November 27, 2014

FOUNDATION INVESTIGATION AND DESIGN REPORT

BEAR CREEK CULVERT AT STA 14+960, SITE 43-231/C
HIGHWAY 17 REHABILITATION BETWEEN WARREN AND VERNER
FROM HIGHWAY 539 EASTERLY TO 0.2 KM EAST
OF WEST JUNCTION OF HIGHWAY 64
TOWNSHIP OF KIRKPATRICK, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 300-8-00, WP300-98-01

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PART A

FOUNDATION INVESTIGATION REPORT

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (MH), on behalf of the Ministry of Transportation, Ontario (MTO), to provide foundation engineering services for the replacement of the Highway 17 Bear Creek culvert (Site 43-231/C) at STA 14+960 in the Township of Kirkpatrick, Municipality of West Nipissing, Ontario. The Key Plan showing the general location of this section of Highway 17 and the location of the investigated area are shown on Drawing 1. The purpose of this investigation is to establish the subsurface conditions at the location of the culvert by borehole drilling, in situ testing and laboratory testing on selected soil samples.

2.0 SITE DESCRIPTION

The Bear Creek culvert is located on Highway 17 east of Warren, approximately 7 km east of the junction of Highway 539. In general, the topography in the area of the overall project limits consists of flat terrain primarily utilized as farmland, with moderate tree cover. The existing highway grade is at about Elevation 208 m with Bear Creek crossing under the embankment about 7 m below the existing highway grade. The side slopes of the existing embankment are inclined at about 2 Horizontal to 1 Vertical (2H:1V) and are vegetated with various grasses. The existing structure is a twin cell reinforced concrete culvert constructed in 1948. The culvert is 40 m long and each cell has a span of 5.3 m and an opening of 3.2 m high.

3.0 INVESTIGATION PROCEDURES

The fieldwork for the investigation was carried out on December 10, 2013, January 13 and 20, February 3 and 4 and March 4 to 12, 2014, during which time a total of six boreholes and one Dynamic Cone Penetration Test (DCPT) were advanced at the approximate locations shown on Drawing 1:

- three boreholes for the culvert alignment (Boreholes BE-1, BE-4 and BE-5 with an adjacent DCPT);
- one borehole for the proposed roadway protection (Borehole BE-2); and
- two boreholes for the proposed cofferdam (Boreholes BE-3 and BE-6).

Boreholes BE-1 and BE-2, located on the existing highway embankment, were advanced using a truck-mounted CME 55 drill rig outfitted with 108 mm inside diameter continuous flight hollow-stem augers and NW casing, supplied and operated by Landcore Drilling Inc. (Landcore) of Sudbury, Ontario. Boreholes BE-3, BE-5 and BE-6, were advanced by portable equipment using wash boring methods with NQ casing, supplied and operated by Landcore or George Downing Estate Drilling Ltd. of Grenville-Sur-La-Rouge, Quebec. Borehole BE-4 was advanced using a track mounted CME 55 drill rig operated by Landcore. At Borehole BE-6, an excavator was used to remove about 2.0 m of cobble and boulder sized rock fill material mixed with sand and gravel prior to advancing the borehole with the portable equipment. A DCPT was advanced adjacent to Borehole BE-5 using portable equipment.

Soil samples were obtained at intervals of depths of about 0.75 m and 1.5 m, using a 50 mm outer diameter (O.D.) split-spoon sampler driven by an automatic hammer at Boreholes BE-1, BE-2 and BE-4 and a manual hammer at Boreholes BE-3, BE-5 and BE-6, and performed in accordance with Standard Penetration Test (SPT) procedure (ASTM D1586). The DCPT was driven by a manual hammer. Selected samples of the cohesive soils were obtained using 76 mm O.D. thin-walled 'Shelby' tubes (ASTM D1587, Standard Practice for Thin-Walled Tube Sampling) for relatively undisturbed samples. Field vane shear tests were conducted in cohesive soils for determination of undrained shear strengths (ASTM D2573, Standard Test Method for Field Vane Strength Shear Test) using MTO Standard 'N' size vanes. The groundwater conditions and water levels in the open boreholes were observed during the drilling operations and are described on the Record of Borehole sheets in Appendix A. All boreholes were backfilled with bentonite upon completion of drilling in accordance with Ontario Regulation 903 (as amended).

The fieldwork was supervised throughout by members of our technical staff, who located the boreholes and DCPT, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, organic content, Atterberg limits and grain size distribution) was carried out on selected soil samples. The results of the laboratory testing are presented on the Record of Borehole sheets in Appendix A and are also included in Appendix B.

A sample of the creek water was obtained on March 10, 2014, using appropriate sampling protocols and submitted to a specialist analytical laboratory under chain of custody procedures for testing for a suite of parameters. The results of the analytical testing are summarized in Table B1 in Appendix B, together with the detailed analytical laboratory test results.

The as-drilled borehole locations and ground surface elevations were measured and surveyed by members of our technical staff, referenced to the marked stations and offsets on the highway or the ends of the culvert, as applicable. The MTM NAD 83 northing and easting coordinates, ground surface elevations referenced to Geodetic datum and borehole depth at each borehole are presented on the Record of Borehole sheets in Appendix A and are summarized below.

Borehole	**Borehole Location**		**Ground/Ice Surface Elevation (m)**	**Borehole/DCPT Depth Below Ground/Ice Surface (m)**
	Northing	**Easting**		
BE-1	5 143 672.4	248 753.9	207.6	18.3
BE-2	5 143 666.4	248 728.6	207.4	16.3
BE-3	5 143 691.3	248 722.7	200.9	14.2*
BE-4	5 143 685.2	248 742.9	203.4	14.3
BE-5	5 143 655.6	248 749.4	202.1	20.7/14.6
BE-6	5 143 649.4	248 769.8	202.7	14.7

*includes 1.5 m column of water

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on terrain mapping (Ontario Geological Survey[1]), the site is located on a glaciolacustrine plain in an area of sand and silt deposits with bedrock knobs/outcrops located approximately 100 m to the south of the site.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions, as encountered in the boreholes advanced for this investigation, together with the results of the laboratory tests carried out on selected soil core samples, are given on the Record of Borehole sheets in Appendix A. The results of the in situ tests (i.e., SPT "N"-values and undrained shear strengths from the field vanes) as presented on the Record of Borehole sheets and in Section 4 are uncorrected. Detailed results of the laboratory testing of the soil samples are provided in Appendix B. The stratigraphic boundaries shown on the Record of Borehole sheets and on the stratigraphic profile and cross-section shown on Drawing 1 are inferred from non-continuous sampling, observations of drilling progress and the results of SPTs and in situ testing. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

4.2.1 Ice/Water

Borehole BE-3 was advanced from a wooden platform extending across the creek. At the time of drilling, the ice/water column was 1.5 m deep and the creek water surface was at Elevation 200.9 m.

4.2.2 Fill

Boreholes BE-1 and BE-2 penetrated a layer of asphalt 250 mm and 175 mm thick at Elevation 207.6 m and 207.4 m, respectively. Underlying the asphalt, the boreholes encountered 7.1 m and 7.2 m of fill comprised of sand and gravel and blast rock of cobble and boulder size; ranging from 90 mm to 910 mm in diameter. In Borehole BE-4, a 2.1 m thick layer of blast rock fill was encountered from ground surface at Elevation 203.4 m. In Borehole BE-5, a 4.6 m thick layer of fill was encountered from ground surface at Elevation 202.1 m, consisting of an upper thick layer of 1.2 m of sandy silt with trace organics and a lower layer 3.4 m thick clayey silt to silty clay mixed in places with sand or sand and gravel. In Borehole BE-6, a 2.0 m thick layer of blast rock fill consisting of cobble and boulder sites mixed with sand and gravel was encountered at ground surface at Elevation 202.7 m underlain by a 1.9 m thick layer of clayey silt to silty clay fill.

SPT 'N'-values measured within the fill are as follows:

- Sand and gravel and blast rock: 9 blows to 65 blows per 0.3 m of penetration, indicating a loose to very dense relative density, with two SPT penetrating only up to 0.1 m, indicative of the blast rock fill;

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[1] Southern Ontario Engineering Geology Terrain Study, 1980. Ontario Geological Survey.

- Sandy silt: 3 blows per 0.3 m of penetration indicating a very loose relative density; and
- Clayey silt to silty clay: between 2 blows and 34 blows per 0.3 m of penetration. Two situ field vane tests within the clayey silt to silty clay measured undrained shear strengths of 15 kPa and 30 kPa, with sensitivities of 3 and 5. The SPT N-values together with the in situ vane test results indicate that the deposit has a soft to hard consistency.

4.2.3 Gravelly Sand to Silty Sand

Underlying the fill in Boreholes BE-1, BE-4, to BE-6 a deposit of gravelly sand to silty sand was encountered about Elevation 201.3 m and 197.5 m, and the thickness of the deposit is between 0.7 m and 3.0 m.

SPT ‘N’-values measured within this deposit range between 11 blows and 29 blows per 0.3 m of penetration, indicating a compact relative density.

Grain size distribution analyses were carried out on two samples of this deposit and the results are presented on Figure B1 in Appendix B.

The natural moisture content measured on two samples of this deposit is about 18 per cent.

4.2.4 Sand and Silt to Silt

Underlying the gravelly sand in Borehole BE-1 at about Elevation 198.9 m and underlying the fill in Borehole BE-2 at about Elevation 200.1 m, a 1.5 m and 1.3 m thick deposit of sand and silt to silt was encountered in the respective boreholes.

Two SPT ‘N’-values measured within this deposit are 5 blows and 6 blows per 0.3 m of penetration, indicating a loose relative density.

Grain size distribution analyses were carried out on two samples of this deposit and the results are presented on Figure B2 in Appendix B.

An Atterberg limits test was carried out on a sample of the sand and silt deposit and measured a liquid limit of about 37 per cent, a plastic limit of about 26 per cent, and a plasticity index of about 11 per cent. The results, which are plotted on a plasticity chart on Figures B3, indicate that the tested sample consist of silt of intermediate plasticity. One Atterberg limits test carried out on a select sample of the silt deposit indicates that the material is non-plastic.

The natural moisture content measured on samples of this deposit range from about 27 per cent and 39 per cent.

4.2.5 Clayey Silt to Silty Clay

A deposit of clayey silt to silty clay was encountered in all of the boreholes between Elevation 200.5 m and 195.8 m and was explored for a thickness between 7.7 m and 15.4 m without fully penetrating the deposit. Silt laminations/layers were observed in numerous samples of this deposit.

The SPT ‘N’-values measured within the clayey silt to silty clay deposit range between 0 blows (weight of hammer) and 11 blows per 0.3 m of penetration. In situ field vane testing at various depths within the deposit measured undrained shear strengths ranging from 15 kPa to 72 kPa with sensitivities between 1 and 5. The in situ vane test results indicate that the deposit has a soft to stiff consistently with the majority of the undrained shear strengths generally in the firm range.

The results of the grain size distribution tests completed on three samples of this deposit are shown on Figure B4; one sample of the silt layer in Borehole BE-4 is shown on Figure B2.

Atterberg limits tests were carried out on sixteen samples of the cohesive deposit and measured liquid limits ranging from about 28 per cent to 44 per cent with one result of about 5 per cent, plastic limits ranging from about 19 per cent to 23 per cent, and plasticity indices ranging from about 8 per cent to 28 per cent. The results, which are plotted on a plasticity chart on Figures B5, indicate that the tested samples of the overall deposit consist of clayey silt of low plasticity to silty clay of intermediate plasticity with one result indicating a clay of high plasticity. One Atterberg limits test carried out on a sample of the silt layer indicates that the material is non-plastic.

The natural moisture content measured on selected samples of this deposit generally ranges between about 24 per cent and 48 per cent.

4.2.6 Groundwater Conditions

The depth to the water level in Boreholes BE-1, BE-2, BE-4 to BE-6 upon completion of drilling was measured at depths between 0.9 m and 5.2 m below ground surface, between Elevation 206.7 m and 198.7 m. The creek ice/water surface at Borehole BE-3 was Elevation 200.9 m on March 4, 2014.

Groundwater levels encountered in the boreholes during and shortly after drilling may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized on completion of drilling. Groundwater levels are subject to seasonal fluctuations and to fluctuations due to precipitation events and snowmelt.

5.0 CLOSURE

The drilling program was supervised by Messrs. Trevor Moxam, Mat Riopelle, Gabriel Mathieu and Shane Albert. This report was prepared by Mr. Adam Core, E.I.T. The technical aspects were reviewed by Mr. André Bom, P.Eng., and Mr. Jorge M. A. Costa, P.Eng., Principal and Golder’s Designated MTO Contact for this project, conducted an independent quality control review of the report.

Report Signature Page

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PART B

BEAR CREEK CULVERT AT STA 14+960, SITE 43-231/C

HIGHWAY 17 REHABILITATION BETWEEN WARREN AND VERNER

FROM HIGHWAY 539 EASTERLY TO 0.2 KM EAST

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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides an interpretation of the factual geotechnical data obtained during the investigation and conclusions and recommendations on the foundation aspects of design of the proposed works. The recommendations provided are intended for the guidance of the design engineer. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the design of the project. Those requiring information on aspects of construction must make their own interpretation of the subsurface information provided as such interpretation may affect their proposed construction methods, costs, equipment selection, scheduling and the like.

6.1 General

The existing Bear Creek culvert is a 40 m long, twin cell reinforced concrete structure with each cell at 5.3 m wide and 3.2 m high. The top of the existing embankment is at about Elevation 208 m and the existing side slopes of the 7 m high embankment are inclined at about 2H:1V.

The existing culvert will be rehabilitated, however, in the event that further consideration is given to replacing the structure, the following sections of this report provide recommendations for design of the replacement structure, including comments on roadway protection to facilitate traffic and construction staging.

As part of the Highway 17 rehabilitation to be carried out in the vicinity of the culvert, a grade raise to the existing embankment of about 150 mm may be required. Further, we understand that there will be no change to the inclination of or the addition of fill to the embankment side slopes.

6.2 Culvert Types

In the event that the culvert is replaced instead of rehabilitated, analysis and recommendations for culvert replacement are presented in this report for a concrete box culvert of overall dimensions similar to the existing structure. Due to the presence of the cohesive deposit of low strength (soft to stiff) consistency at shallow depth below ground surface and under the embankment and the thickness of the fill that will be placed over the culvert to match the surface grade of the existing embankment (and potentially an additional 0.15 m grade raise), an open bottom concrete culvert (on strip footings) at this site is not considered feasible.

6.3 Stability

Limit equilibrium slope stability analyses were performed for the proposed embankment geometry, based on the a culvert being replaced, using the commercially available program GeoStudio 2007 (Version 7.23), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted on MTO projects for the design of embankment slopes under static conditions. This FoS is considered adequate for the embankment at this site considering the design requirements and the field data available and is based on deep-seated, global failure surfaces that would affect the operation of the roadway. The stability analyses were performed to check that the target minimum FoS was achieved for the embankment height and geometry at the culvert location.

The analyses assume that, for a full culvert replacement, the organic soils (if present) beneath the culvert and immediate area of excavation under the embankment footprint will be removed prior to construction of the new culvert and re-constructed embankment. The analyses assume that new granular fill (sand and gravel, or OPSS.PROV 1010 Granular 'A' or 'B' Type I or II), is used for embankment reconstruction, and would be keyed into the existing fill as per OPSD 208.010 (Benching of Earth Slopes) and constructed with horizontal to/vertical or flatter 2H:1V side slopes.

For the cohesionless fill and native cohesionless soils, effective stress parameters were employed in the stability analysis assuming drained conditions and the parameters were estimated from empirical correlations using the results of the in situ SPT 'N'-values. The correlations proposed in NAVFAC (1982) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

For the cohesive deposit, total stress parameters were employed in the analysis. The total stress parameters (i.e., undrained shear strength – s_u) for the cohesive soil were assessed based primarily on the results of the in situ field vane tests. Bjerrum's (1973) correction factor as a function of the plasticity index of the soil was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests.

The piezometric condition required in the analyses is based on the groundwater level consistent with the creek water level at Elevation 200.9 m.

The simplified stratigraphy together with the associated strength and unit weight employed for the different native soil types at the culvert location are summarized below.

Soil Type	Unit Weight (kN/m^3)	Undrained Shear Strength (kPa)	Angle of Internal Friction (^{o})
New Granular Fill	21	-	35
Silt/Silty Sand/Gravelly Sand	19	-	30
Clayey Silt to Silty Clay	18	30 to 60 kPa (increasing with depth)	-

The stability analysis performed on the embankment cross-section at the culvert location indicates that the embankment will have a FoS greater than 1.3 for deep-seated, global failure surfaces that would impact the operation of the roadway, as shown on Figure 1.

6.4 Settlement

Assuming a proposed grade raise of about 150 mm and the side slope being reconstructed at the same inclination as the existing embankment and the embankment not widened, the total and differential settlement of the foundation soil below the box culvert will be less than 25 mm. However, if a greater increase to the existing embankment grade or a widening is required along the side slopes, the magnitude of settlement will be different than that estimated for the presently proposed construction and measures to mitigate settlement may be required.

It is recommended that OPSS.PROV 1010 (Aggregates) Granular 'A' or 'B' Type I or II be used for embankment reconstruction at the culvert location. Where granular fill will be placed below the water level, Granular 'B' Type II should be used. The material placed above the water level should be compacted in accordance with OPSS 501 (Compacting). Compression settlement of the fill placed below water (Granular 'B' Type II in this case) and from properly compacted embankment fill above water is expected to occur during construction.

6.5 Horizontal Strain

Horizontal strain along the culvert is not expected to occur provided the proposed grade raise and side slope geometry is consistent with the embankment cross-section presented in Section 6.4. Should the embankment be widened or raised by a greater height than presently proposed, a reassessment of the potential magnitude of horizontal strain acting on the culvert will be required.

Replacement of the culvert can be carried out concurrent with embankment reconstruction without the need for any foundation mitigation measures or provision of a camber to the culvert designed to tolerate the estimated and total differential settlement noted above.

6.6 Geotechnical Resistance

For the box culvert replacement, we recommend that a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 175 kPa be used for design for a box culvert of similar dimensions to the existing culvert founded on a properly prepared subgrade surface of the underlying soils, comprised of loose to compact silt to silty sand deposits underlain by the generally firm to stiff clayey silt to silt clay deposits. As there will not be any embankment widening at this site, the geotechnical resistance at SLS (for 25 mm settlement) for two times 5.3 m wide cell box culvert constructed on the properly prepared granular subgrade may be taken as 75 kPa.

It should be noted that the loading on the foundation soils below the culvert replacement will be governed by the embankment fills. If a widening is contemplated the structural engineer must exercise caution when utilizing the value of the geotechnical axial resistance at Serviceability Limit States (SLS) in the design of the culvert and that consideration is given to the sequence and staging of construction.

The geotechnical resistances are given for loads applied perpendicular to the surface of the base of the replacement culvert. Where loads are not applied perpendicular to the base of the culvert, inclination of the loads should be taken into account in accordance with Section 6.7.4 and Section C6.7.4 of the Canadian Highway Bridge Code (CHBDC) and its Commentary. For the estimation of the factored ULS value, a minimum culvert embedment depth of the height of the culvert was used in this analysis.

6.6.1 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the base of a concrete box culvert and the granular fill/bedding should be calculated in accordance with Section 6.7.5 of the CHBDC. The following summarizes the unfactored values of coefficient of friction for the interface materials for a precast and cast-in-place concrete box culvert:

Interface Materials	Coefficient of Friction
Precast Concrete Box on Compacted Granular ‘B’ Type II	tan δ = 0.45
Cast-in-Place Concrete on Compacted Granular ‘B’ Type II	tan δ = 0.58

These values represent unfactored values.

6.6.2 Frost Protection

The estimated frost penetration depth for the Sturgeon Falls area is 2.0 m, as per OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario).

Box culverts are typically not provided with the standard depth of cover on the base for frost protection as box culverts are tolerant to small magnitudes of movement related to freeze – thaw cycles should these occur. The box culvert should, however, be founded below any existing fill and organic materials. It is recommended that the box culvert segments be placed on a minimum thickness of 300 mm of OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II bedding material. If placed in the wet, Granular “B” Type II material should be used.

6.7 Lateral Earth Pressures – Culverts and Wing Walls

The lateral earth pressures acting on the side walls and wing walls of culverts will depend on the type and method of placement of backfill materials, the nature of soils/embankment fill behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of the culvert and any wing walls for this site. It should be noted that these design recommendations and parameters are applicable to 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 requirements of OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II but with less than 5 percent passing the 200 sieve (0.075 mm) should be used as backfill against the culvert and behind the wing walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub drains and frost taper immediately behind the culvert walls should be in accordance OPSD 3121.150 (Walls Retaining, Backfill).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the culvert and wing walls, in accordance with CHBDC Section 6.9.3 and Figure 6.6.

Compaction equipment should be used in accordance with OPSS 501 (Compaction). Other surcharge loadings should be accounted for in the design as required.

- Granular fill may be placed either in a zone with the width equal to at least 2.0 m behind the back of the wing walls and culvert walls for a restrained wall (see Figure C6.20(a) of the Commentary to the CHBDC), or within the wedge shaped zone defined by a line drawn at 1.5H:1V extending up and back from the rear face of the base of the wing walls and culvert walls for an unrestrained wall (see Figure C6.20(b) of the Commentary to the CHBDC). The pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used:

Fill Type	**Unit Weight**	**Coefficients of Static Lateral Earth Pressure**	
		At-Rest, K_o	**Active, K_a**
Granular ‘A’	22 kN/m^3	0.43	0.27
Granular ‘B’ Type II	21 kN/m^3	0.43	0.27

If the culvert structure allows for lateral yielding, active earth pressures may be used in the foundation design. If the culvert structure does not allow for lateral yielding, at-rest earth pressures should be assumed for culvert design. The movement to allow active pressures to develop within the backfill, and thereby assume a restrained structure, may be taken as per Table C6.6 of the Commentary to the CHBDC.

6.8 Culvert Construction Considerations

6.8.1 Excavation, Subgrade Preparation, Bedding and Backfill above Base of Culvert

Construction of the pre-cast concrete box culvert with respect to excavation, bedding, backfilling and cover materials, should be carried out in accordance with OPSS 422 (Concrete Box Culverts and Box Sewers in Open Cut).

All excavations must be carried out in accordance with Ontario Regulation 213 Ontario Occupational Health and Safety Act for Construction Projects (as amended by Ontario Regulation 443). In addition, provisions for traffic control measures should be included in the Contract Documents to maintain the safe operation of the existing Highway 17. Temporary excavation support systems should be designed and constructed in accordance with OPSS 539 (Temporary Protection Systems) and take into consideration the presence of cobble and boulder size blast rock within the embankment fill and is discussed further in Section 6.8.2.

Prior to the placement of any bedding material and fill for new construction, all organic soils where encountered should be stripped from the plan limits of the proposed works. The native subgrade soils may be susceptible to disturbance from construction traffic and/or ponded water. To limit degradation of the foundations area, it is recommended that a concrete working slab be placed on the subgrade if culvert construction is not carried out within four hours after preparation, inspection and approval of the subgrade. A sample NSSP to address this requirement is included in Appendix C.

For a cast-in-place box culvert replacement, groundwater control will be required as discussed in Section 6.8.4. As an alternative to a working slab discussed above, the box culvert could be constructed on a minimum 300 mm thick layer of OPSS.PROV 1010 (Aggregates) Granular 'B' Type II material for bedding purposes and partial frost protection. The Granular 'B' Type II may likely be placed in the wet and when nominally compacted (i.e., with an excavator shovel) should achieve a density of 90 per cent of the Standard Proctor Maximum Dry Density (SPMDD). The structural design of the culvert should take into consideration the conditions for bedding placement and compaction in accordance with the requirements of Section 7.8.3.6 of the CHBDC.

The thickness of backfill placed around the culvert as backfilling proceeds should be maintained equal on both sides of the culvert with one side not exceeding the other by more than 500 mm.

The culvert should be designed for the full overburden stress and appropriate live loads, assuming a fill unit weight of 22 kN/m^3 for Granular 'A' and 21 kN/m^3 for Granular 'B' Type II backfill above and surrounding the culvert.

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

The new granular fill used to reconstruct the embankment, including the culvert surround, will need to be keyed into the existing fill as per OPSD 208.010 (Benching of Earth Slopes). Fill for reconstruction of the embankment could consist of the material excavated from the embankment except for the material placed against the culvert walls and the initial cover over the culvert roof, provided that the excavated material can be properly compacted consistent with accepted standard for roadway embankment construction.

6.8.2 Temporary Shoring

Temporary protection systems will be required to support the embankment fill during the culvert replacement if the construction is carried out in stages and the highway continues in operation during the construction period. The temporary support systems could consist of driven steel sheet piling, however, soldier pile and lagging system may be more practical due to the presence of the cobble and boulder size blast rock within the embankment fill, as inferred from augers guiding during advancement of the boreholes, and as observed on the embankment side slopes and at/near the toes of the embankment. The Contractor should be alerted to the potential presence of these obstructions; an example NSSP (or Notice to Contractor) to be included in the Contract Documents is presented in Appendix C.

The temporary excavation support systems should be designed and constructed in accordance with OPSS 539 (Temporary Protection Systems). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539. The contractor is responsible for the complete detail design of the protection system.

The design of the temporary excavation support systems should be based on a rectangular earth pressure distribution using the design parameters given below. Where the support to the wall is provided by anchors or rakers, the wall design should be based on a triangular earth pressure distribution using the design parameters given below. The raker/anchor support must be designed to accommodate the loads applied from earth pressures and surcharge pressures from area, line or point loads as well as the impact of sloping ground behind the system. Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile diameter.

The unfactored triangular earth pressure distribution (p in kN/m^2; increasing with depth), can be calculated as follows:

p = $K_a (\gamma H + q)$

where H = the depth of the excavation at any point (m)

K_a = active coefficient of earth pressure

γ = soil unit weight (kN/m^3)

q = surcharge for traffic and other loading (kN/m^2)

For a braced excavation in granular fill and native cohesionless soils, the unfactored rectangular earth pressure distribution (p in kN/m^2; constant with depth), can be calculated as follows:

p = $0.65\, K_a(\gamma H + q)$

where H = the total depth of the excavation (m)

K_a = active coefficient of earth pressure

γ = soil unit weight (kN/m^3)

q = surcharge for traffic and other loading (kN/m^2)

For a braced excavation in cohesive soil, the unfactored rectangular earth pressure distribution (p in kN/m^2; varying with depth), can be calculated as follows:

p = 0 at ground surface increasing linearly to a depth of $0.25\, H_T$ to:

p = $\gamma H_T - 4mS_u$ at $0.25\, H_T$ and from $0.25\, H_T$ to H_T below ground surface

where H_T = the total depth of the excavation (m)

γ = soil unit weight (kN/m^3)

q = surcharge for traffic and other loading (kN/m^2)

m = 0.4 if an extensive soft clay layer underlies the excavation

1.0 if more resistant layer is present at the excavation base

S_u = undrained shear strength (kN/m^2).

The support systems may be designed using the following parameters:

SOIL TYPE	COEFFICIENT OF EARTH PRESSURE			INTERNAL ANGLE OF FRICTION	UNIT WEIGHT	UNDRAINED SHEAR STRENGTH
	Active, K_a	At Rest, K_o	Passive, K_p	(ϕ, degrees)	(γ, kN/m^2)	(S_u, kPa)
Existing Embankment Fill	0.33	0.50	3.0	30	20	-
Silt/Silty Sand/ Gravelly Sand	0.33	0.50	3.0	30	19	-
Clayey Silt to Silty Clay	0.30	0.55	2.7	27	18	30 at Elev 198 m to 60 kPa

The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficients should be adjusted accordingly.

6.8.3 Erosion Protection

Provisions should be made for scour and erosion protection (suitable non-woven geotextiles and/or rip-rap) at the culvert location and at the creek bends at either end of the culvert. In order to prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a concrete cut-off wall or clay seal should be provided at the upstream end of the culvert. If a clay seal is adopted, the clay material should meet the requirements of OPSS 1205 (Clay Seal), and the seal should be a minimum 1 m thick and constructed of natural clay or soil-bentonite mix and extend from a depth of 1 m below the scour level to a minimum horizontal distance of 2 m on either side of the culvert inlet opening, and a minimum vertical height equivalent to the high water level including along the embankment slope. Alternatively, a 0.6 m thick clay blanket (of natural clay or a soil-bentonite mix) may be constructed, extending upstream three (3) times the culvert height and along the adjacent slopes to a height of two (2) times the culvert height or the high water level, whichever is greater.

The requirements for and design of erosion protection measures for the inlet and outlet of the culvert should be assessed by the hydraulics design engineer. As a minimum, rip-rap treatment for the outlet of the culvert should be consistent with the standard presented in OPSD 810.010 (Rip-Rap Treatment for Sewer and Culvert Outlets). Erosion protection for the inlet of the culverts should follow the standard presented in OPSD 810.010 similar to the outlet but with the rip-rap placed up to the toe of slope level, in combination with the cut-off measures noted above. Similarly, rip-rap should be provided over the full extent of the clay blanket, including the creek side slopes and fill slope over the culvert.

6.8.4 Control of Groundwater and Surface Water

Excavation within the plan limits of the proposed culvert replacement and wing walls will require the removal of organics (where encountered), existing fill, and native soils prior to the placement of bedding material, and construction of the box culvert. The existing culvert flows will likely need to be diverted/piped during construction. Construction of the precast concrete culvert can be carried out in the wet provided the water level in the excavation/construction zone is maintained at the same level (or higher) than the water level outside the construction zone to prevent loosening of the base of the excavation due to unbalanced hydrostatic pressure.

Surficial water seepage into the excavation should be expected and will be heavier during periods of sustained precipitation. Seepage from the granular fills and near surface native granular materials should be expected, particularly after precipitation events. It is anticipated that this surficial seepage can be controlled by using properly filtered sumps within a shored/braced excavation.

Dewatering of the excavation will be required to allow for construction of either a precast or cast-in-place concrete box culvert in-the-dry. If this construction method is adopted, as the excavation will be advanced through cohesionless soils above the clay deposit, appropriate dewatering of the water-bearing granular soil deposits would be required to a water level at least 0.6 m below the base of the excavation prior to the start of excavation operations and continued until after the culvert is in place and the area backfilled to the culvert invert. It is recommended that an NSSP be included in the Contract to address unwatering for the site; a sample NSSP is included in Appendix C.

6.8.5 Analytical Testing for Construction Materials

The analytical test results on a sample of creek water taken adjacent to the culvert site are summarized in Table B1 in Appendix B, together with the detailed analytical laboratory test results. The suite of parameters tested is intended to allow the structural engineer to assess the requirements for the appropriate type of cement to be used in construction and the need for corrosion protection.

7.0 CLOSURE

This report was prepared by Mr. André Bom, P.Eng. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and a Principal with Golder, conducted an independent quality control review of the report.

Report Signature Page

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AC/AB/JMAC/kp

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\\golder.gds\gal\whitby\active_2013\1184 pavement and materials\13-1184-0074 mh hwy 17 warren to verner\1191-foundation\reporting\final\r04 - bear\13-1184-0074-r4 rpt 14nov27 final fidr bear creek.docx

REFERENCES

Bjerrum, L., 1973. Problems of Soil Mechanics and Construction of Soft Clays and Structurally Unstable Soils. State of the Art Report, Session 4. Proceedings, 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, Vol. 3, pp. 111-159.

Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA-S6-06. 2006. CSA Special Publication, S6.1-06. Canadian Standard Association.

Unified Facilities Criteria, NAVFAC Design Manual, DM-7.2. Soil Mechanics, Foundation and Earth Structures. U.S. Navy, 1982, Alexandria, Virginia.

Ontario Geological Survey. Southern Ontario Engineering Geology Terrain Study, 1980.

STANDARDS

ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D1587	Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes
ASTM D2573	Standard Test Method for Field Vane Shear Test in Cohesive Soil

Commercial Software

GeoStudio 2007 (Version 7.23) by Geo-Slope International Ltd.

Ontario Occupational Health and Safety Act

Ontario Regulation 213/91	Construction Projects
Ontario Regulation 443/09	Amendment to Ontario Regulation 213

Ontario Provincial Standard Drawing

OPSD 208.010	Benching of Earth Slopes
OPSD 810.010	Rip-Rap Treatment for Sewer and Culvert Outlets
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement
OPSD 202.010	Slope Flattening Using Surplus Excavated Material on Earth or Rock Fill Embankment

Ontario Provincial Standard Specification

OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut
OPSS 501	Construction Specification for Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSS 1205	Clay Seal

Ontario Water Resources Act

Ontario Regulation 372/97	Amendment to Ontario Regulation 903

MINISTRY OF TRANSPORTATION, ONTARIO

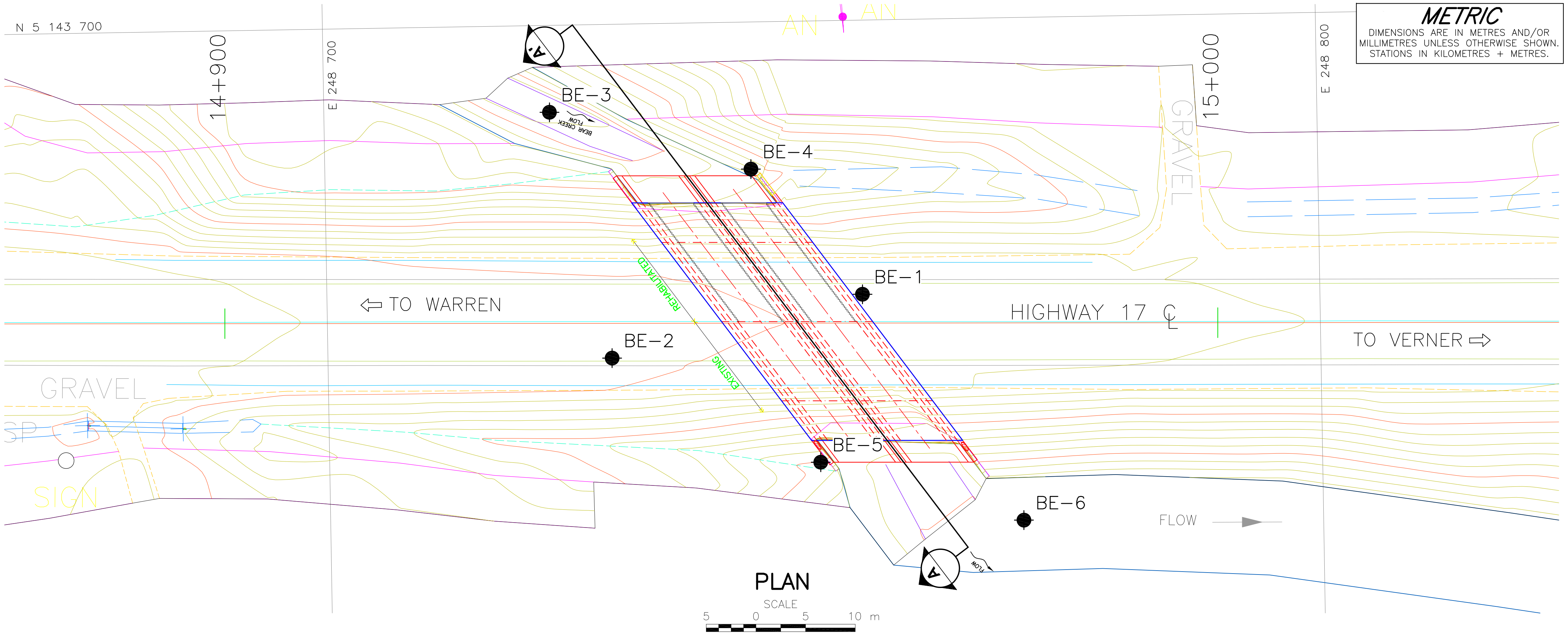
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 300–98–01

SHEET

HIGHWAY 17
BEAR CREEK CULVERT – STA 14+960
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



PLAN
SCALE
5 0 5 10 m

KEY PLAN
SCALE
8 0 8 km

LEGEND

- ● Borehole – Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- ⊽ WL upon completion of drilling

BOREHOLE CO–ORDINATES

No.	ELEVATION	NORTHING	EASTING
BE–1	207.6	5143672.4	248753.9
BE–2	207.4	5143666.4	248728.6
BE–3	200.9	5143691.3	248722.7
BE–4	203.4	5143685.2	248742.9
BE–5	202.1	5143655.6	248749.4
BE–6	202.7	5143649.4	248769.8

NOTES

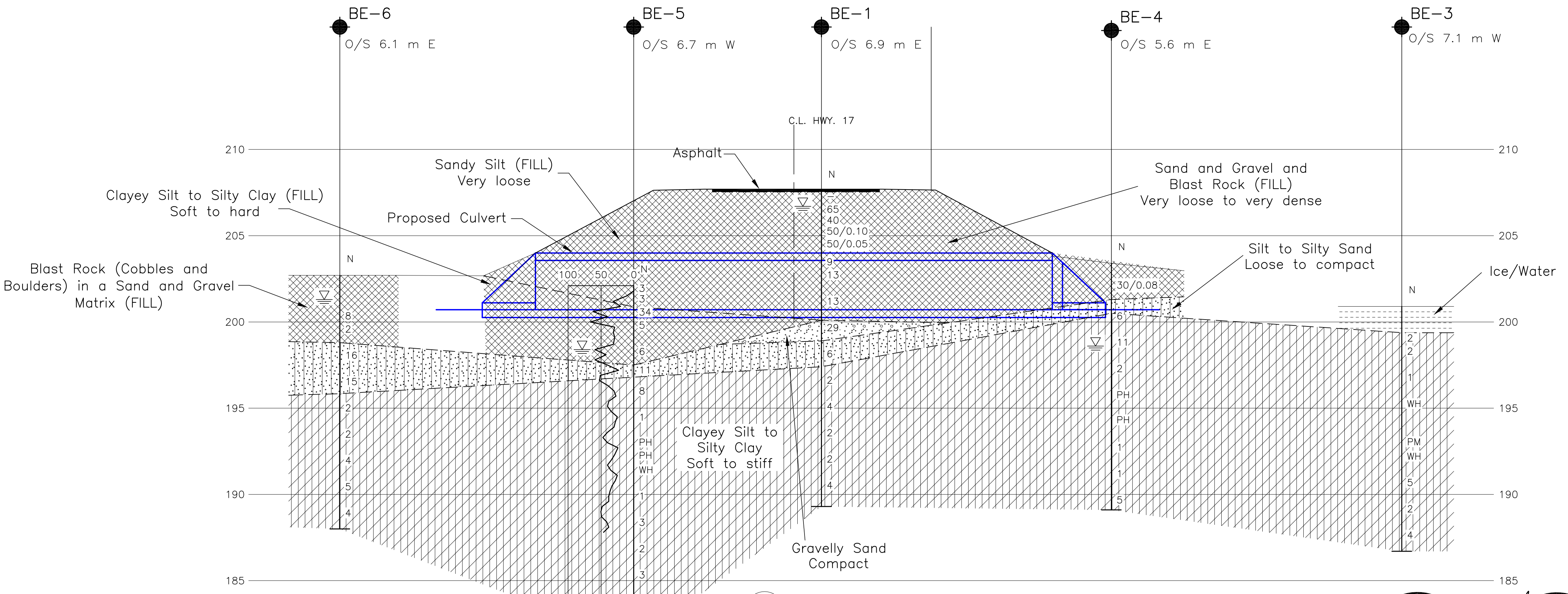
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

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

REFERENCE

Base plans provided in digital format by Morrison Hershfield, drawing file no. X–1130113_Base Plan.dwg, received NOV 13, 2014 and 43–231C_01.dwg, received NOV 13, 2014.



A–A'
PROFILE
BEAR CREEK
HORIZONTAL SCALE
3 0 3 6 m
3 0 3 6 m
VERTICAL SCALE

A.J.K. BOM
100075715
NOV 27, 2014

J.M.A. COSTA
NOV 27, 2014

NO.	DATE	BY	REVISION

Geocres No. 411–321

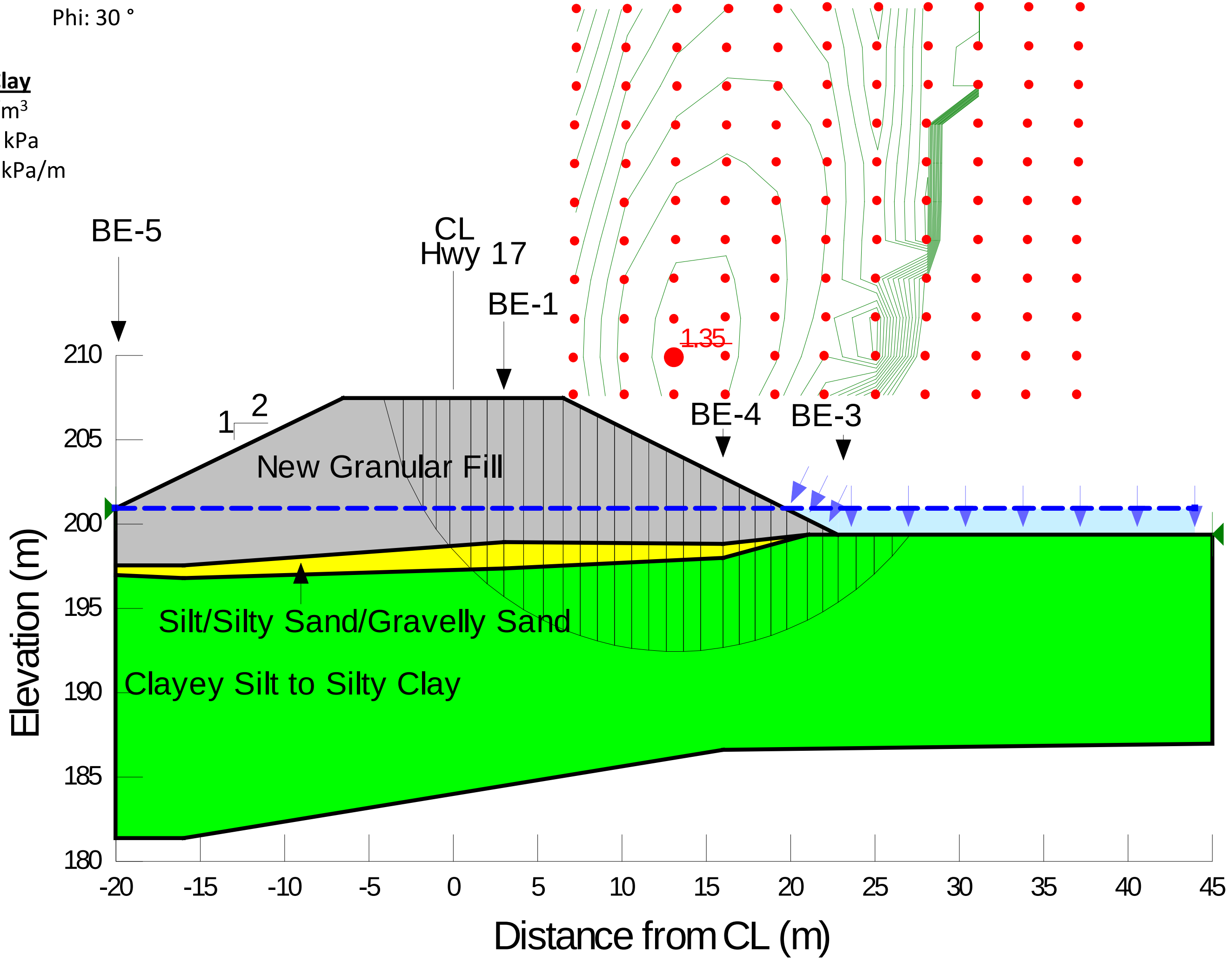
HWY. 17		PROJECT NO. 13–1184–0074	DIST.
SUBM'D. MT	CHKD.	DATE: NOV 2014	SITE: 43–231/C
DRAWN: TB	CHKD. AB	APPD. JMAC	DWG. 1

PLOT DATE: November 26, 2014

New Granular Fill
Unit Weight: 21 kN/m^3
Phi: 35 °

Silt/Silty Sand/Gravelly Sand
Unit Weight: 19 kN/m^3
Phi: 30 °

Clayey Silt to Silty Clay
Unit Weight: 18 kN/m^3
Upper Cohesion: 30 kPa
Rate of Change: 1.5 kPa/m



PROJECT	HIGHWAY 17 – Bear Creek Culvert			
TITLE	**STABILITY ANALYSIS EMBANKMENT NORTH SIDE SLOPE**			
Golder Associates	PROJECT No. 13-1184-0074			FILE No. ----
	DESIGN			SCALE AS SHOWN REV.
	CADD	--		**FIGURE 1**
	CHECK	AB	Oct 2014	
	REVIEW	JMAC	Octl 2014	

APPENDIX A

Record of Boreholes

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes:
1 $\tau = c' + \sigma' \tan\phi'$
2 shear strength = (compressive strength)/2

---

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

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils

Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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

RECORD OF BOREHOLE No BE-1

1 OF 2 **METRIC**

PROJECT	13-1184-0074		
W.P.	300-98-01	LOCATION	N 5143672.4; E 248753.9
DIST		HWY 17	
BOREHOLE TYPE	108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring		
DATUM	GEODETIC	DATE	December 10, 2013
ORIGINATED BY	SA	COMPILED BY	MT
CHECKED BY	DAM		

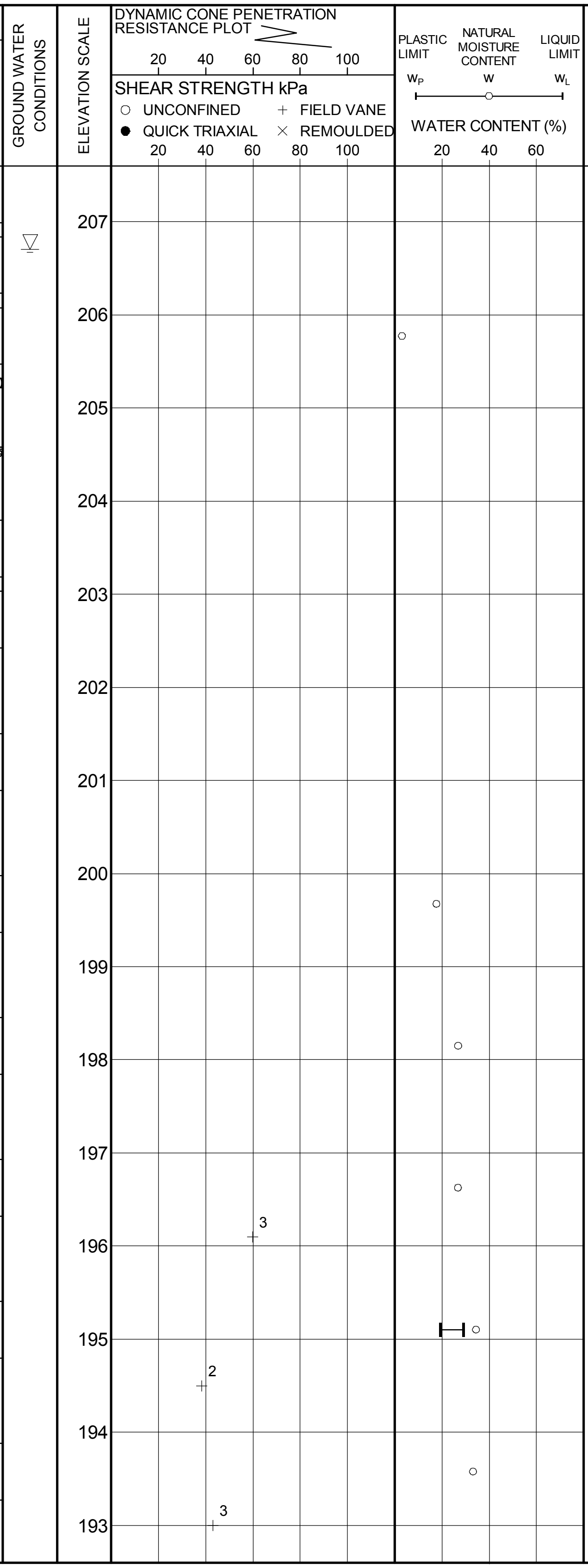
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
207.6	GROUND SURFACE
0.0	ASPHALT (250 mm)
0.3	Sand and gravel and blast rock (FILL) Loose to very dense Brown Moist to wet Switched to casing at 3.0 m depth. Cobbles recovered in casing barrel as follows: Depth (m) / Thickness (mm) 3.1 / 150 3.5 / 100 7.3 / 200
200.1 7.5	Gravelly SAND, some silt, trace to some clay Compact Grey Wet
198.9 8.7	SILT, trace to some clay Loose Grey Wet
197.4 10.2	CLAYEY SILT to SILTY CLAY Firm to stiff Grey Wet

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
1	AS	-					
2	SS	65					
3	SS	40					
4	SS	50/0.10					
5	SS	50/0.05					
6	SS	9					
7	SS	13					
8	SS	13					
9	SS	29		23	53	17	7
10	SS	6	NP	0	1	88	11
11	SS	2					
12	SS	4		0	0	71	29
13	SS	2					



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

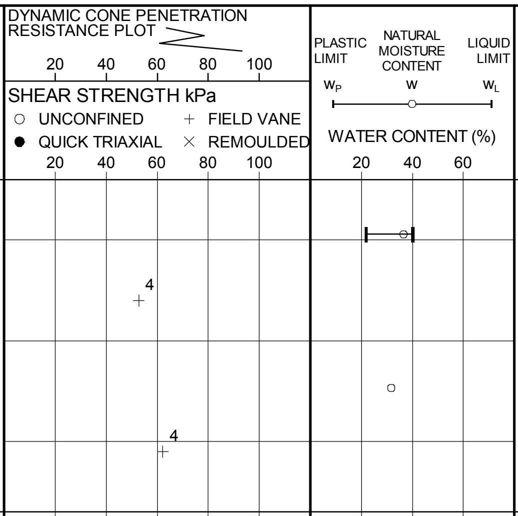
Golder Associates

RECORD OF BOREHOLE No BE-1

2 OF 2 **METRIC**

PROJECT 13-1184-0074
W.P. 300-98-01
LOCATION N 5143672.4; E 248753.9
ORIGINATED BY SA
DIST HWY 17
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring
COMPILED BY MT
DATUM GEODETIC
DATE December 10, 2013
CHECKED BY DAM

SOIL PROFILE			SAMPLES						REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	CLAYEY SILT to SILTY CLAY Firm to stiff Grey Wet		14	SS	2		192		0	0	62	38
							191					
	Silt laminations encountered below 16.8 m depth.		15	SS	4							
							190					
189.3 18.3	END OF BOREHOLE Note: 1. Water level at a depth of 0.9 m below ground surface (Elev. 206.7 m) upon completion of drilling.											



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

RECORD OF BOREHOLE No BE-2

1 OF 2 **METRIC**

PROJECT 13-1184-0074			
W.P. 300-98-01	LOCATION N 5143666.4; E 248728.6	ORIGINATED BY SA/TM	
DIST	HWY 17	BOREHOLE TYPE NW Casing, Wash Boring	COMPILED BY MT
DATUM GEODETIC	DATE January 14 and February 3 and 4, 2014	CHECKED BY DAM	

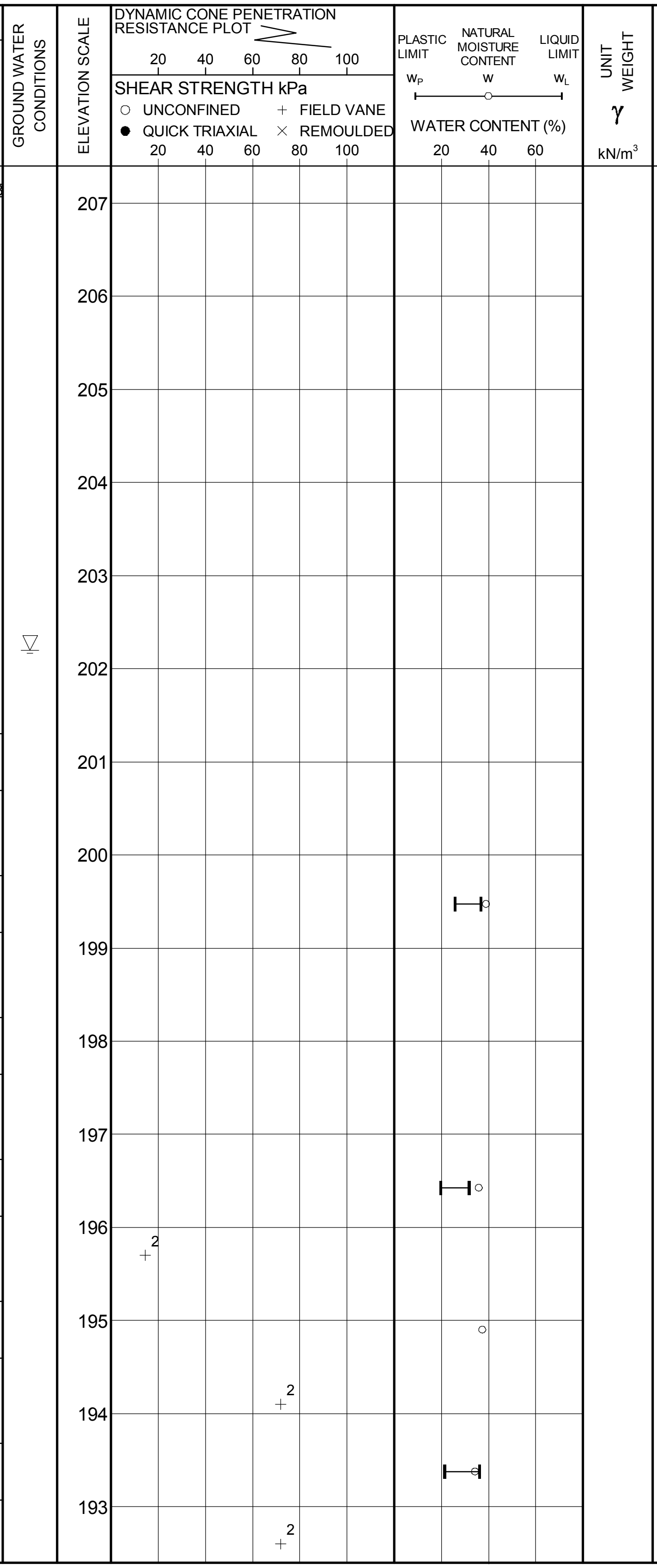
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
207.4	GROUND SURFACE
0.0	ASPHALT (175 mm)
0.2	Sand and gravel and blast rock (FILL) Compact Brown Wet Inferred cobbles and boulders as follows:
200.1 7.3	SILT and SAND, trace to some clay Loose Grey Wet
198.8 8.6	CLAYEY SILT to SILTY CLAY Soft to stiff Grey Wet

Depth (m)	Thickness (mm)
0.8	90
0.9	150
1.1	150
1.5	200
2.0	85
2.3	460
3.0	100
3.2	100
3.7	360
4.7	700
5.4	300
5.8	230
6.9	125

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	50/0.13				
2	SS	19				
3	SS	5	0	33	57	10
4	SS	7				
5	SS	1				
6	SS	WH				
7	SS	2				



Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BE-2

2 OF 2 **METRIC**

PROJECT 13-1184-0074
W.P. 300-98-01
LOCATION N 5143666.4; E 248728.6
ORIGINATED BY SA/TM
DIST
HWY 17
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY MT
DATUM GEODETIC
DATE January 14 and February 3 and 4, 2014
CHECKED BY DAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	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 (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- *CONTINUED FROM PREVIOUS PAGE* ---										
	CLAYEY SILT to SILTY CLAY Soft to stiff Grey Wet		8	SS	3		192	2			
191.1 16.3	END OF BOREHOLE Note: 1. Water level at a depth of 5.2 m below ground surface (Elev. 202.2 m) on February 4, 2014 prior to advancing borehole into native soils.										

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

SUD-MTO 001 13-1184-0074.GPJ GAL-MISS.GDT 28/10/14 DATA INPUT:

RECORD OF BOREHOLE No BE-3

1 OF 1 **METRIC**

PROJECT 13-1184-0074

W.P. 300-98-01

DIST ____ HWY 17

DATUM GEODETIC

LOCATION N 5143691.3; E 248722.7

BOREHOLE TYPE NW Casing, Wash Boring

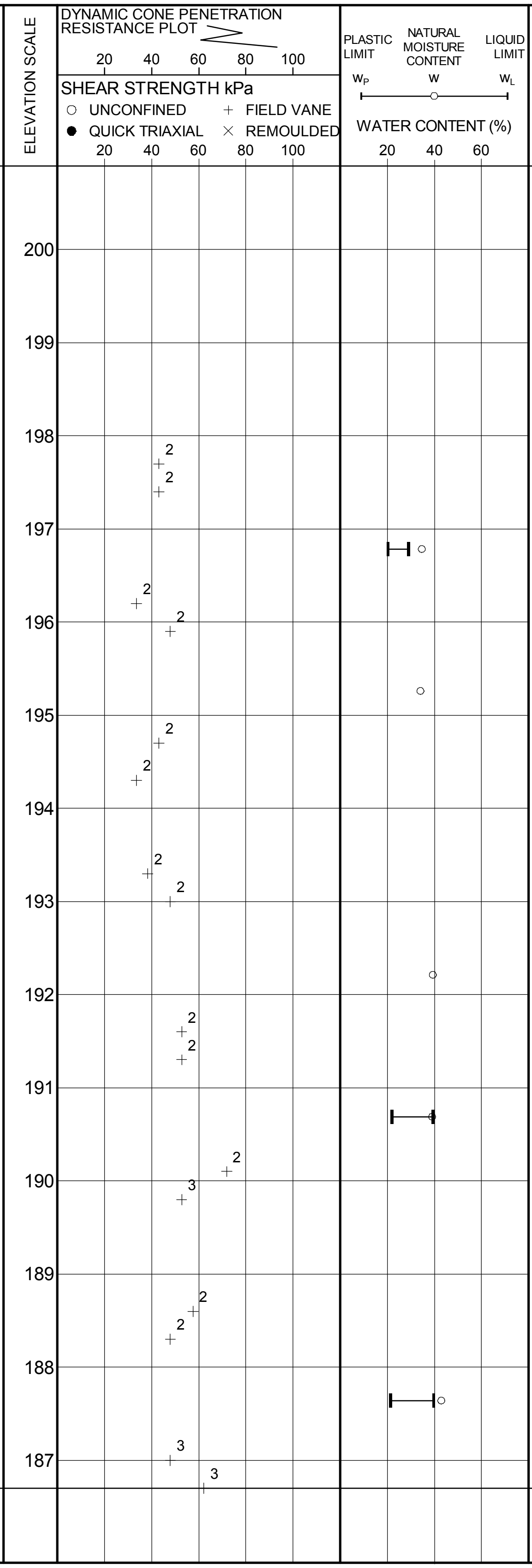
DATE March 4, 2014

ORIGINATED BY MR

COMPILED BY MT

CHECKED BY DAM

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
200.9	ICE SURFACE							
0.0	ICE/WATER							
199.4 1.5	CLAYEY SILT to SILTY CLAY Firm to stiff Grey Wet		1	SS	2			
			2	SS	2			
			3	SS	1			
			4	SS	WH			
			5	TO	PM			
			6	SS	WH			
			7	SS	5			
			8	SS	2			
			9	SS	4			
186.7 14.2	END OF BOREHOLE							



+3, ×3: Numbers refer to Sensitivity

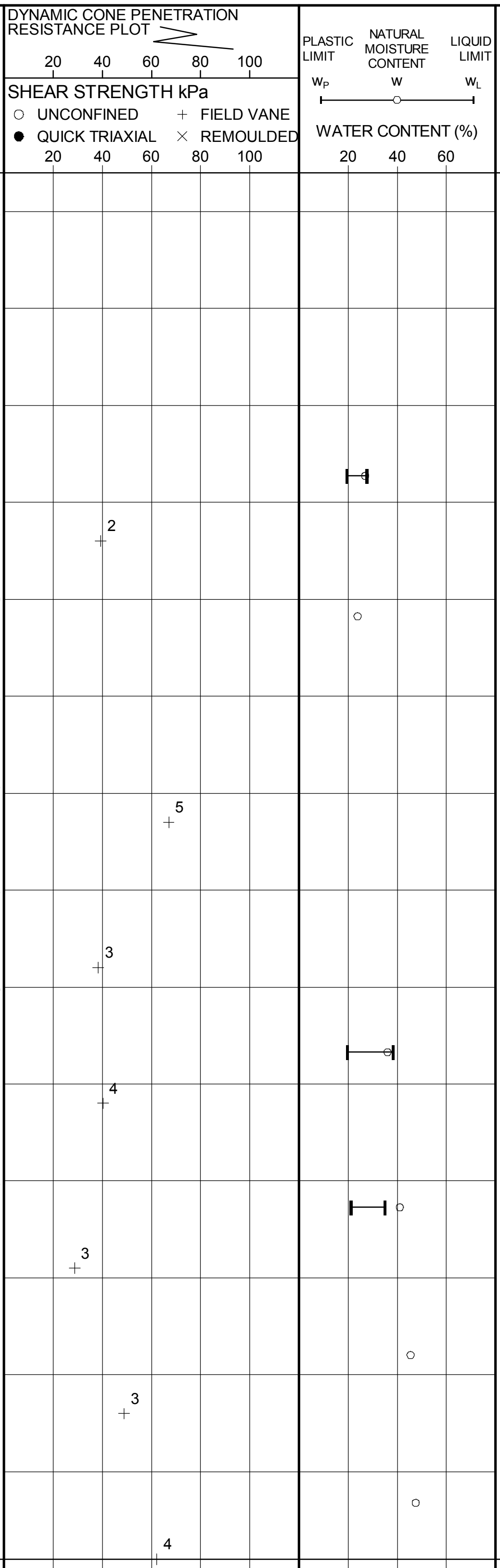
○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BE-4

1 OF 2 **METRIC**

PROJECT 13-1184-0074
W.P. 300-98-01
LOCATION N 5143685.2; E 248742.9
ORIGINATED BY GM
DIST HWY 17
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY MT
DATUM GEODETIC
DATE March 10 and 11, 2014
CHECKED BY DAM

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
203.4	GROUND SURFACE					203					
0.0	Blast rock (FILL)	1	SS	30/0.08		202					
201.3 2.1	Silty SAND, trace clay Brown Wet				▽	201					
200.5 2.9	CLAYEY SILT to SILTY CLAY, trace sand Firm to stiff Brown to grey Wet Approximately 0.6 m thick silt layer at 4.0 m depth.	2A 2B	SS	6		200		0	10	70	20
		3	SS	11		199	NP	0	2	89	9
		4	SS	2		198 197					
		5	TO	PH		196					
		6	TO	PH		195 194		0	0	87	13
		7	SS	1		193					
		8	SS	1		192 191					
		9	SS	5		190					
189.1 14.3											



Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BE-4

2 OF 2 **METRIC**

PROJECT 13-1184-0074
W.P. 300-98-01
LOCATION N 5143685.2; E 248742.9
ORIGINATED BY GM
DIST HWY 17
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY MT
DATUM GEODETIC
DATE March 10 and 11, 2014
CHECKED BY DAM

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 ○ 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 (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE Note: 1. Water level at a depth of 2.3 m below ground surface (Elev. 201.1 m) upon completion of drilling.										

SUD-MTO 001 13-1184-0074.GPJ GAL-MISS.GDT 28/10/14 DATA INPUT:

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

RECORD OF BOREHOLE No BE-5

1 OF 2 **METRIC**

PROJECT	13-1184-0074		
W.P.	300-98-01	LOCATION	N 5143655.6; E 248749.4
DIST	HWY 17	BOREHOLE TYPE	NW Casing, Wash Boring
DATUM	GEODETIC	DATE	January 13 and 20, 2014

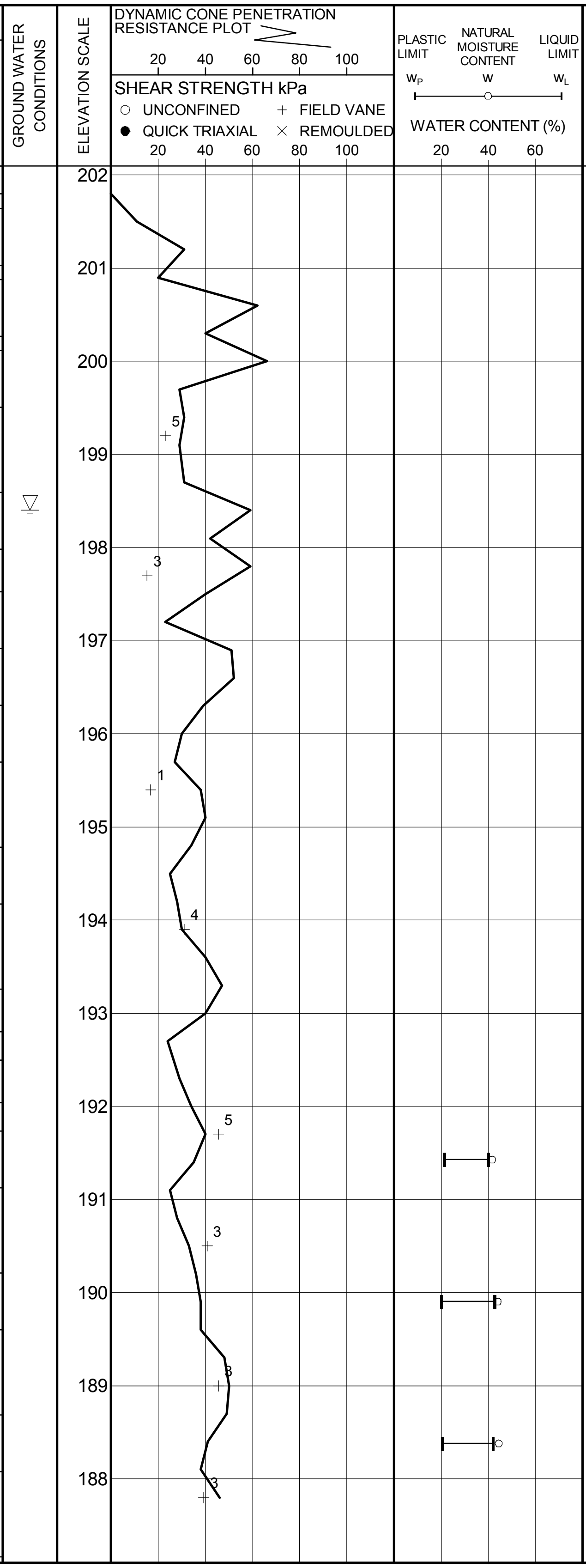
ORIGINATED BY GM
COMPILED BY MT
CHECKED BY DAM

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
202.1	GROUND SURFACE
0.0	Sandy silt, trace organics, trace to some clay (FILL) Very loose Brown Wet
200.9 1.2	Clayey silt to silty clay, trace to some sand, trace gravel (FILL) Soft to hard Grey Wet Approximately 150 mm thick layer of sand and 1.4 m depth. Refusal to further casing advancement at 1.7 m depth. Approximately 0.5 m and 0.3 m thick sand and gravel layer at 2.3 m and 3.5m depth respectively.
197.5 4.6	SILTY SAND Compact Brown Wet
196.8 5.3	CLAYEY SILT to SILTY CLAY Soft to stiff Grey Wet Silt laminations between 7.3 m and 8.4 m depth. Silt laminations between 10.7 m and 11.4 m depth. Silt laminations encountered between 13.8 m and 17.5 m depth.

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	3
2	SS	3
3	SS	34
4	SS	5
5	SS	6
6	SS	11
7	SS	8
8	SS	1
9	TO	PH
10	TO	PH
11	SS	WH
12	SS	1
13	SS	3



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)
GR SA SI CL

Continued Next Page

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

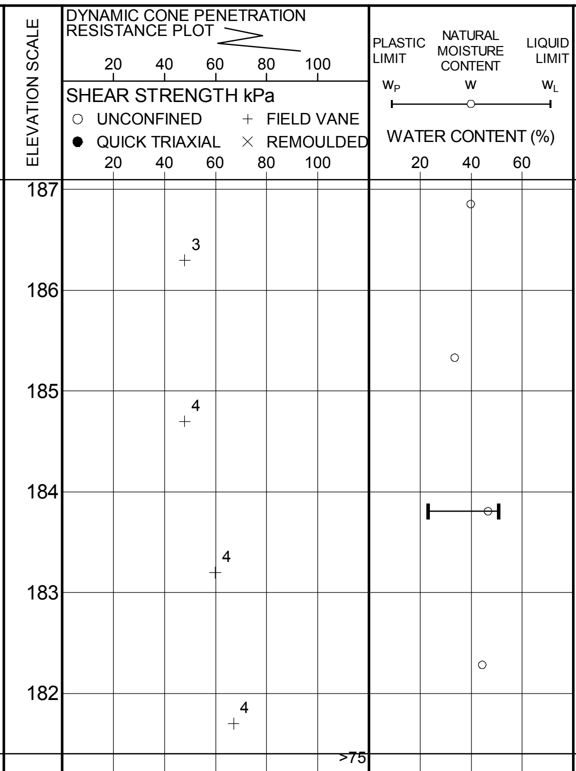
RECORD OF BOREHOLE No BE-5

2 OF 2 **METRIC**

PROJECT 13-1184-0074
W.P. 300-98-01
LOCATION N 5143655.6; E 248749.4
ORIGINATED BY GM
DIST
HWY 17
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY MT
DATUM GEODETIC
DATE January 13 and 20, 2014
CHECKED BY DAM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT to SILTY CLAY Soft to stiff Grey Wet		14	SS	2			
	Silt laminations between 16.5 m and 17.5 m depth.		15	SS	3			
			16	SS	3			
			17	SS	2			
181.4 20.7	END OF BOREHOLE Note: 1. Refusal to further casing advancement encountered at 1.7 m depth. 2. Water level at a depth of 3.4 m below ground surface (Elev. 198.7 m) upon completion of drilling.							



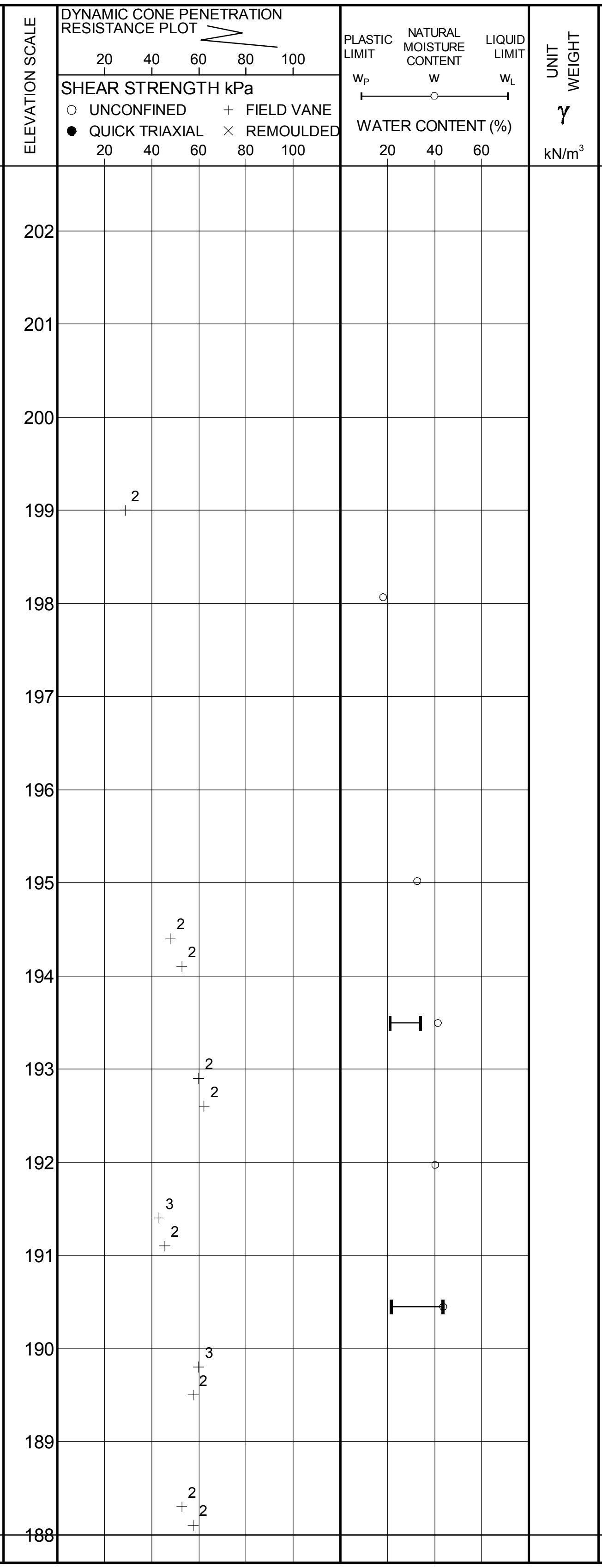
+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BE-6

1 OF 2 **METRIC**

PROJECT 13-1184-0074
W.P. 300-98-01
DIST
HWY 17
DATUM GEODETIC
LOCATION N 5143649.4; E 248769.8
BOREHOLE TYPE Excavator, NW Casing, Wash Boring
DATE March 12, 2014
ORIGINATED BY MR
COMPILED BY MT
CHECKED BY DAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
202.7	GROUND SURFACE									
0.0	Blast rock (cobbles and boulders) in a sand and gravel matrix (FILL)					∇				
200.7 2.0	Clayey silt to silty clay trace to some sand (FILL) Soft to firm Grey Wet		1	SS	8					
			2	SS	2					
198.8 3.9	SILTY SAND, some gravel, trace clay Compact Grey Wet		3	SS	16		12	58	23	7
			4	SS	15					
195.8 6.9	CLAYEY SILT to SILTY CLAY Firm to stiff Grey Wet Silt laminations between 6.9 and 8.4 m depth.		5	SS	2					
			6	SS	2					
	Silt laminations between 10.4 and 11.5 m depth.		7	SS	4					
			8	SS	5					
			9	SS	4					
188.0 14.7										



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

RECORD OF BOREHOLE No BE-6

2 OF 2 **METRIC**

PROJECT 13-1184-0074
W.P. 300-98-01
LOCATION N 5143649.4; E 248769.8
ORIGINATED BY MR
DIST HWY 17
BOREHOLE TYPE Excavator, NW Casing, Wash Boring
COMPILED BY MT
DATUM GEODETIC
DATE March 12, 2014
CHECKED BY DAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	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 (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION *--- CONTINUED FROM PREVIOUS PAGE ---*	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	END OF BOREHOLE Note: 1. Excavated blast rock fill to a depth 2.0 m. 2. Water level at a depth of 1.4 m below ground surface (Elev. 201.3 m) upon completion of drilling.										

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

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

APPENDIX B

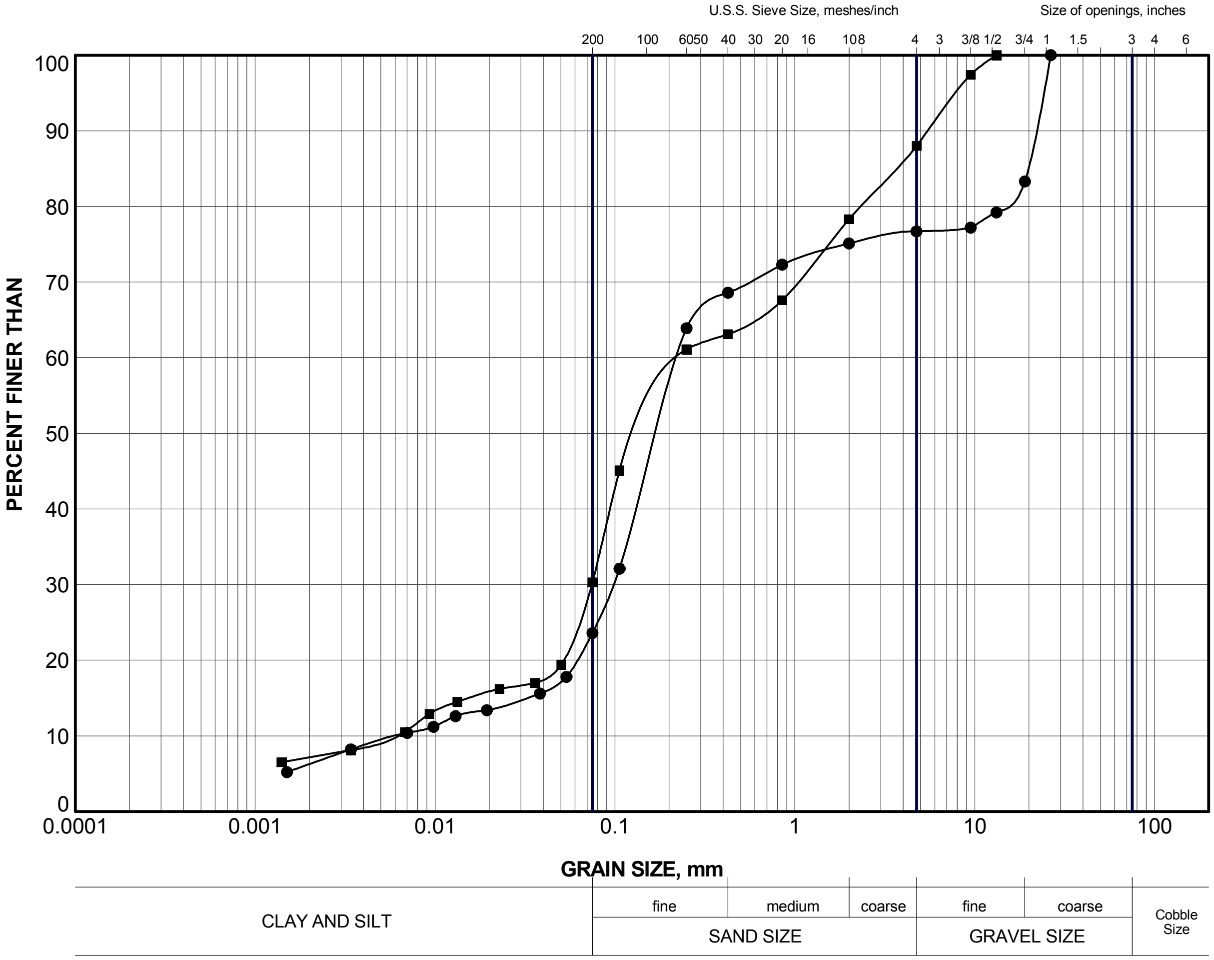
Laboratory Test Results

Table B1 - Summary of Analytical Testing of Bear Creek Water Sample

Parameter	Units	Reportable Detection Limit	Result
Dissolved Chloride	mg/L	1	4
Dissolved Sulphate	mg/L	1	2
Conductivity	µohm/cm	1	95
Resistivity	ohm-cm	n/a	11,000
pH	n/a	n/a	7.21

Notes: 1. Sample obtained on March 10, 2014.
2. Analytical testing carried out by Maxxam Analytics.

Checked by: AB



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BE-1	9	199.7
■	BE-6	3	198.1

PROJECT

HIGHWAY 17
BEAR CREEK CULVERT - STA 14+960

TITLE

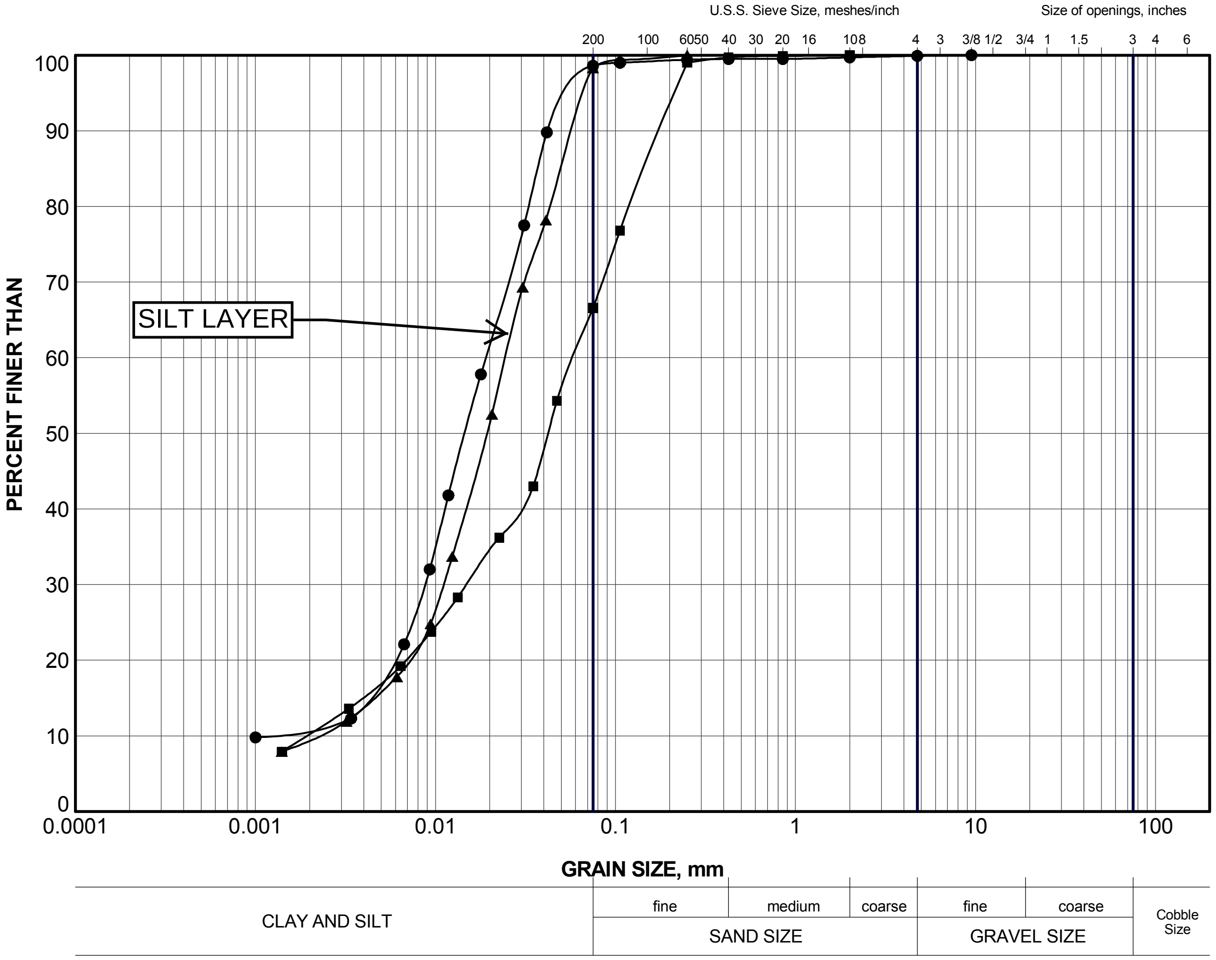
GRAIN SIZE DISTRIBUTION

GRAVELLY SAND to SILTY SAND

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		13-1184-0074	FILE No.	13-1184-0074.GPJ	
DRAWN	TB	Oct 2014	SCALE	N/A	REV.
CHECK	AB	Oct 2014	**FIGURE B1**		
APPR	JMAC	Oct 2014			

SUD-MTO GSD (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BE-1	10	198.2
■	BE-2	3	199.5
▲	BE-4	3	198.8

PROJECT

HIGHWAY 17
BEAR CREEK CULVERT - STA 14+960

TITLE

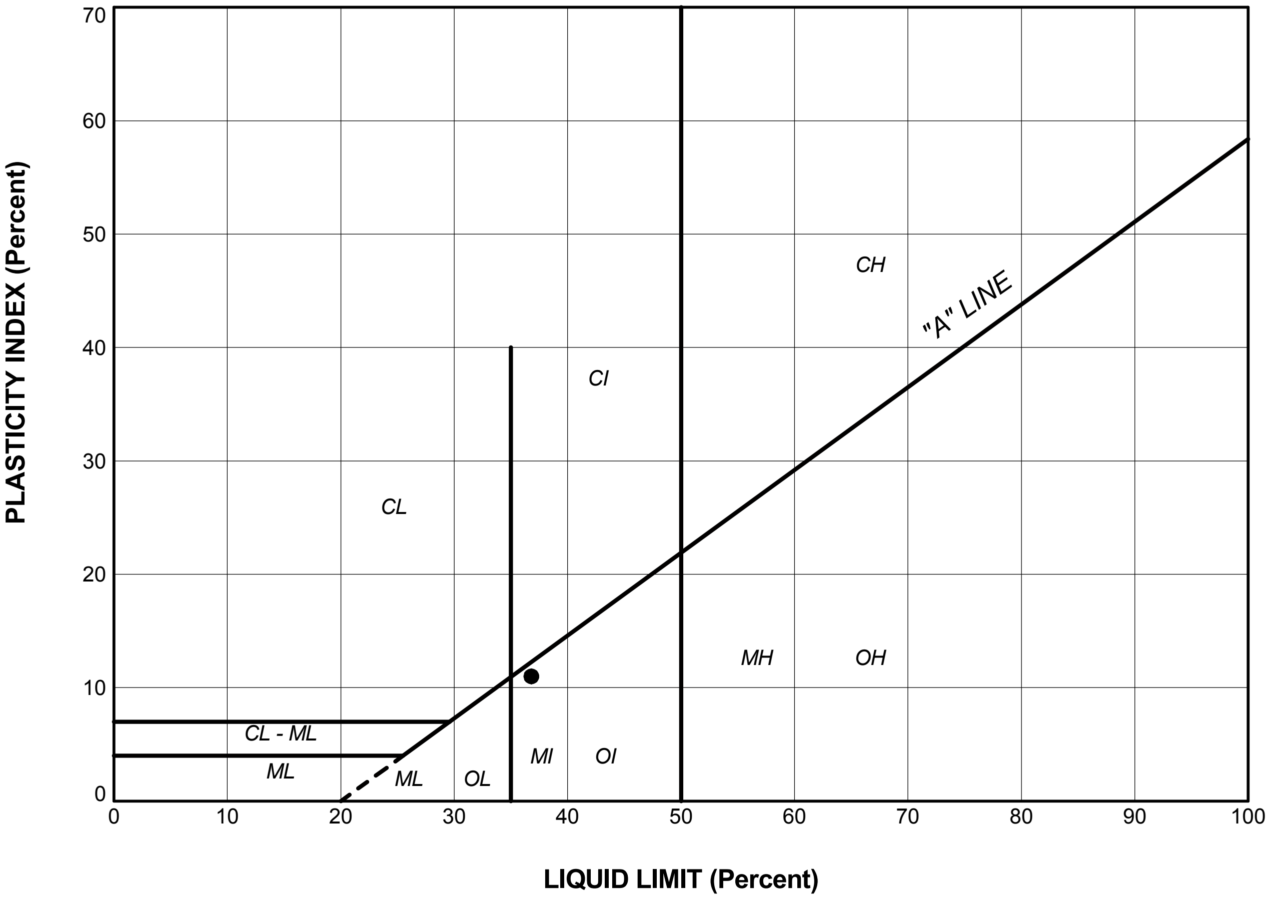
GRAIN SIZE DISTRIBUTION

SILT to SILT and SAND

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		13-1184-0074	FILE No.	13-1184-0074.GPJ	
DRAWN	TB	Oct 2014	SCALE	N/A	REV.
CHECK	AB	Oct 2014	**FIGURE B2**		
APPR	JMAC	Oct 2014			

SUD-MTO GSD (NEW) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BE-2	3	36.8	25.8	11.0

PROJECT

HIGHWAY 17
BEAR CREEK CULVERT - STA 14+960

TITLE

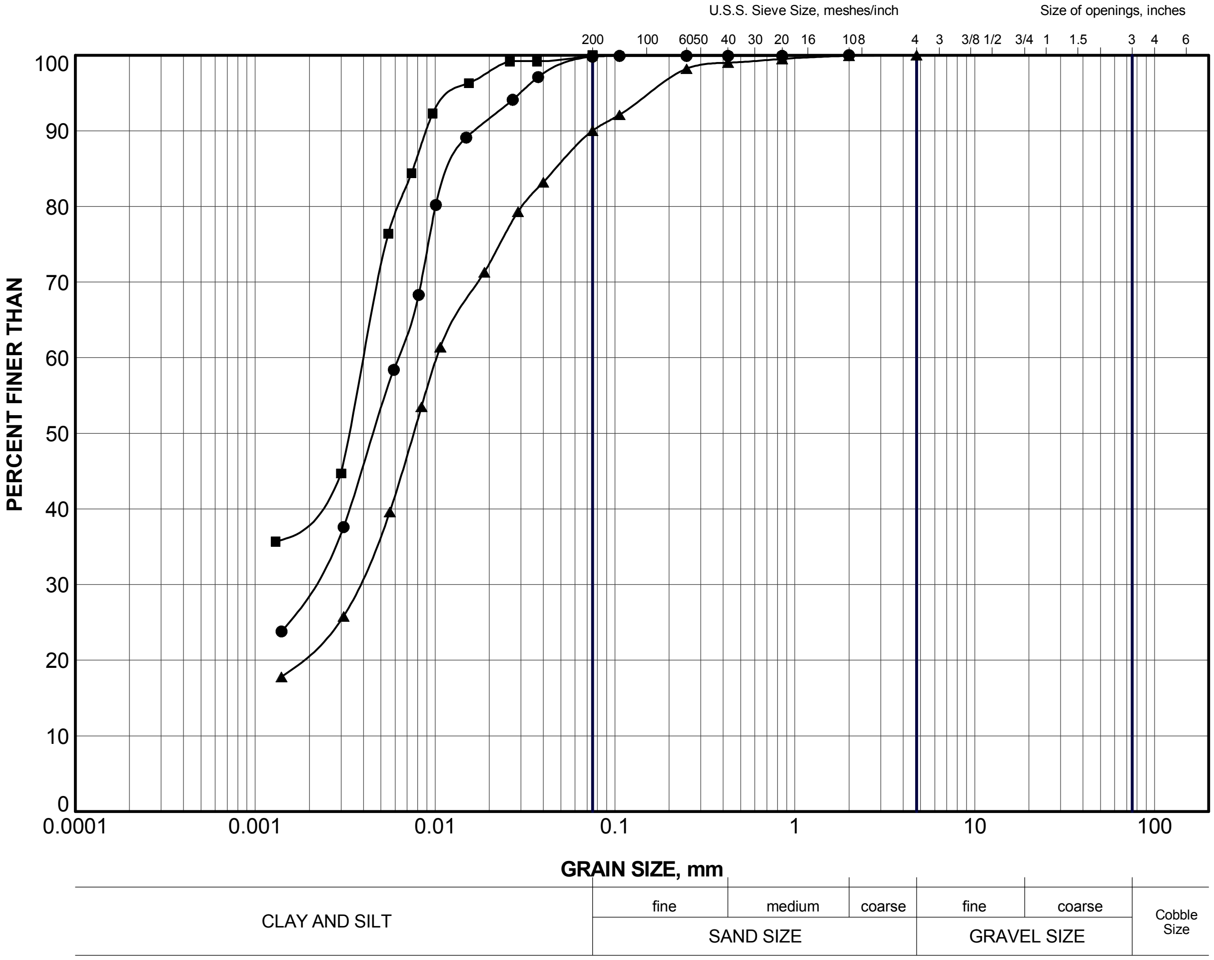
PLASTICITY CHART

SILT and SAND

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		13-1184-0074	FILE No.	13-1184-0074.GPJ	
DRAWN	TB	Oct 2014	SCALE	N/A	REV.
CHECK	AB	Oct 2014	**FIGURE B3**		
APPR	JMAC	Oct 2014			

SUD-MTO PL (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BE-1	12	195.1
■	BE-1	14	192.1
▲	BE-4	2B	200.3

PROJECT

HIGHWAY 17
BEAR CREEK CULVERT - STA 14+960

TITLE

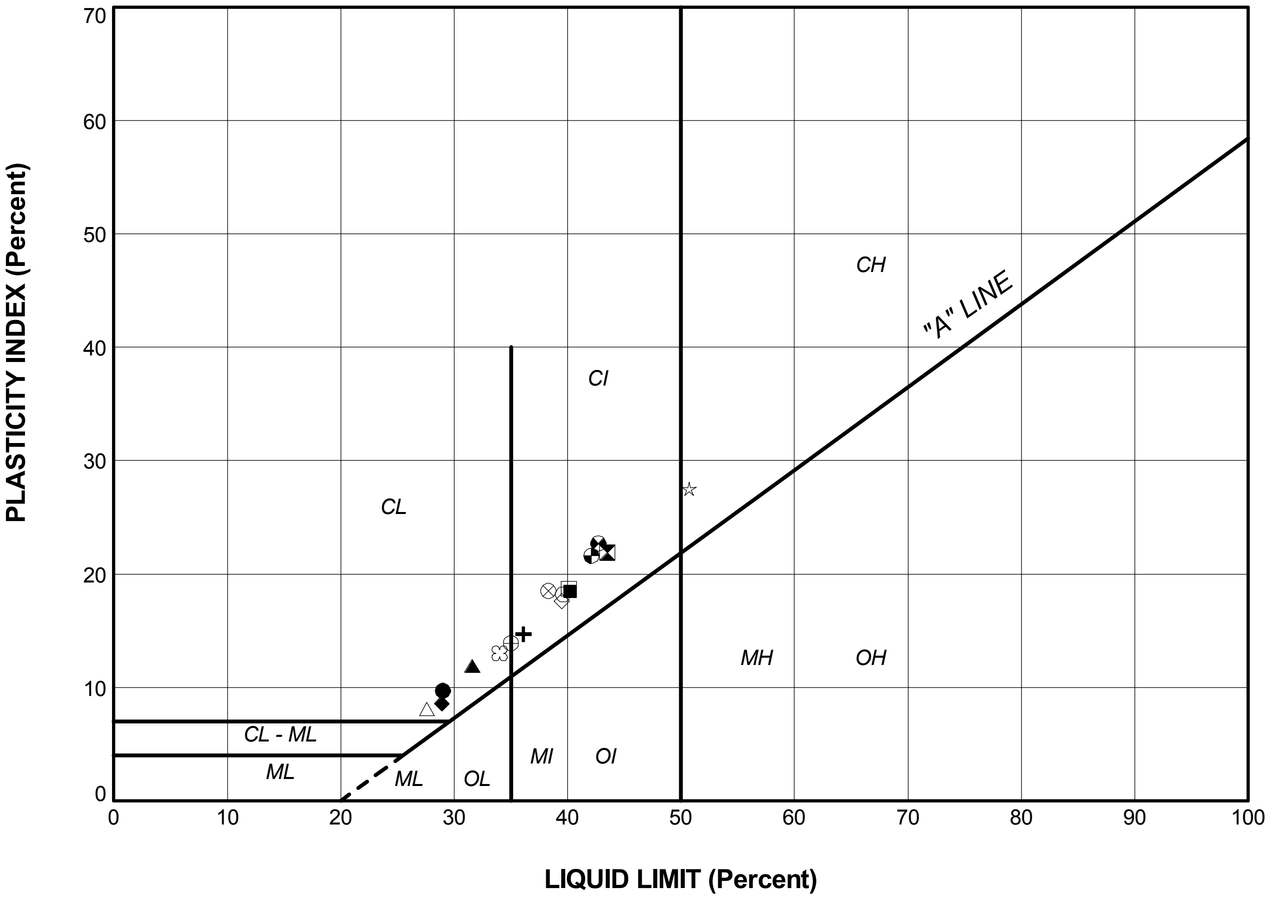
GRAIN SIZE DISTRIBUTION

CLAYEY SILT to SILTY CLAY

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	13-1184-0074		FILE No.	13-1184-0074.GPJ
DRAWN	TB	Oct 2014	SCALE N/A	REV.
CHECK	AB	Oct 2014		
APPR	JMAC	Oct 2014	**FIGURE B4**	

SUD-MTO GSD (NEW) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BE-1	12	29.0	19.3	9.7
■	BE-1	14	40.2	21.7	18.5
▲	BE-2	5	31.6	19.7	11.9
✚	BE-2	7	36.1	21.4	14.7
◆	BE-3	3	28.9	20.3	8.6
◇	BE-3	7	39.5	21.9	17.6
○	BE-3	9	39.6	21.4	18.2
△	BE-4	2B	27.6	19.5	8.1
⊗	BE-4	6	38.3	19.8	18.5
⊕	BE-4	7	35.0	21.1	13.9
□	BE-5	11	40.1	21.4	18.7
◑	BE-5	12	42.7	20.0	22.7
◕	BE-5	13	42.1	20.5	21.6
☆	BE-5	16	50.7	23.2	27.5
✤	BE-6	6	34.0	21.0	13.0
⧗	BE-6	8	43.5	21.6	21.9

PROJECT

HIGHWAY 17
BEAR CREEK CULVERT - STA 14+960

TITLE

PLASTICITY CHART

CLAYEY SILT to SILTY CLAY

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	13-1184-0074		FILE No.	13-1184-0074.GPJ
DRAWN	TB	Oct 2014	SCALE N/A	REV.
CHECK	AB	Oct 2014		
APPR	JMAC	Oct 2014	**FIGURE B5**	

APPENDIX C

Non-Standard Special Provisions

WORKING SLAB – Item No.

Non-Standard Special Provision

Scope of Work

The subgrade soils for the box culvert foundations may be susceptible to disturbance and loosening from construction traffic and ponded water.

Where precast box culverts are used, if all of the box segments are not placed on the prepared subgrade within four hours of its inspection and approval, a concrete working slab of 20 MPa compressive strength at 28 days with minimum thickness of 100 mm, shall be placed on the foundation subgrade. A minimum 75 mm thick uncompacted levelling pad consisting of Granular ‘A’ material (OPPS.PROV 1010) or concrete fine aggregate (meeting the grading requirements specified in OPSS.PROV 1002) shall be provided on top of the concrete working slab.

Basis of Payment

Payment at the lump sum contract price for the above tender item includes full compensation for all labour, equipment and material for completion of the work.

END OF SECTION

DEWATERING/UNWATERING OF STRUCTURE EXCAVATION - Item No.

Non-Standard Special Provision

Construction of the culvert in the dry within an excavation that extends below the groundwater level will require dewatering of the construction zone maintaining the water level at a depth of at least 0.6 m below the base of the excavation prior to carrying out the excavation operations. The cohesionless soils that are present below the groundwater table will slough, run, boil or cave into the excavation unless appropriate groundwater controls are in place. The Contractor is to design and install an appropriate dewatering system, and excavation protection and unwatering system to enable construction in dry conditions, to prevent disturbance to the founding soils.

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

OBSTRUCTIONS

Non-Standard Special Provision

The Contactor shall be alerted to the presence of cobble and boulder size blast rock likely present within the embankment fill as inferred from auger and casing advancement, and as observed on the slopes and at/near the toes of the embankment. Considerations of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for sub-excavation and installation of the temporary shoring and roadway protection system, if required.

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

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Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 56 2 2616 2000

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