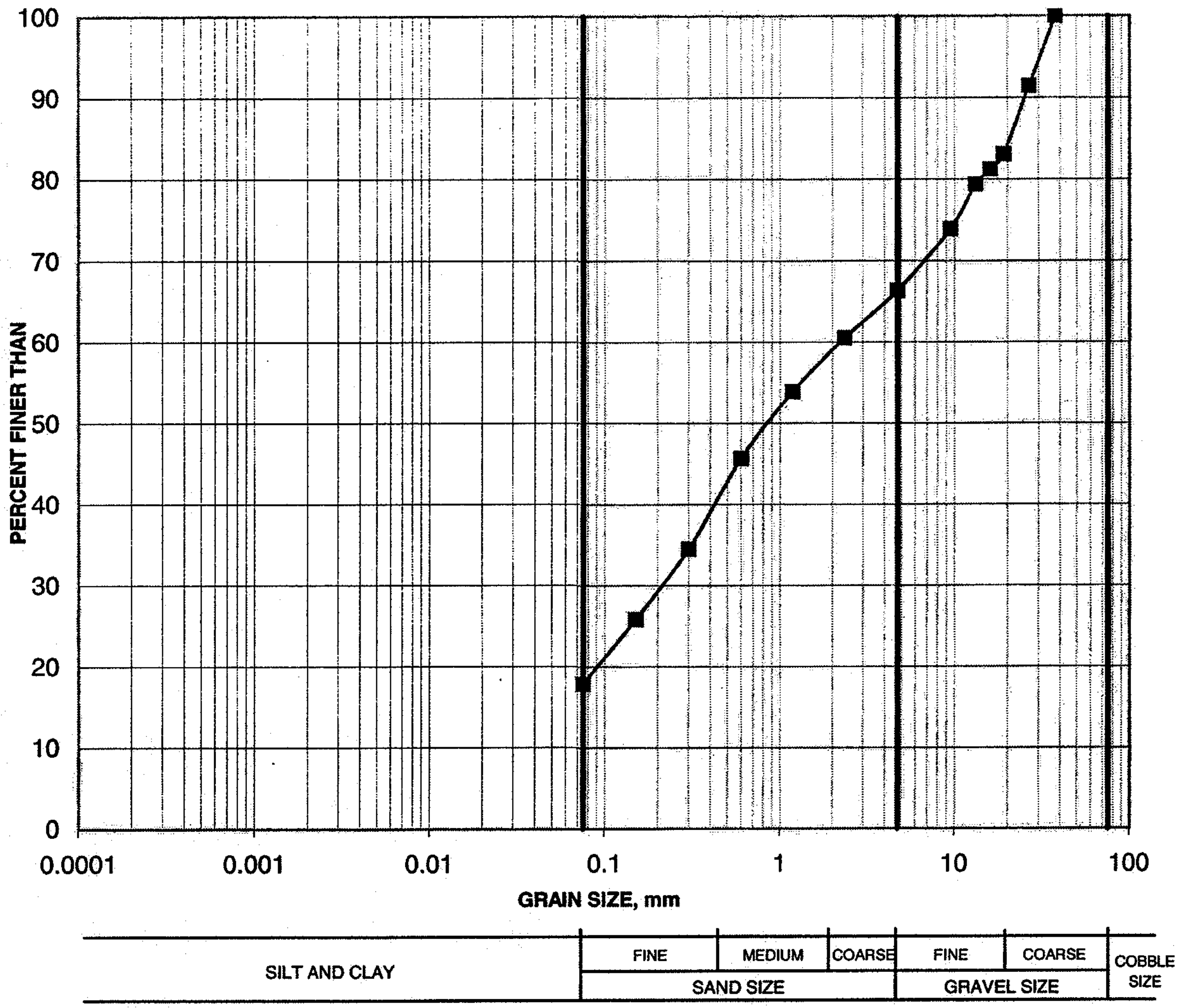
NO.	DATE	BY	REVISION

Geocres No. –

HWY. 62		PROJECT NO.07–1111–0044	DIST.
SUBM'D. T.M.S.	CHKD. T.M.S.	DATE: APR. 2008	SITE:
DRAWN: J.M.	CHKD. F.J.H.	APPD. L.C.C.	DWG. 2

PLOT DATE: June 22, 2009

GRAIN SIZE DISTRIBUTION TEST RESULTS - SAND AND GRAVEL FILL -	Figure 1



Borehole	Sample	Elevation (m)
07-22	2	303.5

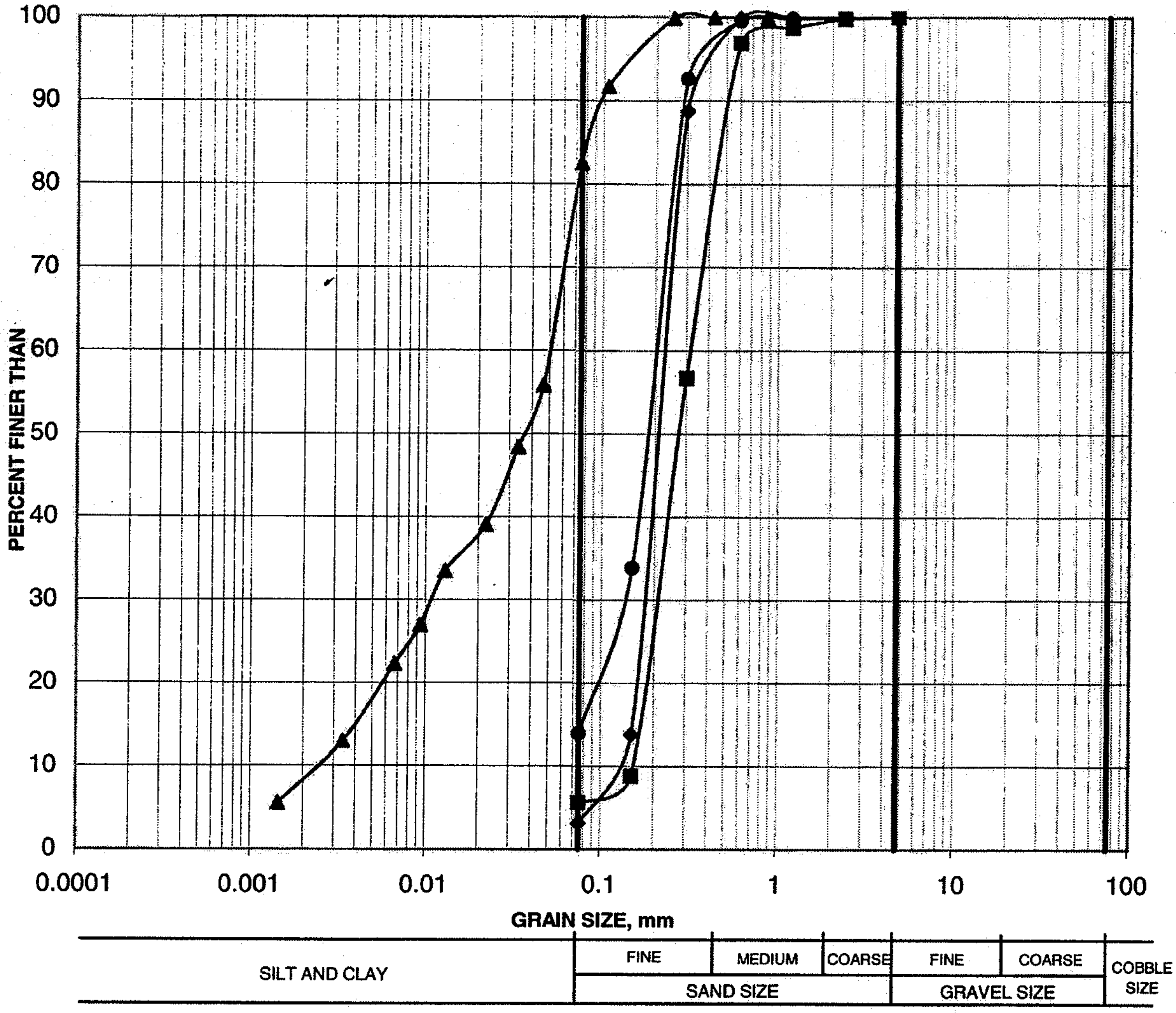
Project: 0711110044

Golder Associates

Created by:

Checked by:

GRAIN SIZE DISTRIBUTION TEST RESULTS - SURFICIAL SAND TO SILT -	Figure 2

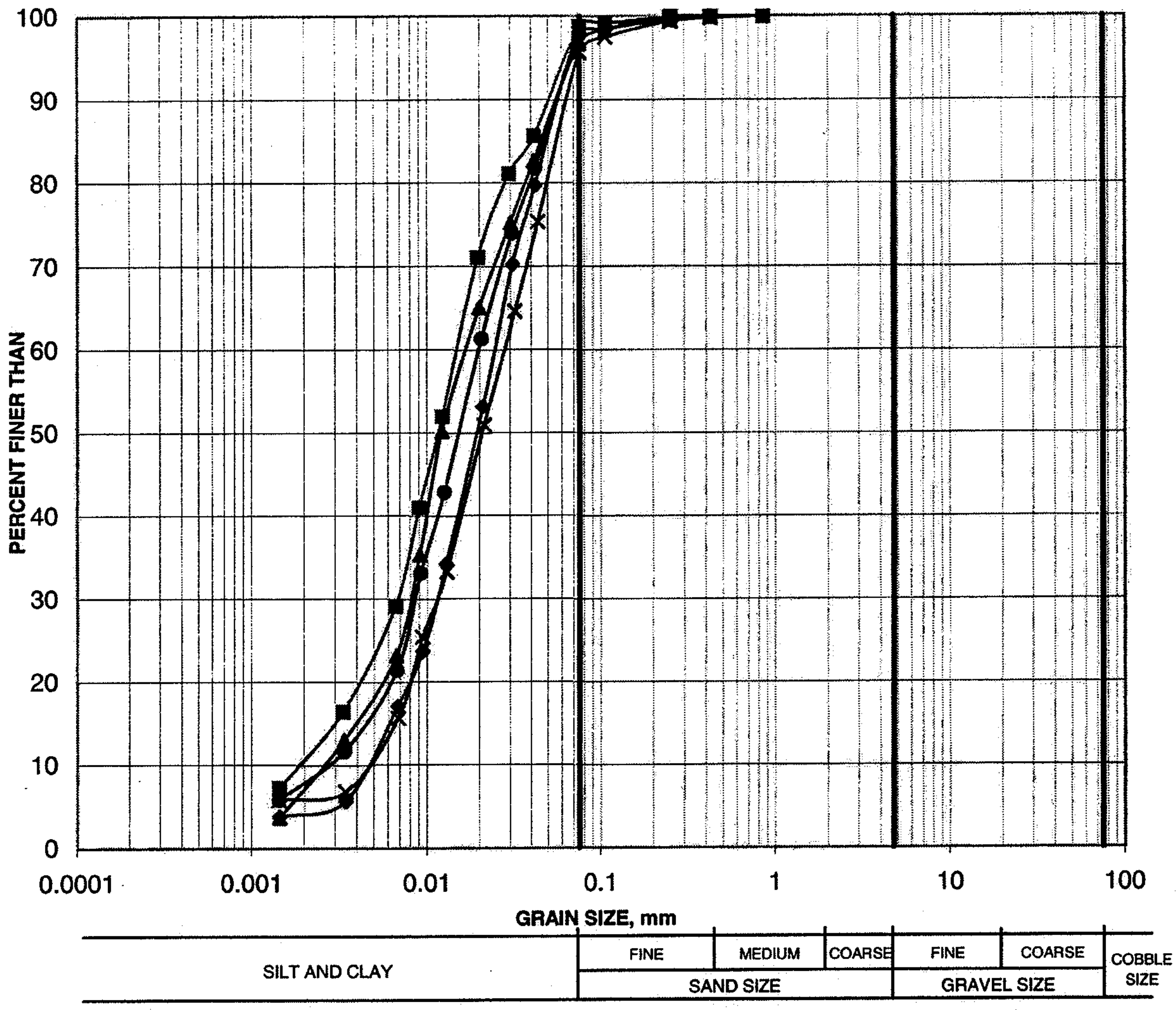


Borehole	Sample	Elevation (m)
07-22	7	299.7
07-23	2	300.8
07-24	2	301.6
07-24	4	300.0

Project: 0711110044 **Golder Associates** Created by: Checked by:

GRAIN SIZE DISTRIBUTION TEST RESULTS
- LAYERED SILT, CLAY AND SAND -

Figure 3A



Borehole	Sample	Elevation (m)
07-22	14	292.9
07-23	8	294.7
07-23	10	291.6
07-24	9	294.7
07-24	11	291.6

Project: 0711110044

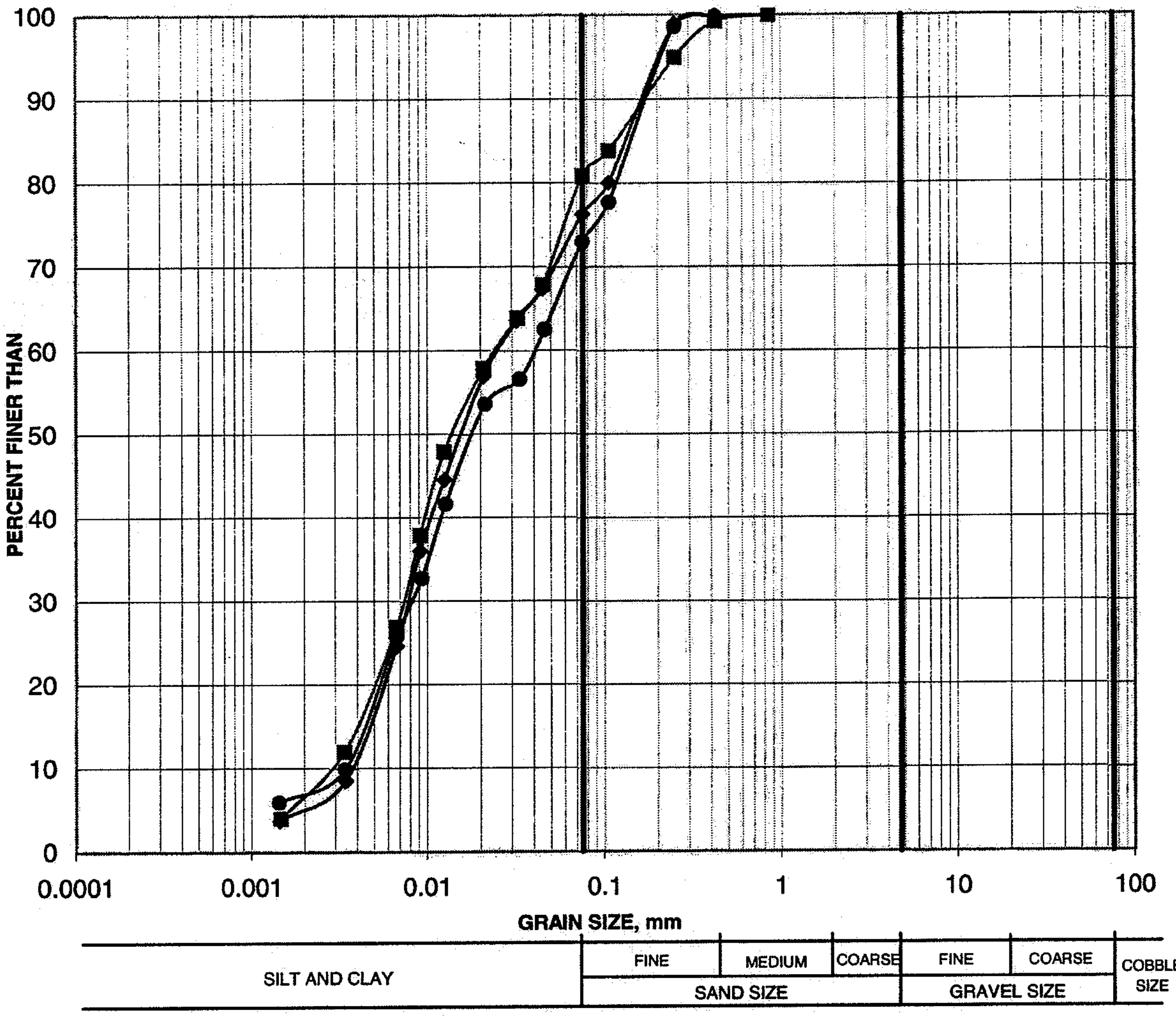
Golder Associates

Created by:

Checked by:

GRAIN SIZE DISTRIBUTION TEST RESULTS
- LAYERED SILT, CLAY AND SAND -

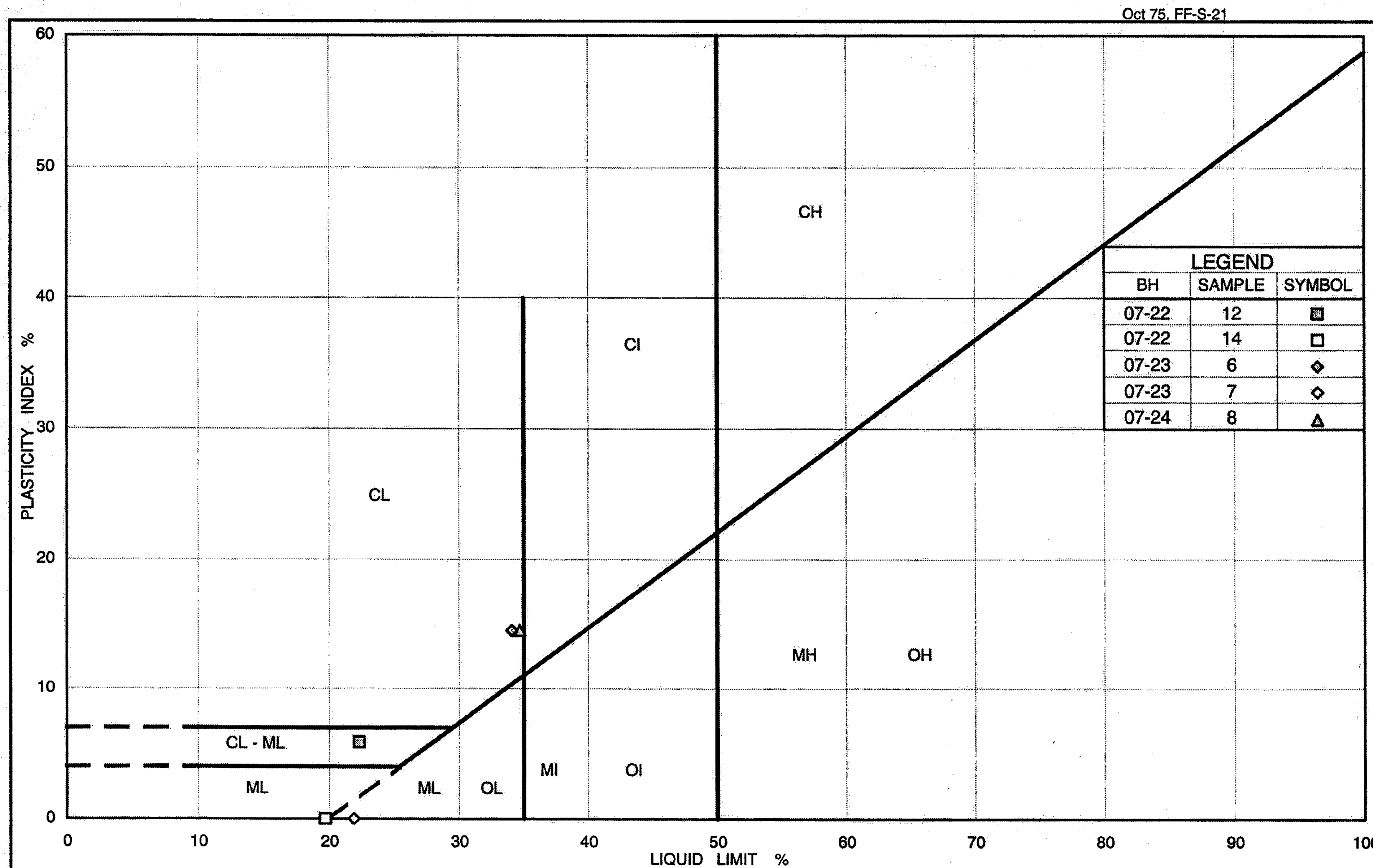
Figure 3B



Borehole	Sample	Elevation (m)
07-22	9	297.4
07-23	4	299.3
07-24	6	298.5

Project: 0711110044 **Golder Associates**

Created by:
Checked by:



Ministry of Transportation

Ontario

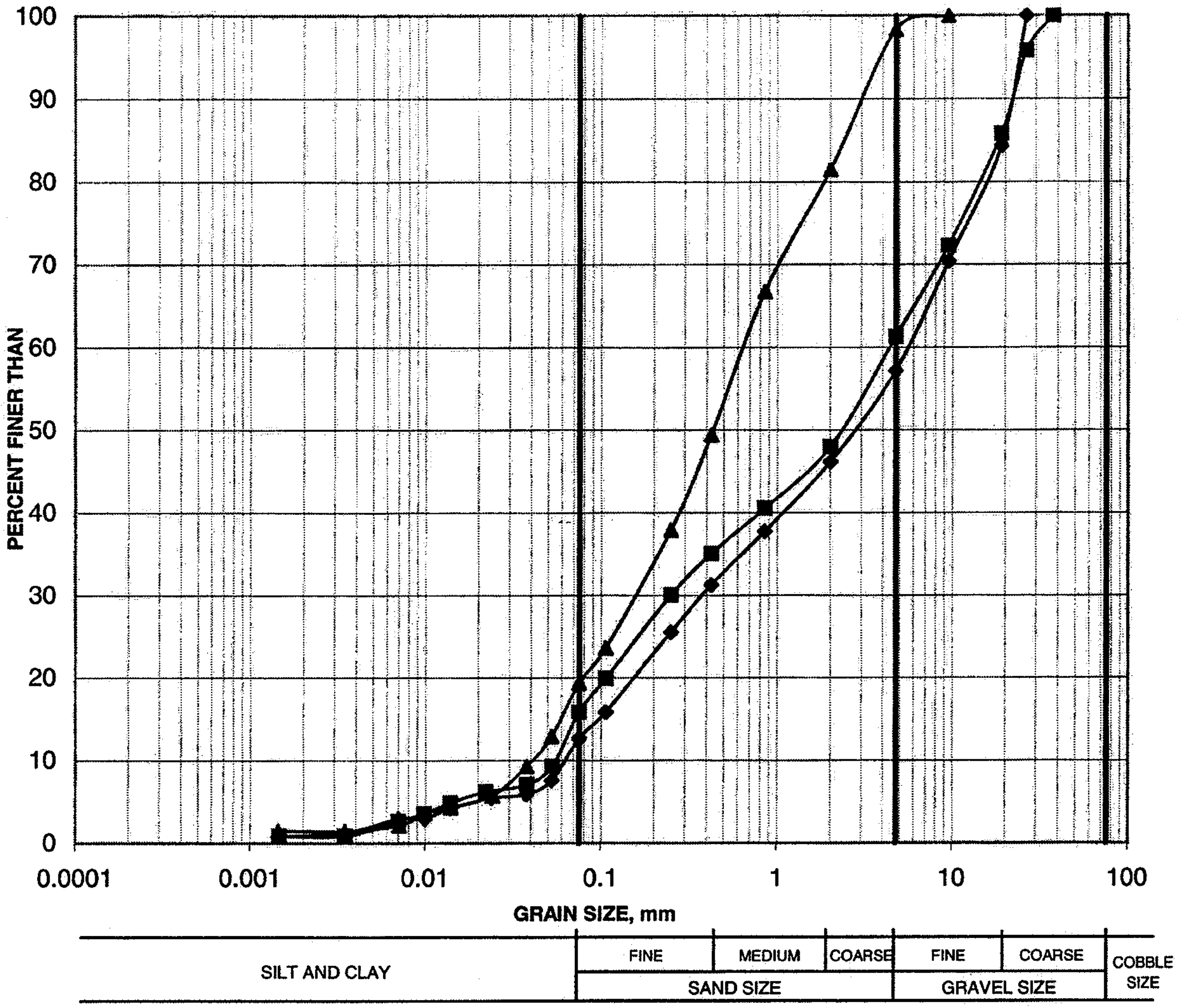
PLASTICITY CHART

- LAYERED SILT, SAND AND CLAY -

Figure 4

Project No. 07-1111-0044

GRAIN SIZE DISTRIBUTION TEST RESULTS - SAND TO SAND AND GRAVEL TILL -	Figure 5



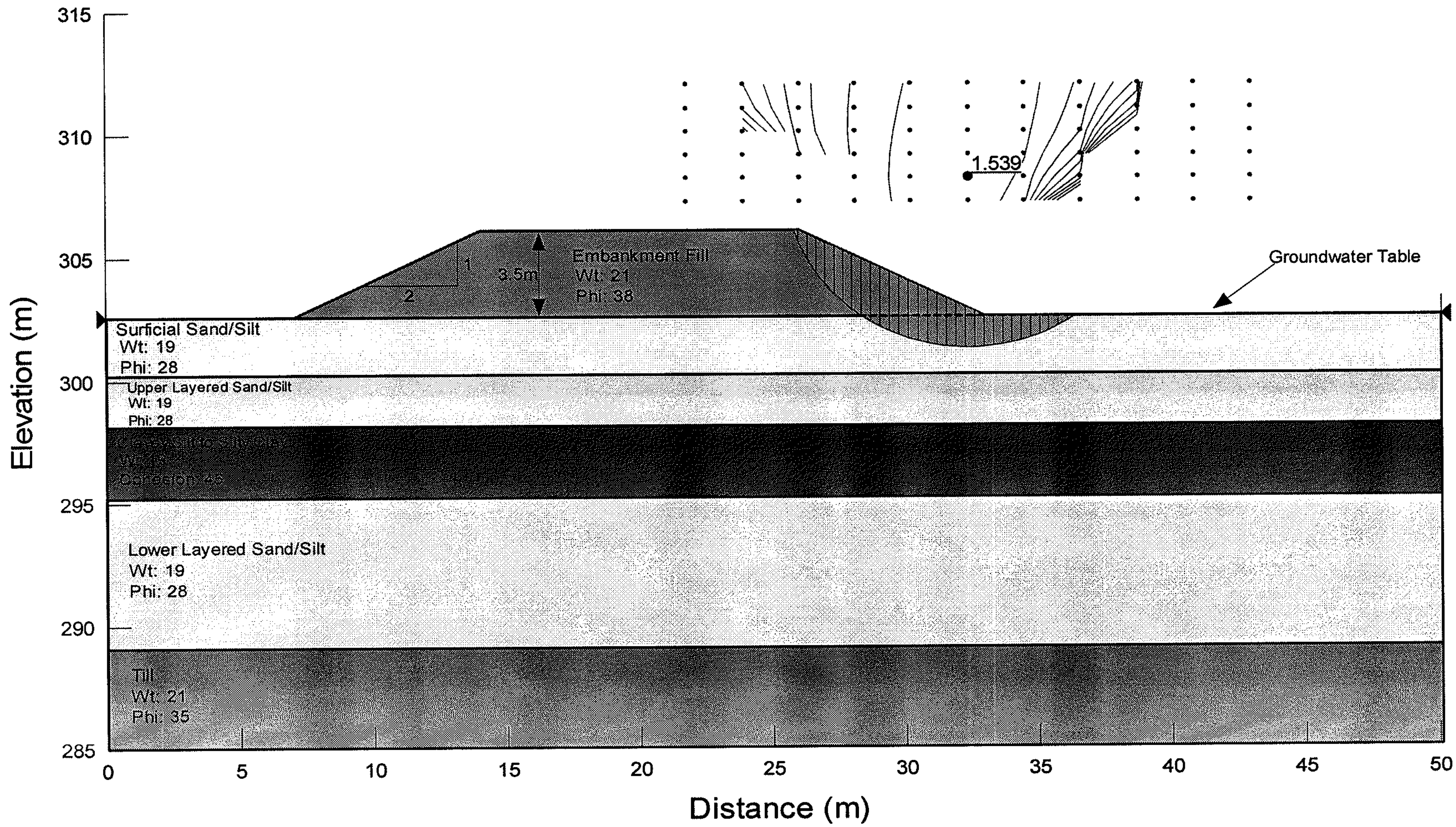
Borehole	Sample	Elevation (m)
07-22	20	284.5
07-23	13	287.1
07-24	13	288.6

Project: 0711110044 **Golder Associates**

Created by:
Checked by:

Beaver Creek Bridge Approach Embankment
Static Global Stability

FIGURE 6



Date: July 2008
Project: 07-111-0044

Golder Associates

Drawn: JEB
Checked: LCC

APPENDIX A

RECORDS OF BOREHOLES FROM
2005-2006 PRELIMINARY INVESTIGATION
BY JACQUES WHITFORD LIMITED
(MTO GEOCRES NO. 31C-172)

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No BH05-1

METRIC

W.P. 248-99-00 LOCATION Beaver Creek Bridge, Site No. 11-034, N4964874.3, E215016.4 ORIGINATED BY JD

DIST 43 HWY 62 BOREHOLE TYPE Portable, Cased, Splitspoons COMPILED BY JD

DATUM Geodetic DATE 28.11.05 - 28.11.05 CHECKED BY PC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
302.6	Tall Grass						
0.0	SILTY SAND, trace organics, trace roots, very loose, dark brown	1	SS	2	▽		
302.0	(SM)						4 54 40 2
0.8	SANDY SILT with seams of silty sand and silty clay, very loose to compact, brownish grey to grey	2	SS	7		302	
		3	SS	9		301	
		4	SS	2			
	(ML)	5	SS	5		300	0 25 65 10
		6	SS	6		299	
		7	SS	4			
		8	SS	8		298	
						297	
	(ML)	9	SS	11			0 0 95 5
		10	SS	9		296	
295.2		11	SS	13			
7.4	End of Borehole						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED × FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

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

WATER CONTENT (%) 10 20 30 (68.9)

UNIT WEIGHT γ kN/m³

End of Borehole

Standpipe Installed (25 mm diameter slotted flexible polytube)

Water level measured December 1, 2005

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

RECORD OF BOREHOLE No BH05-2

METRIC

W.P. 248-99-00 | LOCATION Beaver Creek Bridge, Site No. 11-034, N4964894.9, E215023.9 | ORIGINATED BY JD

DIST 43 HWY 62 | BOREHOLE TYPE Portable, Cased, Splitspoons | COMPILED BY JD

DATUM Geodetic | DATE 29.11.05 - 29.11.05 | CHECKED BY PC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	GR	SA	SI	CL
302.4 / 0.0	Water				302					
301.7 / 0.7	SILTY SAND, with seams of sandy silt and silty clay, very loose, grey	1	SS	3						
	(SP - SM)	2	SS	9	301	~20	0	90	8	2
300.2 / 2.2	SANDY SILT with seams of silty sand and silty clay, very loose to compact, grey				300					
		3	SS	4						
	(ML)	4	SS	6	299	~20	0	33	60	7
					298					
		5	SS	10	297					
		6	GS							
		7	SS	10	296					
		8	GS		295					
		9	SS	16	294					
		10	SS	13	293					
					292					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, X FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No BH05-2

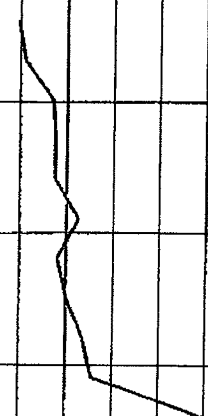
METRIC

W.P. 248-99-00 LOCATION Beaver Creek Bridge, Site No. 11-034, N4964894.9, E215023.9 ORIGINATED BY JD

DIST 43 HWY 62 BOREHOLE TYPE Portable, Cased, Splitspoons COMPILED BY JD

DATUM Geodetic DATE 29.11.05 - 29.11.05 CHECKED BY PC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, ✕ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SANDY SILT, with seams of silty sand and silty clay, compact, grey (ML)		11	SS	19				○ (approx. 24)		0 4 88 8
							291				
			12	SS	13		290				
							289				
288.1 14.3	Sandy silt, some gravel, occasional cobble, compact to very dense, grey (TILL) (ML)		13	SS	8		288				
							287				
							286				
							285				
284.5 17.9	End of Borehole Refusal										



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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No BH05-3

METRIC

W.P. 248-99-00 LOCATION Beaver Creek Bridge, Site No. 11-034, N4984919.6, E215012.1 ORIGINATED BY JD

DIST 43 HWY 62 BOREHOLE TYPE Portable, Cased, Splitspoons COMPILED BY JD

DATUM Geodetic DATE 30.11.05 - 30.11.05 CHECKED BY PC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	GR SA SI CL
302.4	Water					
0.0	Beaver Creek					
301.0 / 1.4	SILTY SAND, some organics, trace wood chips, trace roots, very loose, brown (SP - SM)	1	SS	1	52.6	
		2	SS	3	74.8	
		3	GS			
299.1 / 3.3	SANDY SILT with seams of silty sand and silty clay, very loose to compact, grey	4	ST			
	(SM)	5	SS	5		0 64 30 6
		6	GS			
		7	SS	7		
		8	SS	8		
		9	SS	2		
		10	GS			
	(ML)	11	SS	9		0 22 71 7
		12	SS	12		

ELEVATION SCALE: 302, 301, 300, 299, 298, 297, 296, 295, 294, 293, 292

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED ✕ FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No BH05-3

METRIC

W.P. 248-99-00 LOCATION Beaver Creek Bridge, Site No. 11-034, N4964919.8, E215012.1 ORIGINATED BY

DIST 43 HWY 82 BOREHOLE TYPE Portable, Cased, Splitspoons COMPILED BY

DATUM Geodetic DATE 30.11.05 - 30.11.05 CHECKED BY

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: C UNCONFINED, × FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100) | PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT	GR	SA	SI	CL
	SANDY SILT with seams of silty sand and silty clay, very loose to compact, grey	13	SS	13						
					291					
	(ML)	14	SS	10	290	o	0	3	92	5
					289					
288.1 14.3	Inferred sandy silt, some gravel, occasional cobbles, compact to dense, grey (TILL)				288					
					287					
					286					
					285					
284.4 18.0	End of Borehole									

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

MTO 11686MTO.GPJ ON_MOT.GDT 27/06/06

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No BH05-4

METRIC

W.P. 248-99-00 LOCATION Beaver Creek Bridge, Site No. 11-034, N4964939.9, E215015.7 ORIGINATED BY JD

DIST 43 HWY 62 BOREHOLE TYPE Portable, Cased, Splitspoons COMPILED BY JD

DATUM Geodetic DATE 30.11.05 - 01.12.05 CHECKED BY PC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT (%)	GR	SA	SI	CL
302.4	Tall Grass									
302.2 / 0.2	SANDY SILT, trace organics, trace roots, very loose, dark brown									
	SANDY SILT, very loose, grey to brown	1	SS	2	302					
301.8 / 0.6	SANDY SILT with seams of silty sand and silty clay, loose, grey	2	SS	4						
	(ML)	3	SS	7	301	~24	0	0	92	8
		4	SS	6	300					
		5	SS	7	299					
					298					
	(SM)	6	SS	6	297	~21	0	53	41	6
		7	SS	4	296					
295.7 / 6.7	End of Borehole									

Standpipe Installed (25 mm diameter slotted flexible polytube)

Water level measured December 1, 2005

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED ✕ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No BH06-3

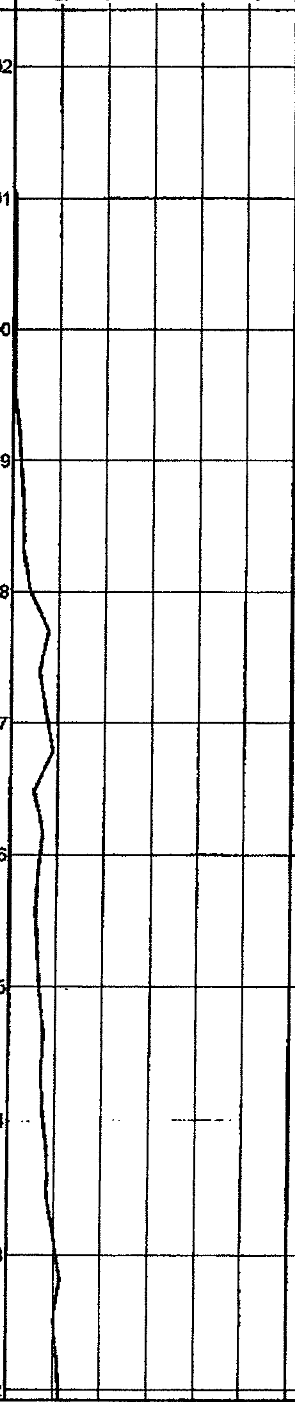
METRIC

W.P. 248-99-00 LOCATION Beaver Creek Bridge, Site No. 11-034, N4964919.6, E215012.6 ORIGINATED BY JD

DIST 43 HWY 62 BOREHOLE TYPE Portable, Dynamic Cone Penetration Test COMPILED BY JD

DATUM Geodetic DATE 17.01.06 - 17.01.06 CHECKED BY PC

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa O UNCONFINED × FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
302.4	Ice										
302.2	Ice										
0.2	Water						302				
301.2 / 1.2	Inferred SILTY SAND, some organics, trace wood chips, trace roots, very loose, brown						301				
							300				
299.0 / 3.4	Inferred SANDY SILT with seams of silty sand and silty clay, very loose to compact, grey						299				
							298				
							297				
							296				
							295				
							294				
							293				
							292				



Continued Next Page

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No BH06-3

METRIC

W.P. 248-99-00	LOCATION Beaver Creek Bridge, Site No. 11-034, N4964919.5, E215012.6	ORIGINATED BY JD
DIST 43 HWY 62	BOREHOLE TYPE Portable, Dynamic Cone Penetration Test	COMPILED BY JD
DATUM Geodetic	DATE 17.01.06 - 17.01.06	CHECKED BY PC

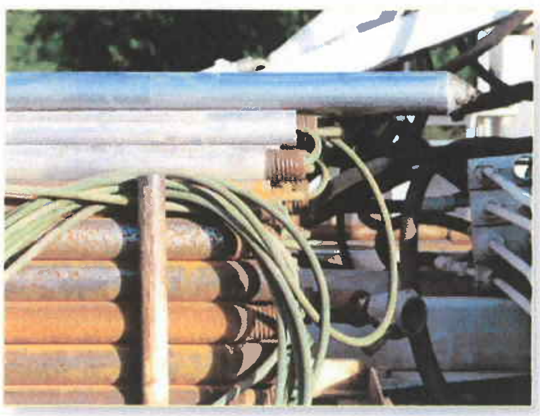
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, × FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Inferred SANDY SILT with seams of silty sand and silty clay, very loose to compact, grey						291, 290, 289				
288.0 / 14.4	Inferred sandy silt, some gravel, occasional cobbles, compact to dense, grey (TILL)						288, 287, 286, 285, 284, 283				
282.3 / 20.1	End of Borehole Refusal										

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

APPENDIX B

FIELD DATA REPORT
SEISMIC CONE PENETRATION TESTING
BY CONETEC INVESTIGATIONS LTD.

Field Data Report



ConeTec Investigations Ltd.

12140 Vulcan Way
Richmond, BC V6V 1J8

Tel: 604-273-4311
Fax: 604-273-4066

Toll Free: 800-567-7969

Email: insitu@conetec.com

Prepared for:

Golder Associates Ltd.
Beaver Creek - Bancroft, ON

Cone Penetration Test Data
Job No: 08-019

- January 14, 2008 -

Cone Penetration Tests (CPTU)

The cone penetration tests (CPTU) with pore pressure measurement were carried out by ConeTec using an integrated electronic cone system.

All soundings were performed using compression type cone penetrometers (refer to Figure CPTU). ConeTec has cones of various cross sectional areas and capacities. ConeTec's 10 ton cones have a tip area of 10 cm^2, a friction sleeve area of 150 cm^2 and tip capacity of 1000 bar. ConeTec's 20 Ton cones have a tip area of 15 cm^2, a friction sleeve area of 225 cm^2 and a tip capacity of 1500 bar. ConeTec's Medium Capacity cones (MC375) have a tip area of 15 cm^2, a friction sleeve area of 225 cm^2 and a tip capacity of 375 bar. The compression cones are designed with an equal end area friction sleeve and a tip end area ratio of 0.85. A porewater pressure filter is located directly behind the cone tip. The filter is made of porous plastic and is 5.0 mm thick. Each porewater pressure filters is saturated under vacuum pressure prior to penetration. Porewater pressure dissipation data is recorded at 5-second intervals during pauses in penetration as directed by the field representative.

The cone system is capable of recording the following parameters at varying depth intervals:

- Tip Resistance (q_c)
- Sleeve Friction (f_s)
- Dynamic Pore Pressure (u)
- Temperature (T)
- Cone Inclination (I)

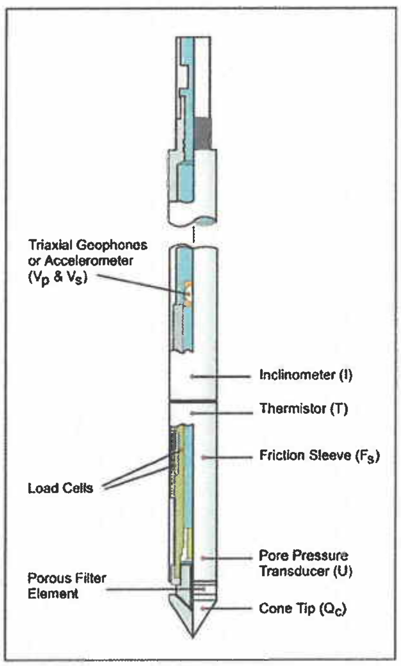


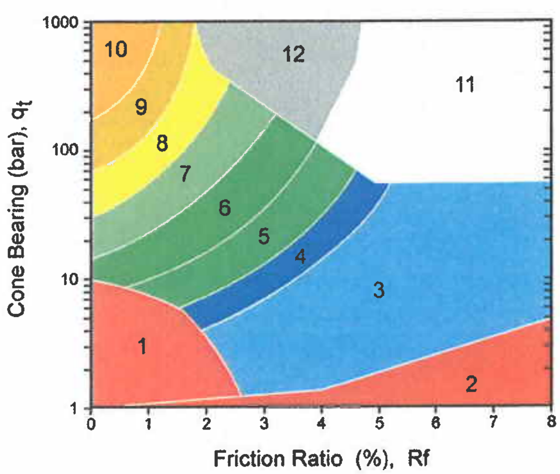
Figure – CPTU

A summary of the cone penetration tests carried out is presented in Table CPTU (Appendix CPTU).

Selected parameters were printed simultaneously on a printer and stored on a floppy disk for future analysis and reference. All cone penetration testing was carried out in accordance with ASTM D-5778-95.

A complete set of baseline readings was taken prior to and at the completion of each sounding to determine temperature shifts and any zero load offsets. Corrections for temperature shifts and zero load offsets can be extremely important, especially when the recorded loads are relatively small. In sandy soils, however, these corrections are generally negligible. Graphical plots of all CPT data are presented in Appendix CPTU.

The inferred stratigraphic profile at each CPT test location is included with this report. The stratigraphic interpretations are based on relationships between cone bearing, q_t, sleeve friction, f_s, and dynamic pore pressure, u. The friction ratio, R_f (100 x f_s/q_t), is a calculated parameter which is used to identify the type of soil and hence gives an indication of its behavior. Generally, soft cohesive soils have high friction ratios, low cone bearing pressures and generate large porewater pressures during penetration. Cohesionless soils have lower friction ratios, high cone bearing pressures and generate little in the way of excess porewater pressure during penetration. The classification of soils is based on correlations summarized by Robertson (1990), as shown in Figure SBT. It is not always possible to clearly identify a soil type based on q_t and f_s alone. Experience, judgment and analyses of porewater pressure generation during penetration and subsequent dissipation tests should be used in arriving at the soil type in these ambiguous situations.



Zone	q_t / N	Soil Behavior Type
1	2	Sensitive fine grained
2	1	Organic soil
3	1	Clay
4	1.5	Silty clay to clay
5	2	Clayey silt to silty clay
6	2.5	Sandy silt to clayey silt
7	3	Silty sand to sandy silt
8	4	Sand to silty sand
9	5	Sand
10	6	Gravelly sand to sand
11	1	Very stiff fine-grained soil *
12	2	Very stiff sand to clayey sand *

* overconsolidated or cemented

Figure SBT – Non-Normalized Soil Behavior Type Chart, Robertson (1990)

It should be noted that stratigraphic interpretation using CPTU data can also be carried out using a normalized (stress corrected) soil behavior type chart (Robertson, 1990). The Robertson publication emphasizes when normalized stratigraphic interpretation is appropriate.

CONETEC Seismic Cone Penetration Testing (SCPTU)

Seismic wave velocity measurements were conducted at regular intervals during the cone penetration test. Seismic wave velocity measurements were made according to the procedures described by Robertson et.al. (1986). Before taking wave velocity measurements, the rods were decoupled from the CPT rig to avoid transmission of energy down the rods.

The seismic waves were generated using a hammer striking a steel beam that was coupled to the ground by a hydraulic cylinder under the CPT rig (refer to Figure S). The sledgehammer striking the beam acts as an electrical contact trigger, initiating the recording of the seismic wave traces. The offset of the beam from the cone was taken into account during calculation of the seismic wave velocities.

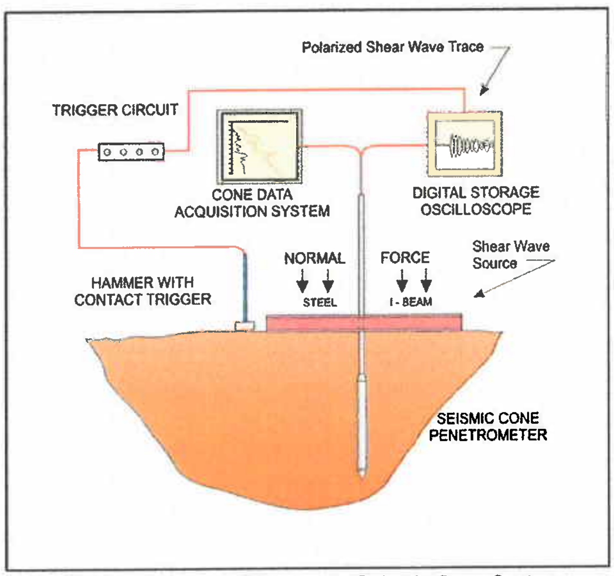


Figure S – Layout of Downhole Seismic Cone System

At each test depth, at least two waves were recorded. Multiple waves are recorded at each end of the beam to enable the operator to check the consistency of the waveforms. The seismic wave receiver used was a horizontally active geophone located in the body of the cone penetrometer. The geophone is located approximately 0.2 meters behind the cone tip. This offset is accounted for in all calculations. Data was sampled at a frequency of 20kHz (i.e. 20,000 samples per second) with a total of 5000 points being recorded per wave trace. To maintain the desired signal resolution, the input sensitivity (gain) of the receiver was increased with depth.

Table CPTU (Appendix CPTU) provides a summary of the seismic cone penetration tests carried out. The seismic wave velocity results are presented in both tabular and graphical form in Appendix SCPTU.

Pore Pressure Dissipation Testing (PPD)

The penetration of the piezocone was halted at specific depths to carry out pore pressure dissipation tests as directed by the field representative. The variation of the penetration pore pressure (u) with time was measured and recorded. All pore pressure data was recorded immediately behind the cone tip at the u_2 location (refer to Figure PTL).

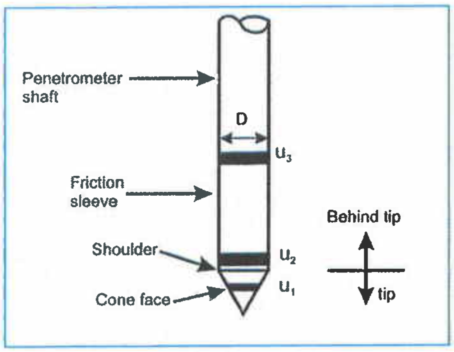


Figure – PTL

Pore pressure dissipation data can be interpreted to provide estimates of :

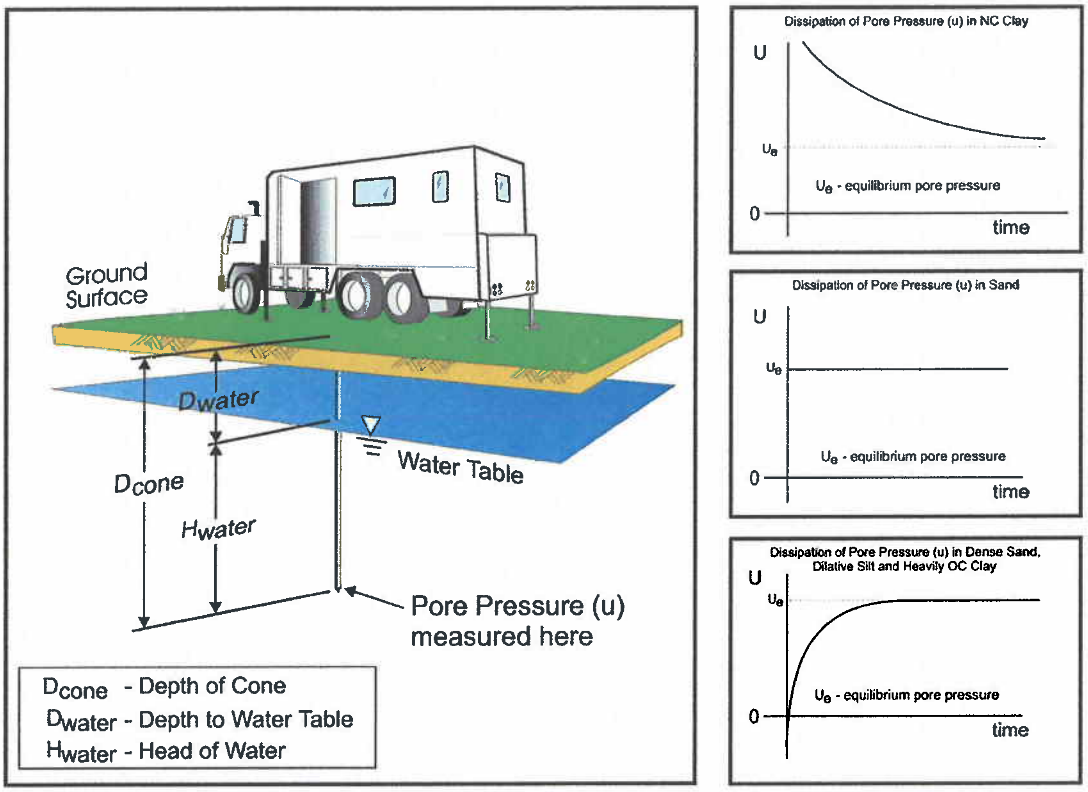
- equilibrium piezometric pressure
- phreatic surface
- in situ horizontal coefficient of consolidation, c_h
- in situ horizontal coefficient of permeability, k_h

In order to interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until such time as there is no variation in pore pressure with time (refer to Figure PPD). This time is commonly referred to as t_{100}, the point at which 100% of the excess pore pressure has dissipated.

Interpretation of either c_h and k_h from dissipation results can be most easily achieved using either of two analytical approaches: cavity-expansion theory or the strain-path approach. Comparisons of the available solutions and results from field studies suggest that the cavity-expansion method of Torstensson (1977) and the strain-path approaches of Levadous (1980) and Teh (1987) all provide similar predications of consolidation parameters from CPTU dissipation data (Gillespie 1981; Kabir and Lutenegger 1990; Robertson et al. 1991). Robertson et al. (1991) have shown that these methods, although developed for normally consolidated soils, can be equally applied to overconsolidated soils. Furthermore, comparisons of field and laboratory data indicate that the trends in the measured (laboratory) and predicated (CPTU) data are consistent provided the micro fabric and nature of the soils being tested are taken into consideration (Danziger 1990; Robertson et al. 1991).

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1991.

The pore pressure dissipation tests are summarized in Table PPD (Appendix PPD).



Water Table Calculation

$$D_{water} = D_{cone} - H_{water}$$

where $H_{water} = U_e$ (depth units)

Useful Conversion Factors:	1psi = 0.704m = 2.31 feet (water)
	1tsf = 0.958 bar = 13.9 psi
	1m = 3.28 feet

Figure - PPD

ConeTec Digital File Formats

CPT Data Files (COR Extension)

ConeTec data files are stored in ASCII text files that are readable by almost any text editor. ConeTec CPT data files are named such that the first 3 characters contain the job number, the next two characters are CP followed by two characters indicating the sounding number. The last 8th character position is reserved for the letters a, b, c, d etc to uniquely identify multiple soundings at the same location. The CPT sounding file has the extension COR, and pore pressure dissipation files have the extension PPD or PPF. As an example, for job number 06-127 the first sounding will have file names 127CP01.COR and 127CP01.PPD.

The sounding (COR) file consists of the following components:

1. Two lines of header information
2. Data records
3. End of data marker
4. Units information

Header Lines

Line 1: Columns 1-6 may be blank or may indicate the version number of the recording software
Columns 7-21 contain the sounding Date and Time
Columns 22-36 contain the sounding Operator

Line 2: Columns 1-16 contain the Job Location
Columns 17-31 contain the Cone ID
Columns 32-47 contain the sounding number

Data Records

The data records contain 4 or more columns of data in floating point format. A comma (and spaces) separates each data item:

Column 1:	Sounding Depth (meters)
Column 2:	Tip (q_c) data uncorrected for pore pressure effects. Recorded in units selected by the operator.
Column 3:	Sleeve (f_s) data. Recorded in units selected by the operator
Column 4:	Dynamic pore pressure readings. Recorded in units selected by the operator
Column 5:	Empty, Resistivity, UVIF or Gamma data

End of Data Marker

After the last line of data there will be a line containing ASCII 26 (CTL-Z) and a newline (carriage return/ line feed) character. This is used to mark the end of data.

Units Information

The last section of the file contains information about the units that were selected for the sounding. A separator bar makes up the first line. The second line contains the type of units used for depth, q_c, f_s and u. The third line contains the conversion values required for ConeTec's software to convert the recorded data to an internal set of base units (bar for q_c, bar for f_s and meters for u).

CPT Dissipation Files (PPF Extension)

CPT Dissipation files have the same naming convention as the CPT sounding files and have the extension PPD or PPF. PPF (and PPD) files consist of the following components:

1. Two lines of header information
2. Data records

Header Lines (same as COR file):

Line 1: Columns 1-6 may be blank or may indicate the version number of the recording software
Columns 7-21 contain the sounding Date and Time
Columns 22-36 contain the sounding Operator

Line 2: Columns 1-16 contain the Job Location
Columns 17-31 contain the Cone ID
Columns 32-47 contain the sounding number

Data Records

The data records immediately follow the header lines. Each data record can occupy several lines in the file and is a complete record of a dissipation test at a particular depth. Each data record starts with a line containing two values separated by spaces; the first value being an index number (not currently used by the Software) and the second being the dissipation test depth in meters. Following this line are the dissipation pore pressure values stored at 5 second intervals with a maximum of 12 entries per line. The last line of the dissipation record may not contain a full 12 entries. The data record is terminated with an ASCII 30 character (appears as a triangle in some editors).

This sequence is repeated for every dissipation test in the sounding. No marker is used to indicate end of file. Units information is not stored in this file. Users would have to check the CPT file for the units that were used.

CPT Basic Interpretations (TBL Extension)

ConeTec's basic CPT interpretation output files are generally delivered in text files with a TBL extension. The root file name is the same as the COR files. A number of calculated geotechnical parameters are presented in these files. The files are stored as ASCII text files that can be viewed using any text editor such as Notepad or Wordpad. The files do not contain any page formatting. These files are not distributed if the enhanced interpretation files are provided.

CPT Enhanced Interpretations (IFI, IFP, XLS Extension)

ConeTec's enhanced CPT interpretation output files are delivered in several formats, each file type containing the exact same information but formatted slightly differently. The files typically have any of the following file extensions:

1. IFI an importable TAB delimited ASCII text file containing approximately 47 data columns of geotechnical interpretations. The file is designed for easy import to Excel. A companion document describes the techniques used for the interpretations (usually reproduced at the beginning of the Interpretation Appendix). Text editors can be used to view the file contents, however, they may remove the tabs or replace the tabs with spaces upon saving the file destroying the feature that makes them easy to import into Excel.

   Because Excel imports the data as text and the sheet is protected two steps may be necessary to modify the data or use the values in certain Excel functions:

   a) Under Tools (Excel 2000) Select the Protection Option and then Unprotect the sheet
   b) Select the entire sheet, copy and then use Paste Special to paste as values to a second sheet.

   Future versions of our interpretation routine will address these inconveniences.

2. IFP a printable ASCII text file containing the same 47 columns of geotechnical interpretations as the IFI file. This file type has been formatted as a multi-page document with up to 132 characters per line and up to 68 lines per page. Each page has been separated into multiple sections to accommodate all the data fields. Each physical page has a header section and a page/section number. The file is designed for direct printing to laser printers set into compressed font mode. This output is typically provided in the Interpretation Appendix.

   An abbreviated set of interpretations (containing 36 columns of output) may be generated instead. These files usually have the extensions NLI and NLP. XLS files can be generated from these as well.

3. XLS an Excel format file that has been generated directly from the corresponding IFI file. IFI and IFP files are not distributed if the XLS files are generated. The XLS files may have been generated from abbreviated NLI interpretation files.

In each case root file name is the same as the COR files.

Bibliography

Robertson, P.K., "Soil Classification using the Cone Penetration Test", Canadian Geotechnical Journal, Vol 27, 1990 pp 151-158.

Robertson, P.K., R.G. Campanella, D. Gillespie and A. Rice, "Seismic CPT to Measure In-Situ Shear Wave Velocity", Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8, 1986 pp 791-803.

Campanella, R.G. and I. Weemees, "Development and Use of An Electrical Resistivity Cone for Groundwater Contamination Studies", Canadian Geotechnical Journal,Vol. 27 No. 5, 1990 pp 557-567.

Kurfurst, P.J. and D.J. Woeller, "Electric Cone Penetrometer – Development and Field Results From the Canadian Arctic", Penetration Testing 1988 ISOPT, Orlando,Volume 2 pp 823-830.

Zemo, D.A., T.A. Delfino, J.D. Gallinatti, V.A. Baker and L.R. Hilpert, "Field Comparison of Analytical Results from Discrete-Depth Groundwater Samplers" BAT Enviroprobe and QED Hydropunch, Sixth national Outdoor Action Conference, Las Vegas, Nevada Proceedings, 1992, pp 299-312.

Lunne, T., Robertson, P.K. and Powell, J.J.M., 1997, "Cone Penetration Testing in Geotechnical Practice", E & FN Spon, ISBN 0 419 23750, www.sponpress.com, 312 pages, 3rd printing.

Greig, J.W., R.G. Campanella and P.K. Robertson, "Comparison of Field Vane Results With Other In-Situ Test Results", International Symposium, on Laboratory and Field Vane Shear Strength Testing, ASTM, Tampa, FL, Proceedings, 1987.

DeGroot, D.J. and A.J. Lutenegger, "Reliability of Soil Gas Sampling and Characterization Techniques", International Site Characterization Conference - Atlanta, 1998.

Daniel, C.R., J.A. Howie and A. Sy, "A Method for Correlating Large Penetration Test (LPT) to Standard Penetration Test (SPT) Blow Counts", 55th Canadian Geotechnical Conference, Niagra Falls, Ontario, Proceedings ,2002.

Woeller, D.J., P.K. Robertson, T.J. Boyd and Dave Thomas, "Detection of Polyaromatic Hydrocarbon Contaminants Using the UVIF-CPT", 53rd Canadian Geotechnical Conference Montreal, QC October pp 733739, 2000.

Appendix CPTU
Cone Penetration Tests

CONETEC

Job No: 08-019
Client: Golder Associates Ltd.
Project: Beaver Creek, Bancroft, ON
Date: 14-Jan-08

CPT SUMMARY

CPT Sounding	File Name	Date	Cone	Assumed Phreatic Surface (m)	Final Depth (m)
SCPT-07-24	019SCP01	01/14/08	Med. Cap. AD-199	0.0	12.18
SCPT-07-23	019SCP02	01/14/08	Med. Cap. AD-199	0.0	13.43

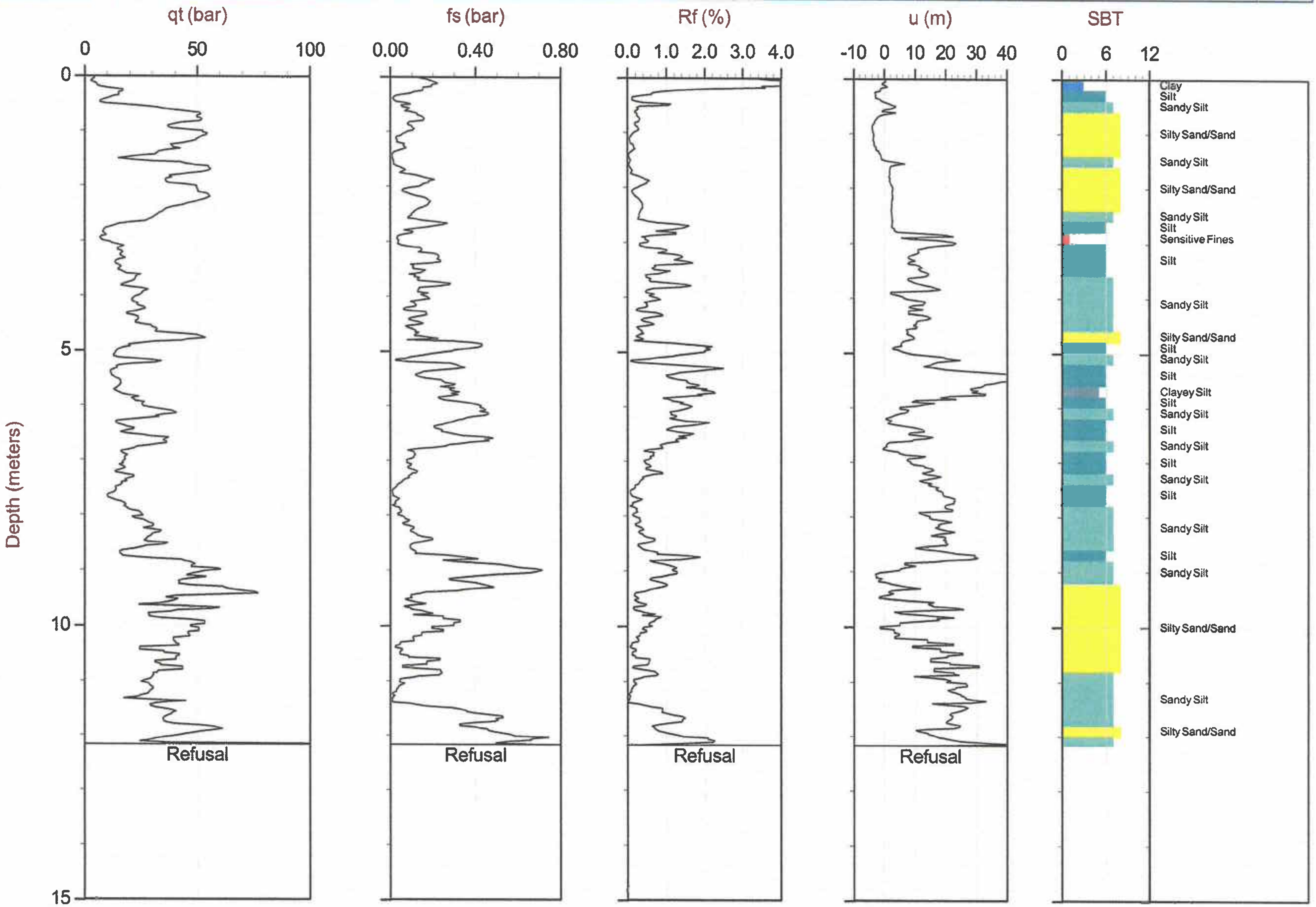
Note: Hydrostatic condition assumed for interpretation tables and based on pore pressure dissipations.

ConeTec

Golder Associates

Job No: 08-019
Date: 01:14:08 12:15
Site: Beaver Creek, Bancroft, ON

Sounding: SCPT-07-24
Cone: Med. Cap. AD-199



Max Depth: 12.175 m / 39.94 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: 0.200 m

File: 019SCP01.COR
Unit Wt: SBT Chart Soil Zones

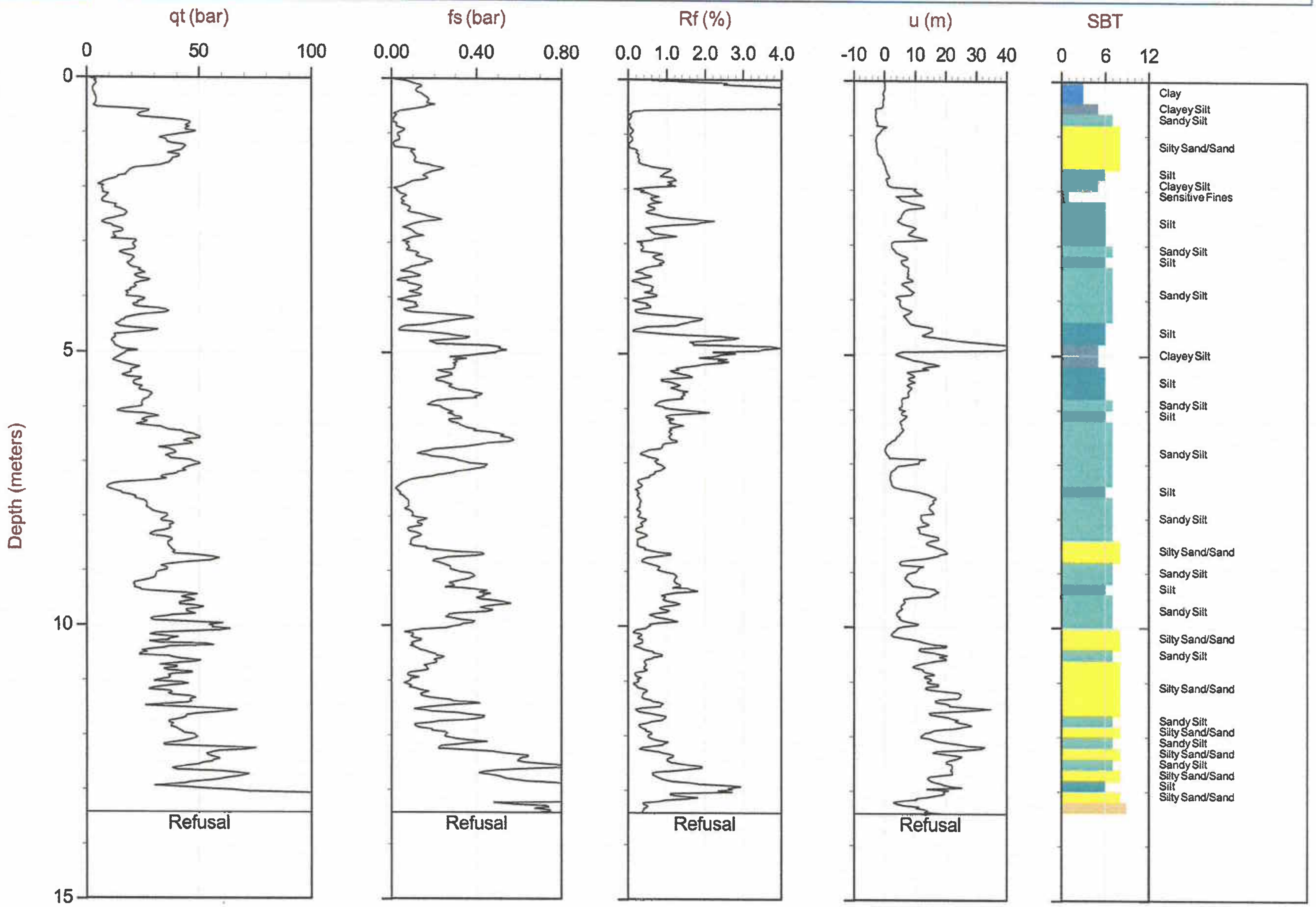
SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

CONETEC

Golder Associates

Job No: 08-019
Date: 01:14:08 15:14
Site: Beaver Creek, Bancroft, ON

Sounding: SCPT-07-23
Cone: Med. Cap. AD-199



Max Depth: 13.425 m / 44.04 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: 0.200 m

File: 019SCP02.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

Appendix SCPTU
Seismic Cone Penetration Test

CONETEC

Job No: 08-019
Client: Golder Associates
Project Title: Beaver Creek, Bancroft, ON
Hole: SCPT-07-23
Date: Jan. 14, 2008

Seismic Source: Auger
Source Offset (m): 1.94
Source Depth (m): 0.00
Geophone Offset (n 0.20

SEISMIC

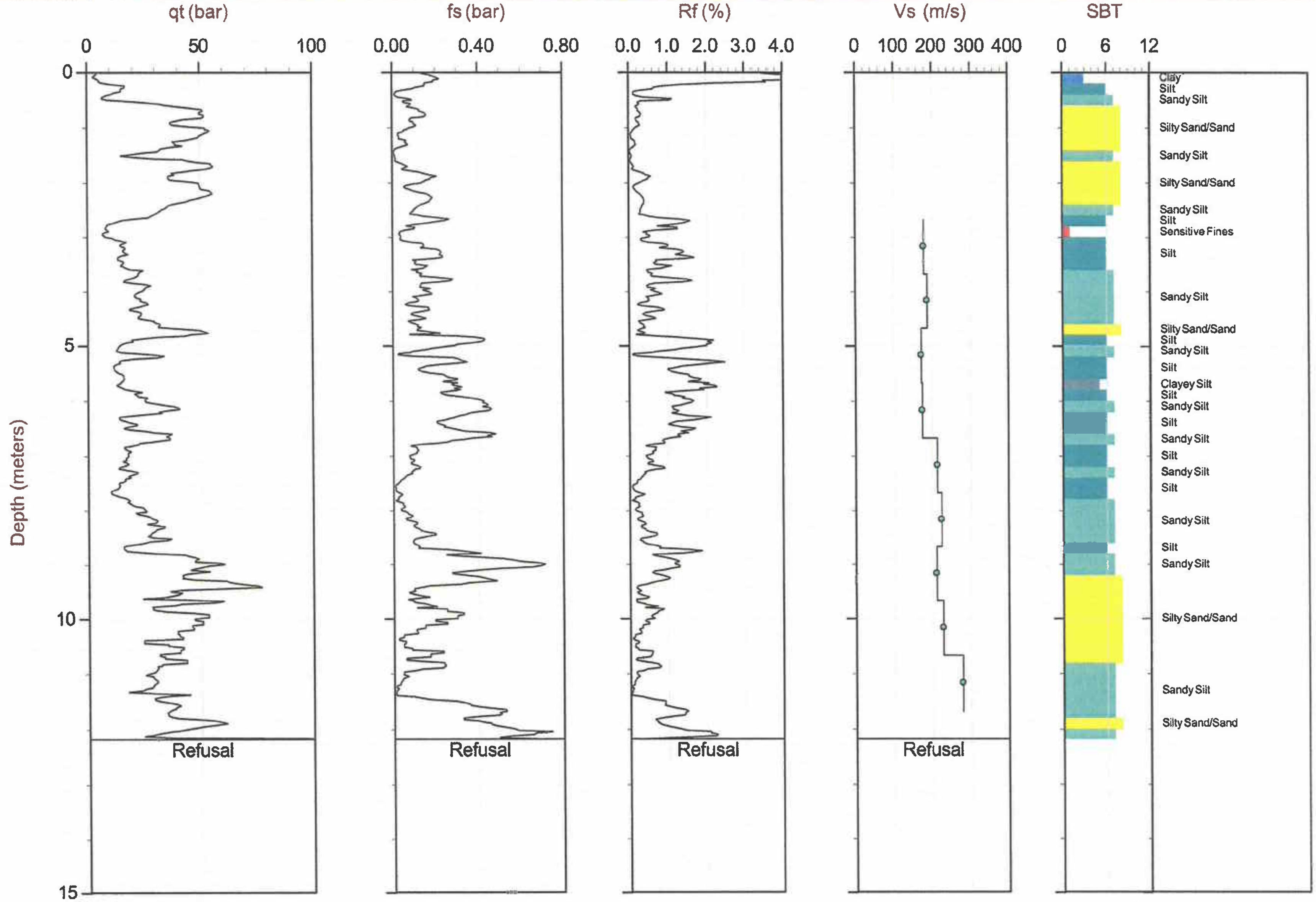
Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Depth Interval (m)	Time Interval (ms)	Vs (m/s)	Mid Layer (m)
2.92	2.72	3.34				
3.92	3.72	4.20	0.85	6.08	141	3.22
4.92	4.72	5.10	0.91	5.69	160	4.22
5.92	5.72	6.04	0.94	4.38	214	5.22
6.92	6.72	6.99	0.95	4.22	226	6.22
7.92	7.72	7.96	0.97	3.95	244	7.22
8.92	8.72	8.93	0.97	3.87	251	8.22
9.92	9.72	9.91	0.98	3.38	290	9.22
10.92	10.72	10.89	0.98	3.63	271	10.22
11.92	11.72	11.88	0.99	3.40	290	11.22

CONETEC

Golder Associates

Job No: 08-019
Date: 01:14:08 12:15
Site: Beaver Creek, Bancroft, ON

Sounding: SCPT-07-24
Cone: Med. Cap. AD-199



Max Depth: 12.175 m / 39.94 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: 0.200 m

File: 019SCP01.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

CONETEC

Job No: 08-019
Client: Golder Associates
Project Title: Beaver Creek, Bancroft, ON
Hole: SCPT-07-24
Date: 14 Jan. 2008

Seismic Source: Auger
Source Offset (m): 1.97
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SEISMIC

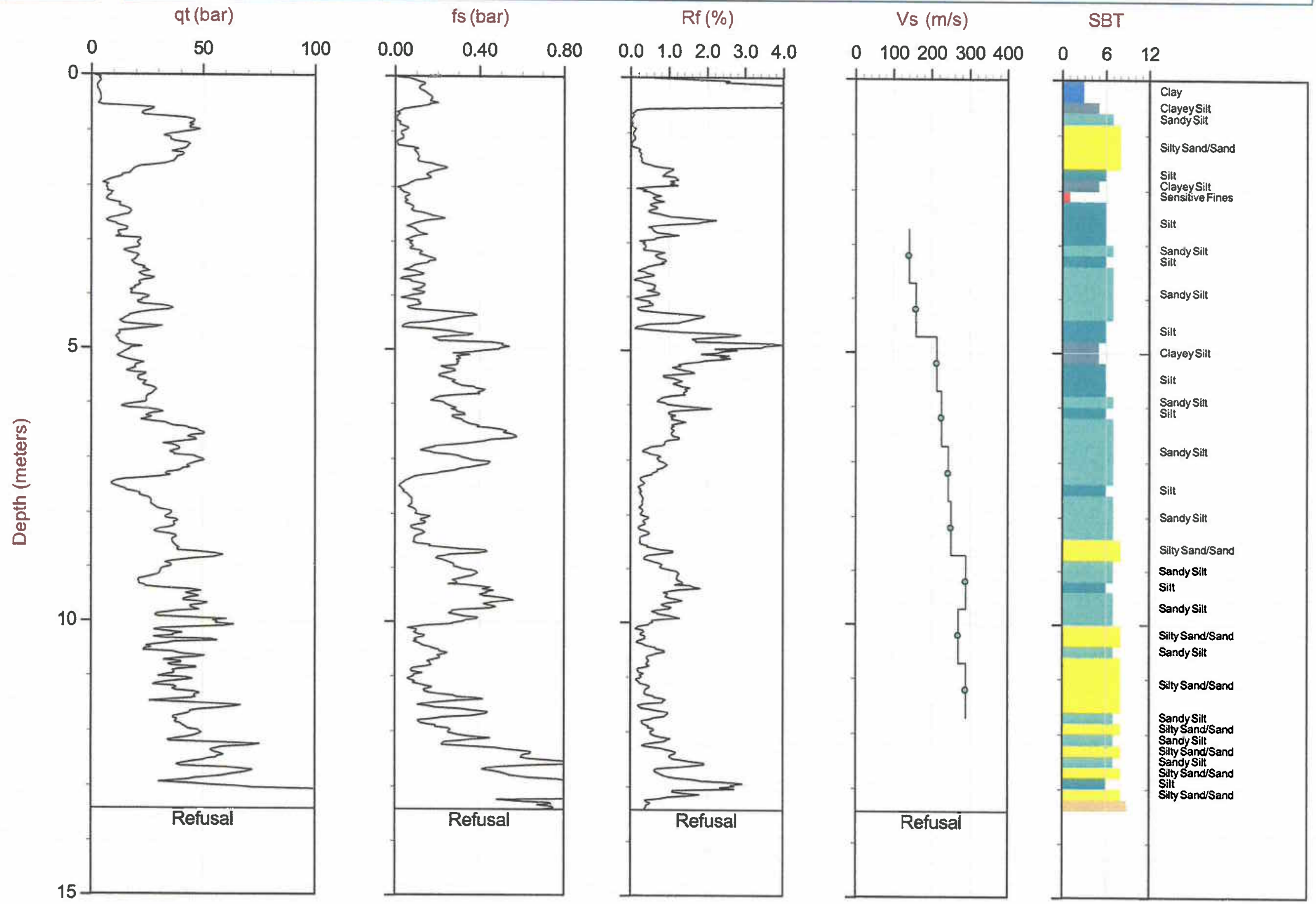
Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Depth Interval (m)	Time Interval (ms)	Vs (m/s)	Mid Layer (m)
2.88	2.68	3.33				
3.88	3.68	4.17	0.85	5.11	166	3.18
4.88	4.68	5.08	0.90	5.34	169	4.18
5.88	5.68	6.01	0.93	5.03	186	5.18
6.88	6.68	6.96	0.95	4.94	193	6.18
7.88	7.68	7.93	0.96	4.51	214	7.18
8.88	8.68	8.90	0.97	4.32	225	8.18
9.88	9.68	9.88	0.98	4.62	212	9.18
10.88	10.68	10.86	0.98	4.28	229	10.18
11.88	11.68	11.84	0.98	3.53	279	11.18

CONETEC

Golder Associates

Job No: 08-019
Date: 01:14:08 15:14
Site: Beaver Creek, Bancroft, ON

Sounding: SCPT-07-23
Cone: Med. Cap. AD-199



Max Depth: 13.425 m / 44.04 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: 0.200 m

File: 019SCP02.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 1

Appendix PPD
Pore Pressure Dissipation Tests

CONETEC

Job No: 08-019
Client: Golder Associates Ltd.
Project: Beaver Creek, Bancroft, ON
Date: 14-Jan-08

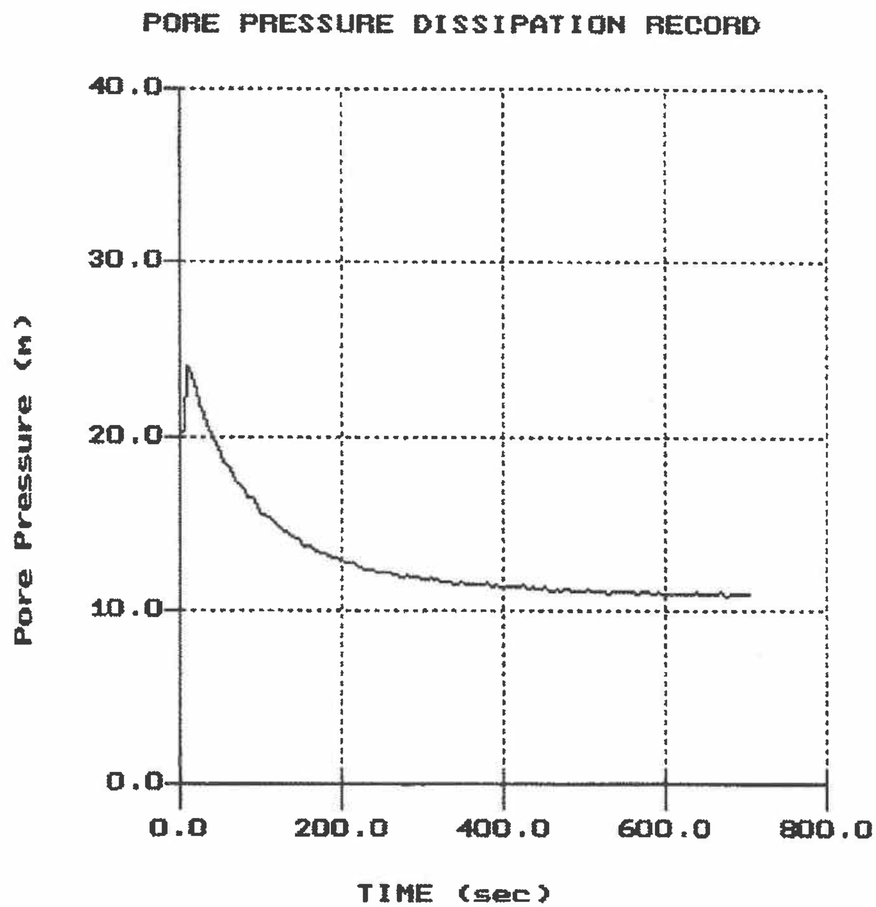
PPD SUMMARY

CPT Sounding	Duration (s)	Test Depth (m)	Equilibrium Pore Pressure U_{eq}(m)*	Calculated Phreatic Surface (m)*	Estimated Phreatic Surface (m)	T_{50} (sec)
SCPT-07-24	190	2.88	Not Achieved	-	-	-
	415	4.88	5.1	-0.2	-	-
	705	9.88	11.0	-1.1	-	-
SCPT-07-23	900	4.93	Not Achieved	-	0.0	78

* Equilibrium pore pressure estimated from dissipation tests.
** Negative phreatic surface indicate artesian conditions

Golder Associates | Hole: SCPT07-24 | Location: Bancroft | Cone: MedCap.AD-199 | Date: 01:14:08 12:15

File: 019SCP01.PPD
Depth (m): 9.88
(ft): 32.40
Duration : 705.0s
U-min: 10.89 675.0s
U-max: 24.02 10.0s



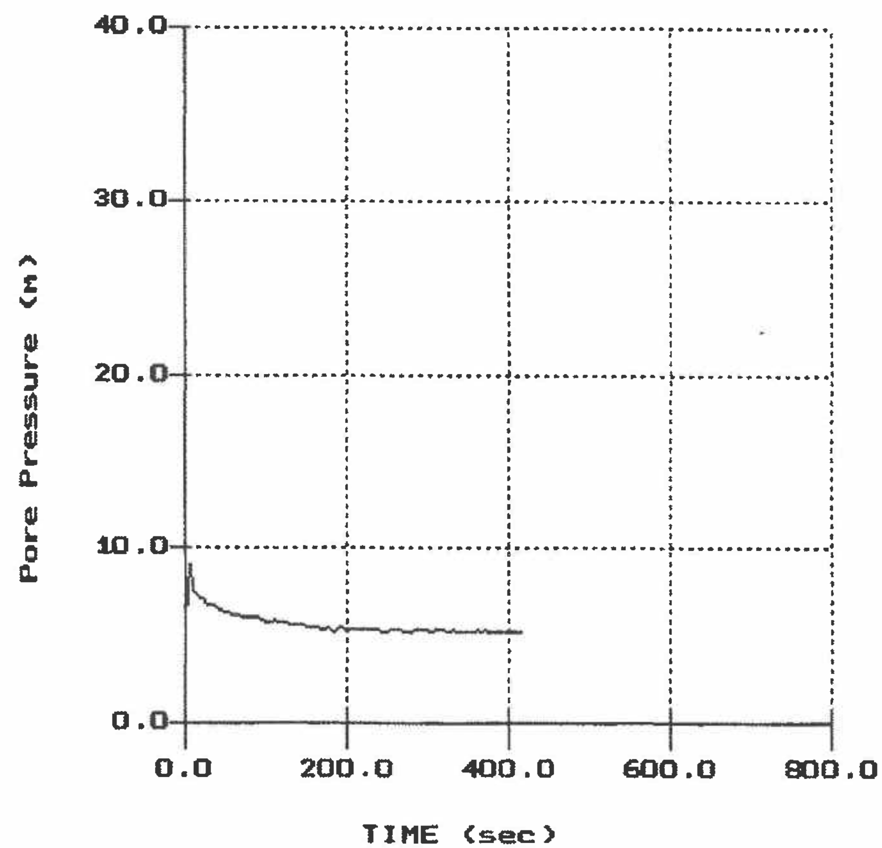
Golder Associates

Hole: SCPT07-24
Location:Bancroft

Cone:MedCap.AD-199
Date:01:14:08 12:15

File: 019SCP01.PPD
Depth (m): 4.88
(ft): 15.99
Duration : 415.0s
U-min: 5.08 385.0s
U-max: 9.07 5.0s

PORE PRESSURE DISSIPATION RECORD

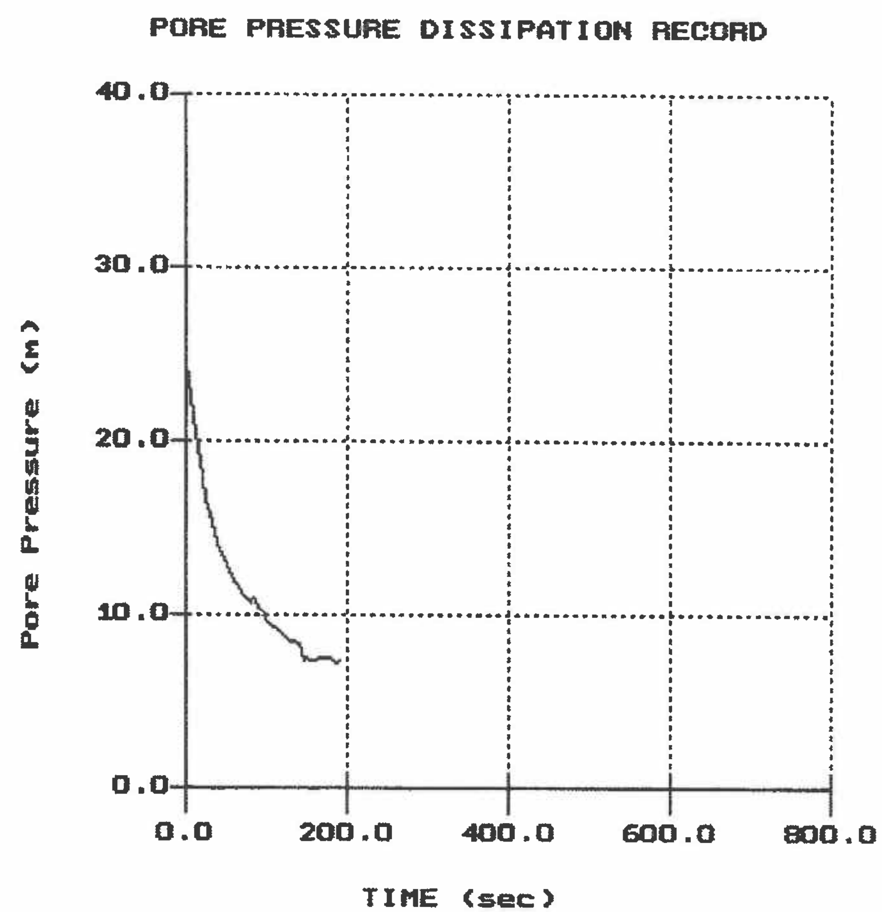


Golder Associates

Hole: SCPT07-24
Location:Bancroft

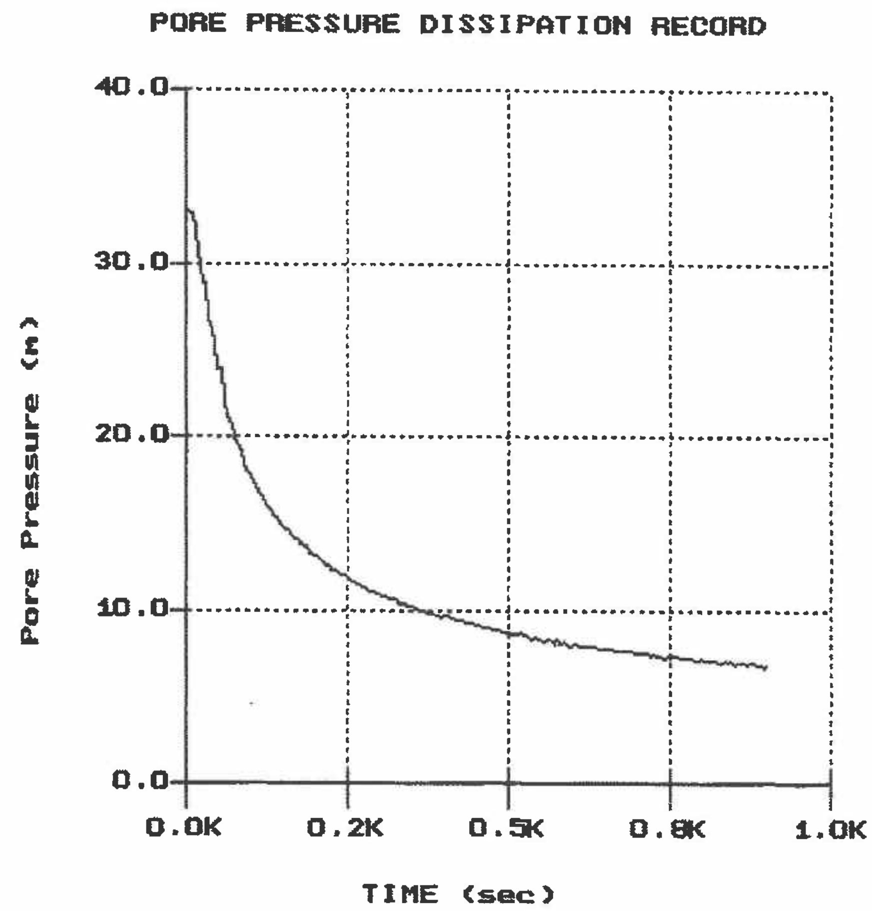
Cone:MedCap.AD-199
Date:01:14:08 12:15

File: 019SCP01.PPD
Depth (m): 2.88
(ft): 9.43
Duration : 190.0s
U-min: 7.26 185.0s
U-max: 24.38 0.0s



Golder Associates | Hole:SCPT-07-23
Location:Beaver Creek | Cone:Med. Cap. AD-199
Date:01:14:08 15:39

File: 019SCP02.PPD
Depth (m): 4.93
(ft): 16.16
Duration : 900.0s
U-min: 6.71 895.0s
U-max: 33.03 5.0s



APPENDIX C

NON-STANDARD SPECIAL PROVISIONS

AMENDMENT TO OPSS 539, NOVEMBER 2003

Special Provision No. 105S19M June 2009

OPSS 539, November 2003, Construction Specification for Protection Systems is deleted in its entirety and replaced with the following:

CONSTRUCTION SPECIFICATION FOR PROTECTION SYSTEMS AND COFFER DAMS

539.01 SCOPE

This specification covers the requirements for the design, construction, maintenance, monitoring and removal of a protection system made necessary by excavation or other work.

As part of the work under this item, the Contractor shall design, supply, install and maintain coffer dams to construct the pile caps for the new north and south abutments as shown on the Contract Drawings and, following completion of construction, cut off the top of the coffer dams below grade to the limits indicated in the Contract Drawings.

This special provision also describes requirements for vibration monitoring on the existing Beaver Creek bridge during protection system and/or cofferdam installation works at the bridge site.

539.02 REFERENCES

This specification refers to the following standards, publications or specifications:

Ontario Provincial Standard Specifications, General:

OPSS 180 Management of Excess Material

Ontario Provincial Standard Specifications, Construction:

OPSS 903 Piling
OPSS 904 Concrete Structures
OPSS 906 Structural Steel

Ontario Provincial Standard Specifications, Material:

OPSS 1350 Concrete Materials and Production
OPSS 1601 Wood Material, Preservative Treatment and Shop Fabrication

Ontario Ministry of Labour:

Occupational Health and Safety Act, R.S.O. 1990, c.O.1, as amended

American Association of State Highways Transportation Officials:

AASHTO Guide Design Specification for Bridge Temporary Works, 1995

Canadian Standards Association

CAN/CSA-S6-06, Canadian Highway Bridge Design Code

539.03 DEFINITIONS

For the purpose of this specification, the following definitions apply.

Anchorage System: means a system consisting of tendons installed in predrilled holes in soil or rock and encapsulated in grout or concrete that derives its load carrying capacity in bond between the grout/concrete body and the surrounding soil or rock; or tie back to deadmen.

Bracing: means the system of walers, struts, anchorages and like members that connect frames, shores or panels of a sheathing system to resist external pressures and to provide stability against lateral movement.

Coffer Dam: means a water-tight enclosure.

Design Engineer: means the Engineer retained by the Contractor who produces the original design and working drawings.

Design Checking Engineer: means the Engineer retained by the Contractor who checks the original design and working drawings.

Dredge Line: means the exposed lower limit of the Protection System.

Erector: means a person that undertakes the construction of a Protection System.

Protection System: means the construction necessary to mechanically support existing or proposed work such that its function will not be affected, or, construction necessary to support work, such as open excavations, during actual construction operations for safety and convenience.

Quality Verification Engineer: means the Engineer, retained by the Contractor, qualified to determine that the work is in general conformance with the Contract Documents and issue Certificate(s) of Conformance.

Raker: means a structural member inclined to the front of the shoring wall providing lateral support.

Shoring Wall: means a structural wall consisting of wood, steel, concrete or combination of these materials that supports earth or rock and any structure, materials, utilities or other facility contained in or on the supported earth or rock mass.

Stamped: means drawings or details that have been reviewed and stamped "In General Conformance with Contract Documents". The stamp shall include the date and signature of the Quality Verification Engineer.

Top of Shoring Wall: means the upper limit of the Protection System.

539.04 SUBMISSION AND DESIGN REQUIREMENTS

539.04.01 Submissions

539.04.01.01 Working Drawings/Details

Three (3) copies of stamped working drawings shall be submitted to the Contract Administrator for information purposes at least one (1) week before commencement of construction of the protection system and/or coffer dam.

All submissions shall bear the seal and signature of the Design Engineer and Design Checking Engineer.

For contracts where another authority, such as a railway or navigable waters, is affected the Contractor shall submit working drawings to each authority (number of sets of drawings to be determined by the authority). The requirements of each authority shall be satisfied before commencement of protection system and/or coffer dam installation.

The Contractor shall have a copy of the stamped working drawings at the site during protection system and/or coffer dam construction.

For protection systems and/or coffer dams that are not specified in the Contract Documents, the Contractor shall submit to the Owner working drawings of these systems at least three weeks prior to the commencement of any construction.

539.04.01.02 Working Drawings/Details Requirements

539.04.01.02.01 Information To Be Shown on Working Drawings/Details

a) Plans, Elevations and Details
   i. Location of protection system and/or coffer dam and station limits.
   ii. Plan and elevation of shoring or coffer dam showing the extent of the protection system.
   iii. Details of the shoring or coffer dam system including cross-sections.
   iv. Details of internal bracing.

b) Design Criteria
   i. Pressure diagrams including values of horizontal and vertical loads, dead load and live load surcharge.
   ii. Design assumptions and parameters.
   iii. Anchor bond stresses.
   iv. Pile design.
   v. Anchor system stressing schedule specifying working loads, stressing loads and lock in loads.
   vi. Details of preload where required.
   vii. For protection systems not specified in the Contract, the performance level shall be designated.

c) Materials
   i. Grade of structural steel and grade and species of structural wood.
   ii. Concrete strengths.
   iii. Grout strengths.
   iv. Details of protection from rain and frost action.
   v. Wood lagging and size.

vi. Mill certificates or test reports from an independent organization certified by the Standards Council of Canada certifying that the steel meets the requirements of the grade specified.
vii. Details of patented accessories, including load test data.

d) Installation Procedure
   i. Installation sequence and procedure including but not limited to the installation of piling, lagging, anchor systems and rakers.

e) Monitoring Method
   i. The proposed method of monitoring the performance of the Protection System during installation and use. The method of monitoring shall be consistent with the requirements specified in Section 539.07 of this special provision.

f) Removal of Protection System or Coffer Dam
   i. The details of the procedures associated with the removal of the protection system or coffer dam indicating: method, sequence of work, and removal limits.

539.04.01.02.01 Vibration Monitoring Plan

The Contractor shall submit details of the vibration monitoring plan to the Quality Verification Engineer. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

1. Qualifications of vibration monitoring specialist.
2. Proposed instrumentation.
3. Proposed location of instruments on the existing Beaver Creek bridge.
4. Proposed frequency of readings.
5. Proposed methods for adjusting protection system or coffer dam installation methods if readings show vibrations exceeding tolerable levels.

The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

539.04.01.03 Qualifications

Design Engineer: The Design Engineer shall have demonstrated expertise for the work. The Design Engineer shall have a minimum of five (5) years experience in designing protection systems and coffer dams of similar nature and scope to the required work. One person cannot perform both the Design Engineer and Design Checking Engineer roles for a protection system.

Design Checking Engineer: The Design Checking Engineer shall have demonstrated expertise for the work. The Design Checking Engineer shall have a minimum of five (5) years experience in designing protection systems and coffer dams of similar nature and scope to the required work.

Erector: All supervisory personnel involved in the work performed under this specification shall be experienced in the method of construction of protection systems and coffer dams. Such experience shall have been obtained within the preceding five years on projects of similar nature and scope to the required work.

Quality Verification Engineer: The Quality Verification Engineer shall have a minimum of five (5) years experience in the design of comparable protection systems and coffer dams, or alternatively with demonstrated expertise through providing satisfactory quality verification services for a minimum of two (2) projects in which the work was of similar scope to that in the Contract. The Quality Verification Engineer shall be retained by the Contractor to determine if the work is in general conformance with the Contract Documents and to issue Certificate(s) of Conformance.

539.04.01.04 Certificates of Conformance

539.04.01.04.01 Excavation Depths Less Than or Equal to Three (3) Metres

For protection systems to facilitate excavation depths less than or equal to three (3) metres and provided that surcharge loading due to vehicular traffic, construction equipment and materials or other is beyond a horizontal distance defined by a 1H:2V line projected from the dredge line at the face of the protection system to the roadway surface, the Contractor shall submit, to the Contract Administrator, a Certificate of Conformance sealed and signed by the Quality Verification Engineer following the installation of Protection System to the Dredge Line.

Should traffic be within a horizontal distance defined by a 1H:2V line projected from the dredge line at the face of the protection system to the roadway surface, the certificate of conformance requirements as specified in clause 539.04.01.04.02 shall apply

Upon completion of the operation of the protection system and removal of the protection system, the Contractor shall submit to the Contract Administrator a final Certificate of Conformance sealed and signed by the Quality Verification Engineer. The Certificate of Conformance shall state that the protection system was monitored and subsequently removed, and it performed in general conformance with the stamped working drawings and contract documents.

539.04.01.04.02 Excavation Depths Exceeding Three (3) Metres

For protection systems to facilitate excavation depths that exceed three (3) metres, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer upon completion of each of the following operations, prior to commencement of each subsequent operation:

a) Layout and Extent of Protection System
b) Piling
c) Installation of Protection System including excavation to Dredge Line
d) Removal of Protection System
e) Management of Excess Material (in accordance with OPSS 180 and as specified in the Contract).

The Certificates of Conformance shall state that the materials and work have been supplied and installed in general conformance with the working drawings.

Upon completion of the operation of the protection system and removal of the system, the Contractor shall submit to the Contract Administrator a final Certificate of Conformance sealed and signed by the Quality Verification Engineer. The Certificate shall state that the protection system was monitored and removed, and it performed in general conformance with the stamped working drawings and contract documents.

539.04.01.04.03 Coffer Dams

For coffer dams, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer after each coffer dam has been completed. The Certificates of Conformance shall state that the coffer dams have been installed in general conformance with the working drawings and the Contract Drawings.

539.04.01.05 Amendments to Protection Systems or Coffer Dams

Work shall not proceed on amendments to protection systems or coffer dams until the Contractor has received sealed and signed approval to proceed from the original Design Engineer and Design Checking Engineer and has submitted a copy of the approval to the Contract Administrator.

Amendments to the protection systems or coffer dams shall be submitted to the Contract Administrator on revised Working Drawings/Details bearing the seal and signature of the original Design Engineer and Design Checking Engineer.

539.04.01.06 Preconstruction Survey

Prior to commencing the work, the Contractor shall submit to the Contract Administrator a condition survey of property and structures that may be affected by the work. The survey shall include, but not be limited to, the locations and conditions of adjacent properties, buildings, underground structures, utility services and structures such as walls abutting the site within a horizontal distance of $2H_w$ from the face of the protection system, where H_w is the height of the wall from the ground surface to the dredge line

539.04.02 Design

539.04.02.01 General

The protection system shall be designed for the performance level specified in the Contract Documents.

Protection systems that are not specified in the Contract Documents shall be assigned an appropriate performance level for design by the Design Engineer. The Contract Administrator shall review the performance level selected at the time of submission of the specified working drawings.

The Contractor shall be responsible for the complete detailed design of the protection system needed to fulfill the requirements specified in the contract drawings.

The geotechnical/foundation portion of the design shall be based on a method published in AASHTO Guide Design Specification for Bridge Temporary Works and in general conformance with the CAN/CSA-S6 Canadian Highway Bridge Design Code (CHBDC). Design methods not meeting this design specification may be used on a particular contract only if prequalified by the Owner.

A protection system shall be designed to provide protection for excavations as required by the Occupational Health and Safety Act, at the locations specified in the Contract, and at any other location where the stability, safety or function of an existing structure and/or utility may be impaired by construction work.

The temporary slope geometry used to determine requirements of the protection system shall be in accordance with the Occupational Health and Safety Act.

Performance levels for protection systems are as follows:

Performance Level	Maximum Angular Distortion	Maximum Horizontal Displacement
1a	1:1000	5 mm
1b	1:1000	10 mm
2	1:200	25 mm
3	1:100	50 mm

Where:

Angular Distortion = $\pm \Delta / H$

Δ = Horizontal displacement (mm) at height H

H = Height (mm) above dredge line to point of measurement or height above the nearest system restraining support.

When performance level 1a is specified the bracing system shall be preloaded.

Where the bracing systems are preloaded, the effects of the preload shall not cause damage to adjacent facilities.

Protection systems with a face within a horizontal distance of 1/3 H of any part of a structure foundation shall be designed for performance level 1a.

539.04.02.02 Design Assumptions

The design assumptions shall accurately represent the subsurface conditions prevalent at the site, and shall be specific to the type of protection system used. The design shall address the subsurface conditions at the project site reported in the Foundation Investigation Report described in the Contract Documents.

539.04.02.03 Vertical and Horizontal Loadings

Vertical and horizontal design loadings used shall represent existing conditions and accepted design practice. Future loadings that are known and may affect the protection system during its useful life shall be considered.

539.05 MATERIALS

539.05.01 Wood

Wood shall be according to OPSS 1601, shall be of the size, grade and species shown on the working drawings and shall be in sound condition, free from defects which will impair its strength. Wood lagging does not have to be grade-stamped.

539.05.02 Structural Steel

539.5.02.01 Mill Certificates

The Contractor shall submit to the Contract Administrator at the time of delivery one copy of the mill certificate, indicating that the steel meets the requirements for the appropriate standards for H-piles, tube piles, casings and sheet piles.

Where mill test certificates originate from a mill outside Canada or the United States of America the Contractor shall have the information on the mill certificate verified by testing by a Canadian laboratory. The laboratory shall be accredited by a Canadian National Accreditation Body to comply with the requirements of ISO/IEC Guide 25 for the specific tests or type of tests required by the material standard specified on the mill test certificate. The mill test certificates shall be stamped with the name of the Canadian testing laboratory and appropriate wording stating that the material conforms to the specified material requirements. The stamp shall include the appropriate material specification number, the date and the signature of an authorized officer of the Canadian testing laboratory.

539.05.03 Proprietary Shoring and Patented Accessories

Where proprietary shoring or patented accessories are to be used, the Contractor shall follow the manufacturers' recommendations for load carrying capacity. The recommended load carrying capacities shall be supported by test results from an accredited testing laboratory approved by the Owner.

539.05.04 Concrete

Concrete shall be according to OPSS 1350.

539.05.05 Other Materials

The Design Engineer may consider other suitable materials when sufficient information is available to quantify the allowable design loads or when the manufacturer's recommendations as to load carrying capacities are supported by test results from an independent organization accredited by the Standards Council of Canada.

Earthen materials shall not be used for the coffer dams.

539.07 CONSTRUCTION

539.07.01 General

The Contractor shall be responsible for the design, materials, construction, maintenance, monitoring, and removal of a temporary protection system.

Protection systems shall be built according to the specifications and the stamped working drawings.

Concrete construction shall be according to OPSS 904.

Structural steel shall be according to OPSS 906.

Piling shall be according to OPSS 903.

Prestressed anchors shall be supplied, installed and stressed according to the Contract Documents.

The protection system shall be protected from the detrimental effects of rain and frost action.

Material used in the protection system shall remain the property of the Contractor unless otherwise specified.

Loss of soil from behind the shoring shall be prevented during and following the installation of the lagging.

The soils at the site are glacially and glaciofluvially derived and should be expected to contain cobbles and boulders. Appropriate equipment and procedures will be required to penetrate these obstructions during installation of protection systems and/or coffer dams.

The Contractor shall carry out dewatering as required to facilitate the installation of the protection system. Concrete shall be placed in the dry unless otherwise specified in the Contract. Where cofferdams are used they shall be sealed sufficiently to permit concrete to be placed in the dry. When concrete cannot be placed in the dry, tremie techniques shall be employed according to OPSS 904.

539.07.02 Removal of Protection Systems

Protection systems shall be removed from the right-of-way unless otherwise specified in the Contract that the protection system may be left in place.

Where piles or sheetpiles are left in place the top shall be removed to at least 1.2 m below the finished grade or ground level or at least 200 mm below the streambed.

The method and sequence of removal shall be such that there will be no damage to new work, existing work and the facility being protected.

Unless otherwise specified, the area remaining disturbed after removal of the protection system shall be restored to as close to its original condition as possible.

539.07.03 Quality Control

539.07.03.01 General

The Contractor shall complete a preconstruction condition survey and monitor the protection system installation as specified herein, or as shown on the Working Drawings.

539.07.03.02 Inspection of Welds

The Contractor shall be responsible for visual inspection of all welds. Any required testing of welds shall be as specified by the Design Engineer of the protection system.

539.07.03.03 Monitoring

539.07.03.03.01 General

Monitoring shall be conducted by a Registered Ontario Land Surveyor or an Engineer according to the program submitted with the construction drawings/details.

The minimum requirements for monitoring shall include the survey measurements of scaled targets attached to the shoring wall at the elevations specified. The scaled targets shall be placed at a maximum spacing of 6 metres with targets placed at the extreme ends and the targets distributed between the outer limits. The survey targets shall be monitored for horizontal displacement from the vertical at the frequency specified.

All test results, observations and records, including the construction survey taken during construction and operation of the protection system shall be available on the site for review by the Contract Administrator.

If movement of the protection system is more rapid than is expected, or if movement approaches the allowable limit, the Contract Administrator shall be notified immediately and suitable measures shall be taken to ensure stability of the protection system and to ensure movement does not exceed the performance level specified.

539.07.03.03.02 Excavation Depths Less Than or Equal to Three (3) Metres

The protection systems shall be monitored during construction. Readings shall be taken during installation of the protection system at the top of the protection system at each construction stage during the installation of the protection system. After installation the above readings shall be taken bi-weekly.

539.07.03.03.03 Excavation Depths Exceeding Three (3) Metres

The protection systems shall be monitored during construction. Readings shall be taken during installation of the protection system at the top, at each restraint point, at the dredge line and halfway between the restraint points at each construction stage during the installation of the protection system. After installation the above readings shall be taken weekly.

539.07.03.03.04 Vibration Monitoring

The Contractor shall take readings on the existing Beaver Creek bridge structure during installation of each pile and/or sheetpile. The measured vibrations shall not exceed 100 mm/s (peak particle velocity).

The results shall be submitted to the Contract Administrator after each pile/sheetpile has been installed, prior to continuing with the subsequent piles/sheetpiles. As a minimum, the pile number, location, set criteria (if applicable) and driving/installation log must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with subsequent piles/sheetpiles with readings taken during installation of each pile/sheetpile. The results of subsequent piles/sheetpiles should be submitted to the Contract Administrator after each pile/sheetpile has been installed.

If the readings are not within the limits stated above, the Contractor must alter the installation procedures until the vibrations are within acceptable levels.

539.10 BASIS OF PAYMENT

539.10.01 Protection System – Item

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and material to do the work.

For protection systems not specified in the Contract Documents, the cost shall be included in the protection system tender item, if available, and shall be full compensation for all labour, equipment and material required to carry out the work, including subsequent removal of the protection system and any necessary restoration work.

If the protection system tender item is not included in the Contract Documents, the cost shall be included in the item or items directly associated with the protection system, and shall be full compensation for all labour, equipment and material required to carry out the work, including subsequent removal of the protection system and any necessary restoration work.

539.10.02 Coffer Dams – Item

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and material to do the work.

For coffer dams not specified in the Contract Documents, the cost shall be included in the coffer dam tender item, if available, and shall be full compensation for all labour, equipment and material required to carry out the work, including subsequent removal of the protection system and any necessary restoration work.

If the coffer dam tender item is not included in the Contract Documents, the cost shall be included in the item or items directly associated with the coffer dam, and shall be full compensation for all labour, equipment and material required to carry out the work, including subsequent removal of the coffer dam and any necessary restoration work.

WARRANT: All contracts.

SUPPLY EQUIPMENT FOR DRIVING PILES - Item No. 80
H-PILES-HP 310x110 – Item No. 81
ROCK POINTS – Item No. 82

Special Provision No. 903S01M June 2009

Piling

OPSS 903, December 1983, Construction Specification for Piling is deleted and replaced with the following:

903.01 SCOPE

This specification covers the requirements for the supply and installation of deep foundation units comprised of wood, steel, concrete or a combination of these materials.

The soils at the site are glacially or glaciofluvially derived and should be expected to contain cobbles and boulders. Appropriate equipment and procedures will be required to penetrate obstructions (cobbles and boulders) that are encountered during foundation construction for the new Beaver Creek Bridge.

Also, this provision describes requirements for vibration monitoring during pile driving.

903.02 REFERENCES

This specification refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction:

OPSS 904 Concrete Structures
OPSS 905 Steel Reinforcement
OPSS 909 Prestressed Concrete - Precast
OPSS 911 Coating Structural Steel Construction

Ontario Provincial Standard Specifications, Material:

OPSS 1302 Water
OPSS 1350 Concrete -Materials and Production
OPSS 1440 Steel Reinforcement for Concrete

Canadian Standards Association Standards:

CAN/CSA 3-G40.20/G40.21-M92 - General Requirements for Rolled or Welded Structural Quality Steel/Structural Quality Sheets
CAN3-056-M79 - Round Wood Piles
CSA 080 Series-M97 - Wood Preservation
W47.1-92 - Certification of Companies for Fusion Welding of Steel Structures
W48.1 - M1991 - Carbon Steel Covered Electrodes for Shielded Metal Arc Welding
W59 - M1989 - Welded Steel Construction (Metal Arc Welding)

American Society for Testing and Materials Standards:

ASTM A 252-93 Welded and Seamless Steel Pipe Piles
ASTM A 328/ A 328M-93A Steel Sheet Piling

American Petroleum Institute:

API 13A-86 Oil Well Drilling Fluid Materials
API 13B Standard Procedures for Field Testing Drilling Fluids

903.03 DEFINITIONS

For the purposes of this specification, the following definitions apply:

Anvil: means the component of a diesel hammer that acts as an impact block for the ram

Bedrock: means a natural solid bed of the hard, stable, cemented part of the earth's crust, igneous, metamorphic or sedimentary in origin which may or may not be weathered. The actual surface of the bedrock, weathered or unweathered, exists immediately below the overburden.

Casing: means open ended enclosing cylindrical steel tubing or pipe permanently installed in the ground with caisson piles that is structurally required and can be used to render a stable excavation hole.

Caisson Pile: means a cast in place deep foundation unit with or without an enclosing liner formed by placing concrete in a bored or excavated hole.

Cap Block: means a material placed on top of the helmet to cushion the blow of the hammer and to attenuate the peak impact energy without causing excessive loss of the impact energy.

Deep Foundation Unit: means a structural member, driven or otherwise installed in the ground to transfer the loads from a structure to soil or rock and derives supporting resistance from the surrounding soil or rock or from the soil or rock strata below its tip or a combination of both.

Displacement Caisson Pile: means a pile formed in the ground by driving a casing or liner by means of a concrete plug or an expendable metal plate and replacing the displaced soil with plain or reinforced concrete.

Driving Shoe: means a reinforcement attached to the bottom of the pile and designed to protect the pile during driving or to penetrate into a hard stratum.

Driving to a Set: means driving the pile to a penetration that satisfies pile driving criteria correlated to a required pile resistance.

Follower: means a removable extension which transmits the hammer blows to the head of the pile.

Helmet: means a formed steel cap that fits over the top of a pile head to retain in position a resilient cap block.

Jetting: means the use of a jet of water at high pressure directed into the ground below the pile tip to assist its penetration

Liner: means open ended enclosing steel tubing or pipe temporarily installed in the ground to facilitate the construction of caisson piles

Pile: means a relatively slender structural element which is installed, wholly or partly in the ground by driving, drilling, auguring, jetting or other means.

Pile Cap: means a footing or some other structural component used to transfer the load to the piles as well as maintaining them in position.

Pile Cushion: means a pad of resilient material placed between the helmet and the top of a reinforced concrete or timber pile to minimize damage to the head during driving.

Pile Group: means the piles supporting a pile cap.

Pumped Concrete: means a method of transporting concrete through hose or pipe by means of positive and continuous pressure.

Quality Verification Engineer (QVE): means an Engineer who has a minimum of five (5) years experience in the field of installation of piling or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and to issue Certificate(s)of Conformance.

Retapping: means verifying that the specified resistance previously attained has been sustained by imparting appropriate hammer energy to the pile and monitoring pile penetration.

Rock Points: means a specially designed steel tip, fitted to piles to enable them to be driven into hard, sound sloped bedrock.

Sheet Pile: means a pile that is designed to interlock with adjacent piles and form a continuous wall for the purpose of resisting mainly lateral forces and to reduce seepage.

Slurry: means a drilling fluid, consisting of water mixed with one or more of various solids or polymers, used to maintain the stability of the side walls and bottom of an excavation.

Stamped: means drawings or details that have been reviewed and stamped "Conforms With Contract Documents". The stamp shall include the date and signature of the Quality Verification Engineer.

Tremie: means a hopper with a vertical pipe leading out of the bottom of it, used for placing concrete under water. The foot of the pipe is always submerged in concrete except during commencement of concreting and the upper level of the concrete is always above water level.

903.04 SUBMISSION AND DESIGN REQUIREMENTS

All submissions shall bear the seal and signature of an Engineer experienced in this field. This Engineer, under this section, will not be permitted to carry out the work of the Quality Verification Engineer.

The Contractor shall submit to the Quality Verification Engineer for review and stamping, the equipment and installation procedure and the procedure for monitoring installation.

903.04.01 Site Survey

Prior to commencing the work, the Contractor shall submit to the Contract Administrator, a condition survey of property and structures that may be affected by the work. The survey shall include, but not be limited to, the locations and conditions of adjacent properties, buildings, underground structures, utility services and structures such as walls abutting the site.

903.04.02 Materials

903.04.02.01 Mill Certificates

The Contractor shall submit to the Contract Administrator at the time of delivery one copy of the mill certificate, indicating that the steel meets the requirements for the appropriate standards for H-piles, tube piles, casings and sheet piles.

Where mill test certificates originate from a mill outside Canada or the United States of America the Contractor shall have the information on the mill certificate verified by testing by a Canadian laboratory. The laboratory shall be accredited by a Canadian National Accreditation Body to comply with the requirements of ISO/IEC Guide 25 for the specific tests or type of tests required by the material standard specified on the mill test certificate. The mill test certificates shall be stamped with the name of the Canadian testing laboratory and appropriate wording stating that the material conforms to the specified material requirements. The stamp shall include the appropriate material specification number, the date and the signature of an authorized officer of the Canadian testing laboratory.

903.04.02.02 Concrete

Concrete and concrete work shall conform to OPSS 1350 and OPSS 904. The Contractor shall submit a suitable, site specific concrete mix design that meets the requirements of the hardened concrete specified. The Contractor is responsible for providing plastic concrete with suitable characteristics for installation. The concrete shall be flowable, non-segregating concrete that does not exhibit rapid slump loss. The concrete mix design shall be submitted as specified in the Contract Documents.

903.04.02.03 Slurry

The Contractor shall submit, for information purposes only, one (1) week prior to construction:

1. The type, source, physical and chemical properties of the bentonite or polymer.
2. Slurry mix proportions and procedure.
3. Quality Control Plan to control properties of slurry mix.
4. Method of disposal.

903.04.03 Installation

903.04.03.01 Driven Piles

The Contractor shall submit, for information purposes only, one (1) week prior to construction:

1. Type of equipment and hammer details including Contractors stated potential energy (rated energy) of the hammer, operating efficiency, weight of ram, anvil and helmet.
2. Procedure including sequence for pile installation.
3. Procedure for monitoring installation.

903.04.03.02 Caisson Piles

The Contractor shall submit, for information purposes only, one (1) week prior to construction:

1. Shop drawings that describe and illustrate equipment, materials.
2. Procedure for caisson excavation and construction.
3. Procedure for monitoring installation and caisson inspection.

903.04.03.03 Displacement Caisson Piles

The Contractor shall submit, for information purposes only, one (1) week prior to construction:

1. Equipment to be used for installation.
2. Procedure for installation
3. Procedure for monitoring installation.

903.04.03.04 Certificate of Conformance

Upon completion of the deep foundation work, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer. The certificate shall state that the work has been carried out in general conformance with the contract documents, specifications and stamped working drawings.

903.04.04 Vibration Monitoring

The Contractor shall submit details of the vibration monitoring plan to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

1. Qualifications of vibration monitoring specialist.
2. Proposed instrumentation.
3. Proposed location of instruments on the existing Beaver Creek bridge.
4. Proposed frequency of readings.
5. Proposed methods for adjusting pile installation methods if readings show vibrations exceeding tolerable levels.

The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

903.05 MATERIAL

903.05.01 Wood Piles

Wood piles shall be according to CSA CAN3-056 and shall be clean and peeled. Treated piles shall be pressure treated with creosote according to CSA 080.

Wood piles shall not be spliced.

903.05.02 Steel Piles

903.05.02.01 Steel H Piles

Steel H piles shall be according to CSA G40.20/G40.21 and shall be 350 W grade.

903.05.02.02 Steel Tube Piles

Steel tube piles shall be according to ASTM A252 minimum Grade 2.

903.05.02.03 Steel Sheet Piles

Steel sheet piles shall be according to ASTM A328. Steel sheet piles shall not be spliced.

903.05.02.04 Straightness Tolerance for Steel Piles

All steel piles shall conform to a straightness tolerance of 1.5 mm maximum per metre of length.

903.05.03 Driving Shoes and Rock Points

Rock points and driving shoes shall be as specified. Driving shoes shall transfer the driving stresses to the pile over the full cross-sectional area of the pile.

Where the contract shows details of "Splice and Driving Shoe Details for Steel 'H' Piles, the Contractor may substitute the Titus "H" Bearing Pile Point, Standard model, in place of the driving shoe details shown.

Where the contract shows details of "Oslo Points for HP310 H-Piles" the Contractor may substitute the Titus "H" Bearing Pile Point, Rock Injector model in place of the pile point details shown.

Welding of Titus Points shall conform to the manufacturer's specifications.

Where the Contractor elects to use any of the above substitutions, the cost shall be deemed to be included in the contract price for the appropriate item.

903.05.04 Casing for Caissons

Casings shall be according to ASTM A252 Grade 2. If welded they shall be welded by the electric arc method according to CSA W59.

The wall thickness specified is the minimum that shall be supplied. The wall thickness shall be increased as required to ensure the casing is not damaged during handling and installation.

903.05.05 Steel Reinforcement

Steel reinforcement shall be according to OPSS 1440.

903.05.06 Concrete

903.05.06.01 General

Concrete shall be according to OPSS 1350.

903.05.06.02 Tube Piles

Concrete shall have a slump of 150 to 180 mm.

903.05.06.03 Caisson Piles

Concrete shall have a slump of 150 to 180 mm. When approved by the Contract Administrator in writing, admixtures may be used. Where the liner is to be withdrawn, sufficient retarder shall be added to prevent arching of concrete during liner withdrawal, and to prevent setting of concrete until after the liner is withdrawn.

903.05.07 Slurry

903.05.07.01 Solids

Bentonite and polymers shall be according to API 13A.

903.05.07.02 Slurry Composition

Slurry shall be according to API 13B

903.05.08 Helmets and Striker Plates

The head of piles shall be protected by a striker plate or a helmet. Helmets shall have adequate and suitable cushioning material. Helmets and striker plates shall distribute the blow of the hammer evenly throughout the cross-section of the pile head.

903.06 EQUIPMENT

The hammers shall be capable of driving the piles and liners/casings to the prescribed depth or to the specified resistance without damage to portions that are not cut off.

903.07 CONSTRUCTION

903.07.01 Subsurface Conditions

A Foundation Investigation Report that describes the subsurface conditions for the project is available, as specified elsewhere in the Contract. The Ministry warrants that the information provided in the Foundation Investigation Report can be relied upon with the following limitations and exceptions:

> Any interpretation of data or opinions expressed in the report are not warranted.

Regarding the data presented in the report, although the raw measured data presented is warranted, the Contractor must satisfy itself as to the sufficiency of the information presented for the intended construction purpose and obtain any updating or additional information as required to facilitate the deep foundation works.

903.07.02 Transportation, Handling, Storage

Piles, casings and reinforcing steel cages shall be transported, stored and handled in such a manner that damage and distortion is prevented and that the strength and integrity are maintained.

903.07.03 Driven Piles

903.07.03.01 Pile Driving Requirements and Restrictions

Piles shall be installed at the locations indicated and to the set or depth specified without being damaged.

Damage to adjacent structures, utilities and fresh concrete shall be prevented during pile installation. Piles shall not be driven within a radius of 7.5 m of concrete which has been in place for less than 72 hours. Piles shall not be driven within a radius of 15 m of concrete that has been in place for less than 72 hours without the approval of the Contract Administrator.

The tops of all piles shall be either square to the longitudinal axis of the pile or horizontal as indicated on the Contract Drawings.

Piles shall not be forced into their proper position by the use of excessive manipulation. Pile damage due to excessive driving shall be avoided.

903.07.03.02 Splicing

903.07.03.02.01 General

Splices within 6 m of the pile cut-off shall be certified by the Quality Verification Engineer as being equal to the full strength of the pile. Any damaged material shall be cut-off prior to splicing. The certificate shall be sealed and signed by the Quality Verification Engineer and shall be submitted to the Contract Administrator.

903.07.03.02.02 H Piles, Tube Piles and Sheet Piles

Welding shall be according to CSA W59 and shall be done by a qualified welder employed by a firm certified according to CSA W47.1, Division 1 or Division 2.1.

Steel H piles and steel tube piles may be spliced providing the pieces being spliced are not less than 3 m long. Splices in marine structures shall be located below the low water level unless otherwise encased in concrete.

Sheet piles shall not be spliced without approval by the Contract Administrator.

903.07.03.02.03 Precast Piles

Precast piles shall only be spliced when specified and the splices shall only be made with approved mechanical splicing devices.

903.07.03.03 Concrete in Steel Tube Piles

Concrete in steel tube piles shall be placed according to the OPSS 904 requirements.

903.07.03.04 Cutting Off Piles

903.07.03.04.01 General

Driven piles shall be cut to the elevation as specified in the contract.

The length of pile supplied shall be sufficient to ensure there is no damaged material below the cut off. Damaged material at the pile head shall be cut off.

903.07.03.04.02 Wood Piles

Where wood piles are broomed, splintered or otherwise damaged below the cutoff elevation, the pile shall be considered defective and shall be replaced.

903.07.03.05 Protective Coating for Steel H and Steel Tube Piles

Exposed steel H and steel tube piles shall have a protective coating applied from an elevation 600 mm below the low water level or finished ground surface up to the top of the exposed steel.

The steel surfaces shall be cleaned according to SSPC-SP10 prior to application of a coal tar epoxy system which shall be according to OPSS 911.

903.07.03.06 Reinforcing Steel

Reinforcing steel shall be installed according to OPSS 905.

The reinforcing steel cage shall be fabricated in one piece.

Welding of reinforcing steel and use of splices shall not be done unless specified in the contract.

903.07.04 Caisson Piles

903.07.04.01 Installation - General

Caissons shall be constructed as specified in the contract.

The final bearing elevation shall be as specified in the contract or shall be an elevation determined by the Contract Administrator. When permanent casings are not specified the caisson shall be constructed in a drilled hole with or without the use of a temporary liner or slurry as determined by the Contractor.

903.07.04.02 Excavation

Sidewall stability shall be maintained throughout the excavation and concrete placement operation. Soil cave-in into the excavation hole shall be prevented.

Excavation methods shall be such that the sides and bottoms of the hole are straight and free of loose material.

Except when founded on sloping rock, the caisson bottom shall be level. On sloping rock, the caisson bottom may be stepped with each step not greater than ¼ the diameter of the bearing area.

903.07.04.03 Unwatering

Where unwatering is required, the Contractor shall effect a dewatering scheme in such a manner as to prevent any disturbance to the base founding material, or prevent subsidence or ground loss that may adversely affect the work of adjacent structures.

903.07.04.04 Backfilling Liners Left in Place

The annular space between a liner permanently left in place and shaft excavation shall be filled with concrete or fluid grout.

903.07.04.06 Concrete

903.07.04.06.01 General

Concrete shall be placed in the caisson according to OPSS 904. Concrete shall be placed immediately following acceptance of the caisson hole by the QVE.

The reinforcement shall not be displaced or distorted during the construction of the caisson.
Arching of concrete during casing withdrawal shall be prevented.

The QVE shall provide inspection throughout the concreting operation.

903.07.04.06.02 Concrete Placed in the Dry

The concrete may be placed free fall provided the fall is vertically down the centre of the opening and transverse ties, spacers or other do not impede the free fall. In the event of interference with the concrete free fall, an elephant trunk or other means shall be used to prevent concrete segregation.

Concrete shall be placed in a continuous operation from the bottom to the top of the caisson, or where columns are cast integral with the caisson, to the elevation of the bottom of the column reinforcing cage. The concrete shall be vibrated for the last 1.5 m of the pour.

903.07.04.06.03 Concrete Placed Under Water or Under Slurry

Tremie or pumped concrete shall be carried out in one continuous operation. The Contractor shall carry out the tremie or pumping operation to ensure a continuous flow of concrete that prevents the inflow of water or slurry.

903.07.04.07 Reinforcing Steel

The reinforcing steel cage shall be checked to ensure conformance to the approved shop drawings prior to installation and during concrete placement.

903.07.05 Displacement Caisson Piles

903.07.05.01 General

Work shall be carried out in accordance with displacement caisson pile suppliers installation procedures. A permanent liner shall be used when specified.

The pile shall not be extended below the specified pile tip elevation without approval in writing from the Contract Administrator.

903.07.06 Tolerances

903.07.06.01 Driven Piles

1. Cut off ± 25 mm.
2. Deviation from vertical not more than 1 in 50, except in the case of a pile cap or footing supporting only a single row of piles the deviation shall not be more than 1 in 75 in the direction of the span.
3. The deviation from the specified inclination for battered piles shall not exceed 1 in 25.
4. The centre of the pile at the junction with the pile cap shall be within 150 mm of that specified (measured horizontally) except in the case of a pile cap or footing supported on a single row of piles the deviation shall not be more than 75 mm (measured horizontally) in the direction of the span.

903.07.06.02 Caissons

1. Cut off elevation ± 25 mm.
2. Horizontal location at cut-off not more than 5% of shaft diameter nor 75 mm.
3. Vertical alignment not more than 2% of the caisson length from vertical for vertical caissons, nor 2% of the caisson length from the specified inclination for battered caissons.

903.08 QUALITY CONTROL

903.08.01 Monitoring Driven Piles

903.08.01.01 General

The driving of piles shall be carefully monitored and controlled and pile driving records produced for each pile. All driving records shall be certified by the Quality Verification Engineer and submitted to the Contract Administrator.

903.08.01.02 Driving to a Set

The founding elevation shall be established by driving to a set determined in accordance with the dynamic formula specified or by the application of the wave equation analysis procedure that verifies the pile resistance. This set shall be established on the first pile of every ten piles driven in a pile group.

The other piles shall be controlled by the pile penetration rate in blows per mm that correlates to the set.

When new conditions such as change in hammer size, change in pile size or change in soil material occur, new sets shall be determined.

903.08.01.03 Driving to Bedrock

When driving piles to bedrock, the Contractor shall adequately seat the pile on bedrock without damaging the pile.

Where rock points are used the rock points shall penetrate into the rock. Piles driven using rock points shall be driven to ensure adequate seating on the bedrock without damaging the pile.

903.08.01.04 Hammer Performance

When requested by the Contract Administrator, the Contractor shall verify the hammer performance using the Pile Driving Analyzer or other approved equivalent. The Contractor shall provide all instrumentation, related access and assistance for the testing and monitoring as directed by the Contract Administrator.

Hammer performance shall be verified to ensure that the actual potential energy is not less than 90% of the stated potential energy.

903.08.01.05 Retapping Tests on Piles

In each pile group, 10% of the piles (actual number of piles to be rounded off to higher number) but no fewer than two piles shall be retapped no sooner than 24 hours after installation of the individual pile to confirm the bearing resistance has been sustained.

Retapping of piles driven to bedrock is not required.

903.08.01.06 Retapping/Redriving Piles

Where the retapping tests indicate the bearing resistance has not been sustained, all piles in the group shall be retapped.

Where the retapping reveals that the bearing resistance of the piles has not been achieved, the piles shall be redriven to the specified resistance. Where piles have risen, the piles shall be redriven to the original depth.

903.08.01.07 Vibration Monitoring

The installation of the deep foundations shall commence with piles located furthest from the existing Beaver Creek bridge structure. The Contractor shall take readings during driving of each pile. The measured vibrations shall not exceed 100 mm/s (peak particle velocity).

The results shall be submitted to the Contract Administrator after each pile has been driven, prior to continuing with the subsequent piles. As a minimum, the pile number, location, set criteria (if applicable) and driving/installation log must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with subsequent piles with readings taken during installation of each pile. The results of subsequent piles should be submitted to the Contract Administrator after each pile has been installed.

If the readings are not within the limits stated above, the Contractor must alter the installation procedures until the vibrations are within acceptable levels.

903.08.02 Inspection of Caisson Holes

The caisson holes shall be inspected and approved by the QVE.

903.09 MEASUREMENT FOR PAYMENT

903.09.01 H Piles, Tube Piles, Wood Piles and Precast Concrete Piles

Measurement is in metres of the piling left in place after cut-off.

903.09.02 Sheet Piles

Measurement is in square metres based on the driving lines specified and the length of piling left in place after cut-off.

903.09.03 Driving Shoes and Rock Points

Measurement is for each driving shoe and rock point specified and used.

903.09.04 Caissons and Displacement Caisson Piles

Measurement is in metres of the depth along the centre line between the approved bearing surface at the bottom and the specified elevation at the top.

903.09.05 Retapping Piles

Measurement is lump sum for retapping the piles above and beyond the minimum 10% but no fewer than two piles requirement for the pile group.

For measurement purposes a count will made of the number of piles retapped above and beyond the minimum 10% but no fewer than two piles requirement and the number of piles in the pile group and a ratio will be determined.

Where retapping is not required above and beyond the minimum, no measurement for payment will be made for this item.

903.10 BASIS FOR PAYMENT

903.10.01 Supply Equipment for Installing Driven Piles - Item
Supply Equipment for Installing Caisson Piles - Item
Supply Equipment for Installing Displacement Caisson Piles - Item

Payment at the contract price for the above items shall be full compensation for all labour, testing, equipment and material required to do the work.

It will be assumed, for payment purposes, that 50% of the work under this item has been completed when the satisfactory performance of the equipment has been demonstrated to the Contractor Administrator by the installation of one (1) pile. The remaining 50% will be paid on the satisfactory completion of the installation.

When the hammer performance is requested to be verified, all costs associated with this work will be included in the contract price when the energy delivered is less than 90% of the stated potential energy (rated energy) specified in the submission.

When the energy is greater than 90% of the stated potential energy (rated energy) stated in the required submission, the cost will be paid as extra work.

903.10.02 H-Piles – Item
Tube Piles – Item
Precast Concrete Piles - Item
Wood Piles - Item
Displacement Caisson Pile - Item
Caisson Piles - Item
Driving Shoes - Item
Rock Points - Item
Sheet Piles - Item

Payment at the contract price for the above items shall be full compensation for all labour, equipment and material to do the work

Payment for redriving piles shall be at the contract price for the applicable item(s) above.

903.10.03 Retapping Piles – Item

Payment for retapping the minimum specified number of piles is included in the Pile Item. Where additional retapping is required, payment will be made based on the ratio of the number of piles retapped in a pile group above the minimum requirement, to the total number of piles in that pile group, times the tender price for retapping all piles for that pile group**.**

WARRANT: Always with these tender items.

<u>CONTROL OF OVERBURDEN SOILS AND GROUNDWATER DURING CAISSON INSTALLATION - Item No.</u>

Special Provision

Caissons for support of the new bridge abutments will be advanced through water-bearing, predominantly cohesionless soils. Appropriate construction procedures and equipment will be required to minimize ground loss during drilling and caisson construction.

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

CAISSON SOCKETS IN BEDROCK - Item No.

Special Provision

The bedrock at this site is medium strong to strong. Appropriate construction equipment and procedures will be required for construction of caisson foundation sockets within the bedrock.

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