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REPORT ON

FOUNDATION INVESTIGATION AND DESIGN REPORT CROSBY CREEK BRIDGE REPLACEMENT HIGHWAY 15, CROSBY, ONTARIO W.P. 479-92-00

Submitted to:

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REPORT



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CROSBY CREEK BRIDGE REPLACEMENT

PART A

**FOUNDATION INVESTIGATION REPORT
CROSBY CREEK BRIDGE REPLACEMENT
HIGHWAY 15
CROSBY, ONTARIO
W.P. 479-92-00**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Genivar on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the proposed rehabilitation of Highway 15 from south of Big Hill Road to south of Young's Hill Road.

Foundation investigation services are required on this project for the following components under W.P. 479-92-00:

- A new bridge to carry Highway 15 over Crosby Creek; and,
- Rehabilitation of the existing structural culvert at Station 26+040.

This report addresses the proposed new bridge over Crosby Creek and its approach embankments.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposals (RFP) dated March 2007, and in the Section 5.8 of the Technical Proposal dated April 2007. The work was carried out in accordance with Golder's Supplementary Specialty Quality Control Plan for foundation engineering services dated, August 22, 2007.



2.0 SITE DESCRIPTION

The site of the proposed bridge over Crosby Creek is located on Highway 15 just south of the town of Crosby, Ontario.

The existing bridge consists of a single span structure, about 12 m in length, with a concrete deck on rolled steel girders. The bridge abutments are understood to be founded on a double row of small diameter steel tube piles. The available information for the existing bridge is limited and the founding strata and lengths of the existing piles are unknown; presumably the existing piles were driven to refusal on the bedrock considering the relatively modest overburden thickness. The bridge was constructed in 1931 and was last rehabilitated in 1975. Due to age and the condition of the bridge, the Ministry of Transportations (MTO) has made the decision to replace the structure.

The existing embankments are about 2 to 2.5 m in height and sloped at about 2H:1V to 4H:1V. At the present time, the highway profile at the approaches does not seem to indicate that significant differential settlement of the roadway relative to the bridge has occurred, although the maintenance history at this location is not known due to the age of the existing structure.

The proposed replacement bridge will be a completely new structure and will be located 16.5 m (measured from centreline to centreline) east of the existing bridge. The new structure and the existing bridge will be separated by a distance of about 3 m, from edge of existing structure to edge of new structure. It is understood that the proposed span of the new structure will be about 22.5 m.

The approach embankments for the replacement bridge will be up to about 3 m in maximum height.

At the location of the crossing, Crosby Creek is oriented approximately east-west and flows in a westerly direction. The creek is approximately 15 to 25 m (from west to east) in width at the proposed crossing and the creek bed is at approximately Elevation 121 m. It is understood that the water level during a 100 year storm event has been estimated at Elevation 123.1 m.

The ground surface slopes up from the creek banks in both the north and south directions. On the north side of the creek, a grass covered marshy area extends northward 5 to 10 m from the edge of the creek.



3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out between August 18 and August 26, 2008. During this period, a total of seven boreholes (Boreholes 08-1 to 08-6, including 08-5A) were put down at the locations shown on Drawing 1. The boreholes were advanced using a CME 55 track-mounted drill rig, supplied and operated by Aardvark Drilling Inc. of Guelph, Ontario.

Boreholes 08-1 and 08-2 were advanced within or near the footprint of the proposed north abutment of the Crosby Creek structure, and Boreholes 08-3 and 08-4 were advanced within or near the proposed south abutment footprint. Boreholes 08-5 and 08-6 were put down approximately 20 m behind the proposed foundation elements. Borehole 08-05A was advanced about 2 m south of Borehole 08-5 to obtain additional samples of the silty clay at that location.

The boreholes were advanced to depths which vary from 2.7 to 8.6 m below present ground surface using both hollow stem auger and rotary wash drilling methods. Sampling and in situ testing within the boreholes was performed as follows: (1) SPT tests were carried out at 0.75 m depth intervals and samples of the soils encountered were recovered using a 50 mm outer diameter (O.D) split-spoon sampler; (2) MTO standard N-sized vane testing was carried out in the silty clay to evaluate the undrained shear strength of this soil unit; and (3) four relatively undisturbed, 75-millimetre diameter thin-walled Shelby tube samples of the silty clay were obtained using a fixed piston sampler in general conformance with ASTM D1587. Four of the boreholes (Boreholes 08-1 to 08-4, inclusive) were advanced between 2.3 and 3.2 m into the bedrock using NQ-size coring equipment. The bedrock core obtained was sequentially packed into core boxes.

The boreholes were backfilled with bentonite pellets, mixed with native soils, and the site conditions restored following completion of work.

The field work was supervised on a full-time basis by members of Golder's staff who located the boreholes in the field, 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 laboratories in Ottawa and Mississauga for further examination and laboratory testing. Index and classification tests consisting of water content determinations, Atterberg limits testing and grain size distribution analyses were carried out on selected soil samples. Laboratory oedometer consolidation testing was carried out on two samples of the silty clay deposit. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate.

The borehole locations were selected and positioned by Golder relative to existing site features. The borehole elevations and locations were subsequently determined by Golder Associates personnel. 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 Drawing 1.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
08-1	North abutment	4945129.0	324023.7	122.3
08-2	North abutment	4945130.1	324033.4	122.3



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Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
08-3	South abutment	4945100.4	324028.0	122.6
08-4	South abutment	4945098.5	324037.0	122.5
08-5	North approach embankment	4945148.5	324024.2	122.3
08-5A	North approach embankment	4945146.0	324040. 1	122.3
08-6	South approach embankment	4945072.3	324040.4	122.5



4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The study area for this assignment lies within the Leeds Knobs and Flats physiographic region, as delineated in *The Physiography of Southern Ontario*¹ that lies within the major physiographic region of the Brockville-Gananoque-Kingston areas.

The Leeds Knobs and Flats are characterized by Precambrian rock out-croppings, or knobs, surrounded by clay flats. The clay deposits vary significantly in depth and areal extent depending on the topography of the underlying rock.

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, seven boreholes were advanced within the limits of the foundation elements and the approach embankments for the proposed Crosby Creek Bridge. The borehole locations and ground surface elevations are shown on Drawing 1 in Appendix A.

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in-situ and laboratory testing are given on the Record of Borehole sheets in Appendix B and Figures 1 to 5 in Appendix C. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the soils encountered within the Crosby Creek channel consist of surficial topsoil over an approximately 3 to 4 m thick deposit of silty clay. The upper 0 m to 2 m of this clay deposit has been weathered to a grey-brown crustal zone, while the underlying portions of the deposit are grey in colour. Below a depth of about 3 m to 4 m, the silty clay is underlain by about 0 m to 2 m of silty sand to sandy silt till. The till is, in turn, underlain by granite or gneiss bedrock that was encountered between about 5 and 6 m depth (at about Elevations 118 to 117 m).

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil and Fill Material

Topsoil was encountered extending from the ground surface at all the borehole locations. The topsoil ranges from about 80 to 600 mm in thickness.

4.2.2 Clayey Silt to Clay

The topsoil is underlain by a deposit of clayey silt to clay that ranges between about 2.7 and 3.9 m in thickness. The surface of the silty clay deposit was encountered at depths below existing ground surface ranging from about 0.1 to 0.6 m (i.e., Elevations 122.2 to 121.7 m).

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



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The upper 1.0 to 1.8 m of the clay deposit has been weathered to a grey-brown crust at Boreholes 08-2 to 08-6. The measured SPT "N" values in this portion of the deposit were between 2 and 7 blows per 0.3 m of penetration and these results indicate that the weathered crust has a stiff to very stiff consistency. The measured water content of the weathered crust ranges from approximately 45 to 72%.

The clayey silt to clay below the depth of weathering in Boreholes 08-2 to 08-6 and the full depth of the deposit in Borehole 08-1 is grey in colour. The results of in-situ vane testing carried out in the boreholes are summarized on Figure 1 and indicate measured undrained shear strengths ranging from about 19 to 61 kilopascals. The results of Atterberg limit testing on selected samples of this portion of the deposit indicate plasticity indices ranging from 15 to 78 per cent, and liquid limits ranging from 31 to 106 per cent. These results, which are summarized on the plasticity chart on Figure 2, indicate that this unweathered material is a clayey silt to clay of low to high plasticity. The measured natural water contents of samples of the unweathered, grey clay range from 37 to 109 per cent, generally near or in excess of the liquid limit.

Oedometer consolidation testing was carried out on two thin-walled Shelby tube samples of the clayey silt to clay below the depth of weathering. The results of that testing are provided on Figures 3 and 4 and are summarized in the table below.

Borehole/ Sample No.	Sample Depth/Elev. (m)	Unit Wt. (kN/m ³)	σ_p' (kPa)	σ_{vo}' (kPa)	$\sigma_p' - \sigma_{vo}'$	Cc	Cr	e _o	OCR
08-3	3.30 / 119.3	18	62	24	38	0.59	0.01	1.06	2.6
08-5A	2.48 / 119.8	18	93	31	62	2.13	0.01	1.14	3.0

Notes:

σ_p' - Apparent preconsolidation pressure

σ_{vo}' - Computed existing vertical effective stress

Cc - Compression index

Cr - Recompression index

e_o - Initial void ratio

OCR - Overconsolidation ratio

The oedometer testing indicates that the clay deposit is preconsolidated by about 40 to 60 kPa above the existing overburden pressure.

4.2.3 Silty Sand to Sandy Silt Till

The native clay deposit is underlain by till that was fully penetrated in Boreholes 08-1 to 08-4. The till ranges in thickness from about 1.6 and 1.5 m at the north abutment (i.e., at Boreholes 08-1 and 08-2) and to about 0.3 and 0.1 m at the south abutment (i.e., at Boreholes 08-3 and 08-4).

The till was not fully penetrated at the north and south approach embankments (i.e., at Boreholes 08-5 and 08-6) but was proved to be about 1.5 and 0.1 m in thickness, respectively.



The till is considered to be a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt or silty sand. Only limited standard penetration testing was possible in the till deposit due to the presence of cobbles and boulders. Diamond drilling techniques were required to penetrate the till deposit at Borehole 08-2. The “N” values obtained from the limited testing possible ranged from 30 to greater than 50 blows per 0.3 m indicating a dense relative density. The higher blow counts likely reflect the presence of the cobbles and boulders in the deposit.

The results of a grain size distribution test on one sample of the till are shown on Figure 5. However this sample was retrieved using a 50 mm diameter sampler and therefore does not reflect the cobble and boulder portions of the deposit.

The measured natural water contents of the till ranged from 7 to 10 percent.

4.2.4 Refusal

Practical refusal to augering was encountered at all the boreholes (with the exception of Borehole 08-5A) at depths between about 3.1 and 5.6 m (i.e., Elevations 119.4 to 116.7 m).

4.2.5 Gneiss and Granite Bedrock

Precambrian bedrock underlies the till at this site. The bedrock encountered consists of granitic gneiss and gneiss at the north abutment (i.e., at Boreholes 08-1 and 08-2) and granite at the south abutment (i.e., at Boreholes 08-3 and 08-4). Granite also underlies the granitic gneiss at Borehole 08-1.

The surface of the bedrock was encountered between Elevation 118.0 and 116.7 at the boreholes put down at the foundation elements.

The following table summarizes the bedrock surface depth and elevation as encountered at the borehole locations:

Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
08-1	122.3	5.6	116.7
08-2	122.3	5.3	117.0
08-3	122.6	4.6	118.0
08-4	122.5	4.5	118.0

Granitic gneiss, gneiss and granite are strong Precambrian rocks. The rock is slightly weathered (at Boreholes 08-1 and 08-2) to fresh. Rock Quality Designation (RQD) values measured on recovered bedrock core samples from the upper 0.8 and 0.3 m of bedrock at the north abutment (i.e., boreholes 08-1 and 08-2, respectively) were 23 and 35 percent. The RQD values below these depths at the north abutment and at the south abutment ranged from 48 to 100 percent, generally increasing with depth. The discontinuities observed in the rock core are typically sub-horizontal to sub-vertical. 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 which precedes the Record of Borehole sheets included with this report.



4.2.6 Groundwater Conditions

A piezometer was installed in Borehole 08-2, sealed within the silty clay overburden. The water level measured in that piezometer is summarized in the following table:

Borehole Number	Borehole Location	Date	Depth (m)	Elevation (m)
08-2	North abutment	October 2, 2008	0.4	121.9

The water levels in Boreholes 08-3 and 08-5 were measured at 0.2 (Elevation 122.4) and 2.2 m (Elevation 120.1) depth during the short time the boreholes remained open following the overburden drilling and prior to commencing the bedrock coring operations.

The water elevation in the creek was not measured at the time of drilling but is shown on the Genivar drawing as Elevation 121.4 m. The water levels recorded in the piezometer and open boreholes are generally near the creek water level.

It should be expected that the groundwater levels will fluctuate seasonally.



5.0 CLOSURE

The investigation was carried out using equipment supplied and operated by Aardvark Drilling Inc. The field portions were supervised by Mr. Doug Grylls, Mr. Paul Hulan and Mr. Jim Samotowka under the direction of Mr. William Cavers, P. Eng. The testing was carried out in the Ottawa and Mississauga laboratories of Golder Associates. The report was prepared by Mr. William Cavers, P. Eng. This report was reviewed by Mr. Fintan J. Heffernan, P. Eng., the designated MTO contact for this project.

Yours truly,

GOLDER ASSOCIATES LTD.

William (Bill) Cavers, P. Eng.
Geotechnical Engineer



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



SSK/WC/FJH/cg

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CROSBY CREEK BRIDGE REPLACEMENT

PART B

**FOUNDATION DESIGN REPORT
CROSBY CREEK BRIDGE REPLACEMENT
HIGHWAY 15
CROSBY, ONTARIO
W.P. 479-92-00**



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the design and construction of the proposed replacement of the existing Crosby Creek Bridge structure associated with the rehabilitation of Highway 15 near Crosby, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are only intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed rehabilitation/widening. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. 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.

The proposed new bridge will be located 16.5 m (measured from centreline to centreline) east of the existing bridge and the new and existing structures will be separated by a distance of about 3 m. The replacement bridge has been proposed as a single span structure with integral abutments and the proposed span of the new structure will be about 22.5 m.

Wing walls extending 3 to 5 m back from the abutments have been proposed for the bridge approaches. The approach embankments for the replacement bridge will be up to about 3.1 m in maximum height above existing grade near the proposed abutments.

6.2 Bridge Foundation Options

The following options have been considered for the foundation system for the replacement bridge:

- Shallow foundations supported on the native soils.
- Shallow foundations supported by the granite to gneiss bedrock.
- Caisson or pile supported abutments.

The first option, using shallow foundations supported on the native clay soils, is not considered practical or appropriate for this site since the bearing resistance of these highly plastic soils of limited strength would be insufficient for support of the abutment loads and the settlement of the foundations would be excessive.

Spread footings supported on the underlying native glacial till or bedrock, or on engineered fill supported on the glacial till or bedrock, are not considered a feasible or practical option due to the about 4 to 6 m deep excavations below the adjacent creek water level that would be required.

It is considered that the most feasible and cost-effective options for the widened bridge abutments are foundations supported on piles, founded on the bedrock. This option is consistent with the existing bridge foundation construction.



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 bridge foundation options is presented in Table 1 following the text of this report.

6.3 Steel H-Pile Foundations

Based on the bedrock surface elevation of approximately 117 m to 118 m and the assumed underside of pile cap elevation of 121 m, the pile length will be about 3 m to 4 m.

Piles that are 4 m in length or longer may be required depending on the anticipated lateral movements for the integral abutments; piles at least 5 m in length are more typically required. It will therefore be necessary to extend the piles into ungrouted sockets in the bedrock to achieve the necessary pile length to provide sufficient flexibility for integral abutment purposes.

In addition, considering the limited overburden thickness at this site and the low strength of the silty clay to clay soils, it will likely be necessary to socket the piles an additional depth into the bedrock (i.e., below the socket required to achieve sufficient pile length for structural flexibility as noted above) to resist lateral or seismic forces. The piles should be placed in the socket and the lower portion of the socket (minimum length of 1 m, or longer as necessary to resist lateral and seismic forces) backfilled with tremie concrete.

Therefore, for integral abutments, the bedrock sockets will extend into the rock at least 2 to 3 m and only the lower portion of the socket (i.e., about the lower 1 m) will be backfilled with tremie concrete.

The granite and gneiss bedrock at the site is strong, and would require socket formation using coring and/or churn drilling to advance the hole.

If integral abutments are adopted, a sand-filled corrugated steel pipe (CSP), 0.6 m in diameter and 3 m in length, is typically provided extending below the underside of the pile cap to allow for the pile movements associated with integral abutments. Following tremie-concreting of the lower portion of the pile within the bedrock socket, the remainder of the bedrock socket and the section around the pile up to the base of the CSP should be loosely backfilled with Ontario Provincial Standard Specification (OPSS) 1010 Granular B Type II. The CSP should be loosely backfilled with sand after backfilling of the bedrock socket with concrete and Granular B Type II.

A Non Standard Special Provision for the supply and installation of CSP's should be included in the contract documents and has been included in Appendix D of this report. The grading of the sand backfill in the CSP is given in Appendix D.

6.3.1 Axial Geotechnical Resistance

For an HP 310x110 pile socketed at least 1 m into the bedrock as discussed above, a factored axial geotechnical resistance at Ultimate Limit States (ULS) of 2,000 kN may be used for design. This value represents a structural limitation for the piles rather than a geotechnical limitation. 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.



The limited thickness of overburden above the bedrock surface at this bridge location should not be relied upon for uplift resistance.

Assuming that granular backfill materials are placed around the pile as discussed above, downdrag forces need not be taken into account in the design.

6.3.2 Resistance to Lateral Loads

For piles socketed at least 1 m into bedrock, the ultimate (unfactored) lateral resistance of the gneiss or granite may be taken as the lesser of 30 MPa or the compressive strength of the tremie concrete.

The lateral resistance of the granular backfill material may be estimated using passive earth pressure theory. Coefficients of passive earth pressure (K_p) of 2.8 and 3.5 may be assumed for the sand within the CSP and for the Granular B Type II fill, respectively.

6.3.3 Frost Protection

The pile caps should be provided with a minimum of 1.6 m of soil cover for frost protection.

6.4 Caissons

Caissons founded on or socketed into the limestone bedrock may be used for support of the bridge abutments if semi-integral abutments are considered. Based on the bedrock surface elevation of approximately 117 m to 118 m and the assumed underside of pile cap elevation of 121 m, the caisson length will be about 3 m to 4 m.

In addition, considering the limited overburden thickness at this site and the low strength of the silty clay to clay soils, it will likely be necessary to nominally socket the caissons into the bedrock to resist lateral or seismic forces.

The native marine (Champlain Sea) clay at this site is a sensitive soil. The disturbed clay could “flow” into the auger hole during caisson installation if left unsupported. The use of a liner or casing will be required in order to advance the caissons with minimal loss of ground. Additionally, it will be difficult to clean the bedrock surface, even with the use of liners, unless the liner is socketed into the bedrock; once disturbed, the sensitive clay soils, as well as the sandy till, could flow under the casings, at the interface with the bedrock.

The granite to gneiss bedrock at the site is strong and the sockets will have to be advanced by rock coring or churn drilling.

6.4.1 Axial Geotechnical Resistance

Caissons socketed nominally (less than 1 m) into the bedrock, should be designed based on end-bearing resistance and a factored geotechnical resistance at ULS of 5 MPa should be used. Serviceability Limit States resistances do not apply to caissons founded on or socketed in the granite or gneiss bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.



Construction of the approach embankments may raise the effective stress level in the grey silty clay deposit at depth close to or above its estimated preconsolidation pressure depending on the embankment configuration selected (see Section 6.7). If an embankment option is chosen that will impose loading above the limits noted in Section 6.7.3, the stress increase will lead to some consolidation of the deposit and will result in downdrag forces on caissons supporting the abutments and wing walls. The unfactored downdrag load acting on a single 0.9 m or 1.2 m diameter caisson over its length is estimated to be 350 or 450 kN, respectively. The structural capacity of the caissons must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *CHBDC*.

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 silty clay deposit versus the elastic shortening of the pile, the neutral plane used in those analyses was assumed to be at the underside of the silty clay deposit.

6.4.2 Resistance to Lateral Loads

For caissons socketed at least 1 m into bedrock, the ultimate (unfactored) lateral resistance of the gneiss or granite may be taken as the lesser of 30 MPa or the compressive strength of the concrete.

6.4.3 Frost Protection

6.5 Seismic Site Coefficient and Liquefaction Assessment

For seismic design purposes, the Site Coefficient, *S*, for this site in accordance with Section 4.4.6 of the *CHBDC* may be taken as 1.2, consistent with Soil Profile Type II.

Seismic liquefaction occurs 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:

- Large lateral movements of even gently sloping ground, referred to as "lateral spreading", which could impact on embankment stability;
- Reduced shear resistance (i.e., bearing capacity) of soils which 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. The following conditions are more prone to experiencing seismic liquefaction:

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



The assessment of the potential seismic liquefaction hazard 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 specific to the site. The CRR is primarily related to the relative density of the soil and its gradation.

It is considered that the silty clay and glacial till soils at this site are too fine-grained to be potentially liquefiable.

6.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stem/ballast walls and associated wing walls/retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. 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 OPSS 1010 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 placed and compacted in accordance with SP 105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 and 3121.150.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. 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 1.6 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 new embankment fill materials and the following parameters (unfactored) may be used, assuming the use of Select Subgrade material for the approach embankments:

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



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

	Granular A	Granular B Type II
Soil Unit Weight:	22 kN/m³	21 kN/m³

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.
- Seismic loading will result in increased lateral earth pressures acting on the abutment stem. The stem should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the CHBDC, this site is located in Seismic Performance Zone 3. The site-specific zonal acceleration ratio for the Crosby area is 0.15. Based on experience, for the subsurface conditions at this site, up to 50% amplification could be expected for the ground conditions at this site, resulting in an increase in the design ground surface acceleration to 0.23. The seismic lateral earth pressure coefficients given below have therefore been derived based on a design zonal acceleration ratio of $A = 0.23$.
- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, for structures which do not allow lateral yielding the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 1.5 times the zonal acceleration ratio (i.e., $k_h = 0.35$). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e., $k_h = 0.12$).



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- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. 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}

Loading Condition	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.37	0.30	0.30
Non-yielding wall	0.60	0.45	0.45

- The above K_{AE} values for yielding walls are applicable provided that the calculated wall displacement is more than 250A (mm), where A is the design zonal acceleration ratio of 0.23. This corresponds to displacements of up to approximately 58 mm at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a \gamma d + (K_{AE} - K_a) \gamma (H-d)$$

Where:

- $\sigma_h(d)$ Is the lateral earth pressure at depth, d, (kPa);
- K_a Is the static active earth pressure coefficient;
- K_{AE} Is the seismic active earth pressure coefficient;
- γ Is the unit weight of the backfill soil (kN/m³), as given previously;
- H Is the total height of the wall (m); and,
- d Is the depth below the top of the wall (m).

6.7 Approach Embankment Design and Construction

The approach embankments for the new bridge, as currently proposed, will be constructed with 2H:1V side slopes. The proposed profile indicates that the embankments will be up to about 3.1 m in maximum height above original ground surface. After removal of the topsoil to expose the subgrade, the embankment height will be about 3.7 m *above the stripped subgrade*. The topographic contours also indicate that the creek bed is about 0.5 m lower than the subgrade elevation.

The maximum embankment heights noted above are near the abutment locations where the ground surface is lower than along the remainder of the alignment. The maximum embankment height will be up to about 3.4 m above stripped subgrade (i.e., about 2.8 m above original ground surface) at about 7.5 m behind the abutments.

Based on the borehole results, the embankment widening subgrade soils will consist of clayey silt to clay which are in turn underlain by loose to very dense sand and silty sand to sandy silt till.



The clay soils at this site are highly plastic, soft and compressible. The settlement analyses (discussed in more detail below) indicate that the consolidation settlements due to the approach embankment loading will be excessive and that settlement mitigation measures such as preloading and/or surcharging and/or light-weight fill (i.e., expanded polystyrene) will need to be considered.

The stability analyses (also discussed in more detail below) indicate that the full height embankments proposed at this site will have static factors of safety of more than 1.3, except near the abutment locations where the lower elevation of the creek bed effectively results in a higher embankment with a reduced factor of safety against global instability.

The stability and settlement mitigation measures that may be undertaken at this site are discussed together in Section 6.7.4 since the stability mitigation options will depend on the settlement mitigation considered.

6.7.1 Subgrade Preparation and Embankment Construction

It is recommended that all topsoil and softened/loosened soils be stripped from below the approach embankment areas, to minimize differential settlement between the existing and widened portions of the approach embankments.

Embankment fill should be placed in regular lifts with a loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the material's Standard Proctor maximum dry density.

The final lift prior to placement of the granular subbase and base courses should be compacted to 100 percent of the Standard Proctor maximum dry density in accordance with MTO Special Provision 105S10. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

To reduce surface water erosion on the embankment side slopes, placement of topsoil and seeding or pegged sod is recommended.

6.7.2 Approach Embankment Stability

6.7.2.1 Static Slope Stability

The slope stability analyses for this embankment configuration were carried out using the commercially available program SLOPE/W produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis.

The table below summarizes the soil parameters that have been used in the stability analyses. The undrained shear strengths used in the analyses are based on the corrected undrained shear strength (based on Bjerrum's correction method) from in situ vane testing as well as shear strengths calculated from the oedometer test results based on the formula $s_u = 0.22\sigma'_p$ (in kPa).



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Material	Bulk Unit Weight (kN/m ³)	Effective Angle of Friction	Undrained Shear Strength (kPa)
New Embankment Fill for Widening (range of parameters for earth fill and granular fill)	20 – 22	32° to 35°	-
Weathered Silty Clay	16.3	-	39
Unweathered Grey Silty Clay	16.5	-	14
Glacial Till	22	32°	-
Gneiss and Granite Bedrock	-	Impenetrable	-

The results of the slope stability analyses using these parameters indicate that approach embankments less than 3.4 m in height above stripped subgrade (i.e., about 2.8 m above original ground surface), with side slopes orientated at 2H:1V, will have a factor of safety of greater than 1.3 against deep-seated global instability (See Figure 6 in Appendix D).

The analyses also indicate that approach embankments greater than 3.4 m in height (which occurs within 7.5 m of the abutment) will have a significantly reduced factor of safety against global instability (see Figure 7 in Appendix D) and that slope stability mitigation measures (discussed in further detail in Section 6.7.4) will be required where the embankment heights exceed 3.4 m *above the stripped subgrade* (i.e., about 2.8 m above original ground surface).

At the abutment locations the lower elevation of the creek bed results in a higher embankment (i.e., greater than 3.4 m in height) with a reduced factor of safety and stability mitigation measures will be required.

6.7.2.2 Seismic Slope Stability

The site falls within the Western Quebec Seismic Zone (WQSZ) according to the Geological Survey of Canada. The WQSZ constitutes a large area that extends from Montreal to Témiscaming, and encompasses the Ottawa area. Within the WQSZ zone, recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montreal-Maniwaki axis. Historical seismicity within the WQSZ from 1900 to 2000 includes the 1935 Témiscaming event which had a magnitude (a measure of the intensity of the earthquake) of 6.2 and the 1944 Cornwall-Massena event which had a magnitude of 5.6. In comparison to other seismically active areas in the world (e.g., California, Japan, New Zealand), the frequency of earthquake activity within the WQSZ is significantly lower but there still exists the potential for significant earthquake events to be generated.

The seismic slope stability evaluations were carried out assuming that the design earthquake would correspond to an event with a 10% probability of occurrence in 50 years. This design criteria corresponds to the design earthquake in the CHBDC. Consistent with this criteria, a “firm ground” peak horizontal ground acceleration (PHGA) of 0.15 g (g=acceleration due to gravity) was selected.

The design ground accelerations associated with an earthquake with a 10% probability of occurrence in 50 years result from the cumulative contributions of a variety of earthquake magnitudes occurring at various distances from the site. For the analyses presented herein, a magnitude of 6.2 was used.



In consideration of the potential for high ground accelerations to be generated during the design earthquake at this site, the seismic performance of these slopes was assessed using either a pseudo-static or a “Newmark Type” displacement based approach. In this latter approach, the seismic displacements that the slope could undergo during the design earthquake are estimated, thus providing an improved assessment of the actual performance of the slopes. For this assessment, the methodology outlined by Bray et al. (1998) was used to predict the seismic displacements. To support this displacement based approach it is necessary to obtain the yield acceleration k_y , using a “pseudo-static” seismic stability analysis. For the seismic stability analyses, the seismic loads imposed on a slope are modelled in a simplified manner by applying a horizontal “pseudo-static” force to the soil mass. The “pseudo-static” force, F_s , is calculated as:

$$F_s = k_s \times M$$

Where: k_s Is horizontal seismic coefficient, taken as 0.078; and,
 M Is mass of soil contained within the failure surface.

If the factor of safety obtained using the pseudo-static approach is greater than 1.1, the slope is expected to perform acceptably. The horizontal seismic coefficient which results in a factor of safety of 1.0 is commonly referred to as the yield acceleration, k_y .

The slopes were analyzed using the “pseudo-static” approach and all the slopes analyzed had a pseudostatic factor of safety of less than 1.1. The embankment sections were therefore analyzed using a “Newmark” displacement based approach. The predicted settlements range from 0 to 30 mm indicating an acceptable performance of the embankments during the design earthquake.

6.7.3 Approach Embankment Settlement

Settlement of the approach embankments will occur as a result of compression of the new embankment fill itself as well as consolidation of the clayey soils on which the approaches will be founded.

Provided that the new embankment fill material consists of granular fill, Select Subgrade Material or clean earth fill, the settlement of the embankment fill itself is expected to be less than about 25 mm. The use of granular fill for the new embankment construction would reduce the magnitude of post-construction settlement (likely to less than 10 mm), since the majority of settlement of granular fills will occur during construction.

Some settlement of the embankment subgrade can be expected due to compression of the clay soils (i.e., the weathered clay crust and, in particular, the underlying grey silty clay to clay). The effective stress level in the clay deposits will likely approach and potentially exceed the deposit's preconsolidation pressure. The resulting consolidation settlements therefore correspond to recompression of the clayey deposits and some potential consolidation in the virgin compression range.

The total estimated magnitude of the primary consolidation settlement ranges from about 20 to 100 mm for fill heights greater than 2 m and up to about 3.4 m above stripped subgrade. It is estimated however that most of the primary consolidation settlement should be completed within about six months.

Up to approximately 60 mm of additional secondary compression is anticipated over a twenty-year time span, by which time resurfacing of the highway might be expected.



The secondary compression index was estimated using Mesri's correlation with the primary compression index. The primary compression index at the anticipated stress level was inferred from the oedometer consolidation test results and the secondary compression index was estimated from that value.

Based on the above, the total magnitude of the settlements over a period of time due to primary and secondary compression of the underlying clayey deposits, for fill heights greater than 2 m and up to about 3.4 m, is anticipated to be up to about 75 to 160 mm.

Along the portions of the embankment where less than 2 m of fill will be placed the estimated settlement magnitudes, over a twenty year time period, are expected to be less than 75 mm.

6.7.4 Embankment Settlement/Stability Mitigation Options

The following options could be considered for mitigation of post-construction settlement and stability of the approach embankments:

- **Option 1:** Excavate the silty clay and replace with engineered fill. This mitigation option would improve the factor of safety against instability of the embankments and would eliminate the anticipated consolidation and creep settlement.
- **Option 2a:** Employ lightweight fill (i.e., expanded polystyrene – EPS) in the construction of the approach embankments to reduce the magnitude of primary consolidation settlement and provide greater factors of safety against global instability. The use of EPS fill would reduce the post-construction settlements and would improve the factor of safety against instability.
- **Option 2b:** Employ lightweight or ultra-lightweight slag fills in the construction of the approach embankments to reduce the magnitude of primary consolidation settlement and provide greater factors of safety against global instability. The use of slag fills may reduce the post-construction settlements and would improve the factor of safety against instability.
- **Option 3:** Preload the widened embankment and allow the settlements to occur prior to paving. This option would reduce the primary settlement consolidation magnitudes prior to paving. However, some primary consolidation settlement would still occur after paving. Additionally, this option would not reduce the long term creep settlements or improve the factor of safety against stability.
- **Option 4:** Install wick drains to accelerate the consolidation settlement within the silty clay. This option may be used, with or without surcharge (see Option 5 below), to reduce the post-paving settlements but would not improve the factor of safety against instability at the abutments or for the surcharged embankments.
- **Option 5:** Surcharge the widened embankment to increase the magnitude of settlement during the preload period, prior to paving. This option would significantly reduce the post-paving consolidation and creep settlements. However, the increased height of fill would further reduce the factor of safety against embankment stability and additional mitigation measures such as berms and/or lightweight fill (i.e., EPS) or ground improvement would still be required.
- **Option 6:** Carry out in situ soil improvement below the affected sections of the embankments by using deep soil mixing or rammed aggregate piers.



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Some additional information regarding each of these mitigation options is provided in the sub-sections that follow, and the advantages, disadvantages, relative costs, and risks/consequences are summarized in tabular format in Table 2 following the text of this report.

Option 1, excavating the silty clay and replacing it with engineered fill is likely not feasible at this site. The clay is about 3 to 4 m in thickness, which is within the reach of typical mechanical excavation equipment. But, the costs for subexcavation would likely be high due to the extensive temporary excavation support that would be required adjacent to the existing roadway and along the creek. Additionally, the excavation would need to extend below the groundwater level at this site, and below the water level in the adjacent creek, requiring the use of water tight shoring or cofferdams.

The shoring options are limited since it would likely not be possible to cantilever sheet piling into the glacial till, which is indicated to contain cobbles and boulders and which is also indicated to be relatively thin (i.e., less than 0.3 m) in thickness at the south abutment. Tie-backs into the bedrock would therefore be required. Soldier pile and lagging shoring would similarly require tiebacks, unless the soldier piles were socketed into the rock which is also potentially costly. In addition, soldier piles and lagging is not generally regarded as a suitable option for watertight shoring. Other shoring methods that are suited to wet conditions, such as a concrete secant pile wall or slurry walls, would be very costly and therefore also likely not cost-effective or practical.

As noted in Option 2a, the amount of time-dependent settlement and the associated roadway maintenance may be reduced by employing EPS fill material, with a unit weight of less than 1 kN/m^3 , below the pavement structure. EPS fill could be used in place of conventional earth fill to reduce the applied loading to below the pre-consolidation range and to increase the factor of safety against global instability.

If EPS fill is adopted for construction of the approach embankments a 1.7 m thick layer of EPS along the approach embankments within 30 m of the abutments would reduce the applied load sufficiently to limit the post-paving, primary consolidation settlement to less than 25 mm. At and beyond 30 m from the abutment the EPS thickness could be reduced to 1 m to limit the settlement to less than 50 mm and then eliminated where the embankment height is 2.5 m or less.

For Option 2b, two types of slag fill are available for use:

- Ultra-lightweight slag fill from Hamilton (Litex-143), with a bulk unit weight of about 11.5 kN/m^3 ; and,
- Lightweight slag fill (Superior Slag) from Sault Ste. Marie or from Hamilton (Litex-149), with a bulk unit weight of about 14 kN/m^3 .

Ultra-lightweight (Litex-143) and lightweight slag fill (Superior Slag) could also be used to construct the embankment. However, since it will not be possible to achieve the required reduction in loading with either ultra-lightweight slag fill or lightweight slag fill, especially since these fills should not be installed below the anticipated high-water levels due to environmental concerns, Option 2b would not eliminate future settlement; some ongoing settlement will occur due to consolidation settlement and creep.

Option 3, preloading without a surcharge is also not considered to be feasible at this site since this option would reduce the magnitude of post paving primary consolidation settlements but would not reduce the long term settlement magnitudes due to secondary compression.



Option 4, in combination with either preloading (Option 3) or surcharging (Option 5, as discussed below), would involve installing wick drains beneath the embankment footprint to accelerate the consolidation settlement. Installing wick drains for this relatively small project and for the limited thickness of clay present at this site would likely not be practical or economic. In addition, while the wick drains would accelerate the settlements, the time for primary consolidation to take place at this site without wick drains (about 6 months) would likely not affect the construction schedule significantly enough to justify the additional cost of wick drains.

Analyses indicate that a 1 m high surcharge (Option 5) would result in approximately 140 to 160 mm of primary consolidation settlement within about 6 months. This would limit the post-paving settlement of the highway to less than 25 mm. However, where the height of fill (including surcharge fill) would exceed 3.4 m above the stripped subgrade level the factor of safety against instability would be significantly less than 1.3 (see Figure 8 in Appendix D) and temporary berms would be required to improve the factor of safety to an acceptable level (see Figure 9 in Appendix D). Additionally, at the abutment locations the lower elevation of the creek bed results in a higher embankment with a reduced factor of safety. Since berms would not be feasible in front of the abutments (i.e., within the creek bed) expanded polystyrene (EPS) would be required extending back about 7.5 m from the abutment locations.

Option 6, involving in situ soil improvement, is considered a feasible alternative to Option 5, although such techniques are not commonly used on MTO projects and may be uneconomical for this relatively small project.

Based on the above, it is therefore considered that Option 5 (surcharging with berms and EPS at the abutments) is the most technically feasible, practicable and cost effective option.

Further details for Options 5 and 6 are provided below.

6.7.4.1 Option 5 – Preloading and Surcharging

At the abutment locations the lower elevation of the creek bed results in a higher embankment with a reduced factor of safety. The embankments at the abutment locations may be constructed to a maximum elevation of 123.9 m. Any additional fill required to achieve the full embankment height should consist of EPS and should extend back from the abutments at least 7.5 m. Surcharging near the abutment locations (i.e., within about 7.5 m of the abutments) will therefore not be feasible.

Following construction of the approach embankments, outside the limits noted above, a surcharge could be placed to increase the settlement magnitude and potentially reduce the preload time. The feasibility of placing a surcharge will have to be evaluated relative to the available right-of-way space and any restrictions on allowable impacts to fisheries and wetlands.

Preliminary analyses indicate that a 1 m high surcharge would result in approximately 140 to 160 mm of primary consolidation settlement within about 6 months. This would limit the post-paving settlement to approximately less than 25 mm.

Stability analyses indicate that the resulting 4.4 m high embankments above stripped subgrade (i.e., 3.8 m above original ground surface), with 2H:1V side slopes, at this site will not have an adequate factor of safety against global instability. Temporary berms will therefore be required on the east side of the approach embankments; the existing roadway on the west side of the proposed embankments would provide a stabilizing influence on the new embankments. The berms will be required where the height of fill *above subgrade level*



exceeds 3.4 m (i.e., 2.8 m above original ground surface). The berms should be 2.2 m in height and should extend laterally 6.5 m from the main embankment to the crest of the berm. The berms should be provided with side slopes at maximum inclinations of 2H:1V.

Ideally, EPS fill should be maintained above the water level at the site to avoid flotation. EPS fill, if adopted, would require a minimum of 1.3 m of conventional fill/pavement structure cover on top of the embankment and side slopes in order to minimize icing potential on the road surface and therefore the maximum elevation of the base of the EPS fill would be about 122.6 m, which is below the indicated high water level of 123.1 m. However, the EPS fill would be ballasted by the fill/pavement structure and this ballast should be sufficient to maintain the EPS fill in place during high water events.

In addition, the EPS would have to be encapsulated within polyethylene sheeting or other material as recommended by the manufacturer/supplier to protect against degradation from exposure to hydrocarbons, and it should be covered by a concrete slab to improve the long-term performance under traffic loading.

6.7.4.2 Option 6 – Soil Improvement

As an alternative to Option 5, and to conventional subexcavation of the silty clay deposit below the approach embankments, the use of deep soil mixing or rammed aggregate piers could be considered to improve the performance of the surficial organic deposit.

The mobilization costs associated with the deep soil mixing equipment would be high; it would likely not be practical to mobilize equipment to this site for the relatively limited improvement works required for this approximately 100 m length of embankment, and so a deep soil mixing option for subsoil improvement is not considered economical.

Rammed aggregate piers, which can be installed to a maximum of about 7.5 m depth, could be feasible for improvement below the approach embankments based on the height of the embankment and thickness of the silty clay deposit. The rammed aggregate piers would likely be installed in an array under the full width and length of the affected embankment areas (i.e., where the height of embankment fill would exceed 2.5 m).

Rammed aggregate piers could be competitive with conventional subexcavation, since it would not be necessary to excavate below the water table or provide temporary excavation support. In addition, rammed aggregate piers could be extended to about 4 m depth (Elevation 118 m), which would extend below the soft to firm grey silty clay to clay deposit. The post-construction consolidation settlement of the underlying grey silty clay to clay should be less than 25 mm and EPS fill would not likely be required at the abutment locations for settlement or stability mitigation.

However, rammed aggregate piers may not be competitive with temporary berms, preloading and surcharging and some use of EPS fill at the abutments. The mobilization and installation costs would be relatively high for a project of this small size. Temporary berms are relatively cost effective, provided that sufficient area is available for their placement, and the cost of the limited amount of EPS fill (i.e., 7.5 m at each abutment) that would be eliminated on this project may not be sufficient to offset the costs for rammed aggregate piers.



In addition, cost advantages would likely be lost due to the relatively small size of this project. Although their use is not uncommon in the United States, rammed aggregate piers have also not been used to date on an MTO project and there is potential for impacts to the schedule. Further design would be required if this option is pursued.

6.8 Construction Considerations

6.8.1 Excavations and Temporary Excavation Support

If it is anticipated that excavations for sub-excavation will be required, these excavations will be advanced through the weathered silty clay and the underlying soft to firm silty clay. Where space permits, open-cut excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The soft to firm silty clay at the site is classified as Type 4 soils, according to the OHSA. Side slopes no steeper than 3 horizontal to 1 vertical (3H:1V), or flatter, may be required through these materials. If excavations below the creek water level, at the time of construction, are planned, appropriate dewatering or groundwater control measures will be required.

It is expected that temporary roadway protection will be required, if subexcavation of the soft to firm silty clay is considered, along the east side of Highway 15. Temporary excavation support would also be required along Crosby Creek. These temporary excavation support systems should be designed and constructed in accordance with MTO's Special Provision SP105S19. The lateral movement of temporary shoring systems should meet Performance Level 2 as specified in SP 105S19.

Based on the subsurface conditions at the site and the likely excavation geometry, roadway protection (i.e., shoring) could consist of soldier piles and lagging or interlocking steel sheet piling. Some form of lateral restraint such as internal bracing or rakers, or rock anchor tie-backs would be required.

6.8.2 Groundwater and Surface Water Control

The groundwater level at the site was measured at Elevation 121.9 m in the piezometer installed in Borehole 08-2 which is generally consistent with the creek water level at the time of the investigation.

Excavations below about Elevation 121.9 m at the abutment areas, though not anticipated at this time, would extend below the groundwater level and below the water level in Crosby Creek. Significant surface and groundwater inflow into the excavations may be expected. It is anticipated that watertight shoring would be required to reduce the potential inflows.

6.8.3 Settlement Monitoring

Although estimates of the settlements and the needed duration of the preload/surcharge have been provided, if that option is chosen, the actual duration of the preload will need to be determined by monitoring of the settlements.

A monitoring program will therefore need to be implemented which includes monitoring of the embankment settlements using settlement plates on the embankment subgrade, monitored via rods extended up through the embankment fill and settlement points installed at the surface of the embankment fill.



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It is proposed that the settlement plates be installed at approximately 50 m intervals along the preload/surcharge area and that the settlement points be installed at 25 m intervals. That spacing is proposed based on the variation in the embankment height, clay thickness, and expected settlements. It is further proposed that the settlements be monitored beneath the embankment centerline.

It is generally best if the supply and installation of the instrumentation is the responsibility of the contractor. This arrangement avoids conflicts with the contractor over delays and interference with the equipment needed for the installation.

The actual monitoring is best carried out by the MTO's Contract Administrator and geotechnical consultant.

The monitoring instrumentation should be monitored with the following frequency:

Construction Stage	Frequency (over given time)
Baseline reading	3 readings on 3 consecutive days Note: Baseline reading shall not commence until a minimum period of 7 days has elapsed since completion of installation
Just prior to start of embankment construction	Once
During embankment construction	Once every 1.0 m fill lift within 20 m of the monitoring section
After end of embankment construction	Daily for 2 weeks Bi-weekly for 2 months Monthly for duration of pre-loading



7.0 CLOSURE

This report was prepared by Mr. William Cavers, P. Eng. This report was reviewed by Mr. Fintan J. Heffernan P. Eng, the designated MTO contact for this project.

Yours truly,

GOLDER ASSOCIATES LTD.

William (Bill) Cavers P. Eng.
Geotechnical Engineer



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



SSK/WC/FJH/cg

n:\active\2007\1111\07-1111-0042 transenco hwy 15 thousand islands\07-1111-0042 crosby creek bridge rpt-001 jan 2010.doc



CROSBY CREEK BRIDGE REPLACEMENT

TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES, HIGHWAY 15
CROSBY CREEK BRIDGE REPLACEMENT, W.P. 479-92-00

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread footings supported on native silty clay soil	<ul style="list-style-type: none"> Not feasible. 	<ul style="list-style-type: none"> N/A. 	<ul style="list-style-type: none"> N/A. 	<ul style="list-style-type: none"> N/A. 	<ul style="list-style-type: none"> N/A.
Spread footings supported on bedrock	<ul style="list-style-type: none"> Not feasible. 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A.
Steel H-pile foundations socketed into bedrock	<ul style="list-style-type: none"> Feasible for support of all foundation elements. 	<ul style="list-style-type: none"> High bearing resistance. Negligible settlement. Allows for integral or semi-integral abutments. 	<ul style="list-style-type: none"> Piles may have to be socketed into strong bedrock, which would require coring or churn drilling. Possibility of encountering cobbles or boulders in the glacial till during installation. If sockets required, temporary liner necessary. 	<ul style="list-style-type: none"> May be less expensive than caisson option. 	<ul style="list-style-type: none"> May not be able to dewater socket for cleaning and inspection.
Caissons socketed into bedrock	<ul style="list-style-type: none"> Feasible for support of all foundation elements. 	<ul style="list-style-type: none"> High bearing resistance. Negligible settlement. Allows for semi-integral abutments. 	<ul style="list-style-type: none"> Liners required to minimize disturbance to surrounding soils. Possibility of encountering cobbles or boulders in the glacial till during installation. Socketing of liner may be required to permit cleaning and inspection. Coring or churn drilling will be required to form rock socket in medium strong bedrock. Not consistent with integral abutments. 	<ul style="list-style-type: none"> May be more expensive than steel H-pile option, particularly if rock sockets are necessary, due to larger socket diameter. 	<ul style="list-style-type: none"> May not be able to dewater socket for cleaning and inspection.



CROSBY CREEK BRIDGE REPLACEMENT

TABLE 2
COMPARISON OF SETTLEMENT/STABILITY MITIGATION OPTIONS, HIGHWAY 15
CROSBY CREEK BRIDGE REPLACEMENT, W.P. 479-92-00

Mitigation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Lightweight fill	<ul style="list-style-type: none"> Feasible. 	<ul style="list-style-type: none"> Reduces the post-construction settlements. 	<ul style="list-style-type: none"> Lightweight slag fills will not sufficiently reduce the loading on the underlying silty clay. Lightweight slag fills may not be used below the groundwater or creek level. 	<ul style="list-style-type: none"> Likely the most costly option. 	<ul style="list-style-type: none"> EPS may de-stabilise embankment by floatation if very high water flows in creek occur.
Preload the embankment	<ul style="list-style-type: none"> Feasible. 	<ul style="list-style-type: none"> Would reduce the post-construction settlement magnitudes. 	<ul style="list-style-type: none"> Would not improve the factor of safety against global instability. Long-term settlements (post-construction) would still exceed 60 mm. EPS required behind abutments. 	<ul style="list-style-type: none"> Least costly. 	<ul style="list-style-type: none"> Long term (post-construction) settlement magnitudes higher than acceptable.
Wick drains (with or without surcharging)	<ul style="list-style-type: none"> Feasible. 	<ul style="list-style-type: none"> Would accelerate time to achieve primary consolidation settlement. 	<ul style="list-style-type: none"> Anticipated time to achieve primary consolidation (3 to 6 months) without wick drains is not long enough to justify their use. 	<ul style="list-style-type: none"> Not cost effective for time gain. 	<ul style="list-style-type: none"> Little practical gain in preloading time.
Surcharging	<ul style="list-style-type: none"> Feasible. 	<ul style="list-style-type: none"> Would reduce post-construction settlement to acceptable magnitudes. 	<ul style="list-style-type: none"> Would not improve the factor of safety against global instability. Berms required where embankments exceed 3.4 m in height. EPS required behind abutments. 	<ul style="list-style-type: none"> Slightly more costly than preloading. 	<ul style="list-style-type: none"> EPS may de-stabilise embankment by floatation if very high water flows in creek occur.
Ground improvement	<ul style="list-style-type: none"> Feasible. 	<ul style="list-style-type: none"> Improves the factor of safety against global instability. Reduces the post-construction settlements to less than 25 mm. 	<ul style="list-style-type: none"> Uncommon on MTO projects. May be difficult to mobilise contractor for a relatively small project. 	<ul style="list-style-type: none"> Potentially more costly than preloading or surcharging. 	<ul style="list-style-type: none"> Higher risk of construction difficulties with uncommon method.

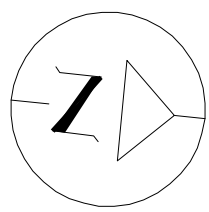


APPENDIX A

Drawing 1 - Borehole Locations and Soil Strata

Drawing 2 - Borehole Locations and Soil Strata

CONT No. -
WP No. 479-92-00

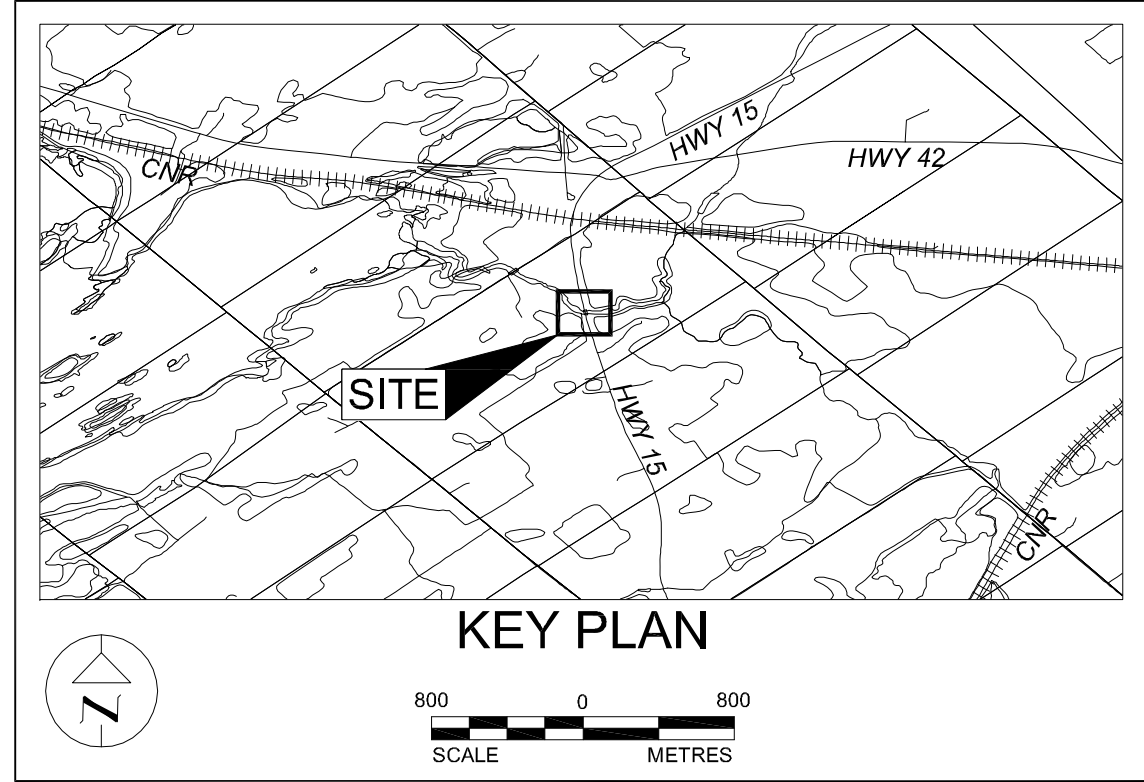


HIGHWAY 15
PROPOSED CROSBY CREEK BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
1



Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



LEGEND	
	Borehole – Current Golder Associates Ltd. Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
	WL in piezometer
	WL upon completion of drilling
	Location of cross-section
	Seal
	Piezometer

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
08-1	122.3	4945129.0	324023.7
08-2	122.3	4945130.1	324033.4
08-3	122.6	4945100.4	324028.0
08-4	122.5	4945098.5	324037.0
08-5	122.3	4945148.5	324024.2
08-5A	122.3	4945146.0	324024.7
08-6	122.5	4945072.3	324040.4

REFERENCE

Base plan supplied by the Genivar Consulting Group

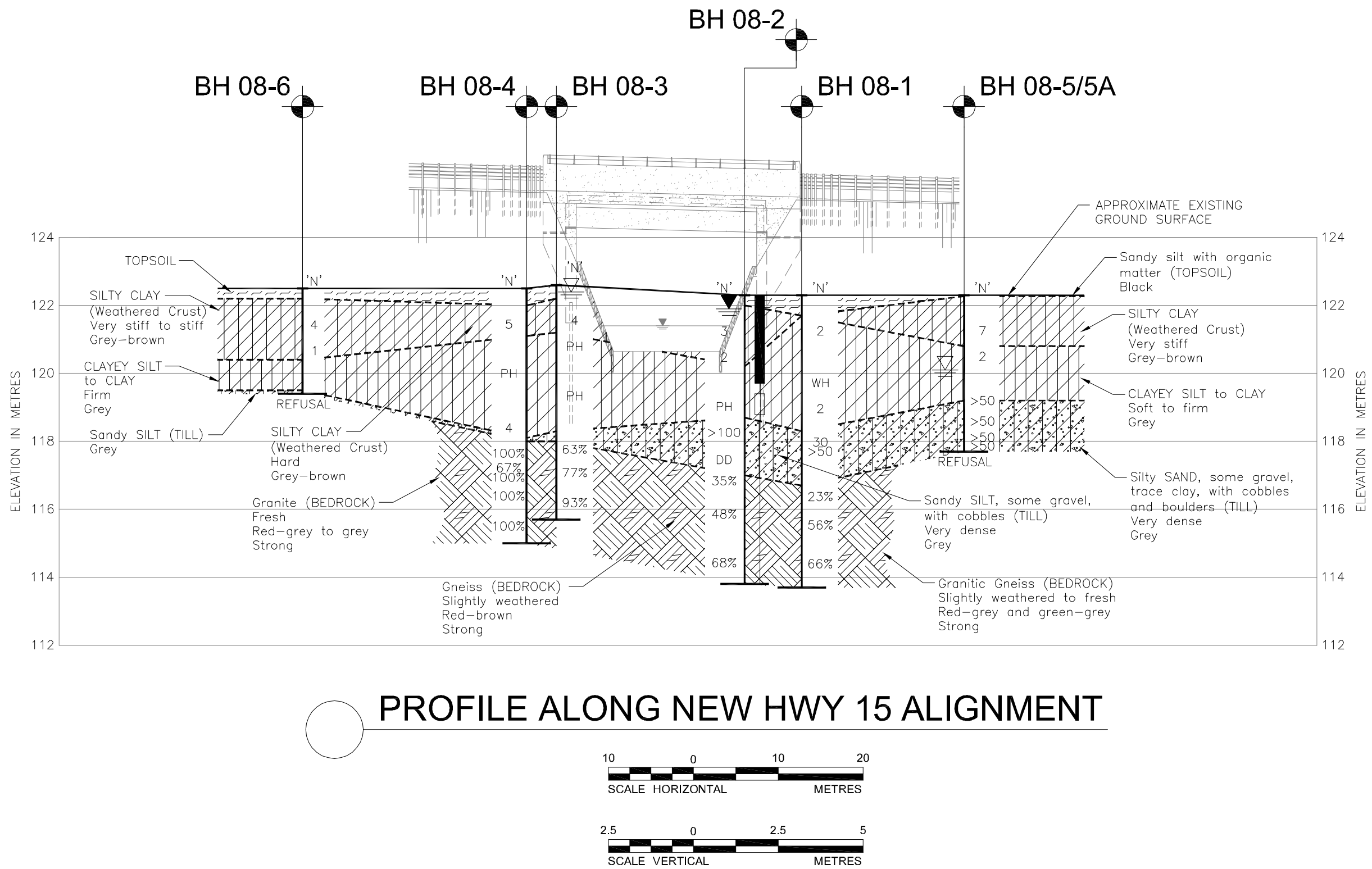
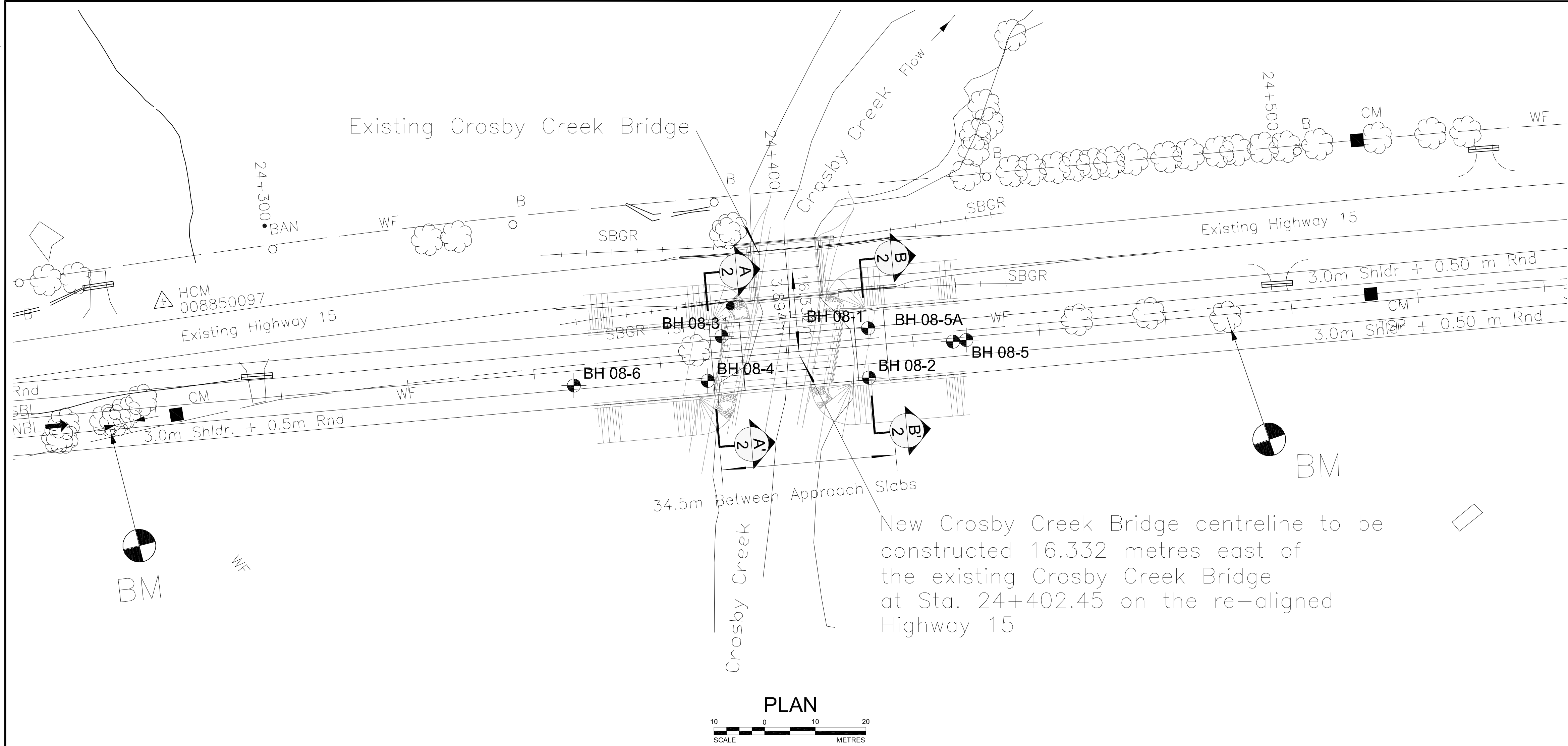
NOTES

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.

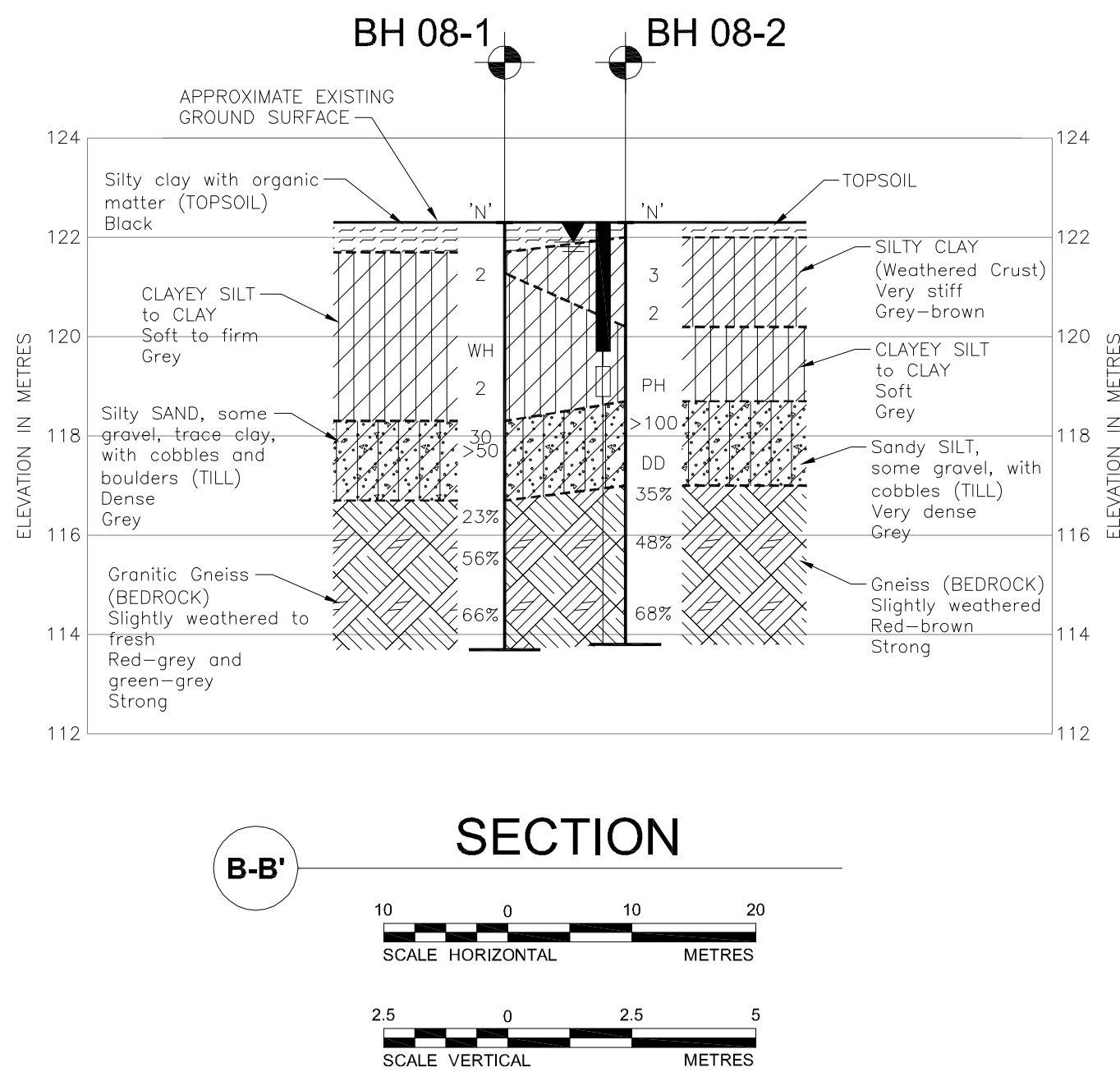
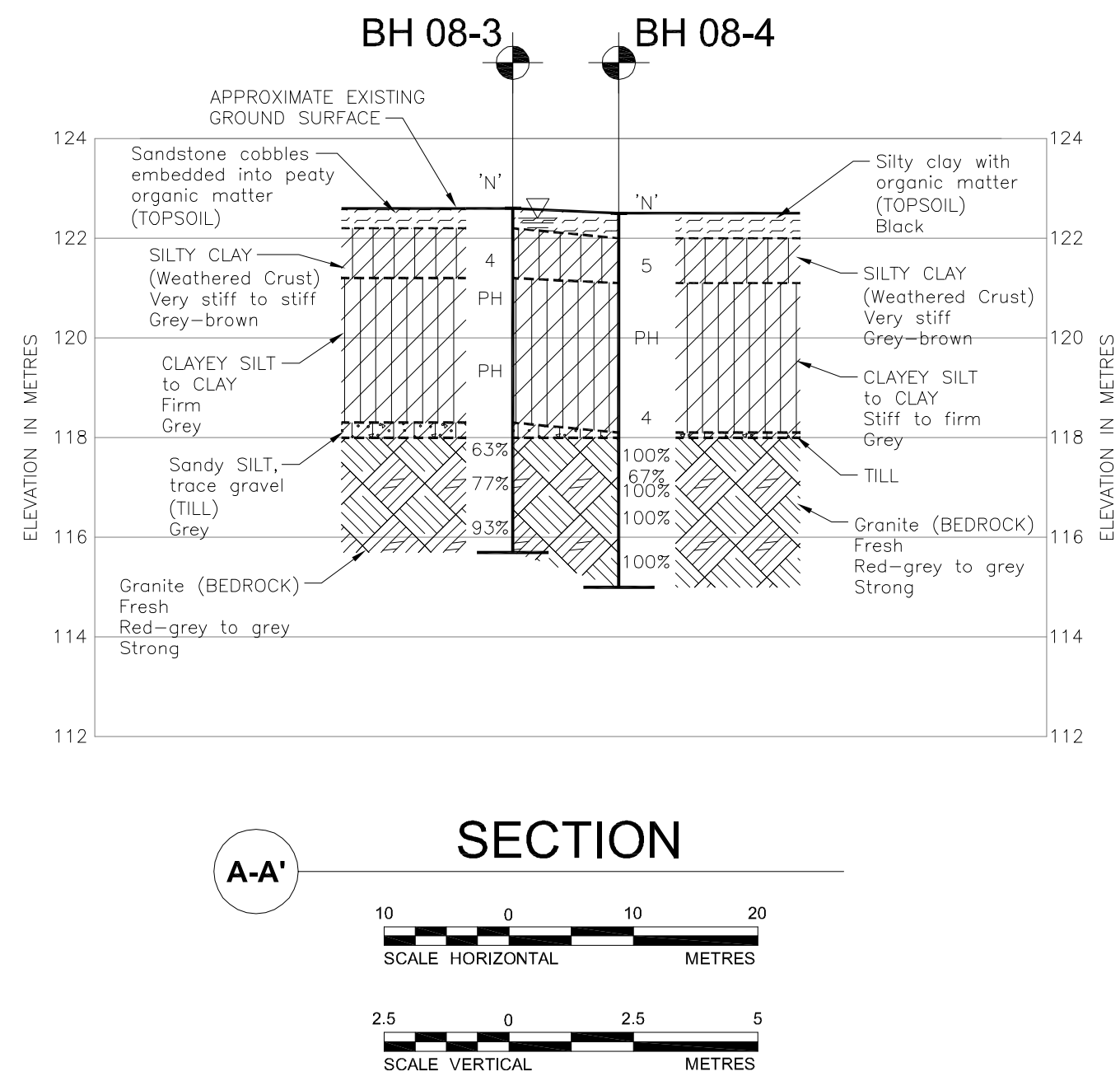
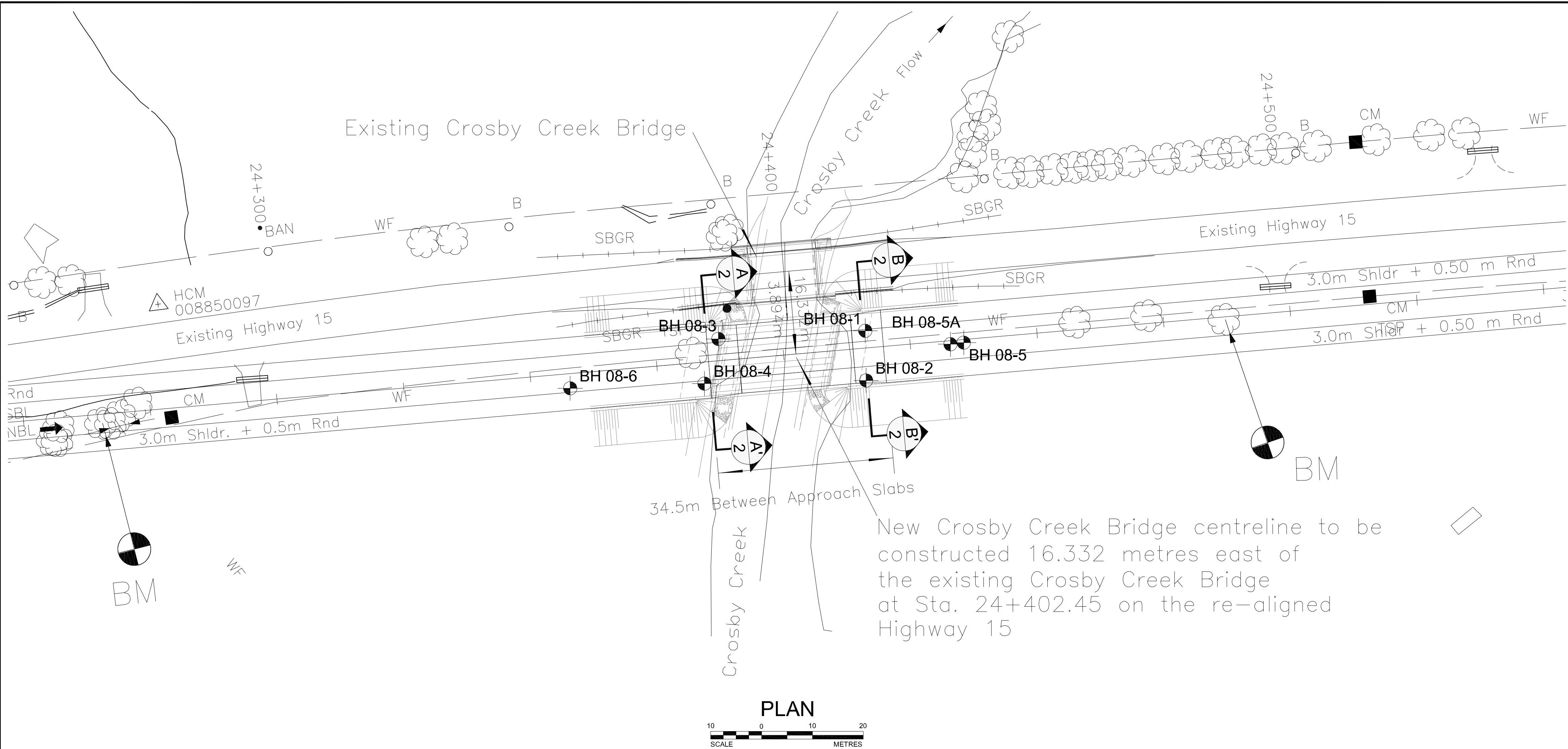
This drawing is for subsurface information only. The proposed details are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

NO.	DATE	BY	REVISION	
Geocres No. —				
HWY. 15			PROJECT NO.07-1111-0042	DIST.
SUBM'D. S.K.		CHKD. W.C.	DATE: JAN. 2009	SITE:
DRAWN: J.M.		CHKD. F.J.H.	APPD.	DWG. 1



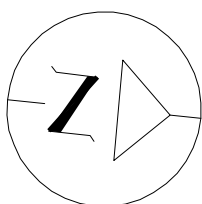
METRIC

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



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

CONT No. -
WP No. 479-92-00

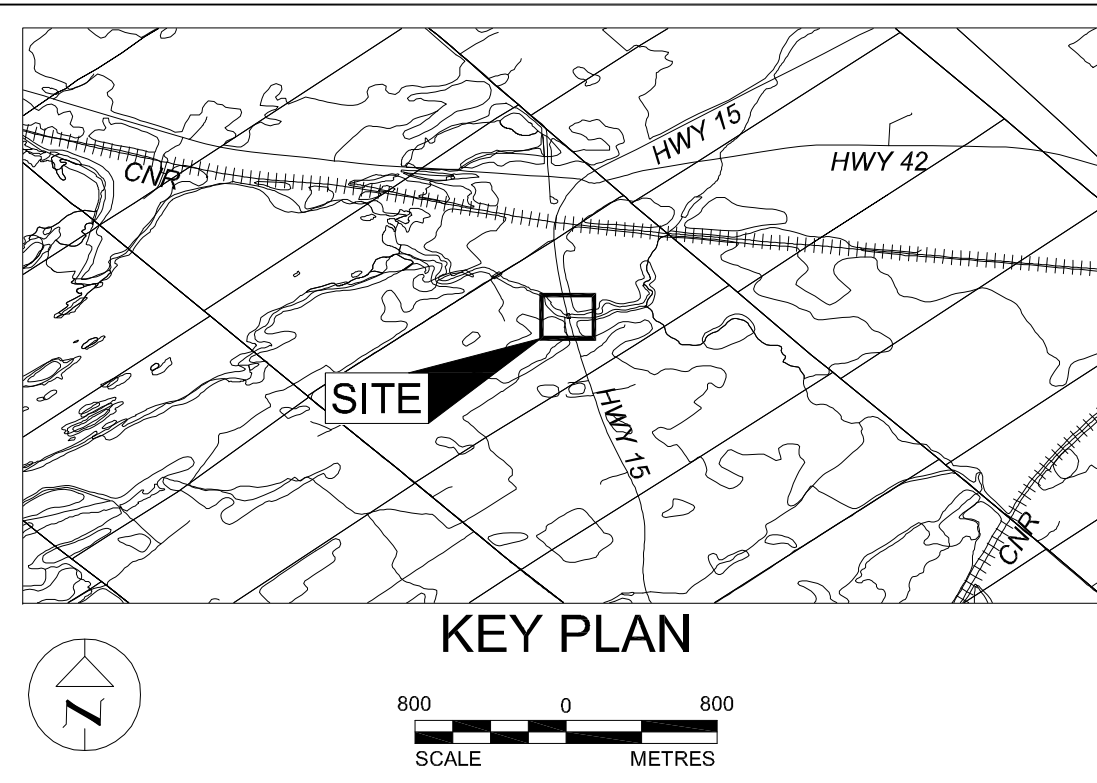


HIGHWAY 15
PROPOSED CROSBY CREEK BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
1



Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



LEGEND

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- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer
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NO.	DATE	BY	REVISION	
Geocres No. —				
HWY. 15		PROJECT NO.07-1111-0042		DIST.
SUBM'D. S.K.	CHKD. W.C.	DATE: JAN. 2009		SITE:
DRAWN: J.M.	CHKD. F.J.H.	APPD.		DWG. 2



APPENDIX B

List of Abbreviations and Symbols

List of Lithological and Geotechnical Rock Description Terminology

Record of Boreholes 08-1 to 08-6

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE		III. SOIL DESCRIPTION	
AS	Auger sample	(a)	Cohesionless Soils
BS	Block sample		
CS	Chunk sample	Density Index	N
DO	Drive open	(Relative Density)	Blows/300 mm
DS	Denison type sample		Or Blows/ft.
FS	Foil sample	Very loose	0 to 4
RC	Rock core	Loose	4 to 10
SC	Soil core	Compact	10 to 30
ST	Slotted tube	Dense	30 to 50
TO	Thin-walled, open	Very dense	over 50
TP	Thin-walled, piston		
WS	Wash sample	(b)	Cohesive Soils
DT	Dual Tube sample	Consistency	C _u or S _u
II. PENETRATION RESISTANCE			
Standard Penetration Resistance (SPT), N:			
The number of blows by a 63.5 kg. (140 lb.)			
hammer dropped 760 mm (30 in.) required			
to drive a 50 mm (2 in.) drive open			
Sampler for a distance of 300 mm (12 in.)			
DD- Diamond Drilling			
Dynamic Penetration Resistance; N_d:			
The number of blows by a 63.5 kg (140 lb.)			
hammer dropped 760 mm (30 in.) to drive			
Uncased a 50 mm (2 in.) diameter, 60° cone			
attached to "A" size drill rods for a distance			
of 300 mm (12 in.).			
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		
Peizo-Cone Penetration Test (CPT):			
An electronic cone penetrometer with			
a 60° conical tip and a projected 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.			
		IV. SOIL TESTS	
		w	water content
		w _p	plastic limited
		w _l	liquid limit
		C	consolidaiton (oedometer) test
		CHEM	chemical analysis (refer to text)
		CID	consolidated isotropically drained triaxial test ¹
		CIU	consolidated isotropically undrained triaxial test
			with porewater pressure measurement ¹
		D _R	relative density (specific gravity, G _s)
		DS	direct shear test
		M	sieve analysis for particle size
		MH	combined sieve and hydrometer (H) analysis
		MPC	modified Proctor compaction test
		SPC	standard Proctor compaction test
		OC	organic content test
		SO ₄	concentration of water-soluble sulphates
		UC	unconfined compression test
		UU	unconsolidated undrained triaxial test
		V	field vane test (LV-laboratory vane test)
		γ	unit weight

Note:

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

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is p. Unit weight symbol is γ where $\gamma = pg$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (cont'd.)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes: 1. $\tau = c' + \sigma' \tan \phi'$

2. Shear strength = (Compressive strength)/2

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

<u>Description</u>	<u>Bedding Plane Spacing</u>
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

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

GRAIN SIZE

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

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

O:\ Templates\Rock Description Terminology

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 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	Ca-	Calcite
FO-	Foliation/Schistosity	P-	Polished
CL -	Cleavage	S-	Slickensided
SH -	Shear Plane/Zone	SM-	Smooth
VN-	Vein	R-	Ridged/Rough
F -	Fault	ST-	Stepped
CO-	Contact	PL-	Planar
J -	Joint	FL-	Flexured
FR-	Fracture	UE-	Uneven
MF -	Mechanical	W-	Wavy
A-	Angular	C-	Curved
BP-	Bedding Plane	H-	Hackly
BL-	Blast Induced	SL-	Sludge Coated
	Parallel To	TCA-	To Core Axis
	Perpendicular To	STR-	Stress Induced

PROJECT		07-1111-0042		RECORD OF BOREHOLE No 08-1		1 OF 1 METRIC														
W.P.		479-92-00		LOCATION		N 4945129.0; E 324023.7														
DIST		HWY 15		BOREHOLE TYPE		Power Auger 108mm I.D. Hollow Stem														
DATUM				DATE		Aug. 18, 2008														
				ORIGINATED BY		D.G./D.J.S.														
				COMPILED BY		J.M.														
				CHECKED BY		W.C.														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa			WATER CONTENT (%)			γ			GR SA SI CL		
122.3	0.0	GROUND SURFACE							○ UNCONFINED + FIELD VANE			W _p W W _L			kN/m ³					
		Silty clay with organic matter (TOPSOIL)							● QUICK TRIAXIAL × REMOULDED			30 60 90								
121.7	0.6	CLAYEY SILT to CLAY																		
		Soft to firm		1	SS	2														
		Grey																		
		Moist to wet																		
				2	SS	WH														
				3	SS	2														
118.3	4.0	Silty SAND, some gravel, trace clay, with cobbles and boulders (TILL)		4	SS	30														
		Dense		5	SS	50														
		Grey																		
		Wet																		
116.7	5.6	Granitic Gneiss (BEDROCK)		6	NQ RC	REC 100%														
		Slightly weathered to fresh		7	NQ RC	REC 100%														
		Red-grey and green-grey		8	NQ RC	REC 100%														
		Strong																		
114.0	8.6	Granite (BEDROCK)																		
		Fresh																		
		Red-grey																		
		Strong																		
		Bedrock cored between 5.6 m and 8.6 m depth.																		
		For bedrock coring details refer to Record of Drillhole 08-1.																		
		End of Borehole																		

MIS-MTO 001 07-1111-0042 GPJ GAL-MISS GDT 12/30/09

PROJECT: 07-1111-0042

RECORD OF DRILLHOLE: 08-1

SHEET 1 OF 1

LOCATION: N 4945129.0; E 324023.7

DRILLING DATE: Aug. 18, 2008

DATUM:

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Aardvark

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH % RETURN	FR/FX-FRACTURE F-FAULT												SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
								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													
								RECOVERY		R.Q.D. %		FRACT INDEX PER 0.3		DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec																	
								TOTAL CORE %		SOLID CORE %						DIP w.r.t. CORE AXIS		TYPE AND SURFACE DESCRIPTION		10 ⁻²		10 ⁻¹		10 ⁰									
								00000		00000		00000		00000		00000				10 ⁻²		10 ⁻¹		10 ⁰									
		Continued from Record of Borehole 08-1		116.70																													
6		Granitic Gneiss (BEDROCK) Slightly weathered to fresh Red-grey and green-grey Strong		5.60		1																											
7		Rotary Drill NQ Core				2																											
8						3																											
		Granite (BEDROCK) Fresh Red-grey Strong End of Drillhole		114.00 8.30 113.70 8.60																													
9																																	
10																																	
11																																	
12																																	
13																																	
14																																	
15																																	
16																																	
17																																	
18																																	
19																																	
20																																	

DEPTH SCALE

1 : 75



LOGGED: D.G./D.J.S.

CHECKED: W.C.

MIS-RCK 001 07-1111-0042 (ROCK) GPJ GAL-MISS GDT 12/30/09 JM

PROJECT		RECORD OF BOREHOLE		No 08-2		1 OF 1		METRIC					
W.P.		LOCATION		N 4945130.1; E 324033.4		ORIGINATED BY		D.J.S.					
DIST		HWY		BOREHOLE TYPE		Power Auger 108mm I.D. Hollow Stern		COMPILED BY					
J.M.		DATE		Aug. 19, 2008		CHECKED BY		W.C.					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	GR SA SI CL
122.3	GROUND SURFACE												
122.0	TOPSOIL												
0.3	SILTY CLAY (Weathered Crust) Very stiff Grey-brown Wet		1	SS	3		122						
			2	SS	2		121						
120.2	CLAYEY SILT to CLAY Soft Grey Wet						120	X +					
2.1							119	X +					
118.7	Sandy SILT, some gravel, with cobbles (TILL) Very dense Grey Wet		3	TP	PH		118						
3.6			4	SS	>100		117						
			5	NQ RC	REC 33%		116						
117.0	Gneiss (BEDROCK) Slightly weathered Red-brown Strong		6	NQ RC	REC 100%		115						
5.3	Bedrock cored between 5.3 m and 8.5 m depth. For bedrock coring details refer to Record of Drillhole 08-2.		7	NQ RC	REC 100%		114						
			8	NQ RC	REC 100%								
113.8	End of Borehole												
8.5	Note: Water level in piezometer at 0.4 m depth (Elev. 121.9 m) on Oct. 2, 2008												

PROJECT: 07-1111-0042

RECORD OF DRILLHOLE: 08-2

SHEET 1 OF 1

LOCATION: N 4945130.1; 324033.4

DRILLING DATE: Aug. 19, 2008

DATUM:

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Aardvark

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm)	FLUSH % RETURN	FR/FX-FRACTURE F-FAULT CL-CLEAVAGE J-JOINT SH-SHEAR P-POLISHED VN-VEIN S-SLICKENSIDED PL-PLANAR	SM-SMOOTH R-ROUGH ST-STEPPED C-CURVED	FL-FLEXURED UE-UNEVEN W-WAVY	BC-BROKEN CORE MB-MECH. BREAK B-BEDDING	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole 08-2		117.00									
6	Rotary Drill NQ Core	Gneiss (BEDROCK) Slightly weathered Red-brown Strong		5.30	1								
7					2								
8					3								
		End of Drillhole		113.80 8.50									
9													
10													
11													
12													
13													
14													
15													
16													
17													
18													
19													
20													

DEPTH SCALE

1 : 75



LOGGED: D.J.S.

CHECKED: W.C.

MIS-RCK 001 07-1111-0042 (ROCK) GPJ GAL-MISS GDT 12/30/09 JIM

[illegible]

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

PROJECT: 07-1111-0042

RECORD OF DRILLHOLE: 08-3

SHEET 1 OF 1

LOCATION: N 4945100.4; 324028.0

DRILLING DATE: Aug, 26, 2008

DATUM:

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Aardvark

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLLOID % RETURN	FR/FX-FRACTURE F-FAULT				SM-SMOOTH		FL-FLEXURED		BC-BROKEN CORE		DIAMETER POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
									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					
RECOVERY		FRACT. INDEX PER 0.3		DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY														
TOTAL CORE %	SOLID CORE %	R.Q.D. %	TYPE AND SURFACE DESCRIPTION		K, cm/sec															
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁹	10 ⁻⁵														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁶	10 ⁻⁴														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁷	10 ⁻³														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁸	10 ⁻²														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁹	10 ⁻¹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁰	10 ⁰														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹¹	10 ¹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹²	10 ²														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹³	10 ³														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁴	10 ⁴														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁵	10 ⁵														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁶	10 ⁶														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁷	10 ⁷														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁸	10 ⁸														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁹	10 ⁹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻²⁰	10 ¹⁰														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻²¹	10 ¹¹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻²²	10 ¹²														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻²³	10 ¹³														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻²⁴	10 ¹⁴														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻²⁵	10 ¹⁵														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻²⁶	10 ¹⁶														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻²⁷	10 ¹⁷														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻²⁸	10 ¹⁸														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻²⁹	10 ¹⁹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻³⁰	10 ²⁰														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻³¹	10 ²¹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻³²	10 ²²														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻³³	10 ²³														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻³⁴	10 ²⁴														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻³⁵	10 ²⁵														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻³⁶	10 ²⁶														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻³⁷	10 ²⁷														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻³⁸	10 ²⁸														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻³⁹	10 ²⁹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁴⁰	10 ³⁰														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁴¹	10 ³¹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁴²	10 ³²														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁴³	10 ³³														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁴⁴	10 ³⁴														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁴⁵	10 ³⁵														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁴⁶	10 ³⁶														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁴⁷	10 ³⁷														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁴⁸	10 ³⁸														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁴⁹	10 ³⁹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁵⁰	10 ⁴⁰														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁵¹	10 ⁴¹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁵²	10 ⁴²														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁵³	10 ⁴³														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁵⁴	10 ⁴⁴														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁵⁵	10 ⁴⁵														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁵⁶	10 ⁴⁶														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁵⁷	10 ⁴⁷														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁵⁸	10 ⁴⁸														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁵⁹	10 ⁴⁹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁶⁰	10 ⁵⁰														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁶¹	10 ⁵¹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁶²	10 ⁵²														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁶³	10 ⁵³														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁶⁴	10 ⁵⁴														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁶⁵	10 ⁵⁵														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁶⁶	10 ⁵⁶														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁶⁷	10 ⁵⁷														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁶⁸	10 ⁵⁸														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁶⁹	10 ⁵⁹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁷⁰	10 ⁶⁰														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁷¹	10 ⁶¹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁷²	10 ⁶²														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁷³	10 ⁶³														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁷⁴	10 ⁶⁴														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁷⁵	10 ⁶⁵														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁷⁶	10 ⁶⁶														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁷⁷	10 ⁶⁷														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁷⁸	10 ⁶⁸														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁷⁹	10 ⁶⁹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁸⁰	10 ⁷⁰														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁸¹	10 ⁷¹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁸²	10 ⁷²														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁸³	10 ⁷³														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁸⁴	10 ⁷⁴														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁸⁵	10 ⁷⁵														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁸⁶	10 ⁷⁶														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁸⁷	10 ⁷⁷														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁸⁸	10 ⁷⁸														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁸⁹	10 ⁷⁹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁹⁰	10 ⁸⁰														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁹¹	10 ⁸¹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁹²	10 ⁸²														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁹³	10 ⁸³														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁹⁴	10 ⁸⁴														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁹⁵	10 ⁸⁵														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁹⁶	10 ⁸⁶														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁹⁷	10 ⁸⁷														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁹⁸	10 ⁸⁸														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻⁹⁹	10 ⁸⁹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁰⁰	10 ⁹⁰														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁰¹	10 ⁹¹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁰²	10 ⁹²														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁰³	10 ⁹³														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁰⁴	10 ⁹⁴														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁰⁵	10 ⁹⁵														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁰⁶	10 ⁹⁶														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁰⁷	10 ⁹⁷														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁰⁸	10 ⁹⁸														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁰⁹	10 ⁹⁹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹¹⁰	10 ¹⁰⁰														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹¹¹	10 ¹⁰¹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹¹²	10 ¹⁰²														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹¹³	10 ¹⁰³														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹¹⁴	10 ¹⁰⁴														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹¹⁵	10 ¹⁰⁵														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹¹⁶	10 ¹⁰⁶														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹¹⁷	10 ¹⁰⁷														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹¹⁸	10 ¹⁰⁸														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹¹⁹	10 ¹⁰⁹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹²⁰	10 ¹¹⁰														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹²¹	10 ¹¹¹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹²²	10 ¹¹²														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹²³	10 ¹¹³														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹²⁴	10 ¹¹⁴														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹²⁵	10 ¹¹⁵														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹²⁶	10 ¹¹⁶														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹²⁷	10 ¹¹⁷														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹²⁸	10 ¹¹⁸														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹²⁹	10 ¹¹⁹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹³⁰	10 ¹²⁰														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹³¹	10 ¹²¹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹³²	10 ¹²²														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹³³	10 ¹²³														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹³⁴	10 ¹²⁴														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹³⁵	10 ¹²⁵														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹³⁶	10 ¹²⁶														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹³⁷	10 ¹²⁷														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹³⁸	10 ¹²⁸														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹³⁹	10 ¹²⁹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁴⁰	10 ¹³⁰														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁴¹	10 ¹³¹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁴²	10 ¹³²														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁴³	10 ¹³³														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁴⁴	10 ¹³⁴														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁴⁵	10 ¹³⁵														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁴⁶	10 ¹³⁶														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁴⁷	10 ¹³⁷														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁴⁸	10 ¹³⁸														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0			10 ⁻¹⁴⁹	10 ¹³⁹														
0 0 0 0 0	0 0 0 0 0	0 0 0 0 0																		

DEPTH SCALE

1 : 75



LOGGED: P.A.H.

CHECKED: W.C.

MIS-RCK 001 07-1111-0042 (ROCK) GPJ GAL-MISS GDT 12/30/09 JM

[illegible]

PROJECT: 07-1111-0042

RECORD OF DRILLHOLE: 08-4

SHEET 1 OF 1

LOCATION: N 4945098.5; 324037.0

DRILLING DATE: Aug. 21, 2008

DATUM:

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Aardvark

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	PENETRATION RATE (m/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE F-FAULT				SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				DIAMETRAL ONLY LOG INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
				DEPTH (m)						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											
RECOVERY		R.Q.D. %	FRACT INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec																					
TOTAL CORE %	SOLID CORE %			DIP W.R.T. CORE AXIS	TYPE AND SURFACE DESCRIPTION	10 ⁰	10 ⁻¹	10 ⁻²	10 ⁻³																		
		Continued from Record of Borehole 08-4		118.00																							
		Granite (BEDROCK) Fresh Red-grey to grey Strong		4.50																							
5	Rotary Drill NO Core			1																							
				2																							
				3																							
6				4																							
7				5																							
8		End of Drillhole		115.00																							
9				7.50																							
10																											
11																											
12																											
13																											
14																											
15																											
16																											
17																											
18																											
19																											

DEPTH SCALE

1 : 75



LOGGED: D.G./P.A.H.

CHECKED: W.C.

MIS-RCK 001: 07-1111-0042 (ROCK) GPU GAL-MISS GDT 12/30/09 JM

PROJECT <u>07-1111-0042</u>		RECORD OF BOREHOLE No 08-5		1 OF 1 METRIC	
W.P. <u>479-92-00</u>		LOCATION <u>N 4945148.5; E 324024.2</u>		ORIGINATED BY <u>D.G.</u>	
DIST <u> </u> HWY <u>15</u>		BOREHOLE TYPE <u>Power Auger 108mm I.D. Hollow Stem</u>		COMPILED BY <u>J.M.</u>	
DATUM <u> </u>		DATE <u>Aug. 15, 2008</u>		CHECKED BY <u>W.C.</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
122.3	GROUND SURFACE													
120.8	Sandy silt with organic matter (TOPSOIL) Black SILTY CLAY (Weathered Crust) Very stiff Grey-brown Moist		1	SS	7									
119.3	CLAYEY SILT to CLAY Soft to firm Grey Wet		2	SS	2									
117.7	Silty SAND, some gravel, trace clay, with cobbles and boulders (TILL) Very dense Grey Wet		3	SS	>50									
			4	SS	>50									
			5	SS	>50									
			6	SS	>50									
4.6	End of Borehole Auger Refusal Note: Water level in open borehole at 2.2 m depth (Elev. 120.1 m) upon completion of drilling on Aug. 15, 2008													

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

MIS-MTO 001 07-1111-0042 GPJ GAL-MISS GDT 12/30/09

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

PROJECT		07-1111-0042		RECORD OF BOREHOLE No 08-6		1 OF 1 METRIC								
W.P.		479-92-00		LOCATION		N 4945072.3; E 324040.4								
DIST		HWY 15		BOREHOLE TYPE		Power Auger 108mm I.D. Hollow Stem								
DATUM				DATE		Aug. 20, 2008								
						ORIGINATED BY D.J.S.								
						COMPILED BY J.M.								
						CHECKED BY W.C.								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
122.5	GROUND SURFACE													
122.0	TOPSOIL													
0.3	SILTY CLAY (Weathered Crust) Very stiff to stiff Grey-brown Wet		1	SS	4		122							
			2	SS	1		121							
120.4	CLAYEY SILT to CLAY Firm Grey Wet						120	X	+					
119.5								X	+					
3.1	Sandy SILT (TILL) Grey End of Borehole Auger Refusal													

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APPENDIX C

Figure 1 - Summary of Undrained Shear Strengths

Figure 2 - Plasticity Chart - Clayey Silt to Clay

Figure 3 - Consolidation Test Results (Borehole 08-3)

Figure 4 - Consolidation Test Results (Borehole 08-5A)

Figure 5 - Grain Size Distribution Test Results (Glacial Till)

Figure 6 - Stability Analysis Output – 3.4 m Approach Embankment

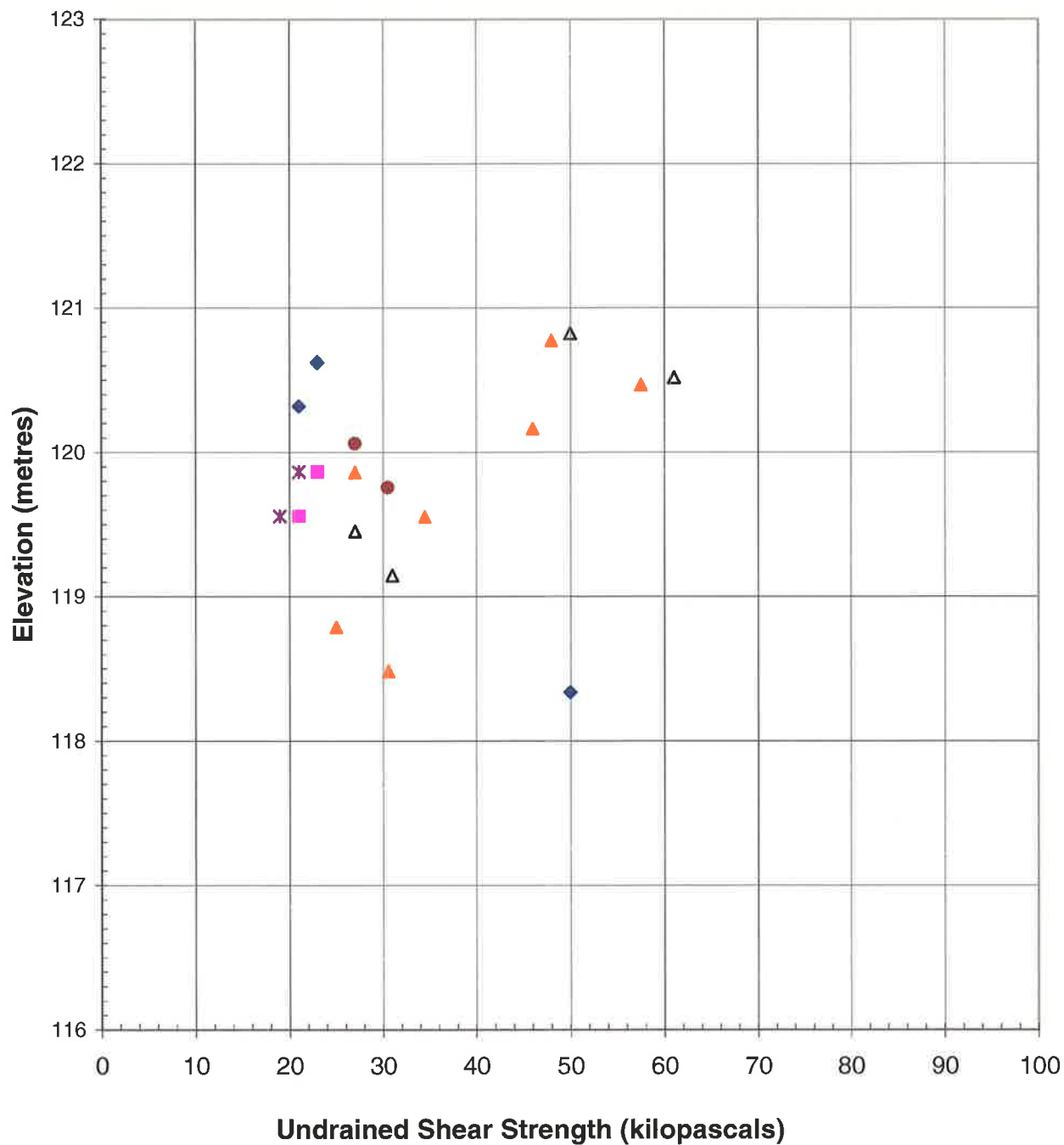
Figure 7 - Stability Analysis Output – 3.7 m Approach Embankment

**Figure 8 - Stability Analysis Output – 3.4 m Approach Embankment
with 1 m Surcharge**

**Figure 9 - Stability Analysis Output – 3.4 m Approach Embankment
with 1 m Surcharge and Temporary Berms**

SUMMARY OF UNDRAINED SHEAR STRENGTHS VERSUS ELEVATION

FIGURE 1



◆ BH 08-1 ■ BH 08-2 ▲ BH 08-3 △ BH 08-4 ✕ BH 08-5 ● BH 08-6

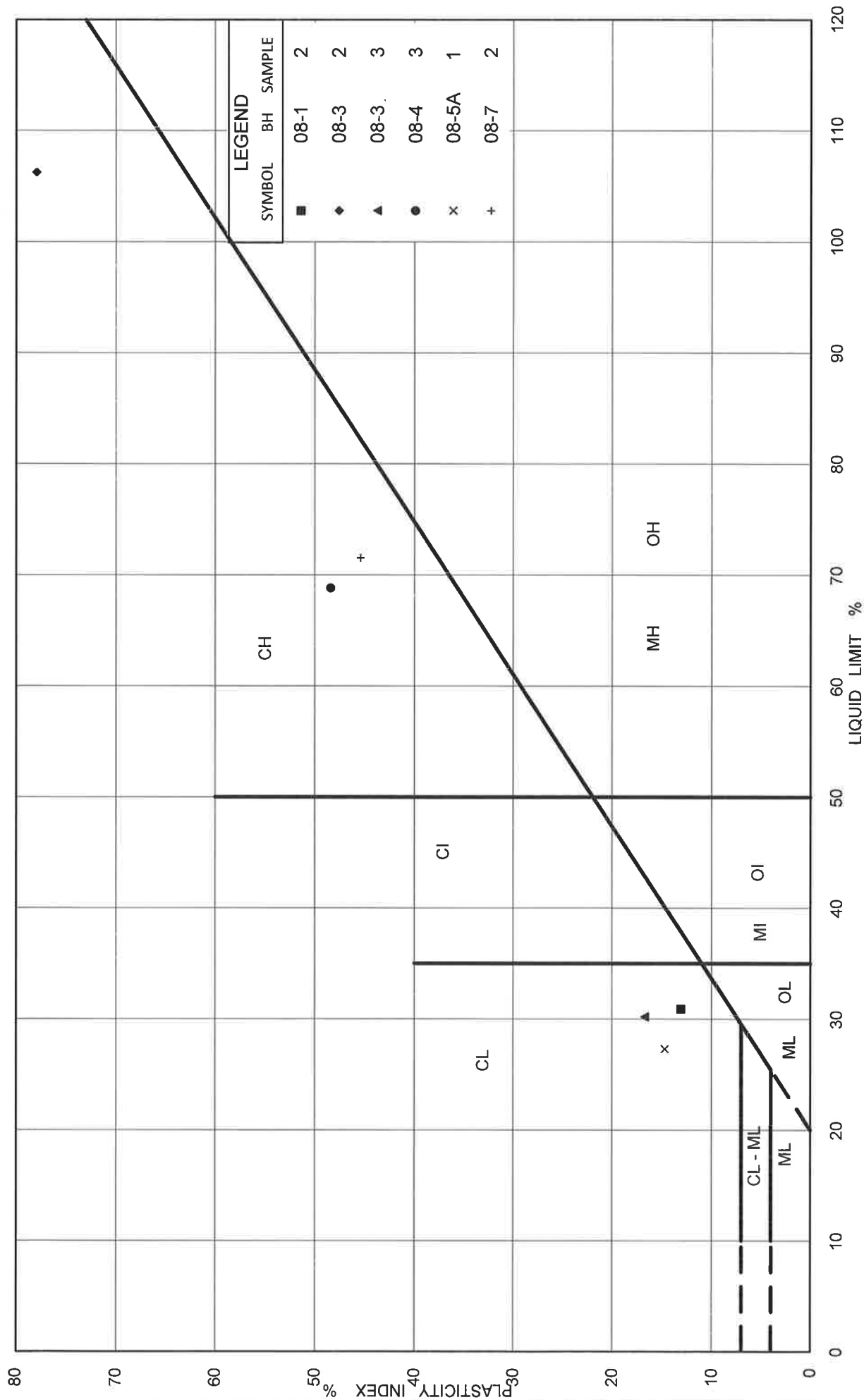
Date February 19, 2009
Project 07-1111-0042

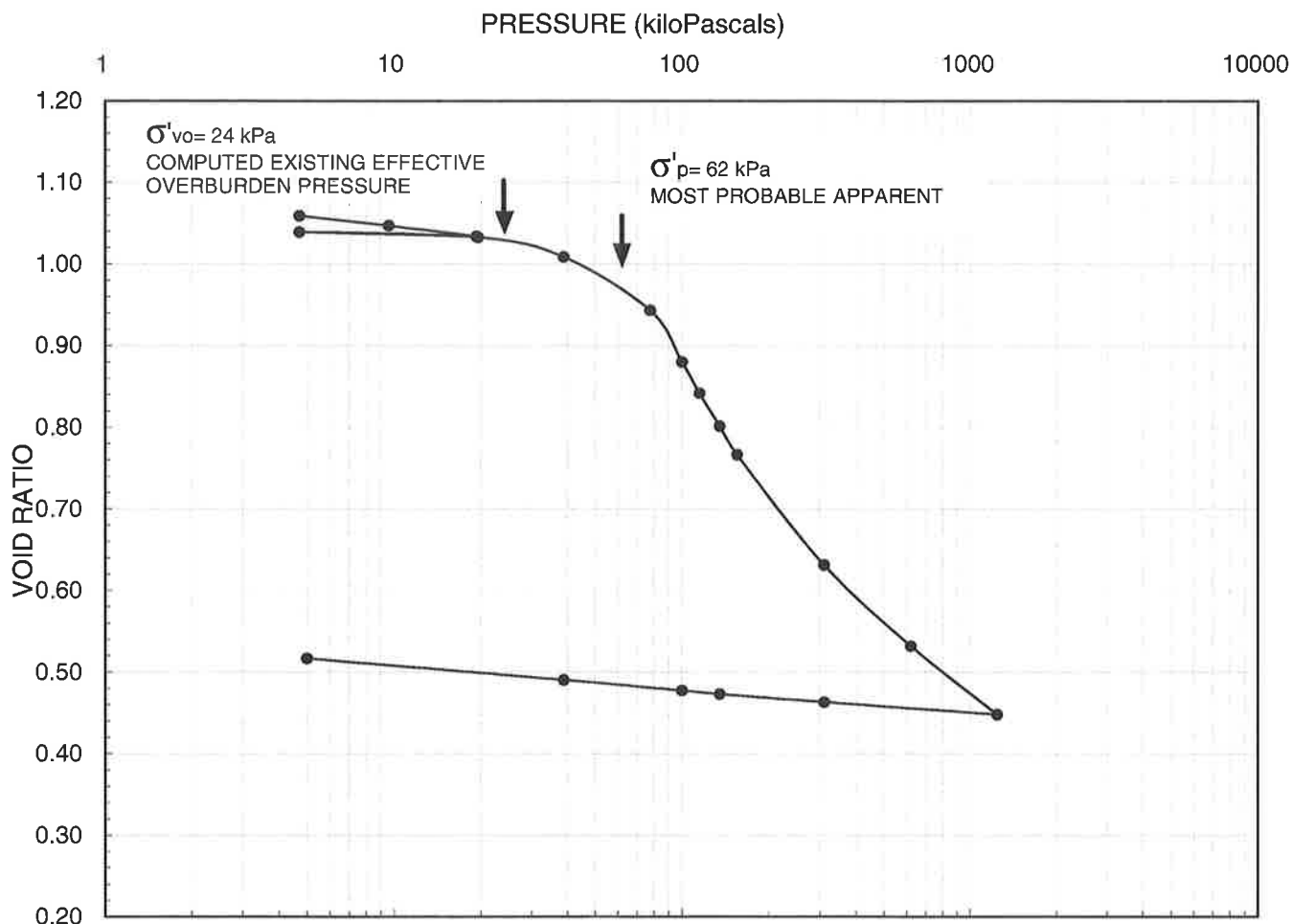
Golder Associates

Drawn
Chkd

SSK

WC





LEGEND

Borehole: 08-3
Sample: 3
Depth (m): 3.1-3.5

$w_i = 37.2\%$
 $w_f = 19.8\%$
 $w_l = 30.2\%$
 $w_p = 13.5\%$

$S_o = 96\%$
 $C_c = 0.59$
 $C_r = 0.010$



SCALE	AS SHOWN
DATE	02/18/09
DESIGN	NA
CADD	NA
CHECK	CNM
REVIEW	LM

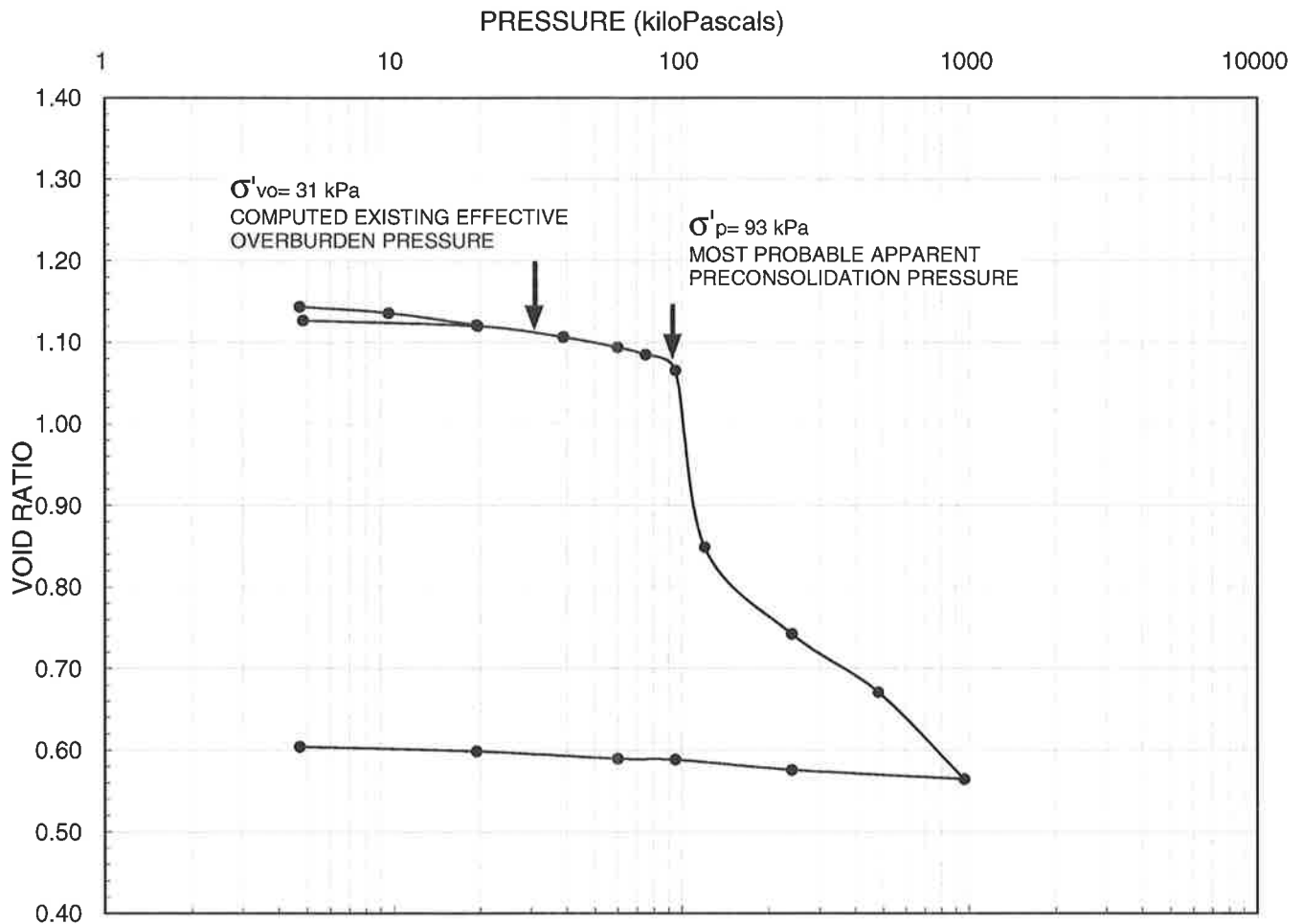
TITLE

CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary
PROJECT No. 07-1111-0042 REV. 2

FIGURE

3



LEGEND

Borehole: 08-5A
Sample: 1
Depth (m): 2.3-2.7

$w_l = 42.9\%$
 $w_f = 24.2\%$
 $w_l = 27.3\%$
 $w_p = 13.5\%$

$S_o = 103\%$
 $C_c = 2.13$
 $C_r = 0.008$



SCALE	AS SHOWN
DATE	02/18/09
DESIGN	NA
CADD	NA
CHECK	CNM
REVIEW	

CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary
PROJECT No. 07-1111-0042 REV. 2

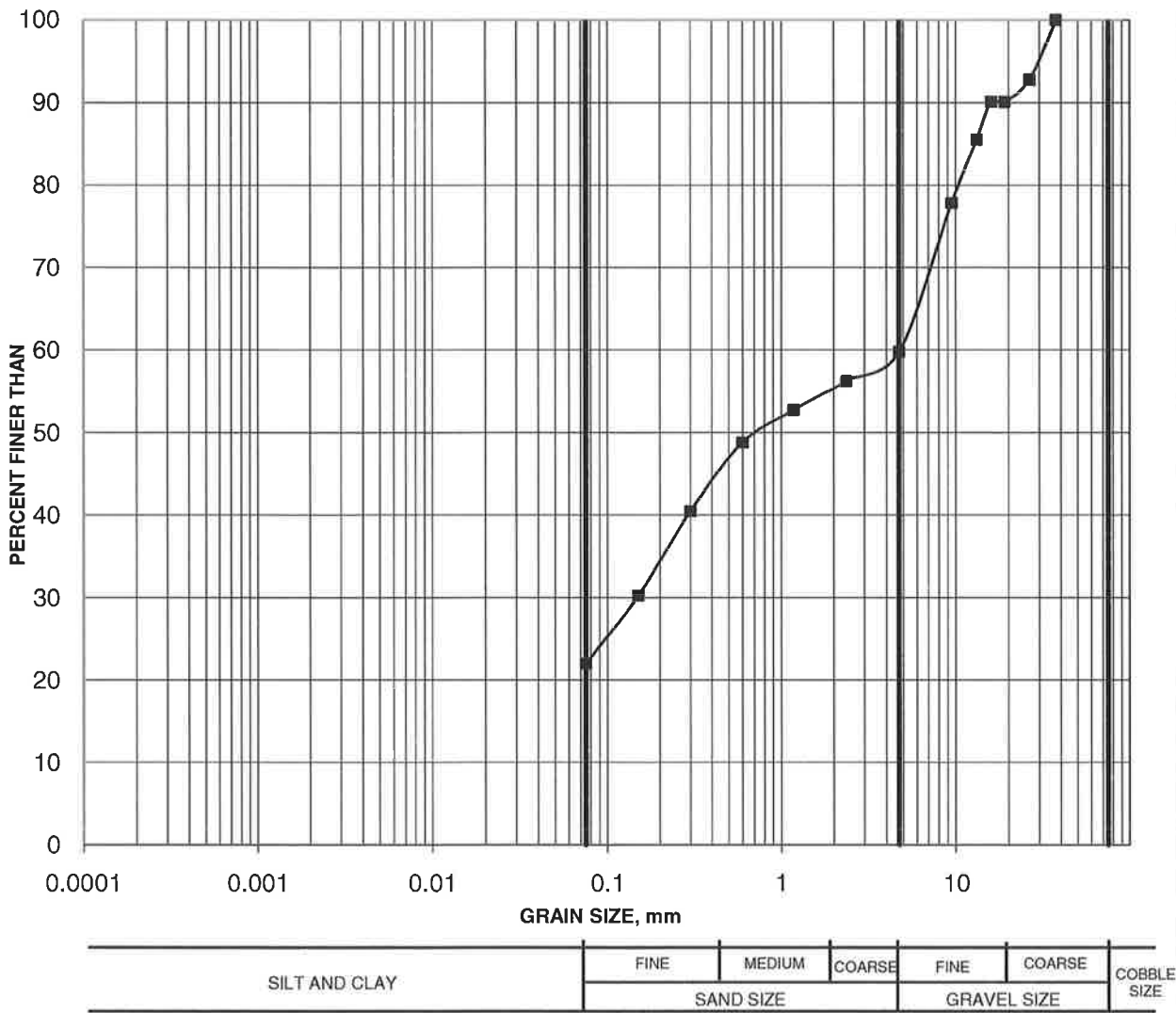
FIGURE

4

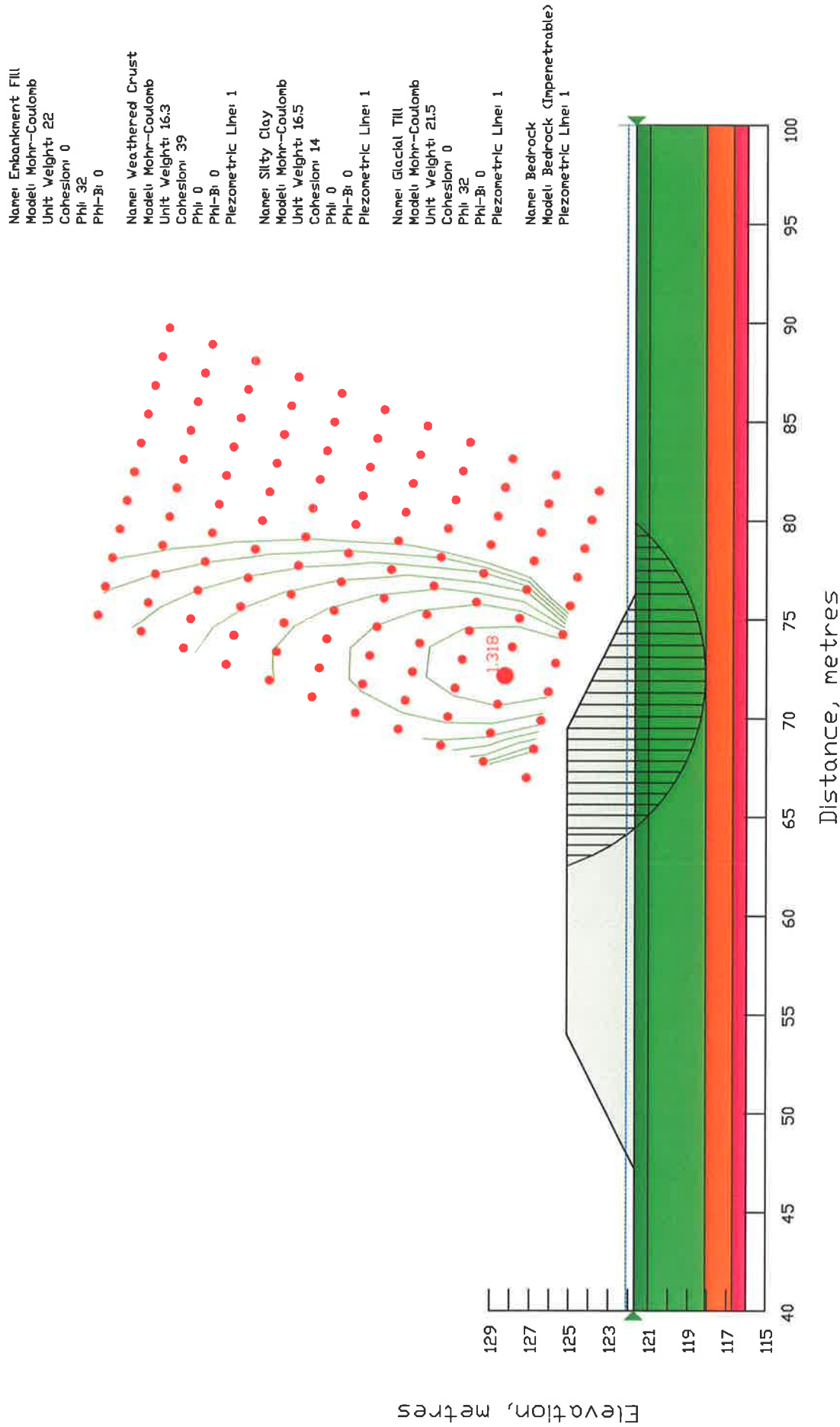
GRAIN SIZE DISTRIBUTION

FIGURE 5

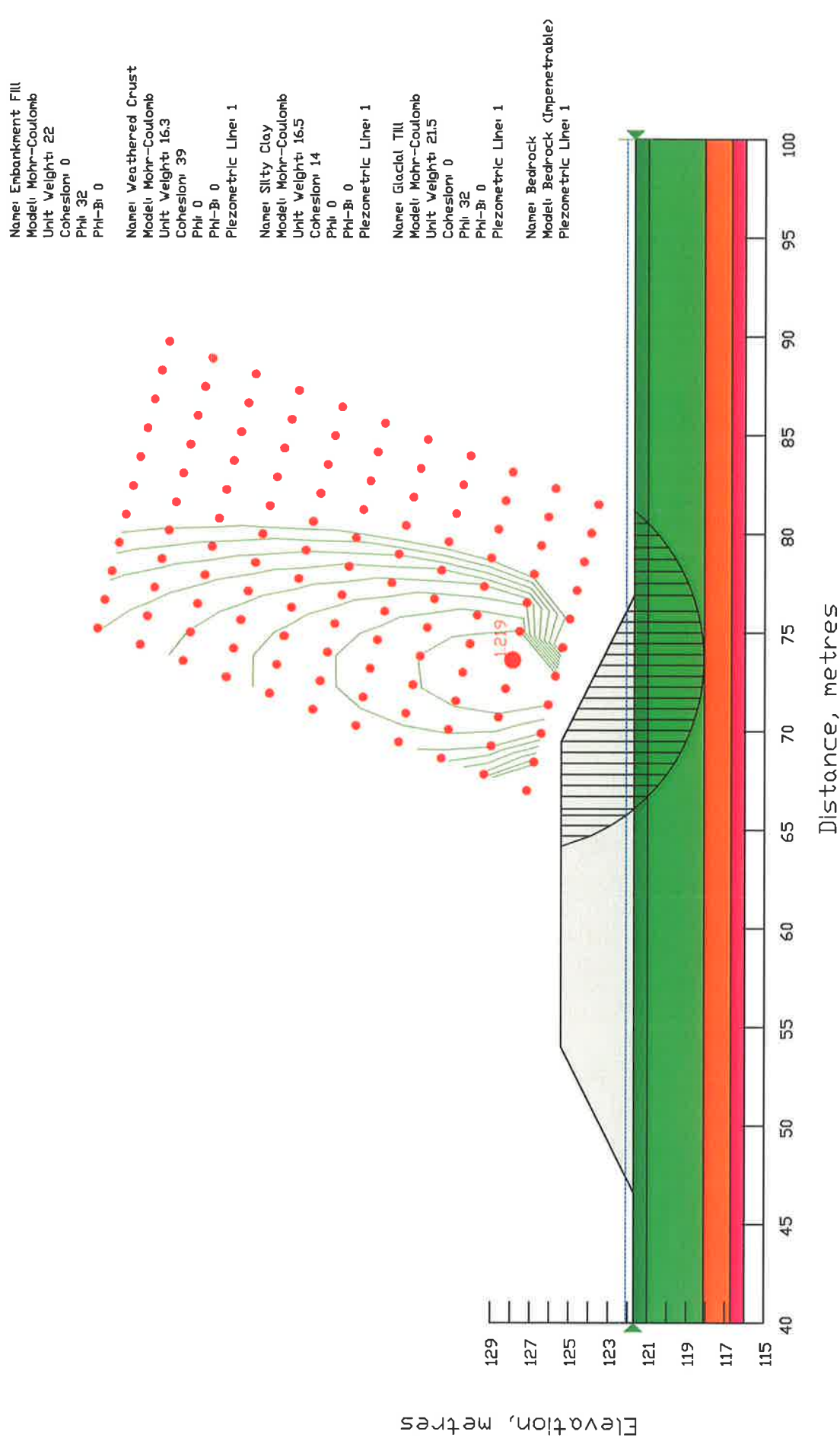
GLACIAL TILL



Borehole	Sample	Depth (m)
08-1	4	4.12-4.57

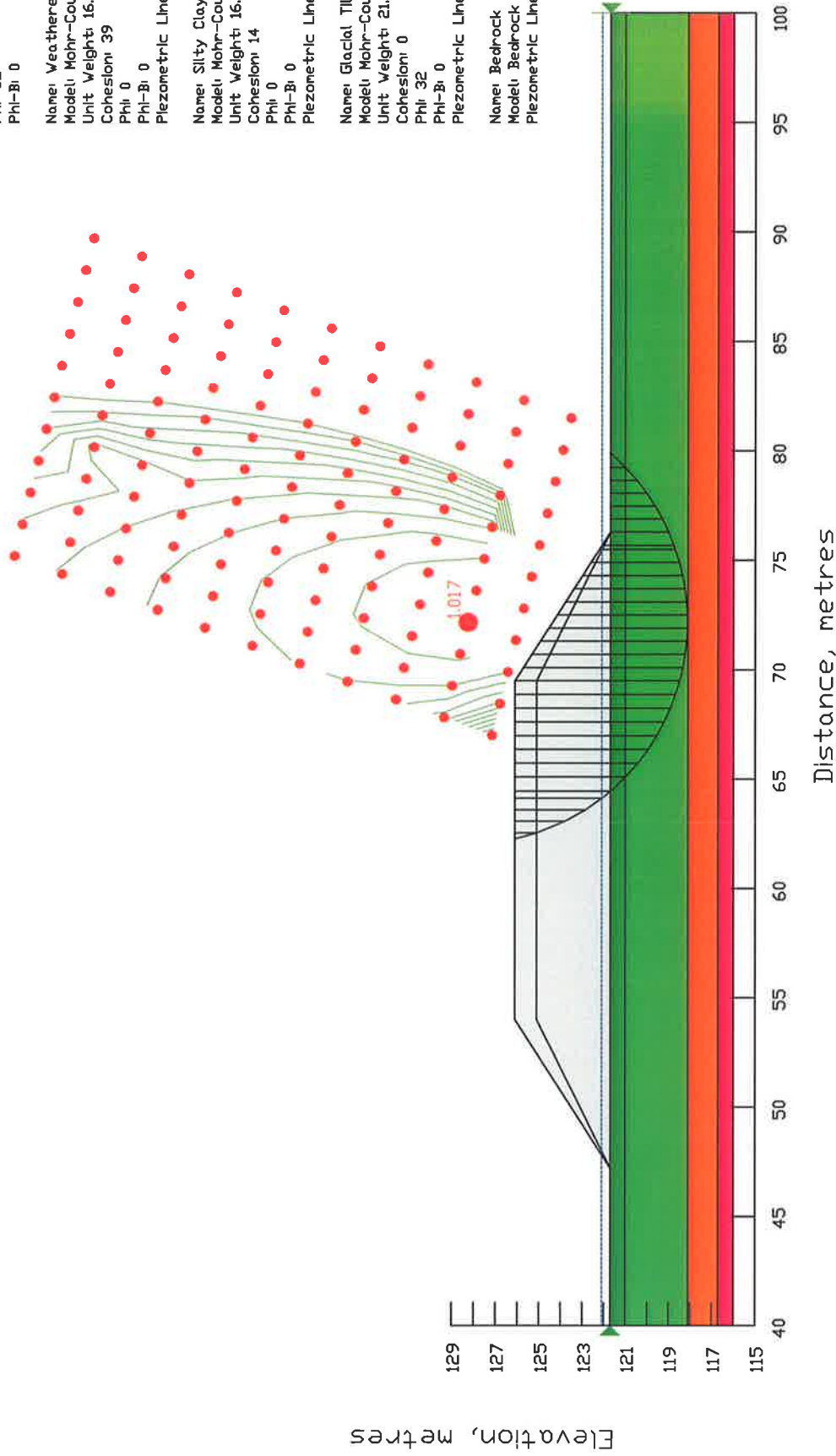


HIGHWAY 15
 PROPOSED CROSBY CREEK BRIDGE
STABILITY ANALYSIS OUTPUT
3.4m APPROACH EMBANKMENT
FIGURE 6

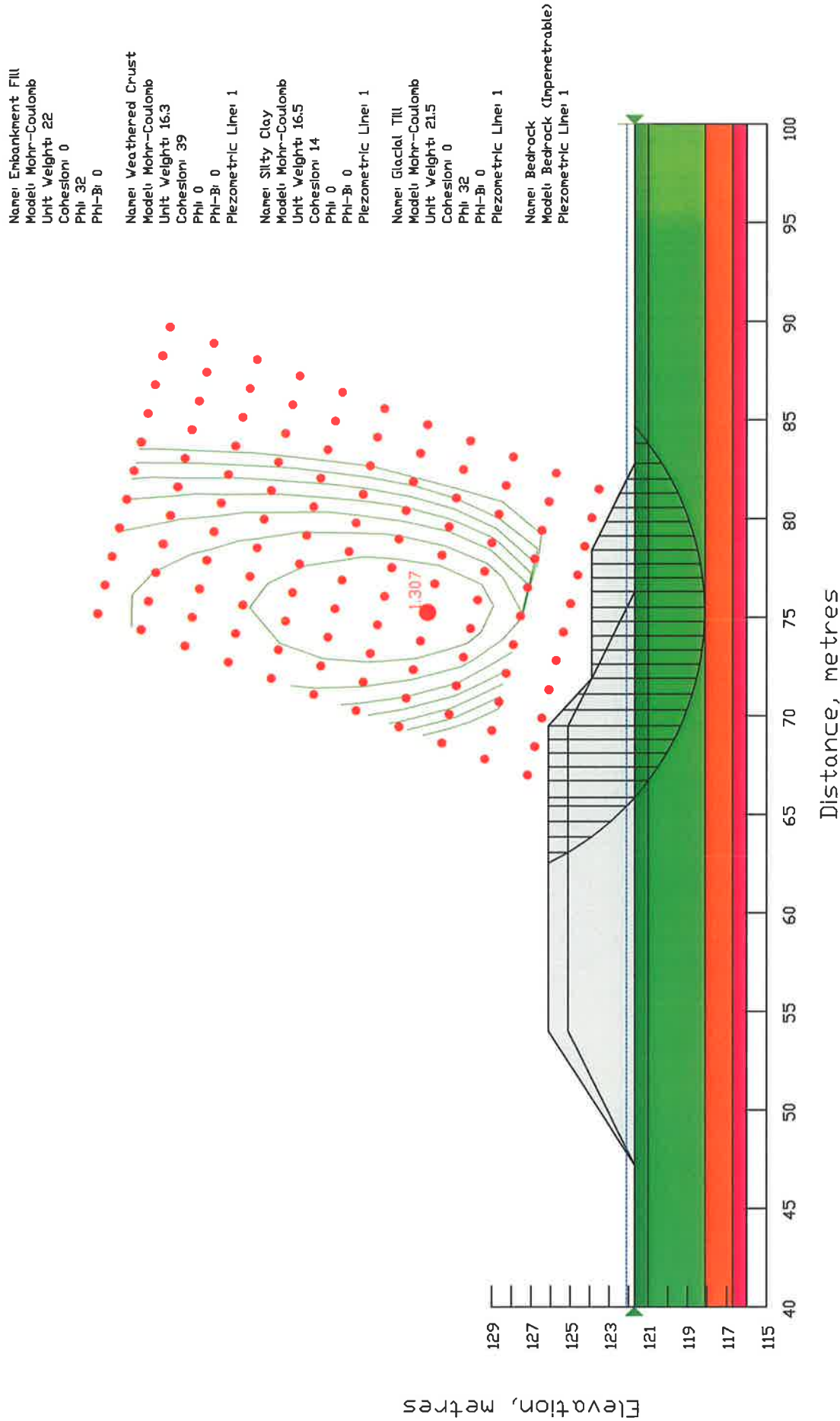


HIGHWAY 15
 PROPOSED CROSBY CREEK BRIDGE
STABILITY ANALYSIS OUTPUT
3.7m APPROACH EMBANKMENT
FIGURE 7

Name: Embankment Fill	Name: Weathered Crust
Model: Mohr-Coulomb	Model: Mohr-Coulomb
Unit Weight: 22	Unit Weight: 16.3
Cohesion: 0	Cohesion: 39
Phi: 32	Phi: 0
Phi-B: 0	Phi-B: 0
Piezometric Line: 1	Piezometric Line: 1
Name: Silty Clay	Name: Silty Clay
Model: Mohr-Coulomb	Model: Mohr-Coulomb
Unit Weight: 16.5	Unit Weight: 16.5
Cohesion: 14	Cohesion: 14
Phi: 0	Phi: 0
Phi-B: 0	Phi-B: 0
Piezometric Line: 1	Piezometric Line: 1
Name: Glacial Till	Name: Glacial Till
Model: Mohr-Coulomb	Model: Mohr-Coulomb
Unit Weight: 21.5	Unit Weight: 21.5
Cohesion: 0	Cohesion: 0
Phi: 32	Phi: 32
Phi-B: 0	Phi-B: 0
Piezometric Line: 1	Piezometric Line: 1
Name: Bedrock	Name: Bedrock
Model: Bedrock (Impenetrable)	Model: Bedrock (Impenetrable)
Piezometric Line: 1	Piezometric Line: 1



HIGHWAY 15
 PROPOSED CROSBY CREEK BRIDGE
STABILITY ANALYSIS OUTPUT
3.4m APPROACH EMBANKMENT WITH 1m SURCHARGE
 FIGURE 8



**HIGHWAY 15
 PROPOSED CROSBY CREEK BRIDGE
 STABILITY ANALYSIS OUTPUT
 3.4m APPROACH EMBANKMENT WITH
 1m SURCHARGE AND TEMPORARY BERMS**

FIGURE 9



APPENDIX D

NSSP - CSP for Integral Abutments

CSP FOR INTEGRAL ABUTMENTS - Item No.

Special Provision

Scope

This specification covers the requirements for the installation of the CSP's, including sand fill and polystyrene sheets, at the integral abutments.

References

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

Ontario Provincial Standard Specifications, Construction:

OPSS 906 Structural Steel
OPSS 909 Prestressed Concrete - Precast Members

Ontario Provincial Standard Specifications, General:

OPSS 180 Management and Disposal of Excess Materials

Ontario Provincial Standard Specifications, Material:

OPSS 1605 Expanded Extruded Polystyrene
OPSS 1801 Corrugated Steel Pipe Products

Canadian Standards Association Standards:

CSA G164-M Galvanizing of Irregularly-Shaped Articles

Ministry of Transportation Publications

MTO Manual of Designated Sources of Materials

Definitions

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

Abutment Stem: means the cast-in-place concrete component of the abutment placed over the top of the piles and forming the bearing seat for the girders.

CSP: means helical corrugated steel pipe.

Design Engineer: means the Engineer who produces the design and/or working drawings, and who has a minimum of five (5) years in the design and/or construction of bridges.

Submission and Design Requirements

Submissions

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

At least two weeks prior to commencement of installation of the abutment, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times.

Working Drawing Requirements

Working drawings shall include at least the following:

1. Layout and Elevations of the CSP's;
2. Source of the sand fill, and description of placing method and equipment;
3. Location and details of all temporary bracing, including permanent and temporary spacers, for the piles, CSP's and abutment stems;
4. Detailed construction sequence for the work, including installation and removal of the temporary bracing.

Design Requirements

The Contractor shall be responsible for the complete detailed design of the construction sequence for the work, including the installation and removal of all temporary bracing. The general sequence of construction shall be as shown on the Contract drawings.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including temporary and permanent spacers, required to maintain the piles, CSP's, abutment stems and girders in their specified positions through all stages of construction until concrete in deck has reached a compressive strength of 25 MPa. All temporary bracing, except spacers identified as permanent on the Contract drawings, shall be removed.

Temporary bracing for prestressed, precast girders shall meet the requirements of OPSS 909. Temporary bracing for structural steel girders shall meet the requirements of OPSS 906.

Material

Corrugated Steel Pipe

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM # 4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

Permanent Spacers and Associated Hardware

Permanent spacers and associated hardware left in place shall not consist of wood and corrodible material.

Sand Fill

The sand fill for backfilling the CSP's shall meet the gradation requirements of Table 1 below:

Table 1 - Sand Fill Gradation Requirements

MTO Sieve Designation		Percentage Passing by Mass
2 mm	# 10	100 %
600 µm	# 30	80 % to 100 %
425 µm	# 40	40 % to 80 %
250 µm	# 60	5 % to 25 %
150 µm	# 100	0 % to 6 %

Expanded Extruded Polystyrene

Expanded extruded polystyrene shall be in accordance with OPSS 1605, and shall be from a supplier listed under DSM # 3.30.30.

Construction

General

The sequence of construction for installing the concrete pads, CSP's, sand fill and abutment stems, including the installation and removal of the temporary bracing, shall be in accordance with the working drawings.

The Contractor shall not proceed with the abutment backfill above the level of the bottom of the CSP's without written permission from the Contract Administrator.

Corrugated Steel Pipe

CSP's shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSP's will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSP's shall be in accordance with the manufacturer's recommendations. Damaged CSP's shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSP's shall be repaired by two coats of zinc-rich paint.

The Contractor shall set the CSP over each pile in the abutment into the concrete pad, following the batter of the pile, while the concrete in the concrete pad is still plastic. The CSP's shall extend at least 150 mm into the concrete pad.

The Contractor shall ensure the full perimeter of the tops of all CSP's at each abutment are at the elevation shown on the working drawings.

After the CSP's have been set, the Contractor shall take all measures necessary to prevent the ingress of water, backfill and debris into the CSP's.

Sand Fill

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand fill itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP's.

After the sand fill has been placed to the top of each CSP, the Contractor shall take all measures necessary to prevent the ingress of water and other liquids into the sand fill until after the concrete in the abutment stem has been placed and cured.

Expanded Extruded Polystyrene

The expanded extruded polystyrene sheets shall completely cover the area under the abutment stem as shown on the Contract drawings. The sheets shall be placed in one piece for the width of the abutment stem, with butt joints perpendicular to the centre-line of abutment bearings. The minimum length of sheet shall be 500 mm.

Joints between sheets within 500 mm of a pile centre-line will not be permitted. At each pile location, a minimum 1000 mm long sheet shall be centred on the pile and a 500 mm diameter hole neatly cut in the sheet so as to fit over the pile in one piece.

The Contractor shall adjust the backfill to ensure full and uniform contact of the sheets with the backfill and the full perimeter of the tops of the CSP's. The vertical step at joints between sheets shall not exceed 5 mm.

The Contractor shall protect the sheets from damage during installation of the reinforcing for the abutment stem, and shall secure the sheets from "floating" during placing of the concrete in the abutment stem. Only hardware approved by the Owner shall be used to secure the sheets. All hardware used to secure the sheets shall be installed so as not to project above the top surface of the sheets into the abutment stem.

Temporary Bracing

Temporary bracing shall be installed and removed in accordance with the working drawings.

The temporary bracing shall not distort, nor pierce the walls of, the CSP's. Welding to the CSP's will not be permitted.

Concrete anchors shall be removed and the holes filled with non-shrink grout.

Tolerances

The CSP's at each pile shall be constructed to the following tolerances:

<u>Criteria</u>	<u>Tolerance</u>
Maximum deviation of CSP from pile centroid.	± 25 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified Elevation.	± 10 mm

Quality Assurance

Prior to placing the CSP's, the Contractor shall establish reference points at each abutment and determine the location of the centroid of each pile in the abutment with respect to these reference points. The Contractor shall maintain the reference points until written permission to proceed with the backfill above the level of the bottom of the CSP's has been given by the Contract Administrator.

Measurement for Payment

There will be no measurement for this item.

Basis of Payment

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