



April 30, 2009

REPORT



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FOUNDATION INVESTIGATION AND DESIGN REPORT BLANCHE RIVER BRIDGE REPLACEMENT AND NORTH EMBANKMENT WIDENING HIGHWAY 11, SITE NO. 47-006 TOWNSHIP OF MAISONVILLE, ONTARIO MINISTRY OF TRANSPORTATION, ONTARIO G.W.P. 168-98-00

Submitted to:
LEA Consulting Ltd.
625 Cochrane Drive, Suite 900
Markham, Ontario

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LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

RECORD OF BOREHOLE AND RECORD OF DRILLHOLE SHEETS (BI-1 TO BI-21 AND PHBI-1)



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PART A

FOUNDATION INVESTIGATION REPORT

BLANCHE RIVER BRIDGE REPLACEMENT AND

NORTH EMBANKMENT WIDENING

HIGHWAY 11, SITE NO. 47-006

TOWNSHIP OF MAISONVILLE, ONTARIO

MINISTRY OF TRANSPORTATION, ONTARIO G.W.P. 168-98-00



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by LEA Consulting Ltd. (LEA) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the detail design of the replacement of the structure carrying Highway 11 over Blanche River in the Township of Maisonville, north of Kirkland Lake and west of Sesekinika, Ontario.

The terms of reference for the scope of work are outlined in Golder's proposal P7-1191-0008, dated March 12, 2007, and subsequently revised in an email dated June 11, 2008, that forms part of the Consultant's Agreement (P.O. Number 5006-E-0042) for this project. The work was carried out in accordance with the Quality Control Plan for this project dated September 18, 2007. The general arrangement drawing for the bridge structure was provided to Golder by LEA in June 2008.

As a result of the findings of the subsurface investigation for the pavement design component of this project, the Terms of Reference for the Foundations Investigation and Design were subsequently expanded on October 3, 2008 to include investigation and design of the new highway embankment for a length of about 400 m to the north of the north bridge approach (from Station about 12+300 to Station 12+650).

The purpose of this investigation is to establish the subsurface conditions at the proposed replacement structure and the new embankment north of the north approach by borehole drilling, rock coring, in situ testing and laboratory testing on selected samples. The boreholes for the current investigation were located in the field by Golder relative to the centreline stakes laid out at the site by the surveyor retained by LEA. The location of the investigated area is shown in plan on Drawing 1.

2.0 SITE DESCRIPTION

The site is situated in the Township of Maisonville on Highway 11 crossing Blanche River, approximately 15 km north of Highway 66. The existing road grade is about 2 m above the river water level. The surrounding land is mainly used for residential development, with grass and tree cover extending beyond the limits of the site. The banks adjacent to the river are vegetated with grass and small shrubs. The river is shallow and mainly used for recreation and is generally about 20 m to 30 m wide adjacent to the site but narrows to a width of about 10 m at the existing highway crossing location.

The existing bridge was constructed in 1952 and has six spans with an overall deck length of about 29.6 m. It is understood that this bridge will be replaced with a new single span (30 m) structure. The existing bridge is supported on a combination of wooden posts and lagging at the abutments and a series of wooden pile bents founded on timber piles likely driven to bedrock. It is unknown if an earlier structure existed at the site prior to 1952 and details of any older foundations that may be present in the subsurface are not available. However, it should also be noted that timber piles and a timber cribbing (possibly an abandoned abutment from an earlier roadway alignment) were observed west of the proposed north abutment area within the river.

The existing highway grade is at about Elevation 310.5 m at the existing south and north bridge abutments. The water level in the river was measured at approximate Elevation 308.4 m in July 2008.

The section of highway north of the structure area from about Station 12+300 to Station 12+650 consists of an approximately 1 m to 3 m high embankment traversing low-lying swampy ground. Highway 570 connects into the east side of Highway 11 at about Station 12+425 and a local driveway or access road connects into the west side of the highway at about Station 12+320.



3.0 INVESTIGATION PROCEDURES

The fieldwork at the bridge site was carried out in several stages, with a total of twenty-one (21) boreholes and one (1) probehole advanced at the site.

- Between July 3 and 9, 2008, four (4) boreholes (BI-1 to BI-4) were advanced for the proposed south and north abutments and approaches, and two (2) boreholes (BI-5 and BI-5a) were advanced for the widening of the west side of the north approach embankment. Boreholes BI-1, BI-2 and BI-4 were drilled on land using a CME 850 track mounted drill rig supplied and operated by Landcore Drilling Inc. (Landcore) of Sudbury, Ontario. Boreholes BI-3, BI-5 and BI-5a were advanced over water in the Blanche River using a D25 drill rig mounted on a modular raft supplied and operated by Walker Drilling Ltd. of Barrie, Ontario.
- Between August 25 and 28, 2008, an additional five (5) boreholes (BI-6 to BI-9 and BI-8a) were advanced within/near the footprint of the proposed south and north abutments and an additional one (1) borehole (BI-1a) was advanced at the south approach embankment using portable equipment supplied by OGS Inc. of Ottawa, Ontario. Boreholes BI-1a, BI-6, BI-7 and BI-9 were advanced on land and BI-8 and BI-8a were advanced from a small raft over water in the Blanche River.
- On September 3 and 4, 2008, four (4) boreholes (BI-10 to BI-13) were advanced within the southbound lane of the existing Highway 11 near the east limits of the proposed south and north abutments, including one (1) borehole drilled through the existing bridge deck (BI-12), using a CME 55 truck-mounted drill supplied and operated by Landcore.
- From October 14 to 17 and October 21 to 23, 2008, six (6) boreholes (BI-14 to BI-19) were advanced between Stations 12+300 and 12+500 using portable equipment supplied by OGS.
- On October 17, 2008, a probehole (PHBI-1) was advanced between Boreholes BI-3 and BI-9, to estimate the depth to bedrock at this location.
- On November 4, 2008, two (2) boreholes (BI-20 and BI-21) were advanced at Stations 12+600 and 12+650 using a CME 55 truck-mounted drill rig supplied and operated by Landcore.

The boreholes were advanced using either 108 mm inside diameter (I.D.) continuous flight hollow stem augers, NW casing and wash boring or portable equipment using 'BW' casing and wash boring. Soil samples were obtained, where possible, continuously or at intervals of depth of about 0.75 m to 1.5 m, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-99). Shelby tube samples were taken in cohesive deposits at some borehole locations. Field vane shear tests were conducted in cohesive soils for assessment of undrained shear strengths (ASTM D2573-01). Rock core samples were obtained using an 'NQ' size core barrel.

Boreholes BI-1a, BI-5a and BI-8a were advanced immediately adjacent to Boreholes BI-1, BI-5 and BI-8, respectively, to carry out additional field vane testing in, and/or to obtain Shelby Tube samples of the cohesive soil deposits.

The land-based boreholes were advanced to depths ranging from about 3.0 m to 14.3 m below the existing ground surface and the water-based boreholes were advanced to depths ranging from about 2.0 m to 8.8 m below the water surface at the time of drilling. Most of the boreholes were advanced to auger, casing or sampler refusal (i.e. on inferred bedrock); however, seven of the boreholes were terminated prior to reaching refusal. Shallow Boreholes BI-1a, BI-5a and BI-8a were terminated in a clayey silt to silty clay stratum after completing additional field vane and/or Shelby tube sampling, Borehole BI-4 was terminated after penetrating through the clayey silt to silty clay stratum and extending approximately 3 m into the underlying sand stratum and Borehole BI-20 was terminated after penetrating through the clayey silt to silty clay stratum and extending approximately 3 m into the underlying silt stratum. Boreholes BI-16 and BI-17 were terminated prior to reaching



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refusal within the clayey silt deposit. A Dynamic Cone Penetration Test (DCPT) was advanced adjacent to Borehole BI-17 to a depth of 14.2 m below existing ground surface.

A minimum of 3 m of rock core was obtained from five of the boreholes drilled at this site within the abutment foundation units, namely Boreholes BI-2, BI-3, BI-6, BI-9 and BI-10. The drilling in Borehole BI-13 (located near the northeast corner of the north abutment foundation unit) was terminated at a depth of 10.5 m below the surface of the roadway shoulder due to boiling sand conditions that were noted in the adjacent river bed next to the existing bridge piles during wash boring operations. Following the end of drilling at BI-13, a DCPT was advanced from the bottom of the borehole to refusal on inferred bedrock at a depth of about 10.8 m.

The groundwater conditions in the open boreholes were observed during the drilling operations and piezometers were installed in two land-based boreholes, BI-1 and BI-4, at the south and north approaches, respectively, to allow monitoring of the groundwater level at these locations. The piezometers consisted of 50 mm outside diameter rigid PVC tubing with a 1.5 m long slotted screen, sealed within the clayey silt or sand stratum. The boreholes were backfilled with bentonite as per Ontario Regulation 903 (as amended by O. Reg. 372) upon completion of drilling and the two piezometers were backfilled in a similar manner after the last water level reading was obtained on September 4, 2008. The installation details and water level readings are presented on the Record of Borehole sheets that follow the text of this report.

The fieldwork was supervised throughout by members of our engineering and technical staff who located the boreholes, arranged for the clearance of underground service locations, supervised the drilling and sampling operations, logged the boreholes, and examined and cared for the soil and rock core samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples. One-dimensional consolidation (oedometer) tests were carried out on Shelby tube samples of the cohesive soil deposit obtained from two of the boreholes. In addition, uniaxial compressive strength (UCS) testing was carried out on selected specimens of the bedrock core recovered from the boreholes.

The locations of the proposed foundation elements were laid out in the field by Golder relative to the proposed centerline alignment as staked in the field by LEA's subconsultant SRQ Geomatics Inc. (SRQ), based on the dimensions shown on the General Arrangement drawing supplied by LEA in June 2008. In July 2008, the ground surface elevation at the location of Boreholes BI-1, BI-2 and BI-4 and the water surface elevation at Boreholes BI-3 and BI-5 were surveyed by SRQ, and the elevations were forwarded to Golder. Golder surveyed the ground surface and water surface elevation at the location of Boreholes BI-6 to BI-9 at the time of our fieldwork, referencing the ground surface elevation at Borehole BI-2 provided by SRQ. The ground surface elevations at Boreholes BI-10 to BI-13, advanced in the existing southbound lane, were taken from the survey information provided by LEA. The elevation of Probehole PHBI-1 was referenced to the ground surface elevation at Borehole BI-9. The ground surface and water surface elevations are referenced to geodetic datum.

The locations of Boreholes BI-14 to BI-21 were laid out in the field by Golder relative to the proposed centreline alignment as staked in the field by LEA's subconsultant SRQ. Golder surveyed the ground surface at Boreholes BI-14 to BI-21 at the time of our fieldwork, referencing the existing ground surface at the proposed centreline alignment stakes. The ground surface elevations are referenced to geodetic datum.

The borehole and probehole locations and ground/water surface elevations are summarized below. The locations of the boreholes for the bridge structure and immediate approach embankments are shown on Drawing 1 and the ground surface profiles and cross-sections at selected locations are shown on Drawings 1 and 2. The locations of the boreholes on the roadway alignment between Station 12+300 and Station 12+650 are shown on Drawing 3.



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Borehole / Probehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
BI-1	South Approach	5341406.1	359752.5	309.9
BI-1a	South Approach	5341406.9	359752.3	309.9
BI-2	South Abutment	5341425.1	359758.7	310.2
BI-3	North Abutment	5341455.4	359760.5	308.4*
BI-4	North Approach	5341471.9	359769.4	310.3
BI-5	West Side of North Approach	5341465.2	359758.6	308.4*
BI-5a	West side of North Approach	5341467.4	359758.1	308.4*
BI-6	South Abutment	5341423.2	359753.2	309.6
BI-7	South Abutment	5341427.1	359754.1	309.3
BI-8	North Abutment	5341458.8	359759.2	308.1*
BI-8a	North Abutment	5341457.6	359759.9	308.1*
BI-9	North Abutment	5341456.1	359766.8	309.2
BI-10	South Abutment	5341421.6	359765.1	310.5
BI-11	South Abutment	5341425.5	359765.9	310.5
BI-12	North Abutment	5341452.8	359772.2	310.5
BI-13	North Abutment	5341458.6	359773.5	310.5
PHBI-1	North Abutment	5341457.9	359762.5	308.1*
BI-14	North Embankment	5341517.4	359774.6	309.2
BI-15	North Embankment	5341565.7	359788.1	309.3
BI-16	North Embankment	5341608.7	359821.9	310.4
BI-17	North Embankment	5341614.7	359798.1	309.1
BI-18	North Embankment	5341638.9	359804.2	309.1
BI-19	North Embankment	5341711.7	359822.5	310.5
BI-20	North Embankment	5341808.7	359846.9	311.4
BI-21	North Embankment	5341854.1	359871.6	311.7

* Water Surface Elevation



4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Published literature indicates that the site is located in the Abitibi Subprovince of the Superior Province (Geology of Ontario; OGS Special Volume 4)¹. The Abitibi Subprovince consists of granite-greenstone-gneiss terrane, generally with minor metasedimentary rock overlying the metavolcanic rock.

Based on terrain mapping by the Ontario Geological Survey², the subsurface soils in the vicinity of the site consist of outwash plain deposits comprising gravels and sands, with peat swamps located within the vicinity of the site.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions, as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil and rock samples, are given on the attached Record of Borehole and Drillhole sheets following the text of this report. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations of drilling progress and cuttings. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

4.2.1 Bridge Structure and Immediate Approach Embankments

The inferred soil stratigraphy based on the results of the boreholes at the bridge location is shown on Drawings 1 and 2.

In general, the subsoils at the structure site consist of embankment fill of the existing Highway 11, underlain by deposits of peat, sand, silt or sandy silt and silty clay to clayey silt over bedrock. In Blanche River, on the west side of the existing highway, a sand alluvium was encountered at the river bed (below up to about 0.9 m of water), underlain by the deposits as described above. The total thickness of overburden is variable at the site, ranging from about 3 m to 5 m in the area of the proposed south abutment and from about 4 m to 11 m in the area of the proposed north abutment. The thickness of overburden (including existing roadway fill) in the area of the north approach ranges from about 10.8 m to greater than 12.8 m.

As described in Section 3, most of the boreholes were terminated on the inferred bedrock surface, with the exception of Borehole BI-4 (north approach borehole) and Boreholes BI-1a, BI-5a and BI-8a (shallow supplementary sampling/testing boreholes) which did not encounter refusal. Boreholes BI-2, BI-3, BI-6, BI-9 and BI-10 were cored at least 3 m into the bedrock.

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

¹ Geology of Ontario, 1991. Ontario Geological Survey, special Volume 4, Part 1. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

² Northern Ontario Engineering Geology Terrain Study, OGS Electronic Map Reference Number 42 A/SE.



4.2.1.1 Topsoil

Moist, brown to black sandy topsoil was encountered at ground surface in Boreholes BI-1, BI-2, BI-6 and BI-7 with a thickness ranging from about 0.2 m and 0.6 m. The ground surface at these boreholes ranges from about Elevation 309.3 m to 310.2 m. The natural water content measured on one sample of the topsoil was about 25 percent.

4.2.1.2 Embankment Fill

All of the boreholes drilled on land (with the exception of BI-1) were advanced through the existing highway embankment fill. In Boreholes BI-10, BI-11 and BI-13 (drilled on the roadway surface), approximately 150 mm of asphalt was encountered overlying a 0.2 m to 0.3 m thick layer of sand and gravel road base fill. Underlying the road base fill in Boreholes BI-10 and BI-11, cobbles and/or a boulder were encountered at a depth of about 0.4 m. Borehole BI-12 was advanced through the existing bridge deck and encountered approximately 110 mm of asphalt overlying 165 mm of concrete and 240 mm of timber before continuing to advance through the embankment fill located about 0.9 m below the deck. The remaining on-land boreholes were advanced through the embankment side slopes.

The embankment fill generally consists of sand to silty sand, trace to some silt, trace to some gravel, trace to some clay and was encountered from the ground surface in Boreholes BI-4 and BI-9, below the bridge deck in Borehole BI-12, underlying the topsoil in Boreholes BI-2, BI-6 and BI-7 and below the road base fill in Boreholes BI-10, BI-11 and BI-13. At the bottom of the fill deposit in Borehole BI-6, approximately 0.3 m of cobbles and/or a boulder were encountered at a depth of about 1.8 m, corresponding to Elevation 307.8 m. The ground surface/top of fill ranges from about Elevation 310.5 m (top of road) to 309.1 m and the fill ranges in thickness from about 0.4 m to 3.2 m.

Fill consisting of clayey silt was encountered below the sand fill in Borehole BI-7 at a depth of 0.6 m, corresponding to Elevation 308.7 m, for a thickness of 1.2 m.

The bottom of the fill materials ranges from about 1.8 m to 3.4 m below ground surface (up to 4.4 m below the bridge deck) and the fill-native ground interface was found between about Elevation 306.1 m and 307.7 m.

SPT 'N' values measured within the fill range from 0 (i.e. weight of hammer) to 25 blows per 0.3 m of penetration, indicating a very loose to compact relative density. Grain size distributions of several samples of the sand fill are shown on Figures A-1a and A-1b in Appendix A.

An Atterberg limits test carried out on one sample of the clayey silt fill deposit indicates a liquid limit of about 21 percent and a plastic limit of about 13 percent, yielding a plasticity index of 8 percent. The results of the Atterberg limits test are shown on the plasticity chart on Figure A-2 in Appendix A and indicate that the material is classified as a clayey silt of low plasticity.

The natural water content measured on samples of the fill ranges between about 3 percent and 24 percent.

4.2.1.3 Peat

Underlying the fill materials in Boreholes BI-7, BI-10, BI-11 and BI-13, an approximately 0.2 m to 0.5 m thick layer of wet, brown to black, very soft to soft peat was encountered. The top of the peat was encountered at depths between about 1.8 m and 3.2 m below existing grade, corresponding to between about Elevation 307.6 m and 307.3 m.

The natural water content measured on samples of the peat ranges between about 97 percent and 145 percent.



4.2.1.4 Sand (Alluvium)

As noted in Section 3, Boreholes BI-3, BI-5, BI-5a, BI-8 and BI-8a were advanced from a raft within Blanche River. Below approximately 0.8 m to 0.9 m of water, an alluvium deposit consisting of brown to black sand, trace to some silt, trace gravel and containing organics was encountered at the river bottom. The top of the alluvium/bottom of the river was encountered at between about Elevation 307.6 m and 307.3 m and the alluvium was about 0.6 m thick.

SPT 'N' values measured within the sand alluvium range from 2 to 7 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

The natural water contents measured on two samples of the sand alluvium are 23 percent and 40 percent.

4.2.1.5 Silt to Sand to Sandy Silt

Below the existing embankment fill and/or peat in Boreholes BI-2, BI-4 to BI-7, BI-11 and BI-13, several cohesionless soil deposits were encountered. The deposits include a grey silt, trace to some clay, trace to some sand, trace organics; a brown to grey sand, trace to some silt; and a grey sandy silt, trace clay.

The top of the silt, sand and silty sand deposits range from about Elevation 307.7 m to 306.8 m and the thickness of the deposits range from about 0.6 m to 3.8 m. Heaving sands occurred during the drilling within the sand deposit at a depth of about 3.8 m (Elevation 306.5 m) in Borehole BI-4 and at a depth of about 5.3 m (Elevation 305.2 m) in Borehole BI-13.

SPT 'N' values measured within the silt, sand and silty sand deposits range from 0 (i.e. weight of hammer) to 23 blows per 0.3 m of penetration, but typically range from about 0 to 9 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

An Atterberg limits test carried out on one sample of the silt deposit from Borehole BI-2 indicates that the silt is non-plastic. Grain size distributions for samples from the sand and the sandy silt to silt deposits are shown on Figure A-3a and Figure A-3b, respectively.

The natural water content measured on samples of the silt, sandy silt and sand deposit ranges from about 24 percent to 33 percent.

4.2.1.6 Sand and Gravel

A thin layer of loose, grey sand and gravel was encountered below the silt deposit at the bottom of Borehole BI-7 at a depth of about 2.9 m below existing ground surface, corresponding to about Elevation 306.4 m. The thickness of the layer was about 0.1 m.

4.2.1.7 Silty Clay to Clayey Silt

Below the existing embankment fill and/or peat in Boreholes BI-1, BI-9, BI-10 and BI-12, and below the sand, silt and sand (alluvium) deposits in Boreholes BI-3 to BI-5, BI-8 and BI-13, a deposit of brown to grey, silty clay to clayey silt, trace to some sand, trace gravel was encountered.

The top of this deposit was encountered at elevations ranging from about Elevation 309.3 m to 303.9 m and the thickness ranges from about 0.7 m to 5.4 m.



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SPT 'N' values measured within the silty clay to clayey silt range from 0 (i.e. weight of hammer) to 7 blows per 0.3 m of penetration suggesting a very soft to firm consistency. In situ field vane testing carried out within this stratum measured undrained shear strengths ranging from about 13 kPa to 51 kPa, but typically range from about 15 kPa to 25 kPa indicating a predominantly soft consistency.

Atterberg limits testing carried out on eighteen (18) samples of the silty clay to clayey silt deposit indicate liquid limits ranging from about 20 percent to 40 percent and plastic limits ranging from about 13 percent to 21 percent, yielding plasticity indices ranging from about 7 percent to 21 percent. The results of the Atterberg limits testing are shown on the plasticity charts for the clayey silt on Figures A-4a and A-4b and the silty clay on Figure A-5 and indicate that the stratum ranges from clayey silt of low plasticity to silty clay of intermediate plasticity. Grain size distribution tests were carried out on ten (10) samples of the clayey silt to silty clay deposit and the results are shown on Figures A-6, A-7a and A-7b.

The natural water content measured on select samples of this deposit ranges between 22 percent and 51 percent.

Two laboratory consolidation (oedometer) tests were carried out on specimens of the silty clay to clayey silt obtained from Boreholes BI-1a and BI-8a and the test results are shown on Figures A-8 and A-9, respectively. The preconsolidation pressures were estimated from the Void Ratio versus logarithmic Pressure plots using the Casagrande method as well as from the Total Work versus Pressure plots. The relevant consolidation test results are summarized below:

Borehole / Sample Number	Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	e_o	C_r	C_c	c_v^* (cm ² /s)
BI-1a / 1	306.4	45	70	25	1.6	0.85	0.07	0.24	0.012
BI-8a / 1	306.5	8	20	12	2.5	1.03	0.06	0.20	0.004

Note: * For approximate stress range of $10 \leq \sigma_v' \leq 80$ kPa

where:

- σ_{vo}' effective overburden pressure in kPa
- σ_p' preconsolidation pressure in kPa
- OCR overconsolidation ratio
- e_o initial void ratio
- C_c compression index (based on void ratio)
- C_r recompression index (based on void ratio)
- c_v coefficient of consolidation in cm²/s in the normally consolidated range

4.2.1.8 Sand to Silty Sand to Silt

Beneath the silty clay to clayey silt stratum in Boreholes BI-4, BI-5, BI-10 and BI-12, a deposit of sand to silty sand to silt was encountered. The deposits are described as a sand to silty sand, some gravel, trace clay; a silty sand, trace to some clay, trace to some gravel; and a silt, trace to some sand, trace to some clay.

The top of these deposits were encountered at elevations ranging from about Elevation 306.4 m to 300.5 m and the thickness ranges from about 0.3 m to 3.2 m. Borehole BI-4 was terminated within the sand to silty sand layer at a depth of 12.8 m.

SPT 'N' values measured within these deposits range from 1 to 12 blows per 0.3 m of penetration, indicating a very loose to compact relative density.



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The test results of grain size distributions performed on samples of the sandy silt to silt and the sand to silty sand deposits are shown on Figures A-10a and A-10b, respectively.

The natural water content measured on samples of the sand to silty sand to silt deposits ranges from about 12 percent to 25 percent.

4.2.1.9 Bedrock

Bedrock was encountered and cored for a minimum of 3 m in Boreholes BI-2, BI-3, BI-6, BI-9 and BI-10. The presence of bedrock was inferred from auger, casing or sampler refusal in all the remaining boreholes and probeholes (i.e. Probehole PHBI-1) except for Borehole BI-4 (north approach borehole) and Boreholes BI-1a, BI-5a and BI-8a (shallow supplementary sampling/testing boreholes).

The top of the bedrock surface within the boreholes and probeholes ranged from about Elevation 299.7 m to 306.3 m, corresponding to depths of about 3.0 m to 10.8 m below ground or water surface. The depth to bedrock below ground or water surface (at the time of drilling) and corresponding bedrock surface elevation estimated at each borehole is summarized below.

Location	Borehole/ Probehole	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Comments
South Approach	BI-1	5.5	304.4	Auger Refusal (Probable Bedrock)
South Abutment	BI-2	4.1	306.1	Bedrock Cored
	BI-6	4.0	305.6	Bedrock Cored
	BI-7	3.0	306.3	Split-Spoon Refusal (Probable Bedrock)
	BI-10	5.0	305.5	Bedrock Cored
	BI-11	5.0	305.5	Split-Spoon and Auger Refusal (Probable Bedrock)
North Abutment	BI-3	3.7	304.7	Bedrock Cored
	BI-8	3.6	304.5	Split-Spoon Refusal (Probable Bedrock)
	BI-9	7.8	301.4	Bedrock Cored
	BI-12	8.4	302.1	Casing Refusal (Probable Bedrock)
	BI-13	10.8	299.7	DCPT Refusal (Probable Bedrock)
	PHBI-1	5.2	302.9	Probe Refusal (Probe sliding on bedrock below 5.2 m depth)
North Approach	BI-5	8.8	299.7	Casing Refusal (Probable Bedrock)



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Based on a review of the bedrock core samples, the bedrock at the site generally consists of fine to medium grained, slightly weathered, grey gabbro. The rock core samples obtained from Borehole BI-2 indicate that a fine to medium grained, slightly weathered, pinkish grey granite is present overlying the gabbro at this location. In general, the bedrock samples are described as slightly weathered, fine to medium grained.

The Rock Quality Designation (RQD) measured on the core samples ranges from 0 percent to 100 percent. This indicates rock mass of variable quality, ranging from very poor to excellent. The RQD values are typically greater than about 50 percent indicating that the bedrock is generally of fair to excellent quality, except in Borehole BI-6 where the RQD ranges from about 0 percent to 53 percent. In all boreholes, excluding BI-3, broken rock was recovered at various depths within the cored zone. The Total Core Recovery (TCR) during bedrock coring was generally 100 percent.

Laboratory UCS testing was carried out on eight core samples of the gabbro bedrock from Boreholes BI-2, BI-3, BI-6, BI-9 and BI-10, and one core sample of the granite bedrock from Borehole BI-2. The UCS results range between 55 MPa and 201 MPa, indicating strong to very strong rock. The depths and corresponding elevations of the tested samples and results of the UCS testing are presented in Table A-1.

4.2.1.10 Groundwater Conditions

The water levels were noted during and after the drilling and coring operations in the boreholes. Piezometers were installed with screened sections sealed within the silty clay to clayey silt deposit in Borehole BI-1 and within the upper sand deposit in Borehole BI-4. Details of the piezometer installations are shown on the Record of Borehole Sheets following the text of this report. In general, the soil samples taken in the boreholes were noted to be moist to wet with free water evident within most of the non-cohesive materials.

The water level of Blanche River was measured at Elevation 308.4 m in July 2008 and at Elevation 308.1 m in August 2008. The water levels measured in the piezometers and open holes upon completion of drilling are summarized below. It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

Borehole	Installation	Groundwater Level Depth (m)	Groundwater Level Elevation (m)	Date
BI-1	Piezometer	2.3	307.6	July 8, 2008
		1.3	308.6	September 4, 2008
BI-2	Open Borehole	1.2	309.0	Upon Completion of Drilling
BI-3	River Water Level		308.4	July 2008
BI-4	Piezometer	2.6	307.7	July 9, 2008
		1.9	308.4	September 3, 2008
BI-5	River Water Level		308.4	July 2008
BI-6	Open Borehole	1.4	308.2	Upon Completion of Drilling
BI-7	Open Borehole	1.2	308.1	Upon Completion of Drilling
BI-8	River Water Level		308.1	August 2008
BI-9	Open Borehole	1.0	308.2	Upon Completion of Drilling
BI-10	Open Borehole	2.3	308.2	Upon Completion of Drilling
BI-11	Open Borehole	2.2	308.3	Upon Completion of Drilling
BI-12	Open Borehole	2.1	308.4	Upon Completion of Drilling
BI-13	Open Borehole	2.2	308.3	Upon Completion of Drilling



4.2.2 North Embankment – Station 12+300 to 12+650

The inferred soil stratigraphy based on the results of the boreholes along the roadway between about 60 m and 400 m north of Blanche River is shown on Drawing 3.

In general, the subsoils encountered at the borehole locations advanced along this section of the north embankment consist of topsoil or fill materials underlain by deposits of peat, sand to silty sand, clayey silt to silty clay, silt and/or sand and gravel. The thickness of overburden (including existing fill) ranges from about 4.3 m to greater than 14.3 m.

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

4.2.2.1 Topsoil

Moist, brown topsoil was encountered at ground surface in Boreholes BI-14 and BI-15 with thicknesses of about 0.3 m and 0.2 m, respectively.

4.2.2.2 Fill

Fill materials consisting of sandy gravel, sand and gravel, sand to silty sand and/or silt were encountered from the ground surface in Boreholes BI-16, BI-19, BI-20 and BI-21 and underlying the topsoil in Borehole BI-15. At the bottom of the fill deposit in Borehole BI-20, approximately 0.3 m of cobbles and/or a boulder was encountered at a depth of about 2.1 m, corresponding to Elevation 309.3 m. The fill ranges in thickness from about 0.4 m to 2.7 m.

SPT 'N' values measured within the fill range from 2 to 24 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The grain size distribution of a sample of the sand and gravel fill is shown on Figure B-1 in Appendix B.

The natural water content measured on two samples of the fill was about 3 percent and 20 percent.

4.2.2.3 Peat

From the ground surface in Boreholes BI-17 and BI-18 and underlying the fill materials in Boreholes BI-15, BI-16, BI-19, BI-20 and BI-21, an approximately 0.5 m to 3.4 m layer of moist to wet, brown, very soft to firm peat was encountered. Below the fill materials, the top of the peat was encountered at depths ranging between about 0.6 m and 2.7 m below existing grade, between about Elevations 307.7 m and 310.5 m.

SPT 'N' values measured within the peat range from 1 to 7 blows per 0.3 m of penetration, indicating a very soft to firm consistency.

The natural water content measured on samples of the peat ranges between about 62 percent and 645 percent.

4.2.2.4 Sand to Silty Sand

Beneath the topsoil in Borehole BI-14 and beneath the peat in Boreholes BI-15, BI-17, BI-18 and BI-20, a deposit of sand to silty sand was encountered. The top of the deposit was encountered from about Elevation 305.7 m to 308.9 m and the thickness ranges from about 0.8 m to 5.5 m. The sand deposit was noted



to underlie the clayey silt deposit in Borehole BI-21 at a depth of about 3.8 m below ground surface, corresponding to Elevation 307.9 m. The sand deposit was about 0.5 m thick at this borehole location and Borehole BI-21 was terminated within the sand deposit on split-spoon and auger refusal.

SPT 'N' values measured within these deposits range from 1 to 25 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The test results of grain size distributions performed on samples of the sand to silty sand deposit are shown on Figure B-2 in Appendix B.

The natural water content measured on samples of the sand to silty sand to silt deposits ranges from about 22 percent to 28 percent.

4.2.2.5 Clayey Silt to Silty Clay

Below the peat in Boreholes BI-16 and BI-21, below the sand to silty sand in Boreholes BI-14, BI-15, BI-17, BI-18 and BI-20, and below a 0.1 m thick deposit of silt at Borehole BI-19, a deposit of grey clayey silt to silty clay was encountered. The clayey silt to silty clay was noted to contain silt seams and layers except within the deposit at Boreholes BI-17 and BI-21; the deposit at Borehole BI-21 was noted to contain sand.

The top of this deposit was encountered at elevations ranging from about Elevation 309.6 m to 303.2 m and the thickness ranges from about 1.7 m to greater than 8.8 m. Boreholes BI-16 and BI-17 were terminated in the clayey silt to silty clay deposit.

SPT 'N' values measured within the silty clay to clayey silt range from 0 (i.e. weight of hammer) to 8 blows per 0.3 m of penetration suggesting a very soft to stiff consistency. In situ field vane testing carried out within this stratum measured undrained shear strengths ranging from about 13 kPa to 51 kPa, but typically ranging from about 20 kPa to 40 kPa, indicating a soft to stiff consistency.

Atterberg limits testing carried out on thirteen (13) samples of the clayey silt to silty clay deposit indicate liquid limits ranging from about 19 percent to 36 percent and plastic limits ranging from about 13 percent to 20 percent, yielding plasticity indices ranging from about 7 percent to 17 percent. The results of the Atterberg limits testing are shown on the plasticity charts on Figures B-3a and B-3b, and indicate that the stratum ranges from a clayey silt of low plasticity to a silty clay of intermediate plasticity. A grain size distribution test was carried out on one (1) sample of the clayey silt deposit in Borehole BI-21 and the results are shown on Figure B-4 in Appendix B.

The natural water content measured on select samples of this deposit ranges between 20 percent and 50 percent.

One laboratory consolidation (oedometer) test was carried out on a specimen of the clayey silt obtained from Borehole BI-17 and the test results are shown on Figure B-5. The preconsolidation pressure was estimated from the Void Ratio versus logarithmic Pressure plots using the Casagrande method as well as from the Total Work versus Pressure plots. The relevant consolidation test results are summarized below.



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Borehole / Sample Number	Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	e_o	C_r	C_c	c_v^* (cm ² /s)
BI-17 / 10	301.5	52	150	98	2.9	1.19	0.04	0.29	0.015

Note: * For approximate stress range of $20 \leq \sigma_v' \leq 100$ kPa

where:

- σ_{vo}' effective overburden pressure in kPa
- σ_p' preconsolidation pressure in kPa
- OCR overconsolidation ratio
- e_o initial void ratio
- C_c compression index (based on void ratio)
- C_r recompression index (based on void ratio)
- c_v coefficient of consolidation in cm²/s in the normally consolidated range

4.2.2.6 Silt

Underlying the clayey silt to silty clay stratum in Boreholes BI-14, BI-19 and BI-20, a deposit of silt was encountered. The top of the deposit was encountered from about Elevation 304.4 m to 297.3 m and the thickness ranges from about 1.2 m to greater than 3.6 m. Borehole BI-14 and BI-20 were terminated within the silt layer at depths of 13.1 m (upon casing refusal) and 14.3 m, respectively.

SPT 'N' values measured within the silt deposit range from 2 to 11 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The test results of grain size distributions performed on samples of the silt deposit are shown on Figure B-6 in Appendix B.

The natural water content measured on samples of the silt deposit ranges from about 22 percent to 28 percent.

4.2.2.7 Sand and Gravel

A thin layer of compact, grey sand and gravel was encountered below the clayey silt to silty clay deposit at the bottom of Boreholes BI-15 and BI-18 and below the silt in Borehole BI-19, at depths between about 7.3 m and 13.3 m below existing ground surface, corresponding to between about Elevations 295.8 m and 303.2 m, respectively. The thickness of the layer was between about 0.1 m and 1.0 m.

SPT 'N' values measured within the deposit range from 18 to greater than 100 blows per 0.3 m of penetration, indicating a compact to very dense relative density.

The test results of grain size distributions performed on a sample of the sand and gravel deposit are shown on Figure B-7 in Appendix B.

The natural water content measured on samples of the sand and gravel deposit ranges from about 22 percent to 28 percent.



4.2.2.8 *Bedrock / Refusal*

Boreholes B1-14, BI-15, BI-18, BI-19 and BI-21 were advanced to auger, casing or sampler refusal between depths of about 4.3 m and 13.9 m (Elevation 307.4 m and 295.2 m). These refusal depths, while they do not confirm bedrock surface elevations, may be inferred to indicate potential proximity to the bedrock surface.

4.2.2.9 *Groundwater Conditions*

Details of the groundwater conditions and water level observed in the open boreholes at the time of drilling are summarized on the Record of Borehole sheets following the text of this report. The unstabilized groundwater level observed in the open boreholes was recorded at depths ranging between 0.3 m and 3.4 m below the existing ground surface upon completion of drilling, corresponding to about Elevations 308.0 m to 310.2 m. It should be noted that these water levels do not represent the stabilized water level and that the groundwater elevation will fluctuate seasonally depending on precipitation and local soil permeability and should be expected to rise during wet periods of the year.

5.0 CLOSURE

The field personnel supervising the drilling program were Mr. Ed Savard, Mr. Evan Childerhose and Mr. Tim Rancourt. This report was prepared by Mr. Tim Rancourt, E.I.T., and Mr. André Bom, P.Eng., and the technical aspects were reviewed by Mr. J. Paul Dittrich, Ph.D., P.Eng., an Associate with Golder. A quality control review of the report was provided by Mr. Jorge Costa, P.Eng., Golder's Designated MTO Contact for this project.



Report Signature Page

GOLDER ASSOCIATES LTD.



André Bom, P.Eng.
Geotechnical Engineer

J. Paul Ditrich, Ph.D., P.Eng.
Associate, Senior Geotechnical Engineer



Jorge M.A. Costa, P.Eng.
Principal, Designated MTO Contact

TR/AB/JPD/JMAC/lb

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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides design recommendations on the foundation aspects of the proposed new Highway 11 bridge structure over Blanche River. In addition, design recommendations are provided for the westerly widening realignment of the existing Highway 11 embankment over the low-lying swampy area north of Blanche River to the north project limits (from Station 12+300 to Station 12+650). The recommendations are based on interpretation of the factual data obtained from the boreholes and probehole advanced during the subsurface investigation at the site.

The interpretation and recommendations presented are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. 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.

6.1 General

6.1.1 Bridge Structure and Immediate Approach Embankments

The existing bridge carrying Highway 11 over Blanche River is a timber structure consisting of six spans with an overall deck length of about 29.6 m and width of about 11.5 m. The structure is supported on a combination of wooden abutment posts and lagging and a series of wooden pile bents founded on timber piles likely driven to bedrock. As indicated in LEA's Preliminary Design Report (PDR), at two of the pile bent locations the timber piles are leaning to the west away from the bridge. The existing approach embankments at the abutments are at about Elevation 310.5 m, about 2 m above the existing river water level.

It is unknown if an earlier structure existed at the site prior to the current bridge and details of any older foundations that may be existing in the subsurface are not available.

It is understood that the proposed new bridge will be a single, 30 m long span with the proposed centreline located about 8 m west of the existing bridge centreline. The new south and north abutments are to be located approximately 2 m and 4 m to the south of the existing south and north abutments, respectively. The proposed new bridge will be about 14.1 m wide and will overlap the west side of the footprint of the existing bridge by about 4 m. The top of the new roadway embankment at the approaches to the south and north abutments will be at about Elevation 311.55 m and 311.40 m, respectively, such that a grade raise of approximately 1 m on the existing roadway will be required on both the south and north sides of Blanche River. In addition, as a result of the westerly shift of the roadway centreline at the crossing, the existing roadway embankments will have to be widened requiring up to about 2 m of fill along the west side of the south approach and up to about 4 m of fill along the west side of the north approach.

We understand that the new bridge is to be constructed in two stages. The first stage will involve the construction of the west-half of the proposed bridge after which traffic will be rerouted to this newly constructed "single lane" bridge. The second phase will include the removal of the existing bridge and construction of the remaining east-half of the structure. The east edge of the new "half" structure during the first stage of construction will be located about 2.4 m west of the west side of the existing structure.

The boreholes advanced for the bridge structure and immediate approach embankments generally encountered embankment fill (land boreholes) or sand alluvium (water boreholes) over deposits of silt to sand and/or silty clay to clayey silt overlying bedrock. Peat was encountered underlying the existing embankment fill at several borehole locations. Within the vicinity of the proposed south abutment, at the investigated locations, the total overburden thickness, including fill materials, ranged from about 3.0 m to 5.0 m, with the bedrock or practical



refusal encountered between about Elevation 305.5 m and 306.3 m. Within the vicinity of the proposed north abutment, at the investigated locations, the total overburden thickness, including fill materials, ranged from about 3.6 m to 10.8 m, with bedrock or practical refusal encountered between about Elevation 299.7 m and 304.7 m. The highest elevation of the bedrock at the north abutment was found at the west edge of the proposed abutment (Elevation 304.7 m at Borehole BI-3) and the bedrock slopes downward to the east side of the proposed abutment (Elevation 302.1 m at Borehole BI-12 and Elevation 299.7 m at Borehole BI-13). A probehole (PHBI-1) was advanced between the proposed centreline of the realigned highway and the west side of the proposed north abutment, between Boreholes BI-3 and BI-9 and encountered refusal on probable bedrock at a depth of about 5.2 m, corresponding to about Elevation 302.9 m.

North of the bridge structure, the current design drawings show the new start of highway taper towards the existing centreline at about Station 12+300 (about 50 m north of Blanche River) and joining the existing centreline at about Station 12+668 (about 418 m north of Blanche River). Based on the sections provided by LEA in November 2008, at about Station 12+260, a grade raise of up to about 1 m is required at the proposed embankment centreline and about 3 m of filling is required at the west embankment crest. The grade raise decreases as the ground surface elevation increases northerly and as the proposed centreline transitions into the existing centreline, reducing the proposed embankment widening northerly.

The recommendations on the foundation design aspects of the new structure presented in this report take into consideration the implications of the new “half” bridge foundations and approach embankments construction sequence in relation to the existing bridge foundations and approach embankments.

6.2 Bridge Foundation Options

Given the presence of very loose silt and soft silty clay deposits within the overburden at the location of both of the proposed new bridge abutments, spread footings founded at shallow depth on either the native soil deposits or perched within the existing (or new) embankment fill are not recommended to support the new bridge structure due to the low geotechnical axial resistance and expected settlement of these strata.

At the proposed south abutment, the following foundation alternatives are technically feasible:

- A spread footing founded on the underlying bedrock (located about 3 m to 5 m below the existing ground surface).
- Short caisson foundations, drilled into the strong to very strong bedrock to achieve an adequate socket length.
- Steel H-piles installed in 600 mm diameter holes drilled in the shallow bedrock in an integral abutment configuration.

At the proposed north abutment, steel H-piles driven to bedrock is considered a suitable foundation alternative. An integral abutment design may be considered at this location given the generally greater depth to bedrock, about 5 m to 6.7 m below the underside of the proposed pile cap – understood to be at Elevation 306.4 m as per the email from D.M. Wills Associates Limited (Wills), a subconsultant to LEA, dated November 7, 2008. However, considering that the bedrock was encountered at a relatively shallower depth (only about 1.7 m below the underside of the proposed pile cap) on the west side of the foundation element, it will be necessary to core large (approximately 600 mm) diameter holes about 3.3 m deep into the bedrock on a portion of this side of the abutment to install pile(s) with sufficient length for an integral abutment design. Caisson foundations could be considered to support the abutment, but achieving adequate socket lengths by drilling into the strong to very strong bedrock may be cost prohibitive. Due to the greater thickness of overburden at this location and the proximity of the proposed abutment to Blanche River, a spread footing design founded on the bedrock is not



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considered a practical alternative given that deep excavation (within temporary shoring) and groundwater control would be required for construction.

At the south abutment, higher costs may be associated with the shoring and dewatering required for the shallow foundation on bedrock option given the need for a relatively deeper shoring system and greater degree of dewatering to expose the bedrock surface prior to construction. Based on the above, and in order to achieve a fully integral abutment structure at this site, the structural engineer's preferred foundation alternative for the north and south abutments are steel H-piles installed in 600 mm diameter holes drilled in the bedrock.

To reduce the cost associated with drilling the 600 mm diameter holes in the bedrock to achieve the minimum pile lengths for an integral abutment design, consideration can be given to the installation of polystyrene insulation to reduce the depth of frost penetration and allow the bottom elevation of the pile cap to be raised thereby increasing the effective length of the piles in order to decrease the required depth of drilling into the bedrock. However, in any event, some of the piles for the north abutment may still require that holes be drilled in the bedrock for lateral resistance (see Section 6.4.5).

Temporary shoring and dewatering will be required to facilitate construction of the pile caps at the south and north abutments. The shoring and dewatering requirements are anticipated to be greater at the new north abutment, part of which will be located within the river.

The details of the recommendations for these options are presented in the following sections. A summary of the advantages, disadvantages, relative costs and risks/consequences of the various foundation alternatives considered for this site is presented in Tables 1A (South Abutment) and 1B (North Abutment) following the text of this report.

6.3 Spread Footings (South Abutment)

The south abutment of the bridge may be supported on a spread footing placed on the properly prepared granite or gabbro bedrock. The details of the bedrock surface elevation as encountered in the boreholes at the south foundation element are summarized below.

Foundation Element	Borehole Numbers	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
South Abutment	BI-2, BI-6, BI-7, BI-10 and BI-11	3.0 to 5.0	305.5 to 306.3

Based on the borehole results, there is some variability in the bedrock surface within the limits of the south foundation element. In addition, although the RQD values for the rock core obtained in the boreholes are generally greater than 50% near the bedrock surface, it may be necessary to sub-excavate loose or fractured portions of the upper bedrock from within some areas of the foundation footprint. For design, the following options for founding levels at the south abutment may be considered:

- 1) A founding level at about Elevation 306.3 m may be assumed (highest bedrock elevation encountered in the boreholes):

For this case, following removal of the overburden, the bedrock surface would have to be cleaned and then mass concrete (placed in the dry) would be used to raise the grade to about Elevation 306.3 m to provide a level bearing surface for construction of the footing. Given the variation in the bedrock surface across the foundation element, the mass concrete for this option would be up to about 0.8 m thick. The benefit of this approach is that excavation into the strong to very strong bedrock is avoided; however, dewatering would be required to allow for placement of the mass concrete in-the-dry. This approach is considered to be the preferred option for this site.



- 2) A founding level at about Elevation 305.5 m could be assumed (lowest bedrock elevation encountered in the boreholes):

For this case, following removal of the overburden, excavation of the higher portions of the bedrock would be required within the foundation footprint. Based on the borehole results, sub-excavation of up to about 0.8 m of bedrock would be required. It is noted that the bedrock is classified as strong to very strong (i.e. estimated uniaxial compressive strengths in the range of about 60 MPa to 200 MPa) and it is probable that the level of fracturing in the upper portions of the rock is variable. This will make excavation difficult, particularly in areas where only small depths and narrow zones of removal are needed. As such, bedrock excavation would have to be carried out using line drilling and pre-shearing techniques to reduce the extent of shattering and over-break and to provide better control over the configuration of the founding surface. However, even with these special techniques, excavation of the bedrock will likely be difficult and, as such, this approach is not considered favourable for this site.

A Non-Standard Special Provision (NSSP) should be made in the Contract Documents for additional mass concrete placement to accommodate variations in the bedrock surface (an example is provided in Appendix C). In the areas where mass concreting is employed, it will be necessary to clean, scale and remove any loose debris to ensure a proper bond to the bedrock. In addition, a check on the sliding resistance between the mass concrete and the sloped bedrock should be carried out (in accordance with the recommendations provided in Section 6.3.2).

Given the need to construct the footing in stages (i.e. west half followed by east half as part of the staged construction of the bridge), the use of a compacted Granular 'A' engineered fill pad to provide a level bearing surface and support the footing is not recommended.

Excavation to bedrock will require temporary shoring, based on the subsurface conditions encountered at the borehole locations and given the proximity to the adjacent river. Further, the shoring will be required in two stages: the first stage for the west half of the abutment footing construction and the second stage for the east half after removal of the existing bridge. Recommendations for excavations and temporary shoring are provided in Section 6.10.1.

It is noted that the excavations to expose the bedrock surface at the south abutment will extend through the existing granular embankment fill and native sand and/or silt deposits below the water table/adjacent river level. As such, a suitable dewatering scheme in conjunction with the temporary shoring (possibly a sheet pile cofferdam) will be required to maintain a dry and stable excavation during construction, including placement of mass concrete in-the-dry.

6.3.1 Geotechnical Resistance

Spread footings placed on the surface of the properly prepared granite and/or gabbro bedrock may be designed based on a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 10,000 kPa. For footings placed on a mass concrete pad (constructed in-the-dry), the factored geotechnical axial resistance at ULS is as given above for bedrock assuming that the strength of the concrete used to form the pad is at least 25 MPa. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS, since the granite and/or gabbro bedrock and the mass concrete (placed directly on the bedrock) are considered to be unyielding materials; as such, ULS conditions will govern for this foundation type.

All loose, shattered and/or fractured rock within the footprint of the footings and at the footing level should be removed and replaced with concrete. A provision should be included in the Contract Documents to address the requirements for field inspection of the exposed bedrock. Groundwater control measures would be required in



order to carry out this inspection in the dry. MTO Special Provision 902S01 – Excavation and Backfilling – should be included in the Contract Documents.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary*.

6.3.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the base of the mass concrete and the bedrock should be calculated in accordance with Section 6.7.5 of the *CHBDC*.

The coefficient of friction, $\tan \delta$, may be taken as 0.70 between the base of the concrete footings and/or mass concrete and the bedrock for construction in-the-dry. This value represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

If necessary, the sliding resistance between the concrete footing and/or mass concrete and the bedrock can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a minimum embedded length within the sound bedrock of 1 m, and the structural strength of the dowel and compressive strength of the grout should not be exceeded. If dowelling into bedrock is adopted at this site, an NSSP should be included in the Contract Documents to specify the installation, materials and testing of the dowels (an example is provided in Appendix C).

6.3.3 Frost Protection

For spread footings or mass concrete founded directly on the properly prepared granite and/or gabbro bedrock at this site, frost susceptibility is not an issue. Otherwise, footings should be provided with a minimum of 2.4 m of soil cover for frost protection.

6.4 Steel H-Pile Foundations (South and North Abutments)

As noted in Section 6.2, the abutments will be founded on steel H-piles in order to achieve a fully integral abutment design. As indicated previously, 600 mm diameter holes will be required to be drilled into the strong to very strong bedrock to incorporate adequate pile lengths at the south abutment and also at the west side of the north abutment. We understand that a single row of vertical piles, each with a minimum length of 5 m, is required for the integral abutment design.

If corrugated steel pipes (CSPs) are installed as part of the integral abutment design (through which the piles will be driven), the CSPs should be backfilled with a loose, fine to medium sand. Further, the sand backfill should also be placed within the drilled holes in the bedrock up to the top of bedrock/bottom of CSP. An NSSP detailing the gradation of this sand should be included in the Contract Documents (see example in Appendix C).

For design, the estimated tip elevations for the piles terminating on the bedrock surface or within the 600 mm diameter drilled holes are presented below. The elevations are based on the depth to bedrock encountered in the boreholes put down at, and immediately adjacent to, the area of the proposed abutments, the requirement to



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have a minimum 5 m long pile (for integral abutment design) and an underside of pile cap at Elevation 306.4 m (as discussed previously). The underside of pile cap elevation has been established assuming a minimum frost cover of 2.4 m above the bottom of the pile cap. There should be a provision made in the Contract for dealing with varying pile lengths.

Foundation Unit	Borehole Numbers	Bedrock Surface Elevation (m)	Approximate Design Pile Tip Elevation* (m)	Approximate Depth of 600 mm Diameter Drilling Required (m)
South Abutment	BI-2, BI-6, BI-7, BI-10 and BI-11	305.5 to 306.3	301.4	4.1 to 4.9
North Abutment – West Side	BI-3, BI-8	304.5 to 304.7	301.4	3.1 to 3.3
North Abutment – Centre to East Side	BI-9, BI-12 and BI-13	299.7 to 302.1	299.7 to 301.4	** to 0.7 m

Note: *For underside of pile cap at Elevation 306.4 m (employing conventional soil cover).

**Requirements need to be checked with lateral pile resistance calculations by structural engineer.

At both abutments, we understand that consideration is being given to installing polystyrene insulation to reduce the amount of conventional soil cover required for frost protection, which would raise the elevation of the underside of the pile cap and decrease the amount of bedrock drilling required to achieve the minimum length of piles of 5 m. For design, MTO has adopted a thickness of 25 mm of rigid insulation to be assumed to be equivalent to about 300 mm of soil cover. Therefore, installing a thickness of 75 mm of rigid insulation can reduce the frost protection requirements by about 0.9 m. The details of the requirements for layout and extent of the rigid insulation are described in Section 6.4.6. The estimated pile tip elevations for a design incorporating the rigid insulation and raising the underside of the pile cap to Elevation 307.4 m are presented below.

Foundation Unit	Borehole Numbers	Bedrock Surface Elevation (m)	Approximate Design Pile Tip Elevation* (m)	Approximate Depth of 600 mm Diameter Drilling Required (m)
South Abutment	BI-2, BI-6, BI-7, BI-10 and BI-11	305.5 to 306.3	302.4	3.1 to 4.0
North abutment – West Side	BI-3, BI-8	304.5 to 304.7	302.4	2.1 to 2.3
North abutment – Centre to East Side	BI-9, BI-12 and BI-13	299.7 to 302.1	299.8 to 302.1	**

Note: *For underside of pile cap at Elevation 307.4 m (using 75 mm thick rigid polystyrene insulation).

**Requirements need to be checked with lateral pile resistance calculations by structural engineer.

The piles installed in the 600 mm diameter drilled holes will have to be fixed at the base in a sufficient depth of concrete (to be determined by the structural engineer) to achieve fixity of the lower section of the pile. As such, the pile tip elevations indicated above may have to be lowered or adjusted as required for structural considerations and depth of bedrock drilling indicated above may increase.

Recommendations for excavations for pile cap construction are provided in Section 6.10.1.



6.4.1 Geotechnical Axial Resistance

For HP 310X110 piles driven to practical refusal on the granite and/or gabbro bedrock, a factored axial resistance at ULS of 2,000 kN may be assumed for design. In the case of the driven H-piles, this value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical resistance at SLS for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored axial resistance at ULS, since the granite and/or gabbro bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

6.4.2 Downdrag (Negative Skin Friction)

At the north approach embankment, the loading from the new fill construction (i.e. widening and raising) will cause consolidation settlement of the underlying soft clayey silt to silty clay strata, since this deposit will likely not be sub-excavated as part of stability/mitigation measures (as discussed in Section 6.8.1). If the piles are installed prior to completion of this settlement, because the piles are end-bearing on bedrock, a small amount of settlement of the cohesive deposit relative to the stiff pile will result in the development of negative skin friction on the piles. In this case, downdrag loads will need to be taken into account for design of the piles supporting the abutments.

At the south abutment, since only a relatively thin layer of silty clay was encountered in the boreholes (i.e. 0.7 m in Borehole BI-10), and if an integral abutment design is employed that utilizes 3 m long corrugated steel pipe around the upper portion of the pile, the downdrag loads are expected to be small and can be neglected in the design of the piles at this location.

Where the cohesive foundation soils remain in place below the north abutment and are not preloaded, the structural design of the abutment pile should be based on the full downdrag load acting on the piles. The estimated unfactored downdrag load acting on the HP 310X110 for this case (assuming the underside of the pile cap is at Elevation 307.4 m) may be taken as 175 kN per pile for the case where no CSPs are installed along the upper 3 m portion of the pile and may be taken as 100 kN per pile for the case where CSPs are installed over the upper 3 m portion of the pile.

The downdrag loads noted above are unfactored loads. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section C6.8.4 of the *Commentary to the CHBDC* for ULS conditions.

6.4.3 Set Criteria

For piles to be driven to bedrock (i.e. those not installed in pre-drilled holes), set criteria are highly dependent on pile driving hammer type and the selected pile. The set criteria can be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known. The choice of set criteria is dependent on the experience of the engineer and traditional use where a substantial database has been developed over the years. The criteria need to be set to also avoid overdriving and possibly damaging the piles.

Based on our experience, consideration should be given to the following preliminary criteria. The piles should be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules. On reaching the required set, the hammer energy should be reduced by about 75 percent and the pile should then be re-driven by increasing the hammer energy slowly in stages up to the maximum rated energy over about 40 blows. This procedure is intended to improve the process of the seating of the pile on the sloping bedrock surface. A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy. Provision should be made to re-tap all piles to confirm the set after adjacent piles have been driven.



All pile installation/driving should be in accordance with SP903S01. The piles should be provided with rock points, Titus Injector or equivalent, for adequate seating on the potentially sloping bedrock surface. A NSSP should be included in the Contract Documents to address the requirements for rock points (see example in Appendix C).

6.4.4 Pile Driving Note

The pile driving note to be added to the drawings for this project is Note 4 in Clause 2.5.11 of the Structural Manual:

“Piles to be driven to bedrock”.

6.4.5 Resistance to Lateral Loads

Lateral loads can be resisted fully or partially by the use of battered steel H-piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The evaluation of the piles subjected to lateral loads should take into account such factors as the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moment, the soil resistance that can be mobilized, the tolerable lateral deflection at the head of the pile and the pile group effects.

The pile should be modelled as a beam-column supported by springs equivalent to the passive soil reaction distributed along the shaft. The passive resistance developed for lateral deformations typical of bridge foundations is generally much less than the passive pressure associated with a full passive resistance. This full passive resistance is calculated from earth pressure theories assuming unlimited deformation of the soil. The lateral resistance of the pile may be limited by the factored structural flexural resistance of the pile rather than the resistance of the soil.

Therefore, in order to develop the full passive resistance, the pile would have to deflect a ‘large’ amount. For piles ‘fixed’ within the pile cap, the magnitude of possible deflection is further reduced and the horizontal geotechnical resistance of the pile is some fraction of the full passive resistance occurring at relatively small horizontal displacements.

It can be assumed, based on the shear strength of the soil, that the pile can be considered a laterally supported compression member. The horizontal load capacity of vertical piles may be limited in three different ways:

- The capacity of the soil may be exceeded, resulting in large horizontal movements of the piles and failure of the foundation;
- The bending moments may generate excessive bending stresses in the pile material, resulting in structural failure of the piles; or
- The deflections of the pile heads may be too large to be compatible with the superstructure.

CFEM (1992) gives two methods by which to assess the lateral capacity of a pile. The first is Broms’ Method (1964), which examines failure criteria (i.e. ultimate horizontal resistance) for two types of piles – ‘short piles’ where the lateral capacity of the soil adjacent to the pile is fully mobilized and ‘long piles’ where the bending resistance of the pile is fully mobilized.

The second method examines the lateral deflections of the pile by using the horizontal subgrade reaction theory where the soil around a pile is modelled using a series of springs. The spring constant is called the coefficient of



horizontal subgrade reaction, k_h (kPa/m). The value of k_h is used as an input parameter into the elastic soil-structure interaction model.

The resistance to lateral loading in front of a vertical pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the equation for cohesionless soils given below:

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

where: n_h = the constant of horizontal subgrade reaction (kPa/m)
 z = the depth (m)
 B = the pile diameter or width (m)

and for cohesive soils:

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

where: s_u = the undrained shear strength of the soil (kPa)
 B = the pile diameter or width (m)

It is understood that an integral abutment foundation design is being considered for both the north and south abutments of the bridge. Where the integral design includes the installation of 3 m long CSP liners (with the annular space between the pile and the liner filled with uniform grained, uncompacted sand), the upper portion of the H-piles will be free to flex and move laterally. With this design, the passive lateral resistance over the length of the CSP liner should be neglected.

At the south abutment, assuming the underside of pile cap is at Elevation 307.4 m and assuming the use of 3 m long CSP liner around the piles (i.e. extending to the top of bedrock), the lateral resistance of the piles will be developed primarily from the fixity (presumably in concrete) at the base of the 600 mm diameter drilled holes. In this case, the structural resistance of the pile will govern the lateral resistance.

At the north abutment, the lateral resistance of the piles will be developed from the passive resistance of the soil over the portion of the piles below the CSP liners. The values of n_h and s_u to be used to calculate coefficient of horizontal subgrade reaction (k_h) to be assumed in the structural analysis for the piles at this location are given in Table 2. The different values reflect the variability in the subsurface conditions below the north abutment. For a single HP 310x110 pile surrounded by a 3 m long CSP liner below the pile cap and embedded about 2.5 m to 3.5 m into the soft clayey silt to silty clay below the CSP, the estimated maximum lateral resistance at ULS is less than about 10 kN and at SLS for 10 mm of deflection is less than about 2 kN. These very low lateral resistance values are due to the short embedded pile length in the soft silty clay below the base of the CSP. In order to achieve sufficient lateral resistance of the piles in an integral abutment configuration, it will be necessary to embed/socket the base of the piles into the bedrock (i.e. within concrete filled 600 mm diameter drilled holes) to achieve fixity. Alternatively, a non-integral abutment design (i.e. without the use of 3 m long CSPs) could be considered.

For a single HP 310x110 pile embedded about 5.5 m to 6.5 m through the lower embankment fill and into the underlying soft clayey silt to silty clay (i.e. without a 3 m long CSP below the pile cap), the estimated maximum lateral resistance at ULS is about 25 kN and at SLS for 10 mm of deflection is about 20 kN.



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Based on the above discussion, it is considered that both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS. At SLS, the horizontal resistance of the piles will be controlled by deflections and the horizontal resistance of the pile should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil as discussed above. The SLS resistance should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting abutments (*CHBDC Commentary C6.8.7.1*).

The upper zone of soil (down to a depth below the pile cap equal to about $1.5 \times B$ after Broms 1964, where B = pile diameter) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

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

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

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

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

6.4.6 Frost Protection

The pile caps should be provided with a minimum of 2.4 m of conventional soil cover for frost protection (per OPSD 3090.100). Where rock fill is employed as a cover material, the minimum cover thickness required will be approximately twice that of a conventional soil cover given the open nature of the rock fill structure.

Alternatively, rigid polystyrene insulation could be used to reduce the required thickness of soil cover. As a guideline for design, the MTO has adopted a thickness of 25 mm of rigid polystyrene insulation assumed to be equivalent to about 300 mm of soil cover. We understand that the design of the pile cap calls for 1.5 m of conventional soil cover only and therefore a thickness of 75 mm of rigid polystyrene insulation should be provided to satisfy the frost protection requirements.

It is recommended that the rigid insulation be placed in all areas around the perimeter of the pile cap where less than 2.4 m of conventional soil cover is available. The insulation should be placed vertically along the face of the abutment stem (to the base of the pile cap) and extend horizontally for a distance of 2.4 m beyond the face of the abutment. A minimum of 1 m of soil cover should be placed over the rigid insulation and the installation should be checked for buoyancy/uplift at the high water level condition.



6.5 Caissons (South and North Abutments)

Consideration could be given to the use of caissons for support of the north and south abutments. Caissons would have an advantage over H-piles in that the length of the rock socket/drill hole into the bedrock (about 2 m deep) would be generally less than that being considered to achieve the integral abutment design to accommodate sufficient length of H-pile (about 3 m to 4 m). Although the socket diameter for the caisson option (1.0 m to 1.5 m) would be larger than that for the H-pile option (0.6 m), the high axial capacity of the caissons would result in fewer piles being required to support the abutment than that required for the H-pile design and therefore less overall bedrock drilling. It should be noted, however, that there may be difficulty in socketting the caissons within the strong to very strong bedrock, particularly where the bedrock surface is steeply sloping (i.e. on the west side of the north abutment) or if the bedrock is fractured. Temporary liners and tremie concrete will likely be required to install caissons at this site.

6.5.1 Geotechnical Axial Resistance

If caissons are considered as a founding alternative, the caissons at this site will derive their axial resistance mainly from the shaft resistance of the rock socket. The contribution from end-bearing will be neglected due to the difficulties in cleaning and inspecting the base of the sockets. The factored geotechnical axial resistance at ULS for two diameter caissons socketted a minimum of 2 m into the bedrock are given below:

Caisson Diameter (m)	Granite and/or Gabbro Bedrock (minimum 2 m socket)	
	ULS (kN)	SLS for 25 mm
1.0	4,750	n/a
1.5	7,000	n/a

The resistance required to achieve 25 mm of settlement is greater than that given for ULS for caissons socketted into the bedrock and, therefore, SLS conditions do not apply.

6.5.2 Downdrag Loads

As discussed in Section 6.4.2, the loading from the new fill construction (i.e. widening and raising) at the north approach embankment will cause settlement of the underlying soft clayey strata which will result in the development of negative skin friction on the caissons that are socketted into bedrock. As a result, downdrag loads will need to be taken into account in the design of the caissons at the north abutment.

At the south abutment, since only a relatively small amount of silty clay was encountered in the boreholes, the downdrag loads are expected to be small and can be neglected in the design of the caissons at this location.

Where the cohesive foundation soils remain in place below the north abutment and are not preloaded, the structural design of the abutment caissons should be based on the full downdrag load acting on the caissons. The estimated unfactored downdrag load acting on the caissons for this case (assuming the underside of the caisson cap is at Elevation 307.4 m) may be taken as follows:



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Caisson Diameter (m)	Unfactored Downdrag Load (kN)
1.0	450
1.5	675

The downdrag loads noted above are unfactored loads. The structural capacity of the caissons must be checked for the factored dead and downdrag loads in accordance with Section C 6.8.4 of the *Commentary to the CHBDC* for ULS conditions.

6.5.3 Resistance to Lateral Loads

The geotechnical resistance to lateral loading for the caissons should be calculated in accordance with Section 6.4.5 and Table 2, using the horizontal subgrade reaction formulas. The recommended maximum lateral resistances for the caissons are as follows:

Caisson Diameter (m)	Factored Lateral Resistance at ULS* (kN)	Lateral Resistance at SLS (kN)
1.0	2,500	200
1.5	5,000	800

*Note: or the subsurface stratigraphy at this site, the factored lateral resistance at ULS for the caissons is controlled by the UCS of the bedrock socket.

6.5.4 Frost Protection

The pile caps for the caissons at the abutments should be provided with a minimum of 2.4 m of conventional soil cover for frost protection or sufficient insulation as described in Section 6.4.6.

6.6 Site Coefficient

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 1.5, consistent with Soil Profile Type III.

6.7 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.



6.7.1 Static Lateral Earth Pressures

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

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) 1010 Granular 'A' or Granular 'B' Type II but containing less than 5 percent passing the No. 200 sieve size should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 percent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. 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 Ontario Provincial Standard Drawings (OPSD) 3101.150 and 3121.150.
- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specifications as outlined in the Northern Region Directive for backfill to structures adjacent to rock fill embankments, dated November 2002. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200.
- 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 OPSS 501.06 or SP 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 2.4 m behind the back of the wall stem (as outlined on Figure C6.20(a), Case I, of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the base of the footing/pile cap (as outlined in Figure C6.20(b), Case II, of the *Commentary to the CHBDC*).
- For Case I, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of earth fill or rock fill:

	Earth Fill 21 kN/m ³	Rock Fill 19 kN/m ³
Soil unit weight:		
Coefficients of static lateral earth pressure:		
Active, K_a	0.31	0.22
At rest, K_o	0.47	0.36

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

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



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Where lightweight, expanded polystyrene (EPS) fill is installed behind the abutment wall, the pressure acting over the depth of the EPS may be calculated as follows:

- EPS unit weight: 0.5 kN/m^3
- Coefficients of static lateral earth pressure:
 - Active, K_a 0.11
 - At rest, K_o 0.11

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 (in accordance with Section C6.9.1 and Table C6.6 of the *Commentary to the CHDBC*):

- 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.

A restrained structure is typically a concrete box culvert or a rigid frame bridge where the rotational and/or horizontal movement is not sufficient to mobilize the active earth pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

6.7.2 Dynamic Lateral Earth Pressures

The potential for seismic (earthquake) loading must also be considered for the design of abutment stems/retaining walls in accordance with Section 4.6 of the *CHDBC*. In this regard, the following should be taken into account in the lateral earth pressures.

- Seismic loading may result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to Table A3.1.7 of the *CHBDC*, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio for the Kirkland Lake area is 0.05. Based on experience, for the subsurface conditions at this site, a 50 percent amplification of the ground motion may occur (i.e. Site Coefficient, $S=1.5$), resulting in an increase in the ground surface acceleration from 0.05 g to 0.075 g (PHA).

It is understood from correspondence with LEA that this highway route/bridge is not designated as a lifeline bridge. As such, based on Section 4.4.4 of the *CHBDC*, this bridge structure is assigned to Seismic Performance Zone 1. Given this, and in accordance with Section 4.4.5.1 of the *CHBDC*, structures located in Seismic Performance Zone 1 need not be analysed for seismic loads.



6.8 Approach Embankment Design and Construction

As part of the replacement of the Blanche River structure, the existing Highway 11 alignment will be shifted to the west by approximately 8 m. In addition, the grade of the existing highway will be raised to about Elevation 311.55 m and 311.4 m at the centreline of the proposed south approach and north approach, respectively. As a result of the grade raise and alignment shift, the existing roadway embankments will be widened by approximately 10 m to the west and raised by approximately 0.9 m and 2.1 m (at the centreline) and by approximately 2.1 m and 4.2 m (at the west crest) of the south and north approaches, respectively.

The north abutment of the new bridge will be relocated about 4 m south of the existing north abutment, requiring filling into the river at the front and west slopes. The south abutment of the new bridge will be relocated about 2 m south of the existing south abutment, further away from the water's edge.

The following sections present the results of settlement and stability analysis for the new immediate approach embankments and the section of new widened roadway embankment to the north of the bridge, including recommendations for settlement and stability mitigation measures, as required.

6.8.1 Stability

Analyses were performed on the critical (i.e. highest or thickest fill) sections of the proposed new approach embankments as well as the sections of the new widened embankment to the north of the bridge to assess the stability and liquefaction potential for the proposed heights and geometries. Critical sections include those through both the front slopes (into the river) and side slopes (into the river or through the existing embankments) of the new immediate approaches. For the embankment widening to the north of the bridge, the critical sections included the following:

- Station 12+293 (2 m high), in the vicinity of Borehole BI-14;
- Station 12+433 (1.5 m high), in the vicinity of Borehole BI-18;
- Station 12+493 (0.75 m high), in the vicinity of Borehole BI-19; and
- Station 12+613 (0.5 m high), in the vicinity of Borehole BI-20.

The geometry of the existing approach embankments (including heights and side slope profiles) has been included in the analyses based on the information provided by LEA.

6.8.1.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2004 (Version 6.20), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety of numerous potential failure surfaces was computed in order to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum Factor of Safety of 1.3 is normally adopted for the design of embankment slopes under static conditions. This Factor of Safety is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum Factor of Safety was achieved for the design embankment height and geometries. In general, circular slip surfaces were analysed in the design.



6.8.1.2 *Parameter Selection*

The soils encountered below the proposed south approach embankment consisted primarily of cohesionless soils immediately behind the abutment and cohesive soils further back under the approach embankment and the soils below the proposed immediate north approach embankment consisted of a combination of cohesionless and cohesive soils. The soils encountered below the proposed north embankment widening consisted of a combination of cohesionless and cohesive soils underlying peat and/or roadway fill materials.

For the cohesionless layers, effective stress parameters were employed in the analysis assuming drained conditions and the shear strength parameters were estimated from empirical correlations using the results of the in situ SPT. The correlations proposed by Peck et al. (1974), Schmertmann (1975) and US Navy (1971) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

For the cohesive layers, total stress parameters were employed in the analysis. The total stress parameters (i.e. average mobilized undrained shear strength – s_u) for the cohesive soils were assessed based on the results of the in situ field vane tests and estimated from correlations with the SPT results and other laboratory test data. Where appropriate, Bjerrum's correction factor (1973) as a function of the plasticity index of the soil was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests.

At all areas, the analyses assume that prior to construction of the new or widened embankments, all surficial organic soils, peat deposits and the sand alluvium at the bottom of the river will be removed, including the peat encountered below the fill materials of the embankment to the north of the bridge. The existing embankment fill materials (as well as the thin deposits of peat underlying the existing fills) will be left in place. Further, the analyses assume that below the river bed, soft clayey silt to silty clay is present.

Along the north embankment, the analysis assumes that the peat was generally removed during construction of the existing roadway embankment and the roadway embankment was constructed of granular fill. A total of 16 boreholes were advanced within the pavement structure of the existing roadway during the pavement investigation to depths ranging between 1.1 m and 3.8 m below existing asphalt surface. The borehole at Station 12+475 (proposed centreline) encountered peat below the pavement structure from a depth of about 1.1 m to 1.8 m below existing asphalt surface and the borehole was terminated within the peat. It has been assumed that the existing roadway embankment granular fill will remain in place and have similar properties (unit weight and strength) of the new fill based on the results of the pavement investigation boreholes and the length of time the roadway embankment has been in use.

The piezometric conditions required in the analyses were assessed based on the water levels observed in Blanche River in July and August 2008 (Elevation 308.4 m to 308.1 m) as well as the groundwater levels noted during drilling of the boreholes in and immediately adjacent to this area and the water levels measured in the piezometers. The water level in the piezometer installed at the south approach (in Boreholes BI-1) was measured at about Elevation 308.6 m on September 4, 2008 and at the north approach (in Borehole BI-4) was measured at about Elevation 308.4 m on September 3, 2008. For design purposes in our analysis, the groundwater level at the immediate approaches has been assumed to be consistent with the adjacent low river level, at about Elevation 308.1 m. For the north embankment, the piezometric conditions employed in the analyses are assumed to be the same elevation/depth as the groundwater levels noted during drilling of the boreholes (at about Elevation 308.7 m).

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the approach areas. For the purpose of analysis, both earth fill and rock fill have been considered for the construction of the approach embankments as indicated below. Earth fill is assumed to have side slopes at 2H:1V and rock fill is assumed to have side slopes at 1.25H:1V. A discussion on the different fill types, with respect to stability, is provided in Section 6.8.1.5.



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South Approach Embankment

Soil Type	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Angle of Internal Friction
New Earth Fill Above Water Level (Sand and Gravel)	21	--	35°
New Earth Fill Below Water Level (Sand and Gravel)	20	--	30°
New Rock Fill	19	--	40°
Existing Granular Fill (Very Loose)	20	--	28°
Silt (Very Loose to Loose)	18	--	27°
Clayey Silt to Silty Clay (below existing embankment, Soft)	18.5	17	--
Clayey Silt to Silty Clay (outside of existing embankment, Soft)	18.5	13	--

Note: Groundwater and Blanche River water level assumed to be at Elevation 308.1 m.

Immediate North Approach Embankment

Soil Type	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Angle of Internal Friction
New Earth Fill Above Water Level (Sand and Gravel)	21	--	35°
New Earth Fill Below Water Level (Sand and Gravel)	20	--	30°
New Rock Fill	19	--	40°
Existing Granular Fill (Very Loose)	20	--	28°
Granular Alluvium (Very Loose)	17	--	28°
Silt (Very Loose to Loose)	18	--	27°
Clayey Silt to Silty Clay (below existing embankment, Soft)	18.5	17	--
Clayey Silt to Silty Clay (outside of existing embankment, Soft)	18.5	13	--
Sand to Silty Sand (Very Loose-Upper, Loose to Compact-Lower)	20	--	28° – upper 30° – lower

Note: Groundwater and Blanche River water level assumed to be at Elevation 308.1 m



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North Embankment

Soil Type	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Angle of Internal Friction
New Earth Fill Above Water Level (Sand and Gravel)	21	--	35°
New Earth Fill Below Water Level (Sand and Gravel)	20	--	30°
New Rock Fill	19	--	40°
Existing Granular Fill below Existing Roadway	20	--	28°
Sand to Silty Sand (Very Loose to Compact)	20	--	28°
Clayey Silt to Silty Clay (Soft)	18.5	20	--
Silt (Very Loose to Compact)	18	--	27°
Sand and Gravel (Compact to Very Dense)	20	--	35°

6.8.1.3 Results of Analysis

The results of the stability analyses for the two embankment fill options (earth and rock fill) are summarized below for the south and north approach embankments including front slopes (towards the river) as well as side slopes. The minimum Factor of Safety is based on a deep-seated, global trial failure surface that would impact the operation of the roadway.

Location	Embankment Height at Critical Section above water surface (m)	Earth Fill Option		Rock Fill Option	
		Recommended Side Slope Profile	Minimum Factor of Safety	Recommended Side Slope Profile	Minimum Factor of Safety
South Approach (front, west and east side slopes)	3	2H : 1V	≥ 1.3	1.25H : 1V	≥ 1.3
North Approach (east side slope)	3				
North Embankment	3				
North Approach (west slope)	3	(see Section 6.8.1.4)			
North Approach (front slope)	3				



As discussed in the following sections, a stabilizing berm is required at the front slope and west slope of the north approach embankment to achieve the required minimum Factor of Safety.

6.8.1.4 North Approach – Front and West Slope Stability

Due to the location of the proposed north abutment being approximately 4 m south of the existing abutment, and as a result of the required roadway grade raise and westerly alignment shift, filling into the river will be required for the new north approach embankment. The majority of the fill placement will be on the west side of the north approach, as the existing river shoreline is located near the centreline of the new proposed north abutment (see Drawing 1).

The proposed wider and higher embankment fill constructed over the soft silty clay strata at the north abutment area will create instability of the west slope and front slope of the new north approach embankment. It should be noted that based on discussions with LEA, it is understood that granular earth fill is the preferred fill for use in the new embankment construction at this site. As such, all analyses of stability mitigation measures have considered the use of earth fill (sand and gravel) only. Limit equilibrium analysis indicates a Factor of Safety of about 1.0 for the critical sections of the front and west slopes assuming construction at 2H:1V profiles with granular earth fill.

To achieve a Factor of Safety of 1.3 or greater for the new, up to 4 m high, embankment fill in this area, stability analyses indicate that berms approximately 8 m wide and 1 m above water surface (i.e. to Elevation 309.4 m) would be required at the toe of the front slope and of the west slope of the approach as shown on Figures 1 and 2, respectively. Toe berms of this size would encroach about halfway into the existing Blanche River channel. Alternatively, other stability mitigation measures could be considered including using lightweight fill, staged construction and sub-excavation and removal of the weak/soft soils.

As discussed in Section 6.8.3, the soft clayey strata in this area will also cause some (consolidation) settlement of the new approach fill. Some of the measures that can be considered to mitigate the stability problems will also help minimize the long-term (i.e. post-construction) settlements. The alternatives to mitigate stability and settlement are discussed in Section 6.8.4. The advantages, disadvantages, relative costs and risks/consequences for the stability and settlement mitigation options at this area are summarized and ranked in Table 3. The lightweight (EPS) fill option is considered the preferred alternative for this location.

6.8.1.5 Embankment Fill Types and Benching Requirements

The different embankment fill alternatives (i.e. granular earth fill and rock fill) provide relative advantages and disadvantages in terms of weight (i.e. driving force and applied load to founding subsoils/bedrock), construction cost and time, and ease of construction/availability.

6.8.1.5.1 Earth Fill

The main advantage of using earth fill (i.e. sand and gravel) for embankment construction is the ease of placement and compaction and the negligible amount of post-construction settlement that will occur within the fill embankment itself. However, this option will require a larger volume of fill and wider right-of-way because the side slopes will be flatter than for rock fill slopes. For this project, acceptable earth fill is considered to be well graded, locally available and/or imported, granular material.



6.8.1.5.2 Rock Fill

The main advantage of using rock fill is the ability to achieve steeper embankment side slopes. This is useful in areas with a limited right-of-way, which we understand is not of concern at this site. Rock fill would likely be available locally. The disadvantage of using rock fill for the construction of high embankments is that some post-construction settlement of the embankment fill itself will occur within about the first year of construction.

6.8.2 Liquefaction Potential and Seismic Analysis

As noted in Section 6.7.2, this site is located in Seismic Zone 1 with a $PHA < 0.08$. Further, the bridge structure is not a lifeline structure. As such, based on Section 4.4.4 of the *CHBDC*, the site is assigned a Seismic Performance of 1 and, therefore, in accordance with Section 4.4.5.1 of the *CHBDC*, no liquefaction analysis is required.

6.8.3 Settlement

Settlement of the approach embankments can be expected as a result of the loading from the new fills on the compressible foundation soils at this site, including the existing embankment fill material. In addition, depending on the type of fill materials employed in the construction, settlements may also occur due to compression of the new embankment fill itself.

6.8.3.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach embankments using either the commercially available program *Settle*^{3D} (by Rocscience Inc.), hand and/or spreadsheet calculations. The rate of settlement of the cohesive foundation soils was assessed using Terzaghi's one-dimensional consolidation theory.

For the settlement analyses, the critical sections were assessed considering the location of the following at each approach area:

- the greatest new embankment height; and/or
- the thickest cohesive deposit.

Based on the above, the analyses were carried out at the following critical sections:

- South Approach (West Crest of proposed embankment, approximate Station 12+206 - near the vicinity of Borehole BI-7) - with a new embankment height of approximately 2.3 m over existing cohesionless and cohesive fill materials, underlain by peat and native cohesionless soil deposits;
- South Approach (Centreline of proposed embankment, about 20 m south of river, approximate Station 12+186 - near the vicinity of Borehole BI-1) - with a new embankment height of about 2 m over the native cohesive deposit;
- North Approach (Centerline of proposed embankment, approximate Station 12+236 - near the vicinity of BI-9) - with a new embankment height of about 2 m over the existing embankment fill and native cohesive deposits;



- North Approach (West Crest of proposed embankment, approximate Station 12+246 - near the vicinity of BI-5) - with a new embankment height of up to 4.2 m (from bottom of the alluvium in the river bed) over native cohesive deposits; and
- The embankment widening at Stations 12+400 and 12+425 (Boreholes BI-17 and BI-18) – with a new embankment height up to 5 m from the bottom of the peat underlain by a cohesive deposit up to about 8.6 m thick.

6.8.3.2 Parameter Selection

At the immediate south approach, the foundation soils are composed primarily of existing embankment fill overlying soft to firm silty clay/clayey silt or very loose to compact silt. At the immediate north approach, the foundation soils are composed of varying thicknesses of existing embankment fill overlying deposits of very loose to loose sand and/or silt, soft silty clay/clayey silt and loose to compact silty sand. At the north embankment, below the proposed embankment widening, the foundation soils are composed primarily of very loose to compact sand to silty sand underlain by soft to stiff clayey silt to silty clay, very loose to compact silt and/or very dense sand and gravel.

The immediate compression of the existing very loose to loose embankment fill material and the very loose to compact silt and sand layers was assessed by estimating an elastic modulus of deformation based on the SPT 'N'-values and empirical correlations found in literature by Bowles (1984) and Kulhawy and Mayne (1990).

The consolidation settlement of the soft to firm silty clay to clayey silt layers was assessed using the results of the in situ field vane and SPT tests and/or laboratory consolidation tests to estimate the deformation parameters for these soils. In addition, the results of the laboratory index testing were also employed to estimate deformation parameters using empirical correlations proposed in literature by Terzaghi and Peck (1967), Kulhawy and Mayne (1990), Azzouz et al. (1976) and Britto and Gunn (1987).

The degree of over-consolidation in the cohesive strata, required in the analyses, was estimated from the results of the in situ field vane tests and the following correlations relating mobilized undrained shear strength to preconsolidation pressure:

$$s_{u(mob)} = 0.22\sigma_p' \text{ (after Mesri, 1975)}$$

where: $s_{u(mob)}$ = average mobilized undrained shear strength (kPa)
 σ_p' = preconsolidation pressure (kPa)

and

$$s_{u(mob)} = \mu s_{u(FV)} \text{ (after Bjerrum, 1973)}$$

where : $s_{u(mob)}$ = average mobilized undrained shear strength (kPa)
 $s_{u(FV)}$ = undrained shear strength from field vane test (kPa)
 μ = Bjerrum's correction factor based on Plasticity Index (i.e. about 1 for this site since PI is less than about 20 percent)



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The unit weights and embankment geometry/slope profiles for the new fill described in Section 6.8.1 were employed in the analyses. The analyses performed assume that all peat, organic soils/topsoil/alluvium will be removed prior to construction and that earth fill (as instructed by LEA and assuming sand and gravel) has been used for the new embankment construction.

At the immediate north and south approaches, the piezometric conditions required in the analyses were based on the groundwater levels noted during drilling and measured in the piezometers. In general, the groundwater level was assumed to be at the same level as Blanche River in July 2008, at Elevation 308.4 m. At the north embankment, the piezometric conditions required in the analyses were based on the groundwater levels noted during drilling at Borehole BI-17, at about 0.3 m below existing ground surface (Elevation 308.7 m).

The following sections summarize the simplified stratigraphy, unit weights and deformation parameters employed for the different soils types in the approach areas. The maximum estimated settlement of the foundation soils in these areas (due to the loading imposed by the new approach embankment fills) is presented and a discussion on the rate of settlement is included.

6.8.3.3 Settlement of Foundation Soils

6.8.3.3.1 South Approach – West Crest of Proposed Embankment (at abutment)

Based on the variable subsurface conditions encountered in the boreholes advanced at the south abutment area (including existing cohesionless and cohesive fill materials overlying peat, silty clay, silt, sand and sandy silt), the settlement of the proposed new higher and wider approach embankment is difficult to predict at this location. As such, the following simplified stratigraphy and deformation parameters have been conservatively estimated and employed in the settlement analysis of the proposed 2.3 m high earth fill embankment at the west side of the south approach in the abutment area.

Soil	Thickness (m)	Unit Weight (kN/m ³)	Estimated Deformation Properties
New Earth Fill (2.3 m embankment)	2.3 (high)	21	-
Existing Embankment Fill (Sandy to Silty Sand)	1.0	20	E' = 5 MPa
Existing Embankment Fill (Clayey Silt)	1.2	19	E' = 2 MPa
Soft to Very Soft Peat	0.5	13	$m_v = 3.4 \times 10^{-3} \text{ kPa}^{-1}$
Firm Silty Clay	0.7	18	E' = 3 MPa
Very Loose to Compact Sand to Sandy Silt to Silt	1.1	20	E' = 3 MPa

Based on the results of the settlement analysis, the total settlement of the foundation soils in the approach embankment immediately south of the abutment is estimated to be up to about 75 mm. This total settlement is estimated to be comprised of about 35 mm of immediate settlement due to compression of the cohesionless soil layers and about 40 mm of consolidation settlement of the cohesive soil layers and organic peat deposits.



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Assuming a coefficient of consolidation (c_v) of about $3.2 \times 10^{-3} \text{ cm}^2/\text{s}$ (based on empirical correlations with liquid limit using US Navy (1971) for a normally consolidated soil) and assuming two-way drainage of the approximately 1.2 m thick clayey silt/peat layer, it is estimated that about 90 percent of the consolidation settlement will be completed in less than about 1 month.

The magnitude of creep settlement for the silty clay/clayey silt and/or peat strata is expected to be negligible (i.e. less than about 5 mm per log-cycle of time) at this location.

In order to mitigate the above noted settlement, consideration should be given to preloading the embankment fill area or removing the fill materials (i.e. very loose silty sand fill and very soft to soft clayey silt fill encountered at Borehole BI-7) and the underlying peat to expose the native cohesionless deposits prior to embankment construction, as discussed in Section 6.8.4.

6.8.3.3.2 South Approach – Centreline of Proposed Embankment (approximately 20 m south of abutment)

At Borehole BI-1, approximately 20 m south of the proposed south abutment, about 0.6 m of sandy topsoil was encountered overlying about 4.9 m of clayey silt to silty clay. The height of the embankment at this location will be about 2 m after removal of the surficial topsoil. The following simplified stratigraphy and deformation parameters have been employed in the settlement analysis of the proposed 2 m high earth fill embankment in the south approach area.

Soil	Thickness (m)	Unit Weight (kN/m^3)	Estimated Deformation Properties
New Earth Fill (1.4 m embankment + removal of 0.6 m of organics)	2.0 (high)	21	-
Soft to Firm Silty Clay	4.9	19	(see below)

The following consolidation parameters were estimated for the silty clay to clayey silt deposit based primarily on the results of a laboratory consolidation test performed on a specimen of the silty clay obtained from Borehole BI-1a. These results were compared with values estimated from empirical correlations using the results of the in situ tests and laboratory index testing as described previously.

Location	Elevation (m)	σ'_{vo} (kPa)	σ'_p (kPa)	OCR	e_o	C_r	C_c	c_v (cm^2/s)
South Approach (20 m South of River)	309.3 to 304.4	29	70	2.4	0.85	0.07	0.24	0.012

Note: Elevations given are for top and bottom of the clayey silt to silty clay layer and the values are estimated at the middle of the silty clay to clayey silt layer.

Based on the results of the settlement analysis, the total consolidation settlement of the foundation soils in this area of the south approach is estimated to be up to about 75 mm.

Assuming a coefficient of consolidation (c_v) of $1.2 \times 10^{-2} \text{ cm}^2/\text{s}$ (as noted above from the results of the laboratory consolidation test) and assuming two-way drainage of the approximately 4.9 m thick cohesive deposit, it is estimated that about 90 percent of the consolidation settlement will be completed in about 2 months.



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In order to mitigate the above noted settlement, consideration should be given to preloading the embankment fill area, as discussed in Section 6.8.4.

6.8.3.3.3 North Approach – Centreline of Proposed Embankment (from abutment up to 20 m north)

At boreholes BI-4, BI-9, BI-12 and BI-13 located along the centreline and east side of the new approach embankment, the height of the new embankment fill will be about 2 m over the existing granular fill, underlain by deposits of soft to firm clayey silt to silty clay and very loose to loose sand.

The following simplified stratigraphy and deformation parameters have been employed in the settlement analysis of the proposed 2 m high earth fill embankment in the north approach area.

Soil	Thickness (m)	Unit Weight (kN/m ³)	Estimated Deformation Properties
New Earth Fill (2.0 m embankment)	2.0 (high)	21	-
Existing Embankment Fill (Sandy to Silty Sand)	2.8	20	E' = 7 MPa
Soft to Firm Clayey Silt to Silty Clay	5.4	18	(see below)
Very Loose Sand to Silty Sand	3.0	20	E' = 5 MPa

The following consolidation parameters were estimated for the silty clay deposit based on empirical correlations using the results of the in situ tests and laboratory index testing as described previously. In addition, the parameters were compared with the results of the laboratory consolidation test performed on a specimen of the silty clay/clayey silt obtained from Borehole BI-8a.

Location	Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	OCR	e_o	C_r	C_c	c_v (cm ² /s)
North Approach – Centre	306.8 to 301.4	63	70	1.1	0.82	0.06	0.20	0.004

Note: Elevations given are the range for top and bottom of the clayey silt to silty clay layers and the values are estimated at the middle of the clayey silt to silty clay layer.

Based on the results of the settlement analysis, the total settlement of the foundation soils in the north approach area (along the centreline) is estimated to be up to about 160 mm. This total settlement is estimated to be comprised of about 40 mm of immediate settlement due to compression of the cohesionless soil layers and about 120 mm of consolidation settlement of the cohesive soil layers.

Assuming a coefficient of consolidation (c_v) of about 4.0×10^{-3} cm²/s (as noted above from the results of the laboratory consolidation test) and assuming two-way drainage of the approximately 5.4 m thick silty clay/clayey silt layer, it is estimated that about 90 percent of the consolidation settlement will be completed in about 6 months.

The magnitude of creep settlement for the silty clay/clayey silt strata is expected to be small (i.e. less than about 15 mm per log-cycle of time) at this location.



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In order to mitigate the above noted settlement, consideration should be given to preloading the embankment fill area and/or constructing a portion of the embankment with lightweight fill, as discussed in Section 6.8.4.

6.8.3.3.4 North Approach – West Crest of Proposed Embankment (from abutment up to 20 m north)

At Boreholes BI-3, BI-5 and BI-8 located along the west side of the new north approach embankment, the height of the new embankment fill will be up to about 4.2 m over the river bed and underlying deposits sandy, silt and silty clay/clayey silt.

The following simplified stratigraphy and deformation parameters have been employed in the settlement analysis of the proposed 4.2 m high earth fill embankment at the west slope of the north approach area.

Soil	Thickness (m)	Unit Weight (kN/m ³)	Estimated Deformation Properties
New Earth Fill (3.6 m embankment + removal of 0.6 m of alluvium)	4.2 (high)	21	-
Very Loose Silt	1.2	17	E' = 2 MPa
Soft Silty Clay/Clayey Silt	3.0	18	(See Below)
Loose to Compact Silty Sand	3.2	20	E' = 5 MPa

The following consolidation parameters were estimated for the silty clay/clayey silt deposit based primarily on the results of a laboratory consolidation test performed on a specimen of the silty clay obtained from Borehole BI-8a. These results were compared and adjusted based on the values estimated from empirical correlations using the results of the in situ tests and laboratory index testing as described previously.

Location	Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	OCR	e_o	C_r	C_c	c_v (cm ² /s)
North Approach – West Side	305.8 to 302.8	25	25	1.0	1.03	0.06	0.20	0.004

Note: Elevations given are the range for top and bottom of the clayey silt to silty clay layers and the values are estimated at the middle of the clayey silt to silty clay layer.

Based on the results of the settlement analysis, the total settlement of the foundation soils along the west side of the north approach area is estimated to be up to about 285 mm. This total settlement is estimated to be comprised of about 90 mm of immediate settlement due to compression of the cohesionless soil layers and about 195 mm of consolidation settlement of the cohesive soil layers.

Assuming a coefficient of consolidation (c_v) of about 4.0×10^{-3} cm²/s (as noted above from the results of the laboratory consolidation test) and assuming two-way drainage of the approximately 3 m thick silty clay/clayey silt layer, it is estimated that about 90 percent of the consolidation settlement will be completed in about 2 months.

The magnitude of creep settlement for the silty clay/clayey silt strata is expected to be small (i.e. less than about 15 mm per log-cycle of time) at this location.



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In order to mitigate the above noted settlement, consideration should be given to preloading the embankment fill area and/or constructing a portion of the embankment with lightweight fill, as discussed in Section 6.8.4.

6.8.3.3.5 North Approach – Station 12+425

Borehole BI-18 advanced at Station 12+425 encountered about 1.3 m of sand underlain by about 8.6 m of clayey silt to silty clay. The total height of the embankment at this location will be about 5 m after removal of the peat. The following simplified stratigraphy and deformation parameters have been employed in the settlement analysis of the proposed 5 m high granular earth fill in this area.

Soil	Thickness (m)	Unit Weight (kN/m ³)	Estimated Deformation Properties
New Earth Fill (1.5 m embankment + removal of 3.5 m of peat)	5 (high)	21	--
Very Loose Sand	1.3	20	E' = 7 MPa
Firm Clayey Silt to Silty Clay	8.6	18.5	(see below)

The following consolidation parameters were estimated for the clayey silt/silty clay deposit based primarily on the results of a laboratory consolidation test performed on a specimen of the clayey silt obtained from Borehole BI-17. These results were compared with values estimated from empirical correlations using the results of the in situ tests and laboratory index testing as described previously.

Location	Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	OCR	e_o	C_r	C_c	c_v (cm ² /s)
Station 12+400	304.4 to 295.8	52	125	2.4	1.19	0.04	0.44	0.015

Note: Elevations given are the range for top and bottom of the clayey silt to silty clay layers and the values are estimated at the middle of the clayey silt to silty clay layer.

Based on the results of the settlement analysis, the total settlement of the foundation soils for the embankment widening is estimated to be up to about 65 mm. This total settlement is estimated to be comprised of about 15 mm of immediate settlement due to compression of the cohesionless soil layers and about 50 mm of consolidation settlement of the cohesive soil layers.

Assuming a coefficient of consolidation (c_v) of 1.5×10^{-2} cm²/s (as noted above from the results of the laboratory consolidation test) and assuming two-way drainage of the approximately 8.6 m thick cohesive deposit, it is estimated that about 90 percent of the consolidation settlement will be completed in about 4 months.

The magnitude of creep settlement for the clayey silt strata is expected to be less than about 10 mm per log-cycle of time at this location.

In order to mitigate the above noted settlement, consideration should be given to preloading the embankment fill area, as discussed in Section 6.8.4.3.



6.8.3.4 Settlement of New Embankment Fill

6.8.3.4.1 Earth Fill

We understand that consideration is being given to the use of Granular 'B' Type II for the new embankment construction (i.e. grade raise and widening) required at this site. The material placed below the water level will compress/settle under its selfweight as additional fill is placed over it. The material placed above the water level should be compacted in accordance with MTO Special Provision 105S10. The magnitude of compression settlement from the below-water and from properly compacted embankment Granular 'B' Type II fill above water is expected to occur during construction. It is recommended that the fines content of the Granular 'B' Type II fill used for embankment construction be restricted to a maximum of 5 percent passing the No. 200 sieve and the material should meet the requirements of MTO Special Provision 110S13, to reduce the potential for post-construction settlement and associated maintenance needs.

6.8.3.4.2 Rock Fill

If rock fill is used for the construction of the new embankment grade raises and widenings, in addition to the settlement due to compression of the foundation soils as described above, there will be some settlement due to compression of the rock fill itself. Settlement of the rock fill depends on the type of rock and on the method and sequence of placement and compaction of the fill. Assuming that the rock fill is not end dumped in its final position and is placed in accordance with the requirements as outlined in Special Provision SP 206S03, the settlement of the newly placed rock fill is expected to be small for the relatively low fill heights required at this site. In general, it is estimated that the settlement of the rock fill at this site would be about 0.5% of the new effective height of rock fill (i.e. on the order to about 10 mm (south approach) and about 20 mm (north approach)). It is anticipated that the majority (approximately 60%) of this settlement will occur in the first year following construction.

6.8.3.5 Embankment Widening

In accordance with the requirements of MTO NRE 98-200, the minimum required embankment widening at this site to account for the potential for post-construction settlement and for future pavement overlays is 1.0 m per embankment side.

6.8.4 Mitigation of Stability Issues / Consolidation Settlements

As discussed in Sections 6.8.1 and 6.8.3, the approach embankment stability and estimated magnitude of post-construction consolidation settlement differ at the proposed immediate south and north approaches. The following sections summarize the foundation challenges at each approach and outline the mitigation options and recommended measures to be adopted.

6.8.4.1 Immediate South Approach

At the south approach, where the existing embankment will be widened to the west and the proposed new embankment requires up to about 2.3 m of new fill to construct the embankment to the design grade, no stability issues are anticipated provided the granular earth fill side slopes are constructed at a profile not steeper than 2H:1V and all near surface peat/organic layers are removed prior to construction.



Consolidation settlements on the order of about 40 mm to 75 mm are estimated to occur as a result of constructing the widened section of the approach embankment up to 2.3 m high due to the presence of existing clayey embankment fill, peat and silty clay strata below the approach area. It is estimated that it will take approximately 1 month to 2 months to achieve 90 percent consolidation of these strata as a result of the construction of the widened section of the embankment. Some of these soft, compressible strata will be removed and replaced with granular backfill as part of the construction of the new south abutment area and, at these locations, the magnitude of the consolidation settlements will decrease. In areas where the compressible strata is not removed as part of the new construction, in order to minimize post-construction settlement, it is recommended that the new embankment sections be constructed to full height and the foundation soils preloaded for a minimum period of two (2) months prior to final grading and construction of the final pavement structure.

6.8.4.2 Immediate North Approach

At the north approach, the greater height of new fill required for widening the existing embankment (in particular on the west side into the river) combined with the softer consistency of the silty clay stratum creates a stability problem for the front slope and west side slope of the proposed new approach embankment. In addition, larger consolidation settlements are anticipated to occur (ranging from about 120 mm to 195 mm) over a period of up to six (6) months following completion of construction of the roadway to the design surface level. In this area, consideration should be given to adopting a design and/or following a construction sequence to achieve the minimum target Factor of Safety of 1.3 for the proposed new embankment height and to limit the post-construction settlements and subsequent maintenance on the new roadway surface.

The following sections outline the options and recommendations for achieving the target Factor of Safety for the required embankment geometry and for minimizing post-construction settlements that could affect roadway performance. The advantages, disadvantages, relative costs and risks/consequences for the stability mitigation options at this area are summarized and ranked in Table 3. A combination of lightweight (EPS) fill and preloading is the recommended stability/settlement mitigation option for this location. In all cases, all peat/organic layers and the alluvium in the riverbed below the footprint of the proposed embankment should be sub-excavated prior to embankment construction.

6.8.4.2.1 Toe Berms and Preloading

For the approximately 3.0 m to 5.4 m thick silty clay to clayey silt stratum at this location, it is estimated that about 90 percent of the post-construction foundation soil settlements will be completed within about 6 months. If the construction schedule can accommodate this period, preloading of the foundation soils by constructing the embankment as early as possible should be considered. However, for this alternative, for the full height of proposed fill, toe berms approximately 8 m wide by 2 m to 3 m high (or 1 m above average river water level) will be required to maintain the stability of both the front slope and west slope of the approach (refer to Figures 1 and 2). Stability berms of this size will encroach significantly into the relatively narrow Blanche River at this site. Given these constraints, toe berms are not considered to be a practical stability mitigation alternative at this location.

6.8.4.2.2 Lightweight (EPS) Fill and Preloading

The loading imposed by the up to 4.2 m high new north approach embankment fills on the soft and compressible foundation soils in this area could be reduced by using ultra-lightweight (EPS) fill. The use of this material for the embankment fill would eliminate the need for stabilizing toe berms (and still achieve a Factor of Safety greater than 1.3 – refer to Figures 3 and 4) and would also reduce the magnitude of the consolidation settlement of the clayey strata.



When estimating the maximum thickness of EPS fill that could be installed at this area, several practical construction constraints and design issues must be considered including:

- a need to maintain the base of the EPS above the average river water level at the time of construction (about Elevation 308.4 m);
- a desire to construct a minimum 125 mm thick reinforced concrete slab on top of the EPS blocks;
- a requirement to have at least about 1.0 m of conventional fill cover (including reinforced concrete slab, compacted granular materials and asphalt) on top of the EPS blocks to minimize the potential for differential icing between the EPS and non-EPS fill areas; and
- a requirement to have an adequate Factor of Safety against uplift (i.e. buoyancy) of the EPS fill at the design high water level in the river (about Elevation 308.77 m).

Given the above requirements, the maximum thickness of EPS that can be installed at the north approach behind the abutment is 1.9 m. The EPS fill should extend for a width of about 15 m east to west across the embankment section (and be stepped to avoid conflict with the guide rail posts) and for a distance of at least 25 m north of the abutment wall (and be stepped in a minimum of 0.3 m increments over a distance of up to about 4 m as it transitions to the granular fill embankment).

With the EPS installed in this manner, it is estimated that the consolidation settlements of the clayey strata will be reduced to about two-thirds of that estimated for the full conventional earth fill scenario. As such, it is estimated that settlements of about 130 mm (west side) and 90 mm (centreline) will still occur after completion of filling. Settlements of this magnitude may result in tilting of the guide rail as well as cause cracking of the reinforced concrete slab on top of the EPS, cause differential settlement of the EPS blocks and affect the long-term performance of the roadway. Given this, a partial preloading with conventional embankment fill should be considered and the following sequence of construction adopted:

- construct up to 3 m high preload embankment (no higher than Elevation 310.0 m) with conventional earth fill (no toe berms required);
- preload for a minimum period of 6 months;
- remove 1.6 m of embankment earth fill (to Elevation 308.4 m);
- install sand levelling layer and 1.9 m thickness of EPS for a distance of 25 m north of the abutment; and
- construct 125 mm reinforced concrete slab, granulars and pavement structure on top of EPS.

This approach would result in a partial off-loading of the foundation soils prior to final embankment construction that would reduce the majority of the post-construction settlement and improve the long-term performance of the roadway.

6.8.4.2.3 Full Sub-excavation

Sub-excavation and replacement of the soft clayey strata (up to 7 m deep) below the existing roadway surface in the north approach area would be a challenge at this site for several reasons. First, given that a part of the excavation would have to advance through Blanche River, a sheet pile cut-off/shoring system would be needed to minimize impacts from the work on the adjacent river. Second, given the presence of the existing highway that must remain in operation during the new construction, sub-excavation of the soft soils would also require a temporary shoring protection system to support the embankment adjacent to the excavation area. Considering this, the costs associated with the required shoring/cut-off systems would likely make this alternative impractical and, as such, sub-excavation is not recommended as a stability mitigation measure at this site.



6.8.4.2.4 Staged Construction

As noted above, it is possible to construct the embankment with conventional earth fill up to a maximum of about Elevation 310.0 m without the use of toe berms. In addition, about 90 percent of the primary consolidation settlement of the silty clay to clayey silt strata would be completed in about 6 months following completion of the filling to this stage.

If there is sufficient time in the construction schedule, consideration could be given to constructing the full embankment with conventional earth fill in two (2) separate stages (i.e. first stage to Elevation 310.0 m and second stage to Elevation 311.3 m) with a minimum 6-month preload period after each stage. However, the estimated strength gain in the silty clay strata as a result of the first stage of construction is not sufficient to achieve a Factor of Safety of 1.3 for the final fill stage to Elevation 311.3 m. It would still be necessary to construct toe berms on the front and west slopes approximately 2 m to 3 m high (or 1 m above average river water level) and 5.2 m and 4 m wide, respectively, to achieve the target Factor of Safety. In addition, it should be noted that some additional long-term settlements due to secondary consolidation (i.e. creep) of the clayey strata (on the order of about 15 mm per log-cycle of time) should be expected with this option. As such, about 30 mm of post-construction settlement should be expected over a 50-year design life of the roadway.

Given the above constraints on the schedule, requirements for small toe berms into the river and the likelihood of some post-construction creep settlement, staged construction is not recommended for stability/settlement mitigation at this location.

6.8.4.2.5 Surcharging

As noted above, toe berms approximately 2 m to 3 m high and 8 m wide are necessary to maintain the stability of the front slope and west slope of the approach if the embankment is built up to the required final grade height (i.e. Elevation 311.3 m) without staging or control of the rate of construction (see Figures 1 and 2). Even larger front slope and west side slope toe berms would be required if a surcharge was to be placed on top of the new embankment fill to reduce the time to complete the consolidation settlements. Given the navigational, hydraulic and potential environmental constraints associated with filling in the river channel, this option is not considered to be feasible at this site.

6.8.4.3 North Embankment – Station 12+300 to 12+650

Stability issues are not anticipated for the widening of the northern portion of the embankment between about Station 12+300 and 12+650 provided that granular earth fill side slopes are constructed at a profile not steeper than 2H:1V and all peat/organic layers are removed from below the embankment footprint prior to construction.

Consolidation settlements on the order of up to 50 mm are estimated to occur as a result of the new section of embankment construction due to the presence of clayey silt to silty clay strata below the embankment widening. It is estimated that it will take up to about 4 months to achieve 90 percent consolidation of these strata as a result of the new embankment construction. In order to minimize post-construction settlement between the embankment widening and the existing embankment, it is recommended that the new embankment fills be constructed to full height and the foundation soils preloaded for a minimum period of six (6) months prior to final grading and construction of the final pavement structure.



6.9 Subgrade Preparation and Embankment Construction

Prior to embankment construction, all topsoil/vegetation/peat/organic soils and riverbed alluvium must be removed below the footprint of the proposed embankments. The existing fill and native subsoils are considered to be an appropriate subgrade; however, all softened/loosened soils should be stripped from below the approach embankment areas and subgrade soils should be proof-rolled, where possible, prior to placement of new fill. The existing peat should be excavated in accordance with OPSS 209 and OPSD 203.020 which maintains the slope of the existing embankment during construction. The peat should be excavated in strips perpendicular to the highway alignment no wider than 3 m and immediately backfilled with Granular 'B' Type II. This assumes that the excavation will take place with equipment working from the existing embankment. Since this may not be practical from a traffic standpoint, alternatively, the peat could be removed in small areas (3 m by 3 m) or small strips starting from the extremes of the embankment working towards the bridge.

Placement of Granular 'B' Type II backfill and other granular earth fill for embankment construction should be carried out in accordance with the requirements as outlined in Special Provision SP206S03 and OPSS209. Granular earth fill above the water level should be placed in regular lifts with loose thickness not exceeding 300 mm and compacted to at least 95 percent of the standard Proctor maximum dry density. Side slopes for earth fill embankments should be no steeper than 2H:1V.

If rock fill is used for construction, the rock should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Voids and bridging should be minimized by blading, dozing and 'chinking' the rock to form a dense, compact mass. Side slopes for rock fill embankments should be no steeper than 1.25H:1V.

The final lift of fill prior to placement of the granular subbase and base courses should be compacted to 100 percent of the Standard Proctor maximum dry density. 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.

In order to minimize differential settlement between the existing embankment slopes and the newly placed embankment fill, the new fill should be keyed into the existing embankment side slope per the requirements of OPSD 208.010.

The abutment front slopes and any side slopes adjacent to the river require erosion protection. Erosion protection should be placed on the slopes to at least 0.5 m above the design high water level (Elevation 308.77 m). Erosion protection could consist of a minimum 0.6 m thick layer of rip rap (300 mm diameter), rock protection or concrete slope paving. The potential for scour below the pile caps should be taken into account in the design of the bridge foundations.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding should be carried out as soon as possible. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw or gravel sheeting to prevent erosion, will be required to reduce the potential for remedial works on the side slopes in the spring prior to topsoil and seeding. The requirement to vegetate the embankment side slopes does not apply to rock fill slopes.

6.10 Design and Construction Considerations

6.10.1 Excavations

As noted in Section 6.3, excavations for construction of a spread footing on bedrock at the south abutment would extend to about Elevation 306 m (up to about 5 m below the existing ground surface). Further, as noted in Section 6.4, excavations for construction of the pile cap at the south and north abutment would extend to about



Elevation 306.4 m and 307.4 m, respectively (up to about 2.6 m below existing ground surface at the south abutment and up to about 3.1 m below existing roadway surface and about 1 m below river water surface at the north abutment).

At the south abutment, excavations up to 5 m deep (i.e. to bedrock for construction of a spread footing) would have to be supported by a temporary shoring system that controls groundwater inflows (and allow work to be carried out in-the-dry), limits the extent of the excavation and supports and maintains the stability of the existing adjacent roadway embankment.

At both the north and south abutment, excavations up to about 3 m deep (i.e. for construction of pile caps) would also have to be supported by a temporary shoring system that controls groundwater inflows (especially where the excavation extends into the river), limits the excavation extent and supports and maintains the stability of the existing adjacent roadway embankment.

Conventional excavation equipment should be suitable for excavation through the on-site soils; however, the contractor shall be made aware of the potential for obstructions in the foundation strata as discussed in Section 6.10.4.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects and good construction practice. The existing fill materials and the native soils should be classified as Type 3 soil, according to the OHSA.

6.10.2 Temporary Shoring

As noted above, given the depth of excavation required and the proximity of the excavation area to the adjacent highway embankment and river, it is unlikely that open (i.e. unsupported) cuts can be utilized for the abutment construction at this site.

Excavation support for protection of the existing roadway at both abutment areas will likely be required during the first stage of construction, while the new first “half” bridge and approach embankment are being constructed. Temporary excavation support systems should be designed and constructed in accordance with Special Provision SP105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP105S19.

6.10.3 Groundwater and Surface Water Control

Where temporary shoring is employed for the construction of the spread footing on bedrock (at above Elevation 306 m) at the south abutment, groundwater inflow should be expected considering that the excavation will extend up to about 2.4 m below the river water level (at Elevation 308.4 m) and that cohesionless layers of silt, sand and sandy silt are present immediately above the bedrock surface. While the use of a sheet pile shoring system should be considered to control groundwater in a deep excavation (to bedrock) at this location, given the variable depth to bedrock and the strong to very strong nature of the bedrock, it will not be possible to toe the sheet piling into the rock to achieve complete cut-off of inflows.

Where temporary shoring is employed for construction of the pile cap (underside at Elevation 307.4 m, or deeper if rigid polystyrene insulation is not used for frost protection) at the south abutment, some groundwater inflow should be expected considering that the excavation will extend up to about 1 m below the river water level (at Elevation 308.4 m) and considering that cohesionless fills and layers of silt, sand and sandy silt are present below the elevation of the base of excavation.



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At the north abutment, the western edge of the footprint of the foundation element is located within Blanche River. In order to construct the pile cap (underside at Elevation 307.4 m), groundwater inflow should be expected and controlled dewatering within the temporary shoring will be required in accordance with the special provision and performance level specified above. Consideration should be given to constructing the north abutment pile cap within a sheet pile shoring system/cofferdam. It is understood that Special Provision 902S01 is typically used in MTO Contracts for this purpose. The cofferdam should be designed so that the disturbance to the existing embankments is minimized. An NSSP will be required to inform the contractor that the pile cap construction must be carried out in-the-dry; an example is included in Appendix C for reference.

Based on grain size distributions for the silt and sand and gravel strata encountered in the boreholes advanced at the north and south abutments and using the limits of dewatering proposed by Powers (1992), it is considered that the volume of seepage through the silt layers may be slow enough that adequate groundwater control could be attained through the use of pumping from properly filtered sumps in the excavation. However, the seepage through the sand and gravel fills and sandy native strata will be faster and larger pumps and/or a more extensive dewatering system may be required to control the seepage through these layers. Given this, consideration should be given to the use of sheet pile shoring toed into the silts (below the south abutment area) or clays (below the north abutment area) to cut off and control groundwater flows.

If temporary sheet pile shoring is installed to facilitate excavation to bedrock at the south abutment (i.e. for spread footing construction), some measures such as the placement of sand bags between the tips of the sheet piles and the sloping bedrock surface could be employed to cut off seepage inflows, if necessary.

In all cases, surface water runoff should be directed away from the excavation at all times.

6.10.4 Obstructions

As part of the design and construction of the new foundations, careful consideration should be given to the location of the existing (and possibly older) bridge foundations (for example, wooden piles) relative to the new construction. Specifically, the designer should check that any new piles (batter and orientation) and any temporary shoring do not interfere with the older abandoned piles. This should be checked to the full extent of the pile/shoring length.

It should also be noted that timber piles and a timber cribbing (possibly an abandoned abutment from an earlier roadway alignment) were observed west of the proposed north abutment area within the river. As such, it is possible that older structure foundations may be present in the area of the proposed new construction. Consideration should be given to carrying out an underwater inspection (in the river) and a test pitting program (on the shores) prior to finalizing design and/or the start of new construction to ensure/check that there are no obstructions in the areas of the proposed new construction (i.e. temporary shoring, piling and abutment construction).

Where obstructions in the form of old pile foundations do exist, it is recommended that they be left in place and not pulled out or removed. During the foundation investigation at this site, problems were encountered with heaving sands during the borehole drilling below the clay strata and, in one case, at the north abutment, boiling sands occurred around the existing timber piles during drill casing advancement. Based on these observations and considering the present condition of the existing bridge timber pile foundations (i.e. piles leaning to the west), it is possible that extraction of any existing piles from previous foundations may cause disturbance of the foundation soils and loss of ground which could affect the performance of the existing structure.

The existing timber piles should be cut off at the river bed level after installation of the new piles to minimize obstructions during subsequent construction of the abutments and to enhance future use of waterway.



Cobbles and/or a boulder, about 0.3 m thick, were encountered below the existing fill material in Borehole BI-6. In addition, cobbles were encountered within the existing pavement structure at several borehole locations at the south approach. A layer of cobbles and/or a boulder, about 0.3 m thick, was encountered below the existing fill material in Borehole BI-20. In addition, cobbles were encountered within the peat at Borehole BI-21. Several pavement boreholes did not penetrate through the fill materials, potentially due to the presence of cobbles and/or boulders.

An NSSP should be included in the contract document to alert the contractor to such potential construction difficulties, such as the existing piles or cobbles/boulders at the site. An example NSSP is included in Appendix C for reference.

6.10.5 Existing Structure Monitoring

Given the condition of the existing structure, the close proximity of the first half of the new structure construction relative to the existing structure, and the requirement for the existing structure to remain in operation during construction of the first half of the new structure, it is recommended that the existing structure be monitored for settlement and lateral movement while in operation during the new construction.

7.0 CLOSURE

This report was prepared by Mr. André Bom, P.Eng., a geotechnical engineer with Golder Associates Ltd. The technical aspects were reviewed by Mr. Paul Dittrich, Ph.D., P.Eng., an Associate with Golder and Mr. Jorge Costa, P.Eng., Principal with Golder and the Designated MTO Contact, who also conducted a quality control review of the report.



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GOLDER ASSOCIATES LTD.



André Bom, P.Eng.
Geotechnical Engineer

J. Paul Ditrich, Ph.D., P.Eng.
Associate, Senior Geotechnical Engineer



Jorge M.A. Costa, P.Eng.
Principal, Designated MTO Contact

AB/JPD/JMAC/lb

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TABLE 1A
EVALUATION OF FOUNDATION ALTERNATIVES – SOUTH ABUTMENT
HIGHWAY 11 STRUCTURE AT BLANCHE RIVER
W.P 168-98-00, SITE NO. 47-006

Options	Ranking	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-Piles set in 600 mm diameter holes cored up to 4 m deep into bedrock (HP310x125 may be required)	1	<ul style="list-style-type: none"> Allows for integral abutment design. Pile cap can generally be constructed in-the-dry, without tremied concrete placement. 	<ul style="list-style-type: none"> Drilling 600 mm diameter holes into the bedrock for lengths up to 4 m to achieve minimum required 5 m pile length for integral abutment design. Corrugated Steel Pipe (CSP) installed below groundwater level to bedrock surface and sand infilling, for pile installation. Additional drill length and concreting at the bottom of the holes drilled in the bedrock to achieve pile fixity. 	<ul style="list-style-type: none"> Drilling up to 10 600 mm diameter holes into strong to very strong bedrock is expensive. Two mobilizations to site for specialized drilling equipment due to staged construction. 	<ul style="list-style-type: none"> Heavier pile section (i.e. HP 310x125) may be required to satisfy lateral loading and buckling consideration. Drilling vibrations may negatively impact existing structure. Monitor existing structure for displacement during drilling/pile installation.
Spread Footing on mass concrete pad (Founding Level about Elev. 306.3 m)	2	<ul style="list-style-type: none"> Can minimize or eliminate bedrock excavation. Conventional construction. 	<ul style="list-style-type: none"> Allows only for semi-integral abutment design. Subexcavation depths through fill materials and native soil may be up to 5 m below existing ground surface and up to 3 m below the groundwater level. Dewatering, cleaning bedrock surface and concrete placement required within dry, shored excavation. 	<ul style="list-style-type: none"> Lower relative costs compared with piled foundations due to less bedrock excavation Additional costs required for installation of temporary shoring and dewatering. 	<ul style="list-style-type: none"> If bedrock is higher than anticipated, bedrock removal may be required Variability in bedrock surface will impact mass concrete quantities and/or excavation depths. Fractures in bedrock may cause seepage into excavation from adjacent river. Shored excavations could negatively impact existing structure.
1.0 m Diameter Caissons socketted 2 m into bedrock	3	<ul style="list-style-type: none"> Reduced number of deep elements and number of rock coring locations compared to steel H-piles. Possible elimination of pile cap. 	<ul style="list-style-type: none"> Allows only for semi-integral abutment design. Temporary liners would be required for groundwater control and support through overburden. Concrete for caissons would have to be placed by tremie methods below the water level. May be difficult socketting caissons into strong to very strong bedrock. 	<ul style="list-style-type: none"> Socketting large diameter (1.0 m) holes into the strong to very strong bedrock may be more expensive than drilling required for steel H-pile sockets. Two mobilizations to site for specialized drilling equipment due to staged construction. 	<ul style="list-style-type: none"> Risk of difficulties achieving seal and drilling large diameter bedrock socket. Drilling vibrations may negatively impact existing structure. Monitor existing structure for displacement during caisson construction.

NOTES:

1. This table should be read in conjunction with Section 6.2 of the Foundation Investigation and Design Report.

Compiled By: AB
 Checked By: JPD
 Reviewed By: JMAC

TABLE 1B
EVALUATION OF FOUNDATION ALTERNATIVES – NORTH ABUTMENT
HIGHWAY 11 STRUCTURE AT BLANCHE RIVER
W.P 168-98-00, SITE NO. 47-006

Options	Ranking	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-piles set in 600 mm diameter holes cored up to 2.5 m deep into bedrock (HP310x125 may be required)	1	<ul style="list-style-type: none"> Allows for integral abutment design. 	<ul style="list-style-type: none"> Drilling 600 mm diameter holes into the bedrock for lengths up to 2.5 m to achieve minimum required 5 m pile length for integral abutment design. Corrugated Steel Pipe (CSP) installed below groundwater level and to bedrock surface (on west side) and sand infilling, for pile installation. Additional drill length and concreting at the bottom of the holes drilled in the bedrock to achieve pile fixity. Heavier pile section required to provide for lateral resistance. 	<ul style="list-style-type: none"> Drilling up to 10 600 mm diameter holes into strong to very strong bedrock is expensive. Two mobilizations to site for specialized drilling equipment due to staged construction. 	<ul style="list-style-type: none"> Heavier pile section (i.e. HP 310x125) may be required to satisfy lateral loading and buckling considerations. Drilling vibrations could negatively impact existing structure. Monitor existing structure for displacement during drilling/pile installation.
Steel H-piles driven to bedrock (HP 310x110)	2	<ul style="list-style-type: none"> Piles can be battered to achieve required lateral resistance. Shorter pile lengths as piles driven to bedrock only (no rock drilling required to achieve minimum pile length for integral abutment design or for lateral fixity of vertical piles). Straightforward construction. 	<ul style="list-style-type: none"> Allows for semi-integral abutment design. Depending on elevation of bottom of pile cap, piles on west side of abutment may be less than 3 m in length; consideration is being given to raising the bottom of pile cap elevation and reducing depth of soil cover required for frost protection by installation of polystyrene insulation. 	<ul style="list-style-type: none"> Lower relative costs compared with caisson option and steel H-piles set in 600 mm diameter cored holes in bedrock option. 	<ul style="list-style-type: none"> Sloping bedrock may make seating piles onto bedrock difficult. Monitor existing structure for displacement during drilling/pile installation.

Options	Ranking	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Caissons socketted into bedrock	3	<ul style="list-style-type: none"> Reduced number of deep elements compared to steel H-piles. Possible elimination of pile cap. Compatible with South Abutment (i.e. similar construction methods could be employed at each abutment if caissons selected for each). 	<ul style="list-style-type: none"> Allows only for semi-integral abutment design. Temporary liners would be required for groundwater control and support through overburden. Concrete for caissons would have to be placed by tremie methods below the water level. May be difficult socketting caissons into strong to very strong bedrock. 	<ul style="list-style-type: none"> Socketting large diameter (1.0 m) holes into the strong to very strong bedrock may be more expensive than drilling required for steel H-pile sockets. Two mobilizations to site for specialized drilling equipment due to staged construction. 	<ul style="list-style-type: none"> Risk of difficulties achieving seal and drilling large diameter bedrock socket. Drilling vibrations could negatively impact existing structure. Monitor existing structure for displacement during caisson construction.
Spread Footings on mass concrete pad (Lower Founding Elev. 302 m and Upper Founding Elev. 304 m)	NF	<ul style="list-style-type: none"> No bedrock drilling required. 	<ul style="list-style-type: none"> Subexcavation depths through fill materials and native soil may be up to 9 m below existing ground surface and up to 7 m below the groundwater level. Deep temporary sheet pile shoring/cofferdam construction and dewatering required to expose bedrock and construct in-the-dry. Sloping bedrock will may shoring construction and sub-excavation difficult. Bedrock excavation required at west edge of proposed abutment if lower founding level proposed or required. Large mass concrete quantities required if higher founding level proposed. 	<ul style="list-style-type: none"> Higher relative costs due to subexcavation to bedrock within a shored excavation Increased costs due to mass concrete needs. 	<ul style="list-style-type: none"> If bedrock is higher than anticipated, bedrock removal may be required. Variability in bedrock surface will impact mass concrete quantities and/or excavation depths. Fractures in bedrock may cause seepage into excavation from adjacent river. Shored excavations could negatively impact existing structure.

NOTES:

- This table should be read in conjunction with Section 6.2 of the Foundation Investigation and Design Report.
- NF: Indicates that the founding option is not considered feasible.

Compiled By: AB
 Checked By: JPD
 Reviewed By: JMAC

TABLE 2
PARAMETERS FOR HORIZONTAL SUBGRADE REACTION
HIGHWAY 11 STRUCTURE AT BLANCHE RIVER
W.P 168-98-00, SITE NO. 47-006

Foundation Element	Relevant Borehole	Soil Unit	Elevation (m)	n_h (kPa/m)	s_u (kPa)
North Abutment - West Side	BI-3 and BI-8	Embankment Fill above Water/Groundwater Level (assumed to be compact granular fill)	above 308.4 m	6,600	-
		Embankment Fill below Water/Groundwater Level (assumed to be loose granular fill)	307.0 m to 308.4 m	1,300	-
		Soft Clayey Silt	304.6 m to 307.0 m	-	15
North Abutment - Centre	BI-9	Embankment Fill above Water/Groundwater Level (assumed to be compact granular fill)	above 308.4 m	6,600	-
		Embankment Fill below Water/Groundwater Level (assumed to be loose granular fill)	307.0 m to 308.4 m	1,300	-
		Soft Clayey Silt	301.4 m to 307.0	-	15
North Abutment - East Side	BI-12 and BI-13	Embankment Fill above Water/Groundwater Level (assumed to be compact granular fill)	above 308.4 m	6,600	-
		Embankment Fill below Water/Groundwater Level (assumed to be loose granular fill)	306.0 m to 308.4 m	1,300	-
		Soft to Firm Clayey Silt	300.0 m to 306.0 m	-	20

NOTES:

1. This table should be read in conjunction with Sections 6.4.5 and 6.5.3 of the Foundation Investigation and Design Report.

Checked By: AB
Reviewed By: JMAC

TABLE 3
EVALUATION OF STABILITY/SETTLEMENT MITIGATION ALTERNATIVES
NORTH APPROACH EMBANKMENT - HIGHWAY 11 STRUCTURE AT BLANCHE RIVER
W.P 168-98-00, SITE NO. 47-006

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Lightweight Fill (EPS) and Partial Preloading (to Elev. 310 m) for about 6 months	1	<ul style="list-style-type: none"> Reduces load on compressible soils thereby increasing stability and reducing settlement of foundation soils. Combined preloading plus EPS fill reduces magnitude of post-construction differential settlement across embankment section. Stabilizing toe berms not required. Reduces impact of construction on adjacent river. 	<ul style="list-style-type: none"> High EPS fill material cost. Restricted use within embankment as it has to be placed above the groundwater/river water level and requires concrete slab and minimum 1 m of conventional soil cover to mitigate potential for buoyancy at high water level. Pre-loading to Elev. 310 m still required in order to reduce post-construction settlement. 	<ul style="list-style-type: none"> Cost savings in berm fill or sub-excavation, but relative cost of EPS is up to an order of magnitude higher than for the other materials. Some additional costs required for basic instrumentation and monitoring. 	<ul style="list-style-type: none"> 6 month preloading period to be carried out early in construction schedule to minimize delays.
Toe Berms (up to 8 m wide and 1 m above normal water level) and Preloading for about 6 months	NF	<ul style="list-style-type: none"> Relatively simple operation. 	<ul style="list-style-type: none"> Toe berms (on front slope and west slope) will extend into middle of river causing environmental, hydraulic and navigational problems. 	<ul style="list-style-type: none"> Low cost, some additional costs are required for berm construction. Some additional costs required for basic instrumentation and monitoring. 	<ul style="list-style-type: none"> Settlement of embankment/foundation soils will occur. Secondary consolidation (creep) will occur.
Full Subexcavation and Replacement (up to 5.6 m below water surface elevation in July 2008)	NF	<ul style="list-style-type: none"> Stability and long-term settlement issues minimized since all or nearly all weak, soft and compressible soils are removed. Stability berms not required. 	<ul style="list-style-type: none"> Potential for impact on the adjacent river unless extensive shoring system is employed. Potential for impact to the existing piles and approach embankment with existing bridge in operation during first stage of construction. Excavation and backfill 'in -the-wet'. 	<ul style="list-style-type: none"> Additional costs for subexcavation, extra fill materials and disposal of excavated material. Additional costs for extensive temporary shoring protection system to support existing embankment and structure. 	<ul style="list-style-type: none"> Low risk with respect to stability and long-term settlement of new embankment. Risk of impacting stability and performance of existing bridge and roadway embankment during new construction operation.

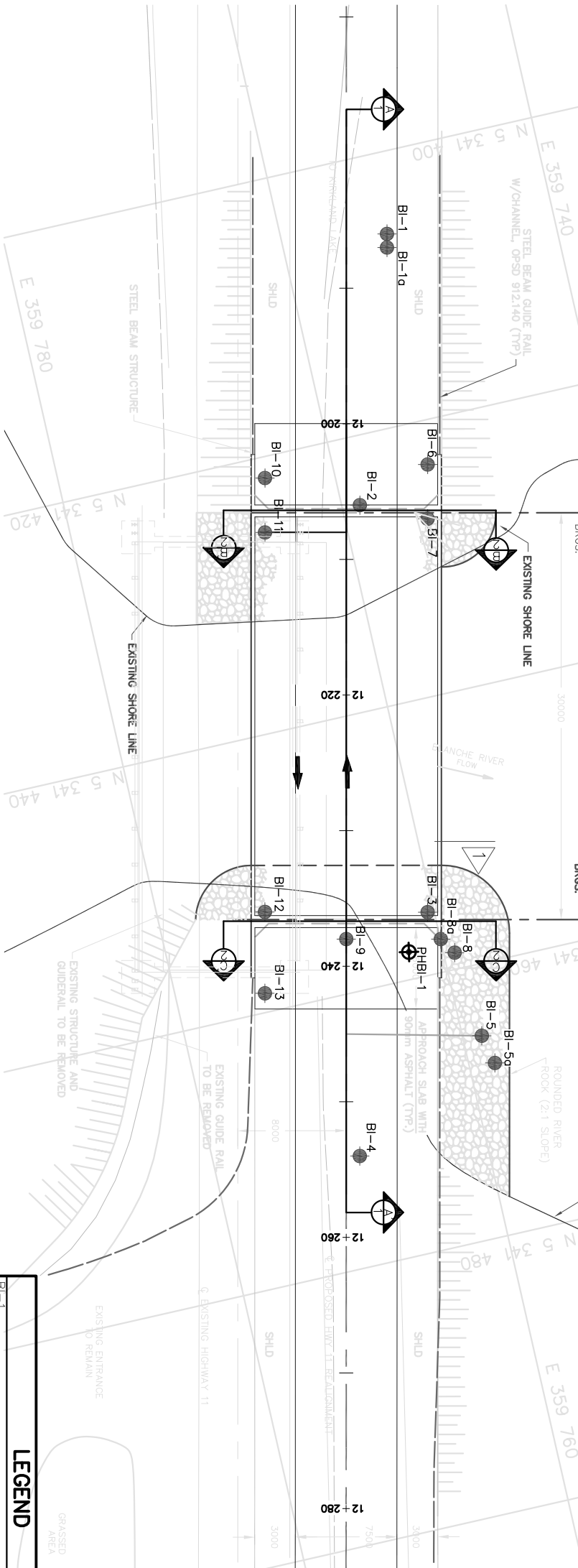
Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Staged Construction	NF		<ul style="list-style-type: none"> • Delay to construction schedule due to the requirement of minimum two stages of construction with a 6 month preload period after each stage. • Stabilizing toe berms will be required up to 5 m wide and about 1 m above average water level. • Toe berms (on front slope and west slope) will extend partially into river and may still cause environmental, hydraulic and navigational problems. 	<ul style="list-style-type: none"> • Low cost, some additional costs are required for berm construction. • Additional costs required for more extensive instrumentation and monitoring. 	<ul style="list-style-type: none"> • Settlement of embankment/foundation soils will occur. Secondary consolidation (creep) will occur. • Uncertainty associated with required preloading time period between each stage of construction.
Surcharging	NF	<ul style="list-style-type: none"> • Reduce time period for consolidation of foundation soils to less than 6 months. 	<ul style="list-style-type: none"> • Large toe berms will be required to maintain stability, larger than 8 m wide as noted above due to addition of surcharge. • Toe berms (on front slope and west slope) will extend beyond middle of river causing environmental, hydraulic and navigational problems. 	<ul style="list-style-type: none"> • Additional costs for placement and removal of surcharge and for larger toe berms. • Some additional costs required for basic instrumentation and monitoring. 	<ul style="list-style-type: none"> • Risk of embankment instability due to addition of surcharge load. • Risk to stability and performance of the existing bridge and road embankment.

NOTES:

1. NF – Not considered feasible.
2. This table should be read in conjunction with Section 6.8.4 of the Foundation Investigation and Design Report.

Compiled By: ABChecked By: JPDReviewed By: JMAC

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

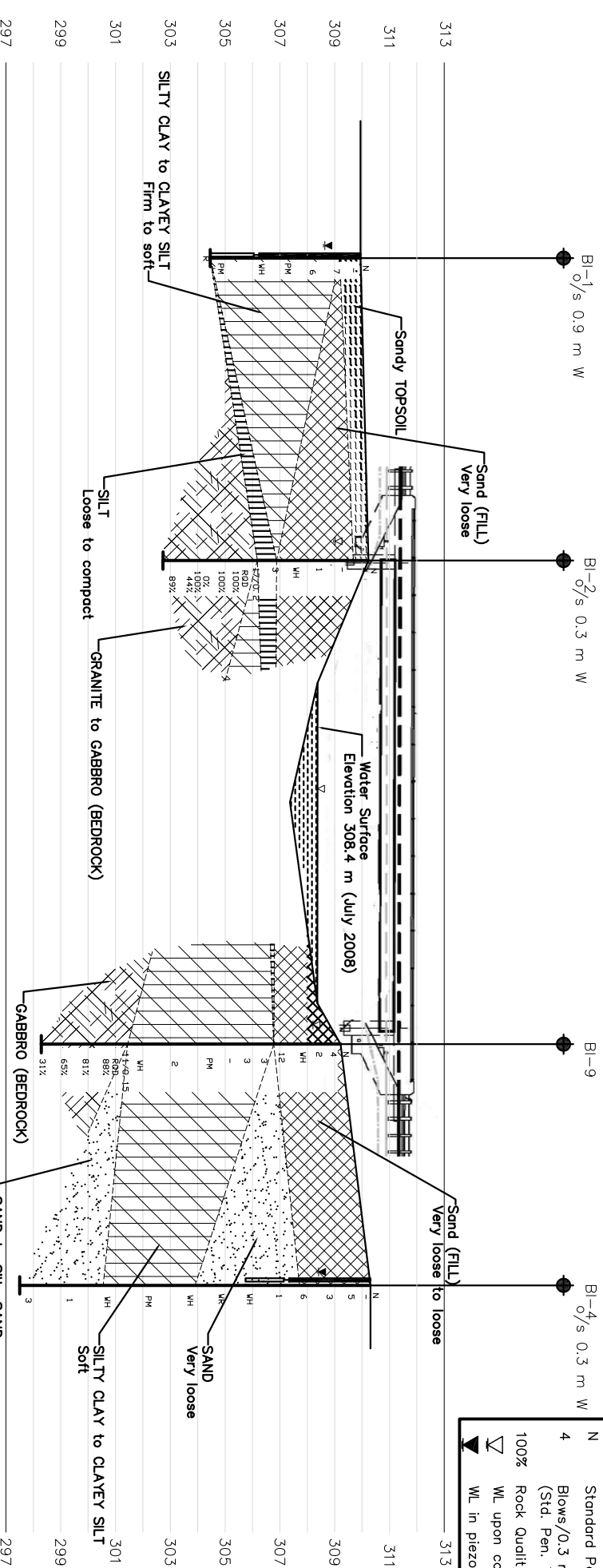


PLAN

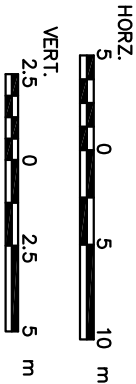


LEGEND

- BI-1 Borehole – Current Investigation
- PHBI-1 Piezometer
- Seal
- Piezometer
- Refusal
- Standard Penetration Test
- Blows/0.3 m unless otherwise stated (Std. Pen. Test, 4/51/blow)
- Rock Quality Designation (RQD)
- 100% WL upon completion of drilling
- WL in piezometer, measured on Sept. 3 or 4, 2008



PROFILE AA'



CONT No.
WP No. 168-98-00

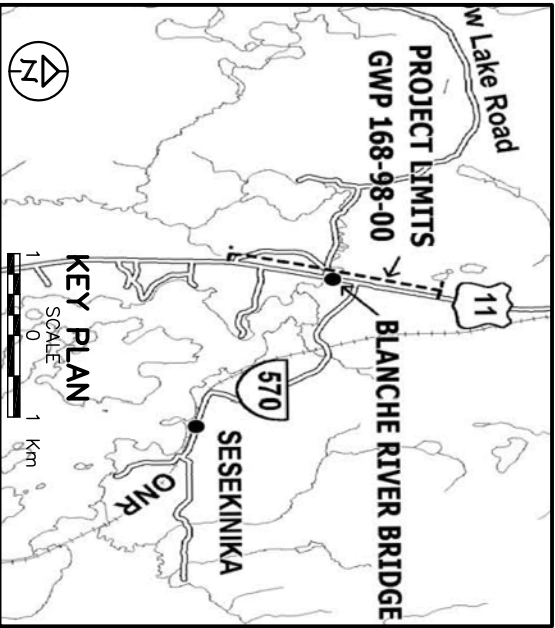


HIGHWAY 11
CROSSING BLANCHE RIVER
BOREHOLE LOCATION AND
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



No.	ELEVATION(m)	CO-ORDINATES	
		NORTHING	EASTING
BI-1	309.9	5341406.1	359752.5
BI-1a	309.9	5341406.9	359752.3
BI-2	310.2	5341425.1	359758.7
BI-3	308.4	5341455.4	359760.5
BI-4	310.3	5341471.9	359769.4
BI-5	308.4	5341465.2	359758.6
BI-5a	308.4	5341467.4	359758.1
BI-6	309.6	5341423.2	359753.2
BI-7	309.3	5341427.1	359754.1
BI-8	308.1	5341458.8	359759.2
BI-8a	308.1	5341457.6	359759.9
BI-9	309.2	5341456.1	359766.8
BI-10	310.5	5341421.6	359765.1
BI-11	310.5	5341425.5	359765.9
BI-12	310.5	5341452.8	359772.2
BI-13	310.5	5341458.6	359773.5
PHBI-1	308.1	5341457.9	359762.5

NOTES

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

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

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

REFERENCE

Base plans provided in digital format by WLLS, drawing file name BLANCHE-PO-GA-only.dwg received August, 2008 and Key Plan.dwg received September 11, 2008.

NO.	DATE	BY	REVISION	DIST.
HRV. 11				
SUBWD. AB	CHKD. AB	DATE: MAR. 2009	SITE: 47-006	
DRAWN. MM	CHKD. JPD	APPD. JMAG	DWG. 1	

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

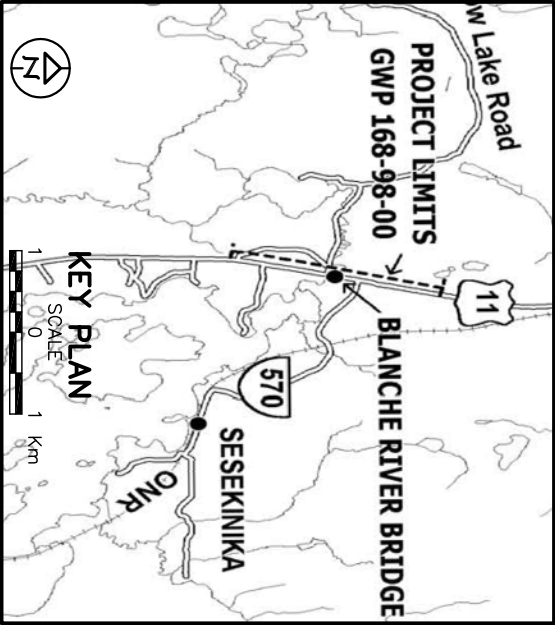
CONT No.
WP No. 168-98-00

HIGHWAY 11
CROSSING BLANCHE RIVER
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



No.	ELEVATION(m)	CO-ORDINATES	
		NORTHING	EASTING
BI-1	309.9	5341406.1	359752.5
BI-1a	309.9	5341406.0	359752.3
BI-2	310.2	5341425.1	359758.7
BI-3	308.4	5341455.4	359760.5
BI-4	310.3	5341471.9	359769.4
BI-5	308.4	5341465.2	359758.6
BI-5a	308.4	5341467.4	359758.1
BI-6	309.6	5341423.2	359753.2
BI-7	309.3	5341427.1	359754.1
BI-8	308.1	5341458.8	359759.2
BI-8a	308.1	5341457.6	359759.1
BI-9	309.2	5341456.1	359766.8
BI-10	310.5	5341421.6	359765.1
BI-11	310.5	5341425.5	359766.0
BI-12	310.5	5341452.8	359772.2
BI-13	310.5	5341458.6	359773.5
PHBI-1	308.1	5341457.9	359762.5

NOTES

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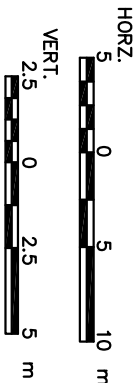
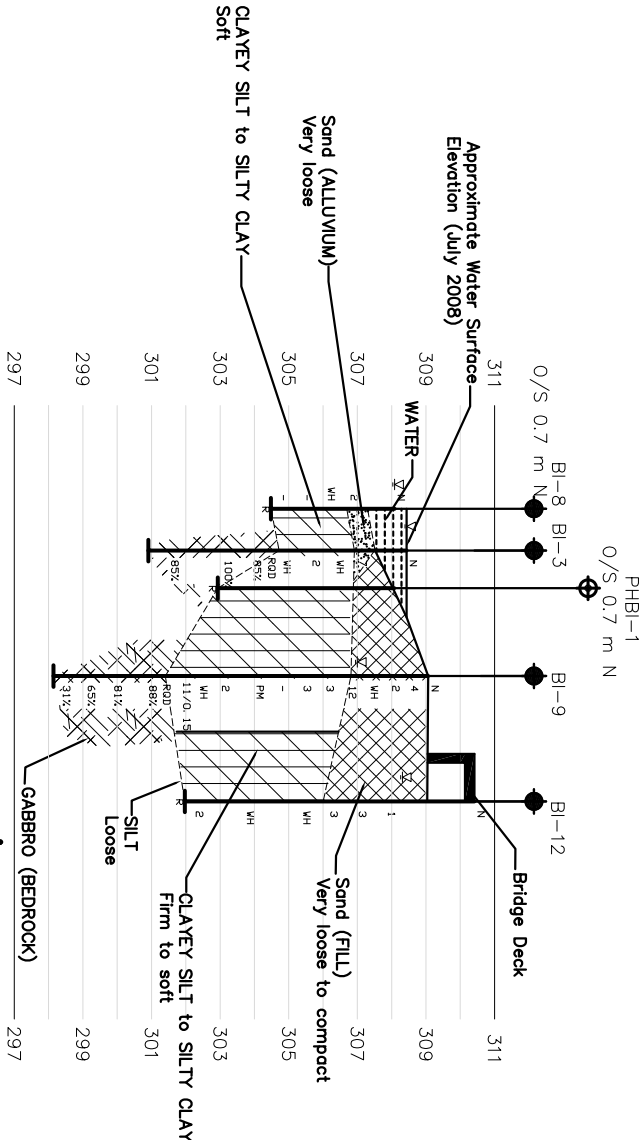
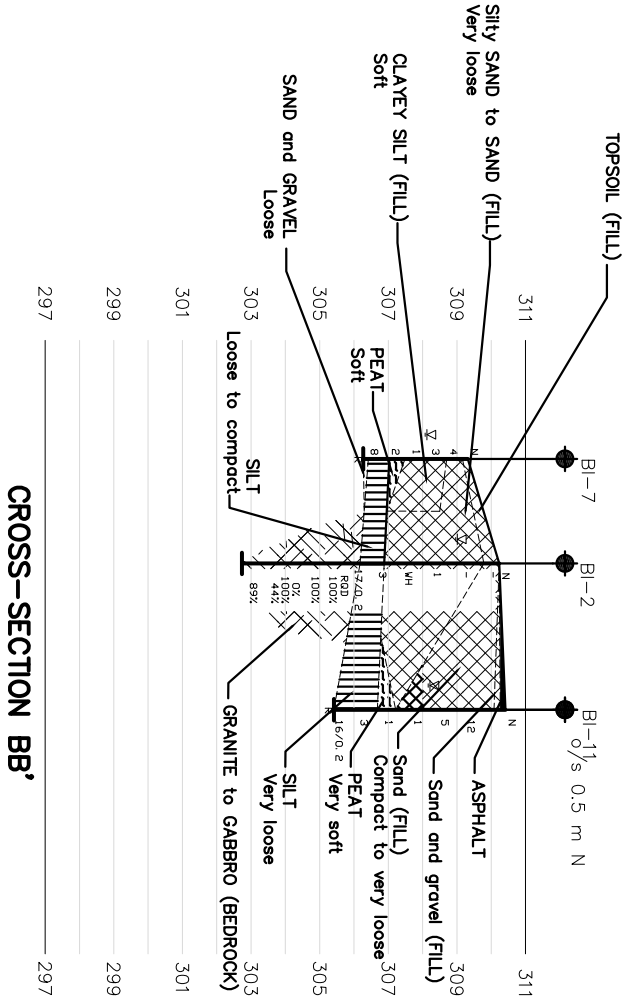
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REFERENCE

Base plans provided in digital format by WLLS, drawing file name BLANCHE-PO-GA-only.dwg received August, 2008 and Key Plan.dwg received September 11, 2008.

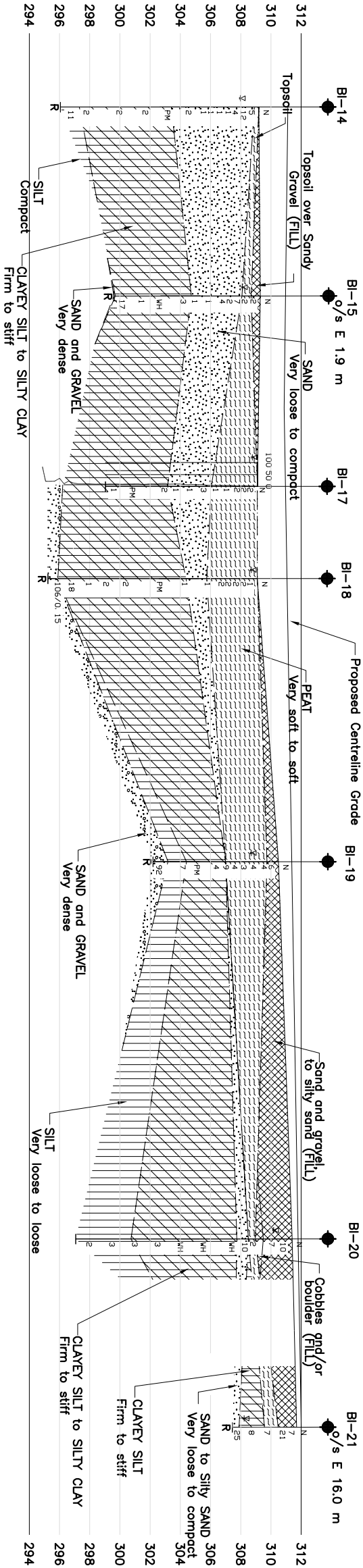
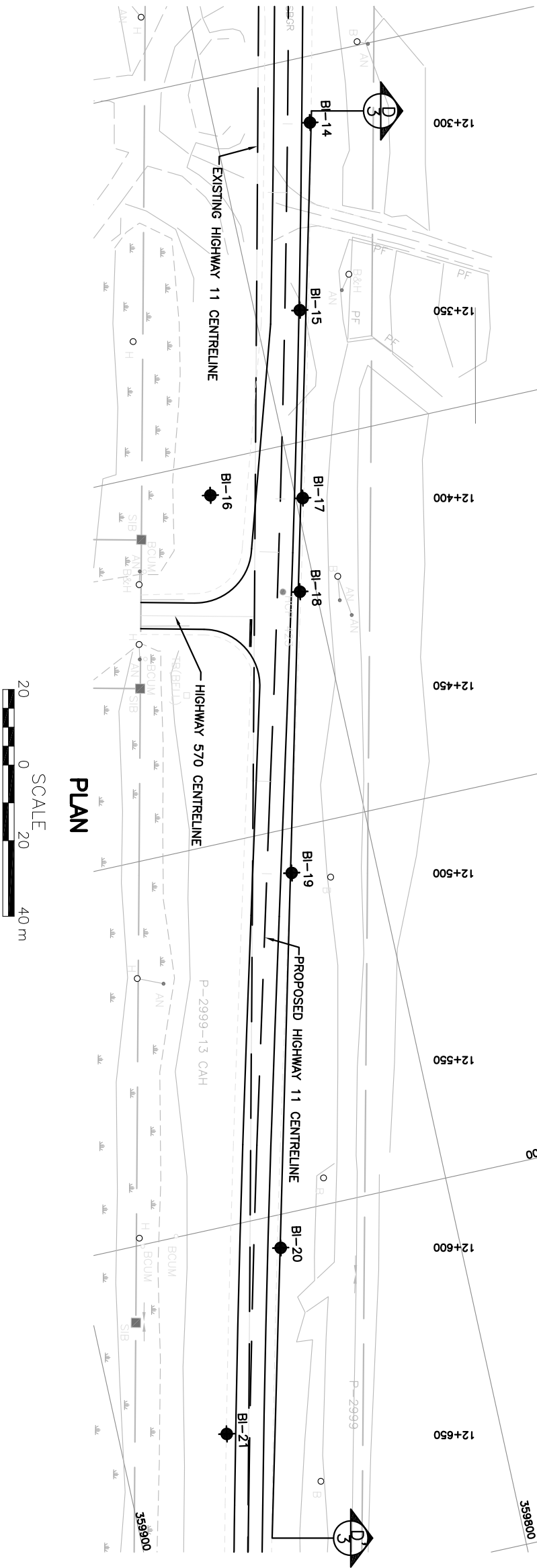
LEGEND

- BI-1 Borehole – Current Investigation
- PHBI-1 Probehole
- R Refusal
- N Standard Penetration Test
- 4 Blows/0.3 m unless otherwise stated (Std. Pen. Test, 4/5l/blow)
- 100% Rock Quality Designation (RQD)
- WL upon completion of drilling

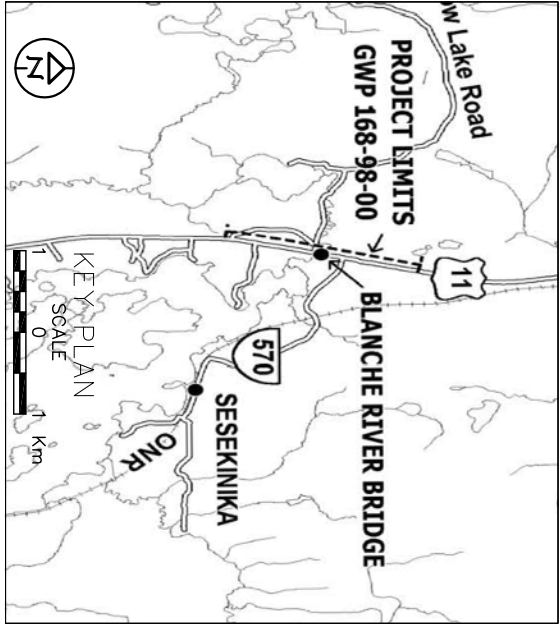
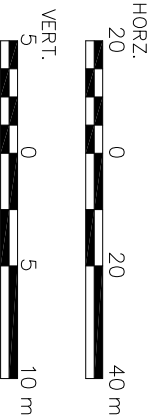


NO.	DATE	BY	REVISION	DIST.
HWY. 11	CHRD. AB	DATE: MAR. 2009	SITE: 47-006	
DRAWN: MM	CHKD. JPD	APPD. JMAC	DWG. 2	

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



PROFILE DD'



No.	ELEVATION(m)	CO-ORDINATES	
		NORTHING	EASTING
BI-14	309.2	5341517.4	359774.6
BI-15	309.3	5341565.7	359788.1
BI-16	310.4	5341608.7	359821.9
BI-17	309.1	5341614.7	359798.1
BI-18	309.1	5341638.9	359804.2
BI-19	310.5	5341711.7	359822.5
BI-20	311.4	5341808.7	359846.9
BI-21	311.7	5341854.1	359871.6

LEGEND	
	Borehole – Current Investigation
	Refusal
	Standard Penetration Test
	Blows/0.3 m unless otherwise stated (Std. Pen. Test, 475i/blow)
	WL upon completion of drilling

NOTES

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REFERENCE

Base plans provided in digital format by WLLS, drawing file name PROPOSED HIGHWAY.dwg received November, 2008 and Key Plan.dwg received September 11, 2008.

NO.	DATE	BY	REVISION	
Geocses No.	42A-73			
HWY. 11			PROJECT NO. 07-1191-0008	DIST.
SUBWD. AB	CHKD. AB	DATE: MAR. 2009		SITE: 47-006
DRAWN: MM	CHKD. JMAC	APPD. JMAC		DWG. 3

CONT No.
WP No. 168-98-00



HIGHWAY 11
STA. 12+270 to 12+675
BOREHOLE LOCATION AND
SOIL STRATA

SHEET

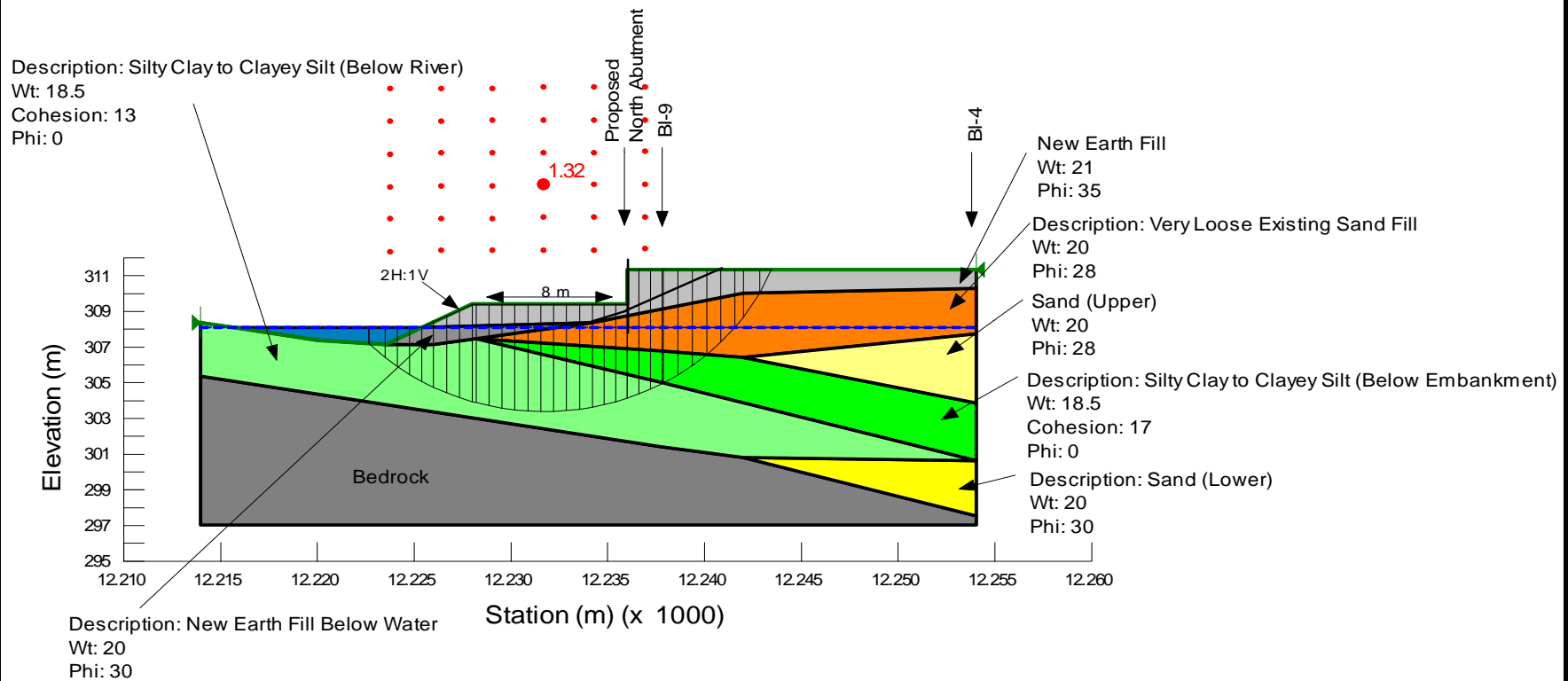


Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

STABILITY ANALYSIS

North Abutment Front Slope - Toe Berm Alternative

FIGURE 1



Date: April 2009
Project: 07-1191-0008

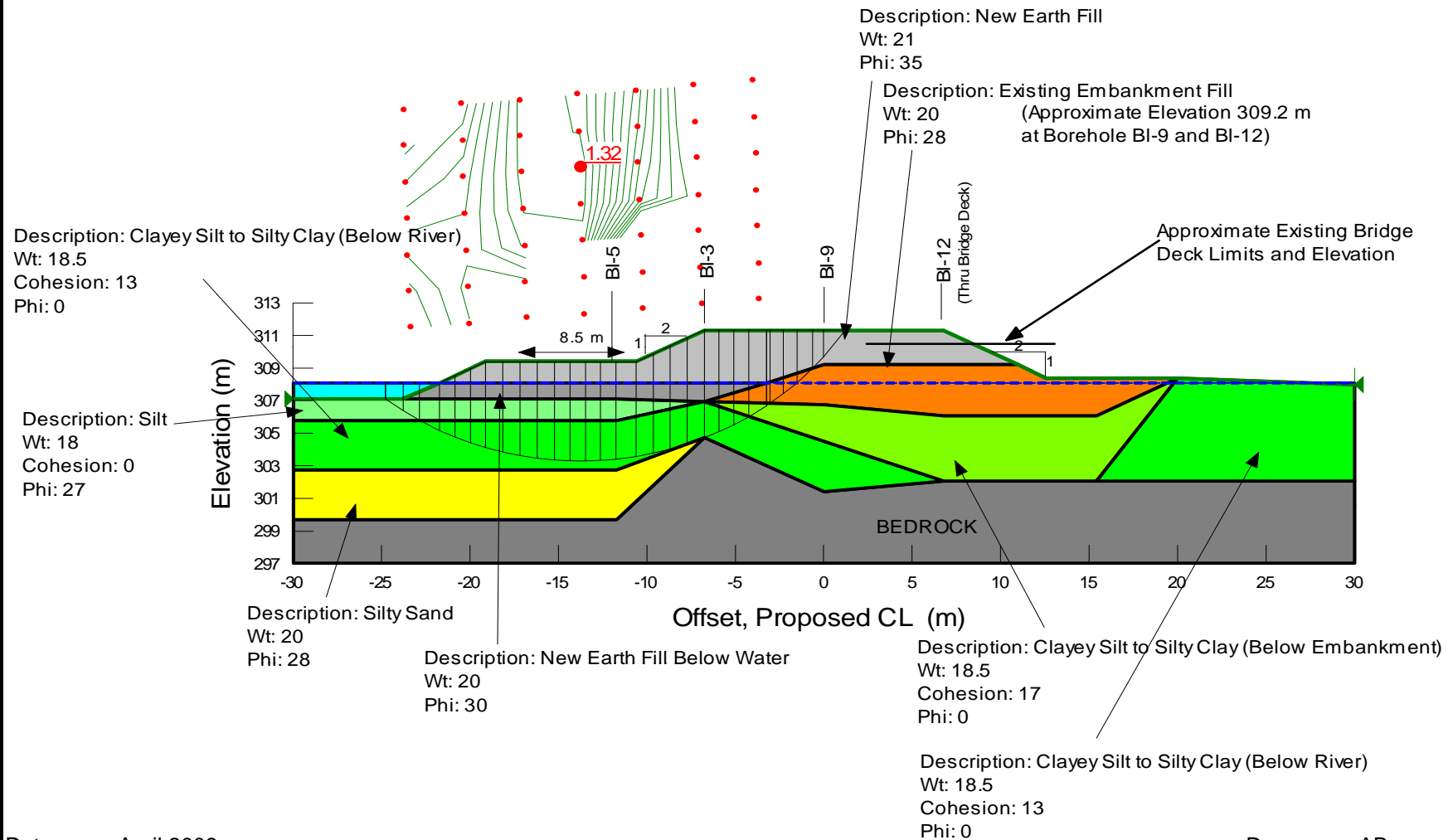
Golder Associates

Drawn: AB
Checked: JPD

STABILITY ANALYSIS

North Abutment West Slope - Toe Berm Alternative

FIGURE 2



Date: April 2009
Project: 07-1191-0008

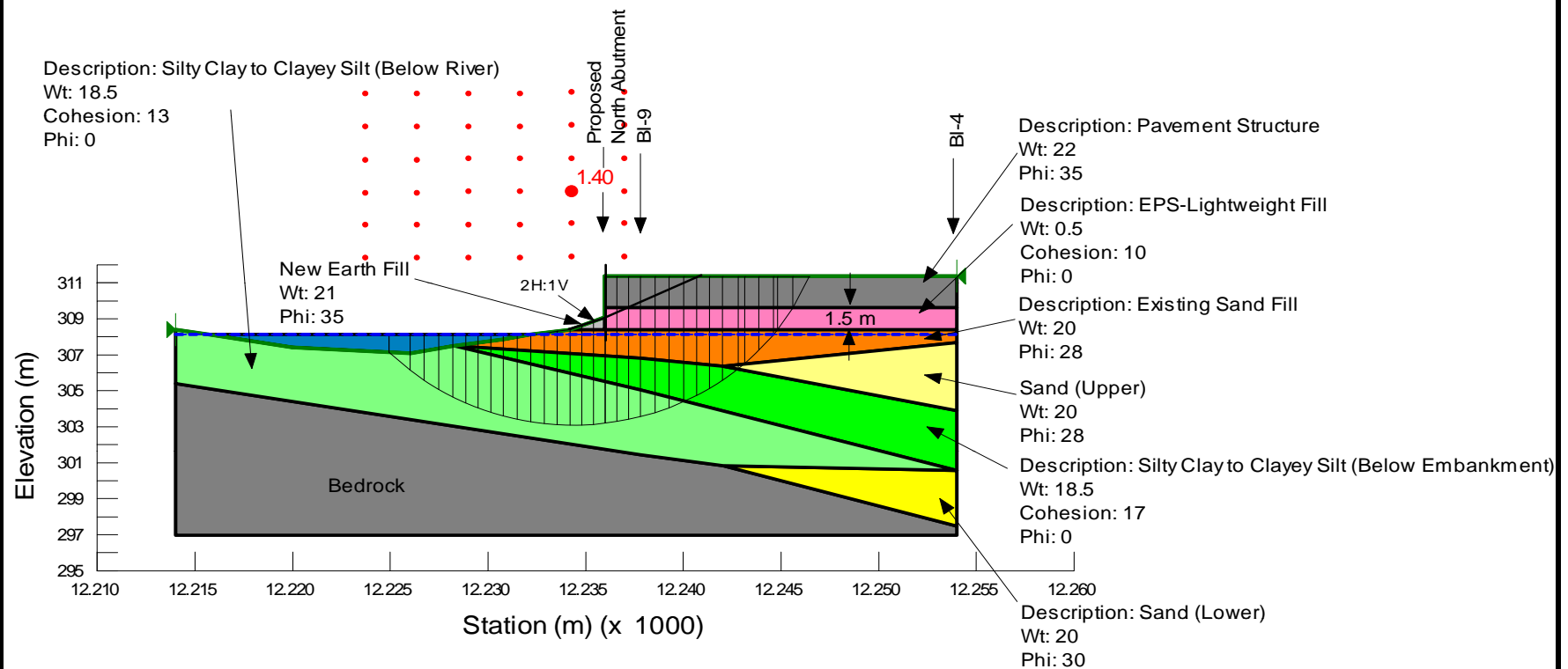
Golder Associates

Drawn: AB
Checked: JPD

STABILITY ANALYSIS

North Abutment Front Slope - EPS Fill Alternative

FIGURE 3



Date: April 2009
Project: 07-1191-0008

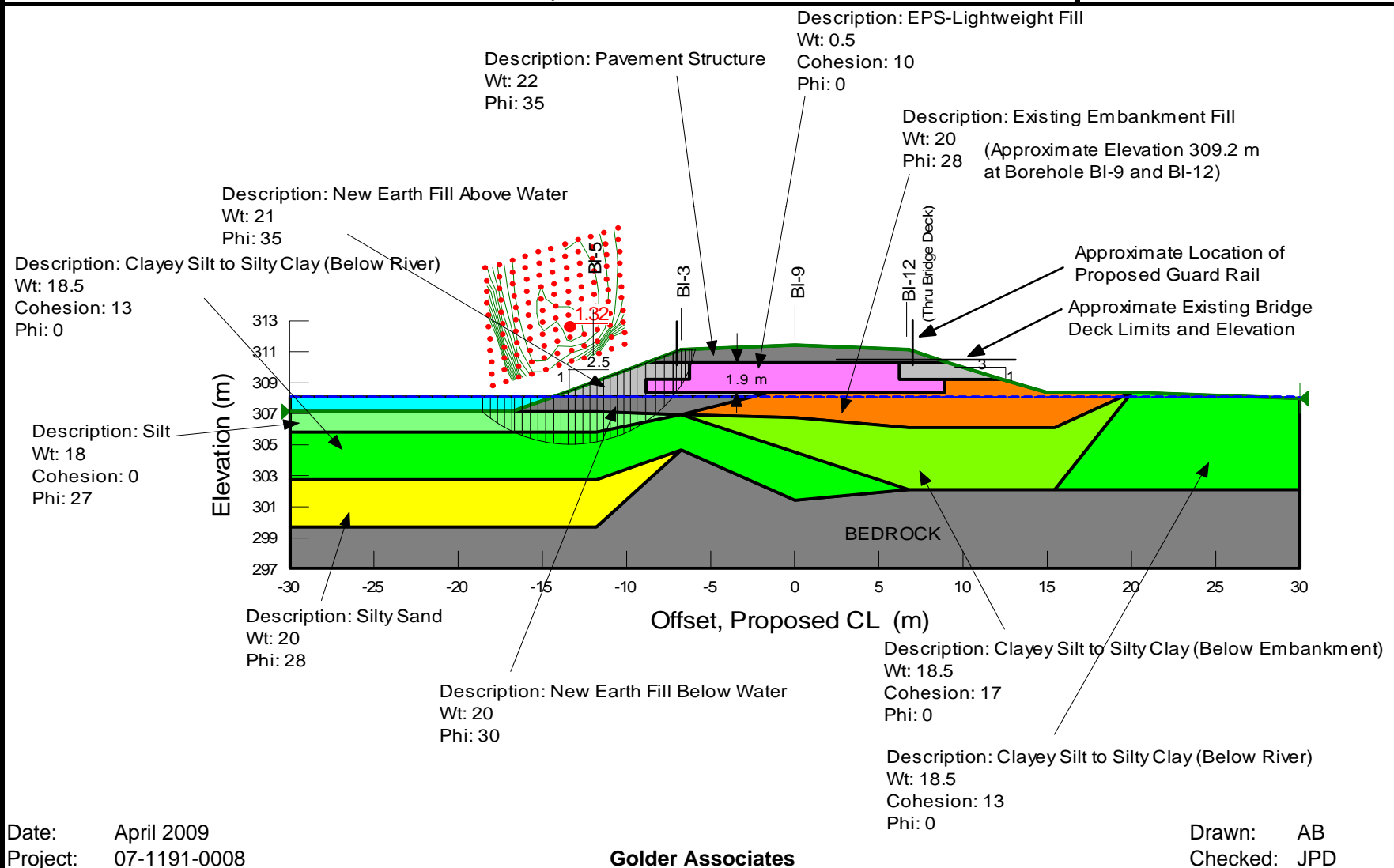
Golder Associates

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Checked: JPD

STABILITY ANALYSIS

North Abutment West Slope - EPS Fill Alternative

FIGURE 4



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance, N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezcone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

Consistency

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separation) 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 separation such as fractures, bedding planes and foliation planes or mechanically induced fractures 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	⊥ - Perpendicular To
FO - Foliation / Schistosity	- Parallel To
CL - Cleavage	P - Polished
SH - Shear Plane / Zone	K - Slickensided
VN - Vein	SM - Smooth
F - Fault	R - Rough
CO - Contact	ST - Stepped
J - Joint	PL - Planar
FR - Fracture	U - Undulating
MF - Mechanical Fracture	C - Curved

PROJECT 07-1191-0008			RECORD OF BOREHOLE No BI-1			1 OF 1 METRIC											
W.P. 168-98-00			LOCATION N 5341406.1 ; E 359752.5			ORIGINATED BY EC											
DIST HWY 11			BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers			COMPILED BY MM											
DATUM Geodetic			DATE July 8, 2008			CHECKED BY AB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W _p W W _L			γ	GR SA SI CL
309.9	GROUND SURFACE							20 40 60 80 100									
0.0	Sandy TOPSOIL Brown to black Moist		1	AS	-												
309.3	SILTY CLAY to CLAYEY SILT, trace to some sand Soft to firm Brown Moist to wet		2	SS	7		309										
0.6			3	SS	6		308										0 1 55 44
	Becoming grey and wet below 2.1 m depth.		4	TO	PM		307										
			5	SS	WH		306										
			6	SS	PM		305										0 13 61 26
304.4	End of Borehole Auger Refusal																
5.5	Note: 1. Water level measured in piezometer at a depth of 1.3 m below ground surface (Elev. 308.6 m) on September 4, 2008. 2. Shelby tube Sample # 4 had zero recovery. On August 27, 2008 Borehole BI-1a was advanced 1.0 m north of Borehole BI-1 to obtain shelly tube sample and additional field vane testing. Refer to Record of Borehole No. BI-1a.																

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

MIS-MTO 001 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/4/09 ACM

PROJECT: 07-1191-0008

RECORD OF DRILLHOLE: BI-2

SHEET 1 OF 1

LOCATION: N 5341425.1 ;E 359758.7

DRILLING DATE: July 7, 2008

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME - 850

DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	PENETRATION RATE min(m)	FLUSH	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES WATER LEVELS INSTRUMENTATION	
								RECOVERY	R.Q.D. %	FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q AVG.
								TOTAL CORE %	SOLID CORE %	B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr Ja Jn	
		Refer to Previous Page		306.00										
5	July 7, 2008 NQ RC	GRANITE Fine to medium grained Slightly weathered Very strong Pinkish grey		4.10	1						J, U, R			
					2									
		Broken core between 5.4 m and 5.5 m depth			3						J, U, R			UCS = 168 MPa
					4						J, U, R			
6					5						J, U, R			
		GABBRO Fine to medium grained Slightly weathered Strong to very strong Grey		303.90 6.20							J, U, R J, U, SM			UCS = 131 MPa
7					6						J, U, R J, U, R J, U, R J, U, R			
		End of Drillhole		302.60 7.50							J, U, R			
8														
9														
10														
11														
12														
13														
14														

DEPTH SCALE

1 : 50



LOGGED: EC

CHECKED: AB

MIS-RCK 004 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/4/09 ACM

PROJECT		RECORD OF BOREHOLE No BI-3				1 OF 1 METRIC										
W.P. 168-98-00		LOCATION N 5341455.4 ;E 359760.5				ORIGINATED BY EHS										
DIST HWY 11		BOREHOLE TYPE NW Casing, Wash Boring				COMPILED BY MM										
DATUM Geodetic		DATE July 3, 2008				CHECKED BY AB										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	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									
308.4	WATER SURFACE															
0.0	WATER															
307.5																
0.9	SAND, some silt, trace gravel, trace organics (ALLUVIUM)		1	SS	7											
306.9	Loose															
1.5	Brown to black		2	SS	WH											
	Wet															
	CLAYEY SILT to SILTY CLAY, trace to some sand		3	SS	2											
	Soft															
	Grey		4	SS	WH											
	Wet															
304.7																
3.7	GABBRO (BEDROCK)		1	RC	REC 100%											
	Bedrock cored from 3.7 m to 7.5 m depth.															
	For coring details refer to Record of Drillhole BI-3.		2	RC	REC 100%											
			3	RC	REC 100%											
300.9																
7.5	End of Borehole															

MIS-MTO 001 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/09 ACM

PROJECT: 07-1191-0008

RECORD OF DRILLHOLE: BI-3

SHEET 1 OF 1

LOCATION: N 5341455.4 ;E 359760.5

DRILLING DATE: July 3, 2008

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D25

DRILLING CONTRACTOR: Walker Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE min/(m)	FLUSH	COLLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock	NOTES WATER LEVELS INSTRUMENTATION	
									RECOVERY	R.Q.D. %	FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q AVG.
		Refer to Previous Page		304.70											
4		GABBRO Fine to medium grained Slightly weathered Very strong Grey		3.70	1							J, I, R			
												J, I, R			
												J, I, R			
												J, PL, SM			
5					2							J, U, SM			
												J, I, R			
												J, I, R			
6												J, I, R			
												J, I, R			
7					3							J, I, R			
												J, I, R			
												J, I, R			
												J, U, R			
		End of Drillhole		300.90											
8				7.50											
9															
10															
11															
12															
13															

DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: AB

MIS-RCK 004 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/4/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No BI-4			1 OF 1 METRIC											
W.P. 168-98-00			LOCATION N 5341471.9 ; E 359769.4			ORIGINATED BY EC											
DIST HWY 11			BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers			COMPILED BY MM											
DATUM Geodetic			DATE July 9, 2008			CHECKED BY AB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	15 30 45					
310.3	GROUND SURFACE																
0.0	Sand, trace silt, trace clay, trace organics (FILL) Very loose to loose Brown Moist to wet		1	AS	-		310										
			2	SS	5		309										0 87 (13)
			3	SS	3		308										
307.7			4	SS	6		307										
2.6	SAND, trace silt Very loose Grey Wet		5	SS	1		307										
	Approximately 1.8 m of heaving sands encountered at 3.8 m depth.		6	SS	WH		306										0 98 (2)
			7	SS	WR		305										
303.9			8	SS	WH		304										
6.4	SILTY CLAY to CLAYEY SILT, trace to some sand Soft Grey Wet		9	TO	PM		303										
			10	SS	WH		302										
300.5			11	SS	1		301										0 20 42 38
9.8	SAND to Silty SAND, some gravel, trace clay Very loose Grey Wet		12	SS	3		300										
	Approximately 2.1 m of heaving sands encountered at 12.2 m depth.						299										
297.5							298										15 65 (20)
12.8	End of Borehole																
	Note: 1. Water level measured in piezometer at a depth of 1.9 m below ground surface (Elev. 308.4 m) on September 3, 2008.																

PROJECT 07-1191-0008			RECORD OF BOREHOLE No BI-5			1 OF 1 METRIC									
W.P. 168-98-00			LOCATION N 5341465.2; E 359758.6			ORIGINATED BY EHS									
DIST HWY 11			BOREHOLE TYPE NW Casing, Wash Boring			COMPILED BY MM									
DATUM Geodetic			DATE July 4, 2008			CHECKED BY AB									
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							
308.4	WATER SURFACE														
0.0	WATER														
307.6															
0.8	SAND, containing organics, some silt, trace gravel (ALLUVIUM)		1	SS	5										
307.0	Loose														
1.4	Brown to black Wet														
	SILT, trace sand, trace clay, trace organics		2	SS	WH										
	Very loose														
305.8	Grey Wet														
2.6	CLAYEY SILT to SILTY CLAY, trace sand		3	SS	WH										
	Soft														
	Grey		4	SS	WH										
	Wet														
			5	SS	PM										
			6	SS	PM										
302.8															
5.6	Silty SAND, trace to some clay, trace to some gravel		7	SS	5										
	Loose to compact														
	Grey		8	SS	4										
	Wet														
			9	SS	12										
299.6	End of Borehole														
8.8	Casing Refusal														
	Notes:														
	1. Moved 1.5 m north and 0.5 m west to carry out field vane testing within cohesive deposit. Refer to Record of Borehole BI-5a.														



PROJECT		RECORD OF BOREHOLE				No BI-5a		1 OF 1		METRIC						
W.P.		168-98-00		LOCATION		N 5341467.4 ;E 359758.1		ORIGINATED BY		EHS						
DIST		HWY 11		BOREHOLE TYPE		NW Casing, Wash Boring		COMPILED BY		MM						
DATUM		Geodetic		DATE		July 5, 2008		CHECKED BY		AB						
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								
308.4 0.0	WATER SURFACE															
	Refer to Record of Borehole BI-5 for description of soil stratigraphy.															
302.9 5.5	End of Borehole															

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PROJECT 07-1191-0008		RECORD OF BOREHOLE No BI-6				1 OF 1 METRIC											
W.P. 168-98-00		LOCATION N 5341423.2 ; E 359753.2				ORIGINATED BY TR											
DIST HWY 11		BOREHOLE TYPE Portable Equipment - BW Casing, Wash Boring				COMPILED BY MM											
DATUM Geodetic		DATE August 25, 2008				CHECKED BY AB											
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									
309.6	GROUND SURFACE							20	40	60	80	100					
0.0	TOPSOIL																
0.2	Brown to black Moist		1	SS	5												
	Sand, trace to some silt (FILL)		2	SS	5												
	Very loose to loose																
	Brown																
	Moist to wet																
307.8	COBBLES and/or BOULDER		3	SS	1												
307.5																	
2.1	SAND, some silt		4	SS	4												
	Very loose to loose																
	Brown to grey																
	Wet																
306.9			5	SS	9												
	Sandy SILT, trace clay																
	Loose to compact																
	Grey																
	Wet																
305.6			6	SS	23												
4.0	GABBRO (BEDROCK)		1	RC	REC 100%												
	Bedrock cored from 4.0 m to 7.1 m depth.																
	For coring details refer to Record of Drillhole BI-6.																
			2	RC	REC 85%												
			3	RC	REC 100%												
			4	RC	REC 100%												
			5	RC	REC 100%												
302.5																	
7.1	End of Borehole																
	Note:																
	1. Water level at a depth of 1.4 m below ground surface (Elev. 308.2 m) upon completion of drilling.																

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PROJECT		07-1191-0008		RECORD OF BOREHOLE No BI-7		1 OF 1		METRIC						
W.P.		168-98-00		LOCATION		N 5341427.1 ; E 359754.1		ORIGINATED BY						
DIST		HWY 11		BOREHOLE TYPE		Portable Equipment, BW Casing, Wash Boring		COMPILED BY						
DATUM		Geodetic		DATE		August 25, 2008		CHECKED BY						
								AB						
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						
309.3	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	15 30 45				
0.0	Topsoil (FILL) Brown to black Moist		1	SS	4		309							
0.6	Silty sand, trace clay (FILL) Very loose Grey Moist		2	SS	3		308							
307.5	Clayey silt, some sand, containing sand pockets (FILL) Very soft to soft Brown to grey Moist to wet		3	SS	1									
307.0	PEAT Very soft Brown to black Wet		4	SS	2		307							
2.3														
306.4	SILT, some sand, trace clay Loose Grey Moist to wet		5	SS	8									
3.0	SAND and GRAVEL Loose Grey Moist to wet End of Borehole Split-Spoon Refusal													
Note: 1. Water level in open borehole at a depth of 1.2 m below ground surface (Elev. 308.1 m) upon completion of drilling.														

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PROJECT 07-1191-0008			RECORD OF BOREHOLE No BI-8			1 OF 1 METRIC											
W.P. 168-98-00			LOCATION N 5341458.8 ; E 359759.2			ORIGINATED BY TR											
DIST HWY 11			BOREHOLE TYPE Portable Equipment, BW Casing, Wash Boring			COMPILED BY MM											
DATUM Geodetic			DATE August 27, 2008			CHECKED BY AB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	15 30 45					
308.1 0.0	WATER SURFACE WATER					▽	308										
307.3 0.8	SAND, trace silt, containing organics (ALLUVIUM) Very loose Brown to black Wet		1	SS	2		307										
306.7 1.4	CLAYEY SILT to SILTY CLAY, trace to some sand, trace gravel Soft Grey Wet		2	SS	WH		306										
			3	AS	-		305										
			4	AS	-												
304.5 3.6	End of Borehole Split-Spoon Refusal Note: 1. Moved 1.0 m south and 0.5 m east to obtain shelly tube sample of silty clay stratum. Refer to Record of Borehole BI-8a.																

MIS-MTO 001 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/4/09 ACM



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

MIS-MTO 001 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/4/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No BI-9			1 OF 1 METRIC																
W.P. 168-98-00			LOCATION N 5341456.1 ; E 359766.8			ORIGINATED BY TR																
DIST HWY 11			BOREHOLE TYPE Portable Equipment, BW Casing, Wash Boring			COMPILED BY MM																
DATUM Geodetic			DATE August 27, 2008			CHECKED BY AB																
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			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		ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR	SA	SI	CL		
							20 40 60 80 100	○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× REMOULDED	W _p	W	W _L	15 30 45							
309.2 0.0	GROUND SURFACE Sand, trace to some gravel, trace silt (FILL) Very loose to compact Brown Moist to wet		1	SS	4		309															
			2	SS	2		308															
			3	SS	WH																	
			4	SS	12																	
306.8 2.4	CLAYEY SILT, some sand Soft Brown to grey Wet		5	SS	3																	
			6	SS	3																	
			7	AS	-																	
			8	TO	PM																	
			9	SS	2																	
		10	SS	WH																		
301.4 7.8	GABBRO (BEDROCK) Bedrock cored from 7.8 m to 10.9 m depth. For coring details refer to Record of Drillhole BI-9.		11	SS	11/0.15																	
			1	RC	REC 100%		301													RQD = 88%		
			2	RC	REC 100%		300													RQD = 81%		
			3	RC	REC 100%															RQD = 65%		
			4	RC	REC 100%		299													RQD = 31%		
298.3 10.9	End of Borehole Note: 1. Water level at a depth of 1.0 m below ground surface (Elev. 308.2 m) upon completion of drilling.																					

MIS-MTO 001 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/04/09 ACM

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: OGS Drilling

CHECKED: AB

MIS-RCK 004 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 30/4/09 ACM

PROJECT 07-1191-0008		RECORD OF BOREHOLE No BI-10				1 OF 1 METRIC										
W.P. 168-98-00		LOCATION N 5341421.6 ; E 359765.1				ORIGINATED BY EHS										
DIST HWY 11		BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers				COMPILED BY MM										
DATUM Geodetic		DATE September 4, 2008				CHECKED BY AB										
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								
310.5	GROUND SURFACE															
0.0	ASPHALT															
310.0	Sand and gravel (FILL)		1	AS	-											
309.7	Brown Moist															
0.8	Cobbles and/or boulder (FILL)		2	SS	25											
	Sand, some silt, trace gravel (FILL)															
	Very loose to compact															
	Brown Moist		3	SS	4											
			4	SS	1											
307.6	PEAT															
2.9	Very soft															
307.1	Brown Wet		5	SS	1											
3.4	SILTY CLAY															
306.4	Firm Grey Wet															
4.1	SILT, trace to some sand, trace clay															
305.5	Loose Grey Wet		6	SS	8											
5.0	GABBRO (BEDROCK)															
	Bedrock cored from 5.0 m to 8.1 m depth.		1	RC	REC 100%											
	For coring details see Record of Drillhole BI-10.															
			2	RC	REC 100%											
302.4	End of Borehole															
8.1	Note:															
	1. Water level at a depth of 2.3 m (Elev. 308.2 m) below ground surface upon completion of drilling.															

MIS-MTO 001 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/09 ACM

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Landcore Drilling

CHECKED: AB

MIS-RCK 004 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/4/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No BI-11			1 OF 1 METRIC											
W.P. 168-98-00			LOCATION N 5341425.5 ; E 359765.9			ORIGINATED BY EHS											
DIST HWY 11			BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers			COMPILED BY MM											
DATUM Geodetic			DATE September 3, 2008			CHECKED BY AB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L WATER CONTENT (%)			γ	GR SA SI CL
310.5	GROUND SURFACE							20 40 60 80 100									
0.0	ASPHALT							20 40 60 80 100									
0.3	Sand and gravel (FILL) Brown Moist Sand, trace to some silt, trace gravel (FILL) Very loose to compact Brown Moist to wet		1	SS	12		310										
			2	SS	5		309										
			3	SS	1		308										1 92 (7)
307.3	PEAT Very soft Brown Wet		4	SS	1		307										
306.8	SILT, some sand, some clay Very loose Grey Wet		5	SS	3		306										0 14 71 15
305.5	End of Borehole Split-Spoon and Auger Refusal		6	SS	16/0.2												
5.0	Note: 1. Water level at a depth of 2.2 m (Elev. 308.3 m) below ground surface upon completion of drilling. 2. Cobbles and/or boulder encountered at a depth of 0.4 m depth in original borehole, moved 0.3 m east to continue drilling.																

MIS-MTO 001 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/04/09 ACM

PROJECT		07-1191-0008		RECORD OF BOREHOLE No BI-12		1 OF 1 METRIC												
W.P.		168-98-00		LOCATION		N 5341452.8 ; E 359772.2												
DIST		HWY 11		BOREHOLE TYPE		NW Casing, Wash Boring												
DATUM		Geodetic		DATE		September 3, 2008												
ORIGINATED BY		EHS		COMPILED BY		MM												
CHECKED BY		AB																
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	15 30 45					
310.5		GROUND SURFACE																
310.0		ASPHALT (110 mm) over CONCRETE (165 mm)																
310.2		TIMBER BRIDGE DECK																
0.5		VOID (Beneath Bridge)																
309.1																		
1.4		Sand, some gravel, trace silt (FILL) Very loose Brown Wet																
				1	SS	1												
				2	SS	3												
				3	SS	3												
306.1																		
4.4		CLAYEY SILT to SILTY CLAY, trace sand Soft to firm Grey Wet		4	SS	WH												
				5	SS	WH												
302.4				6	SS	2												
302.1		SILT, trace to some clay, trace sand Loose Grey Wet																
8.4		End of Borehole Casing Refusal																
<p>Note:</p> <p>1. Borehole advanced through existing bridge deck.</p> <p>2. Water level at a depth of 2.1 m (Elev. 308.4 m) below bridge deck upon completion of drilling.</p>																		

PROJECT 07-1191-0008		RECORD OF BOREHOLE No BI-13		1 OF 1 METRIC								
W.P. 168-98-00		LOCATION N 5341458.6 ; E 359773.5		ORIGINATED BY EHS								
DIST _____ HWY 11		BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers - NW Casing, Wash Boring		COMPILED BY MM								
DATUM Geodetic		DATE September 4, 2008		CHECKED BY AB								
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					
310.5	GROUND SURFACE											
0.0	ASPHALT											
0.4	Sand and gravel (FILL) Brown Moist		1	AS	-							
	Sand, trace to some gravel, trace silt (FILL) Very loose to compact Brown Moist to wet		2	SS	22							
			3	SS	18							
			4	SS	2							
307.3	PEAT Brown Wet		5	SS	5							
3.4	SAND, trace to some silt Very loose to loose Brown to grey Wet		6	SS	WH							
			7	SS	1							
305.2	CLAYEY SILT to SILTY CLAY, trace to some sand Soft to firm Grey Wet		8	SS	2							
5.3			9	SS	WH							
			10	SS	WH							
			11	SS	WR							
300.1	Start of DCPT											
299.7	End of DCPT (Hammer Bouncing) End of Borehole											
10.8	Note: 1. Water level at a depth of 2.2 m (Elev. 308.3 m) below ground surface upon completion of drilling. 2. Bedrock coring not carried out due to sand boils in adjacent river bed next to existing bridge piles during drilling. Discussed with LEA Consulting Ltd. and MTO Foundations on September 4, 2008.											

PROJECT 07-1191-0008			RECORD OF BOREHOLE No BI-14				1 OF 1 METRIC							
W.P. 168-98-00		LOCATION N 5341517.4 ; E 359774.6		ORIGINATED BY ID										
DIST HWY 11		BOREHOLE TYPE BW Casing, Wash Boring		COMPILED BY MM										
DATUM Geodetic		DATE October 21 and 22, 2008		CHECKED BY AB										
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
309.2	GROUND SURFACE													
308.9	TOPSOIL		1	SS	5		309							
0.3	Brown Moist		2	SS	12		308							
	SAND, trace silt, trace gravel		3	SS	4									0 98 (2)
	Very loose to compact		4	SS	1		307							
	Brown		5	SS	1									
	Moist to wet		6	SS	1		306							2 97 (1)
	Becoming grey below 1.8 m depth.		7	SS	1		305							
			8	SS	2		304							
303.4	CLAYEY SILT, containing silt seams and layers		9	TO	PM		303							
5.8	Firm Grey Wet		10	SS	2		302	2.5 + 2.7 +						
			11	SS	2		301	2.5 + 3.3 +						
			12	SS	2		300	3.5 + 4 +						
297.3	SILT, trace to some sand, trace to some clay		13	SS	11		299							
11.9	Compact Grey Wet						298							0 6 83 11
296.1	End of Borehole Casing Refusal						297							
13.1	Note: 1. Water level in open borehole at a depth of 1.2 m below ground surface (Elev. 308.0 m) upon completion of drilling.													

PROJECT 07-1191-0008			RECORD OF BOREHOLE No BI-15			1 OF 1 METRIC											
W.P. 168-98-00			LOCATION N 5341565.7 ; E 359788.1			ORIGINATED BY ID											
DIST HWY 11			BOREHOLE TYPE BW Casing, Wash Boring			COMPILED BY MM											
DATUM Geodetic			DATE October 21, 2008			CHECKED BY AB											
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 kN/m³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	15 30 45					
309.3	GROUND SURFACE																
0.0	Topsoil (FILL) Brown Moist		1	SS	2		309										
308.7	Sandy gravel (FILL) Very loose Brown Moist		2	SS	2		308										
308.1	PEAT Very soft Brown Moist to wet		3	SS	7												
1.2	SAND, trace gravel, trace silt Very loose to loose Brown to grey Wet		4	SS	2		307										
			5	SS	4												
			6	SS	1		306										
			7	SS	1												
304.7	CLAYEY SILT, containing silt seams and layers Firm Grey Wet		8	SS	3		305										
4.6							304										
			9	SS	WH		303										
							302										
			10	SS	1		301										
							300										
299.7	SAND and GRAVEL Compact Grey Wet		11	SS	17												
9.7	End of Borehole Spoon Refusal																
	Note: 1. Water level in open borehole at a depth of 1.2 m below ground surface (Elev. 308.1 m) upon completion of drilling.																

MIS-MTO 001 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/4/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No BI-16			1 OF 1 METRIC											
W.P. 168-98-00			LOCATION N 5341608.7 ; E 359821.9			ORIGINATED BY ID											
DIST HWY 11			BOREHOLE TYPE BW Casing, Wash Boring			COMPILED BY MM											
DATUM Geodetic			DATE October 14 and 15, 2008			CHECKED BY AB											
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									WATER CONTENT (%)
310.4	GROUND SURFACE							20	40	60	80	100					
0.0	Sand and gravel (FILL) Compact Brown Wet		1	SS	24												
			2	SS	24												
			3	SS	23												
308.3			4	SS	14												
2.1	Silt (FILL) Loose to compact Grey Wet		5	SS	7												
307.7			6	SS	4												
2.7	PEAT Soft to firm Brown Wet		7	SS	7												
306.1			8	SS	2												
4.3	CLAYEY SILT, containing sand and silt seams and layers Firm Grey Wet		9	SS	4												
	Unable to push vane to 7.0 m depth.		10	SS	4												
			11	SS	WH												
			12	SS	1												
297.3	End of Borehole																
13.1	Note: 1. Water level in open borehole at a depth of 1.5 m below ground surface (Elev. 308.9 m) upon completion of drilling.																

MIS-MTO 001 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/4/09 ACM

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

1 OF 2 **METRIC**

CHECKED BY AB

MIS-MTO 001 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/4/09 ACM

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



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

MIS-MTO 001 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/4/09 ACM

PROJECT		07-1191-0008		RECORD OF BOREHOLE No BI-19		1 OF 1 METRIC												
W.P.		168-98-00		LOCATION		N 5341711.7 ; E 359822.5												
DIST		HWY 11		BOREHOLE TYPE		BW Casing, Wash Boring												
DATUM		Geodetic		DATE		October 22 and 23, 2008												
ORIGINATED BY		ID		COMPILED BY		MM												
CHECKED BY		AB																
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	15 30 45					
310.5	0.0	GROUND SURFACE		1	SS	6		310										
309.7	0.8	Sand and gravel, pockets of silty clay (FILL) Loose Brown Moist		2	SS	4		309										
		PEAT Soft Brown Wet		3	SS	4		308										
				4	SS	3		307										
				5	SS	4		306										
				6	SS	9		305										
307.0				7	SS	4		304										
	3.7	SILT Loose Grey Wet		8	TO	PM		303										
		CLAYEY SILT, containing silt seams and layers Firm Grey Wet																
304.4	6.1	SILT, trace to some clay, trace sand Loose Grey Wet		9	SS	7												
303.2	7.3	SAND and GRAVEL Very dense Grey Wet		10	SS	92												
302.2	8.3	End of Borehole Casing Refusal																
		Note: 1. Water level in open borehole at a depth of 1.8 m below ground surface (Elev. 308.7 m) upon completion of drilling.																

MIS-MTO 001 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/4/09 ACM

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



PROJECT		RECORD OF BOREHOLE					2 OF 2		METRIC			
W.P.		LOCATION					ORIGINATED BY		ID			
DIST		BOREHOLE TYPE					COMPILED BY		MM			
DATUM		DATE					CHECKED BY		AB			
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	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	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)		
--- CONTINUED FROM PREVIOUS PAGE ---												
	End of Borehole Note: 1. Water level in open borehole at a depth of 1.2 m below ground surface (Elev. 310.2 m) upon completion of drilling. 2. Auger refusal at 2.1 m depth; moved 1.5 m south to continue advancing borehole.											

MIS-MTO 001 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No BI-21			1 OF 1 METRIC														
W.P. 168-98-00			LOCATION N 5341854.1 ; E 359871.6			ORIGINATED BY ID														
DIST HWY 11			BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers			COMPILED BY MM														
DATUM Geodetic			DATE November 4, 2008			CHECKED BY AB														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W _p W W _L			γ	GR	SA	SI	CL
311.7	GROUND SURFACE							20 40 60 80 100												
0.0	Sand and gravel to sandy gravel (FILL) Grey Moist						311													
310.5			1	SS	7															
1.2	PEAT, containing cobbles and/or boulder Brown Moist						310													
309.6			2	SS	21															
2.1	SILT and SAND to SILT, some clay Loose Grey Moist to wet						309													
			3	SS	7															
			4	SS	8															
307.9							308													
3.8	SAND, some gravel, trace to some silt Compact Grey Wet		5	SS	25															
307.4																				
4.3	End of Borehole Spoon and Auger Refusal Note: 1. Water level in open borehole at a depth of 3.4 m below ground surface (Elev. 308.3) upon completion of drilling. 2. Moved 2.0 m north, auger refusal at 4.1 m depth.																			

MIS-MTO 001 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/4/09 ACM



PROJECT		RECORD OF BOREHOLE				No PHBI-1		1 OF 1		METRIC										
W.P.		LOCATION		N 5341457.9 ; E 359762.5		ORIGINATED BY		ID												
DIST		HWY		BOREHOLE TYPE		COMPILED BY		MM												
DATUM		DATE		October 17, 2008		CHECKED BY		AB												
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		W _p		W		W _L		γ		GR SA SI CL	
308.1	0.0	WATER SURFACE						308	20 40 60 80 100	○ UNCONFINED + FIELD VANE	15 30 45									
								307	20 40 60 80 100	● QUICK TRIAXIAL × REMOULDED										
								306	20 40 60 80 100											
								305	20 40 60 80 100											
								304	20 40 60 80 100											
302.9	5.2	End of Borehole						303	20 40 60 80 100											
		Note: 1. Rods noted to be sliding to the east at a depth of 5.2 m below ground surface (Elev. 302.9 m).																		

MIS-MTO 001 BLANCHE RIVER BH LOGS METRIC.GPJ GAL-MISS.GDT 29/4/09 ACM



APPENDIX A

Laboratory Test Results Bridge Structure and Immediate Approach Embankments

TABLE A-1
UNIAXIAL COMPRESSIVE STRENGTH TEST RESULTS
REPLACEMENT OF BLANCHE RIVER STRUCTURE
W.P 168-98-00, SITE NO. 47-006
HIGHWAY 11, TOWNSHIP OF MAISONVILLE

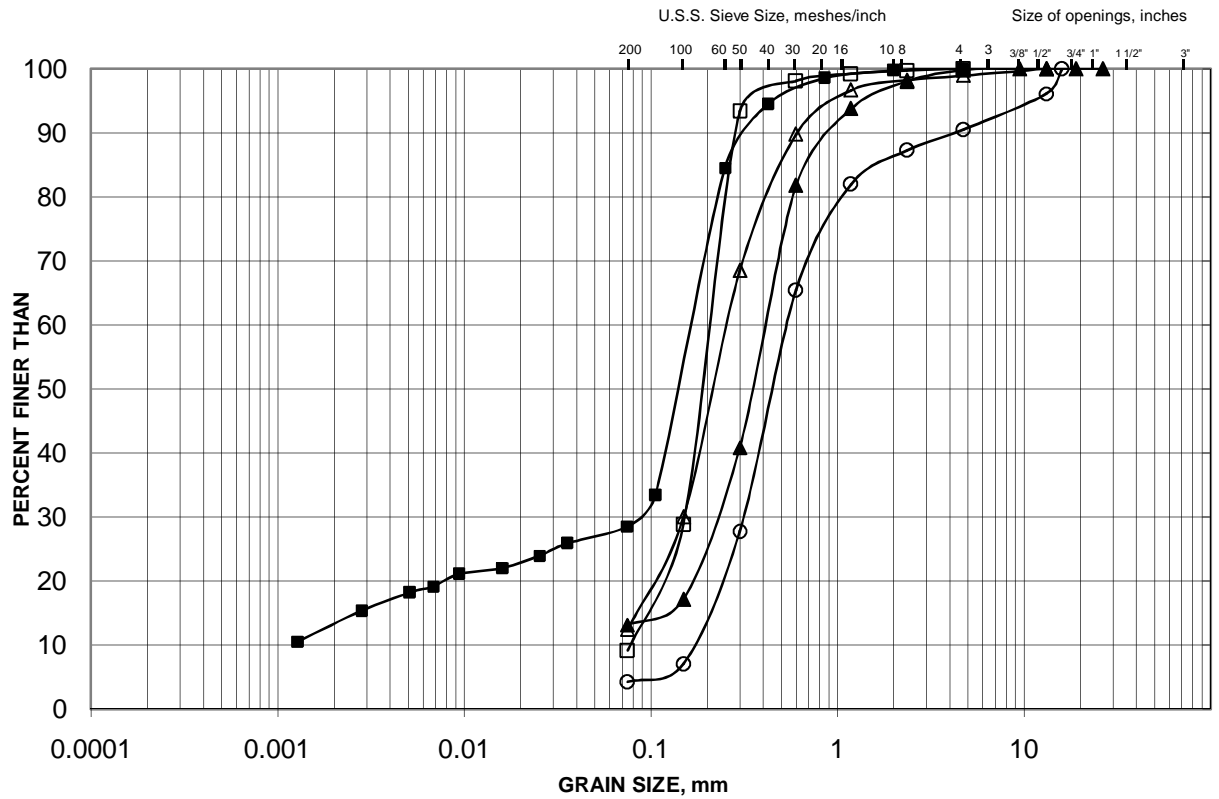
Borehole Number	Sample Depth (m)	Sample Elevation (m)	Rock Type	Core Diameter (mm)	Uniaxial Compressive Strength (MPa)
BI-2	4.8	305.4	Granite	47	168
BI-2	6.4	303.8	Gabbro	48	131
BI-3	4.2	304.2	Gabbro	47	201
BI-3	5.7	302.7	Gabbro	47	170
BI-6	4.8	304.8	Gabbro	51	152
BI-9	8.2	301.0	Gabbro	51	62
BI-9	9.9	299.3	Gabbro	51	59
BI-10	5.9	304.6	Gabbro	47	89
BI-10	7.4	303.1	Gabbro	47	55

Compiled by: AB
Reviewed By: JMAC

GRAIN SIZE DISTRIBUTION

Sand (Fill)

FIGURE
A-1a



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
■	BI-2	2	309.2
▲	BI-4	2	309.2
○	BL-9	3	307.7
△	BL-10	2	309.5
□	BI-6	2	308.8

Project Number: 07-1191-0008

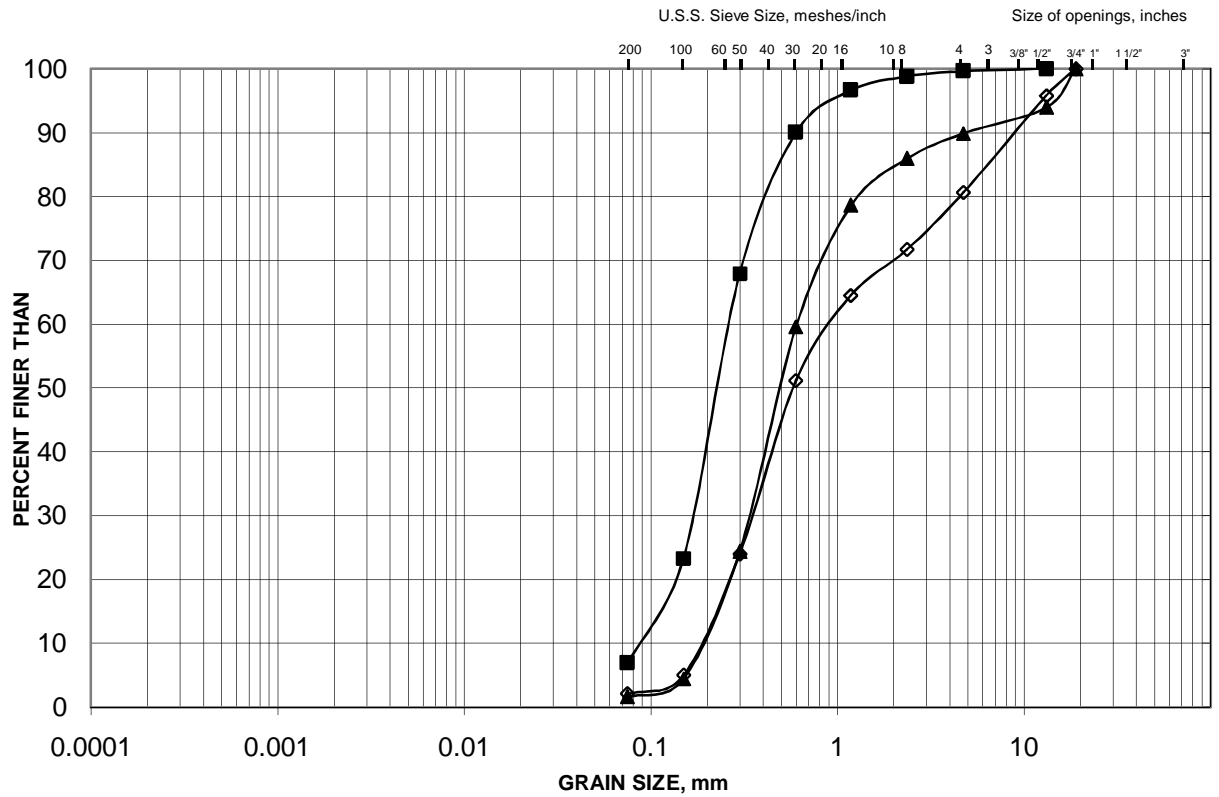
Checked By: AB

Golder Associates

Date: April 2009

GRAIN SIZE DISTRIBUTION Sand (Fill)

**FIGURE
A-1b**



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

LEGEND

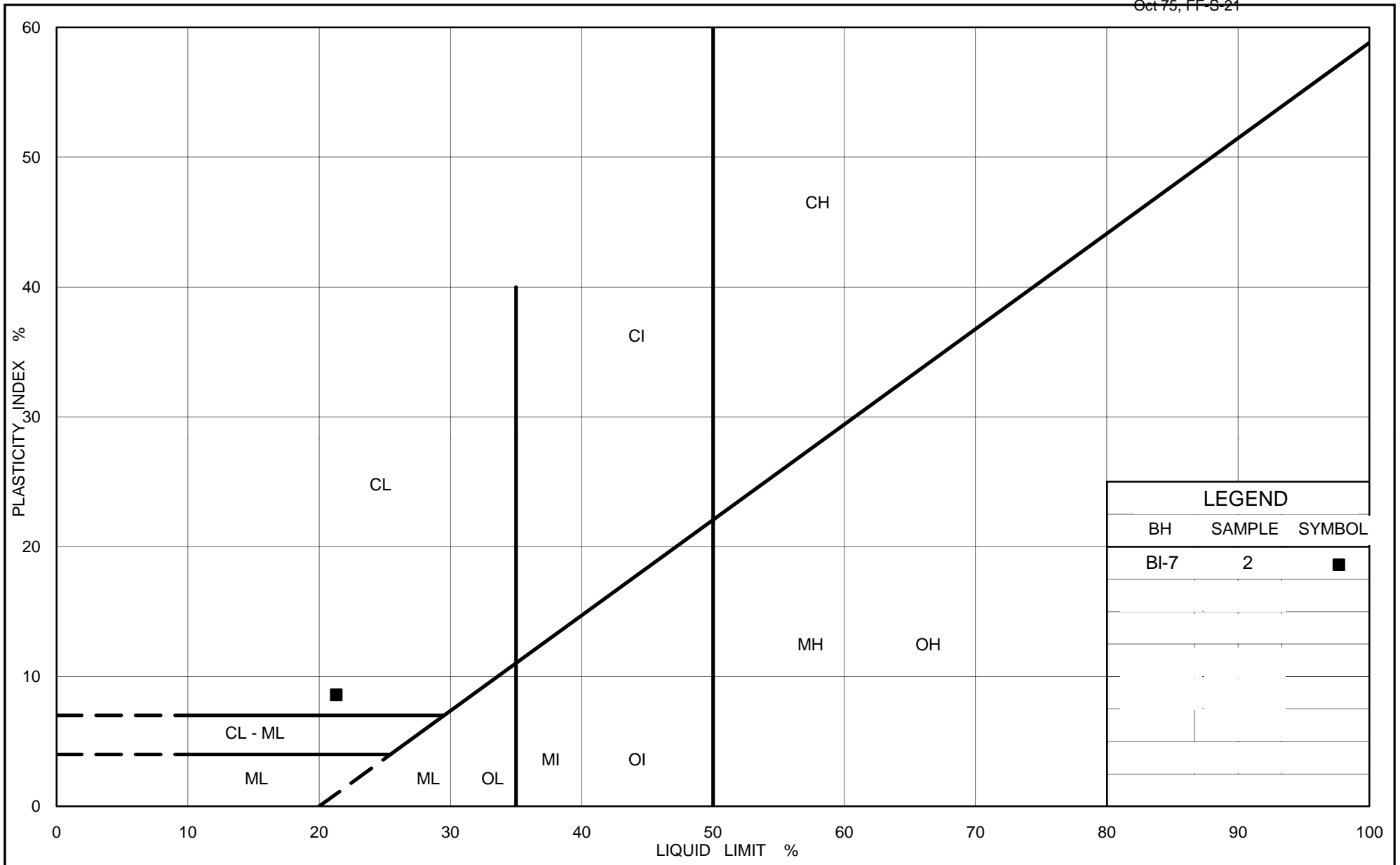
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
■	BI-11	3	307.8
◇	BI-12	2	307.2
▲	BI-13	4	307.9

Project Number: 07-1191-0008

Checked By: AB

Golder Associates

Date: April 2009

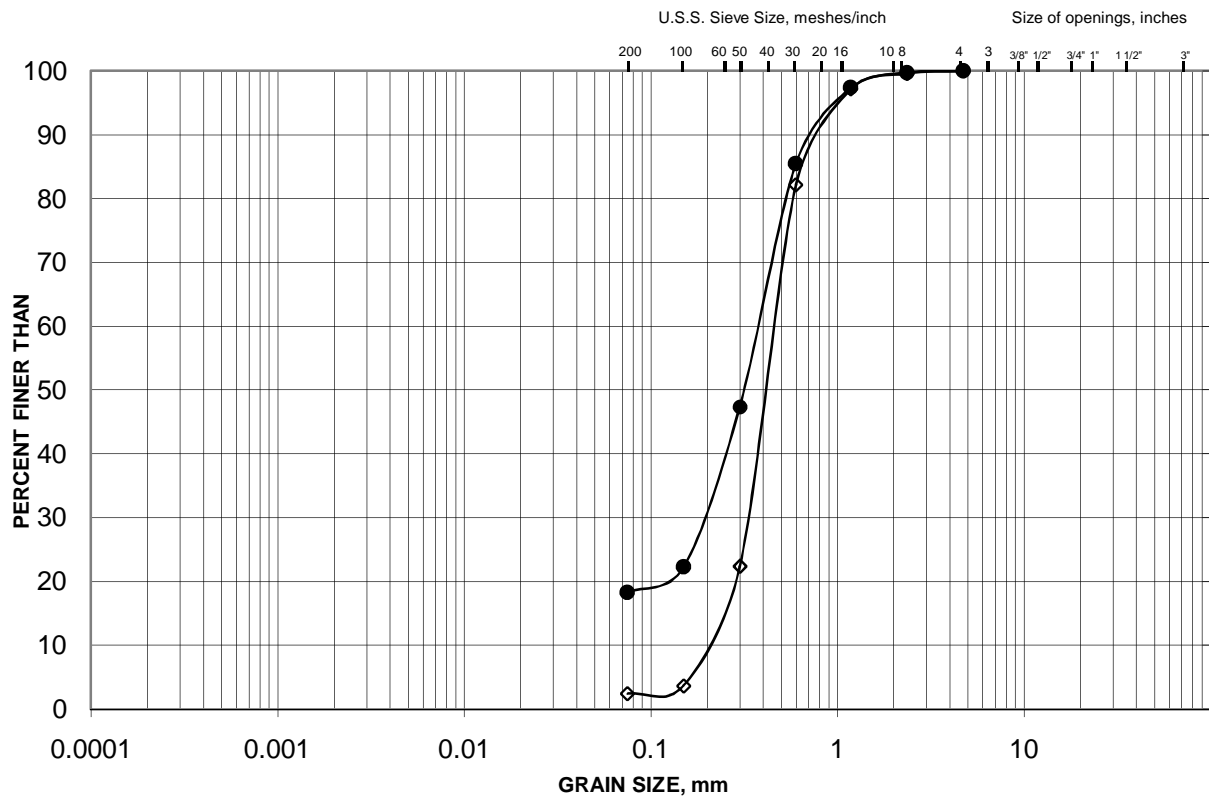


GRAIN SIZE DISTRIBUTION

Sand

FIGURE

A-3a



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—◇—	BI-4	6	306.2
—●—	BI-13	7	305.6

Project Number: 07-1191-0008

Checked By: AB

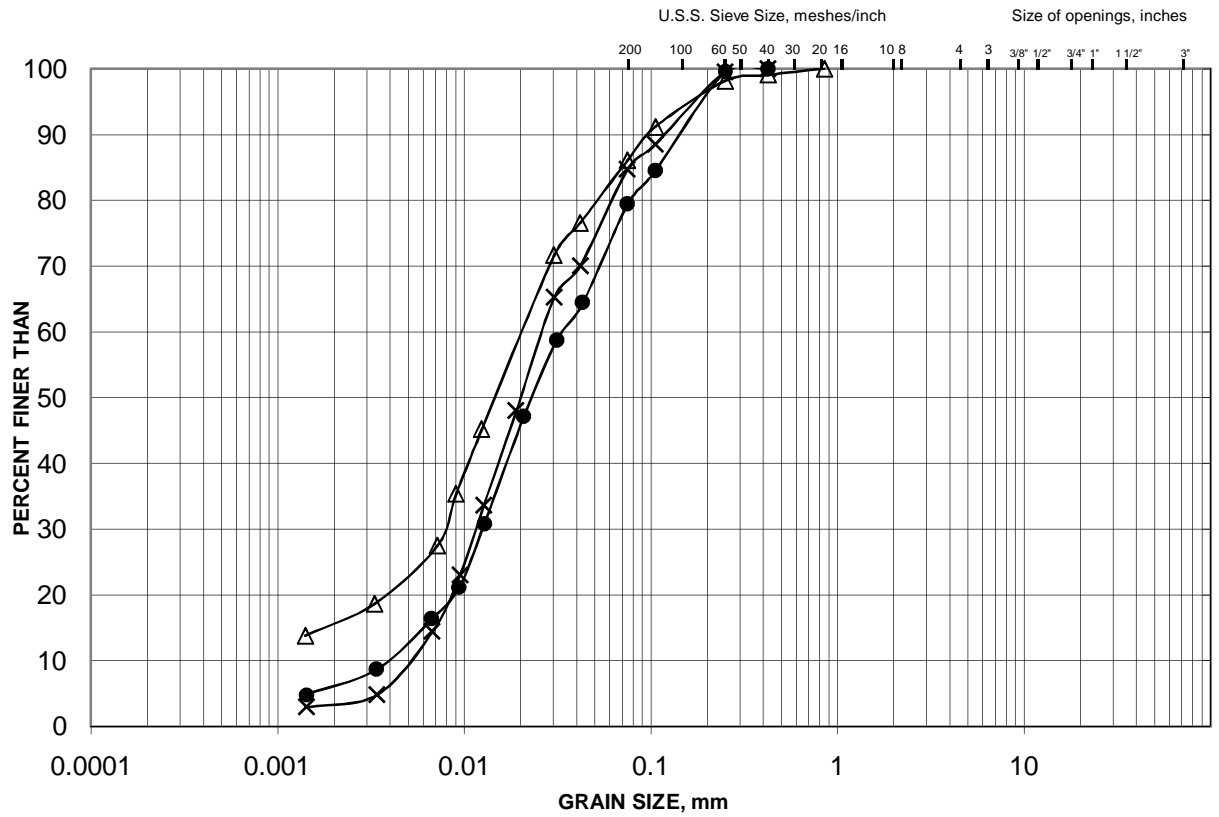
Golder Associates

Date: April 2009

GRAIN SIZE DISTRIBUTION

Sandy Silt to Silt

FIGURE
A-3b



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

LEGEND

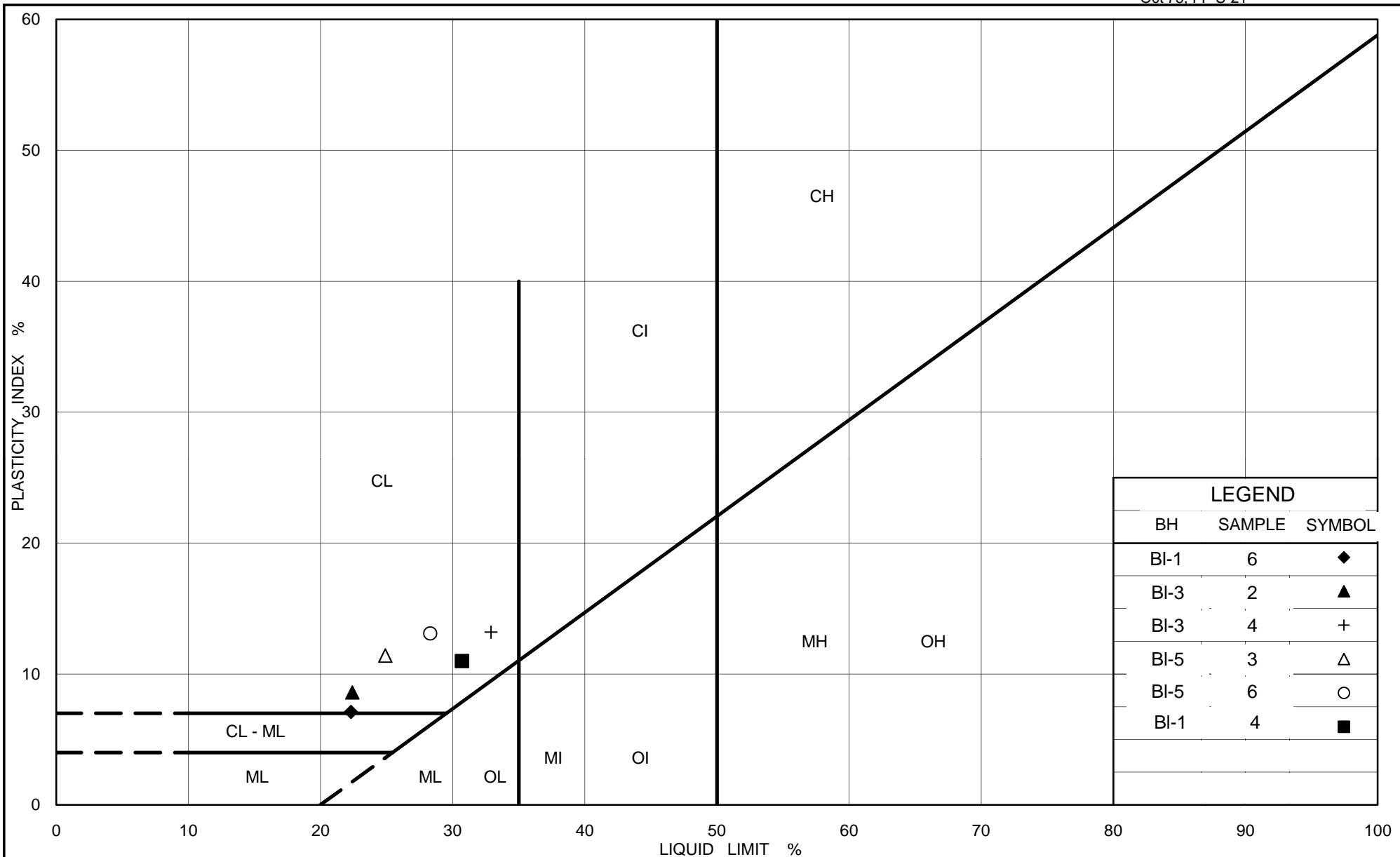
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	BI-6	5	306.5
×	BI-7	5a	306.8
△	BI-11	5	306.3

Project Number: 07-1191-0008

Checked By: AB

Golder Associates

Date: April 2009



Ministry of Transportation

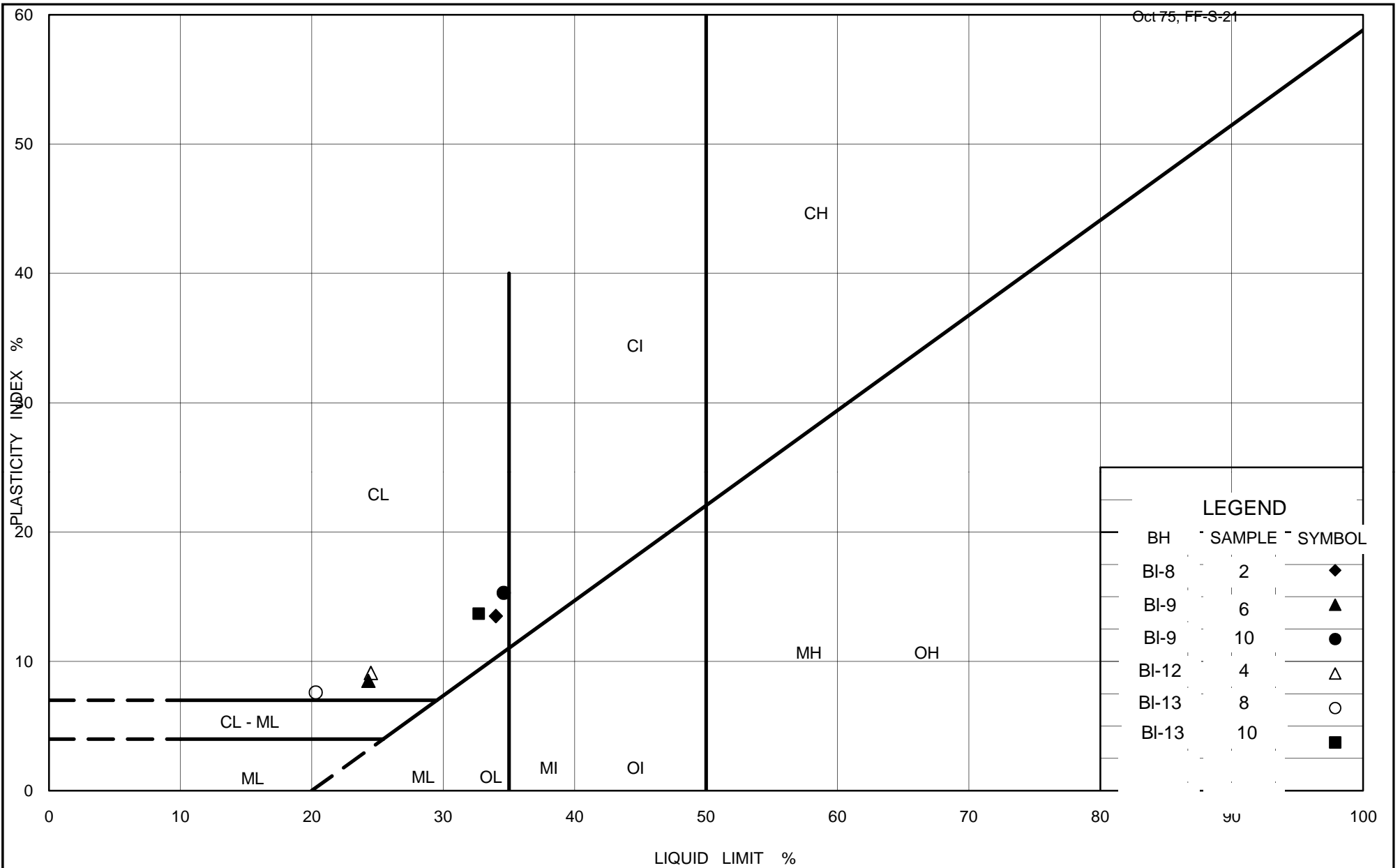
Ontario

PLASTICITY CHART Clayey Silt

Figure A-4a

Project No. 07-1191-0008

Checked By: AB



Ministry of Transportation

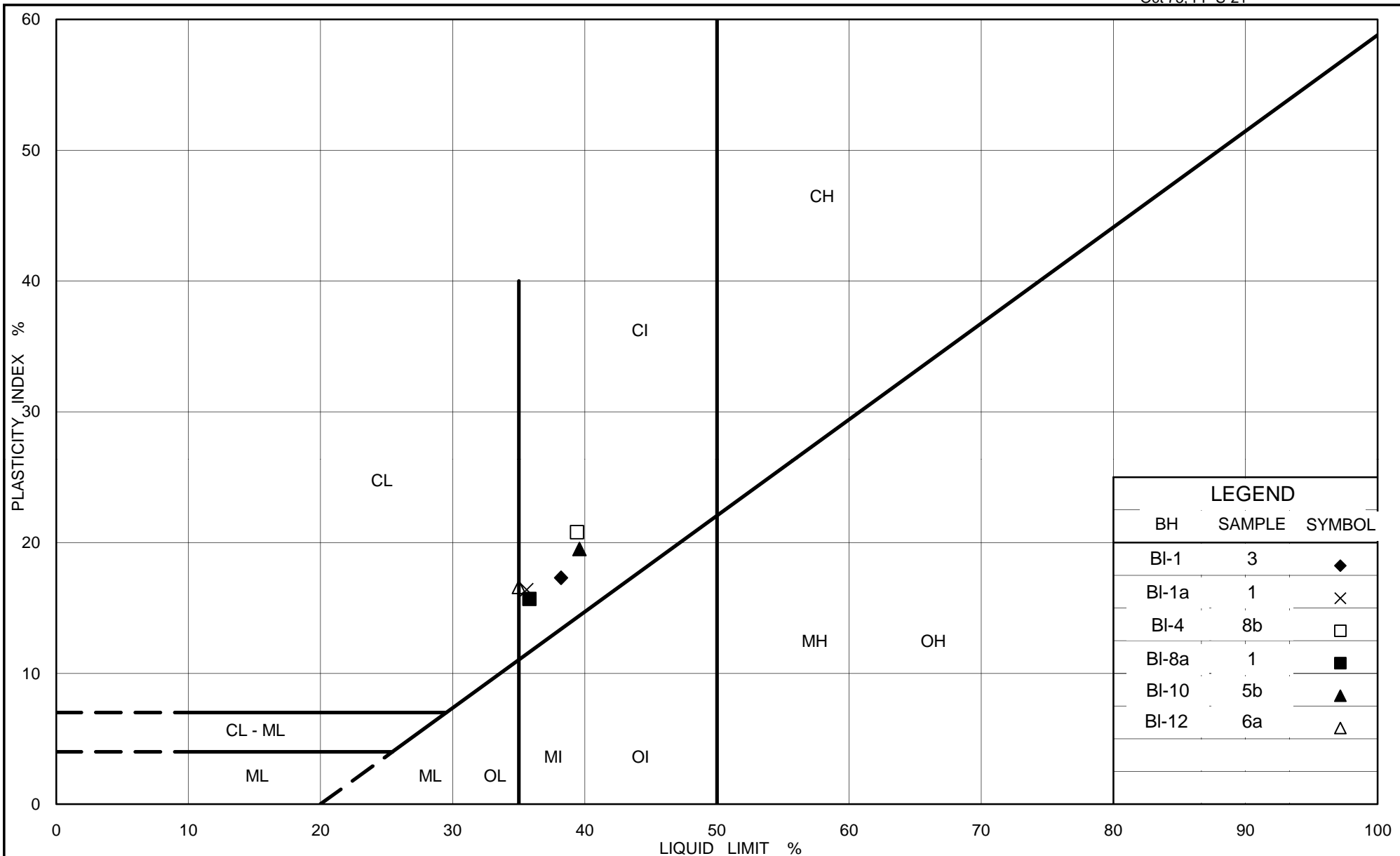
Ontario

PLASTICITY CHART Clayey Silt

Figure A-4b

Project No. 07-1191-0008

Checked By: AB



Ministry of Transportation

Ontario

PLASTICITY CHART Silty Clay

Figure A-5

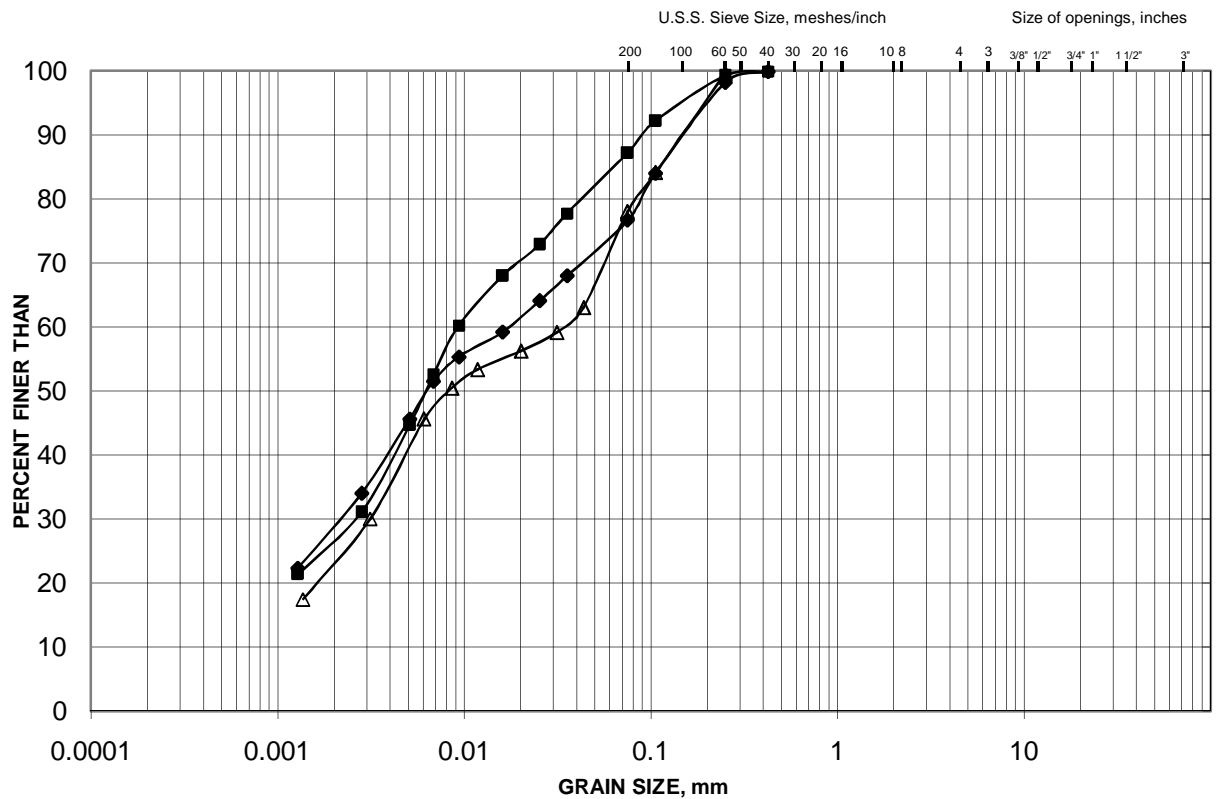
Project No. 07-1191-0008

Checked By: AB

GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE
A-6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
■	BI-1	6	305.1
◆	BI-3	2	306.6
△	BI-9	7	305.2

Project Number: 07-1191-0008

Checked By: AB

Golder Associates

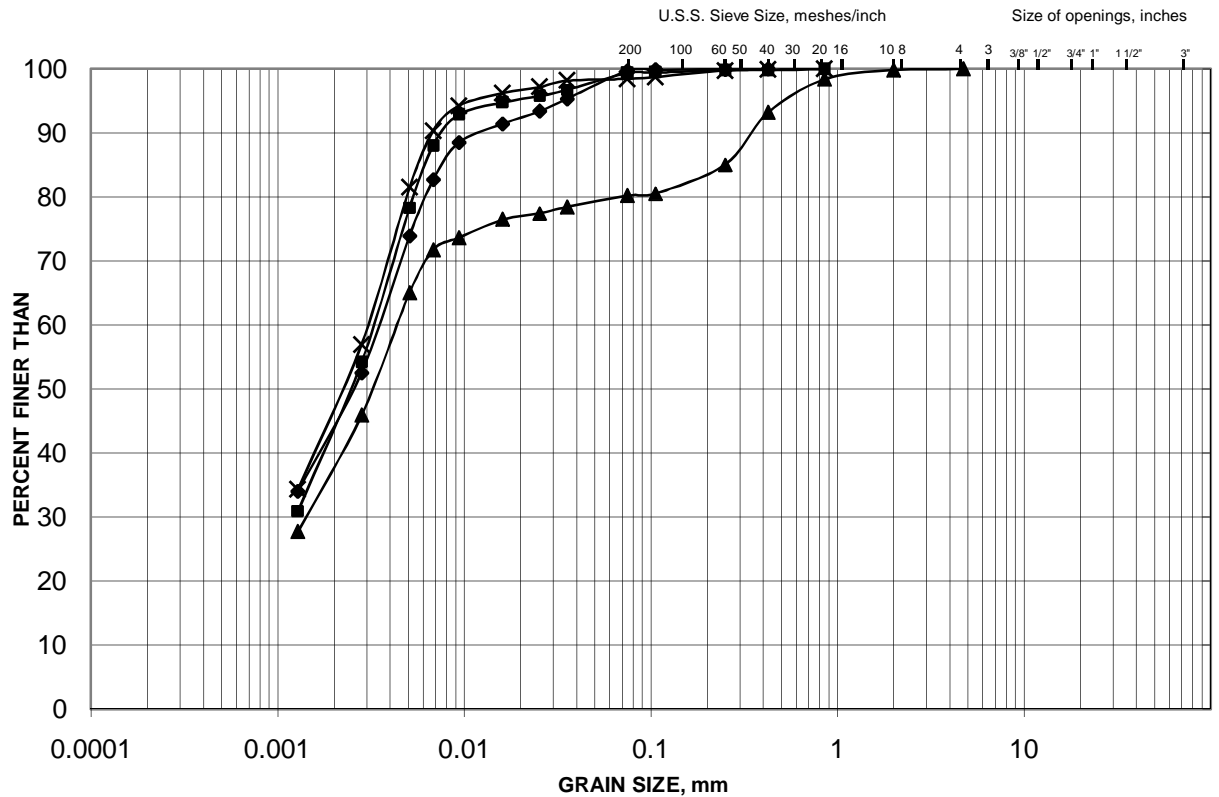
Date: March 2009

GRAIN SIZE DISTRIBUTION

Silty Clay

FIGURE

A-7a



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
■	BI-1	3	308.1
◆	BI-3	4	305.7
▲	BI-4	10	300.8
×	BI-5	4	304.8

Project Number: 07-1191-0008

Checked By: AB

Golder Associates

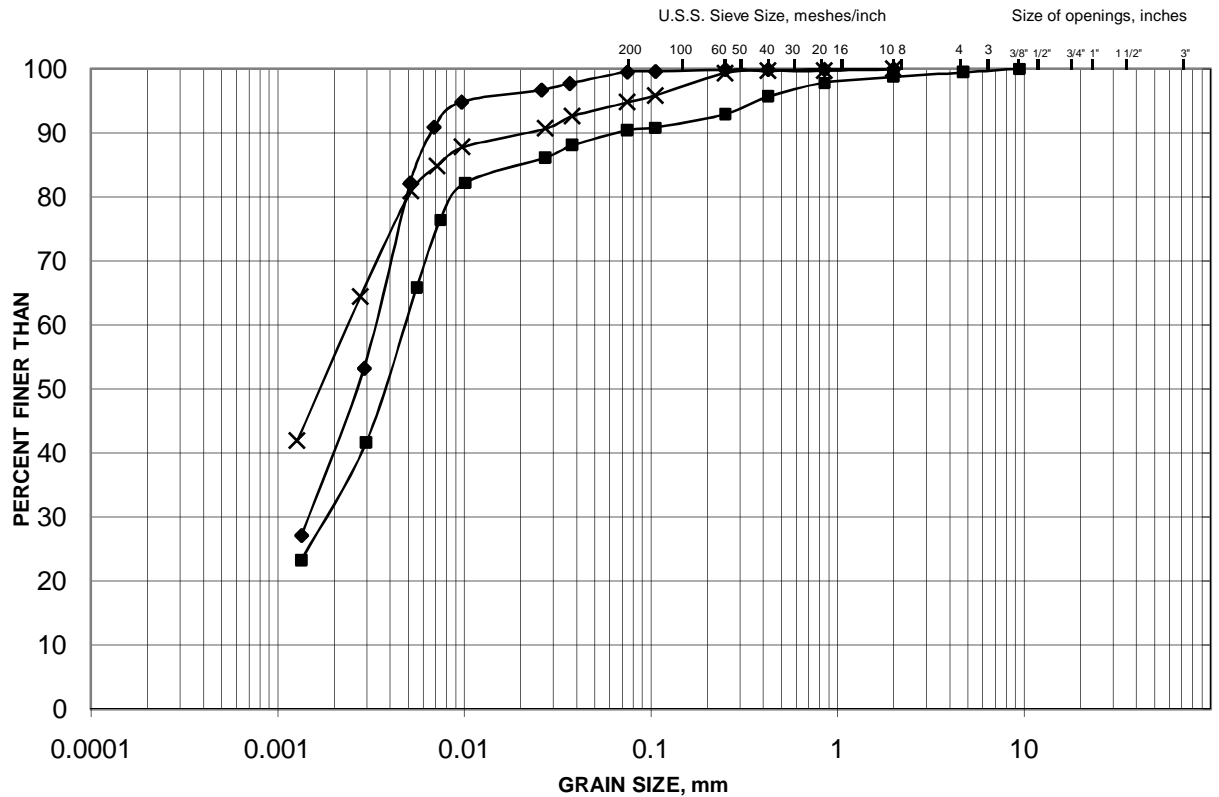
Date: April 2009

GRAIN SIZE DISTRIBUTION

Silty Clay

FIGURE

A-7b



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
■	BI-8	3	305.7
◆	BI-12	5	304.1
×	BI-13	9	304.1

Project Number: 07-1191-0008

Checked By: AB

Golder Associates

Date: April 2009

OEDOMETER CONSOLIDATION SUMMARY

Figure A-8

Page 1 of 4

SAMPLE IDENTIFICATION

Project Number	07-1191-0008	Sample Number	1
Borehole Number	B1-1a	Sample Depth, m	3.3

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	09/23/2008		
Date Completed	10/05/2008		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.22	Unit Weight, kN/m ³	19.14
Sample Diameter, cm	5.00	Dry Unit Weight, kN/m ³	14.59
Area, cm ²	19.60	Specific Gravity, measured	2.75
Volume, cm ³	23.83	Solids Height, cm	0.658
Water Content, %	31.13	Volume of Solids, cm ³	12.89
Wet Mass, g	46.50	Volume of Voids, cm ³	10.93
Dry Mass, g	35.46	Degree of Saturation, %	101.0

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	1.216	0.848	1.216				
5.00	1.199	0.822	1.208	60	5.15E-03	2.76E-03	1.40E-06
10.00	1.190	0.808	1.195	26	1.16E-02	1.51E-03	1.73E-06
20.00	1.170	0.777	1.180	19	1.55E-02	1.68E-03	2.55E-06
40.00	1.164	0.769	1.167	37	7.80E-03	2.30E-04	1.76E-07
80.00	1.139	0.731	1.152	21	1.34E-02	5.14E-04	6.74E-07
160.02	1.093	0.661	1.116	22	1.20E-02	4.73E-04	5.56E-07
319.95	1.057	0.606	1.075	17	1.44E-02	1.85E-04	2.61E-07
640.00	1.024	0.556	1.041	15	1.53E-02	8.48E-05	1.27E-07
1280.00	0.992	0.508	1.008	9	2.39E-02	4.11E-05	9.64E-08
2560.00	0.958	0.456	0.975	26	7.75E-03	2.18E-05	1.66E-08
1280.00	0.966	0.468	0.962				
319.95	0.981	0.491	0.974				
80.00	0.998	0.517	0.990				
20.00	1.014	0.541	1.006				
5.00	1.025	0.558	1.020				

Note:
k calculated using cv based on t₉₀ values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

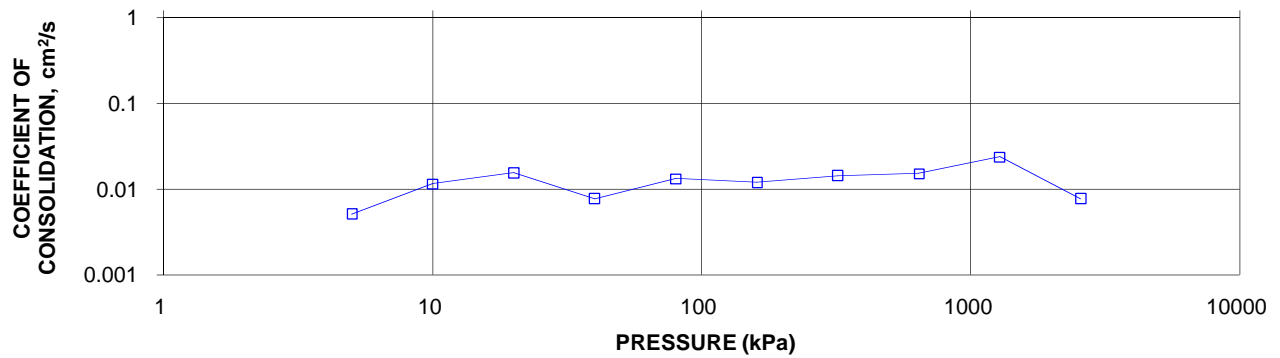
Sample Height, cm	1.03	Unit Weight, kN/m ³	21.60
Sample Diameter, cm	5.00	Dry Unit Weight, kN/m ³	17.31
Area, cm ²	19.60	Specific Gravity, measured	2.75
Volume, cm ³	20.09	Solids Height, cm	0.658
Water Content, %	24.79	Volume of Solids, cm ³	12.89
Wet Mass, g	44.25	Volume of Voids, cm ³	7.19
Dry Mass, g	35.46		

OEDOMETER CONSOLIDATION SUMMARY

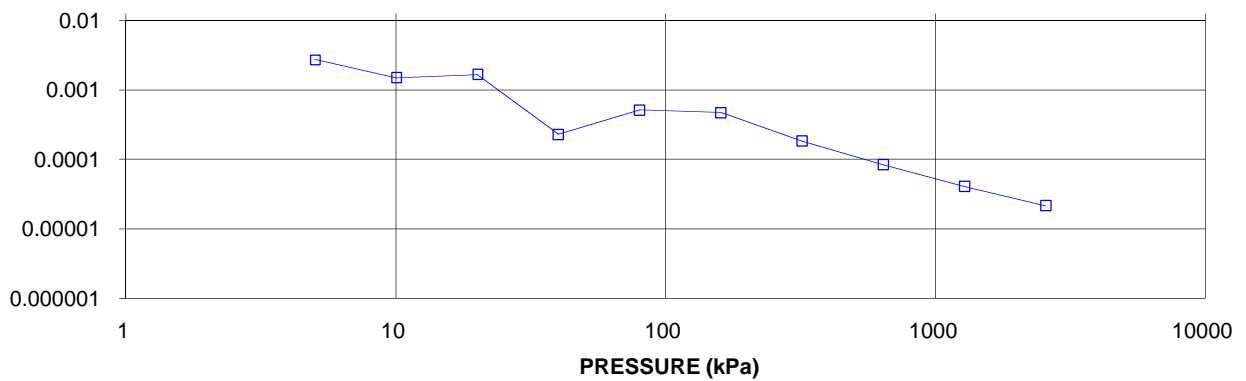
Figure A-8

Page 2 of 4

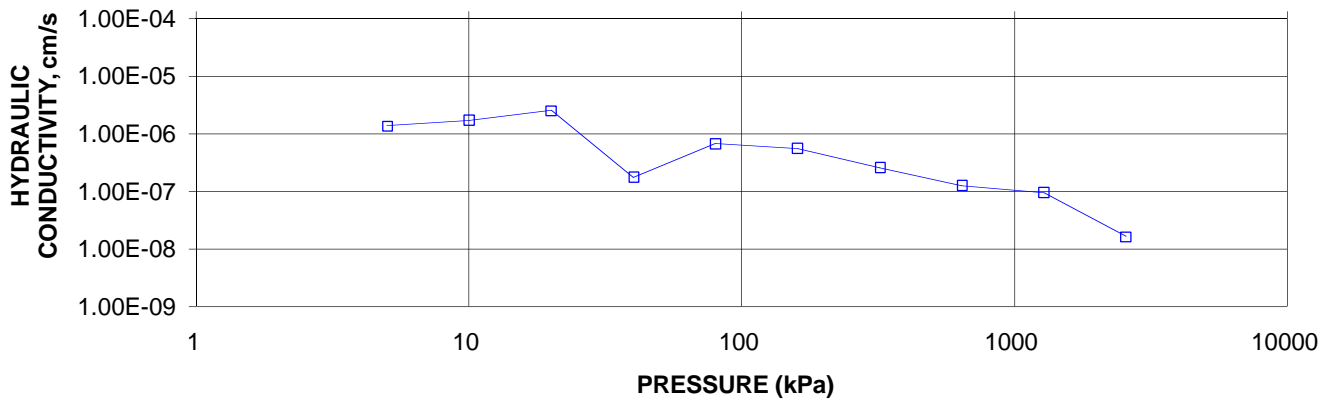
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH BI-1a

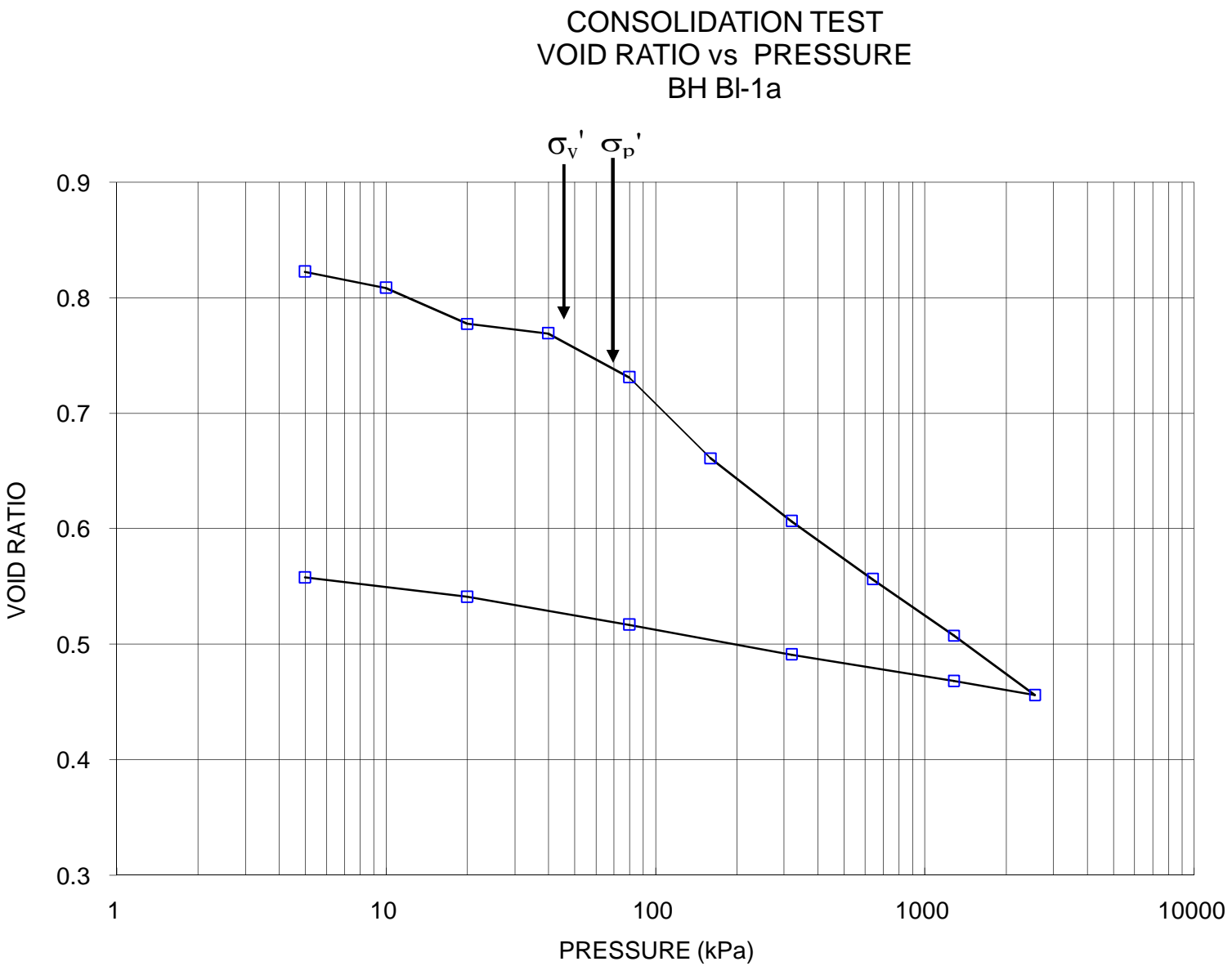


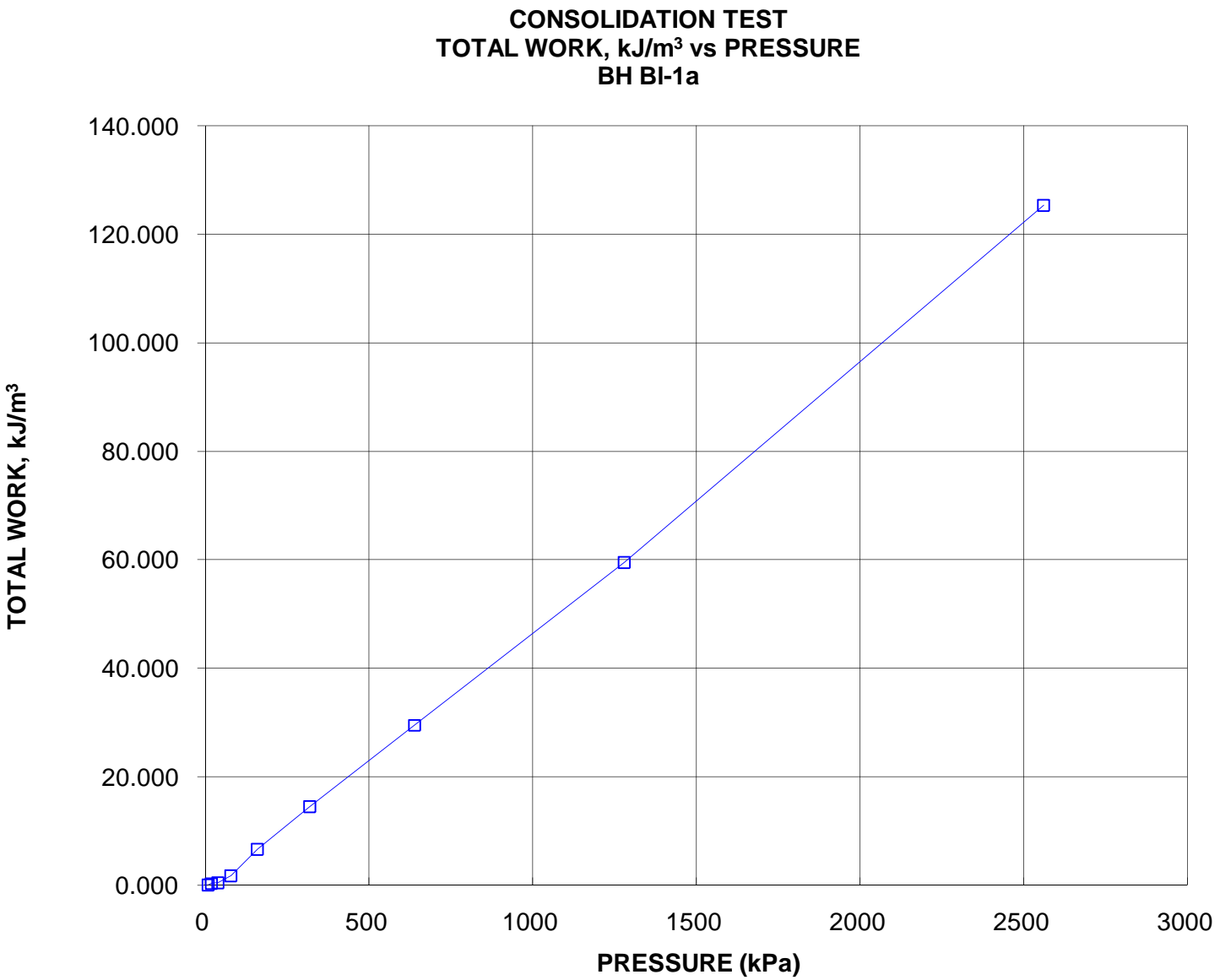
CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH BI-1a



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH BI-1a







OEDOMETER CONSOLIDATION SUMMARY

FIGURE A-9

Page 1 of 4

SAMPLE IDENTIFICATION

Project Number	07-1191-0008	Sample Number	1
Borehole Number	BI-8a	Sample Depth, m	1.6

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	09/11/2008		
Date Completed	09/25/2008		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.22	Unit Weight, kN/m ³	18.32
Sample Diameter, cm	5.00	Dry Unit Weight, kN/m ³	13.31
Area, cm ²	19.60	Specific Gravity, measured	2.76
Volume, cm ³	23.83	Solids Height, cm	0.598
Water Content, %	37.63	Volume of Solids, cm ³	11.72
Wet Mass, g	44.51	Volume of Voids, cm ³	12.11
Dry Mass, g	32.34	Degree of Saturation, %	100.5

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	1.216	1.034	1.216				
7.87	1.175	0.965	1.196	69	4.39E-03	4.28E-03	1.84E-06
15.35	1.161	0.942	1.168	60	4.82E-03	1.54E-03	7.27E-07
20.00	1.152	0.927	1.157	41	6.92E-03	1.59E-03	1.08E-06
39.94	1.122	0.876	1.137	89	3.08E-03	1.24E-03	3.73E-07
80.00	1.085	0.815	1.104	175	1.48E-03	7.60E-04	1.10E-07
160.00	1.052	0.759	1.069	76	3.18E-03	3.39E-04	1.06E-07
320.00	1.021	0.707	1.037	38	5.99E-03	1.59E-04	9.36E-08
640.00	0.989	0.654	1.005	41	5.22E-03	8.22E-05	4.21E-08
1280.00	0.959	0.604	0.974	21	9.58E-03	3.85E-05	3.62E-08
2560.00	0.926	0.549	0.943	60	3.14E-03	2.12E-05	6.52E-09
1280.00	0.932	0.559	0.929				
319.99	0.946	0.582	0.939				
80.00	0.960	0.605	0.953				
20.00	0.977	0.634	0.969				
7.84	0.984	0.646	0.981				

Note:
k calculated using cv based on t₉₀ values.

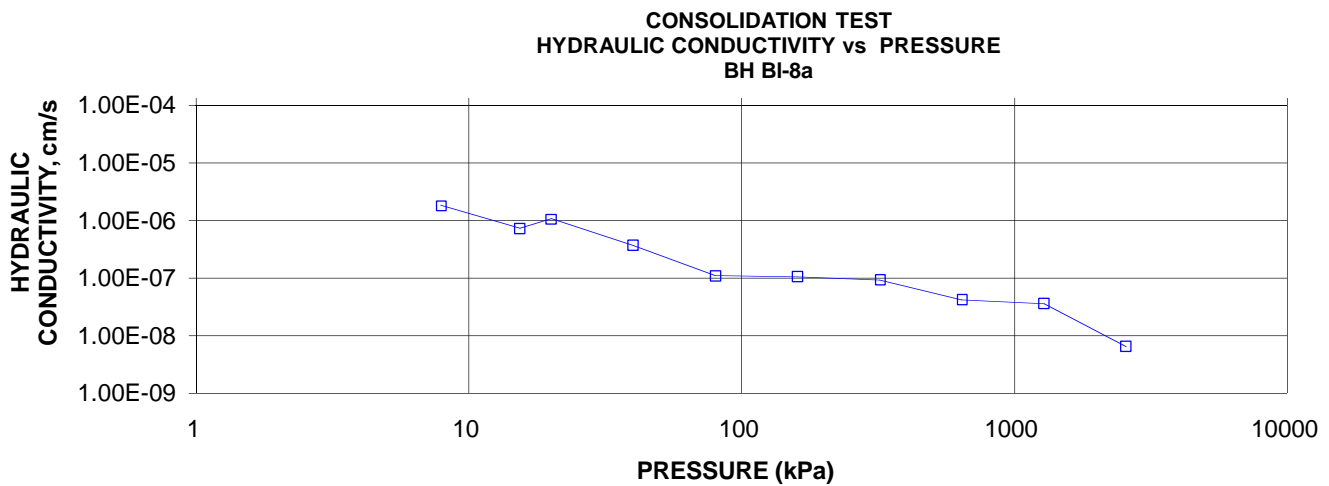
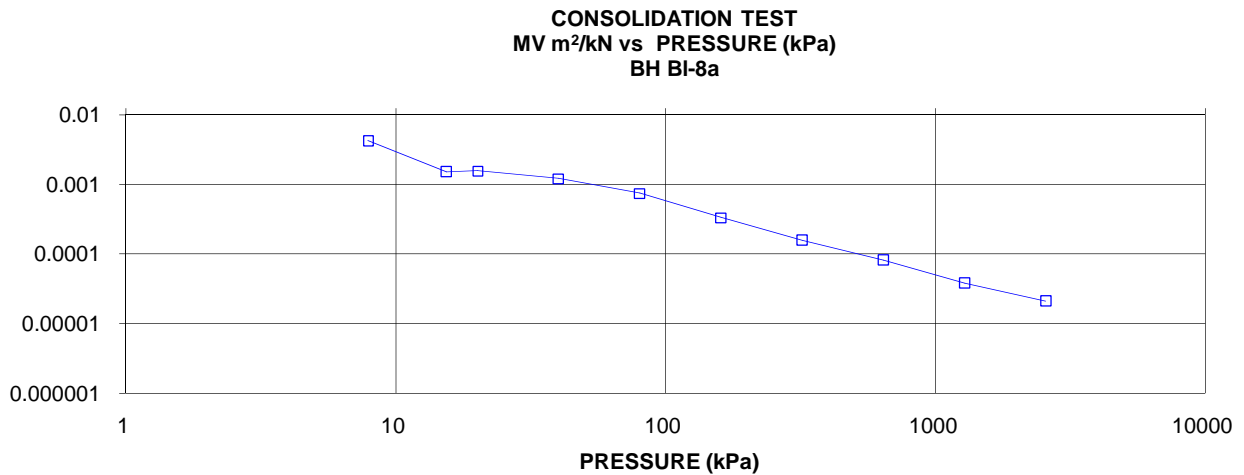
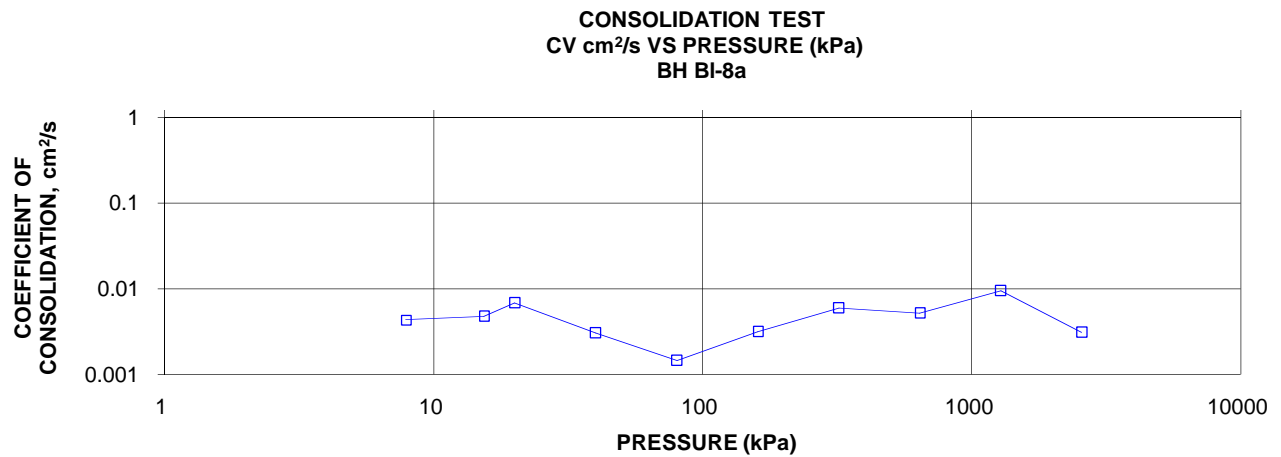
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

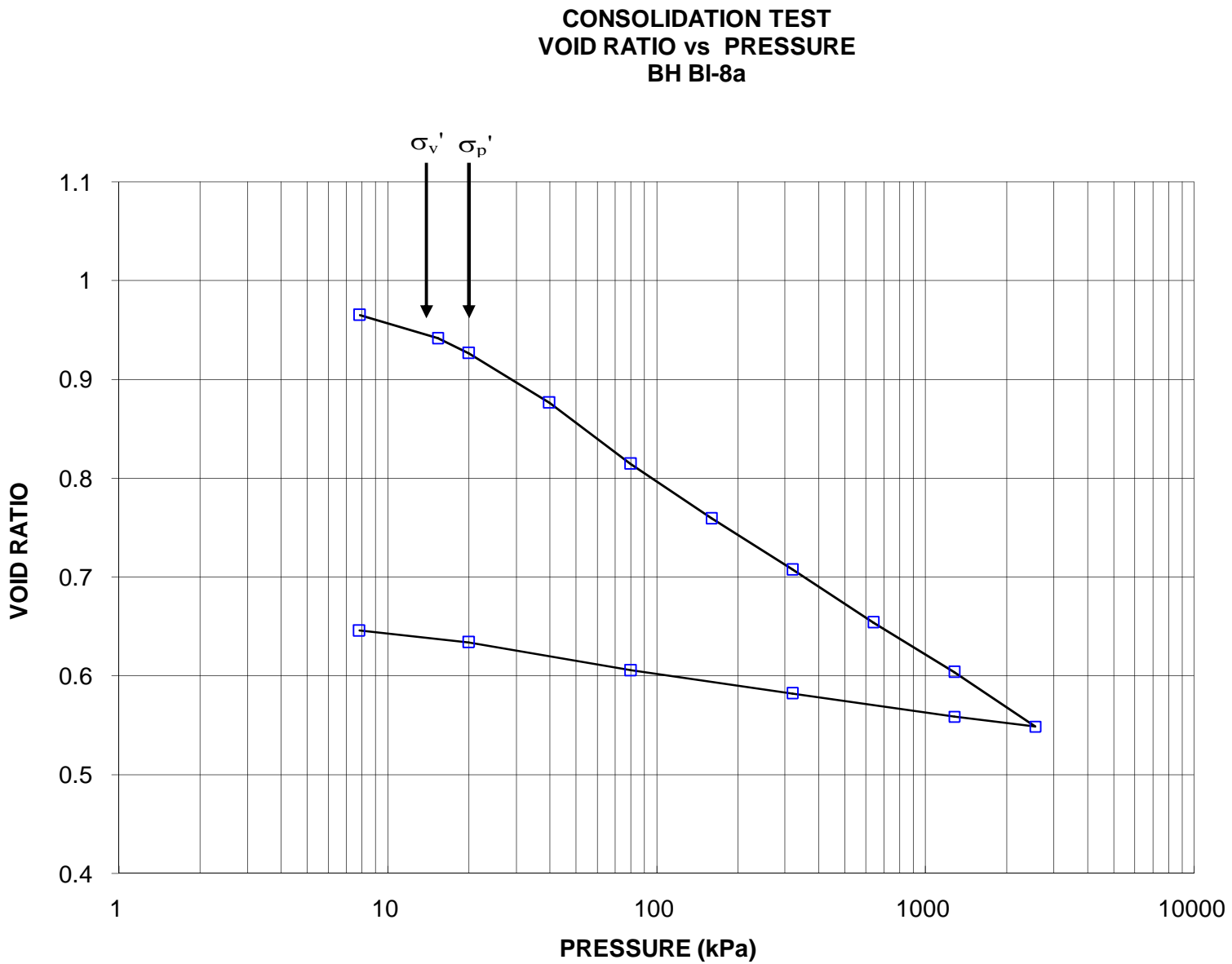
Sample Height, cm	0.98	Unit Weight, kN/m ³	20.67
Sample Diameter, cm	5.00	Dry Unit Weight, kN/m ³	16.45
Area, cm ²	19.60	Specific Gravity, measured	2.76
Volume, cm ³	19.28	Solids Height, cm	0.598
Water Content, %	25.70	Volume of Solids, cm ³	11.72
Wet Mass, g	40.65	Volume of Voids, cm ³	7.56
Dry Mass, g	32.34		

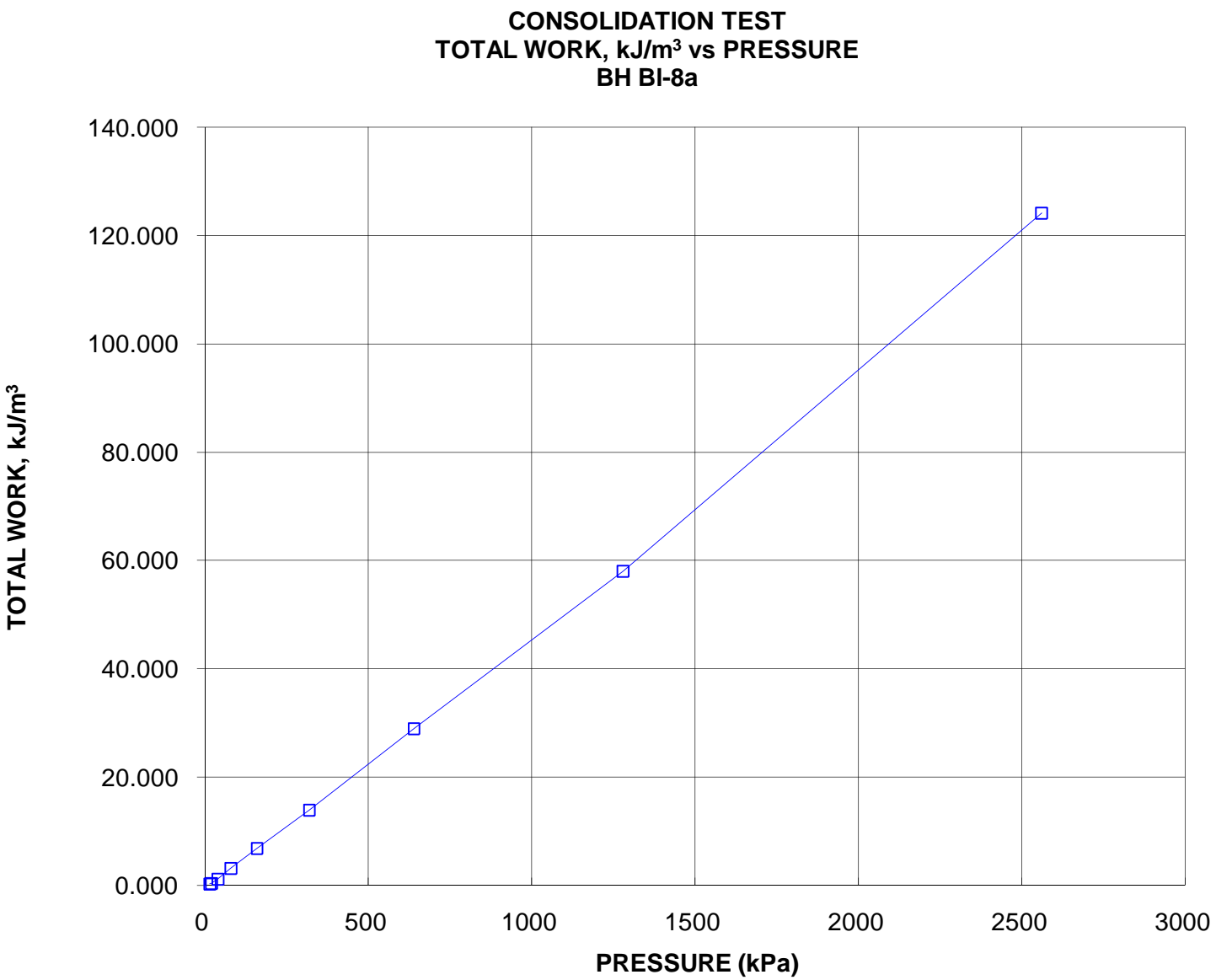
OEDOMETER CONSOLIDATION SUMMARY

FIGURE A-9

Page 2 of 4



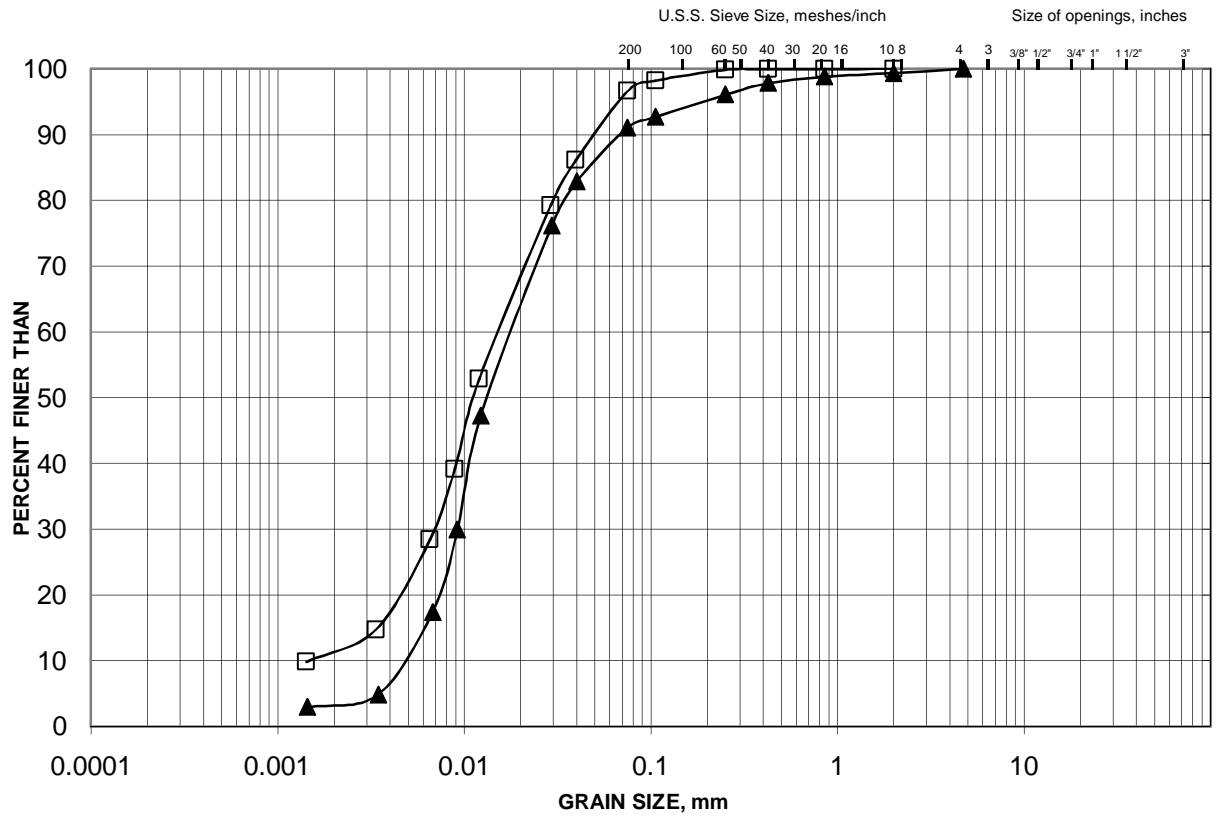




GRAIN SIZE DISTRIBUTION

Sandy Silt to Silt

FIGURE
A-10a



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
▲	BI-10	6	305.8
◻	BI-12	6b	302.3

Project Number: 07-1191-0008

Checked By: AB

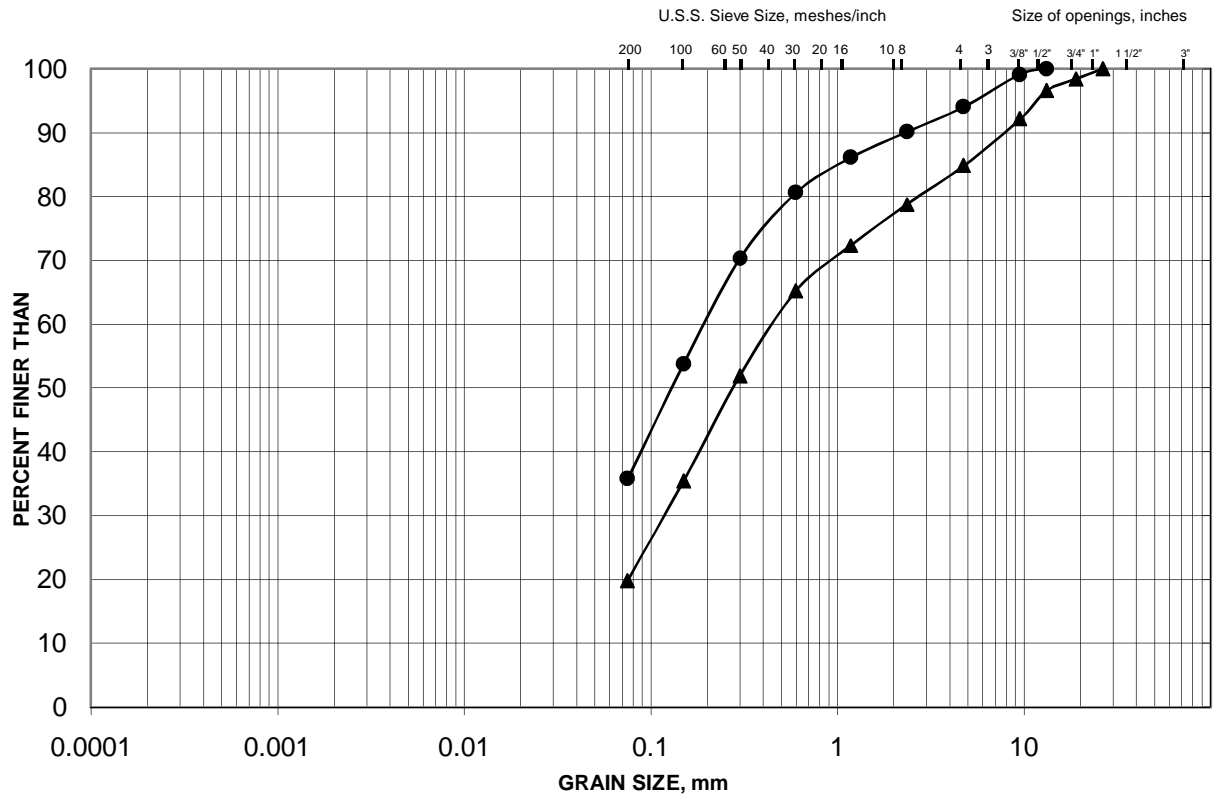
Golder Associates

Date: April 2009

GRAIN SIZE DISTRIBUTION

Sand to Silty Sand

FIGURE
A-10b



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
▲	BI-4	12	297.8
●	BI-5	7	302.5

Project Number: 07-1191-0008

Checked By: AB

Golder Associates

Date: April 2009



APPENDIX B

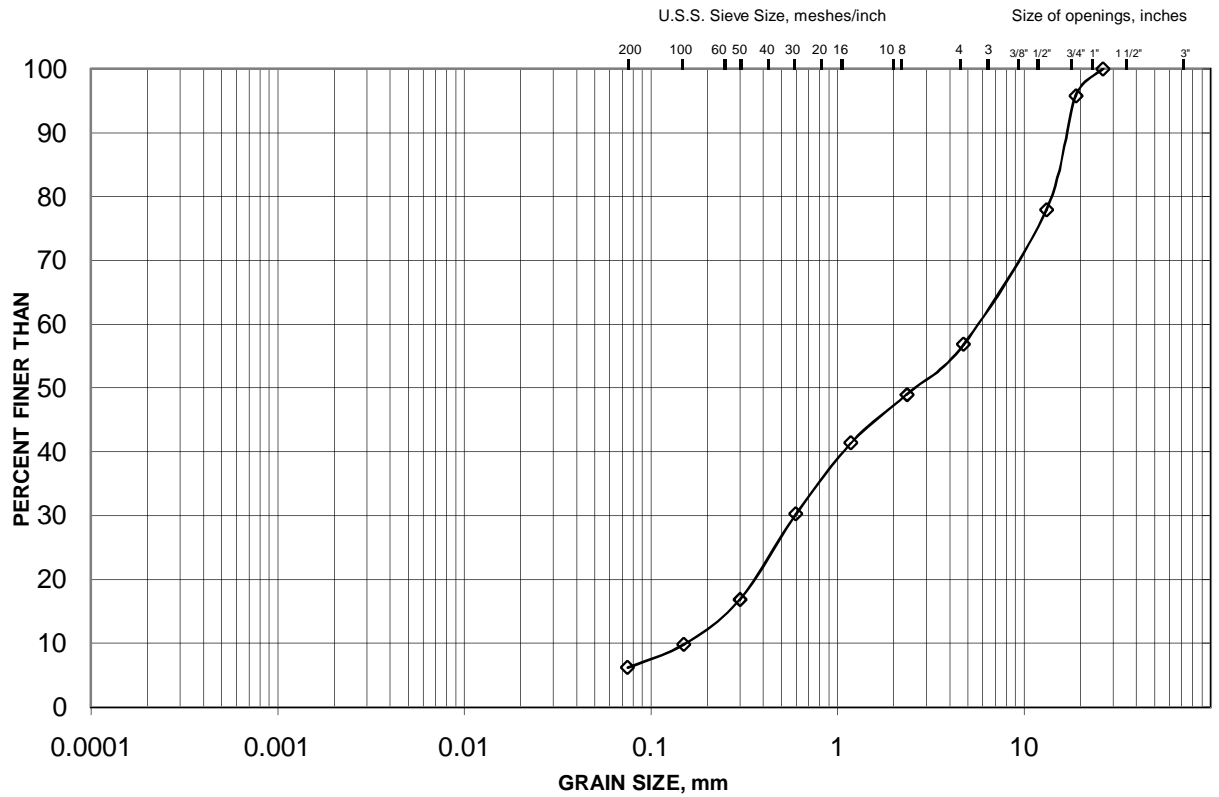
Laboratory Test Results

North Embankment – Station 12+300 to 12+650

GRAIN SIZE DISTRIBUTION

Sand and Gravel (Fill)

**FIGURE
B-1**



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—◇—	BI-16	2	309.5

Project Number: 07-1191-0008

Checked By: AB

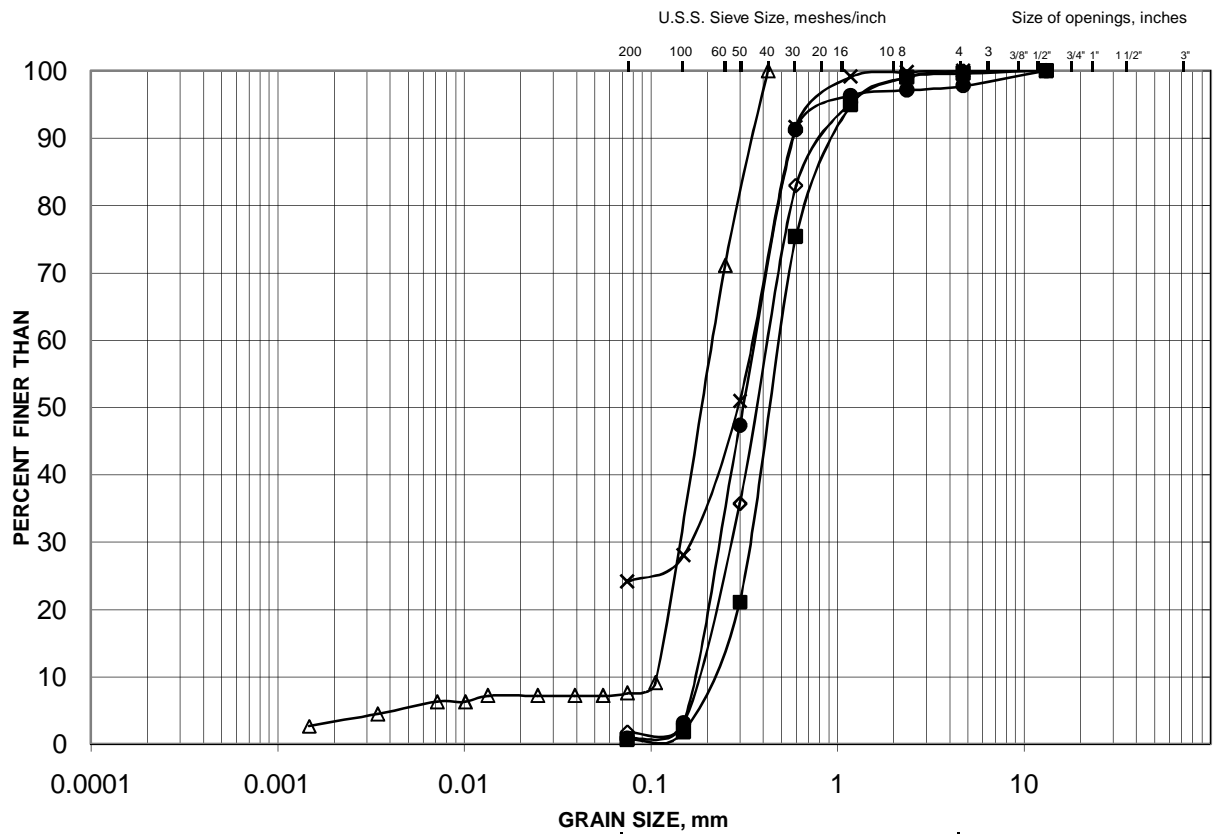
Golder Associates

Date: April 2009

GRAIN SIZE DISTRIBUTION

Sand to Silty Sand

FIGURE
B-2



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

LEGEND

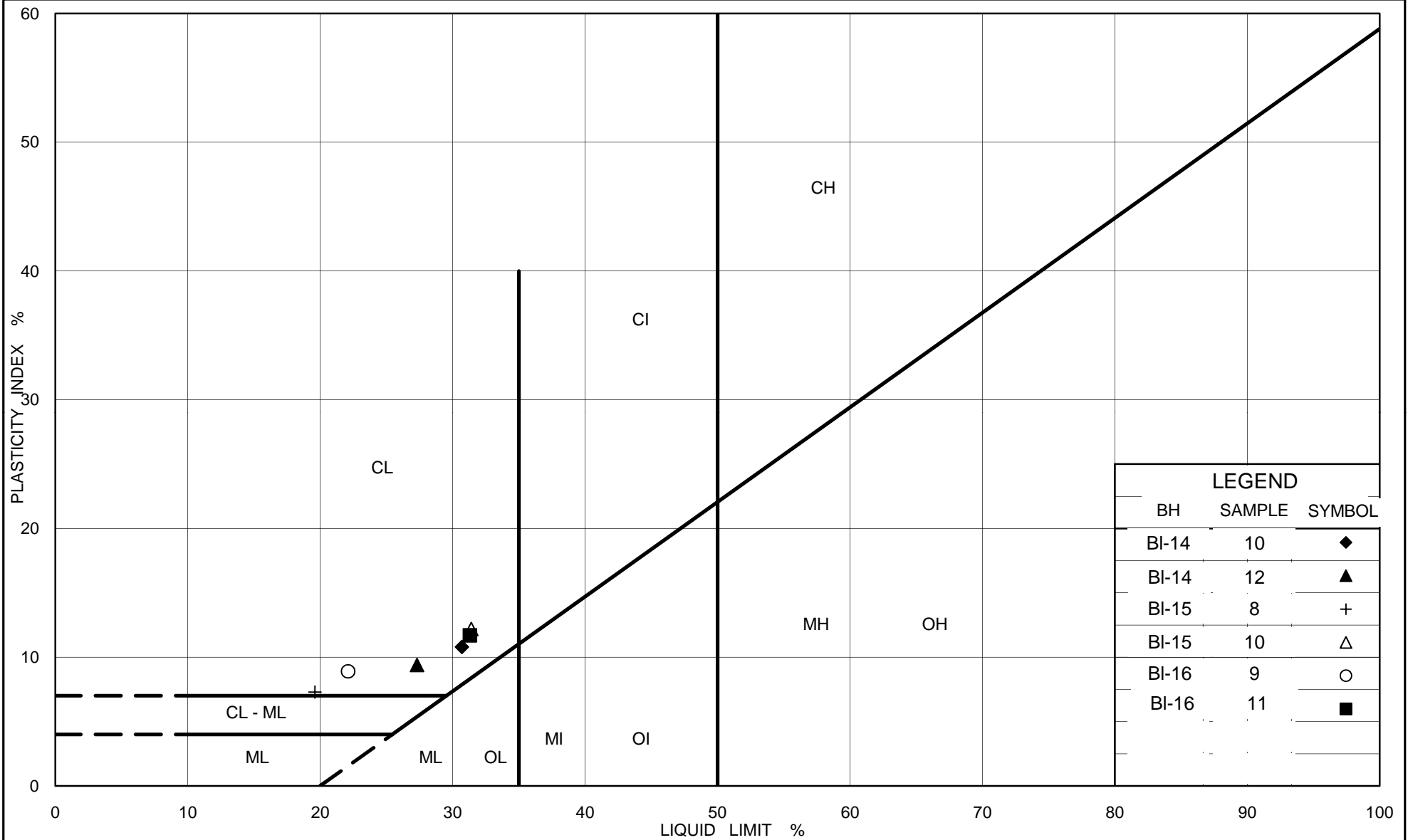
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
◇	BI-14	3	307.7
●	BI-14	6	305.9
■	BI-15	5	306.5
△	BI-17	8	304.6
×	BI-20	4	308.1

Project Number: 07-1191-0008

Checked By: AB

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Date: April 2009



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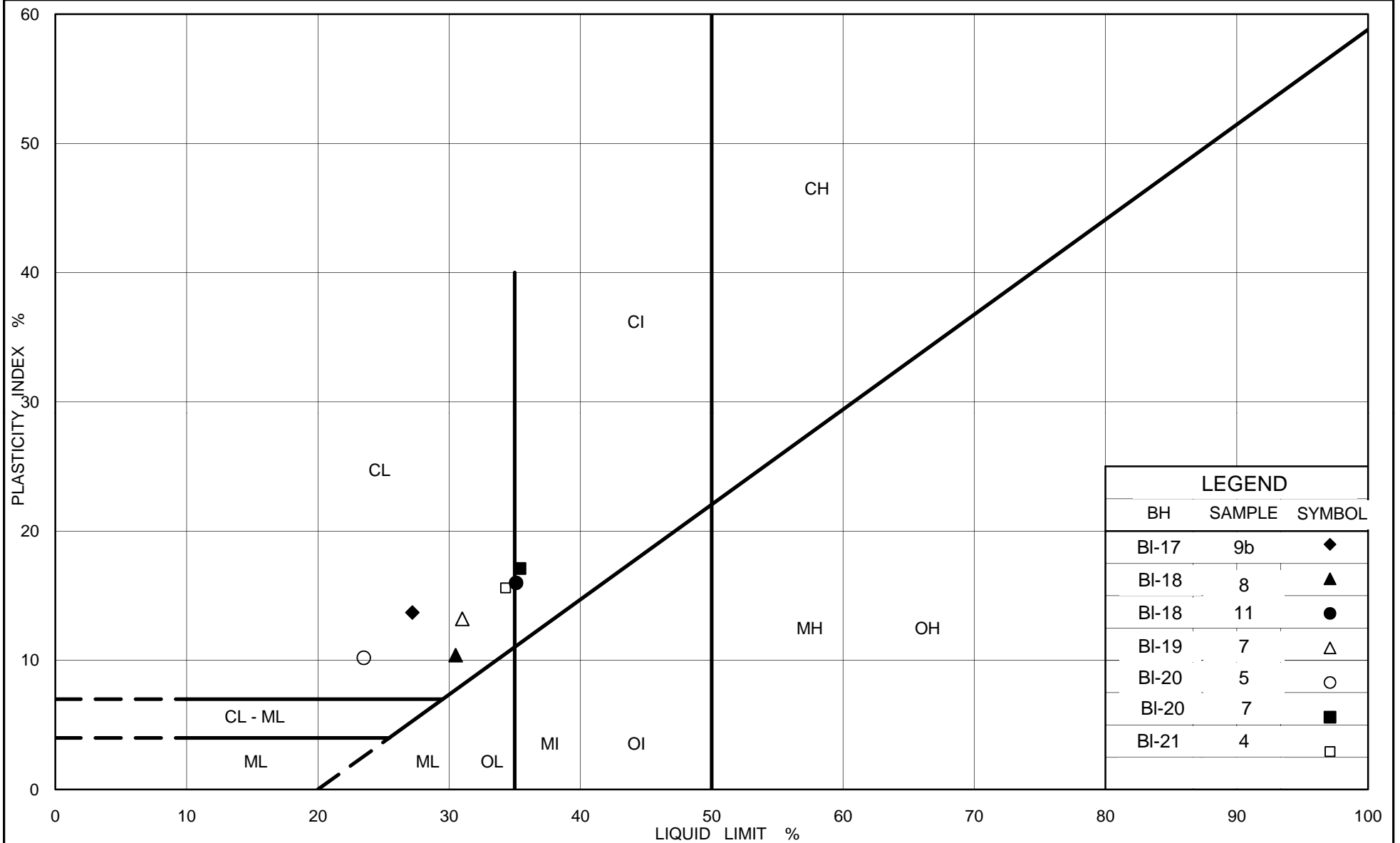
PLASTICITY CHART

Clayey Silt

Figure B-3a

Project No. 07-1191-0008

Checked By: AB



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt to Silty Clay

Figure B-3b

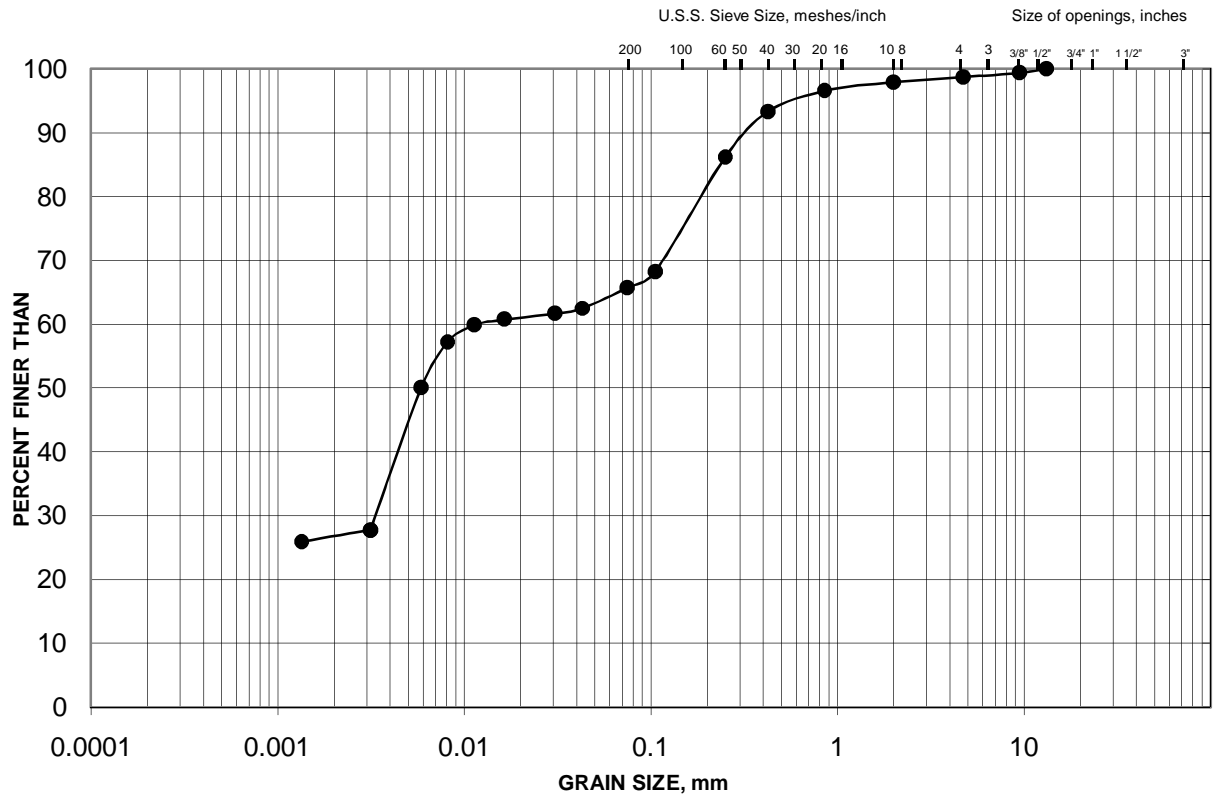
Project No. 07-1191-0008

Checked By: AB

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand

**FIGURE
B-4**



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—●—	BI-21	3	309.1

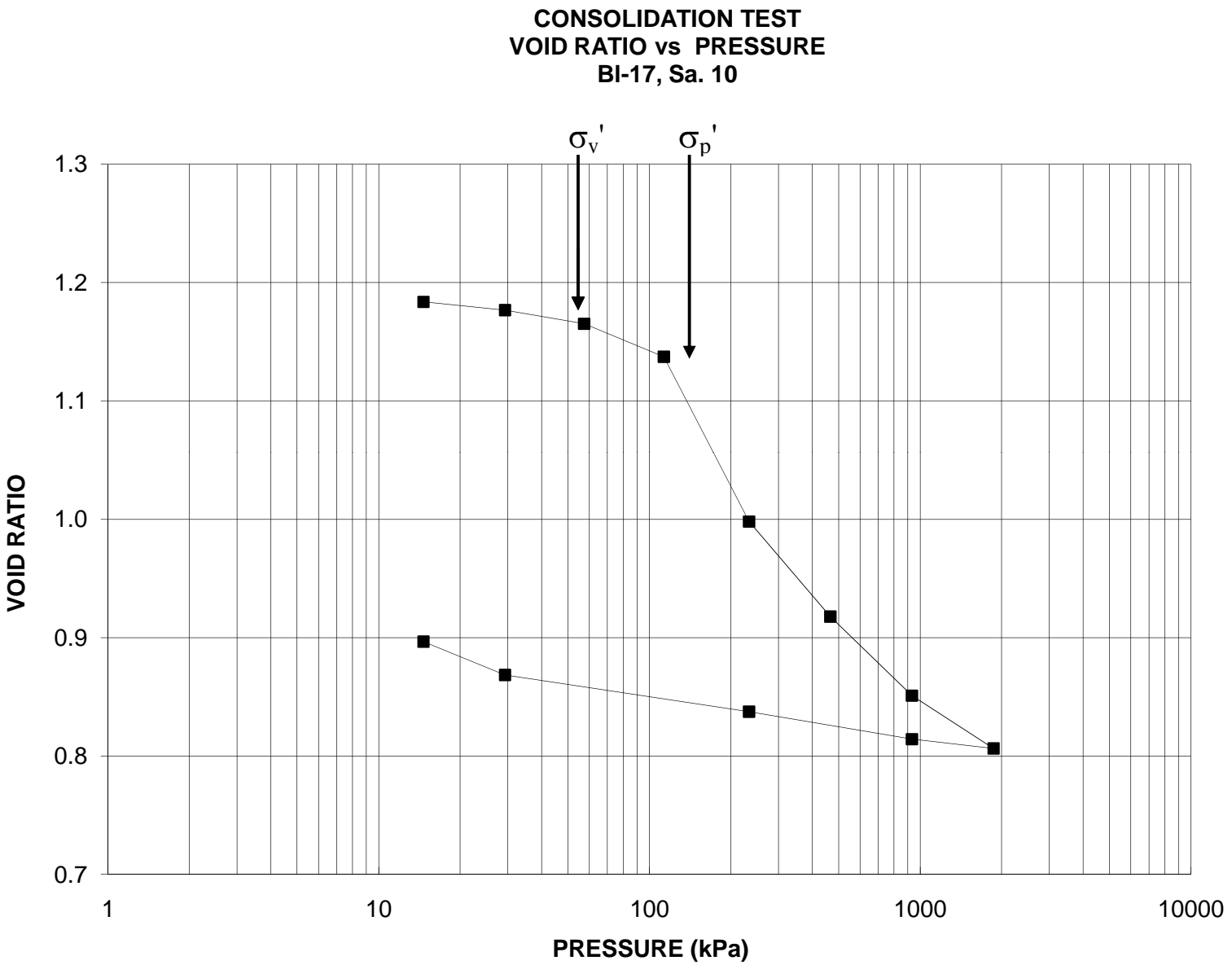
Project Number: 07-1191-0008

Checked By: AB

Golder Associates

Date: April 2009

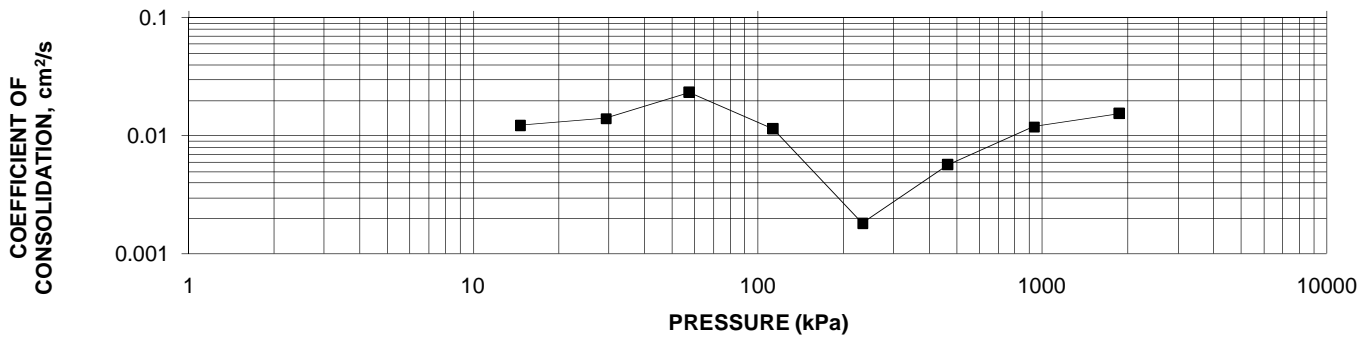
OEDOMETER CONSOLIDATION SUMMARY					FIGURE B-5 Page 1 of 4			
SAMPLE IDENTIFICATION								
Project Number		07-1191-0008		Sample Number		10		
Borehole Number		BI-17		Sample Depth, (m)		7.6		
TEST CONDITIONS								
Test Type		Standard		Load Duration, hr		24		
Oedometer Number		1						
Date Started		30-Oct-08						
Date Completed		11-Nov-08						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL								
Sample Height, cm		1.906		Unit Weight, kN/m ³		17.5		
Sample Diameter, cm		5.000		Dry Unit Weight, kN/m ³		12.1		
Area, cm ²		19.63		Specific Gravity, assumed		2.7		
Volume, cm ³		37.42		Solids Height, cm		0.870		
Water Content, %		44.9		Volume of Solids, cm ³		17.09		
Wet Mass, g		66.85		Volume of Voids, cm ³		20.34		
Dry Mass, g		46.13		Degree of Saturation, %		101.9		
TEST COMPUTATIONS								
Pressure	Primary Consolidation	Corr. Height	Void Ratio	Average Height	t ₅₀	cv.	m _v	k
kPa	mm	cm		cm	s	cm ² /s	m ² /MN	cm/s
0	0.00	1.906	1.190	1.906				
14.6	0.06	1.900	1.184	1.903	58	0.01224	0.208	2.501E-07
29.2	0.06	1.894	1.177	1.897	50	0.01411	0.223	3.089E-07
57.4	0.10	1.884	1.165	1.889	30	0.02331	0.186	4.247E-07
113.1	0.24	1.860	1.137	1.872	60	0.01145	0.232	2.608E-07
233.3	1.21	1.739	0.998	1.799	350	0.00181	0.541	9.629E-08
466.1	0.70	1.669	0.918	1.704	100	0.00569	0.173	9.653E-08
933.2	0.58	1.611	0.851	1.640	44	0.01198	0.074	8.742E-08
1864.2	0.39	1.572	0.806	1.591	32	0.01551	0.026	3.957E-08
933.2	-0.07	1.579	0.814	1.575				
233.3	-0.20	1.599	0.837	1.589				
29.2	-0.27	1.626	0.868	1.612				
14.6	-0.25	1.650	0.897	1.638				
Notes: k calculated using cv based on t ₅₀ values.								
SAMPLE DIMENSIONS AND PROPERTIES - FINAL								
Sample Height, cm		1.650		Unit Weight, kN/m ³		18.2		
Sample Diameter, cm		5.000		Dry Unit Weight, kN/m ³		14.0		
Area, cm ²		19.63		Specific Gravity, assumed		2.7		
Volume, cm ³		32.40		Solids Height, cm		0.870		
Water Content, %		30.6		Volume of Solids, cm ³		17.09		
Wet Mass, (after test) g		60.251		Volume of Voids, cm ³		15.32		
Dry Mass, g (oven)		46.13		Degree of Saturation, %		92.2		
<div> <div>Prepared By: TG</div> <div>Golder Associates</div> <div>Checked By: AB</div> </div>								



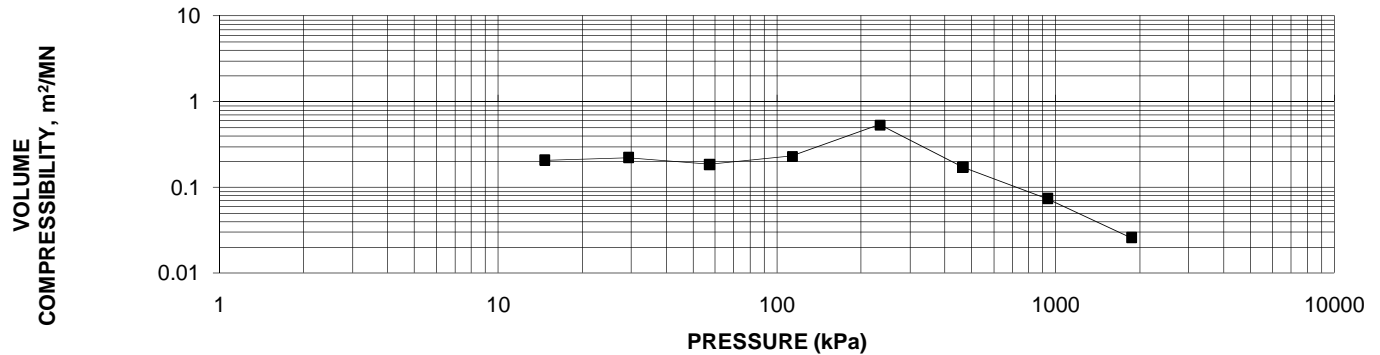
OEDOMETER CONSOLIDATION SUMMARY

FIGURE B-5
Page 3 of 4

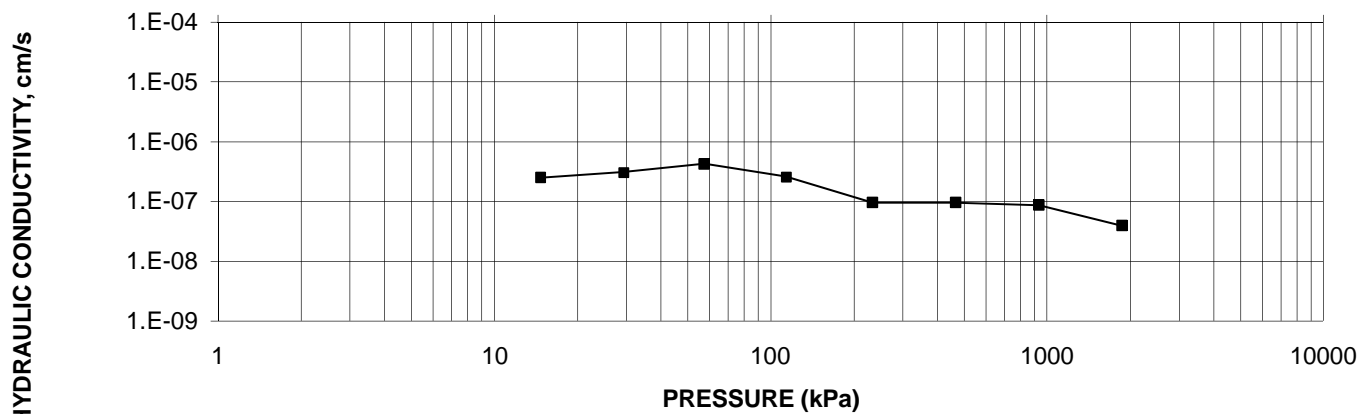
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BI-17, Sa. 10

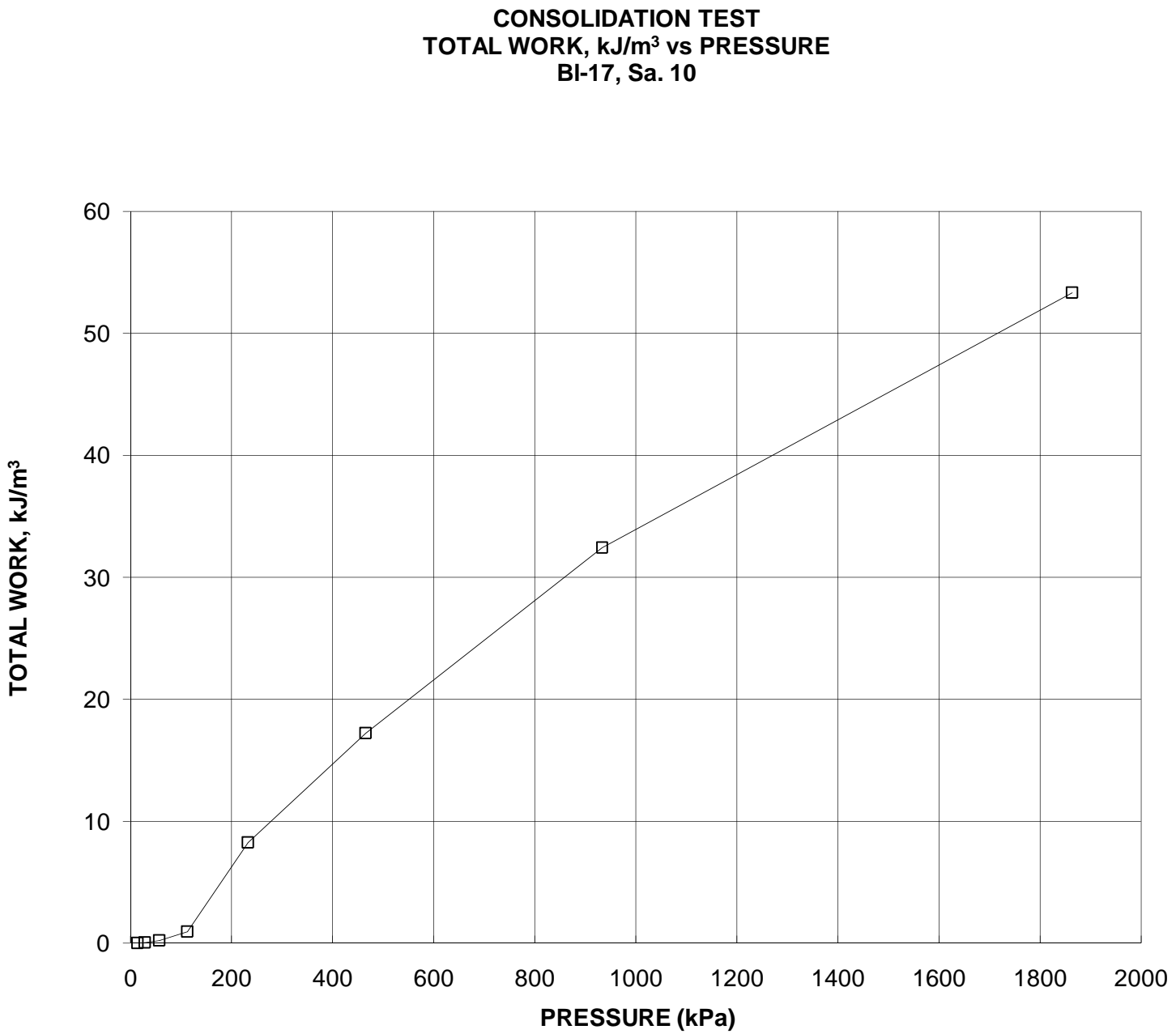


CONSOLIDATION TEST
MV m²/MN vs PRESSURE (kPa)
BI-17, Sa. 10



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BI-17, Sa. 10

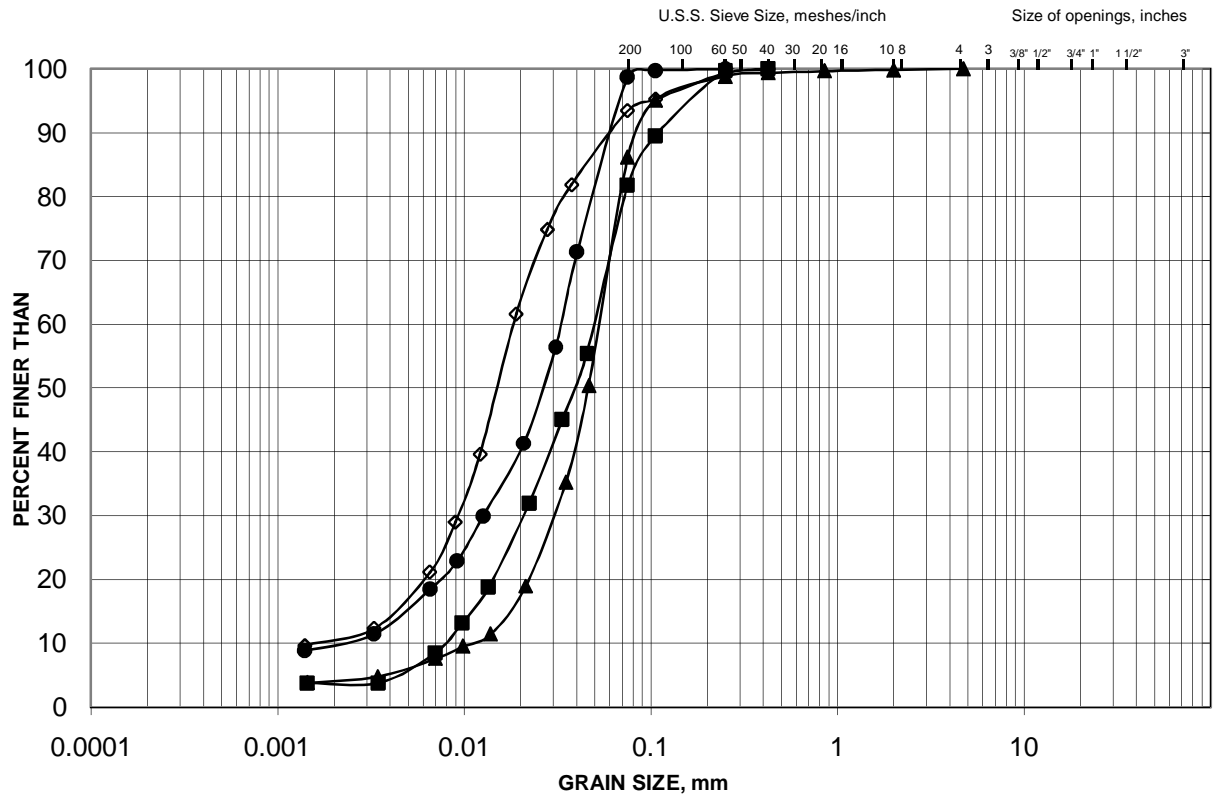




GRAIN SIZE DISTRIBUTION

Silt

**FIGURE
B-6**



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
◇	BI-14	13	296.8
●	BI-19	9	304.1
■	BI-20	9	300.4
▲	BI-20	11	297.5

Project Number: 07-1191-0008

Checked By: AB

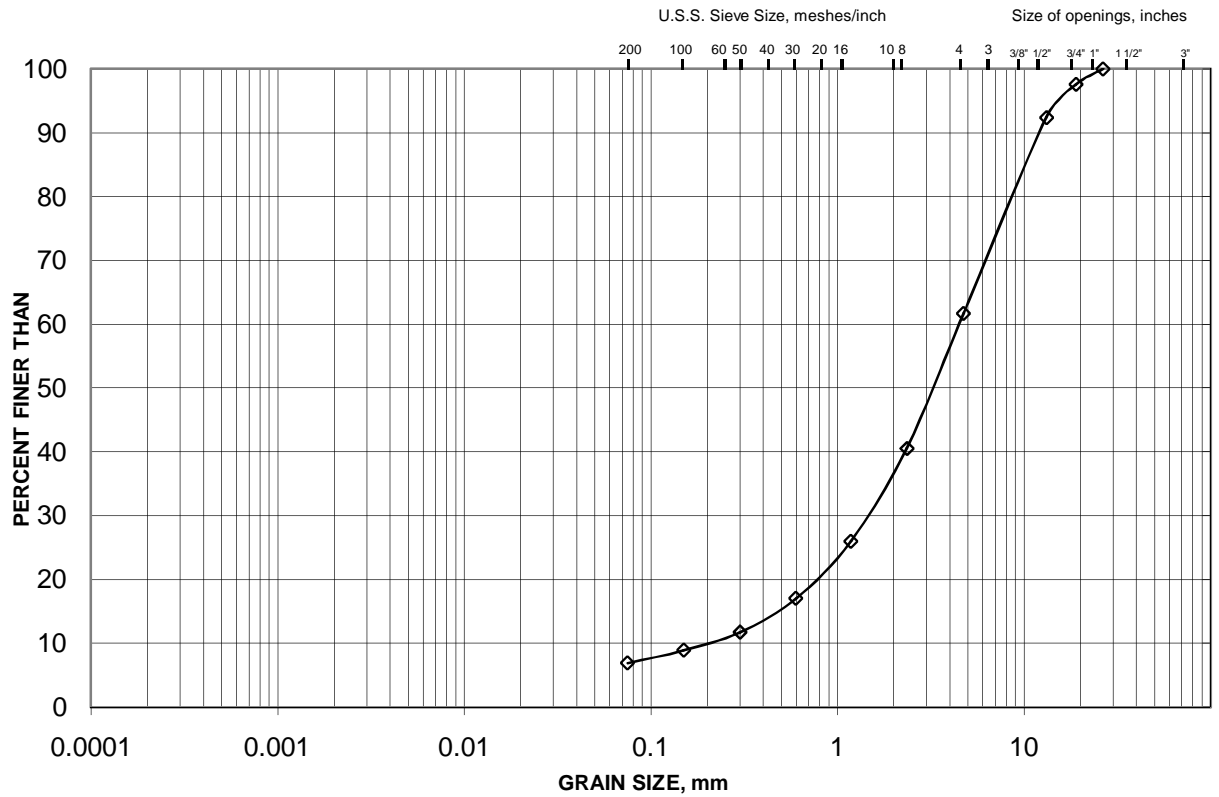
Golder Associates

Date: April 2009

GRAIN SIZE DISTRIBUTION

Sand and Gravel

**FIGURE
B-7**



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—◇—	BI-18	13	295.3

Project Number: 07-1191-0008

Checked By: AB

Golder Associates

Date: April 2009



APPENDIX C

Non-Standard Special Provisions (NSSP)

MASS CONCRETE – Item No.

Special Provision

Scope of Work

The scope of work for the above noted tender item includes the mass concrete under the South abutment footing.

Construction

Concrete shall be of the same strength as the footing concrete and placed in accordance with OPSS 904.

Basis of Payment

Payment at the contract price for the above noted tender item includes full compensation for all labour, equipment and materials to do the required work.

DOWELS Into Rock – Item No.

Special Provision

Scope of Work

Work under this item is for the placement and field testing of dowels into rock.

Construction

Dowels into rock shall be constructed in accordance with OPSS 904. All reinforcing steel supplied shall be in accordance with OPSS 1440 (dowel bars conforming to CSA Standard CSAG30.18, Grade 400).

Where dowels are to be placed in rock, holes shall be drilled to the required depth and size. Hole diameter shall be two times the nominal diameter of the dowel. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete (or at least 25 MPa at 28 days).

If the hole contains water, the contractor shall remove the water otherwise a tremie procedure shall be used to completely fill the hole with grout. The dowel shall be forced into the hole after the grout has been placed and while it is still fresh.

Rock Dowel Testing

All proposed testing procedures shall be in general conformance with ASTM D 3689-90 and ASTM D 114381 (Re-approved 1994). Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

Performance Tests

The following table summarizes the number of rock dowels where performance testing shall be carried out to confirm that the design load of the rock dowels can be achieved. The Contract Administrator will select the rock dowels to be tested.

Bridge	Foundation	Number of Dowels for Performance Testing
Highway 11 over Blanche River	South Abutment	2

Performance test shall be by axial tensioning using a hydraulic jack with a capacity of at least 1.5 times the ultimate strength of the dowels.

Rock dowels shall be loaded and unloaded in 3 cycles and measurements of the displacement of the dowel shall be carried out at each load increment (step) in accordance with the following schedule:

Cycle-Step	1-1	1-2	1-3	2-1	2-2	2-3	2-4
% Design Load	50	75	25	50	75	100	25

DOWELS Into Rock – Item No.

Special Provision

Cycle-Step	3-1	3-2	3-3	3-4	3-5
% Design Load	50	75	100	110	25

The design load shall be taken as 360 kN for 35M dowels, 252 kN for 30M dowels, 180 kN, for 25M dowels, and 108 kN for 20M dowels.

Displacement measurements shall be carried out at each load increment using calibrated displacement gauges capable of measuring movements of 0.0025 cm. Measurements shall be referenced to an independent fixed referenced pint.

Rock dowels which fail to meet the acceptance criteria shall be replaced at the Contractor's expense and re-tested. If a rock dowel fails, 3 additional rock dowels shall be tested at the same abutment and pier footing as directed by the Contract Administrator.

Acceptance criteria for the rock dowels will be in accordance with the Post-tensioning Institute (1985) as follows:

The dowels are acceptable if the total elastic movement is greater than 80% of the theoretical elastic elongation of the free stressing and is less than the theoretical elongation of the free stressing length plus 50% of the bond length.

Basis of Payment

Payment at the Contract Price for the above tender items shall include full compensation for all labour, equipment and material to do work.

Special Provision

SCOPE

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

SUBMISSION AND DESIGN REQUIREMENTS

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

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administer, 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 drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

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.

CSP FOR INTEGRAL ABUTMENTS – Item No.

Special Provision

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

Sand Fill

The sand fill for backfilling the CSP 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%

CONSTRUCTION

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Form concrete levelling pad and place CSPs and spacers.
2. Construct concrete levelling pads.
3. Place loose sand into 600 diameter CSP.
4. Install piles by driving to bedrock.
5. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeters of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

CSP FOR INTEGRAL ABUTMENTS – Item No.

Special Provision

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

<u>Criteria</u>	<u>Tolerance</u>
Maximum deviation of CSP from pile centroid	+/- 50 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation	+/- 10 mm

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 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.

BASIS OF PAYMENT

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

ROCK POINTS - Item No.

Non-Standard Special Provision

Scope

As part of the work under the above tender item, the Contractor shall supply TITUS Rock Injector Pile Points on HP 310 x 110 Piles.

References

OPSS 906 – Structural Steel

Materials

The pile points shall be of the following:

Product

Manufacturer

HPP-R-12

Titus Steel Company Ltd.
6767 Invader Cr.
Mississauga, ON
Tel (905) 564-2446

(Or approved equivalent)

Basis of Payment

Payment at the Contract Price for the above tender items shall be full compensation for all labour, equipment and material to do the work.

OBSTRUCTIONS – Item No.

Non-Standard Special Provision

Scope

As part of the work for the installation of piles and/or caissons as well as excavations for pile caps at the Blanche River structure for the foundation elements, the Contactor shall be alerted that the fill materials may contain cobbles and/or boulders. In addition, timber piles and timber cribbing for the existing bridge structure are present at the site and possibly from an abandoned abutment from an earlier roadway alignment that were observed west of the proposed north abutment area within the river.

Basis of Payment

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

UNWATERING FOR STRUCTURE EXCAVATION – Item No.

Non-Standard Special Provision

Scope of Work

The contractor shall be alerted that the soils at the Blanche River Bridge Replacement site consist of water-bearing sand and silt. Pile caps construction below the groundwater and/or river water levels must be carried out in-the-dry. The excavation shall be kept stable during the work.

Basis of Payment

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

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com



Golder Associates Ltd.
1010 Lorne Street
Sudbury, Ontario, P3C 4R9
Canada
T: +1 (705) 524 6861

