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REPORT ON

FOUNDATION INVESTIGATION AND DESIGN
DETAIL DESIGN
VERNON LAKE NARROWS
REPLACEMENT OF SOUTHBOUND STRUCTURE
HIGHWAY 11
W.P. 94-89-01, SITE NO. 42-018,
MINISTRY OF TRANSPORTATION, ONTARIO
DISTRICT 52, HUNTSVILLE, ONTARIO

Submitted to:

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VERNON LAKE NARROWS
REPLACEMENT OF SOUTHBOUND STRUCTURE
HIGHWAY 11
W.P. 94-89-01, SITE NO. 42-018
MINISTRY OF TRANSPORTATION, ONTARIO
DISTRICT 52, HUNTSVILLE, ONTARIO

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 southbound structure carrying Highway 11 over the Vernon Lake Narrows in Huntsville, Ontario.

The terms of reference for the scope of work are outlined in Golder's proposal P51-1687, dated November, 2005, that forms part of the Consultant's Agreement (P.O. Number 5004-E-0070) for this project. The work was carried out in accordance with the Quality Control Plan for this project dated May 16, 2006. The general arrangement drawing for the bridge structure was provided to Golder by LEA in July 2006.

The investigation was supplemented with information contained in the available existing data supplied by the MTO and LEA, specifically:

- Preliminary Design Report, Vernon Lake Narrows, Site 42-018, Highway 11 NBL and SBL, Huntsville Area, G.W.P. 94-89-00, March 2005, by Stantec Consulting Ltd. (Includes Preliminary Foundation Report by Peto MacCallum Ltd. in Appendix E).
- Pile Driving Records (Pile 4-10 and Pile 9), Contract 77-130, by Ministry of Transportation, dated June 1978.
- Contract Drawings, Structure and Approaches, Vernon Lake Narrows Bridge (Northbound Lane) 1.7 Miles South of Highway 60, W.P. 74-74-03 Contract No. 77-130, Ministry of Transportation and Communications, dated October 1977.
- Foundation Investigation Report for W.P. 74-74-03, Site No. 42-18N, Hwy. 11 District 11, Vernon Lake Narrows, N.B. Lane 1.7 Miles South of Hwy 60, Ministry of Transportation and Communications, dated January 1976.
- Contract Drawings, Vernon Narrows Bridge, Contract Number 57-32, by T.O. Lazarides, Lount and Partners Consulting Engineers, March 1956.
- Foundation Investigation, Vernon Narrows Bridge, by Peto MacCallum Ltd., December 1955.

2.0 SITE DESCRIPTION

The site is situated on the west side of the Town of Huntsville, on Highway 11 crossing Vernon Lake Narrows as shown on Drawing 1. The bridge is located between Vernon Lake in the west and Hunters Bay in the east. The road grade rises up to about 10 m on the south and north sides of the Narrows. 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 lake are vegetated with grass and small shrubs. The lake is used mainly for recreation and is approximately 210 m wide at the crossing location.

The existing southbound lane (SBL) bridge was constructed between 1957 and 1960. It currently has nine spans with eight in-water piers and will be replaced with a five-span structure with four in-water piers. The existing bridge is founded on piles driven to bedrock.

The highway grade is at about Elevation 294 m and 293 m at the existing south and north abutments, respectively. The water level in the lake was measured at approximate Elevation 284.1 m (July and August 2006) as indicated on the General Arrangement drawing. Previous drawings from 1977 indicate water levels as low as Elevation 283.9 m.

3.0 INVESTIGATION PROCEDURES

The fieldwork at the bridge site was carried out in two stages: six (6) boreholes (BH06-11 and BH06-12, BH06-17 to BH06-19 and BH06-24) were drilled on land; and eight (8) boreholes (BH06-13 to BH06-16 and BH06-20 to BH06-23) were drilled over-water. The land-based work was carried out between June 26 and July 6, 2006, and the water-based work was carried out between July 24 and August 18, 2006. The location and elevation of these boreholes are shown on Drawing 1 and noted on the respective Record of Borehole and Drillhole Sheets.

The land-based field investigation was carried out using either a track-mounted D-50 Bombardier drill rig or a truck-mounted D-90 Bombardier drill rig supplied and operated by Walker Drilling Ltd. (Walker) of Utopia, Ontario. The boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers as well as wash boring methods using ‘NW’ casing. Soil samples were obtained at 0.75 m to 1.5 m intervals of depth, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures with an automatic hammer. Shelby tube samples and in situ vane (‘N’ vane) tests were taken in cohesive deposits at some borehole locations. Rock core samples were obtained using an ‘NQ’ size rock core barrel.

The water-based field investigation was carried out using a D-90 Bombardier drill rig mounted on a barge. The rig was supplied by Walker while the barge was supplied by Kashe Barge Services of Gravenhurst, Ontario. These boreholes were advanced by wash boring methods using NW casing and rock coring using ‘NQ’ size rock core barrel. Tri-cone methods were used to advance the boreholes at some locations. Soil samples were obtained from 0.75 m to 1.5 m intervals of depth, from the tip of the casing.

The land-based boreholes were advanced to depths ranging from 15.5 m to 25.3 m below the existing ground surface. The water-based boreholes were advanced to depths ranging from 15.6 m to 26.6 m below the water surface at the time of drilling. A minimum of 3 m of rock core was obtained from seven of the boreholes drilled at this site, while six boreholes were terminated on practical refusal to auger or split spoon advance and one borehole was terminated at a depth of about twice the embankment height.

The groundwater conditions in the open boreholes were observed during the drilling operations and piezometers were installed in two land-based boreholes, BH06-11 and BH06-19, near the south and north abutments, respectively, to permit monitoring of the groundwater level at these locations. The piezometers consisted of a 50 mm outside diameter rigid PVC tubing with a 1.5 m long slotted screen, sealed within the sandy silt and/or the clayey silt/silt stratum. The boreholes and piezometers (after the last water level was obtained) were backfilled with bentonite and/or cement bentonite grout as per Ontario Regulation 128 (amendment to O. Reg. 903). The installation details and water level readings are presented on the Record of Borehole sheets that follow the text of this report.

The soil cuttings from the land-based boreholes were distributed along the slopes of the embankments. The wash water from the water-based boreholes was pumped into a settling tank which was subsequently pumped onto shore behind a silt fence.

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 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 samples. Two one-dimensional consolidation (oedometer) and three unconfined compression tests were carried out on Shelby Tube samples from three boreholes. In addition, point load strength and unconfined compressive strength tests were carried out on selected portions of the bedrock cored from the boreholes.

It should be noted that the location of the existing watermain, which crosses the bridge alignment under the lake channel to the west of and under the bridge, was coordinated by LEA.

The locations of the proposed foundation elements were laid out in the field by Golder staff relative to the existing bridge foundation units and in reference to the general arrangement drawing supplied by LEA. The as-drilled locations were measured in reference to the existing bridge abutments and piers. The ground surface elevation of the boreholes on land were surveyed relative to working points on the bridge abutments and referenced to geodetic datum. The boreholes in the water were referenced to the lake water level at the time of drilling, which was referenced to the bridge pier foundations of known elevation.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Published literature indicates that the site is located in the Huntsville Domain of the Algonquin Terrane, which is located in the Grenville Province of the Canadian Shield (Geology of Ontario; OGS Special Volume 4). The bedrock of this domain consists of thin sheets of shallow dipping orthogneiss (i.e. having igneous origins) with interleaves (1 cm to 10 cm thick) of flaggy high grade gneisses and tectonites. The site occurs within an area mapped as flaggy layered gneiss, which is considered to have plutonic and sedimentary origins. The rock has been metamorphosed to the granulite facies (high temperature and pressure). Steeply dipping shears are common within the area and are typically dipping in the east-northeast direction.

4.2 Subsoil Conditions

The detailed subsurface soil and groundwater conditions, as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil 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. The inferred soil stratigraphy based on the results of the boreholes at the bridge location is shown on Drawings 1 to 3.

In general, the subsoils in the land-based boreholes consisted of embankment fill underlain by silty clay to clayey silt, silt, sandy silt to silty sand and gravelly sand deposits overlying bedrock. The silty sand and gravelly sand deposits contained cobbles and boulders. In the water-based boreholes, the depth of water ranged between 1.7 m and 5.2 m. The subsoils in these boreholes generally consisted of an alluvium layer consisting of silty sand or clayey silt, underlain by deposits of clayey silt, silt, sandy silt and gravelly sand to silty sand overlying bedrock. The gravelly sand to silty sand deposit contained cobbles and boulders. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill and Topsoil

Boreholes BH06-11, BH06-12, BH06-17 and BH06-24 were advanced at the approaches to the bridge structure within the existing southbound lanes. Approximately 125 mm to 280 mm of asphalt was encountered overlying a 150 mm to 225 mm thick layer of sand and gravel road base fill. Borehole BH06-18, advanced in the shoulder of the road, encountered a 150 mm thick layer of sand and gravel fill at the ground surface overlying 75 mm of asphalt.

Borehole BH06-19 was advanced at the toe of the slope on the west side of the existing south abutment. About 100 mm of sandy topsoil was encountered at the ground surface at this borehole.

Underlying the road base materials, boreholes BH06-11, BH06-12, BH06-17, BH06-18 and BH06-24 penetrated layers of granular fill consisting of sand, silty sand, and/or silt. The fill thickness ranged from 8.5 m to 10.6 m on the north side of the bridge and between 2.3 m and 7.6 m on the south side of the bridge. Occasional cobbles were noted within the fill materials in borehole BH06-17.

SPT ‘N’ values measured within the fill ranged between 4 and 78 blows per 0.3 m of penetration, indicating a very loose to very dense relative density. In general, ‘N’ values are less than 50 blows and the fill is considered to be compact to dense. Grain size distributions of two samples of the sand to silty sand fill are shown on Figure A-1.

The natural water content measured on samples of the fill ranged between 4 percent and 18 percent.

4.2.2 Alluvium

In each of the water-based boreholes, alluvium was encountered at the bottom of the lake bed. In boreholes BH06-15, BH06-16, BH06-20 and BH06-21, the alluvium consisted of silty sand or sandy silt, containing trace organics in some boreholes, and was between 0.1 m and 0.5 m thick. In boreholes BH06-13, BH06-14, BH06-22 and BH06-23, the alluvium consisted of clayey silt to silty clay containing trace to some organics and ranged from 0.1 m to 3.5 m in thickness. The surface of the alluvium varied between Elevation 278.9 m and 282.4 m.

Measured SPT ‘N’ values within the alluvium ranged from 0 blows (weight of hammer) to 2 blows per 0.3 m of penetration indicating a very loose relative density of very soft to soft consistency.

An Atterberg limits test carried out on one sample of the clayey silt alluvium deposit measured a liquid limit of about 53 percent and a plastic limit of about 28 percent yielding a plasticity index of about 25 percent. The results of the Atterberg limits test are shown on the plasticity chart on Figure A-2 in Appendix A and classify the deposit as a silty clay of high plasticity. A grain size distribution of a sample of the clayey silt alluvium is shown on Figure A-3.

The natural water content measured on samples of the alluvium ranged between 32 and 73 percent. The higher water contents are likely attributed to the presence of organics.

Directly underlying the fill in borehole BH06-12, a 1.5 m thick layer of silt containing some sand, trace clay and trace organics was encountered at Elevation 282.6 m. In borehole BH06-18, underlying the fill, a 0.4 thick layer of organic silt with trace sand was encountered at Elevation 291.1 m, underlain by a 0.5 m thick layer of silt containing trace to some clay and trace sand. Measured ‘N’ values in the silt layer in borehole BH06-12 and in the organic silt and silt layers in borehole BH06-18 were 20 and 12 blows per 0.3 m of penetration, respectively, indicating a compact relative density. The natural water content of the sample of silt from BH06-12 was 20 percent and the sample of organic silt from BH06-18 was 26 percent.

4.2.3 Clayey Silt

A deposit of grey clayey silt to silty clay was encountered below the surficial fill, topsoil, silt and alluvium deposits in all boreholes. The top of this deposit varied between Elevation 281.1 m and 290.2 m in the land-based boreholes and between Elevation 278.3 m and 281.8 m in the water-based boreholes. The thickness of the deposit ranged from 1.7 m to 8.1 m.

Land-Based Boreholes

The samples of this deposit from the land-based boreholes consisted of clayey silt to silty clay containing trace sand, trace gravel and trace organics near the surface of the deposit. Measured SPT ‘N’ values ranged from 2 blows to 24 blows per 0.3 m of penetration. In situ field vane testing carried out in these boreholes measured undrained shear strengths ranging from 31 kPa to greater than 100 kPa. In general, the field vane shear strengths together with the SPT ‘N’ values suggest that the clayey silt to silty clay stratum has a soft to very stiff consistency.

Atterberg limits testing carried out on six samples of the deposit from the land-based boreholes indicate liquid limits ranging from about 32 percent to 40 percent and the plastic limit ranging from about 20 percent to 24 percent, yielding plasticity indices ranging from about 10 percent to 19 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure A-4 in Appendix A and indicate that the material is classified as a clayey silt of low plasticity to a silty clay of intermediate plasticity. A grain size distribution test was carried out on one sample of the clayey silt deposit and the results are shown on Figure A-5.

The natural water content measured on select samples of this deposit ranged between 25 percent and 47 percent. The natural water content of samples of the clayey silt to silty clay were typically near or greater than the corresponding liquid limit, resulting in liquidity indices up to 1.22.

Three unconfined compression tests were carried out on specimens of the clayey silt to silty clay obtained from boreholes BH06-11, BH06-18 and BH06-19. Details of the test results are shown on Figures A-6 to A-8 in Appendix A and the unconfined compression test results are summarized below.

Borehole and Sample Number	***Elevation (m)***	***Compressive Stress (kPa)***	***Undrained Shear Strength (kPa)***
BH06-11 SA 11	280.3	125	62
BH06-18 SA 7	288.5	562	281
BH06-19 SA 6	282.9	210	105

Two laboratory consolidation (oedometer) tests were carried out on specimen of the clayey silt to silty clay obtained from boreholes BH06-11 and BH06-19 and the test results are shown on Figure A-9 and Figure A-10 in Appendix A. The pre-consolidation pressures were estimated from the voids ratio versus logarithmic pressure plots using the Casagrande method and from the total work versus pressure curve. The relevant oedometer 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)***
BH06-11 SA11	280.3	215	350	135	1.6	0.83	0.032	0.161	0.060
BH06-19 SA6	282.9	80	300	220	3.1	0.77	0.016	0.203	0.073

Note: [*]For stress range of $300 \leq \sigma_v' \leq 500$ 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 cm^2/s in the normally consolidated range

Water-Based Boreholes

The samples of this deposit from the water-based boreholes consisted of clayey silt, trace sand. SPT ‘N’ values measured in this deposit from the water-based boreholes ranged from 0 blows (weight of hammer) to 11 blows per 0.3 m of penetration. In situ field vane testing carried out in these boreholes measured undrained shear strengths ranging from 9 kPa to 34 kPa. In general, the field vane and SPT ‘N’ values suggest the clayey silt stratum has a very soft to stiff consistency, becoming stiffer near the base of the deposit.

Atterberg limits testing carried out on five samples of the deposit from the water-based boreholes indicate liquid limits ranging from about 27 percent to 40 percent and plastic limits ranging from about 20 percent to 25 percent, yielding plasticity indices ranging from about 6 percent to 15 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure A-11 in Appendix A and indicate that the material is classified as a clayey silt to silt of low plasticity to a silty clay of intermediate plasticity. These test results typically plot at or just below the A-line indicating this material has a significant silt content. Grain size distribution testing was carried out on four samples of the clayey silt deposit and the results are shown on Figure A-12.

The natural water content measured on select samples of this deposit ranged between 26 percent and 51 percent. The natural water content of samples of the clayey silt were typically near or greater than the corresponding liquid limit, resulting in liquidity indices of between 1.0 and 1.3.

4.2.4 Silt

A deposit of silt containing trace sand and trace to some clay was encountered below the clayey silt in all the boreholes, except in borehole BH06-12 where the silt layer was encountered overlying the clayey silt deposit, and in boreholes BH06-11 and BH06-24 where the silt stratum is not present. The surface of the deposit was generally encountered between Elevation 282.0 m and 287.0 m in the land-based boreholes and between Elevation 271.0 m and 279.9 m in the land-based boreholes. The thickness of the silt deposit ranged between 1.5 m and 4.6 m.

Measured SPT 'N' values in the silt deposit ranged from 6 to 23 blows per 0.3 m of penetration, indicating a loose to compact relative density.

Atterberg limits testing carried out on one sample of the silt deposit in (borehole BH06-20) indicates that the sample is non-plastic. Grain size distributions for four samples from the silt deposit are shown on Figure A-13.

The natural water content measured on samples of the silt deposit ranged from 12 percent to 39 percent, and were typically greater than 25 percent.

4.2.5 Silty Sand to Sandy Silt

A deposit of silty sand to sandy silt containing trace clay was encountered below the clayey silt or silt deposits in boreholes BH06-11, BH06-12, BH06-14, BH06-15 and B06-20 to BH06-24. The surface of the deposit was encountered between Elevation 267.2 m and 278.1 m and ranged between 1.1 m and 5.2 m in thickness. Borehole BH06-11 was terminated within this deposit.

Measured SPT 'N' values in the sandy silt to silty sand deposit ranged from 6 blows to 23 blows per 0.3 m of penetration, indicating a loose to compact relative density.

The natural water content measured on samples of the silty sand to sandy silt deposit ranged from 28 percent to 31 percent.

4.2.6 Gravelly Sand to Silty Sand

Underlying the deposits of silt or sandy silt to silty sand, a deposit consisting of gravelly sand to silty sand was encountered in all the boreholes except BH06-11. Cobbles and boulders, inferred from difficult augering, grinding of augers and bouncing of the split spoon sampler, were encountered within the deposit. The surface of the deposit ranges from Elevation 263.7 m to 283.7 m and the thickness varied between 1.1 m and 7.6 m.

Measured SPT ‘N’ values ranged from 10 blows to greater than 100 blows per 0.3 m of penetration, indicating a loose to very dense relative density. Typically, based on the average ‘N’ values, the deposit is considered to be compact to dense.

A grain size distribution for one sample of gravelly sand to silty sand containing some gravel is shown on Figure A-14.

In borehole BH06-18, the gravelly sand to silty sand was noted to heave into the hollow stem augers upon penetrating about 4 m into this deposit.

The natural water content measured on samples of the gravelly sand deposit ranged from 7 percent to 23 percent.

Casing show blockage and auger refusal on probable boulders within this deposit were encountered in boreholes BH06-15 and BH06-17 at depths of 19.4 m and 17.2 m, respectively, corresponding to Elevation 264.7 m and 276.8 m, respectively. In borehole BH06-24, NW casing was used to advance the borehole below 20.0 m due to difficulty advancing the hollow stem augers at this depth.

4.2.7 Bedrock

Split spoon and/or auger refusal on probable bedrock was encountered in boreholes BH06-12, BH06-17 and BH06-22 at depths of 21.6 m (Elevation 271.4 m), 17.2 m (Elevation 276.8 m) and 23.7 m (Elevation 260.4 m), respectively. Gneiss bedrock was confirmed by coring between 3.0 m and 6.8 m in boreholes BH06-13, BH06-14, BH06-16, BH06-19, BH06-21, BH06-23 and BH06-24. The surface of the bedrock was encountered at depths between 14.3 m and 22.2 m below existing ground surface in the land-based boreholes; the bedrock surface in the land-based boreholes ranged from Elevation 273.5 m to 270.8 m. The surface of the bedrock was encountered at depths between 16.1 m and 22.6 m below the water surface in the water-based boreholes; the bedrock surface in the water-based boreholes ranged from Elevation 268.0 m to 261.5 m.

The rock core is described as a gneiss, grey, fined to medium grained and fresh to slightly weathered. In borehole BH06-13, a layer of silt and cobbles were encountered between 18.8 m and 19.3 m of depth, and a silt seam was encountered between 20.8 m and 21.3 m of depth. A sand layer was encountered within the bedrock in borehole BH06-23 between 21.2 m and 21.9 m of depth. The Rock Quality Designation (RQD) measured on the core samples ranged from 0 percent to 100 percent. This indicates rock mass variable in quality, ranging from very poor to excellent. In general, the RQD values were greater than 50 percent indicating that the gneiss is of fair to excellent quality. Generally, the RQD values increase with depth. In borehole BH06-23, broken rock was encountered in the core barrel between 18.8 m and 19.8 m of depth, resulting in a very low RQD value.

Uniaxial compression strength (UCS) testing was carried out on three core samples of the gneiss bedrock from boreholes BH06-14, BH06-16 and BH06-24. The UCS results were between 44 MPa and 86 MPa. The depths and corresponding elevations of the samples and results of the UCS testing are presented in Table A-1. Diametral (i.e. horizontal or perpendicular to the core axis) point load strength tests were performed on twelve core samples of the gneiss bedrock from boreholes BH06-13, BH06-14, BH06-16, BH06-19, BH06-21 and BH06-24. Diametral point load index values ranged from about 3.8 MPa to 7.5 MPa which correspond to estimated UCS values between 76 MPa and 150 MPa with an average strength of about 98 MPa, as presented in Table A-2. Using the Intact Rock Strength Classification table, these results indicate that the gneiss rock is classified as medium strong to very strong.

4.2.8 Groundwater Conditions

The water levels were noted during and after the drilling and coring operations in the boreholes. Piezometers were installed in boreholes BH06-11 and BH06-19 with screened zones sealed within the clayey silt/silt and/or sandy silt deposits. Details of the piezometer installations are shown in the Record of Borehole Sheets following the text of this report. The water levels in the piezometers and open holes upon completion of drilling are summarized below.

Location	***Borehole***	***Installations***	***Groundwater Level Depth (m)***	***Groundwater Level Elevation (m)***	***Date***
South Approach	BH06-18	Open Borehole	4.6	288.8	Upon Completion of Drilling
South Abutment	BH06-19	Piezometer	3.2 3.4	284.6 284.4	July 4, 2006 August 24, 2006
	BH06-17	Open Borehole	12.8	281.2	Upon Completion of Drilling
In-Water Piers	BH06-13 to BH06-16, BH06-20 to BH06-23	Lake Water Level	0	284.1	July and August 2006
North Abutment	BH06-12	Open Borehole	10.4	282.6	Upon Completion of Drilling
	BH06-24	Open Borehole	9.1	283.8	Upon Completion of Drilling
North Approach	BH06-11	Piezometer	7.6 7.7	285.2 285.1	July 4, 2006 August 24, 2006

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. In borehole BH06-18, gravelly sand was noted to flow into the hollow stem augers upon augering at a depth of 13.7 m.

The above groundwater levels are consistent with the adjacent lake water level, rising slightly away from the lake. The water level in the lake was measured at Elevation 284.1 m (July and August 2006), as noted in the General Arrangement drawing. It should be noted that groundwater levels in the area are subject to seasonal fluctuations and precipitation.

4.3 Closure

The field technicians supervising the drilling program were Mr. Ed Savard and Mr. Indulis Dumpis. This report was prepared by Mr. André Bom, P.Eng., a geotechnical engineer; the technical aspects were reviewed by Ms. Sarah Coyne, P.Eng. A quality control review of the report was provided by Mr. Jorge Costa, P.Eng., a Designated MTO Contact for Golder.

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PART B

FOUNDATION DESIGN REPORT
VERNON LAKE NARROWS
REPLACEMENT OF SOUTHBOUND STRUCTURE
HIGHWAY 11
W.P. 94-89-01, SITE NO. 42-018
MINISTRY OF TRANSPORTATION, ONTARIO
DISTRICT 52, HUNTSVILLE, ONTARIO

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides design recommendations on the foundation aspects of the proposed Highway 11 Southbound Lane bridge structure over the Vernon Lake Narrows. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are 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.

The existing bridge is a nine-span structure, about 210 m in overall length. The proposed works involve replacing the existing SBL bridge with a new five-span structure about 230 m in overall length. This requires that the abutments be moved back by about 10 m each, relative to the location of the existing abutments. We understand that the abutment and pier foundations will not be at the same location as the existing foundations. The recommendations provided in this report discuss issues related to the interaction of the proposed foundations with the existing piers which are to remain in place. The existing embankments at the abutments are between 9 m and 10 m in height above the existing lake level. The proposed grade of the highway at the southbound lanes will not change significantly; however, there will be some minor widening of the embankment towards the west. The recommendations provided will also address the settlement and stability of the approach embankments.

It should be noted that the new/proposed NBL structure will be constructed by the time the SBL contract is tendered. The new SBL structure will be located between about 4 m and 8 m west of the new NBL structure. The recommendation on the geotechnical design aspects of the new structure presented in this report also consider any implications of the new widened NBL bridge foundations in relation to the proposed SBL foundations.

5.1 General Bridge Foundation Options

Shallow foundations are not recommended for support of the new SBL bridge due to the presence of soft compressible soil at this site and considering that the existing foundations are deep (i.e. piles). Consideration should be given to the use of deep foundations comprised of either piles or caissons for support of the new footings. The foundation type chosen will depend on:

- The interaction between the existing and proposed foundations;
- The interaction between the two bridges;
- The potential construction techniques for in-water piers; and
- The soil conditions (i.e. depth to end-bearing stratum).

Table 1 (attached), summarizes the advantages, disadvantages, relative costs and risks / consequences of the two deep foundation alternatives. Discussion on the alternatives is given in the sections below. The preferred alternative at this site is steel H-piles driven to bedrock within a pre-cast cofferdam and steel casing.

5.2 Construction Considerations

Based on our assessment of the subsurface conditions at the site and as noted above, it is recommended that driven steel H-piles be used as the preferred founding alternative. The following paragraphs describe the proposed methodology for pile installation and pile cap construction for the driven pile alternative at the piers. Pile installation at the abutments should be carried using standard construction techniques.

We understand from the available contract drawings (1958) that the existing SBL structure piers are supported on 450 mm diameter tube piles driven to bedrock and filled with concrete. For each pier, the piles were installed in groups of three or four, in two groups per pier. The abutments are founded on steel H-piles. The piers were constructed generally using the following method:

- A pre-cast concrete cofferdam (i.e. pile cap) of 12 feet (3.7 m) diameter was floated into place and secured to the lake bed with spud piles; and
- Once the cofferdam was in place, twin groups of 18 inch (450 mm) diameter steel tube piles with 1:6 batter were installed inside the cofferdam and backfilled with concrete, followed by construction of the pile cap.

For information purposes, we understand from the available contract drawings (1977) and pile driving records that the piers for the NBL structure were built using generally the method described below:

- A pre-cast concrete cofferdam was floated into place and positioned such that the top of the cofferdam extended approximately 0.3 m above the water level. The pre-cast cofferdam had six pre-drilled holes with a 50" (1.22 m) diameter steel tube sleeve installed on the design batter;
- Once the cofferdam was positioned at the pier locations, 48" (1.2 m) diameter steel casings with 1:8 batter were installed inside the tube sleeve by wash boring methods to a depth just below the interface of the clay and sand deposits;

- The casings were flushed/pumped and cleaned out to the bottom and three 1:8 battered steel H-piles were driven inside the casing to the bedrock surface;
- Once the piles were driven, the casing was grouted up to the base of the cofferdam; and
- After the tremie plug (i.e. grout) was in place for all six locations, the cofferdam was pumped out and the pile cap was constructed.

This method of steel H-pile installation allowed for relatively straightforward in-water construction of the NBL foundations. In addition, the steel casing left-in-place surrounding the piles provided for scour protection of the piles below the base of the pile cap. The pre-cast cofferdam itself formed part of the pile cap.

We consider and recommend that this same construction technique (or similar) could be used to construct the piers for the new SBL structure. Ultimately, the design of the cofferdam will be the responsibility of the contractor. In addition, the project environmental sub-consultant should confirm regulatory requirements applicable to mucking/air lifting/washing out the casings, in regards to disposal of cuttings and sedimentation into the lake. This pile installation operation will create a significant amount of cuttings for disposal. Therefore, an appropriate Non-Standard Special Provisions (NSSP) should be created by the environmental sub-consultant that details the handling and disposal of this material. It is likely that a sediment containment system will be required on-shore where the material can be treated (i.e. dried) prior to disposal at an appropriate facility.

In the case of the new SBL piers, this construction technique should be compatible with and not affect or be affected by the existing foundations, as the new foundation elements are a minimum of 7 m away from the existing foundation elements.

We assume that the existing SBL pier piles will be cut off at the lake bed level and will not be pulled out. The designer should check that the new piles (batter and orientation) do not interfere with the existing piles. The designer should also check that the new piles do not interfere with the NBL pier piles. This should be checked to the full extent of the pile length to the bedrock surface.

From a vibration standpoint, it is our opinion that the vibrations generated during pile driving will typically be low and should not be a significant issue at this site. Since there is at least 4 m to 8 m separating the widened NBL structure from the new SBL foundation elements, vibration monitoring is not required for this work.

It is possible that contractors may choose to use a standard sheet-pile cofferdam to construct the piers at this site as an alternative. Although sheet-pile cofferdams are feasible at this site, we do not recommend this technique since the sheet piles may have to extend to below the base of the clay deposit (some 10 m below the lake bed), and likely would require a relatively thick tremie plug to prevent base heave and to provide adequate lateral resistance for the sheet piles.

If the caisson alternative is considered, it may be possible to eliminate the requirement for pile caps and therefore cofferdams, by extending the caissons up to the underside of the bridge deck (i.e. pier cap).

At this time, we consider that all the in-water work can be carried out from a barge. However, depending on the lake levels at the time of construction, the piers closest to the banks may not have an adequate depth of water and, therefore, consideration should be given to constructing an access road in the water to these near shore locations.

5.3 Shallow Foundations

Due to the presence of compressible clayey silt to silty clay subsoils in the area of the abutments and thus the potential for differential settlement of the abutment, spread footings are not considered feasible at this site. In addition, the existing abutment footings are also founded on piles and therefore spread footings for the new SBL structure would not be consistent with the existing foundations.

5.4 Steel H-Pile Foundations

Based on the borehole information obtained at this site, piles driven to bedrock are recommended for support of the foundations of the new footings. The existing soils are not suitable for friction piles and the bedrock was encountered at a reasonable depth, therefore piles end-bearing on the bedrock are highly suitable for this site. Also, this foundation method would match the existing foundation condition consisting of piles end-bearing on bedrock.

5.4.1 Axial Geotechnical Resistance

The bedrock was encountered between about Elevation 260.4 m and Elevation 273.5 m at the new abutment and pier locations. This elevation corresponds to the design pile tip level. The current lake bed is at between Elevation 278.9 m and Elevation 282.4 m at the pier borehole locations, resulting in piles up to about 21 m in length at the piers (up to about 24 m below the July/August 2006 water level) and up to about 18.5 m in length at the abutments relative to the underside of the pile caps. The design pile tip elevation and the ground surface/lake bed elevation are given in Table 2. Also presented in Table 2 are the approximate interpreted elevations where very dense gravelly sand containing cobbles and boulders were encountered, as such conditions may impact the final tip elevation. The elevation of the bedrock should be assumed to be the design pile tip elevation; however, practically, the piles could "hang up" on the very dense layer (or boulders) within the gravelly sand layer.

The steel casing of each pile or pile group should be sized to accommodate the piles (as determined by the structural engineer). The casing size should take into account the pile batter and orientation. The casing should extend to about 1 m of depth below the base of the clay deposit at each location, resulting in casing lengths between about 5.5 m and 13.5 m below the July/August 2006 water level. The design base elevation of the casing at each pier location is given in Table 3.

For steel HP310x110 piles driven to bedrock, the factored axial resistance will be dependent upon the structural capacity of the pile; however, a factored axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be used for design. Since bedrock is considered to be a non-yielding material, the geotechnical resistance at Serviceability Limit States (SLS) will be higher than the ULS value and, therefore, the ULS value will govern the design. The above values assume that the pile is not "hanging up" on a boulder.

Pile installation should be in accordance with SP903S01. The piles should be fitted with Titus Ejector rock points or equivalent and appropriate driving procedures must be adopted to ensure adequate/proper seating of the piles on sloping bedrock without damaging the piles. The appropriate NSSP should be included in the Contract Documents; an example is included in Appendix B for reference. The driving procedures to enable seating on bedrock depend on the type of pile driving rig used and these procedures need to be established at the time of construction. Generally, the procedures will involve a reduction in hammer energy once abrupt peaking is met to ease the pile point into the rock. For piles driven into the bedrock, the following note should be included on the drawings:

- "Piles to be driven to bedrock."

Steel casings installed below the lake bed through the very soft to firm clayey silt deposit should be backfilled using tremie concrete to the underside of the cofferdam after pile installation. The steel casings are considered to be permanent and are to be left in place.

5.4.2 Resistance to Lateral Loads

Lateral loading could 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 (e.g. ice 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 Brom's 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 (kN/m^3 or 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 is the constant of horizontal subgrade reaction (kPa/m)
z is the depth (m)
B is the pile diameter/width (m)

and for cohesive soils:

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

Where

s_u is the undrained shear strength of the soil (kPa)
B is the pile diameter/width (m)

The values of n_h and s_u to be assumed in the structural analysis are given in Table 4. The different values reflect the variability in the subsurface conditions as well as the two extremes of design: the requirement for flexibility in the case of integral abutments and the requirement for lateral support in the case of non-integral abutments and the piers. A maximum lateral resistance of 120 kN at ULS and 35 kN at SLS is recommended for HP 310x110 piles.

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. For the proposed piles (HP 310x110) driven to bedrock through the very soft to firm clayey silt at this site, the horizontal resistance at ULS will be controlled by structural limitations such as the yield moment (M_{YIELD}) of the pile (i.e. Brom 1964 method). 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. 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 piers and abutments (CHBDC C6.8.7.1).

The upper zone of soil (down to a depth below the pile cap equal to about 1.5 x B after Brom 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

5.4.3 Downdrag

The subsurface soils consist of overconsolidated clayey silt underlain by granular deposits. Since the widening for the new embankments is expected to be minimal and since there will be a net unloading of the soils as a result of moving the abutments back, downdrag loads on the new piles need not be considered.

5.4.4 Frost Protection

The pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection (at the abutments). If the required soil cover cannot be provided, consideration could be given to the use of rigid polystyrene foam insulation below the footings. As a guideline, one inch of rigid polystyrene foam insulation may be used for every 0.45 m reduction in soil cover.

5.5 Caissons

If it is desirable to eliminate the need for pile caps in the water, then consideration could be given to the use of caissons for support of the piers by extending the caissons up to the underside of the bridge deck (i.e. pier cap). Given that the existing SBL and NBL structures are founded on piles and the depth to bedrock, caissons may not be the most practical alternative for the abutment and pier foundations for the new SBL structure at this site.

5.5.1 Axial Geotechnical Resistance

If caissons are considered as a founding alternative, the caissons at this site will derive their axial resistance mainly from end-bearing. The depth to bedrock at each of the pier and abutment locations is given in Table 2. The factored axial geotechnical resistance at ULS for various diameter caissons socketted a minimum of 2 m into the bedrock are given below:

Caisson Diameter(m)	***Gneiss Bedrock (minimum 2 m socket)***	
	ULS	**SLS**
1.5	8,000 kN	n/a
1.8	10,000 kN	n/a

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

It should be noted that there may be difficulty in socketting the caissons within the hard gneiss bedrock, particularly if the bedrock surface is sloping or if the bedrock is fractured. Temporary liners and tremie concrete will likely be required to install caissons at this site.

5.5.2 Resistance to Lateral Loads

The resistance to lateral loading for the caissons should be in accordance with Section 5.4.2 and Table 4, using the horizontal subgrade reaction formulas. The recommended maximum lateral resistance for the caissons is as follows:

Caisson Diameter (m)	***Factored Lateral Resistance at ULS (kN)***	***Lateral Resistance at SLS (kN)***
1.5	2,400	700
1.8	3,400	1,000

5.5.3 Frost Protection

Caisson caps at the abutments should be provided with a minimum of 1.8 m of soil cover for frost protection or sufficient insulation as described in Section 5.4.4.

5.6 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.

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

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' Type II should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with 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 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.8 m behind the back of the wall stem (Case I in Figure C6.9.1(l) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(l) 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	**Rock Fill**
Soil unit weight:	21 kN/m^3	19 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.31	0.22
At rest, K_o	0.47	0.35

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

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:

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

Seismic (earthquake) loading must also be taken into account in the design 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 will 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 the National Building Code of Canada, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio for Huntsville is 0.05. Based on experience, for the subsurface conditions at this site, a 30 percent amplification of the ground motion will occur, resulting in an increase in the ground surface acceleration from 0.05g to 0.065g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.065.
- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.03$). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.10$). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration, k_v. Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2/3\ k_h$, $k_v = 0$, and $k_v = -2/3\ k_h$.
- The following seismic active pressure coefficients (K_{AE}) for the two cases (Case I and Case II) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I		Case II	
	Earth Fill	**Rock Fill**	**Granular A**	**Granular B Type II**
Yielding wall	0.30	0.22	0.26	0.26
Non-yielding wall	0.34	0.25	0.30	0.30

Note : These CHBDC seismic K_{AE} values include the effect of wall friction ($\delta=\phi'/2$) and are less than the static values of K_a and K_o reported above for the very low zonal acceleration ratio for this site.

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.065. This corresponds to displacements of up to 16 mm at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

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

Where K = either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
K_{AE} = the seismic active earth pressure coefficient;
γ' = the effective unit weight of the soil (kN/m^3):
- taken as soil unit weights given above for fill materials;
- taken as 21 kN/m^3 above Elev. 284 m for the native materials, if present;

d = the depth below the top of the wall (m); and
H = the height of the wall above the toe (m).

5.7 Liquefaction Potential and Seismic Analysis

5.7.1 Analysis Methods

The liquefaction potential of the granular soils below the immediate approach embankments and under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the *CHBDC Commentary*, which correlates the cyclic resistance ratio (CRR) and the cyclic stress ratio (CSR) of the soils with their normalized penetration resistance and fines content for granular soils. The CRR has been determined using the empirical method suggested by the CHBDC based on papers by Seed et al (1984) using SPT 'N' values and accounting for fines content. The method used to determine the CSR will be the simplified procedure suggested by Seed and Idriss (1971) relating to the peak ground acceleration and effective overburden stress.

In general, geologically young, loose deposits of sand and non-plastic silty sands with low fines content (less than 5 percent passing No. 200 sieve) which are below the water table are potentially susceptible to liquefaction.

5.7.1.1 Liquefaction Induced Settlements

Where liquefaction is identified to be a problem either in clayey soils or in granular soils using the methods described above, vertical deformation of the soil under the earthquake loading may occur due to the contraction of the sand deposit using a relationship developed by Tokimatsu and Seed (1987). This deformation can be estimated using relationships proposed by Makdisi and Seed (1978). If deformation is anticipated, soil improvement methods should be considered and could include densification, removal and re-compaction, grouting, or permanent drainage so that the pore water pressure rise necessary to trigger liquefaction is controlled.

5.7.1.2 Stability under Seismic Conditions

The susceptibility of the soil deposits underlying the proposed roadway embankments and the consequent stability of the embankment under seismic loading conditions for this site has been assessed. The peak zonal acceleration for this site (Huntsville) is 0.065g, which is based on a zonal acceleration of 0.05 g multiplied by an amplification factor of 30 percent for the types of soils found in this area. Typically, the seismic loading will be applied to the long-term (drained) conditions.

If liquefaction of the subsoils under the embankment loading is not anticipated, a factor of safety of 1.0 is typically used to assess the stability under magnitude 7.0 earthquake events.

Where liquefaction is triggered in the underlying soil deposit, the stability of the embankment is analyzed using post-liquefaction, residual strength parameters in the liquefied layers using the correlation proposed by Seed and Harder (1990) which is correlated to SPT ‘N’ values. If under these conditions, the embankment is estimated to have a factor of safety less than 1.0 under static conditions, the embankment is considered to be susceptible to a flow slide. Flow slides are characterized by very large lateral and vertical displacements of the embankment. If under residual strength conditions, the static factor of safety is greater than 1.0, lateral displacements may still occur, and are estimated using the Newmark method, which compares the design ground acceleration to that necessary to induce a factor of safety equal to 1.0 in the embankment (i.e. yield acceleration). If the yield acceleration is greater than the maximum acceleration for this site, then no remedial measures are required. If the yield acceleration is less than the maximum acceleration, soil improvement methods may be necessary to improve soil conditions.

5.7.2 Results of Analysis

Using the methods outlined in Section 5.7.1, the soils at this site are not considered to be liquefiable. A factor of safety of greater than 1.0 is obtained for magnitude 7.0 earthquake events.

5.8 Approach Embankment Design and Construction

We assume that there will be no grade raise of the existing highway and that any widening of the existing embankment (i.e. on the west side of the bridge) will be minimal. In addition, the abutments will be moved back by about 10 m on both ends of the bridge. Therefore, there will be a net unloading of the soils in front of the abutment. The new SBL bridge will be skewed relative to the widened NBL bridge; the two bridges will be separated 4 m and 8 m from each other at the south and north ends, respectively. The following sections present the results of settlement and stability analysis for the SBL bridge approach embankments and abutments and subsequent recommendations.

The existing SBL bridge abutments are located very close to the water, as compared to the NBL bridge abutments. Moving the abutments back by about 10 m will allow for a geometry of 3H:1V in front on the abutments. The existing slope geometry on the west side of the south abutment embankment is greater than 2H:1V and on the north abutment embankment it is between 1.6H:1V and 2H:1V. The overall height of the embankments above the water level is between about 9 m and 10 m. For design purposes, the groundwater level is assumed to be consistent with the lake level, at about Elevation 284 m.

The methodology, parameter selection and results of stability and settlement analysis for the widened approach embankments at the abutments are presented in the following sections.

5.8.1 Stability

5.8.1.1 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, excavation depths and geometries. In general, circular slip surfaces were analyzed in the design. Non-circular slip surfaces were not analyzed since there are no obvious thin/weaker zones within the clayey silt deposits.

5.8.1.2 Parameter Selection

The subsoils encountered at the site are composed of granular fill for the existing embankment, and natural deposits of soils (silt, sandy silt and gravelly sand) or cohesive materials (clayey silt and alluvium). For granular soils, effective stress parameters were employed in the analyses assuming drained conditions. The effective stress parameter (effective friction angle) for the granular soils was estimated from empirical correlations using the results of in situ SPT results, in conjunction with engineering judgement considering experience in similar soil conditions.

For cohesive deposits below the embankment, effective-stress parameters were employed in the analyses assuming drained conditions. This assumption is made based on the fact that the embankment has been in place since the 1950s and that no significant change in loading is anticipated. The effective stress parameters for the cohesive soils below the embankment were derived based on correlations with the SPT results and other laboratory test data (natural water content and Atterberg limits) as well as empirical data for similar soils.

For cohesive deposits outside the toe of the existing embankment, total-stress parameters were employed in the analyses assuming undrained (i.e. short-term or during construction) conditions for these soils. The total stress parameters (i.e. average mobilized undrained shear strength – s_u) for the cohesive soils were derived based on the results of field vane shear tests (where applicable) and estimated from correlations with the SPT results and other laboratory test data (natural water content and Atterberg limits).

The parameters used in the stability analysis are given below:

Soil Type	***Unit Weight* (kN/m^3)***	***Undrained Shear Strength (kPa)***	***Angle of Internal Friction***
New Earth Fill (Assume Granular Material)	21	--	35°
New Rock Fill	19	--	40°
Existing Granular Fill	21	--	32°
Alluvium	15	10	n/a
Silty Clay (below north embankment)	18	--	26° to 28°
Clayey Silt (toe of north slope)	18	20	n/a
Clayey Silt (below south embankment)	18	--	30°
Clayey Silt (toe of south slope)	18	20	n/a
Silt	19	--	30°
Silty Sand to Sandy Silt	19	--	30°
Gravelly Sand	21	--	35°

* Groundwater table assumed to be at Elevation 284 m.

5.8.1.3 Results of Analysis

For the south abutment, there will be sufficient space between the abutment face and the shoreline to construct a 3H:1V slope. Based on this slope geometry, assuming drained conditions and assuming no significant grade raise, a factor of safety of greater than 1.3 is obtained for the south abutment front slope, as shown on Figure 1. The west side slope of the south abutment has a factor of safety greater than 1.3 for a slope geometry of 2H:1V as shown on Figure 2. Therefore, no special mitigation measures are required for the south approach embankment side slopes and front slope. The front slope geometry of 3H:1V should be smoothly transitioned into the side slope geometry of 2H:1V.

For the north abutment, there will be sufficient space between the abutment face and the shoreline to construct a 3H:1V slope. Based on this slope geometry, assuming drained conditions and assuming no significant grade raise, a factor of safety of greater than 1.3 is obtained for the north abutment front slope, as shown on Figure 3.

The north embankment existing west side slope geometry varies between 1.6H:1V and 2H:1V. These side slopes extend towards the water as the west shoreline extends to the north in this area. Embankment side slopes adjacent to the water and constructed at 2H:1V will have a factor of safety of greater than 1.3 as shown on Figure 4. However, the existing northwest side slope geometry of 1.6H:1V adjacent to the water has a factor of safety of less than 1.3, as shown on

Figure 5. Depending on the final roadway and slope geometry, a 2H:1V side slope (i.e. projected back from the toe of the slope) may be feasible. If such a slope is not possible, then ground improvement would be required to mitigate potential stability issues in this area to increase the factor of safety to that normally adopted by MTO (i.e. FoS ≥ 1.3). The details of the slope/roadway geometry are still to be developed and confirmed by the designer. These mitigation measures would be required over a limited area of the side slope, extending from the abutment to about 15 m beyond the abutment.

5.8.1.4 Mitigation of Stability (Northwest Slope)

In order to achieve a factor of safety greater than 1.3 for the north embankment slope, where the existing geometry is as steep as 1.6H:1V, consideration could be given to several mitigation alternatives. These measures could consist of sub-excavation of soft materials at the toe of the slope in combination with overall slope flattening (toe berms) and /or placement of rock fill, a pile supported retaining wall, a geogrid reinforced slope, or the use of lightweight fill to reduce embankment loading. The advantages, disadvantages, relative costs and risks/consequences are compared and ranked in Table 5. Based on our comparison of the alternatives and the results of the slope stability analysis presented below, we recommend that sub-excavation of the soft alluvium be carried out at the toe of the slope and replaced with rock fill, in conjunction with replacing a portion of the slope with rock fill and keeping the same slope geometry of 1.6H:1V. Since rock fill is not readily available on this project, the designer will have to consider a potential rock borrow source if the rock fill alternative is used in the design.

Since the slope extends to the water, any modifications to the slope geometry will impact the water channel. All of the mitigation alternatives discussed below will impact the water channel except the pile supported retaining wall and possibly the geogrid reinforced embankment.

Our recommended alternative to achieve a factor of safety greater than 1.3 involves sub-excavation of the soft material at the toe of the existing slope and excavating a portion of the existing granular fill embankment. The sub-excavated material should be replaced with rock fill and the excavated portion of the slope should be reconstructed with rock fill, as shown on Figures 6 and 7. If the current slope geometry of 1.6H:1V is desired, then a permanent toe berm, 6 m wide and 1 m high (above the existing water level), would also be required. If a slope geometry of 2H:1V is permissible, then the toe berm would not be required; however, the sub-excavation would still be required as shown on Figure 7. Sub-excavation of very soft to soft alluvium and clayey silt materials at the toe of slope should be carried out to Elevation 281 m.

It should be noted that normally rock fill embankments are constructed at 1.25H:1V. In this case, sub-excavation of the soft materials at the toe of the slope would still be required to obtain a factor of safety greater than 1.3, although to a slightly lesser extent into the lake as compared with the 1.6H:1V and 2H:1V rock fill side slope alternatives.

Alternatively, consideration could be given to an approximately 15 m long pile-supported retaining wall to support the road embankment, extending back from the abutment wall to the location where a full 2H:1V side slope presently exists or where there is space available to allow for construction for such a slope geometry. Assuming a fill slope of 3H:1V in front of the retaining wall, a factor of safety greater than 1.3 would be obtained for a wall height of about 5 m (similar geometry to Figure 3). Recommendations with respect to piling and backfilling will be the same as for the bridge abutments and are given elsewhere in this report. This wall/embankment configuration would likely have minimal impact to the water channel but would be expensive.

If a retaining wall is not practical or economical, then consideration could be given to a geogrid reinforced slope constructed to the current slope geometry of 1.6H:1V. A geogrid reinforced embankment would result in a factor of safety of greater than 1.3. Such a design would be proprietary. This alternative would require re-construction of the existing embankment using granular fill to incorporate the geogrids. The existing embankment slope would have to be cut back to a width of at least 0.8H to accommodate the reinforcing strips. Some excavation below the water level may also be required.

Alternatively, consideration could be given to the use of lightweight fill such as ultra lightweight slag or expanded polystyrene (EPS) to construct a portion of the northwest embankment. This would reduce the loading and, therefore, the existing west side slope geometry of 1.6H:1V would result in a factor of safety of greater than 1.3. However, the cost of using slag fill or EPS fill is typically an order of magnitude greater than other options and EPS could only be used above the water level. It should be noted both EPS and slag fills can only be used above the water line.

5.8.2 Settlement

Given that there is no significant grade raise or widening proposed in this area, and given that the firm to very stiff clayey silt to silty clay soils at the abutments are overconsolidated, consolidation settlement is expected to be minimal. Any such settlement is expected to occur during construction.

For the minor widening or slope re-construction, granular earth fill should be used to be consistent with the existing embankment material. If cohesive earth fill (i.e. fill containing more than 20% passing the No. 200 sieve) is used for the re-construction of the embankments, the settlement could be up to about 15 mm and this settlement would occur after construction. The existing embankment fill removed as a result of cutting back the abutments by 10 m should be suitable for re-use in embankment re-construction. However, the appropriate geotechnical testing would have to be carried out during construction to confirm this.

If the rock fill configuration described in the stability mitigation alternative is chosen, some settlement of the rock fill will occur and will be differential relative to the remaining embankment. The total magnitude of settlement of the rock fill, properly placed, compacted/chinked and keyed into the existing embankment should be less than 15 mm.

The embankment widening/re-construction should be constructed using acceptable earth fill properly placed and compacted in accordance with SP206S03.

5.9 Subgrade Preparation and Embankment Construction

The existing embankments are to be widened or reconfigured slightly. Where this occurs, the topsoil/vegetation shall be removed from the existing slopes and the subgrade surface along the area of expanded toe before additional fill is placed. 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, where possible, all subgrade soils should be proof-rolled prior to placement of new fill.

Although the embankment height is up to 10 m at this site, the effective embankment heights are less than 6 m and therefore do not require a mid-height berm (in accordance with Northeastern Region Directives).

Due to the limited amount of slope widening/reconfiguration, we assume that earth fill will be used for this purpose; however, rock fill may be used in the remediation of the northwest slope. Earth fill and rock fill materials and placement should be carried out in accordance with the requirements as outlined in Special Provision SP206S03 and the newly exposed subgrade should be proof-rolled prior to fill placement. Side slopes for earth fill embankments should be no steeper than 2H:1V. Special requirements with respect to the abutment front slope geometry are given in Section 5.8.

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 slope as per OPSD 208.01.

The abutment front slopes and any side slopes adjacent to the lake require erosion protection. Erosion protection should be placed on the slopes to at least 0.5 m above the design high water level. 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 footings and pile/caisson caps must 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.

5.10 Design and Construction Considerations

5.10.1 Excavations and Groundwater Control

5.10.1.1 Abutments

It is anticipated that the excavations for the abutment pile caps will extend through compact to dense fill consisting of sand to silty sand to silt, at both abutments. Excavations for abutment pile cap construction should be above the groundwater level which was recorded at between Elevation 284.4 m and Elevation 289.0 m at the south abutment (rising towards the south) and between Elevation 283.8 m (in an open borehole during drilling) and Elevation 285.1 m at the north approach (rising towards the north). Temporary excavation side slopes through these deposits should be made at no steeper than 1.5H:1V. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities and good construction practice. The compact to dense sand to silty sand to silt fill is classified as Type 3 soil, according to the OHSA.

It should be noted that as part of the NBL widening contract, the existing foundations for the SBL structure will be removed to 0.6 m below the ground surface at the abutments and below the lake bed at the piers. Excavations made for the abutment pile caps and to remove the existing abutments could extend below the water table. Groundwater inflow into the excavations is expected to be small and it is expected that the groundwater may generally be controlled by pumping from well-filtered sumps at the base of the excavations. Surface water should be directed away from the excavations at all times.

If excavation support for protection of the existing roadway at the abutments is required at this site, then 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.

5.10.1.2 Piers

At the piers, it will be necessary to excavate below the lake level and, therefore, cofferdams will be required at these locations. It is understood that there is a special provision for cofferdams that is typically used in MTO Contracts for this purpose. The design is the responsibility of the contractor. Sheet-pile cofferdams are feasible at this site. The steel sheet piles would have to extend to sufficient depth into the clayey silt deposit to provide for water cut-off and to prevent basal heave. Alternatively, a pre-fabricated cofferdam could be constructed as discussed in Section 5.2. The cofferdam should be designed so that the disturbance to the existing foundation is minimized. NSSPs will be required to inform the contractor that the pile cap construction must be carried out in the dry; examples are included in Appendix B for reference.

Removal of the existing pier caps and piles to 0.6 m below the lake bed will involve shallow excavation into the very soft clayey silt alluvium and/or very soft to firm clayey silt. Temporary excavations side slopes within this material below the lake bed will likely form naturally between about 3H:1V to 4H:1V although some sloughing may still occur. Cofferdams may be required to allow for removal of the piles below the lake bed.

5.10.2 Obstructions

Cobbles and boulders were encountered within the compact to very dense gravelly sand deposit, typically within 2 m of the bedrock surface. Consequently, there could be difficulties installing piles or caissons at this site. An NSSP should be included in the contract document to alert the contractor to such potential construction difficulties. An example NSSP is included in Appendix B for reference.

5.10.3 Vibration Monitoring

The proposed structure foundations will be located about 7 m from the new widened NBL foundation elements in some locations. Given this (minimum) separation distance, it is not considered necessary to carry out vibration monitoring.

5.11 Closure

This report was prepared by Mr. André Bom, P.Eng. and Ms. Sarah Coyne, P.Eng., both geotechnical engineers with Golder Associates Ltd. The technical aspects were reviewed by 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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TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES
VERNON LAKE NARROWS REPLACEMENT OF SOUTHBOUND STRUCTURE
W.P 94-89-01, SITE NO. 42-018
HIGHWAY 11, HUNTSVILLE

Options	***Advantages***	***Disadvantages***	***Relative Costs***	***Risks/Consequences***
Steel H-piles driven to bedrock	• Can found piles below the scour elevation. • Similar construction to existing foundations.	• Possibility of piles "hanging up" on boulders on very dense deposits at a few locations. • Cofferdam construction required for pile cap construction in lake. Permanent steel caissons tremie backfilled through the clay deposits to provide pile cap construction in the dry.	• Typical pile cost = $200/m	• Minimal disturbance to existing foundations
Caissons socketted into bedrock	• Can found caissons below the scour elevation. • Reduced number of deep elements compared to piles. • Possible elimination of pile caps and therefore cofferdams.	• Temporary liners would be required for groundwater control. • Concrete for caissons would have to be placed by tremie methods below the water level. • May require specialized construction techniques to remove/penetrate cobbles and boulders. • May be difficulty in socketting caissons into strong to very strong gneiss bedrock.	• Typical caisson cost = $4,900/m (plus $95,000 mobilization)	• Potential disturbance to existing foundations

NOTES:

1. This table should be read in conjunction with Section 5.0 of the Foundation Investigation and Design Report.

Checked By: SEMC

Reviewed By: JMAC

TABLE 2
PROPOSED PILE TIP ELEVATION
VERNON LAKE NARROWS REPLACEMENT OF SOUTHBOUND STRUCTURE
W.P 94-89-01, SITE NO. 42-018
HIGHWAY 11, HUNTSVILLE

Foundation Element	*Side of Foundation Element*	*Relevant Borehole*	*Ground Surface/Lake Bed Elevation (m)*	*Surface of Very Dense Gravelly Sand (m)*	*Approximate Surface of Bedrock/Design Pile Tip Elevation (m)*	*Approximate Pile Length(2) (m)*
South Abutment	East	06-17	294.0	279.5(3)	273.5(4)	14.5
	West	06-19	287.8	274.0	273.5	14.5
Pier #1	East	06-16	281.5	270.5	268.0	13.5
	West	06-20	282.3	272.0	268.0(5)	14.3
Pier #2	East	06-15	279.7	266.0	263.0(6)	16.7
	West	06-21	278.9	264.5	263.0	15.9
Pier #3	East	06-14	281.3	--	261.5	19.8
	West	06-22	281.7	--	260.5	21.2
Pier #4	East	06-13	281.8	267.0	266.0	15.8
	West	06-23	282.4	267.0	266.5	15.9
North Abutment	East	06-12	293.0	271.5	271.0	18.0
	West	06-24	292.9	--	270.5	18.5

NOTES:

1. This table should be read in conjunction with Section 5.4 of the Foundation Investigation and Design Report.
2. Approximate pile length below the underside of pile cap, assumed to be at about Elev. 289 m N. Abutment and Elev. 288 S. Abutment; approximate pile length below the lake bed at the piers.
3. Based on difficult drilling in BH06-17.
4. Based on bedrock surface confirmed by coring at BH06-19, located 14 m west of abutment.
5. Based on bedrock surface confirmed by coring at BH06-16.
6. Based on bedrock surface confirmed by coring at BH06-21.

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TABLE 3
PROPOSED BASE OF CASING ELEVATION
VERNON LAKE NARROWS REPLACEMENT OF SOUTHBOUND STRUCTURE
W.P 94-89-01, SITE NO. 42-018
HIGHWAY 11, HUNTSVILLE

Foundation Element	***Relevant Borehole***	***Proposed Base of Casing Elevation (m)***
Pier #1	06-16 & 06-20	278.5
Pier #2	06-15 & 06-21	271.5
Pier #3	06-14 & 06-22	270.0
Pier #4	06-13 & 06-23	273.0

NOTES:

1. This table should be read in conjunction with Section 5.4 of the Foundation Investigation and Design Report.

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TABLE 4
PARAMETERS FOR HORIZONTAL SUBGRADE REACTION
VERNON LAKE NARROWS REPLACEMENT OF SOUTHBOUND STRUCTURE
W.P 94-89-01, SITE NO. 42-018,
HIGHWAY 11, HUNTSVILLE

Foundation Element	*Relevant Borehole*	*Soil Unit*	*Elevation (m)*		*n_h (MPa/m)*	*s_u (kPa)*
			East Side	West Side		
South Abutment	E: 06-17 W: 06-19	Loose to dense sand (fill)	Ground surface to 286.3	n/a	1.8	--
		Firm to stiff clayey silt to silty clay	286.3 to 282.4	Ground surface to 282.0	--	60
		Loose to compact silt	282.4 to 279.4	282.0 to 279.2	1.3	--
		Compact gravelly sand/silty sand	279.4 to 276.8	279.2 to 273.5	4.4	--
Pier #1	E: 06-16 W: 06-20	Very soft to firm clayey silt	Lake bed to 279.5	Lake bed to 279.9	--	40
		Loose to compact silt	279.5 to 275.6	279.9 to 276.9	1.3	--
		Loose to very dense silty sand	n/a	276.9 to 273.6	4.4	--
		Compact to very dense gravelly sand/silty sand	275.6 to 268.0	273.6 to 268.5	11	--
Pier #2	E: 06-15 W: 06-21	Very soft to stiff clayey silt	Lake bed to 272.4	Lake bed to 272.5	--	30
		Compact silt	272.4 to 270.8	272.5 to 270.8	4.4	--
		Compact silty sand	270.8 to 269.3	270.8 to 267.9	4.4	--
		Compact to very dense gravelly sand/silty sand	269.3 to 264.7	267.9 to 263.1	11	--
Pier #3	E: 06-14 W: 06-22	Very soft clayey silt (alluvium)	Lake bed to 279.1	Lake bed to 278.6	--	10
		Very soft to firm clayey silt	279.1 to 271.0	278.6 to 271.0	--	20
		Loose to compact silt	271.0 to 269.3	271.0 to 267.2	1.3	--
		Compact silty sand	269.3 to 265.4	267.2 to 263.7	4.4	--
		Compact to dense gravelly sand/silty sand	265.4 to 261.5	263.7 to 260.4	4.4	--
Pier #4	E: 06-13 W: 06-23	Very soft clayey silt (alluvium)	Lake bed to 278.3	Lake bed to 279.1	--	10
		Very soft to firm clayey silt	278.3 to 273.9	279.1 to 273.9	--	20
		Compact to loose silt	273.9 to 269.3	273.9 to 270.8	1.3	--
		Compact sandy silt to gravelly sand/silty sand	269.3 to 266.0	270.8 to 266.8	4.4	--
North Abutment	E: 06-12 W: 06-24	Very loose to dense sand (fill)	Ground surface to 282.6	Ground surface to 282.3	1.8	--
		Compact silt	282.6 to 281.1	n/a	4.4	--
		Firm silty clay to clayey silt	281.1 to 278.1	282.3 to 276.2	--	30
		Loose to compact sandy silt	278.1 to 272.9	276.2 to 273.1	1.3	--
		Compact gravelly sand/silty sand	272.9 to 271.4	273.1 to 270.8	4.4	--

NOTES:

1. This table should be read in conjunction with Section 5.4.2 of the Foundation Investigation and Design Report.

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TABLE 5
EVALUATION OF STABILITY MITIGATION ALTERNATIVES – NORTHWEST SLOPE
VERNON LAKE NARROWS REPLACEMENT OF SOUTHBOUND STRUCTURE
W.P 94-89-01, SITE NO. 42-018
HIGHWAY 11, HUNTSVILLE

Options	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Final Slope 1.6H:1V Rock Fill Sub-excavation Toe Berm	1	• Keeps same overall slope angle in this area • Removal of soft material at toe of slope increases factor of safety	• Will impact water channel • Will temporarily impact roadway • Excavation of existing fill required • Disposal of soft material (alluvium and clayey silt) required	• Cost of removing existing fill and replacing with offsite rock fill, plus fill for backfill for sub-excavation and toe berm	• Potential for minor differential settlement in final road alignment
Final Slope 2H:1V Rock Fill Sub-excavation	2	• Removal of soft material at toe of slope increases factor of safety	• Greater encroachment length on water channel • Larger rock fill volume required • Will temporarily impact roadway • Final shoreline will be altered • Excavation of existing fill required • Disposal of soft material (alluvium and clayey silt) required	• Cost of removing existing fill and replacing with offsite rock fill, plus fill for backfill of sub-excavation • Increased costs due to larger volume of rock fill required	• Potential for minor differential settlement in final road alignment
RSS Wall Founded on Existing Embankment Material (5 m high)	3	• Minimal impact to shoreline/water channel • Requires nominal bearing resistance • Accommodates some differential settlement	• Increased cost due to reinforcing strips and facing • Requires additional excavation of roadway embankment to accommodate reinforcing strips	• Less expensive than pile-supported retaining wall but more expensive than rock fill embankment	• Low risk • Differential settlement may be accommodated by the RSS wall
Final Slope 1.6H:1V Geogrid Reinforced Embankment	4	• Keeps same overall slope • Sub-excavation of soft alluvium and clayey silt probably not required	• Could impact water channel and require sub-excavation of soft materials, depending on final design • Excavation of existing embankment fill material required, may be able to re-use fill for slope backfill • Preparing a suitable sub-base may be difficult given the proximity to the water channel	• Cost of wall design and construction	• Potentially low risk but slope deformation could occur due to consolidation of the soft materials at the toe of the reinforced slope

Options	***Rank***	***Advantages***	***Disadvantages***	***Relative Costs***	***Risks/Consequences***
Pile Supported Retaining Wall	5	• No impact to roadway or shoreline	• Increased cost due to pile driving, however, abutment will be piled and would save on mobilization cost	• Cost of piling and wall design/construction	• Low risk
Lightweight Fill (slag, EPS)	6	• Minimal impact to water course • Reduction in fill loading increases factor of safety	• Cost is high • Concrete slab may be required • EPS can only be used above the water level	• Typically an order of magnitude higher than other options	• Low risk

NOTES:

1. This table should be read in conjunction with Section 5.0 of the Foundation Investigation and Design Report.

Checked By: SEMC

Reviewed By: JMAC

LIST OF ABBREVIATIONS

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

1. 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

Piezocone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

1. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. 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 δ
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

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

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

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

Description and Notes

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

Abbreviations

B - Bedding
FO - Foliation/Schistosity
CL - Cleavage
SH - Shear Plane/Zone
VN - Vein
F - Fault
CO - Contact
J - Joint
FR - Fracture
MF - Mechanical Fracture
II - Parallel To
⊥ - Perpendicular To

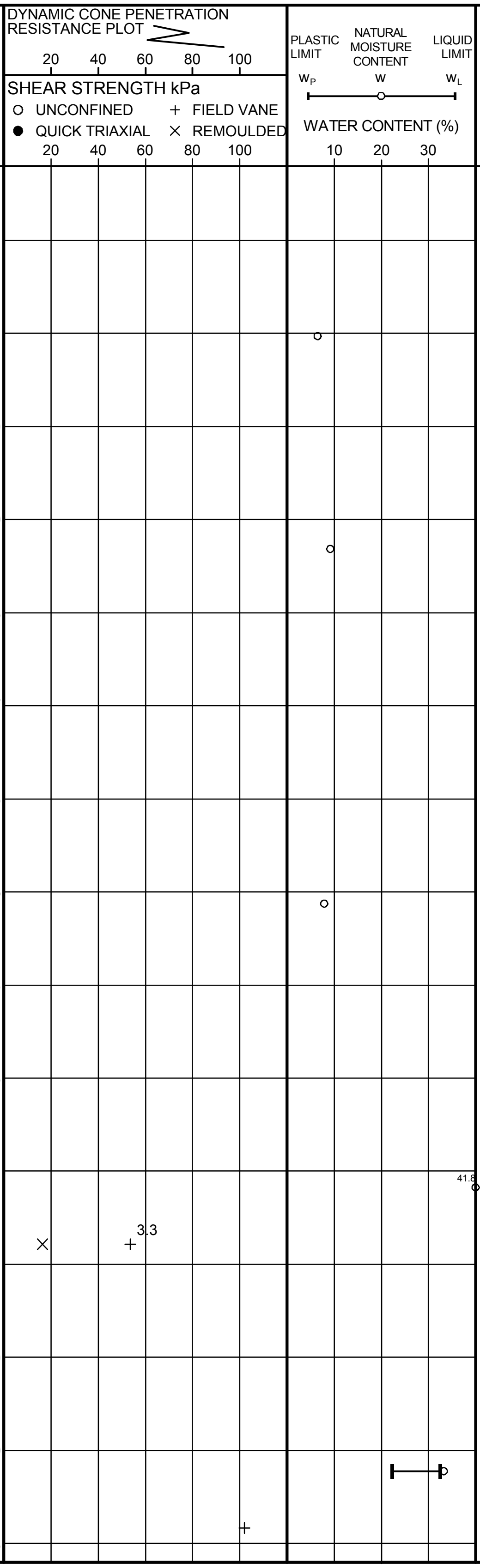
P - Polished
S - Slickensided
SM - Smooth
R - Ridged/Rough
ST - Stepped
PL - Planar
FL - Flexured
UE - Uneven
W - Wavy
C - Curved

RECORD OF BOREHOLE No BH06-11

1 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-00
LOCATION N 5020592; E325164
ORIGINATED BY EHS
DIST 52 HWY 11
BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers
COMPILED BY AB
DATUM Geodetic
DATE 06/26/06
CHECKED BY SEP

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
292.8	GROUND SURFACE						
0.0	ASPHALT						
	Sand and gravel (FILL) Brown						
0.4	Sand to silty sand, trace gravel (FILL) Very dense to compact Brown Moist	1	SS	78	292		
		2	SS	49	291		
		3	SS	35	290		
		4	SS	20	289		0 72 (28)
		5	SS	18	288		
		6	SS	26	287		
		7	SS	26	286		
		8	SS	13	285		
284.0 8.8	CLAYEY SILT, trace sand, occasional sand seams, varved Firm to very stiff Grey Moist to wet	9	SS	10	284 283		
		10	SS	7	282 281		
		11	TO	PH	280	18.6	
		12	SS	9	279		
277.9					278		



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

MIS-MTO 001 06-1191-001 SOIL WP 94-89-01.GPJ GAL-MISS.GDT 7/12/07 RN

RECORD OF BOREHOLE No BH06-11

2 OF 2 **METRIC**

PROJECT 06-1191-001

W.P. 5189-05-00

DIST 52 HWY 11

DATUM Geodetic

LOCATION N 5020592; E325164

BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers

DATE 06/26/06

ORIGINATED BY EHS

COMPILED BY AB

CHECKED BY SEP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
14.9	Sandy SILT, trace clay, occasional clay seams Loose Grey Wet		13	SS	8						
277.0 15.8	End of Borehole Notes: 1. Water level measured in piezometer at 7.7m depth (Elev. 285.1m) on August 24, 2006.						277				

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

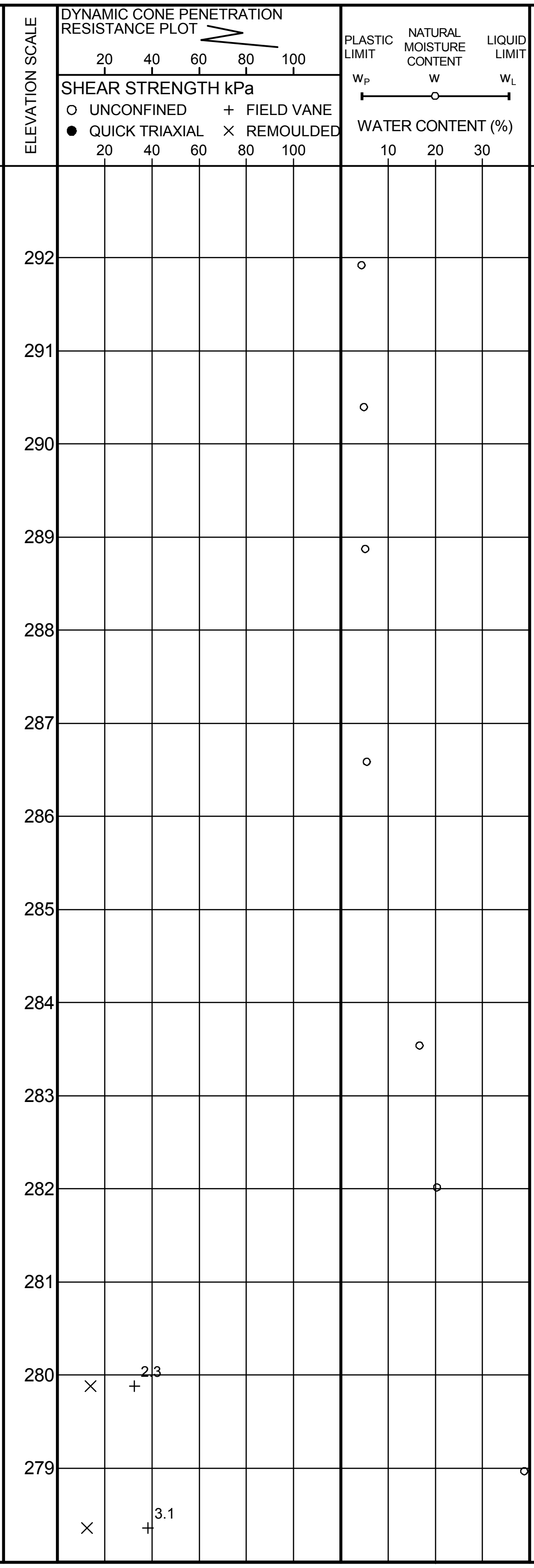
RECORD OF BOREHOLE No BH06-12

1 OF 2 **METRIC**

PROJECT	06-1191-001		
W.P.	5189-05-00	LOCATION	N 5020580; E325169
DIST	52 HWY 11	BOREHOLE TYPE	Power Auger, 108mm ID Hollow Stem Augers
DATUM	Geodetic	DATE	07/05/06

ORIGINATED BY EHS
COMPILED BY AB
CHECKED BY SEP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
293.0	GROUND SURFACE							
0.0	ASPHALT							
	Sand and gravel (FILL) Brown							
0.4	Sand to silty sand, trace to some gravel (FILL) Compact to loose Brown Moist to wet		1	SS	22			
			2	SS	29			
			3	SS	13			
			4	SS	9			
			5	SS	5			
			6	SS	6			
			7	SS	8			
			8	SS	10			
			9	SS	8			
						▽		
282.6 10.4	SILT, some sand, trace clay, trace organics Compact Grey Wet		10	SS	20			
281.1 11.9	CLAYEY SILT to SILTY CLAY, trace sand Very soft to firm Grey Wet		11	SS	3			
			12	SS	5			
278.1								



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+3, ×3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

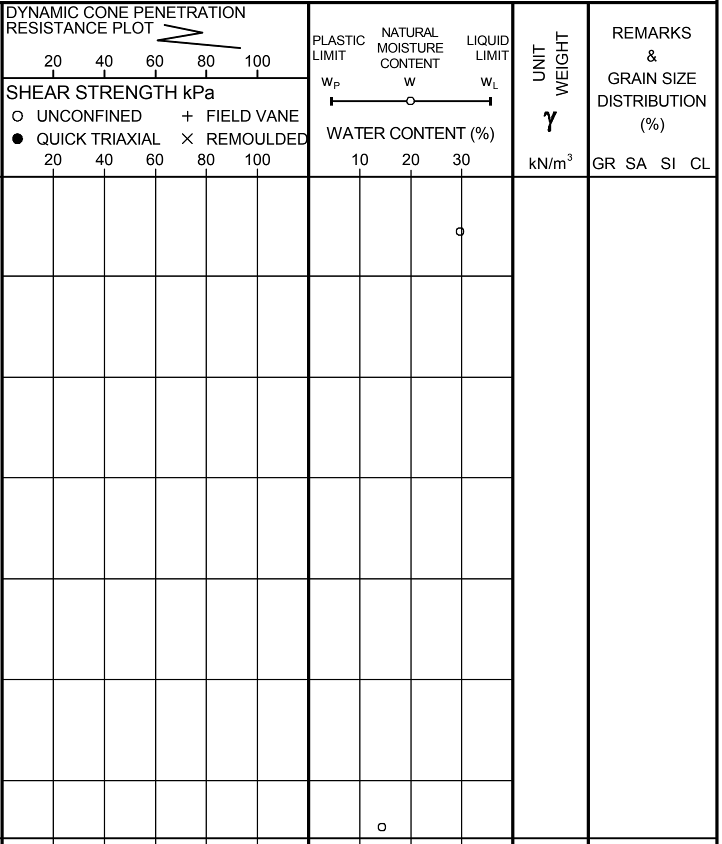
RECORD OF BOREHOLE No BH06-12

2 OF 2 **METRIC**

PROJECT 06-1191-001			
W.P. 5189-05-00	LOCATION N 5020580; E325169		ORIGINATED BY EHS
DIST 52	HWY 11	BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers	COMPILED BY AB
DATUM Geodetic	DATE 07/05/06		CHECKED BY SEP

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
14.9	Sandy SILT, trace clay Loose Grey Wet	13		6		
						277
	Difficult augering below 16.6m depth.					276
						275
		14	SS	9		
						274
272.9 20.1	Gravelly SAND to Silty SAND, containing cobbles Compact Brown Wet					273
						272
271.4	Augers grinding below 20.1m depth.	15	SS	17/.18		
21.6	End of Borehole Split Spoon and Auger Refusal					
	Notes: 1. Water level at 10.4m depth (Elev. 282.6m) upon completion of drilling.					



+3, ×3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH06-13

1 OF 2 **METRIC**

PROJECT	06-1191-001		
W.P.	5189-05-00	LOCATION	N 5020539; E325163
DIST 52	HWY 11	BOREHOLE TYPE	NW Casing, Wash Boring
DATUM	Geodetic	DATE	08/03/06
ORIGINATED BY	EHS	COMPILED BY	AB
CHECKED BY	SEP		

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
284.1	WATER SURFACE
0.0	WATER
281.8 2.3	CLAYEY SILT to SILTY CLAY, some organics (ALLUVIUM) Very soft Brown to black Wet
278.3 5.8	CLAYEY SILT, trace sand, trace organics, layered Very soft to firm Grey Wet
273.9 10.2	SILT, trace sand, trace to some clay Loose to compact Grey Wet
269.3 14.8	

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	WH	
2	SS	WH	
3	SS	WH	
4	SS	WH	
5	SS	1	
6	SS	WH	0 0 70 30
7	SS	6	
8	SS	11	
9	SS	14	
10	SS	10	

GROUND WATER CONDITIONS

ELEVATION SCALE

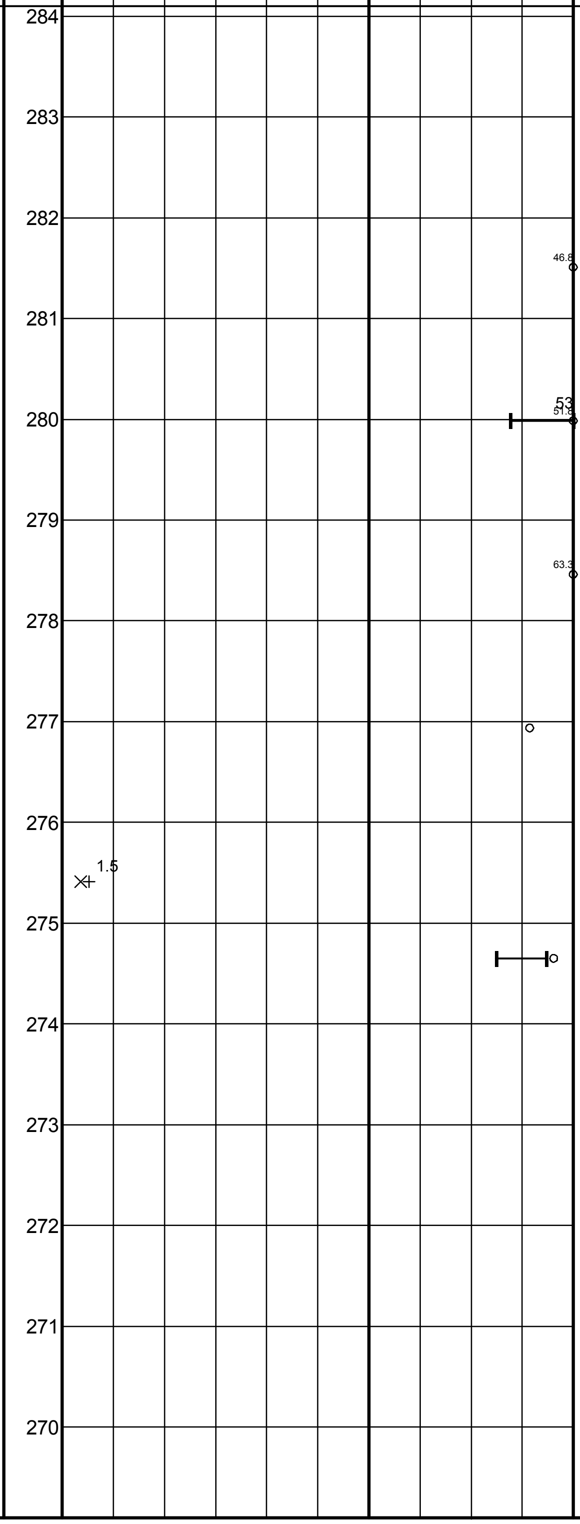
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³



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

MIS-MTO 001 06-1191-001 SOIL WP 94-89-01.GPJ GAL-MISS.GDT 7/12/07 RN

Golder Associates

RECORD OF BOREHOLE No BH06-13

2 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-00
LOCATION N 5020539; E325163
ORIGINATED BY EHS
DIST 52 HWY 11
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY AB
DATUM Geodetic
DATE 08/03/06
CHECKED BY SEP

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Gravelly SAND to Silty SAND, containing cobbles and boulders Compact Grey Wet						269				
			11	SS	12						
							268				
	Spoon bouncing at 16.8m depth		12	SS	50/0.03						
							267				
266.0											
18.1	GNEISS (BEDROCK)		1	RC	REC 100%		266				RQD = 68%
			2	RC	REC 73%		265				RQD = 37%
	Bedrock cored from 18.1m to 24.9m depth						264				
			3	RC	REC 70%		263				RQD = 32%
			4	RC	REC 82%						RQD = 0%
							262				
			5	RC	REC 100%						RQD = 57%
	For coring details see Record of Drillhole BH06-13						261				
			6	RC	REC 100%		260				RQD = 98%
259.2											
24.9	End of Borehole										

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

PROJECT: 06-1191-001

RECORD OF DRILLHOLE: BH06-13

SHEET 1 OF 1

LOCATION: N 5020539; E325163

DRILLING DATE: 08/03/06

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

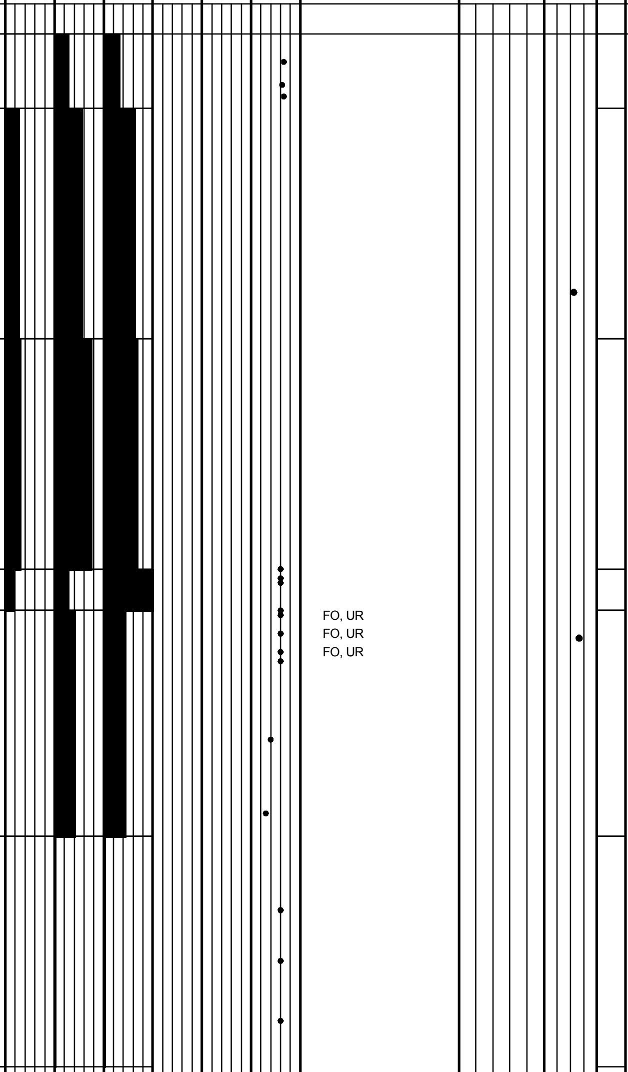
DRILL RIG:

DRILLING CONTRACTOR:

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.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.
		Refer to previous page	266.0	
	NQ Coring	GNEISS Fine to medium grained Fresh to slightly weathered Strong to very strong Grey Joint information not recorded between 18.6m depth and 21.6m depth Silt and cobbles between 18.8m and 19.3m depth	18.1	1
19				2
20				
21		Silt seam between 20.8m and 21.3m depth		3
				4
22		All joints and foliations are planar and smooth except where noted.		5
23				
24				6
25		End of Drillhole	259.2 24.9	
26				
27				
28				



NOTES WATER LEVELS INSTRUMENTATION

SYMBOLIC LOG

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: EHS

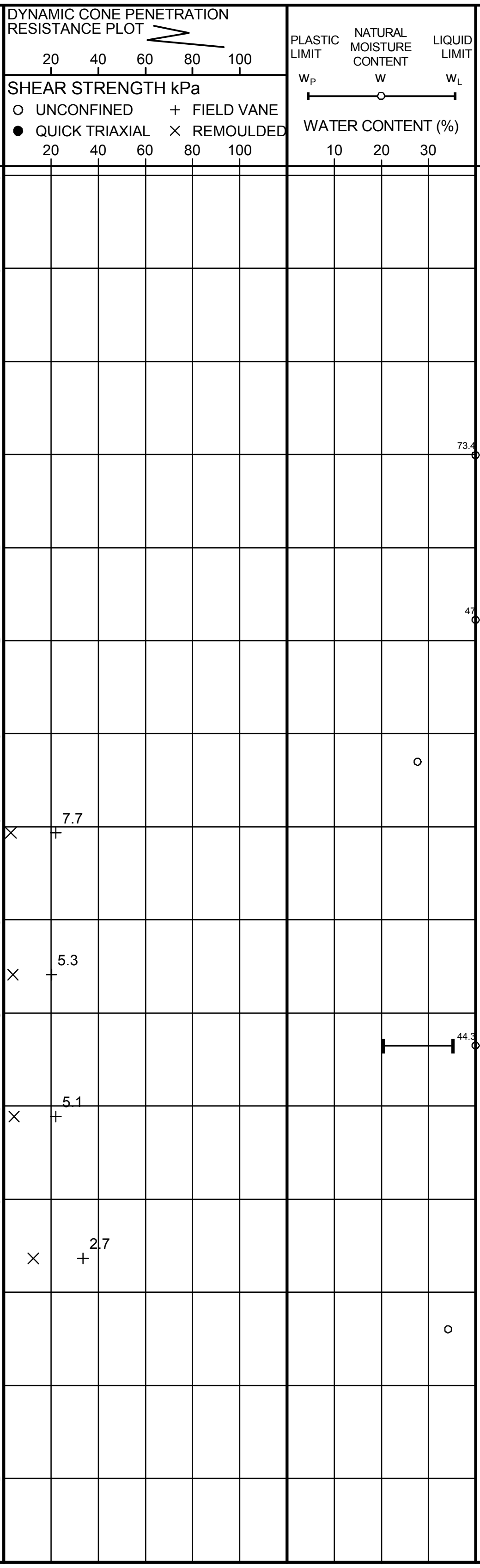
CHECKED: SEP

RECORD OF BOREHOLE No BH06-14

1 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-00
LOCATION N 5020491; E325150
ORIGINATED BY EHS
DIST 52 HWY 11
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY AB
DATUM Geodetic
DATE 08/10/06
CHECKED BY SEP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
284.1	WATER SURFACE				
0.0	WATER				
281.3 2.8	CLAYEY SILT, some organics (ALLUVIUM) Very soft to soft Brown Wet	1	SS	WH	
		2	SS	WH	
		3	SS	2	
279.1 5.0	CLAYEY SILT, trace sand, layered Very soft to firm Grey Wet	4	SS	WH	
		5	SS	1	0 3 74 23
		6	SS	WH	
		7	SS	WH	
		8	SS	WH	
		9	SS	8	
271.0 13.1	SILT, trace to some clay Compact Grey Wet	10	SS	11	
269.3 14.8					



UNIT WEIGHT γ kN/m³

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

MIS-MTO 001 06-1191-001 SOIL WP 94-89-01.GPJ GAL-MISS.GDT 7/12/07 RN

RECORD OF BOREHOLE No BH06-14

2 OF 2 **METRIC**

PROJECT 06-1191-001

W.P. 5189-05-00

LOCATION N 5020491; E325150

ORIGINATED BY EHS

DIST 52 HWY 11

BOREHOLE TYPE NW Casing, Wash Boring

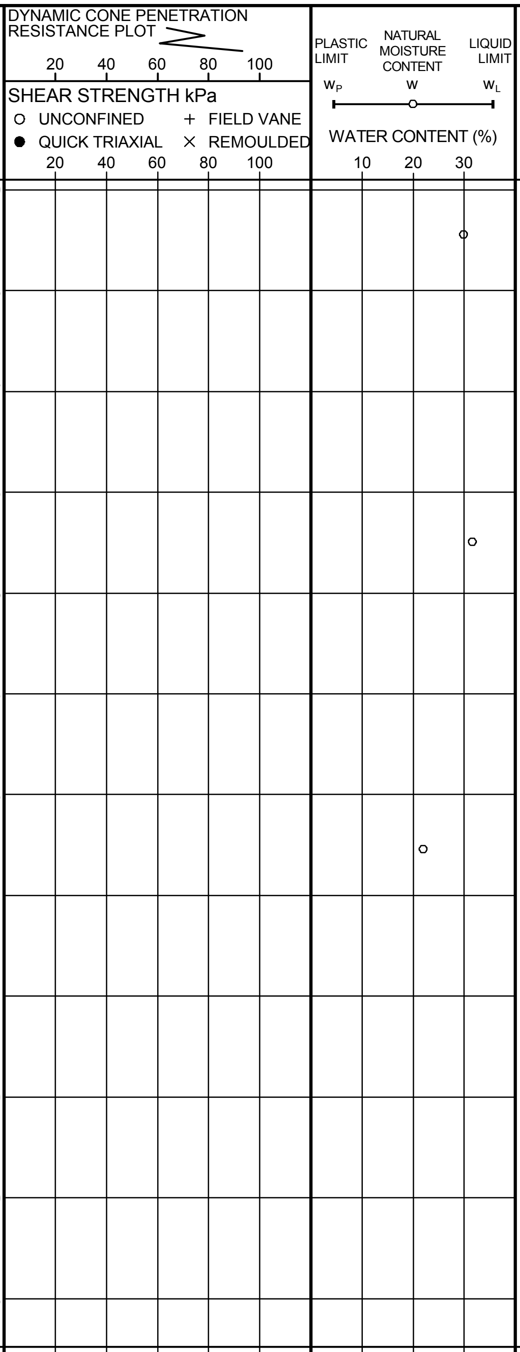
COMPILED BY AB

DATUM Geodetic

DATE 08/10/06

CHECKED BY SEP

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---					
	Silty SAND, trace clay Compact Grey Wet	11	SS	18	269	
					268	
		12	SS	23	267	
					266	
265.4		13	SS	15		
18.7	Gravelly SAND to Silty SAND, containing cobbles and boulders Compact to dense Grey Wet				265	
		14	SS	20	264	
					263	
		15	SS	36	262	
261.5						
22.6	GNEISS (BEDROCK)	1	RC	REC 81%	261	RQD = 44%
	Bedrock cored from 22.6m to 26.6m depth	2	RC	REC 99%	260	RQD = 97%
					259	
	For coring details see Record of Drillhole BH06-14	3	RC	REC 100%	258	RQD = 100%
257.5 26.6	End of Borehole					



UNIT WEIGHT γ kN/m³

GROUND WATER CONDITIONS

STRAT PLOT

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

PROJECT: 06-1191-001

RECORD OF DRILLHOLE: BH06-14

SHEET 1 OF 1

LOCATION: N 5020491; E325150

DRILLING DATE: 08/10/06

DATUM: Geodetic

DRILL RIG:

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR:

JN - Joint
FLT - Fault
SHR - Shear
VN - Vein
CJ - Conjugate

BD - Bedding
FO - Foliation
CO - Contact
OR - Orthogonal
CL - Cleavage

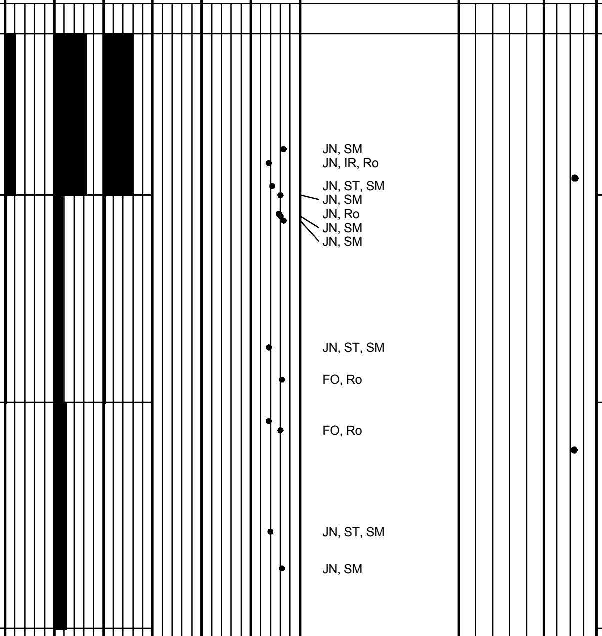
PL - Planar
CU - Curved
UN - Undulating
ST - Stepped
IR - Irregular

PO - Polished
K - Slickensided
SM - Smooth
Ro - Rough
MB - Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN FLUSH	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page		261.5				
23 24 25 26	NQ Coring	GNEISS Fine to medium grained Fresh to slightly weathered Strong to very strong Grey Sand seam between 22.9m and 23.1m depth		22.6 257.5	1 2 3			UCS=44MPa
27 28 29 30 31 32		End of Drillhole		26.6				



TYPE AND SURFACE DESCRIPTION

JN, SM
JN, IR, Ro
JN, ST, SM
JN, SM
JN, Ro
JN, SM
JN, SM
JN, ST, SM
FO, Ro
FO, Ro
JN, ST, SM
JN, SM

DEPTH SCALE
1 : 50

Golder Associates

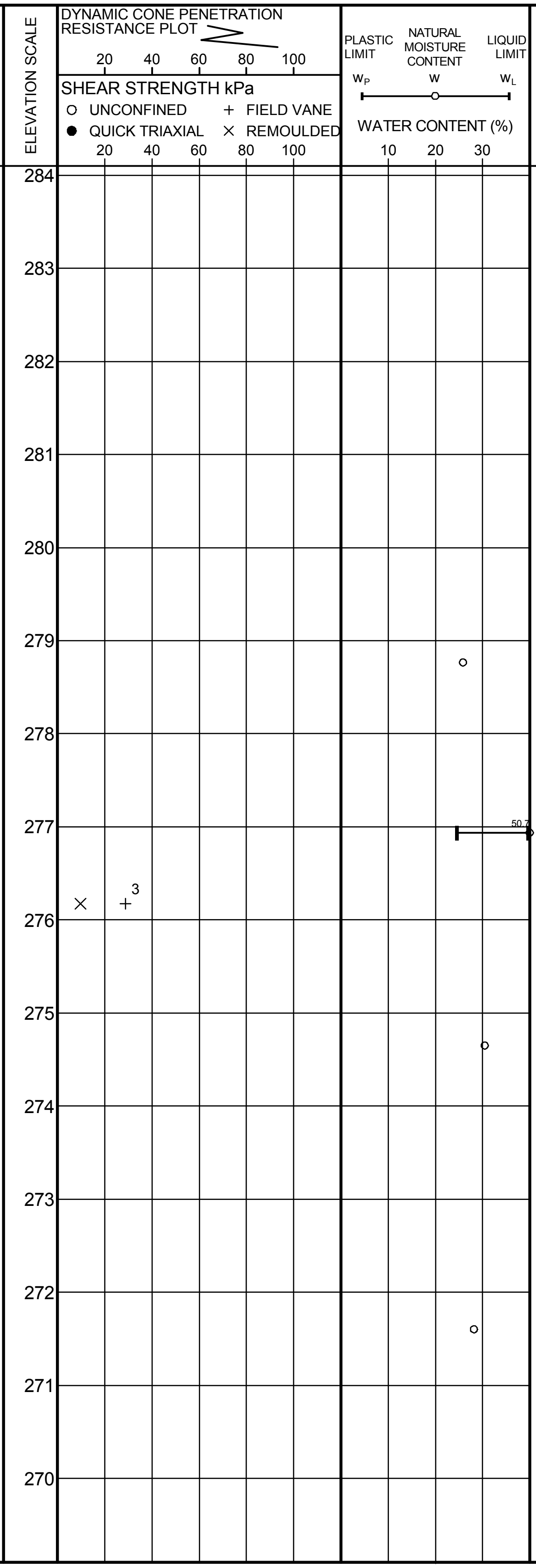
LOGGED: EHS
CHECKED: SEP

RECORD OF BOREHOLE No BH06-15

1 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-00
LOCATION N 5020442; E325136
ORIGINATED BY EHS
DIST 52 HWY 11
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY AB
DATUM Geodetic
DATE 08/11/06
CHECKED BY SEP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
284.1	WATER SURFACE									
0.0	WATER									
279.7 4.6	Sandy SILT, trace organics (ALLUVIUM)									
	CLAYEY SILT to SILTY CLAY, trace sand, layered Very soft to stiff Grey Wet	1	SS	WH						
		2	SS	2						
		3	SS	WH						
		4	SS	WH			0	1	49	50
		5	SS	WH						
		6	SS	7						
		7	SS	11						
272.4 11.7	SILT, trace sand, trace clay Compact Grey Wet	8	SS	17						
270.8 13.3	Silty SAND, trace clay Compact Grey Wet	9	SS	13						
269.3 14.8										



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+3, ×3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

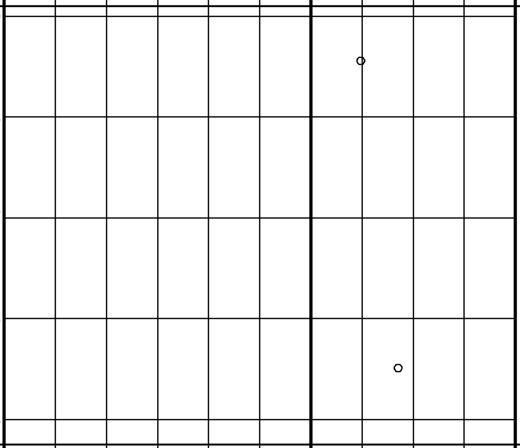
RECORD OF BOREHOLE No BH06-15

2 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-00
DIST 52 HWY 11
DATUM Geodetic
LOCATION N 5020442; E325136
BOREHOLE TYPE NW Casing, Wash Boring
DATE 08/11/06
ORIGINATED BY EHS
COMPILED BY AB
CHECKED BY SEP

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Gravelly SAND to Silty SAND, containing cobbles and boulders Compact to very dense Grey Wet						269				
			10	SS	22						
							268				
			11	SS	30		267				
							266				
			12	SS	57						
264.7							265				
19.4	End of Borehole										



Notes:
1. Casing shoe stuck in borehole at 19.4m depth, no further borehole advancement possible.

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

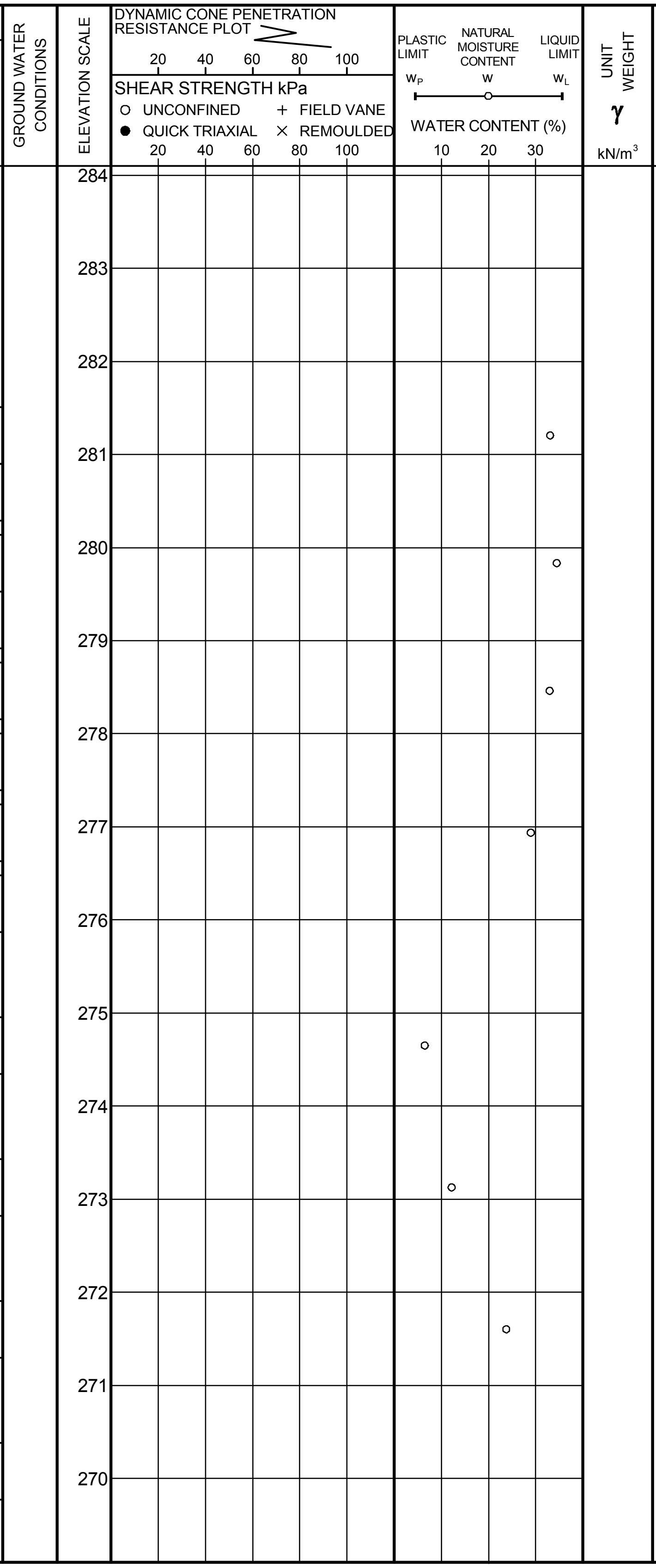
$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No BH06-16

1 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-00
DIST 52 HWY 11
DATUM Geodetic
LOCATION N 5020395; E325123
BOREHOLE TYPE NW Casing, Wash Boring
DATE 08/12/06
ORIGINATED BY EHS
COMPILED BY AB
CHECKED BY SEP

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
284.1	WATER SURFACE							
0.0	WATER							
281.5	Silty SAND (ALLUVIUM) Very loose Grey to black Wet	1	SS	2				
281.2 2.9	CLAYEY SILT, trace sand Firm Grey Wet	2	SS	6				
		3	SS	6				
279.5 4.6	SILT, trace clay, trace sand Loose to compact Grey Wet	4	SS	10				
		5	SS	8				
		6	SS	6				
		7	SS	14				
		8	SS	8				
275.6 8.5	Gravelly SAND to Silty SAND Compact to very dense Grey Wet							
		9	SS	19				
		10	SS	47	17	60	(23)	
		11	SS	34				
	Cobbles below 13.7m depth	12	SS	102				



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

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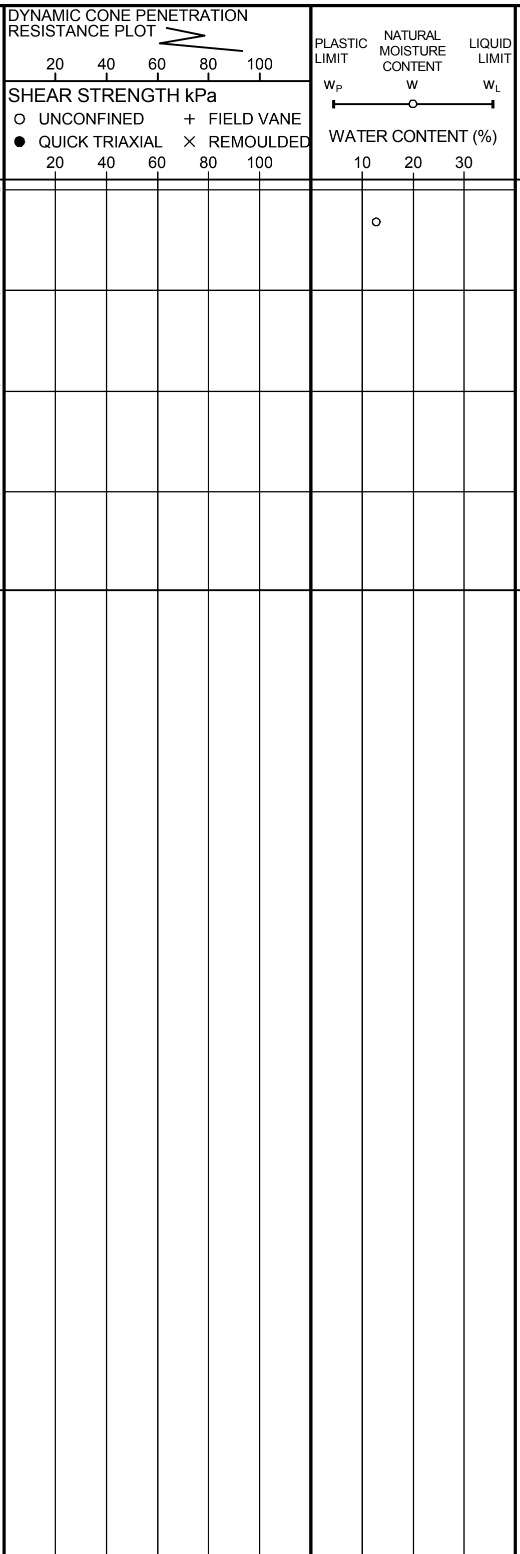
RECORD OF BOREHOLE No BH06-16

2 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-00
LOCATION N 5020395; E325123
ORIGINATED BY EHS
DIST 52 HWY 11
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY AB
DATUM Geodetic
DATE 08/12/06
CHECKED BY SEP



ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---								
	Gravelly SAND to Silty SAND Compact to very dense Grey Wet Spoon bouncing at 15.6m depth		13	SS	119/0.20		269		
268.0 16.1	GNEISS (BEDROCK) Bedrock cored from 16.1m to 19.1m depth		1	RC	REC 100%		268 267		RQD = 81%
	For coring details see Record of Drillhole BH06-16		2	RC	REC 100%		266		RQD = 100%
265.0 19.1	End of Borehole								

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

PROJECT: 06-1191-001

RECORD OF DRILLHOLE: BH06-16

SHEET 1 OF 1

LOCATION: N 5020395; E325123

DRILLING DATE: 08/12/06

DATUM: Geodetic

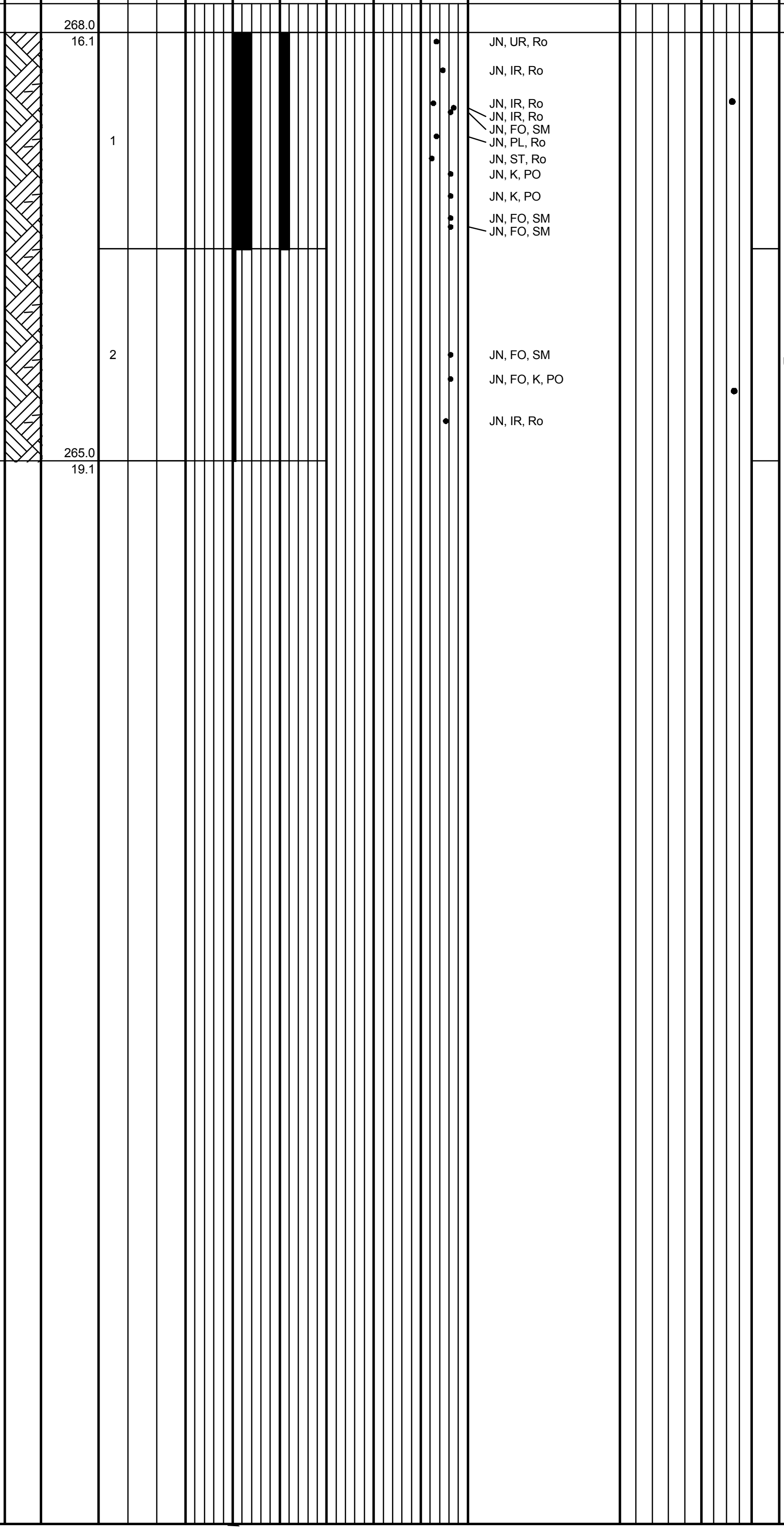
INCLINATION: -90° AZIMUTH: ---

DRILL RIG:

DRILLING CONTRACTOR:

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page		268.0					
17	NQ Coring	GNEISS Fine to medium grained Fresh to slightly weathered Strong to very strong Grey		16.1	1			JN, UR, Ro JN, IR, Ro JN, IR, Ro JN, IR, Ro JN, FO, SM JN, PL, Ro JN, ST, Ro JN, K, PO JN, K, PO JN, FO, SM JN, FO, SM	
18 19				265.0	2			JN, FO, SM JN, FO, K, PO JN, IR, Ro	UCS=86MPa
20 21 22 23 24 25 26		End of Drillhole		19.1					



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: EHS

CHECKED: SEP

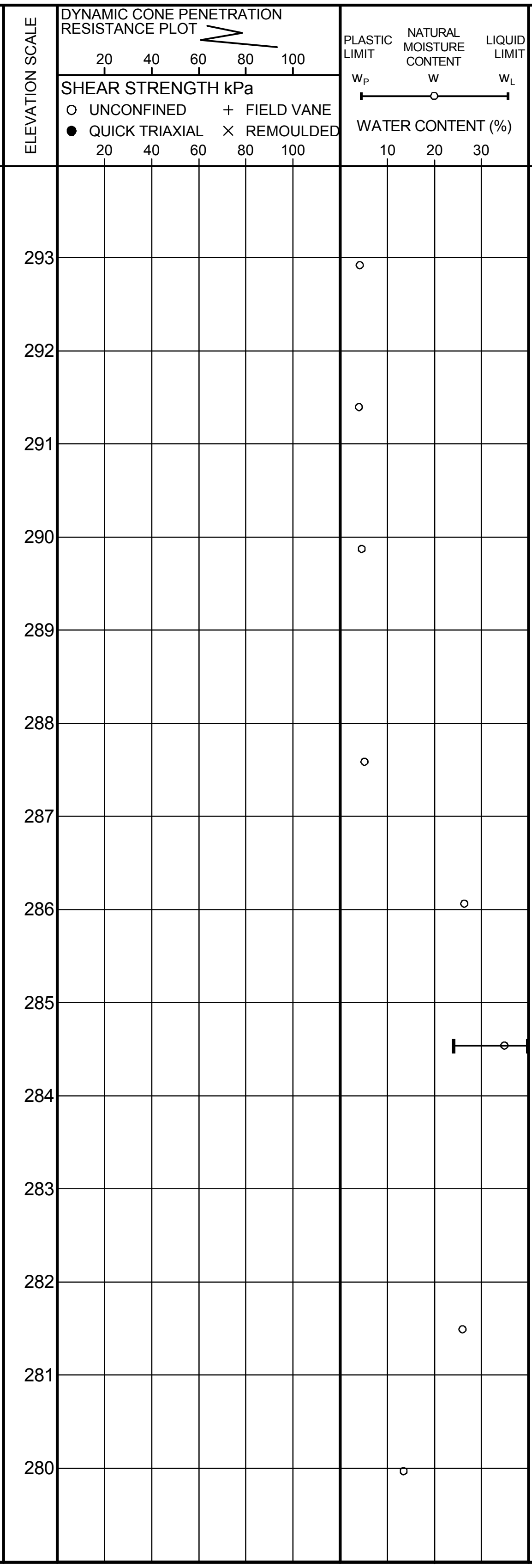
Golder Associates

RECORD OF BOREHOLE No BH06-17

1 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-00
LOCATION N 5020355; E325106
ORIGINATED BY EHS
DIST 52 HWY 11
BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers
COMPILED BY AB
DATUM Geodetic
DATE 07/06/06
CHECKED BY SEP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR	SA	SI	CL
294.0	GROUND SURFACE									
0.0 0.1	ASPHALT Sand and gravel (FILL) Brown									
293.5 0.5	Sand, trace to some gravel, trace silt, occasional cobbles (FILL) Loose to dense Brown Moist	1	SS	32						
		2	SS	38						
		3	SS	23			3	91	(6)	
		4	SS	7						
		5	SS	10						
		6	SS	9						
	Augers grinding at 5.3m depth									
		7	SS	6						
286.3 7.7	CLAYEY SILT to SILTY CLAY, trace sand, trace organics Stiff Brown Moist to wet	8	SS	10						
	Becoming grey at 9.1m depth	9	SS	10						
		10	SS	9						
282.4 11.6	SILT, trace clay Loose to compact Grey Wet	11	SS	8	∇					
		12	SS	12						
279.4 14.6	Augers grinding at 14.6m depth									



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

RECORD OF BOREHOLE No BH06-17

2 OF 2 **METRIC**

PROJECT 06-1191-001			
W.P. 5189-05-00	LOCATION N 5020355; E325106		ORIGINATED BY EHS
DIST 52	HWY 11	BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers	COMPILED BY AB
DATUM Geodetic	DATE 07/06/06		CHECKED BY SEP

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Gravelly SAND to Silty SAND, containing cobbles Compact Grey Wet		13	SS	22						
							278				
	Augers grinding at 16.5m depth		14	AS	-						
							277				
276.8 17.2	End of Borehole Auger Refusal										

Notes:
1. Water level at 12.8m depth (Elev. 281.2m) upon completion of drilling.

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

RECORD OF BOREHOLE No BH06-18

1 OF 2 **METRIC**

PROJECT	06-1191-001		
W.P.	5189-05-00	LOCATION	N 5020323; E325083
DIST	52 HWY 11	BOREHOLE TYPE	Power Auger, 108mm ID Hollow Stem Augers
DATUM	Geodetic	DATE	06/21/06

ORIGINATED BY EHS
COMPILED BY AB
CHECKED BY SEP

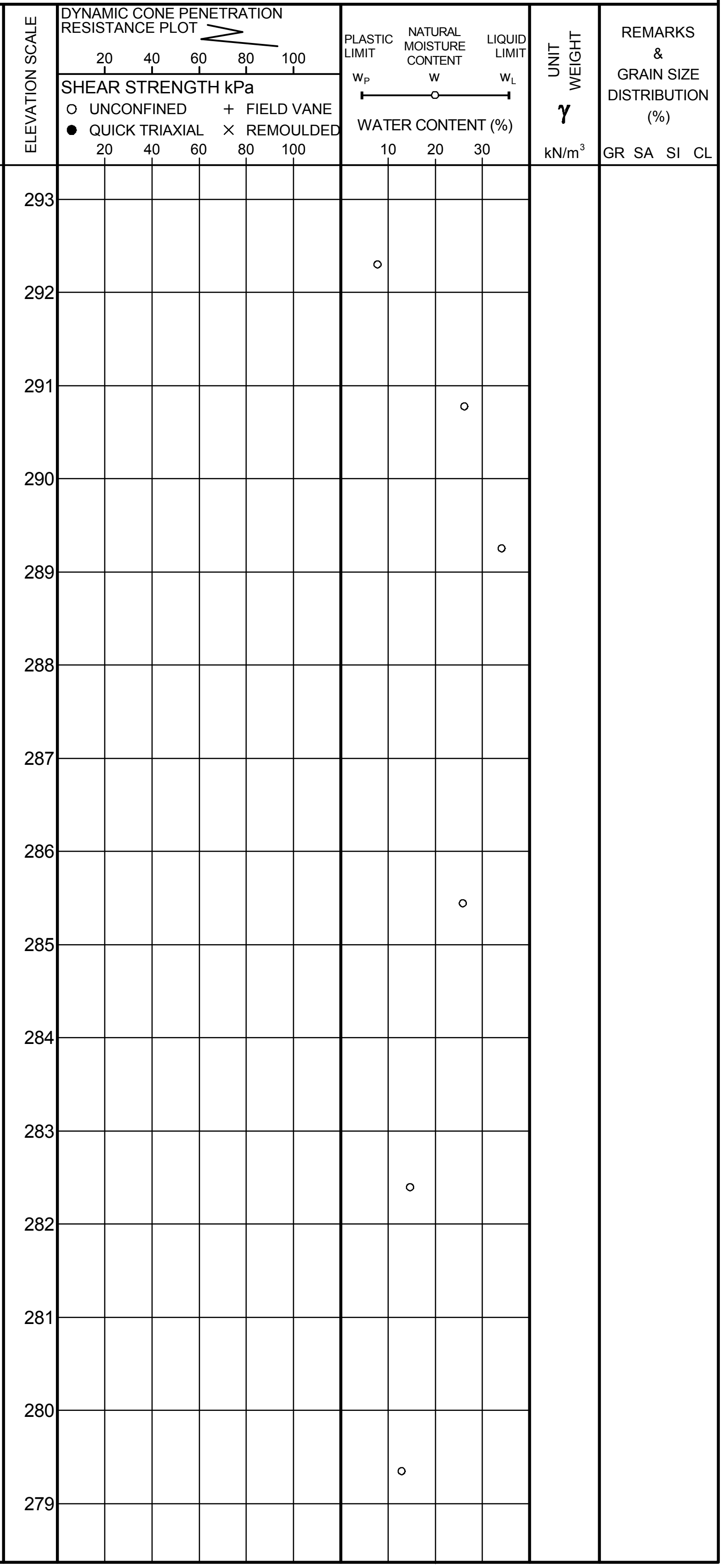
SOIL PROFILE

ELEV DEPTH	DESCRIPTION	STRAT PLOT
293.4	GROUND SURFACE	
0.0	Sand, some gravel (FILL)	▨
	ASPHALT	■
0.2	Sand, trace to some gravel, trace silt, occasional cobbles (FILL) Loose Brown Moist	▨
291.8 1.5	Silt, some clay, trace sand (FILL) Loose Brown and grey Moist	▨
291.1	Organic SILT, trace sand Compact Dark brown Moist	
290.7 2.7	SILT, trace to some clay, trace sand Compact Grey Moist	
290.2 3.2	CLAYEY SILT, trace sand Very stiff Brown Moist Becoming grey below 4.6m depth	
287.0 6.4	SILT, trace to some clay, frequent sand seams, occasional clay seams Compact Brown Wet	
283.7 9.7	Gravelly SAND to Silty SAND, containing cobbles Compact to very dense Brown Wet Augers grinding below 10.0m Difficult advancing augers below 14.0m depth	

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
1	SS	11	
2	SS	4	
3	SS	7	
4	SS	12	
5	SS	26	
6	SS	17	
7	TO	PH	▽
8	SS	15	
9	SS	11	
10	SS	16	
11	SS	12	
12	SS	25	
13	SS	40	



+³, ×³: Numbers refer to Sensitivity

○ ³% STRAIN AT FAILURE

PROJECT 06-1191-001

RECORD OF BOREHOLE No BH06-18

2 OF 2 **METRIC**

W.P. 5189-05-00

LOCATION N 5020323; E325083

ORIGINATED BY EHS

DIST 52 HWY 11

BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers

COMPILED BY AB

DATUM Geodetic

DATE 06/21/06

CHECKED BY SEP

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
277.8			14	SS	78/0.15		278				
15.5	End of Borehole Notes: 1. Heave in augers to 12.5m depth upon augering to 13.7m depth. 2. Water level at 4.6m depth (Elev. 288.8m) upon completion of drilling.										

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

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

RECORD OF BOREHOLE No BH06-19

1 OF 2 **METRIC**

PROJECT	06-1191-001		
W.P.	5189-05-00	LOCATION	N 5020365; E325082
DIST	52 HWY 11	BOREHOLE TYPE	Power Auger, 108mm ID Hollow Stem Augers
DATUM	Geodetic	DATE	06/22/06
ORIGINATED BY	EHS	COMPILED BY	AB
CHECKED BY	SEP		

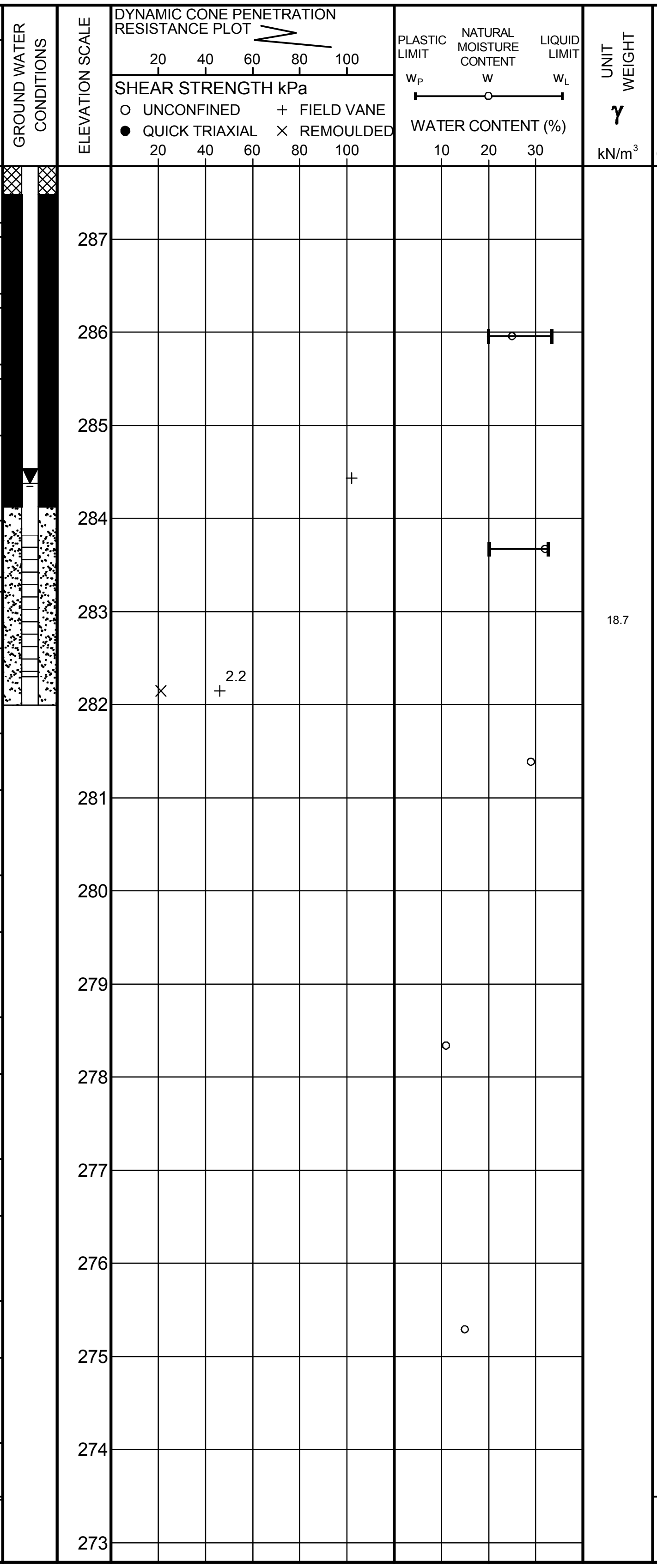
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
287.8	GROUND SURFACE
0.0 0.1	Sandy TOPSOIL Brown Moist
	CLAYEY SILT, trace sand, occasional sand seams, varved Firm to stiff Brown and grey Moist Becoming grey below 3.8m depth Becoming wet below 4.9m depth
282.0 5.8	SILT, trace to some clay, occasional sand seams Loose to compact Grey Wet
279.2 8.6	Gravelly SAND to Silty SAND, occasional cobbles Compact to very dense Brown Wet
273.5 14.3	GNEISS (BEDROCK)

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	AS	-		
2	SS	10		
3	SS	10		
4	TO	PH		
5	SS	8		0 0 81 19
6	TO	PH	18.7	
7	SS	6		0 0 95 5
8	SS	10		
9	SS	25		
10	SS	16		
11	SS	33		
12	SS	108		
1	RC	REC 100%		RQD = 48%



+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH06-19

2 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-00
DIST 52 HWY 11
DATUM Geodetic
LOCATION N 5020365; E325082
BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers
DATE 06/22/06
ORIGINATED BY EHS
COMPILED BY AB
CHECKED BY SEP

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	GNEISS (BEDROCK) Bedrock cored from 14.3m to 17.4m depth For coring details see Record of Drillhole BH06-19		2	RC	REC 100%		272				RQD = 90%
			3	RC	REC 100%		271				RQD = 69%
270.3 17.4	End of Borehole Notes: 1. Water level in piezometer at 3.4m depth (Elev. 284.4m) on August 24, 2006.										

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

PROJECT: 06-1191-001

RECORD OF DRILLHOLE: BH06-19

SHEET 1 OF 1

LOCATION: N 5020365; E325082

DRILLING DATE: 06/22/06

DATUM: Geodetic

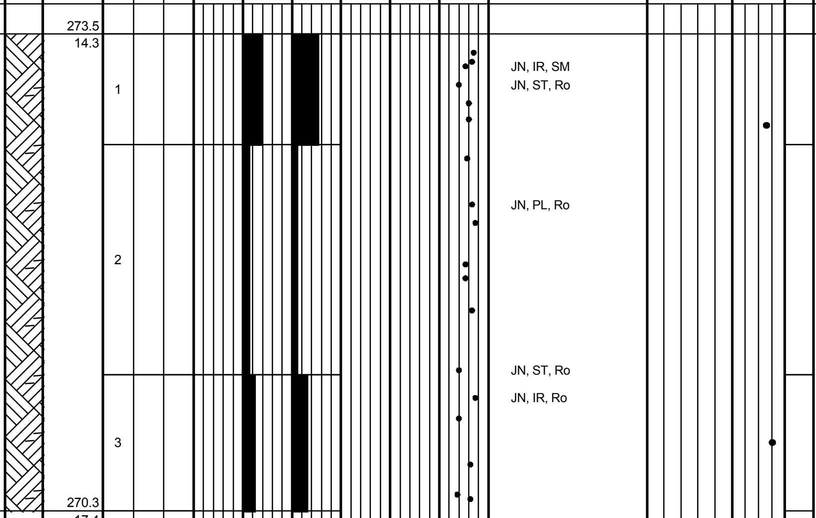
DRILL RIG:

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR:

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page				
	NQ Coring	GNEISS Fine to medium grained Fresh to slightly weathered Strong to very strong Grey	273.5 14.3	1	JN, IR, SM JN, ST, Ro	
15				2	JN, PL, Ro	
16		All joints and foliations are planar and smooth except where noted.			JN, ST, Ro	
17			270.3	3	JN, IR, Ro	
		End of Drillhole	17.4			
18						
19						
20						
21						
22						
23						
24						



DEPTH SCALE
1 : 50

Golder Associates

LOGGED: EHS
CHECKED: SEP

RECORD OF BOREHOLE No BH06-20

1 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-00
LOCATION N 5020400; E325105
ORIGINATED BY EHS
DIST 52 HWY 11
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY AB
DATUM Geodetic
DATE 08/14/06
CHECKED BY SEP

SOIL PROFILE		SAMPLES			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
284.1	WATER SURFACE								
0.0	WATER								
282.3 1.8	Sandy SILT (ALLUVIUM) Very loose Brown Wet	1	SS	WH					
281.8 2.3	CLAYEY SILT, trace sand Very soft to firm Grey Wet	2	SS	WH					
		3	SS	1					
279.9 4.2	SILT, trace sand, trace clay Loose to compact Grey Wet	4	SS	10	NP				
		5	SS	9					
		6	SS	7		0	2	91	7
276.9 7.2	Silty SAND, trace clay Loose to compact Grey Wet	7	SS	12					
		8	SS	8					
273.6 10.5	Gravelly SAND to Silty SAND, containing cobbles and boulders Compact to very dense Grey Wet	9	SS	22					
		10	SS	92					
		11	SS	140/0.23					
	Split spoon bouncing at 13.9m depth								

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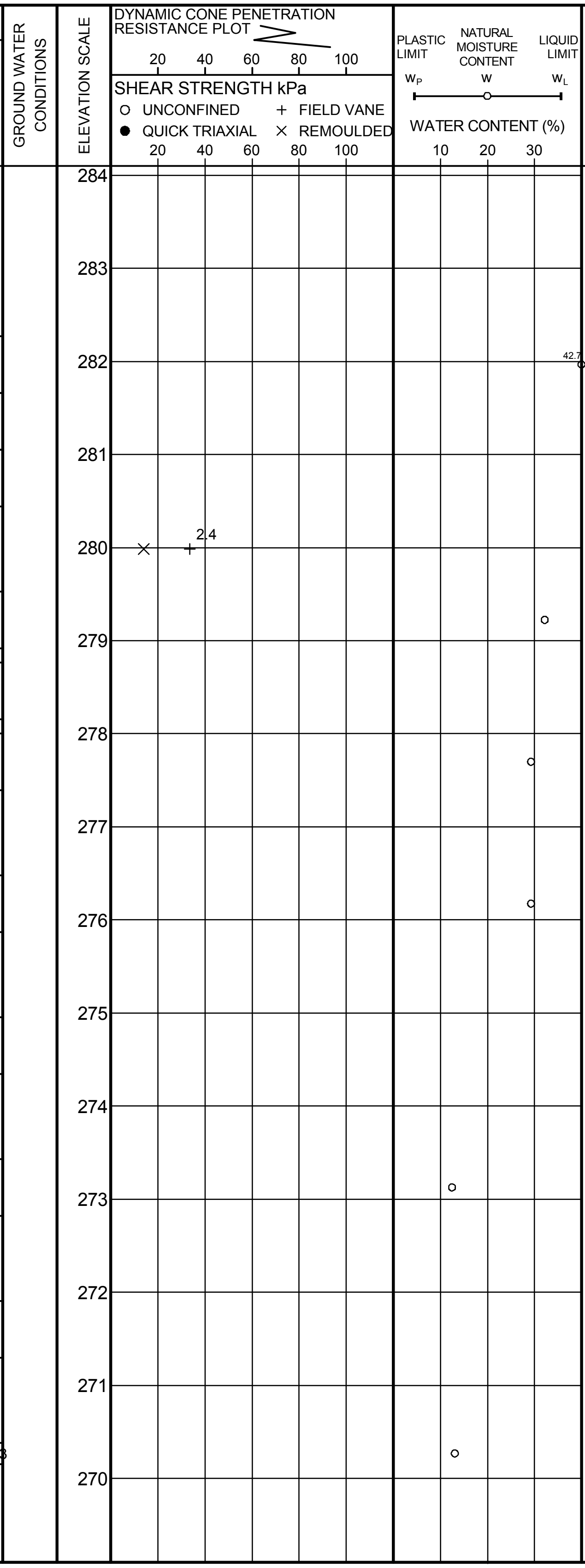


$+^3$, $\times^3$: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No BH06-20

2 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-00
DIST 52 HWY 11
DATUM Geodetic
LOCATION N 5020400; E325105
BOREHOLE TYPE NW Casing, Wash Boring
DATE 08/14/06
ORIGINATED BY EHS
COMPILED BY AB
CHECKED BY SEP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
			12	SS	125/0.15		269				
268.5 15.6	Split spoon bouncing at 15.4m depth End of Borehole										

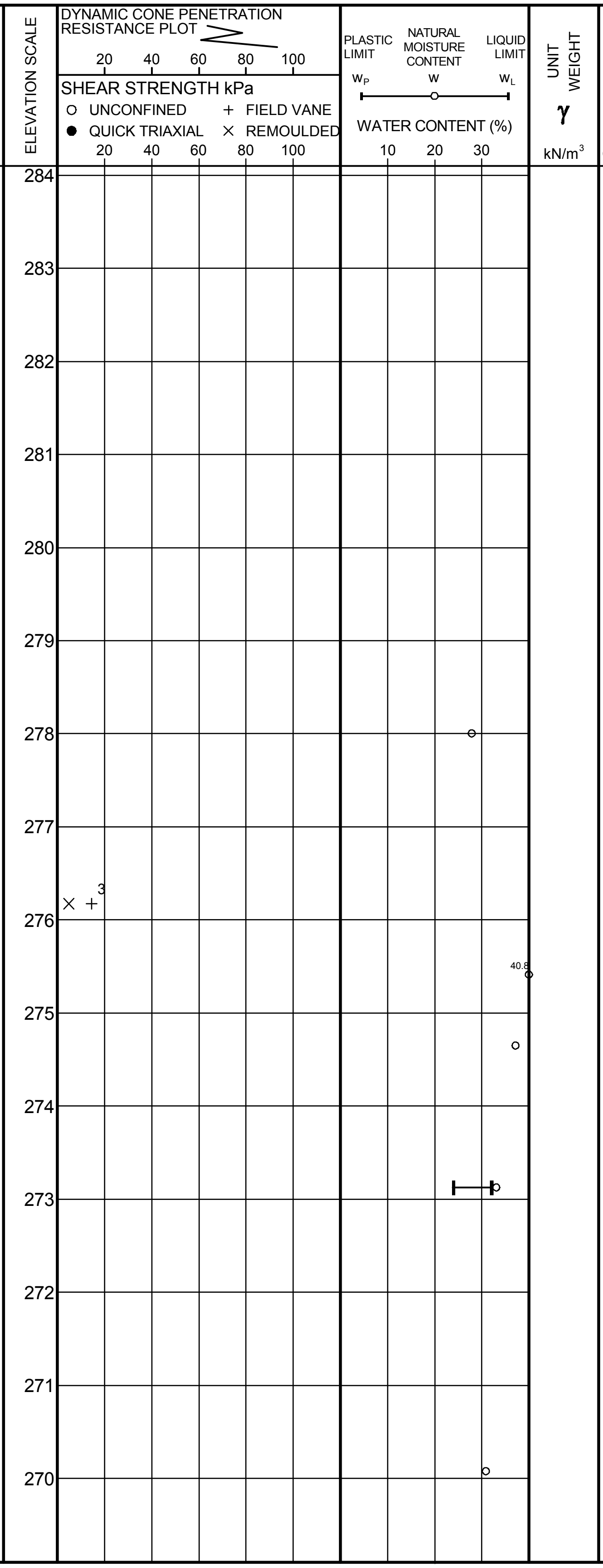
$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No BH06-21

1 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-00
DIST 52 HWY 11
DATUM Geodetic
LOCATION N 5020448; E325119
BOREHOLE TYPE NW Casing, Wash Boring
DATE 08/16/06
ORIGINATED BY EHS
COMPILED BY AB
CHECKED BY SEP

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
284.1	WATER SURFACE							
0.0	WATER							
278.9 5.3	Sandy SILT, trace organics (ALLUVIUM) Grey to brown Wet	1	SS	WH				
	CLAYEY SILT, trace sand, trace organics, layered Very soft to stiff Grey Wet	2	SS	WH				
		3	SS	1				
		4	SS	3	0	0	65	35
		5	SS	3				
		6	SS	9				
272.5 11.6	SILT, trace sand, trace clay Compact Grey Wet	7	SS	11				
270.8 13.3	Silty SAND, trace clay Compact Grey Wet	8	SS	20				



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+3, ×3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

Golder Associates

Foundation Design

RECORD OF BOREHOLE No BH06-21

2 OF 2 **METRIC**

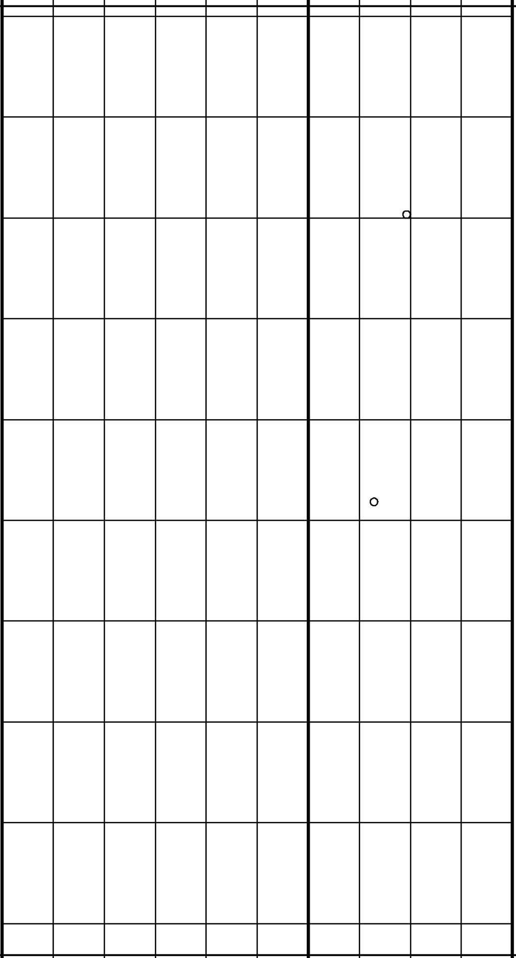
PROJECT 06-1191-001
W.P. 5189-05-00
DIST 52 HWY 11
DATUM Geodetic
LOCATION N 5020448; E325119
BOREHOLE TYPE NW Casing, Wash Boring
DATE 08/16/06
ORIGINATED BY EHS
COMPILED BY AB
CHECKED BY SEP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			
	--- CONTINUED FROM PREVIOUS PAGE ---							
	Silty SAND, trace clay Compact Grey Wet		9	SS	10		269	
267.9 16.2	Gravelly SAND to Silty SAND, containing cobbles and boulders Compact to dense Grey Wet		10	SS	16		268 267	
			11	SS	45		266 265	
	Split spoon bouncing at 20.0m depth		12	SS	102/0.20		264	
263.1 21.0	GNEISS (BEDROCK)		1	RC	REC 86%		263	RQD = 55%
	Bedrock cored from 21.0m to 24.4m depth		2	RC	REC 100%		262 261	RQD = 100%
	For coring details see Record of Drillhole BH06-21		3	RC	REC 100%		260	RQD = 100%
259.7 24.4	End of Borehole							

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
20 40 60 80 100

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

UNIT WEIGHT γ kN/m³



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

MIS-MTO 001 06-1191-001 SOIL WP 94-89-01.GPJ GAL-MISS.GDT 7/12/07 RN

PROJECT: 06-1191-001

RECORD OF DRILLHOLE: BH06-21

SHEET 1 OF 1

LOCATION: N 5020448; E325119

DRILLING DATE: 08/16/06

DATUM: Geodetic

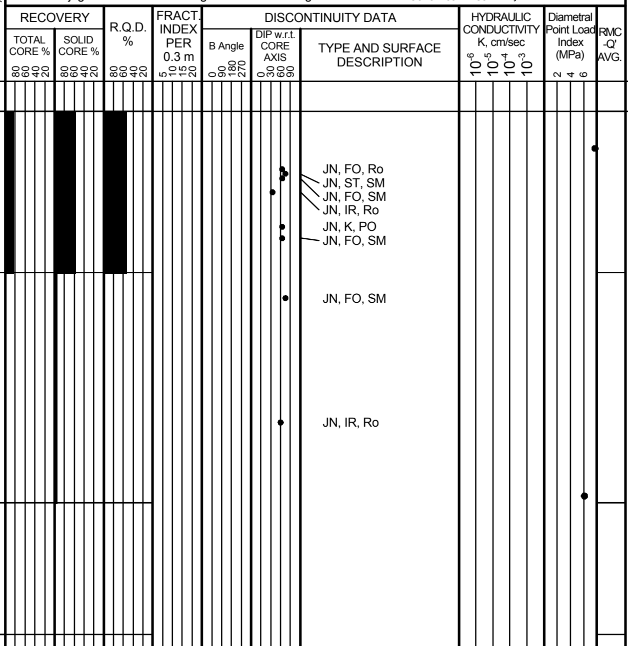
INCLINATION: -90° AZIMUTH: ---

DRILL RIG:

DRILLING CONTRACTOR:

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page		263.1				
21	NQ Coring	GNEISS Fine to medium grained Fresh to slightly weathered Strong to very strong Grey	▨	21.0	1			
22					2			
23								
24				259.7 24.4	3			
		End of Drillhole						
25								
26								
27								
28								
29								
30								



JN, FO, Ro
JN, ST, SM
JN, FO, SM
JN, IR, Ro
JN, K, PO
JN, FO, SM
JN, FO, SM
JN, IR, Ro

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: EHS
CHECKED: SEP

Foundation Design

RECORD OF BOREHOLE No BH06-22

1 OF 2 **METRIC**

PROJECT 06-1191-001			
W.P. 5189-05-00	LOCATION N 5020496; E325132		ORIGINATED BY EHS
DIST 52	HWY 11	BOREHOLE TYPE NW Casing, Wash Boring	COMPILED BY AB
DATUM Geodetic	DATE 08/09/06		CHECKED BY SEP

SOIL PROFILE

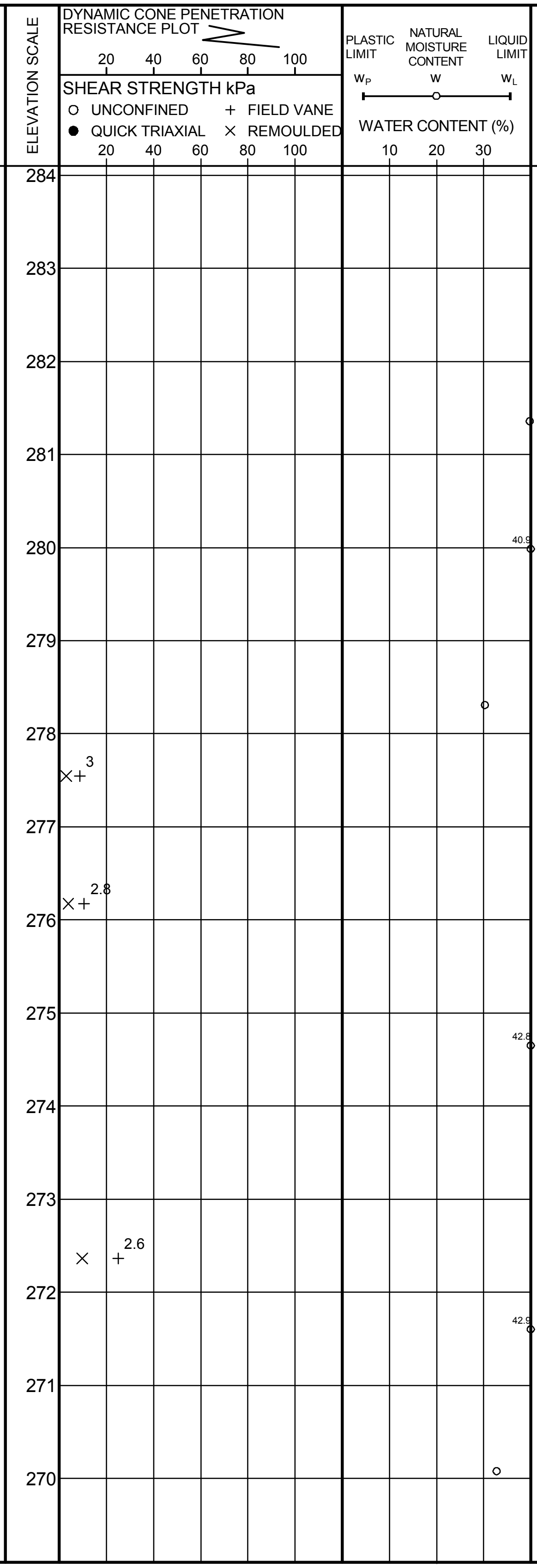
ELEV DEPTH	DESCRIPTION
284.1	WATER SURFACE
0.0	WATER
281.7 2.4	CLAYEY SILT, trace organics, trace to some sand, (ALLUVIUM) Very soft Brown to grey Wet
278.6 5.5	CLAYEY SILT, trace sand, layered Very soft to firm Grey Wet
271.0 13.1	SILT, some clay Loose to compact Grey Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	WH				
2	SS	WH				
3	SS	WH	0	13	75	12
4	SS	1				
5	SS	1				
6	SS	1				
7	SS	WH				
8	SS	WH				
9	SS	7				
10	SS	9	0	0	85	15

GROUND WATER CONDITIONS

ELEVATION SCALE



UNIT WEIGHT γ kN/m³

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

MIS-MTO 001 06-1191-001 SOIL WP 94-89-01.GPJ GAL-MISS.GDT 7/12/07 RN

RECORD OF BOREHOLE No BH06-22

2 OF 2 **METRIC**

PROJECT 06-1191-001

W.P. 5189-05-00

DIST 52 HWY 11

DATUM Geodetic

LOCATION N 5020496; E325132

BOREHOLE TYPE NW Casing, Wash Boring

DATE 08/09/06

ORIGINATED BY EHS

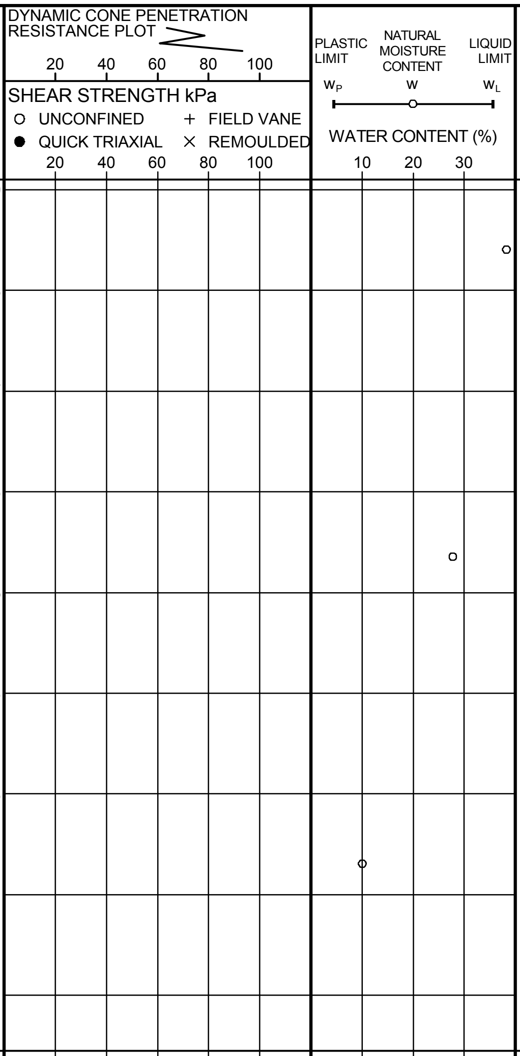
COMPILED BY AB

CHECKED BY SEP

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILT, some clay Loose to compact Grey Wet					269		
		11	SS	23				
						268		
267.2 16.9	Silty SAND Compact Grey Wet	12	SS	11		267		
						266		
		13	SS	23				
						265		
		14	SS	14		264		
263.7 20.4	Gravelly SAND to Silty SAND, containing cobbles and boulders Compact to dense Grey Wet					263		
		15	SS	40		262		
		16	SS	27		261		
260.4 23.7	End of Borehole Refusal							

Notes:
1. Casing seated into bedrock at 23.7m.



+3, ×3: Numbers refer to Sensitivity

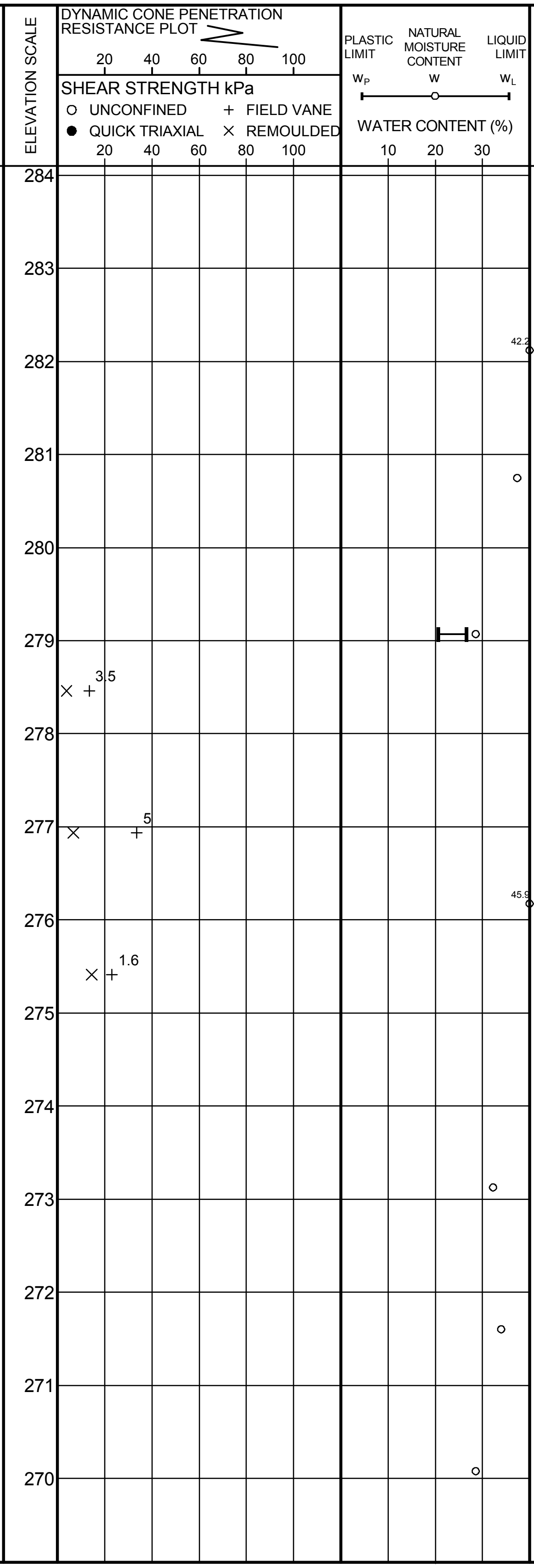
○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH06-23

1 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-00
LOCATION N 5020543; E325146
ORIGINATED BY EHS
DIST 52 HWY 11
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY AB
DATUM Geodetic
DATE 08/02/06
CHECKED BY SEP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
284.1	WATER SURFACE									
0.0	WATER									
282.4 1.7	CLAYEY SILT, trace organics, layered (ALLUVIUM) Very soft Brown to grey Wet	1	SS	WH						
		2	SS	WH						
		3	SS	WH						
		4	SS	1						
		5	SS	1						
279.1 5.0	CLAYEY SILT, trace sand with silt layers Very soft to firm Grey Wet									
		6	SS	1						
		7	SS	1						
		8	SS	7						
273.9 10.2	SILT, trace to some clay, trace sand Compact Grey Wet	9	SS	16						
		10	SS	16			0	1	88	11
270.8 13.3	Sandy SILT Compact Grey Wet	11	SS	22						



Continued Next Page

+3, ×3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

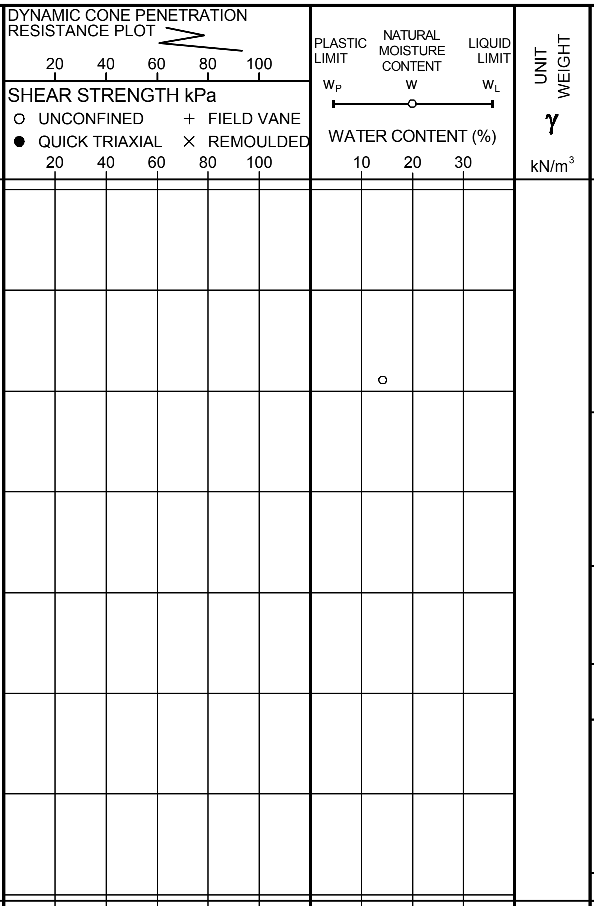
RECORD OF BOREHOLE No BH06-23

2 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-00
DIST 52 HWY 11
DATUM Geodetic
LOCATION N 5020543; E325146
BOREHOLE TYPE NW Casing, Wash Boring
DATE 08/02/06
ORIGINATED BY EHS
COMPILED BY AB
CHECKED BY SEP

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Sandy SILT Compact Grey Wet		12	SS	10		269	
267.9 16.2	Gravelly SAND to Silty SAND, containing cobbles Compact Grey Wet						268	
			13	SS	12/0.23		267	
266.8 17.3	Split spoon bouncing at 17.3m depth GNEISS (BEDROCK) Bedrock cored from 17.3m to 22.2m depth		1	RC	REC 100%		266	RQD = 95%
			2	RC	REC 49%		265	RQD = 18%
	For coring details see Record of Drillhole BH06-23		3	RC	REC 100%		264	RQD = 81%
			4	RC	REC 53%		263	RQD = 42%
261.9			5	RC	REC 100%		262	RQD = 73%
22.2	End of Borehole							



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

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

PROJECT: 06-1191-001

RECORD OF DRILLHOLE: BH06-23

SHEET 1 OF 1

LOCATION: N 5020543; E325146

DRILLING DATE: 08/02/06

DATUM: Geodetic

DRILL RIG:

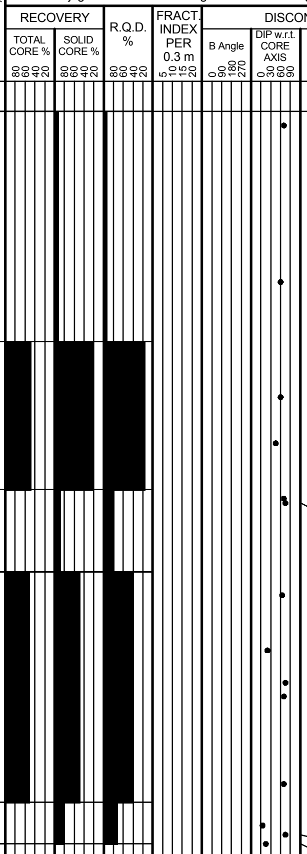
INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR:

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR / % RETURN	FLUSH	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page		266.8						
	NQ Coring	GNEISS Fine to medium grained Fresh to slighly weathered Strong to very strong Grey		17.3	1				JN, IR, SM	
18									JN, Ro	
19					2				BR BR	
20					3				JN, SM JN, Ro	
					4				JN, SM	
21									JN, IR, Ro JN, SM JN, Ro	
		Sand seam from 21.2m to 21.9m depth							JN, SM	
22				261.9 22.2	5				JN, IR, Ro JN, SM JN, IR, Ro	
		End of Drillhole		22.2						
23										
24										
25										
26										
27										



HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3} | Diametral Point Load Index (MPa) 2 4 6 | RMC -Q' AVG.

DEPTH SCALE
1 : 50

Golder Associates

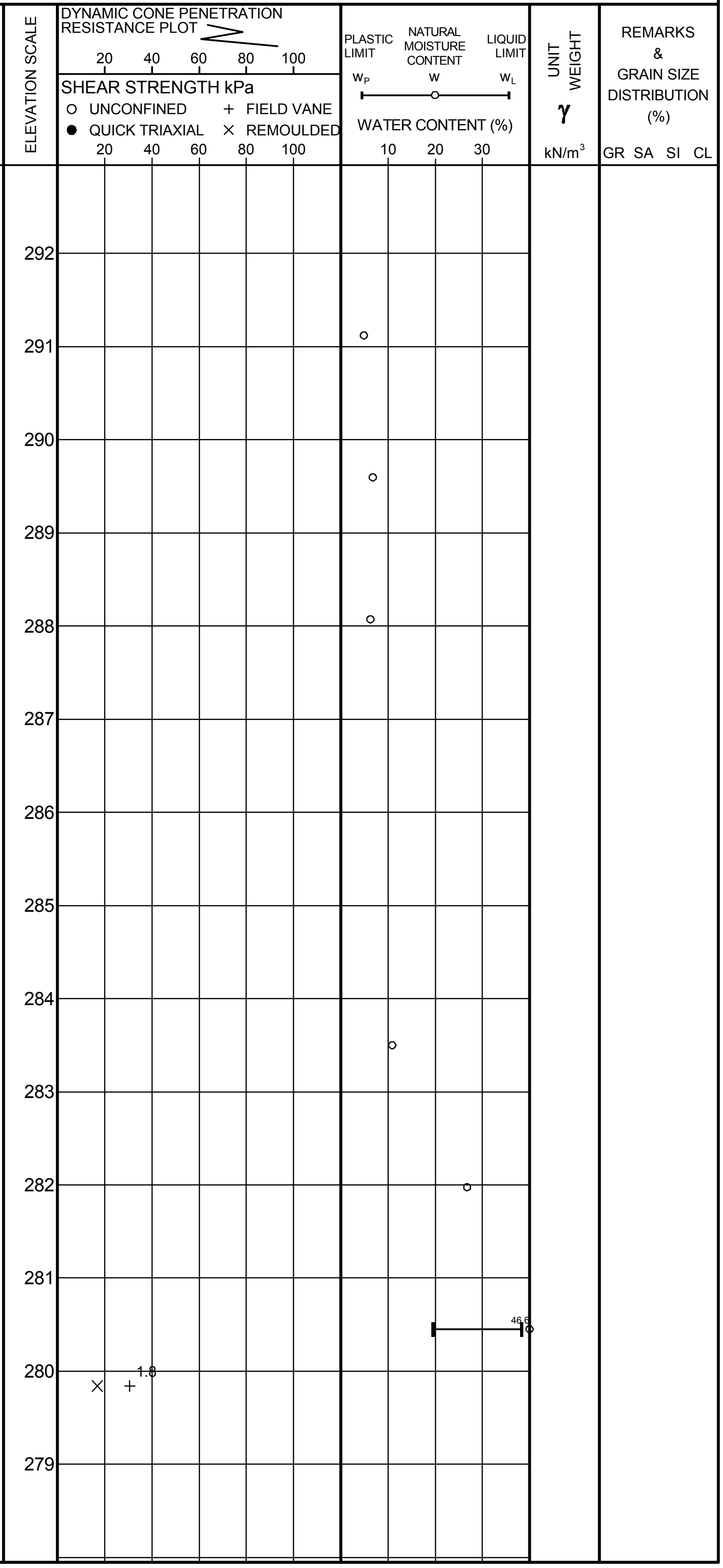
LOGGED: EHS
CHECKED: SEP

RECORD OF BOREHOLE No BH06-24

1 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-00
DIST 52 HWY 11
DATUM Geodetic
LOCATION N 5020581; E325161
BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers
DATE 07/04/06
ORIGINATED BY EHS
COMPILED BY AB
CHECKED BY SEP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
292.9	GROUND SURFACE				
0.0	ASPHALT				
0.3	Sand and gravel (FILL) Brown				
	Sand, trace gravel, trace silt (FILL) Very loose to dense Brown Moist	1	SS	20	
		2	SS	31	
		3	SS	12	
		4	SS	7	
		5	SS	4	
		6	SS	7	
		7	SS	10	
		8	SS	10	
	Becoming wet at 9.1m depth				∇
	Augers grinding between 9.4m and 10.4m depth	9	SS	13	
282.3 10.7	CLAYEY SILT to SILTY CLAY, trace sand, trace gravel Very soft to very stiff Grey Wet	10	SS	9	
		11	SS	2	
		12	SS	15	



Continued Next Page

+3, ×3: Numbers refer to Sensitivity

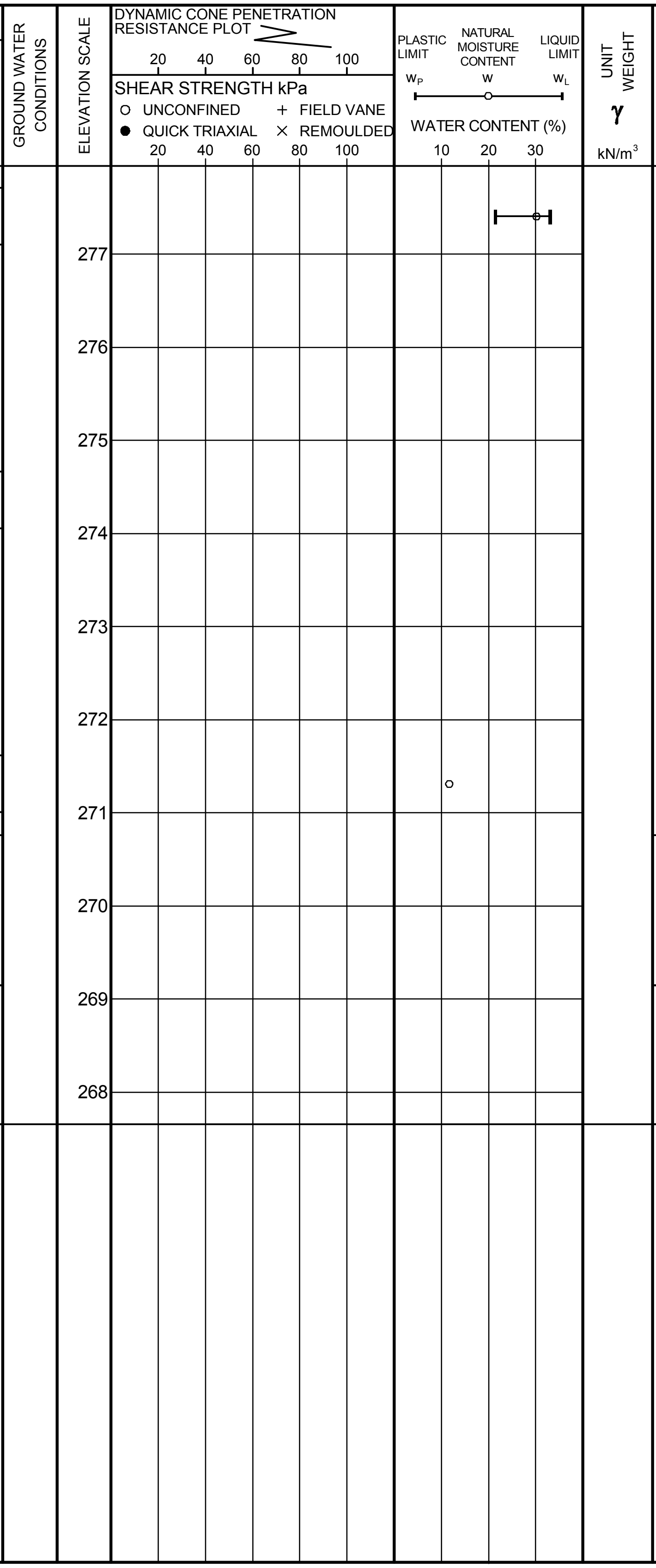
○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH06-24

2 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-00
LOCATION N 5020581; E325161
ORIGINATED BY EHS
DIST 52 HWY 11
BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers
COMPILED BY AB
DATUM Geodetic
DATE 07/04/06
CHECKED BY SEP

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---				
	CLAYEY SILT to SILTY CLAY, trace sand, trace gravel Very soft to very stiff Grey Wet	13	SS	24	
276.2 16.8	Sandy SILT, trace clay Compact Grey Wet	14	SS	12	
273.1 19.8	Gravelly SAND to Silty SAND, containing cobbles Compact Grey Wet	15	SS	13	
270.8 22.2	GNEISS (BEDROCK)	1	RC	REC 100%	RQD = 92%
	Bedrock cored from 22.2m to 25.3m depth For coring details see Record of Drillhole BH06-24	2	RC	REC 100%	RQD = 95%
267.7 25.3	End of Borehole				



Notes:
1. Difficult augering at 20.1m depth. Switch to NW Casing.
2. Water level at 9.1m depth (Elev. 283.8m) upon completion of drilling.

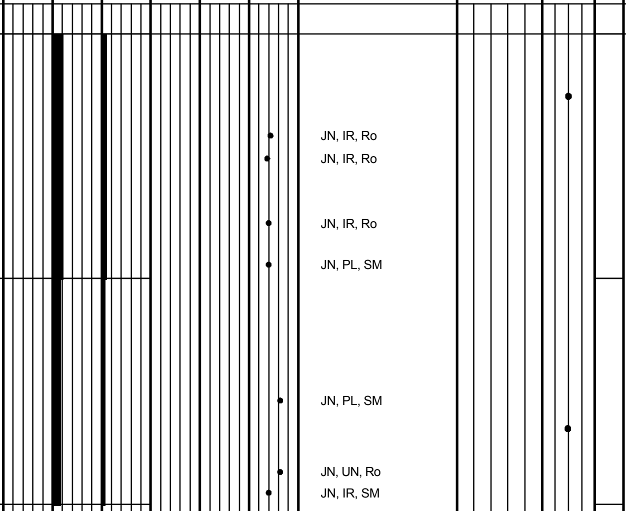
+3, ×3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

RECORD OF DRILLHOLE: BH06-24

PROJECT: 06-1191-001
SHEET 1 OF 1
LOCATION: N 5020581; E325161
DRILLING DATE: 07/04/06
DATUM: Geodetic
INCLINATION: -90° AZIMUTH: ---
DRILL RIG:
DRILLING CONTRACTOR:

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	



DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page	270.8			
	NQ Coring	GNEISS Fine to medium grained Fresh to slightly weathered Strong to very strong Grey	22.2			
					JN, IR, Ro	
23				1	JN, IR, Ro	
						UCS=50MPa
					JN, IR, Ro	
					JN, PL, SM	
24						
				2	JN, PL, SM	
25					JN, UN, Ro	
			267.7 25.3		JN, IR, SM	
		End of Drillhole				
26						
27						
28						
29						
30						
31						
32						

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: EHS
CHECKED: SEP

MINISTRY OF TRANSPORTATION, ONTARIO

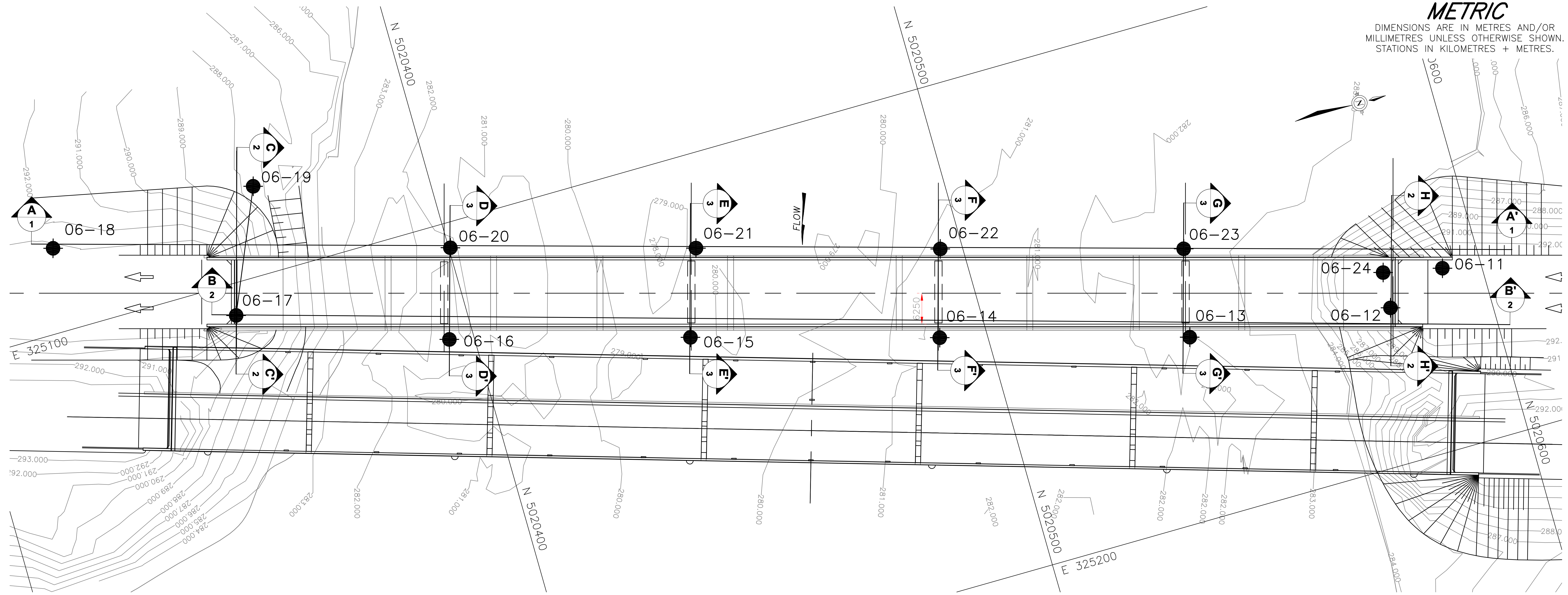
CONT No.
WP No. 94–89–01

VERNON LAKE NARROWS
HWY 11 SBL STRUCTURE
BOREHOLE LOCATION AND
SOIL STRATA

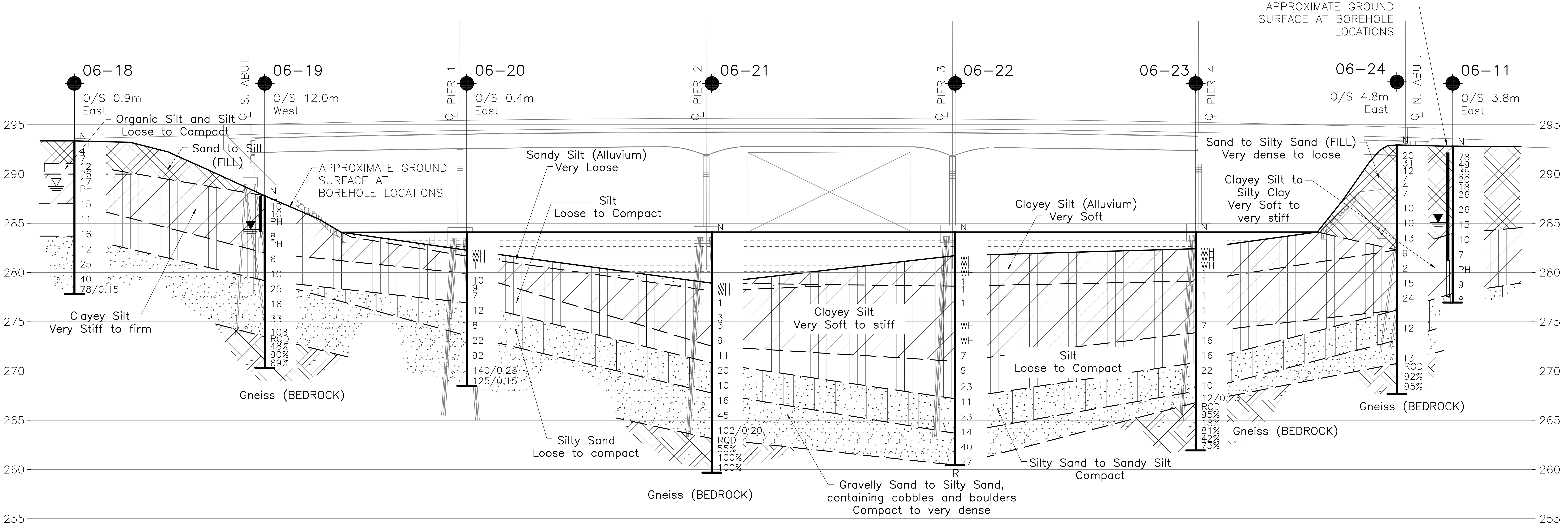
SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

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



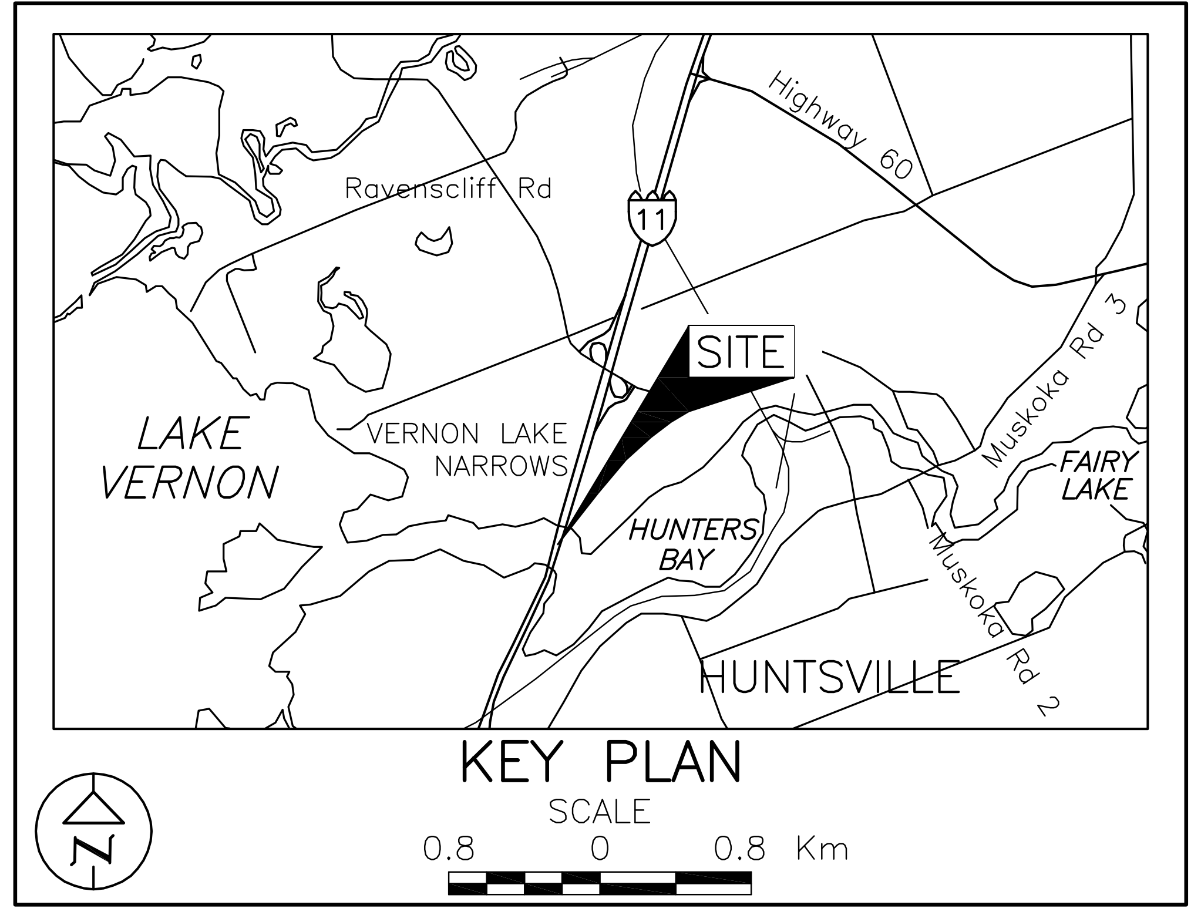
PLAN
SCALE
10 0 10 20 m



PROFILE A–A'
SCALE
HORIZ. 10 0 10 20 m
SCALE
VERT. 5 0 5 10 m

S.E.M.COYNE July 13, 2007

J.M.A.COSTA July 13, 2007



KEY PLAN
SCALE
0.8 0 0.8 Km

LEGEND

●	Borehole
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▼	WL in piezometer, measured on Aug. 24, 2006
▽	WL upon completion of drilling
R	Refusal

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
06–11	292.8	5020592	325164
06–12	293.0	5020580	325169
06–13	284.1	5020539	325163
06–14	284.1	5020491	325150
06–15	284.1	5020442	325136
06–16	284.1	5020395	325123
06–17	294.0	5020355	325106
06–18	293.4	5020323	325083
06–19	287.8	5020365	325082
06–20	284.1	5020400	325105
06–21	284.1	5020448	325119
06–22	284.1	5020496	325132
06–23	284.1	5020543	325146
06–24	292.9	5020581	325161

NOTES

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

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

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

REFERENCE

Base plans provided in digital format by LEA Consulting Ltd., drawing file no. GA SBL.dwg, dated May, 2006, received August 28, 2006.

NO.	DATE	BY	REVISION

Geocres No. 31E–270

HWY. 11		PROJECT NO. 06–1191–001	DIST. 52
SUBM'D. AB	CHKD. SEMC	DATE: July 2007	SITE: 42–018
DRAWN: MSM	CHKD. JMAC	APPD. JMAC	DWG. 1

PLOT DATE: July 13, 2007

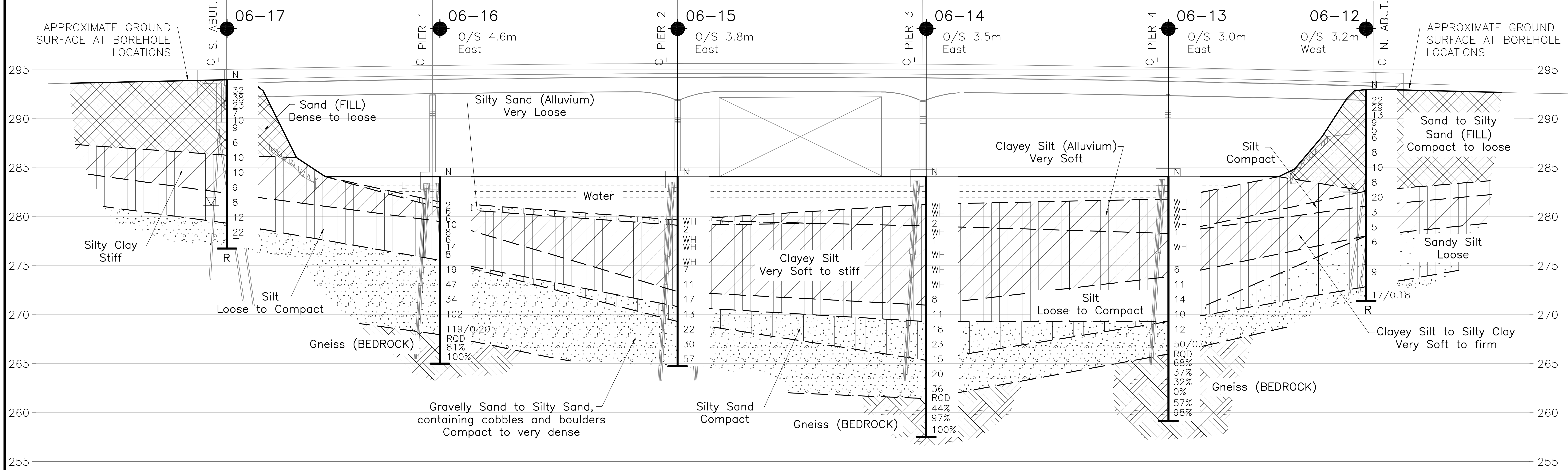
CONT No.
WP No. 94-89-01

VERNON LAKE NARROWS
HWY 11 SBL STRUCTURE
SOIL STRATA I

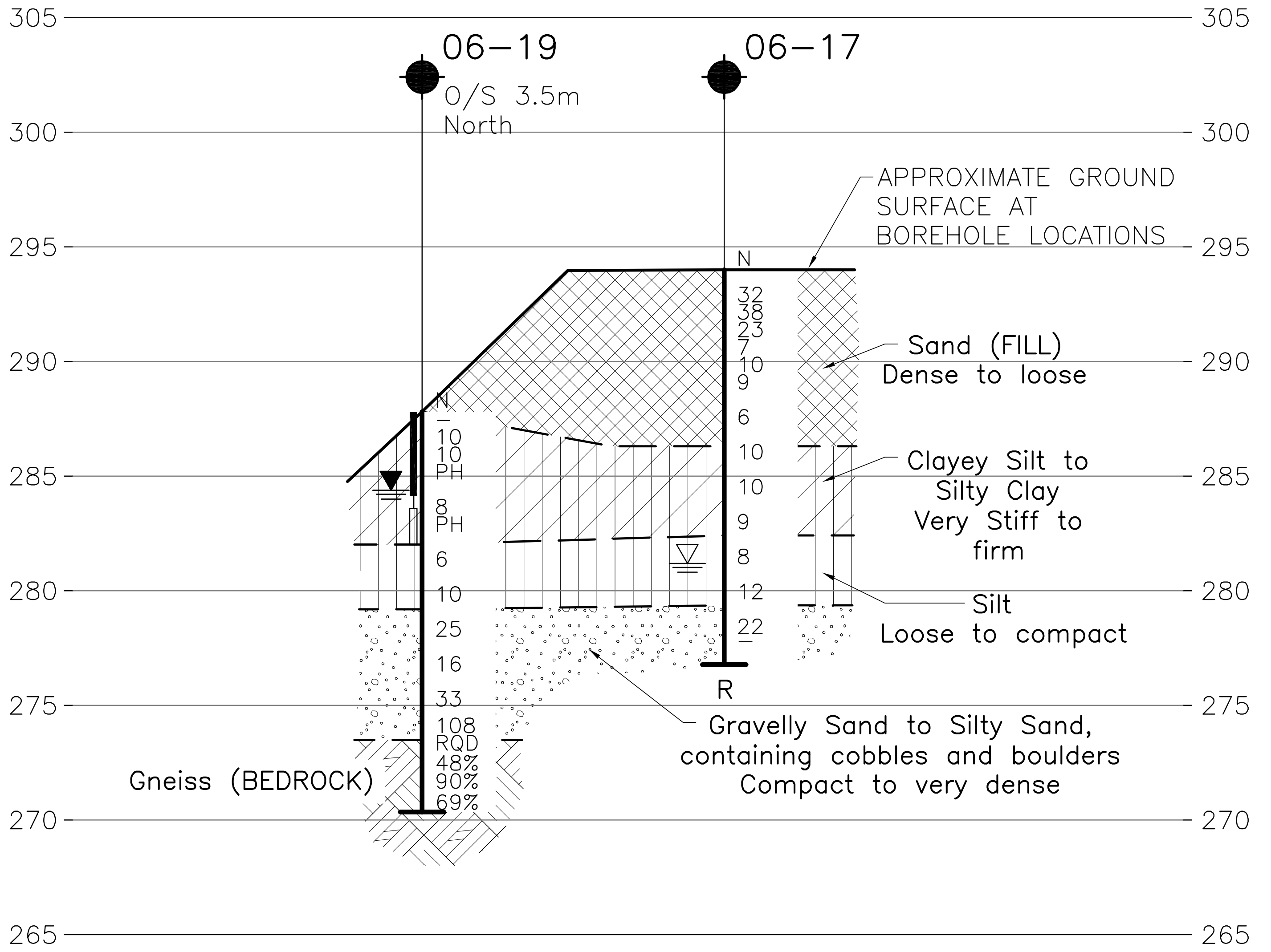
SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

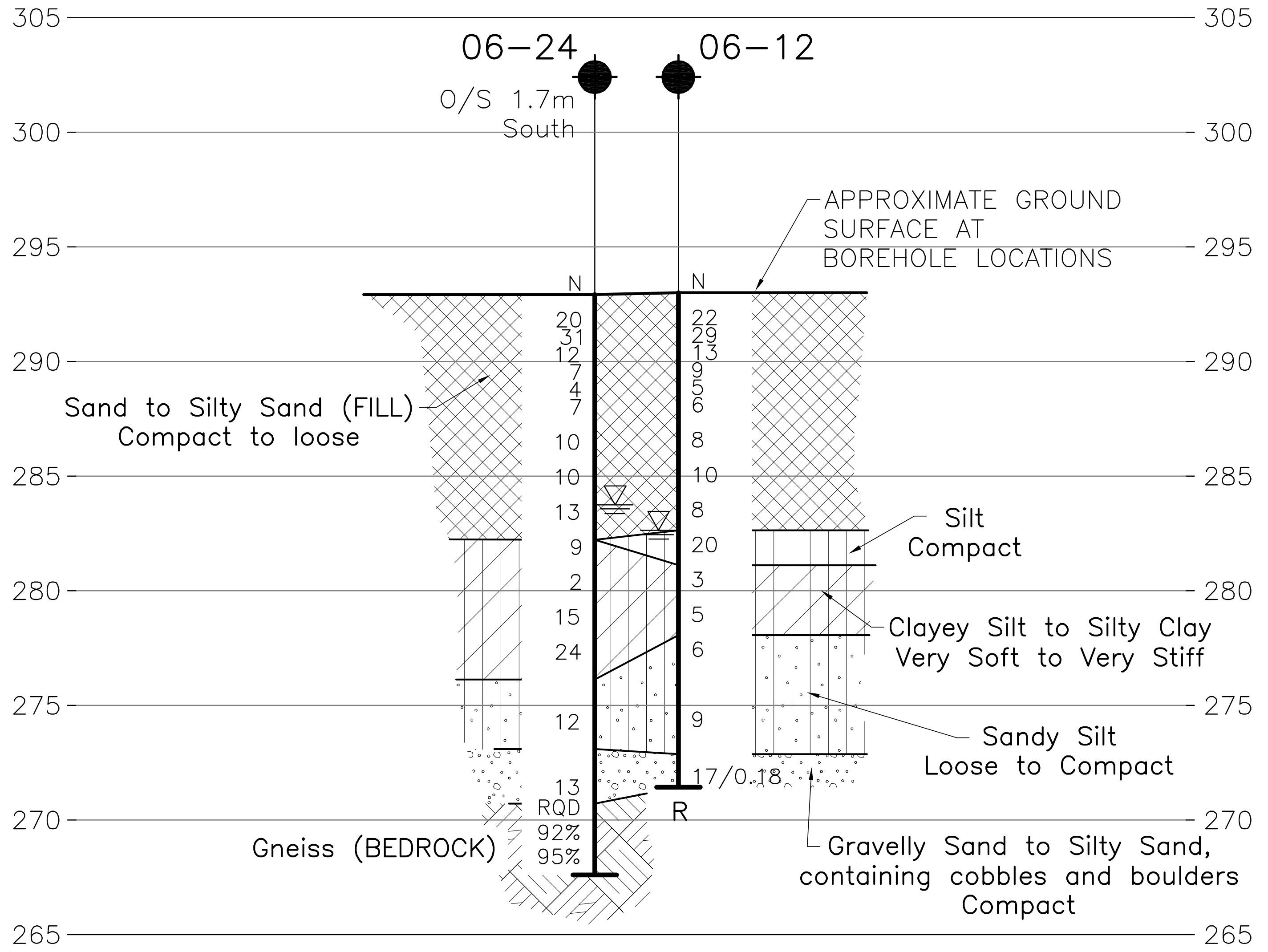
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.



PROFILE B-B'



SECTION C-C'
SOUTH ABUTMENT



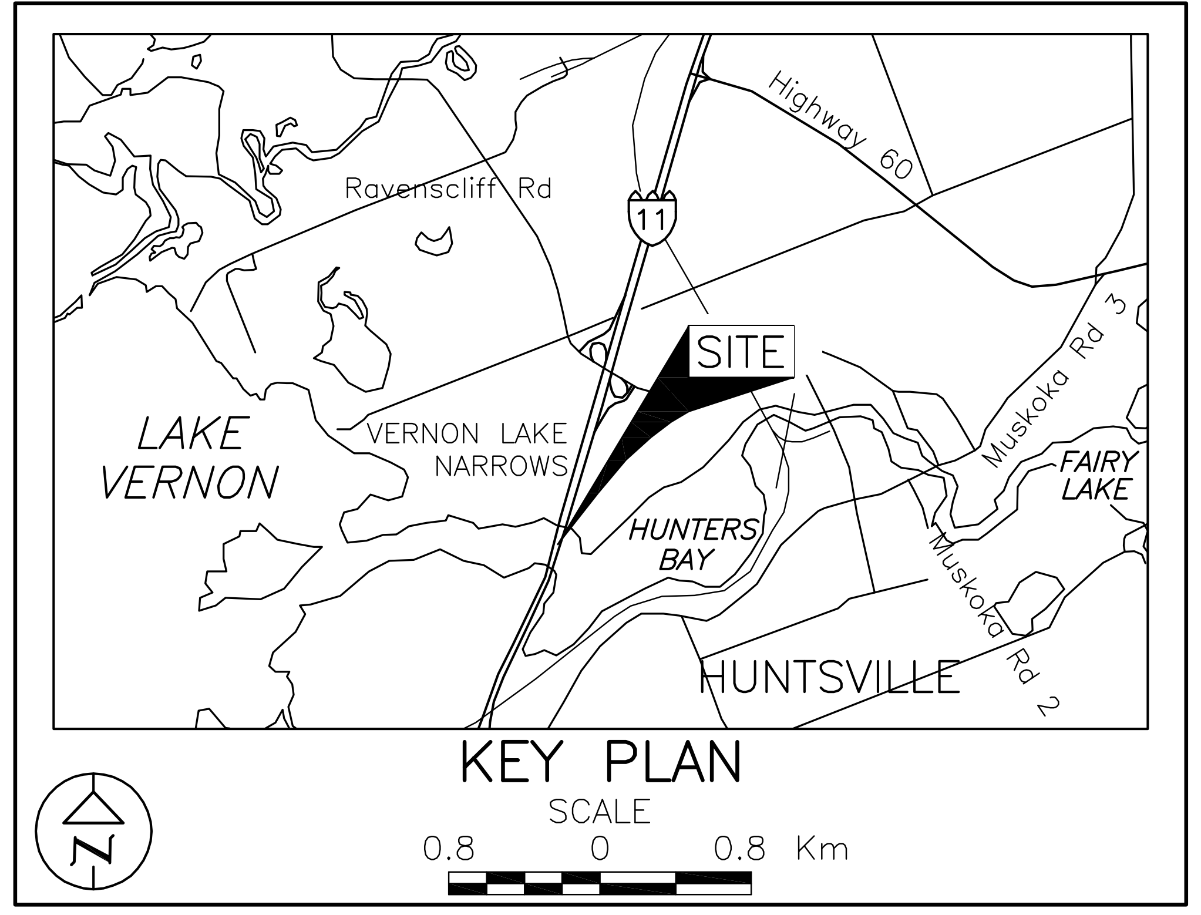
SECTION H-H'
NORTH ABUTMENT

SCALE
HORIZ. 10 0 10 20 m

SCALE
VERT. 5 0 5 10 m

S.E.M.COYNE
July 13, 2007

J.M.A.COSTA
July 13, 2007



KEY PLAN
SCALE
0.8 0 0.8 Km

LEGEND

●	Borehole
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▼	WL in piezometer, measured on Aug. 24, 2006
▽	WL upon completion of drilling
R	Refusal

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
06-11	292.8	5020592	325164
06-12	293.0	5020580	325169
06-13	284.1	5020539	325163
06-14	284.1	5020491	325150
06-15	284.1	5020442	325136
06-16	284.1	5020395	325123
06-17	294.0	5020355	325106
06-18	293.4	5020323	325083
06-19	287.8	5020365	325082
06-20	284.1	5020400	325105
06-21	284.1	5020448	325119
06-22	284.1	5020496	325132
06-23	284.1	5020543	325146
06-24	292.9	5020581	325161

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.

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REFERENCE

Base plans provided in digital format by LEA Consulting Ltd., drawing file no. GA SBL.dwg, dated May, 2006, received August 28, 2006.

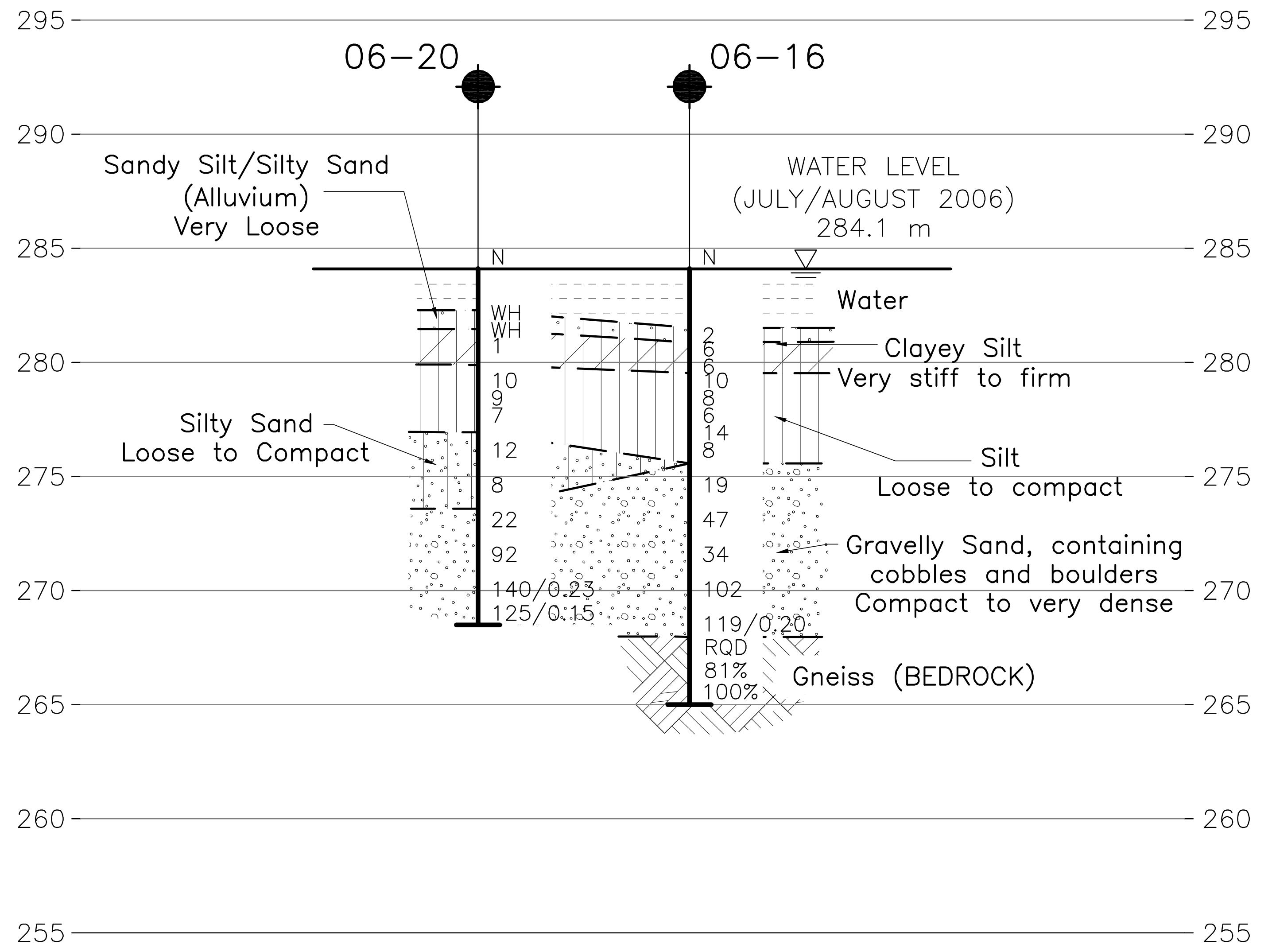
NO.	DATE	BY	REVISION

Geocres No. 31E-270

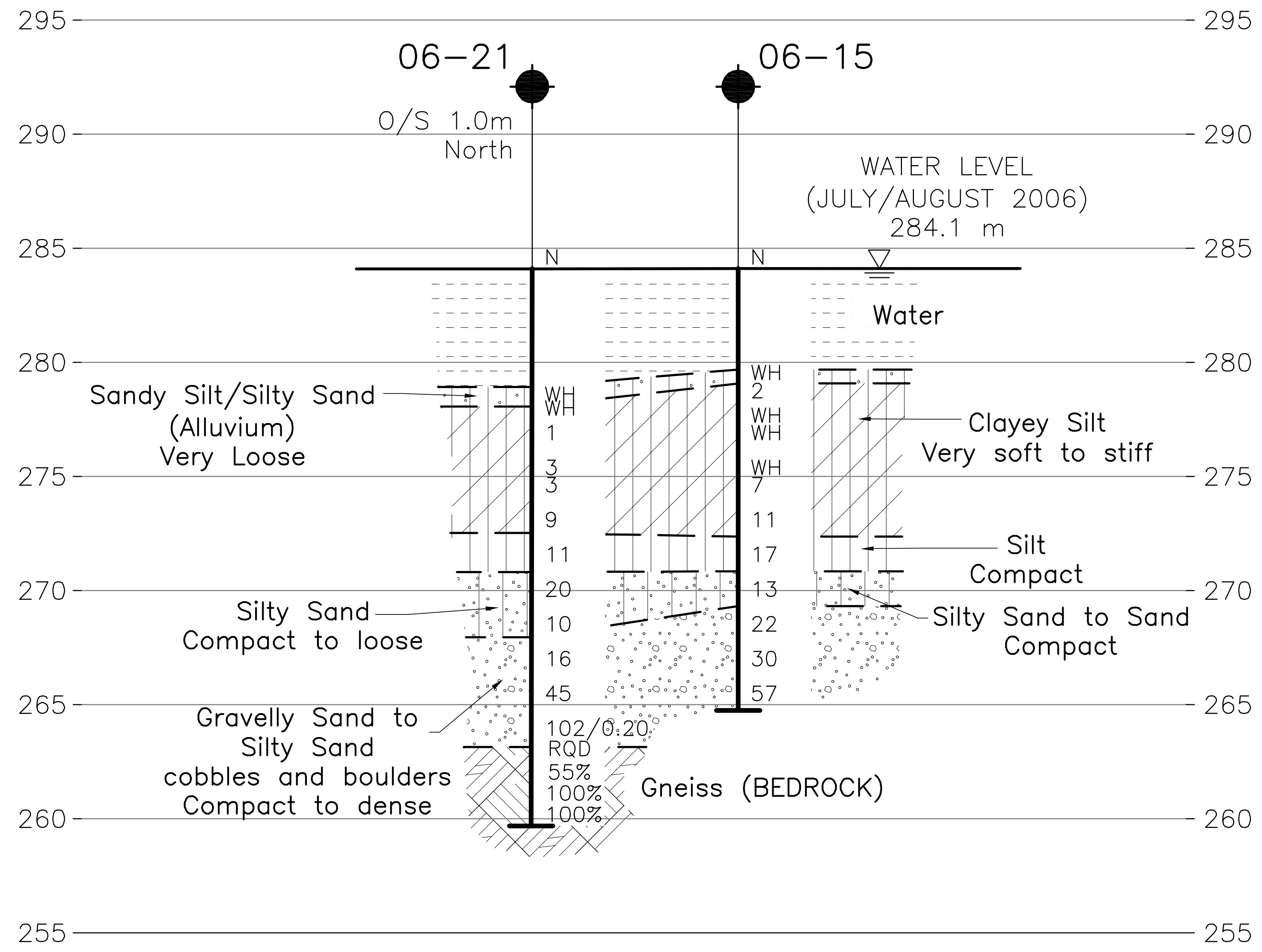
HWY. 11		PROJECT NO. 06-1191-001	DIST. 52
SUBM'D. AB	CHKD. SEMC	DATE: July 2007	SITE: 42-018
DRAWN: MSM	CHKD. JMAC	APPD. JMAC	DWG. 2

PLOT DATE: July 13, 2007

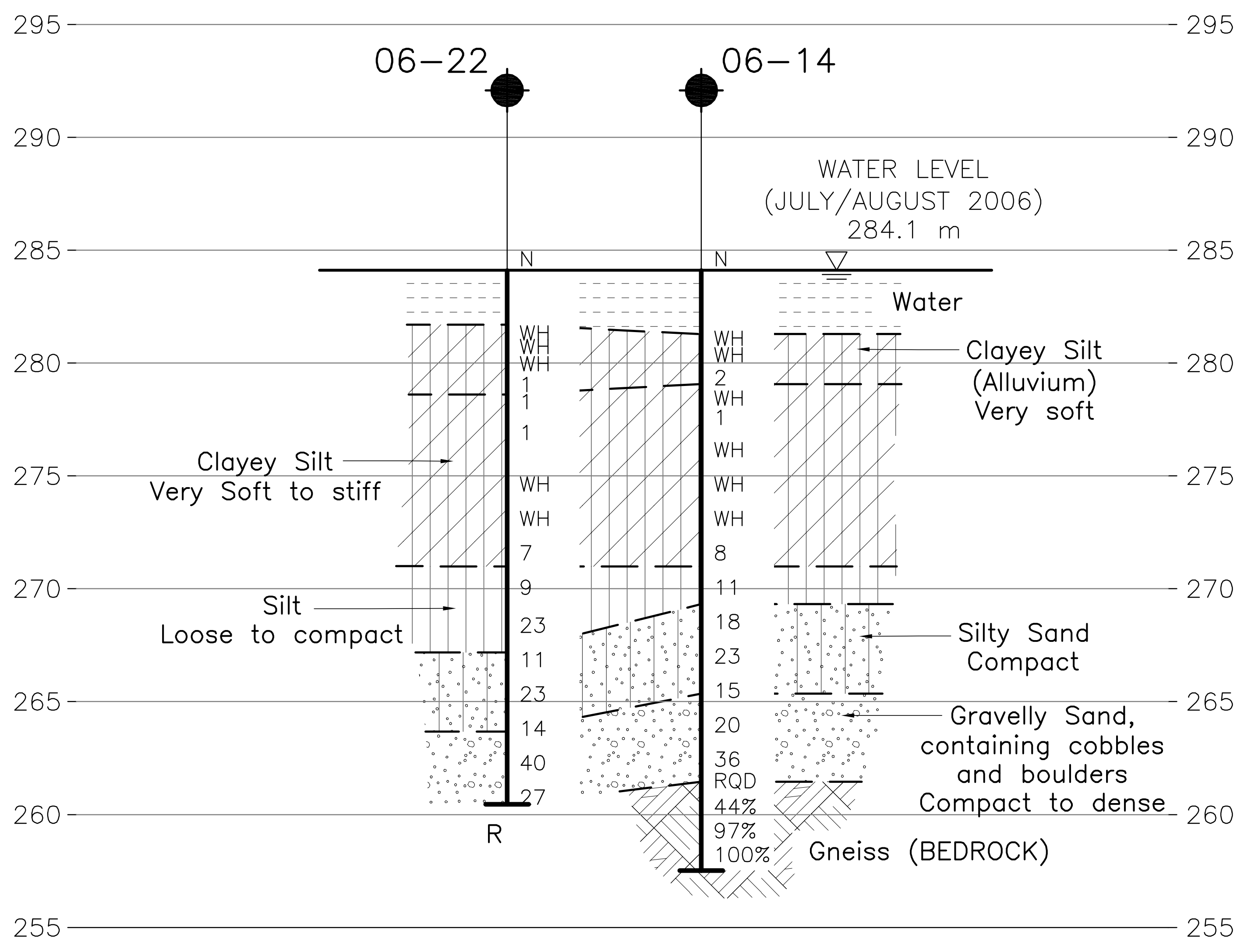
MINISTRY OF TRANSPORTATION, ONTARIO



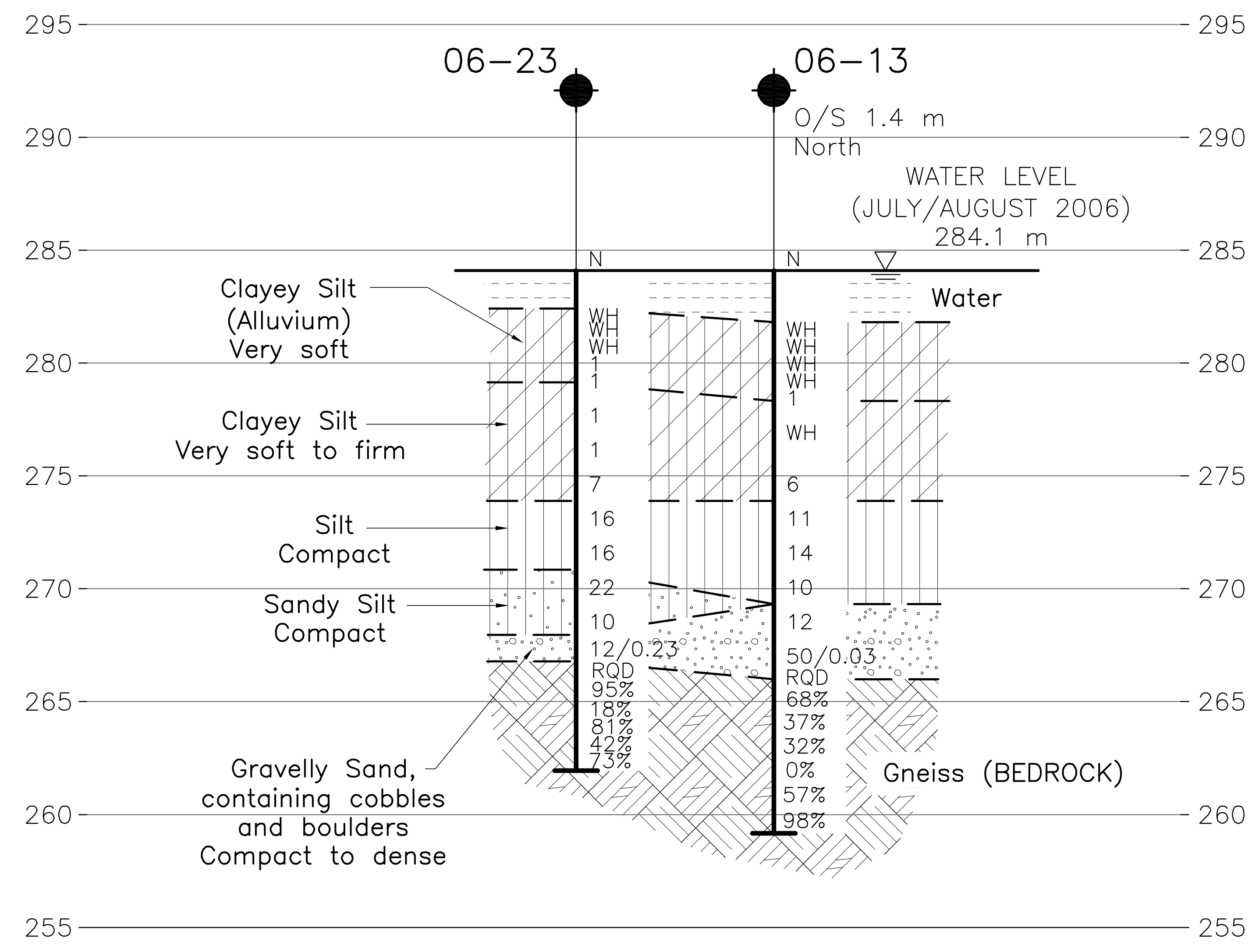
SECTION D–D'
PIER 1



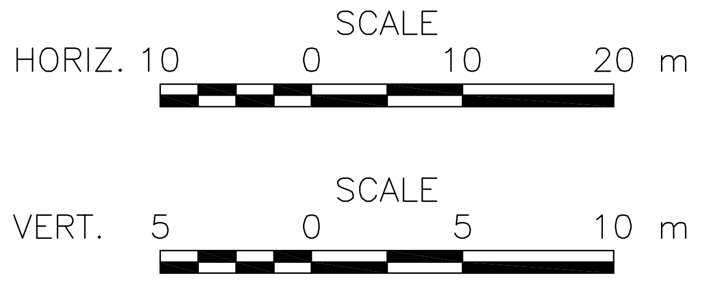
SECTION E–E'
PIER 2



SECTION F–F'
PIER 3



SECTION G–G'
PIER 4



LICENSED PROFESSIONAL ENGINEER
S.E.M.COYNE
July 13, 2007
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
J.M.A.COSTA
July 13, 2007
PROVINCE OF ONTARIO

REFERENCE

Base plans provided in digital format by LEA Consulting Ltd., drawing file no. GA SBL.dwg, dated May, 2006, received August 28, 2006.

METRIC

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

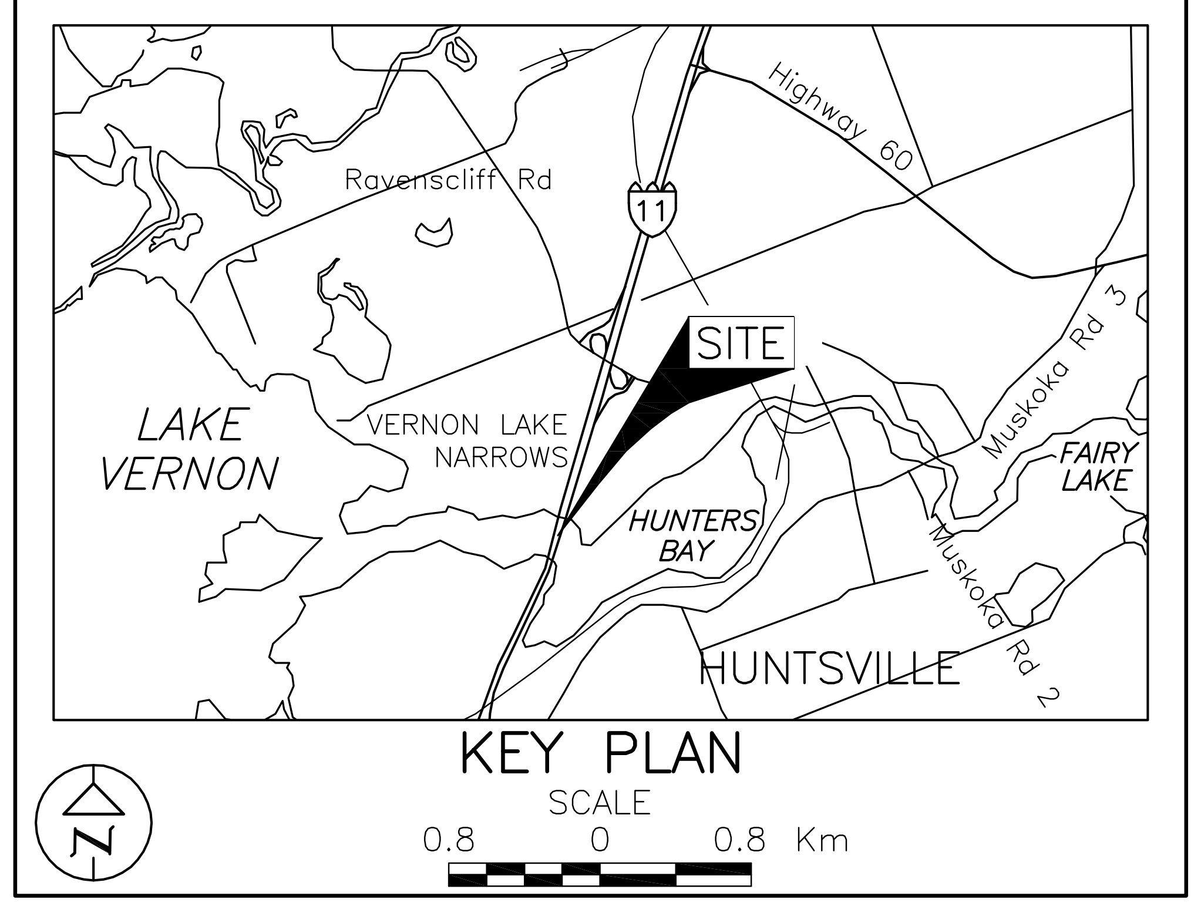
CONT No.
WP No. 94–89–01

VERNON LAKE NARROWS
HWY 11 SBL STRUCTURE
SOIL STRATA II

SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

Golder Associates



KEY PLAN

LEGEND

●	Borehole
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▼	WL in piezometer, measured on Aug. 24, 2006
▽	WL upon completion of drilling
R	Refusal

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
06–11	292.8	5020592	325164
06–12	293.0	5020580	325169
06–13	284.1	5020539	325163
06–14	284.1	5020491	325150
06–15	284.1	5020442	325136
06–16	284.1	5020395	325123
06–17	294.0	5020355	325106
06–18	293.4	5020323	325083
06–19	287.8	5020365	325082
06–20	284.1	5020400	325105
06–21	284.1	5020448	325119
06–22	284.1	5020496	325132
06–23	284.1	5020543	325146
06–24	292.9	5020581	325161

NOTES

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

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

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

NO.	DATE	BY	REVISION

Geocres No. 31E–270

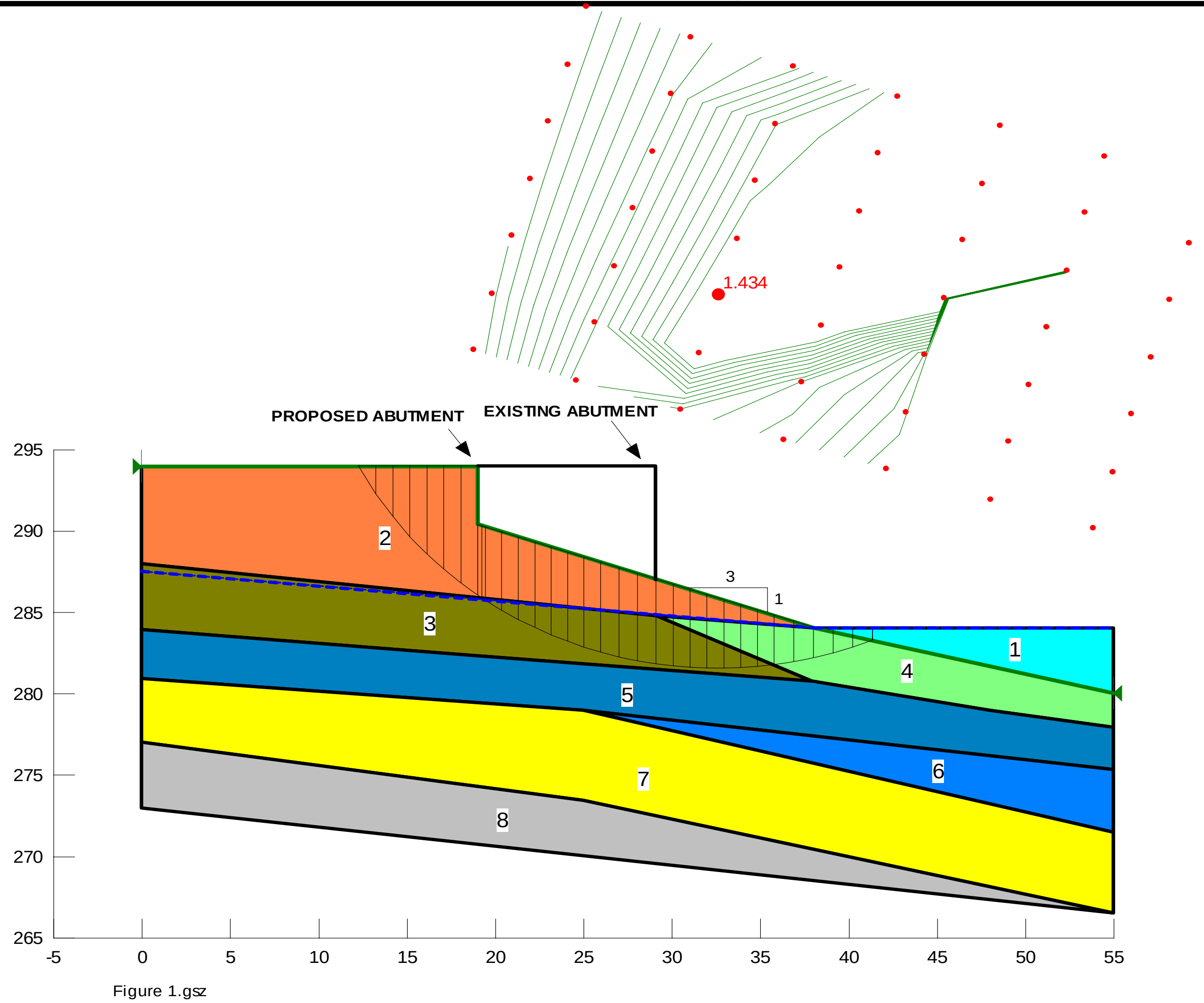
HWY. 11		PROJECT NO. 06–1191–001	DIST. 52
SUBM'D. AB	CHKD. SEMC	DATE: July 2007	SITE: 42–018
DRAWN: MSM/JFC	CHKD. JMAC	APPD. JMAC	DWG. 3

PLOT DATE: July 13, 2007
FILENAME: T:\Projects\2006\06-1191-001 (LEA, Huntsville)\-BB- SBL Bridge\061191001BB003.dwg

STABILITY ANALYSIS

South Abutment Front Slope

FIGURE 1



Material #: 1
Description: Water

Material #: 2
Description: Existing Granular Fill
Wt: 21
Phi: 32

Material #: 3
Description: Clayey Silt to Silty Clay
Wt: 18
Phi: 30

Material #: 4
Description: Clayey Silt (Toe of Slope)
Wt: 18
Cohesion: 20

Material #: 5
Description: Silt
Wt: 19
Phi: 30

Material #: 6
Description: Silty Sand
Wt: 19
Phi: 30

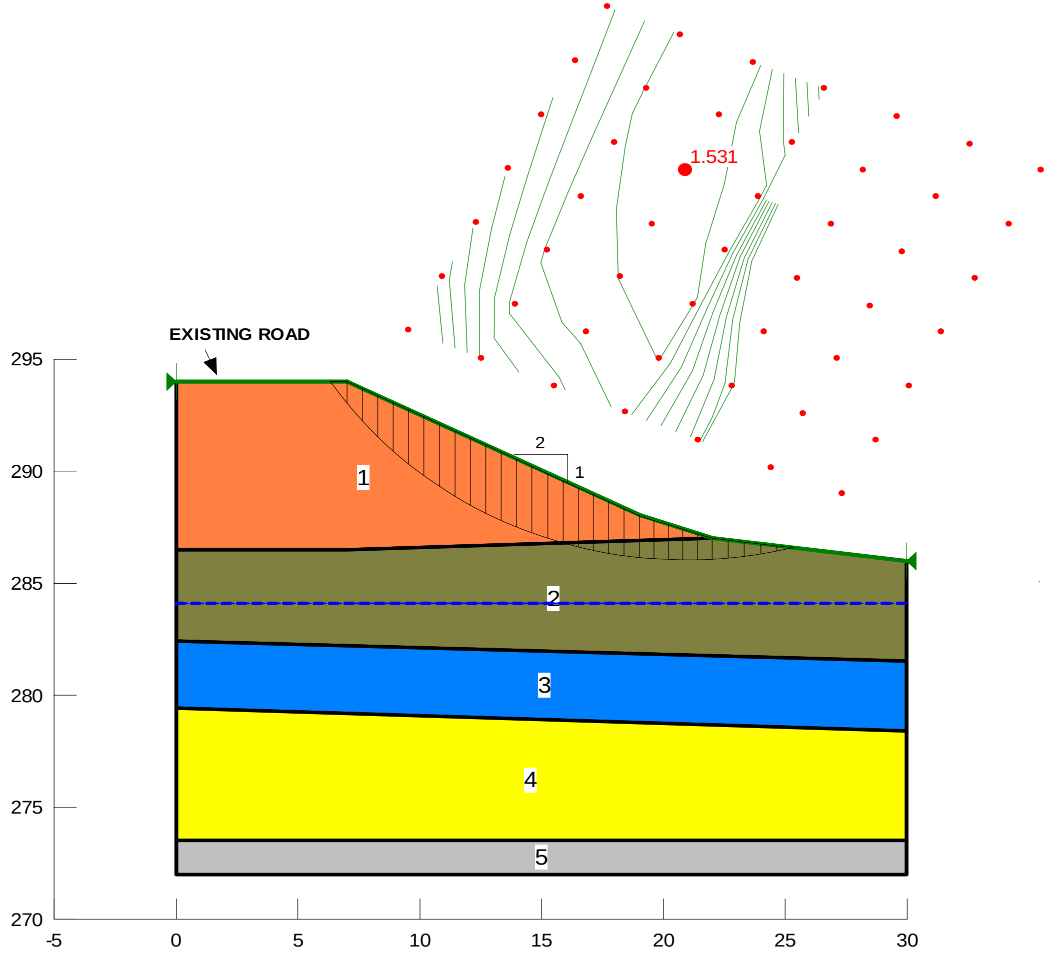
Material #: 7
Description: Sand to Gravelly Sand
Wt: 21
Phi: 35

Material #: 8
Description: Bedrock

Date:	July 2007		Drawn:	AB
Project:	06-1191-001-S	**Golder Associates**	Checked:	SEMC

STABILITY ANALYSIS
South Approach Embankment Side Slope

FIGURE 2



Material #: 1
Description: Existing Granular Fill
Wt: 21
Phi: 32

Material #: 2
Description: Clayey Silt to Silty Clay
Wt: 18
Phi: 30

Material #: 3
Description: Silt
Wt: 19
Phi: 30

Material #: 4
Description: Gravelly Sand to Silty Sand
Wt: 21
Phi: 35

Material #: 5
Description: Bedrock

Figure 2.gsz

Date: July 2007
Project: 06-1191-001-S

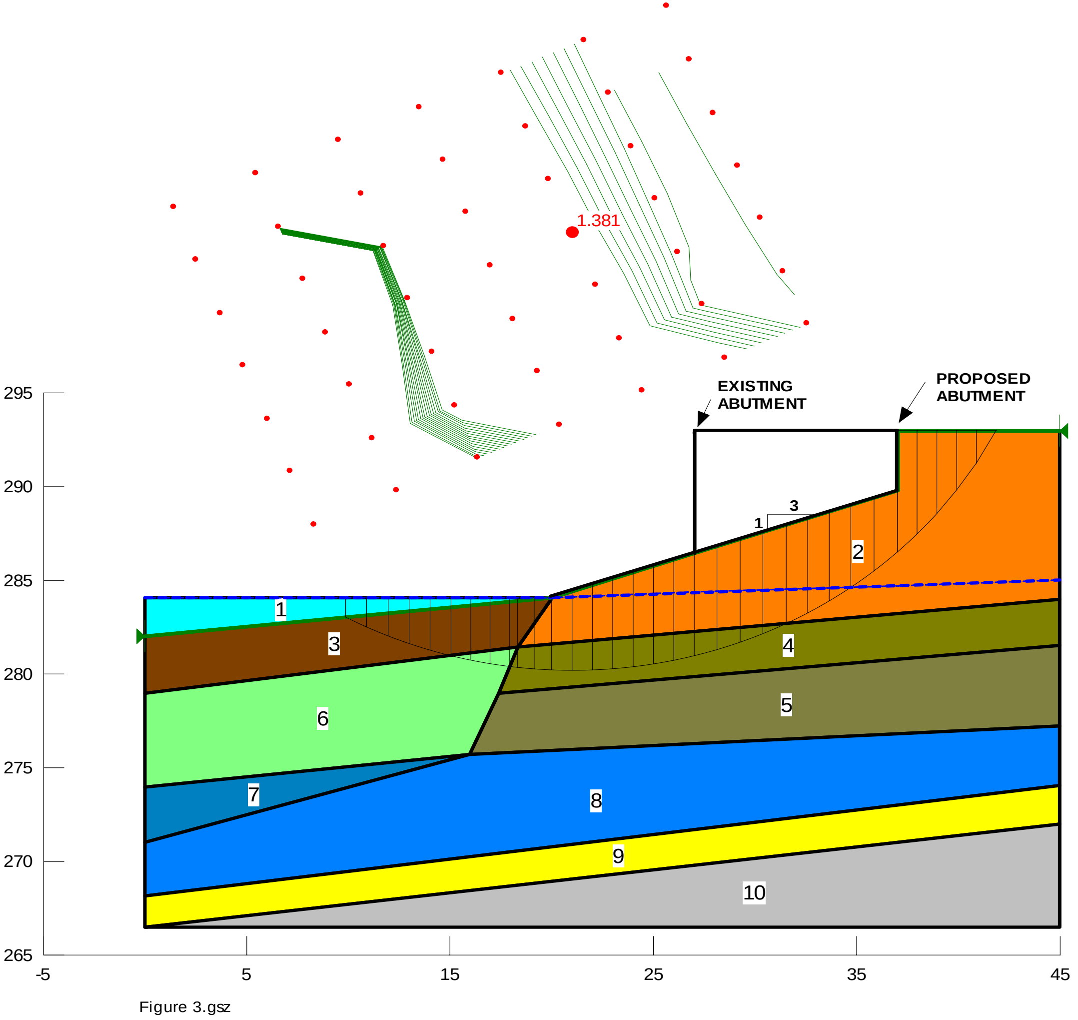
Golder Associates

Drawn: AB
Checked: SEMC

STABILITY ANALYSIS

North Abutment Front Slope

FIGURE 3



Material #: 1
Description: Water

Material #: 2
Description: Existing Granular Fill
Wt: 21
Phi: 32

Material #: 3
Description: Alluvium
Wt: 15
Cohesion: 10

Material #: 4
Description: Clayey Silt to Silty Clay
Wt: 18
Phi: 26

Material #: 5
Description: Clayey Silt to Silty Clay
Wt: 18
Phi: 28

Material #: 6
Description: Clayey Silt (toe of slope)
Wt: 18
Cohesion: 20

Material #: 7
Description: Silt
Wt: 19
Phi: 30

Material #: 8
Description: Silty Sand to Sandy Silt
Wt: 19
Phi: 30

Material #: 9
Description: Gravelly Sand to Silty Sand
Wt: 21
Phi: 35

Material #: 10
Description: Bedrock

Date: July 2007
Project: 06-1191-001-S

Golder Associates

Drawn: AB
Checked: SEMC

STABILITY ANALYSIS
North Approach Embankment
Existing 2H:1V Northwest Side Slope

FIGURE 4

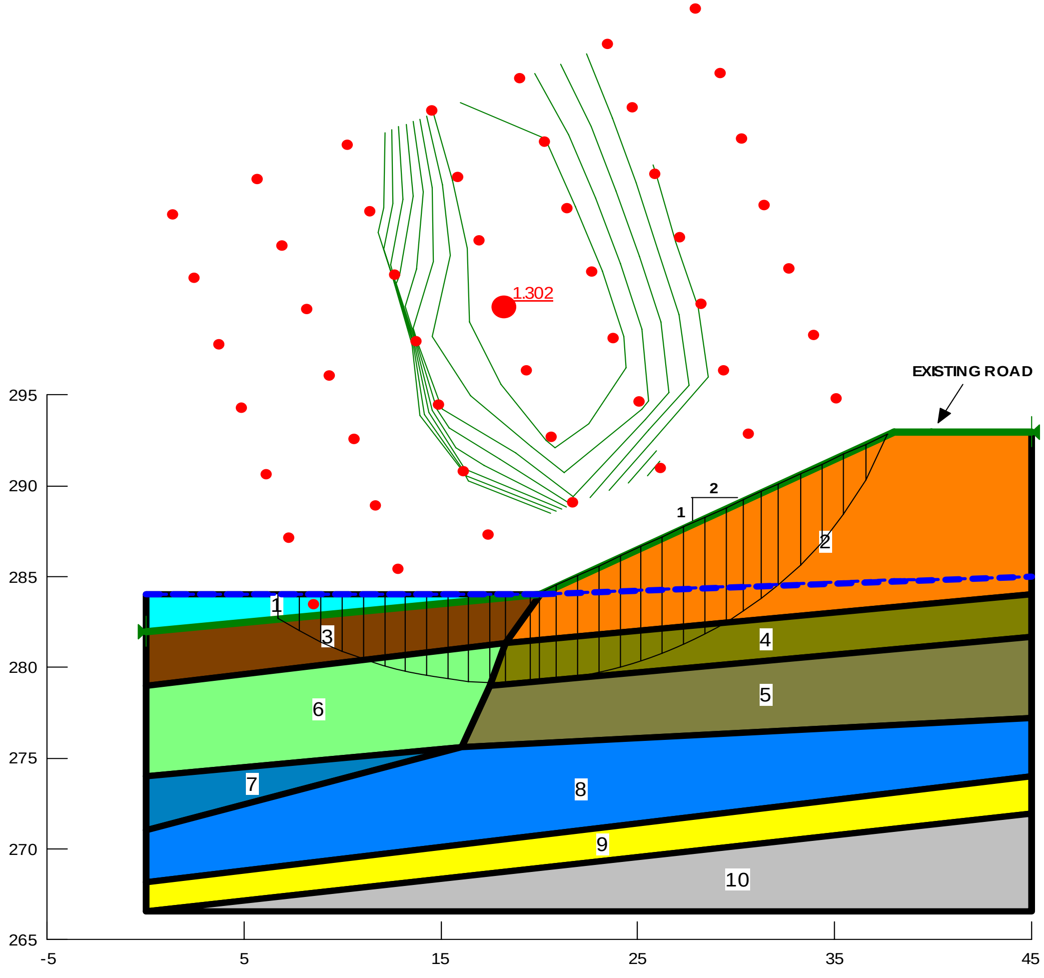


Figure 4.gsz

Material #: 1
Description: Water
Wt: 9.807

Material #: 2
Description: Existing Granular Fill
Wt: 21
Phi: 32

Material #: 3
Description: Alluvium
Wt: 15
Cohesion: 10

Material #: 4
Description: Clayey Silt to Silty Clay
Wt: 18
Phi: 26

Material #: 5
Description: Clayey Silt to Silty Clay
Wt: 18
Phi: 28

Material #: 6
Description: Clayey Silt (toe of slope)
Wt: 18
Cohesion: 20

Material #: 7
Description: Silt
Wt: 19
Phi: 30

Material #: 8
Description: Silty Sand to Sandy Silt
Wt: 19
Phi: 30

Material #: 9
Description: Gravelly Sand to Silty Sand
Wt: 21
Phi: 35

Material #: 10
Description: Bedrock

Date: July 2007
Project: 06-1191-001-S

Golder Associates

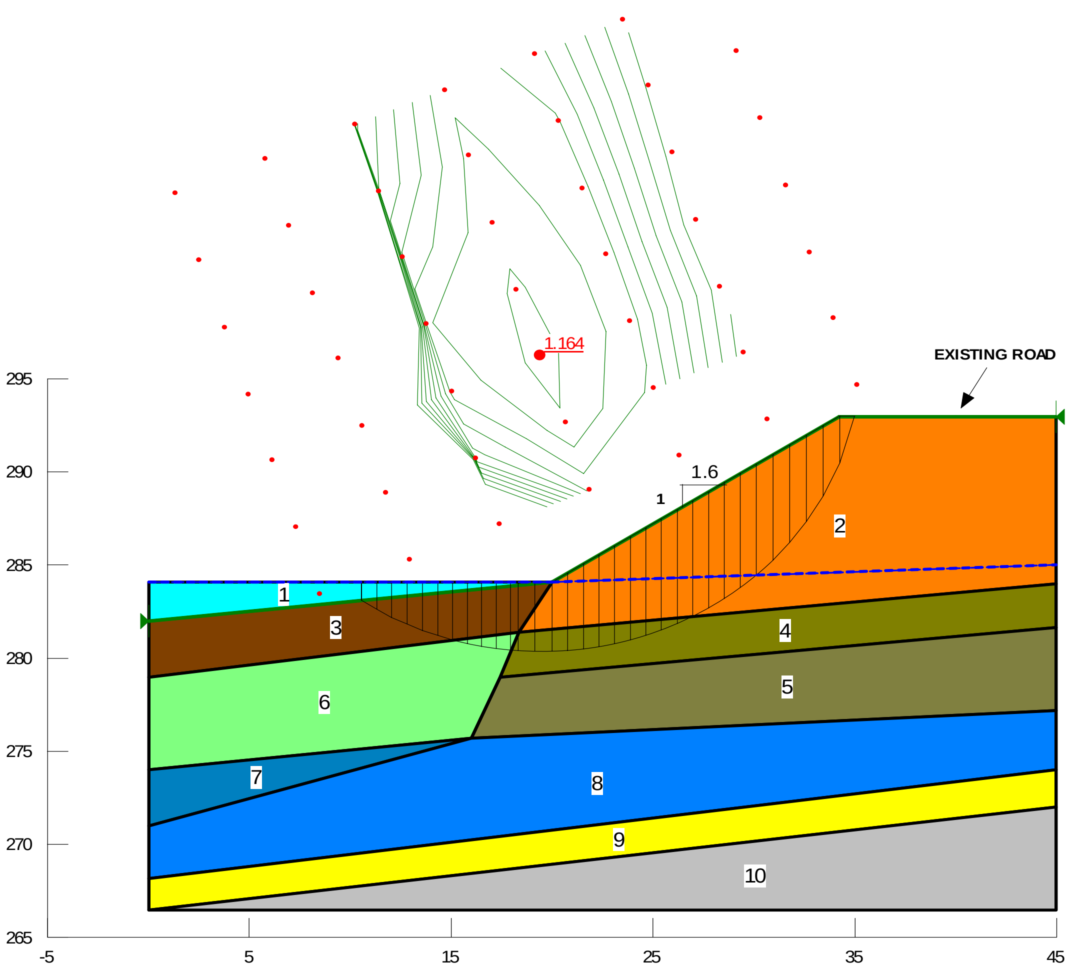
Drawn: AB
Checked: SEMC

STABILITY ANALYSIS

North Approach Embankment

Existing 1.6H:1V Northwest Side Slope

FIGURE 5



Material #: 1
Description: Water
Wt: 9.807

Material #: 2
Description: Existing Granular Fill
Wt: 21
Phi: 32

Material #: 3
Description: Alluvium
Wt: 15
Cohesion: 10

Material #: 4
Description: Clayey Silt to Silty Clay
Wt: 18
Phi: 26

Material #: 5
Description: Clayey Silt to Silty Clay
Wt: 18
Phi: 28

Material #: 6
Description: Clayey Silt (toe of slope)
Wt: 18
Cohesion: 20

Material #: 7
Description: Silt
Wt: 19
Phi: 30

Material #: 8
Description: Silty Sand to Sandy Silt
Wt: 19
Phi: 30

Material #: 9
Description: Gravelly Sand to Silty Sand
Wt: 21
Phi: 35

Material #: 10
Description: Bedrock

Figure 5.gsz

Date: July 2007
Project: 06-1191-001-S

Golder Associates

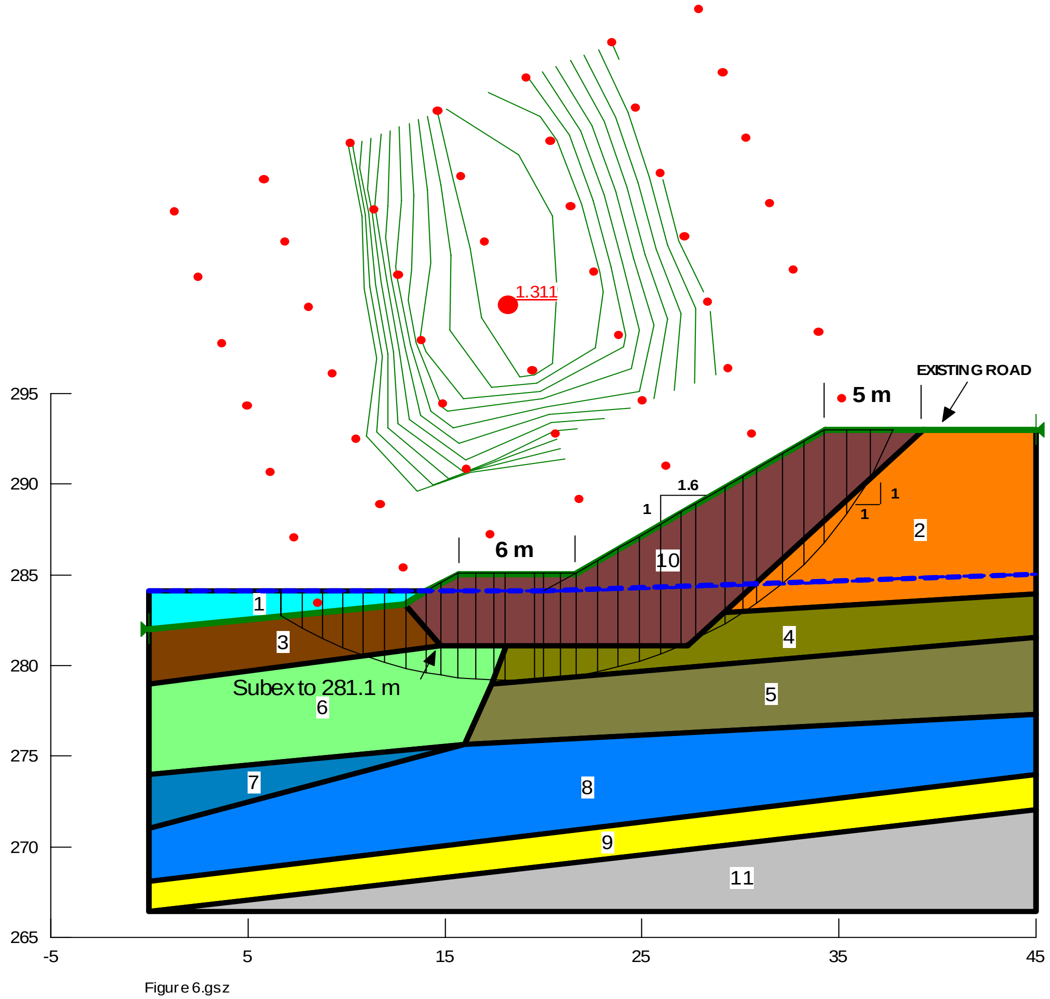
Drawn: AB
Checked: SEMC

STABILITY ANALYSIS

North Approach Embankment

Northwest Rock Fill Side Slope with Sub-excavation and Toe Berm (1.6H:1V)

FIGURE 6



Material #: 1
Description: Water

Material #: 2
Description: Existing Granular Fill
Wt: 21
Phi: 32

Material #: 3
Description: Alluvium
Wt: 15
Cohesion: 10

Material #: 4
Description: Clayey Silt to Silty Clay
Wt: 18
Phi: 26

Material #: 5
Description: Clayey Silt to Silty Clay
Wt: 18
Phi: 28

Material #: 6
Description: Clayey Silt (toe of slope)
Wt: 18
Cohesion: 20

Material #: 7
Description: Silt
Wt: 19
Phi: 30

Material #: 8
Description: Silty Sand to Sandy Silt
Wt: 19
Phi: 30

Material #: 9
Description: Gravelly Sand to Silty Sand
Wt: 21
Phi: 35

Material #: 10
Description: Rock Fill
Wt: 19
Phi: 40

Material #: 11
Description: Bedrock

Date: July 2007
Project: 06-1191-001-S

Golder Associates

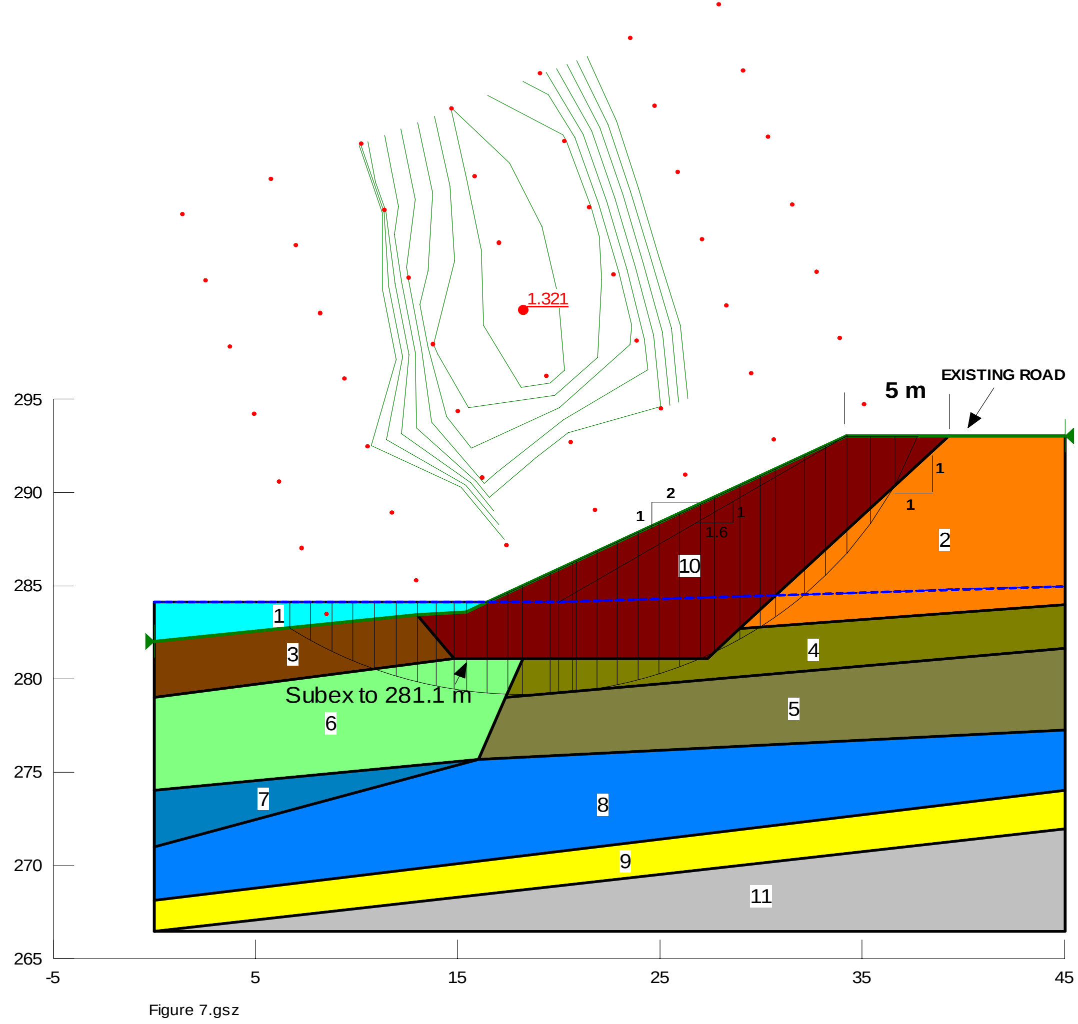
Drawn: AB
Checked: SEMC

STABILITY ANALYSIS

North Approach Embankment

Northwest Rock Fill Side Slope with Sub-excavation (2H:1V)

FIGURE 7



Material #: 1
Description: Water

Material #: 2
Description: Existing Granular Fill
Wt: 21
Phi: 32

Material #: 3
Description: Alluvium
Wt: 15
Cohesion: 10

Material #: 4
Description: Clayey Silt to Silty Clay
Wt: 18
Phi: 26

Material #: 5
Description: Clayey Silt to Silty Clay
Wt: 18
Phi: 28

Material #: 6
Description: Clayey Silt (toe of slope)
Wt: 18
Cohesion: 20

Material #: 7
Description: Silt
Wt: 19
Phi: 30

Material #: 8
Description: Silty Sand to Sandy Silt
Wt: 19
Phi: 30

Material #: 9
Description: Gravelly Sand to Silty Sand
Wt: 21
Phi: 35

Material #: 10
Description: Rock Fill
Wt: 19
Phi: 40

Material #: 11
Description: Bedrock

Date:	July 2007		Drawn:	AB
Project:	06-1191-001-S	**Golder Associates**	Checked:	SEMC

APPENDIX A

LABORATORY TEST RESULTS

TABLE A-1
UNIAXIAL COMPRESSION STRENGTH TEST RESULTS
REPLACEMENT OF SOUTHBOUND STRUCTURE
W.P 94-89-01, SITE NO. 42-018
HIGHWAY 11, HUNTSVILLE

Borehole Number	*Sample Depth (m)*	*Sample Elevation (m)*	*Rock Type*	*Core Diameter (mm)*	*Load (kN)*	*Unconfined Compressive Strength (MPa)*
06-14	24.8	259.3	Gneiss	47.0	75.8	44
06-16	18.4	265.7	Gneiss	47.0	149.3	86
06-24	23.3	269.6	Gneiss	47.0	86.1	50

Checked by: SEMC
Reviewed By: JMAC

TABLE A-2

POINT LOAD STRENGTH TEST RESULTS
REPLACEMENT OF SOUTHBOUND STRUCTURE
W.P 94-89-01, SITE NO. 42-018
HIGHWAY 11, HUNTSVILLE

Borehole Number	*Sample Depth [1] (m)*	*Sample Elevation (m)*	*Rock Type*	*Test Type [2]*	*Core Diameter (mm)*	*Ram Pressure (MPa)*	*Load (kN)*	*I_s Diametral [2] (MPa)*	*I_s 50 mm [2] (MPa)*	*Approximate UCS [2] (MPa)*
06-13	19.8	264.3	Gneiss	D	47.0	10.4	0.010	4.5	4.3	86
06-13	22.1	262.0	Gneiss	D	47.0	12.4	0.012	5.3	5.2	104
06-14	23.6	260.5	Gneiss	D	47.0	11.0	0.010	4.7	4.6	92
06-14	25.4	258.7	Gneiss	D	47.0	10.8	0.010	4.6	4.5	90
06-16	16.6	267.5	Gneiss	D	47.0	11.4	0.011	4.9	4.7	94
06-16	18.6	265.5	Gneiss	D	47.0	12.2	0.012	5.2	5.1	102
06-19	14.9	272.9	Gneiss	D	47.0	12.0	0.011	5.2	5.0	100
06-19	17.0	270.8	Gneiss	D	47.0	14.3	0.014	6.1	6.0	120
06-21	21.2	262.9	Gneiss	D	47.0	18.0	0.017	7.7	7.5	150
06-21	23.5	260.6	Gneiss	D	47.0	14.3	0.014	6.1	6.0	120
06-24	22.6	270.3	Gneiss	D	47.0	9.3	0.009	4.0	3.9	78
06-24	24.8	268.1	Gneiss	D	47.0	9.0	0.009	3.9	3.8	76

Average[3]	4.9	98

NOTES:

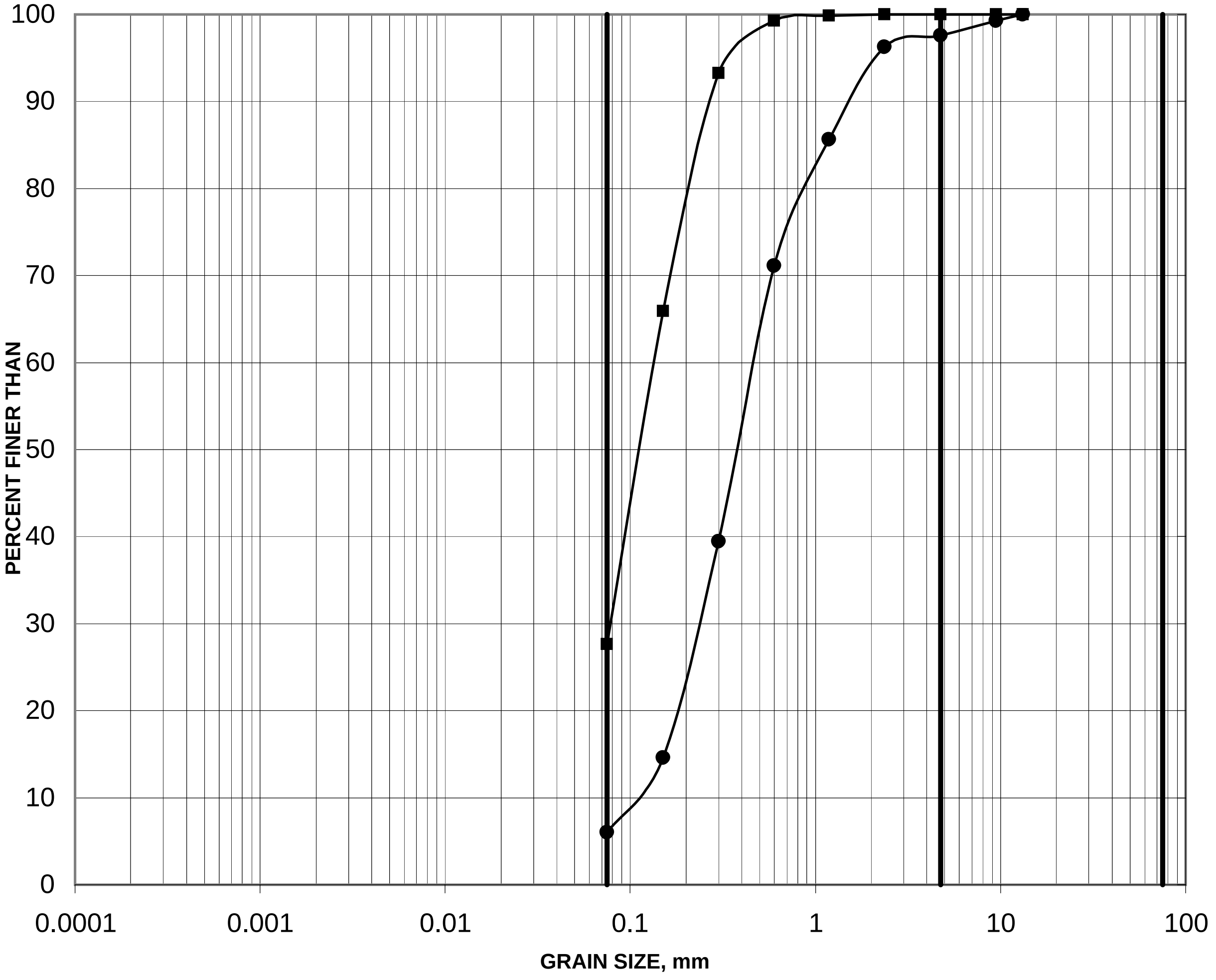
1. Depths are given below the ground surface at the borehole location.
2. Where: D = Diametral test;
   I_s Diametral = Uncorrected point load strength;
   I_s 50 mm = Corrected point load strength; and
   UCS = Unconfined compressive strength = I_s 50 mm X 20 (based on experience with similar rock types)
3. Based on removal of the 2 highest and 2 lowest values

Checked by: SEMC
Reviewed By: JMAC

GRAIN SIZE DISTRIBUTION

Sand to Silty Sand (Fill)

FIGURE
A-1



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

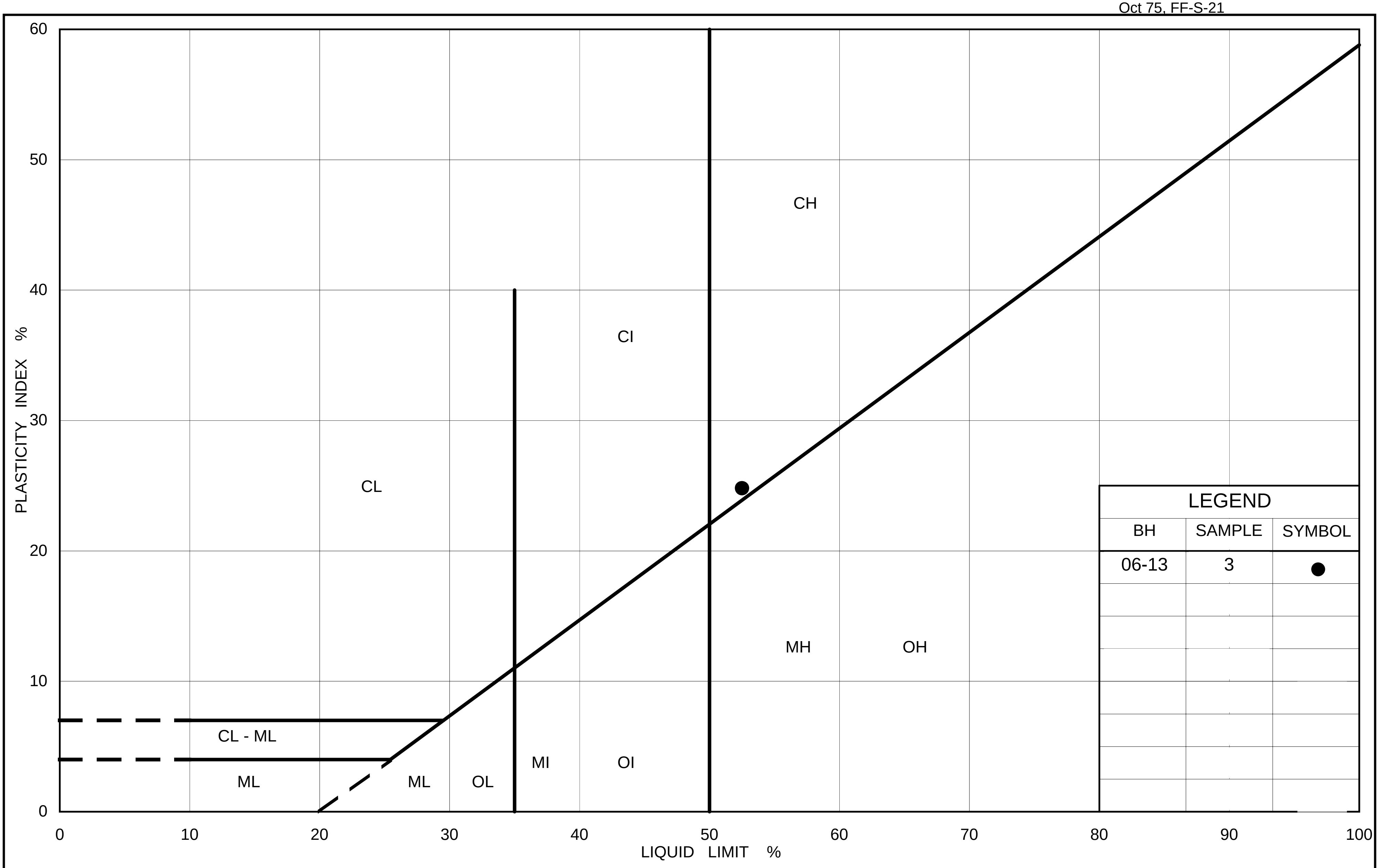
Borehole	Sample	Elevation (m)
—■— 06-11	4	289.5
—●— 06-17	3	291.5

Date: July 2007
Project: 06-1191-001-S

Golder Associates

Created by: SL
Checked by: AB

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Alluvium

FIGURE No. A-2

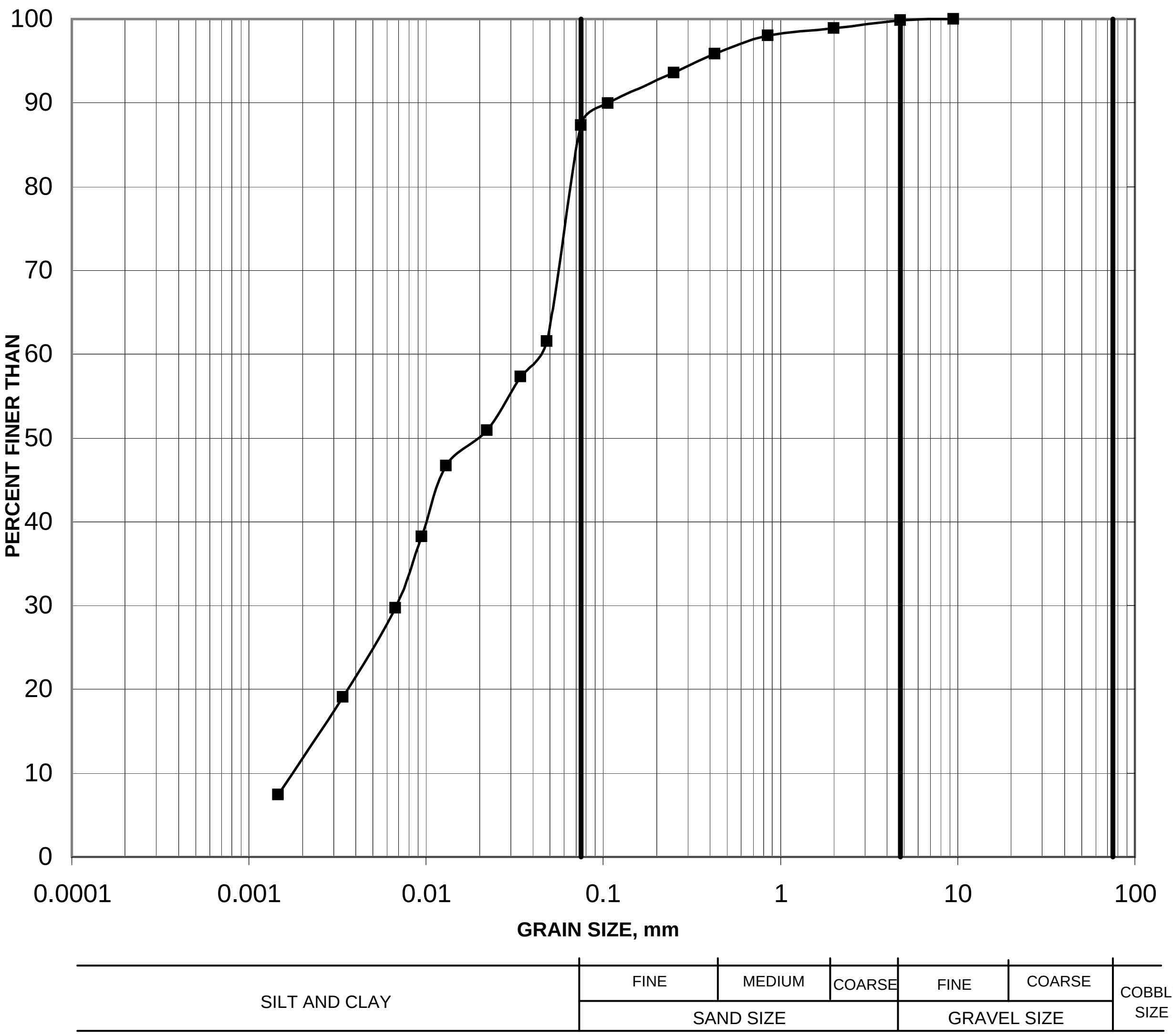
Project No. 06-1191-001-S

Checked: AB

GRAIN SIZE DISTRIBUTION

Alluvium

FIGURE A-3



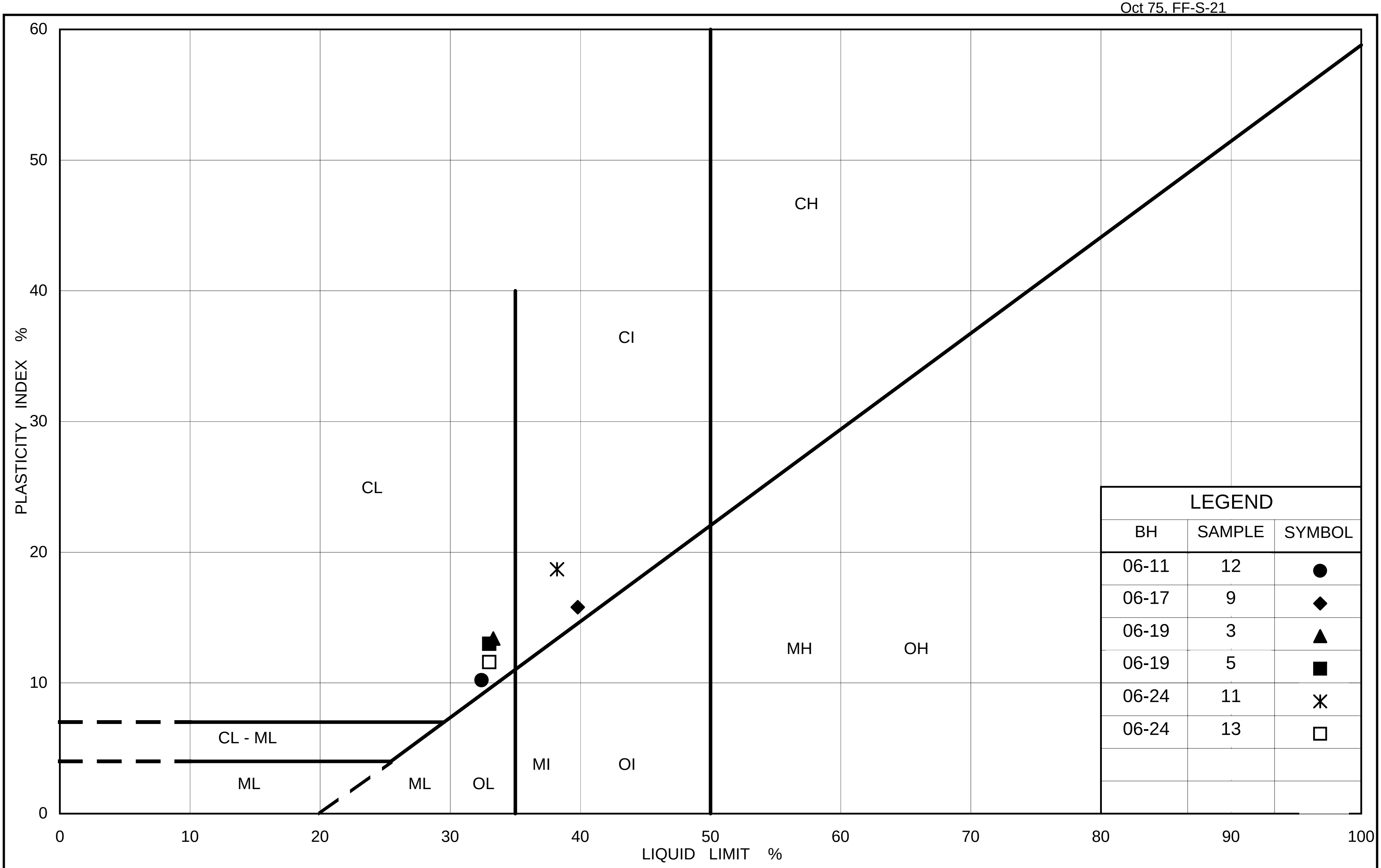
Borehole	Sample	Elevation (m)
—■— 06-22	3	280.0

Date: July 2007
Project: 06-1191-001-S

Golder Associates

Created by:	SL
Checked by:	AB

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
06-11	12	●
06-17	9	◆
06-19	3	▲
06-19	5	■
06-24	11	⋇
06-24	13	□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt (Land-Based Boreholes)

FIGURE No. A-4

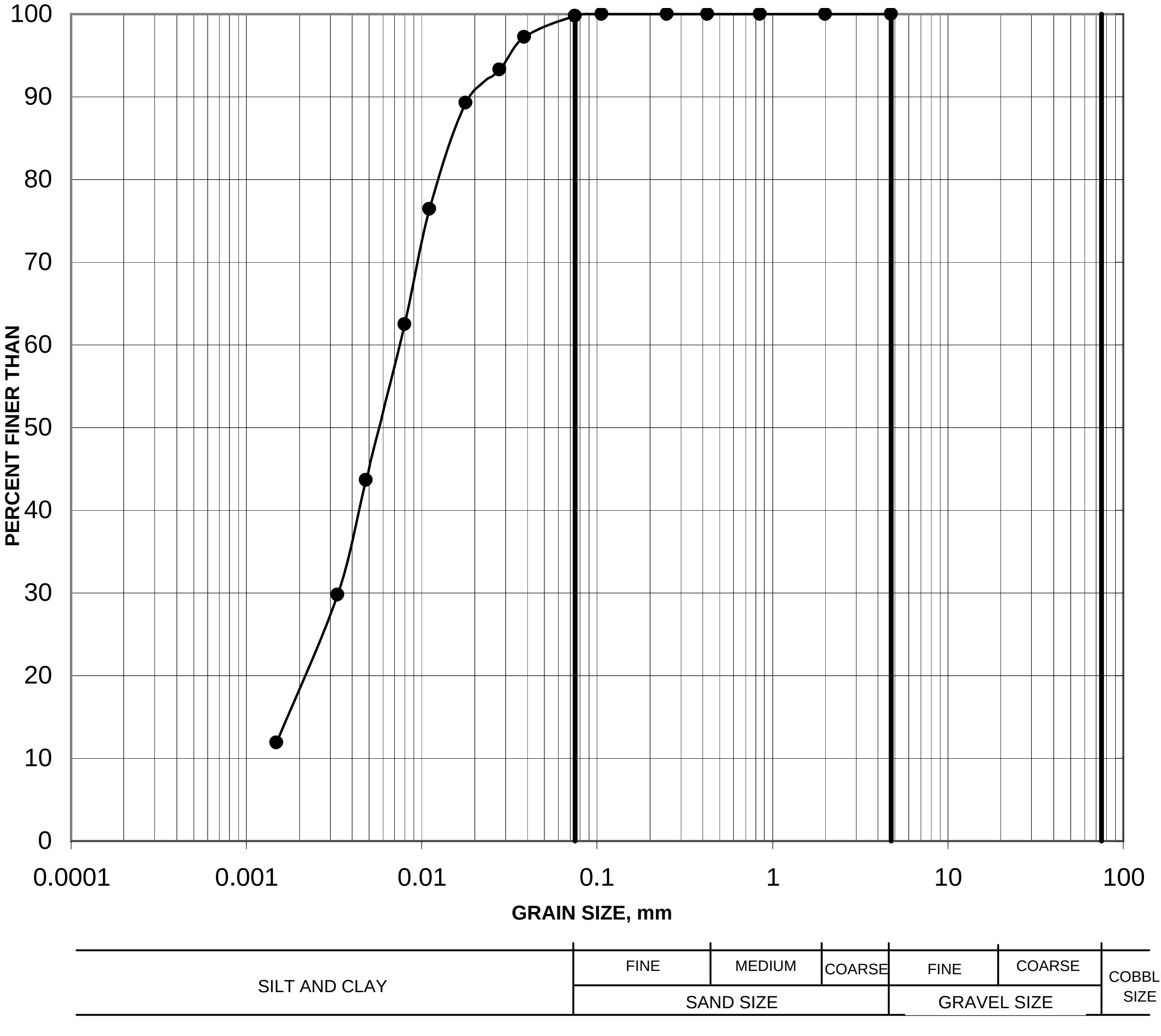
Project No. 06-1191-001-S

Checked: AB

GRAIN SIZE DISTRIBUTION

Clayey Silt (Land-Based Boreholes)

FIGURE A-5



	Borehole	Sample	Elevation (m)
—●—	06-19	5	283.7

Date: July 2007

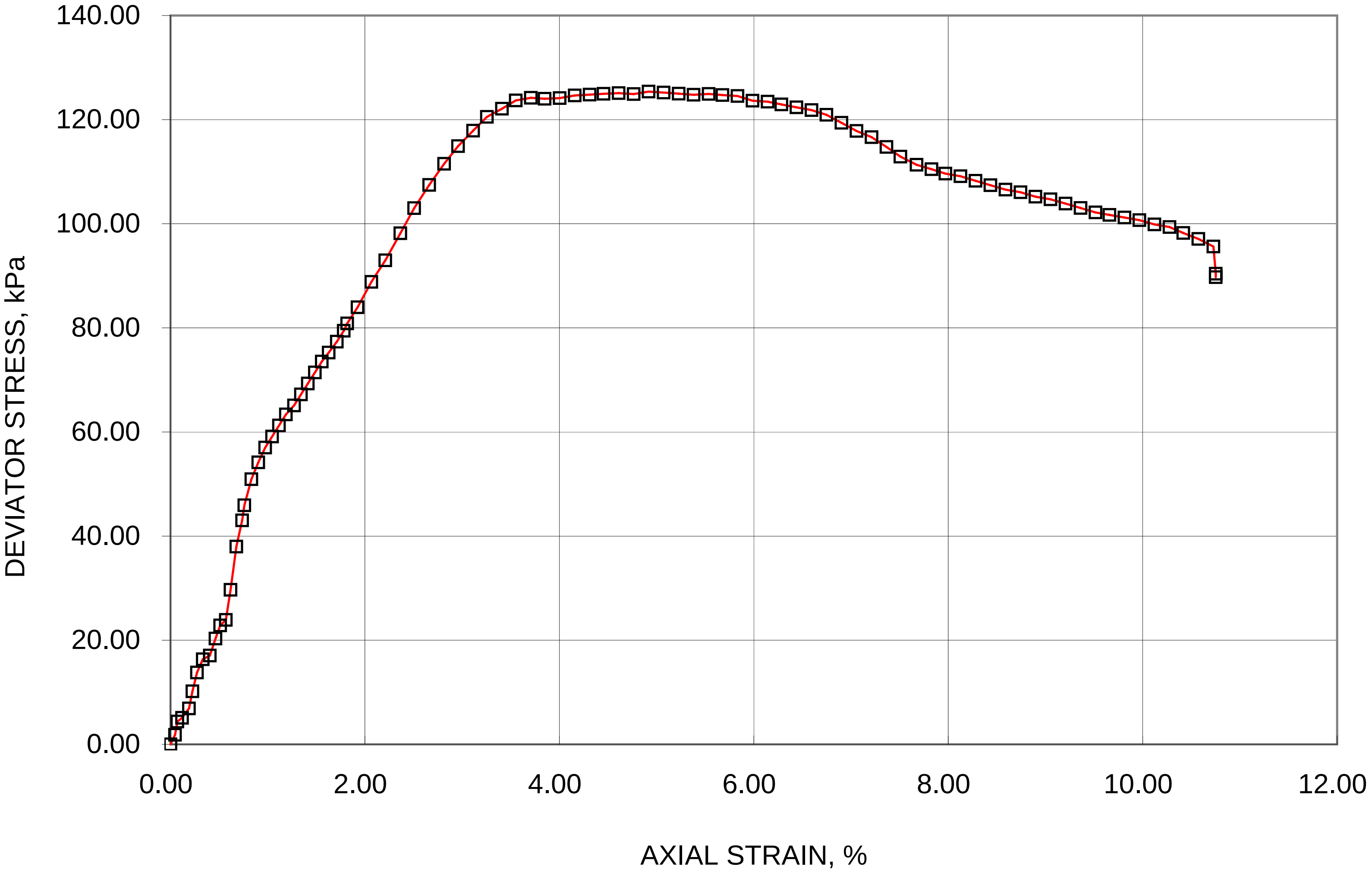
Project: 06-1191-001-S

Golder Associates

Created by:	SL
Checked by:	AB

Project No. 06-1191-001-S

BOREHOLE NUMBER 06-11 SAMPLE NUMBER 11 SAMPLE DEPTH, m 12.19-12.80



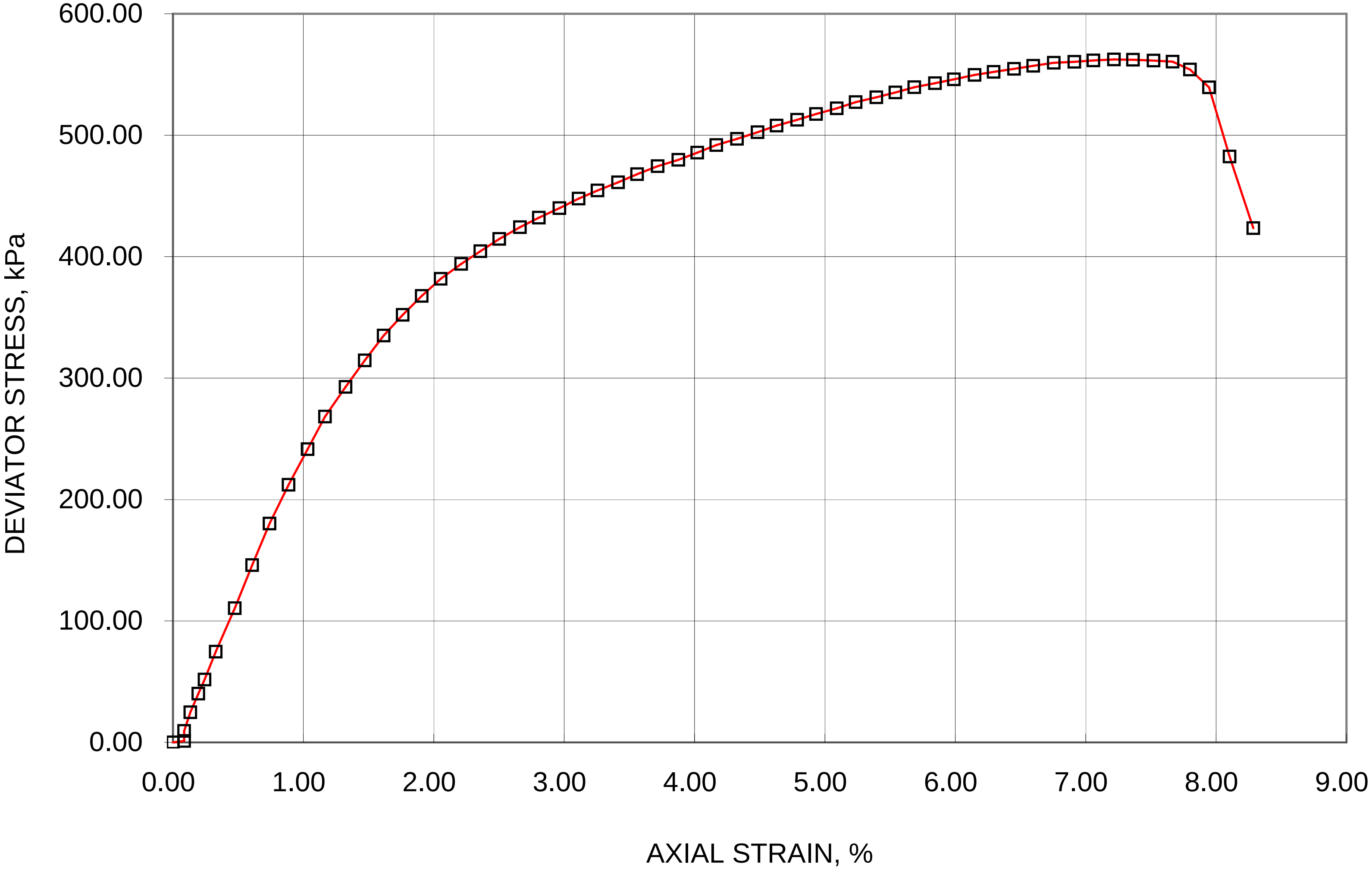
Checked: MM

UNCONFINED COMPRESSION TEST (UC)

FIGURE A-6

Project No. 06-1191-001-S

BOREHOLE NUMBER 06-18 SAMPLE NUMBER 7 SAMPLE DEPTH, m 4.57-5.18



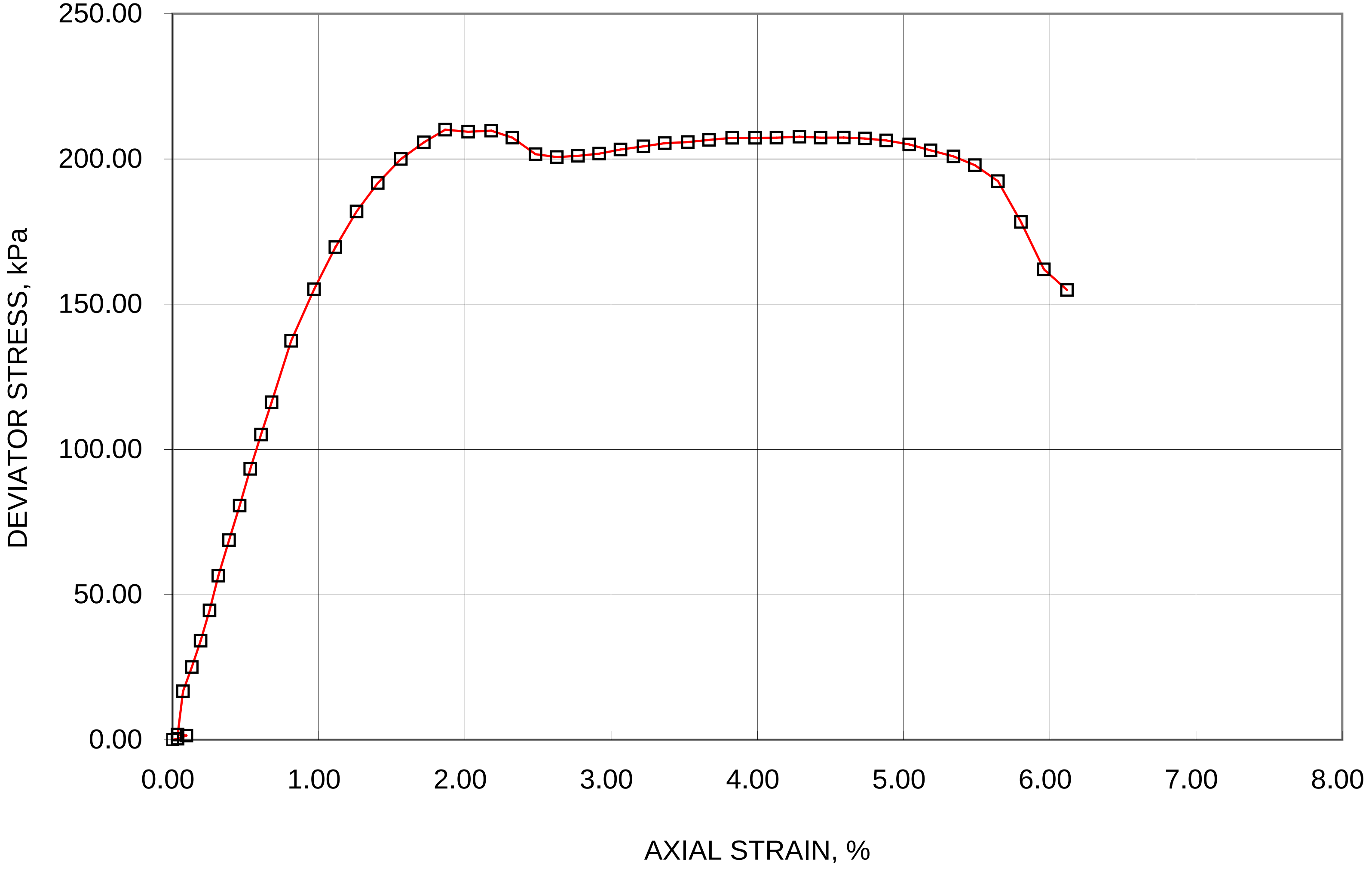
Checked: MM

UNCONFINED COMPRESSION TEST (UC)

FIGURE A-7

Project No. 06-1191-001-S

BOREHOLE NUMBER 06-19 SAMPLE NUMBER 6 SAMPLE DEPTH, m 4.57-5.18



Checked: MM

UNCONFINED COMPRESSION TEST (UC)

FIGURE A-8

OEDOMETER CONSOLIDATION SUMMARY

FIGURE A-9
Page 1 of 4

SAMPLE IDENTIFICATION

Project Number	06-1191-001-S	Sample Number	11
Borehole Number	06-11	Sample Depth, m	12.2-12.8

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	08/08/2006		
Date Completed	08/19/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	18.60
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	14.14
Area, cm^2	31.65	Specific Gravity, measured	2.64
Volume, cm^3	60.13	Solids Height, cm	1.038
Water Content, %	31.53	Volume of Solids, cm ^{3}	32.85
Wet Mass, g	114.08	Volume of Voids, cm ^{3}	27.28
Dry Mass, g	86.73	Degree of Saturation, %	100.3

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	1.900	0.830	1.900				
4.83	1.894	0.825	1.897	7	1.09E-01	6.54E-04	6.98E-06
9.55	1.888	0.819	1.891	16	4.74E-02	6.69E-04	3.11E-06
19.51	1.879	0.810	1.884	15	5.01E-02	4.76E-04	2.34E-06
38.71	1.866	0.798	1.873	15	4.96E-02	3.56E-04	1.73E-06
77.44	1.851	0.783	1.859	26	2.82E-02	2.04E-04	5.63E-07
154.78	1.831	0.764	1.841	21	3.42E-02	1.36E-04	4.56E-07
312.33	1.796	0.730	1.814	11	6.34E-02	1.17E-04	7.26E-07
620.72	1.755	0.691	1.776	12	5.57E-02	7.00E-05	3.82E-07
1239.28	1.708	0.645	1.732	19	3.35E-02	4.00E-05	1.31E-07
2481.22	1.655	0.594	1.682	20	3.00E-02	2.25E-05	6.60E-08
1239.28	1.665	0.604	1.660				
312.33	1.684	0.622	1.675				
77.44	1.702	0.640	1.693				
19.51	1.722	0.659	1.712				
4.83	1.738	0.674	1.730				

Note:
k calculated using cv based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.74	Unit Weight, kN/m^3	19.93
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	15.46
Area, cm^2	31.65	Specific Gravity, measured	2.64
Volume, cm^3	55.01	Solids Height, cm	1.038
Water Content, %	28.89	Volume of Solids, cm ^{3}	32.85
Wet Mass, g	111.79	Volume of Voids, cm ^{3}	22.15
Dry Mass, g	86.73		

Prepared By: LFG

Golder Associates

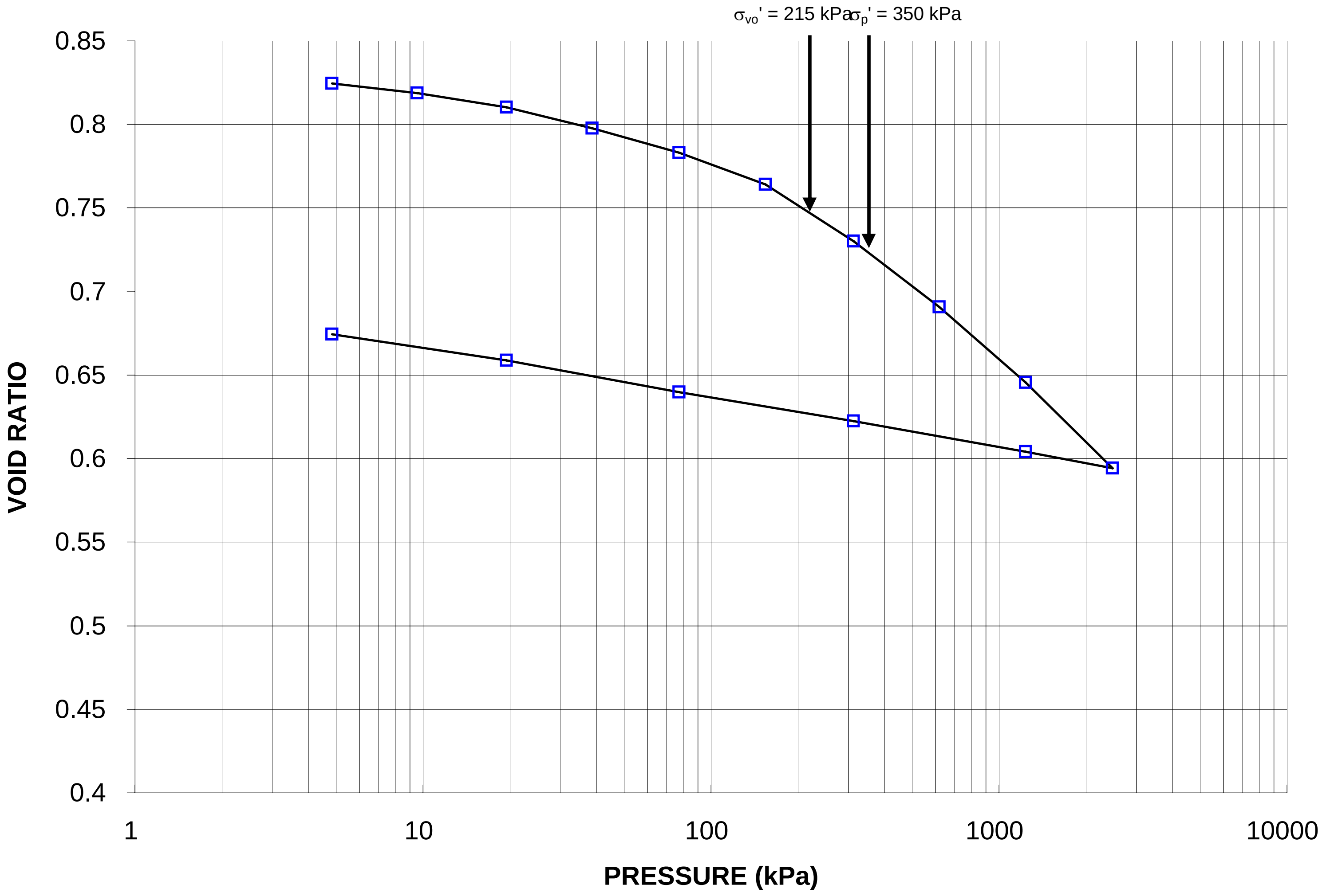
Checked By: MM

Project No. 06-1191-001-S

Prepared By: LFG Checked By:

Golder Associates

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 06-11 SA 11



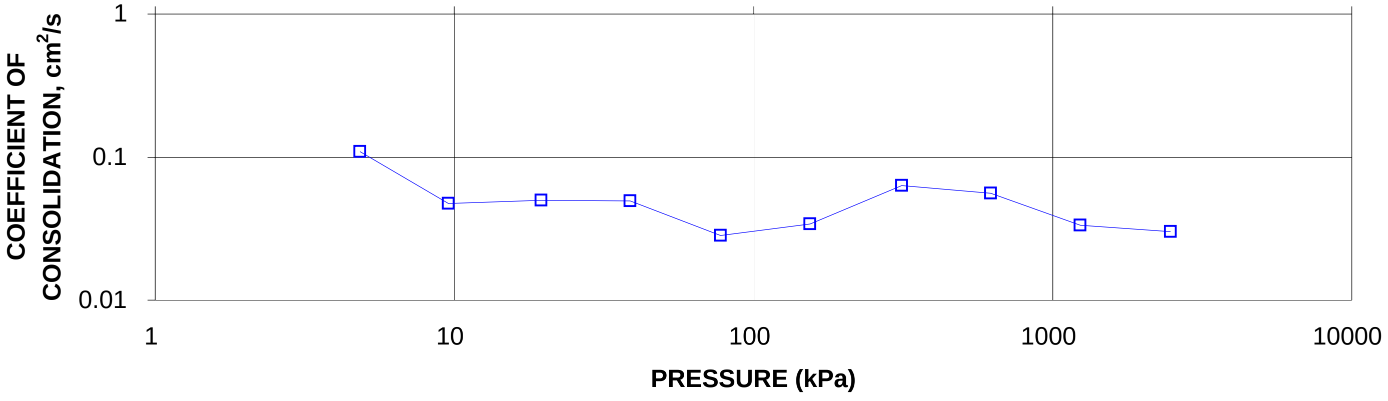
CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A-9
Page 2 of 4

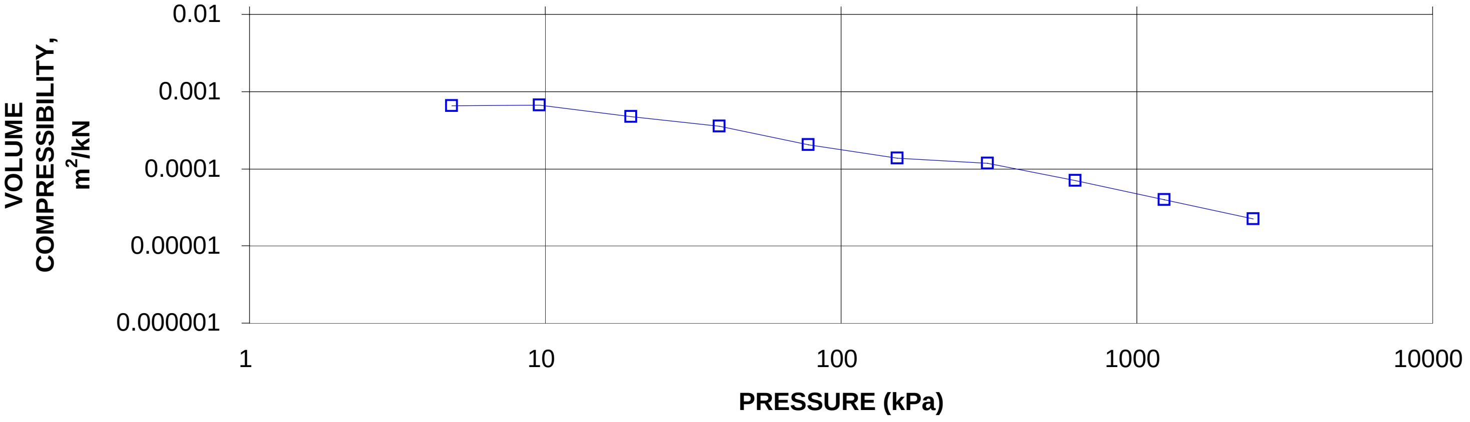
OEDOMETER CONSOLIDATION SUMMARY

FIGURE A-9
Page 3 of 4

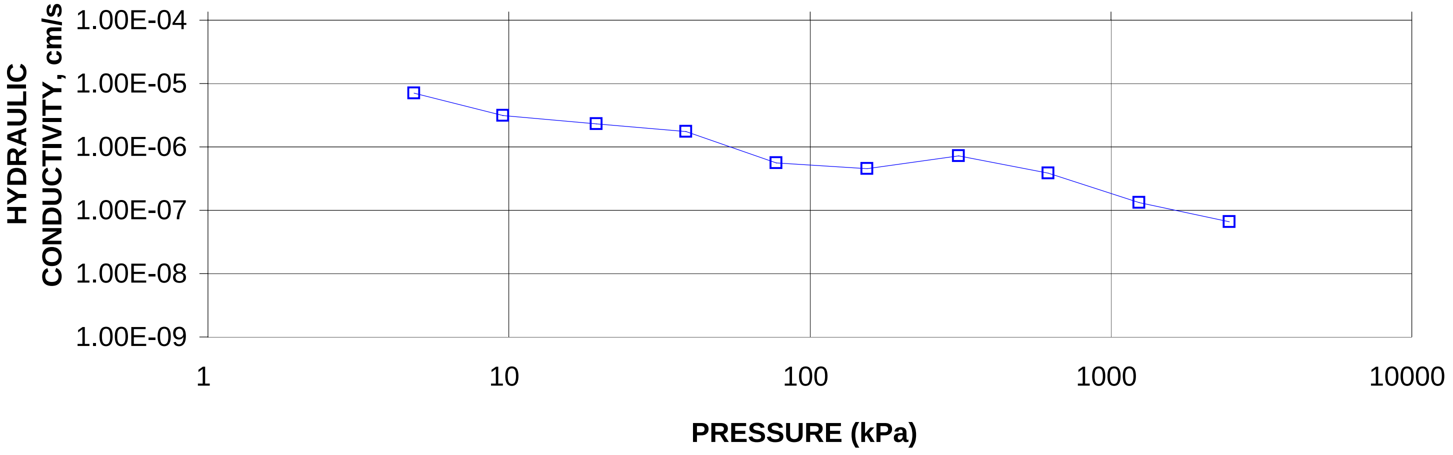
CONSOLIDATION TEST
CV cm^2/s VS PRESSURE (kPa)
BH 06-11 SA 11



CONSOLIDATION TEST
MV m^2/kN vs PRESSURE (kPa)
BH 06-11 SA 11



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 06-11 SA 11

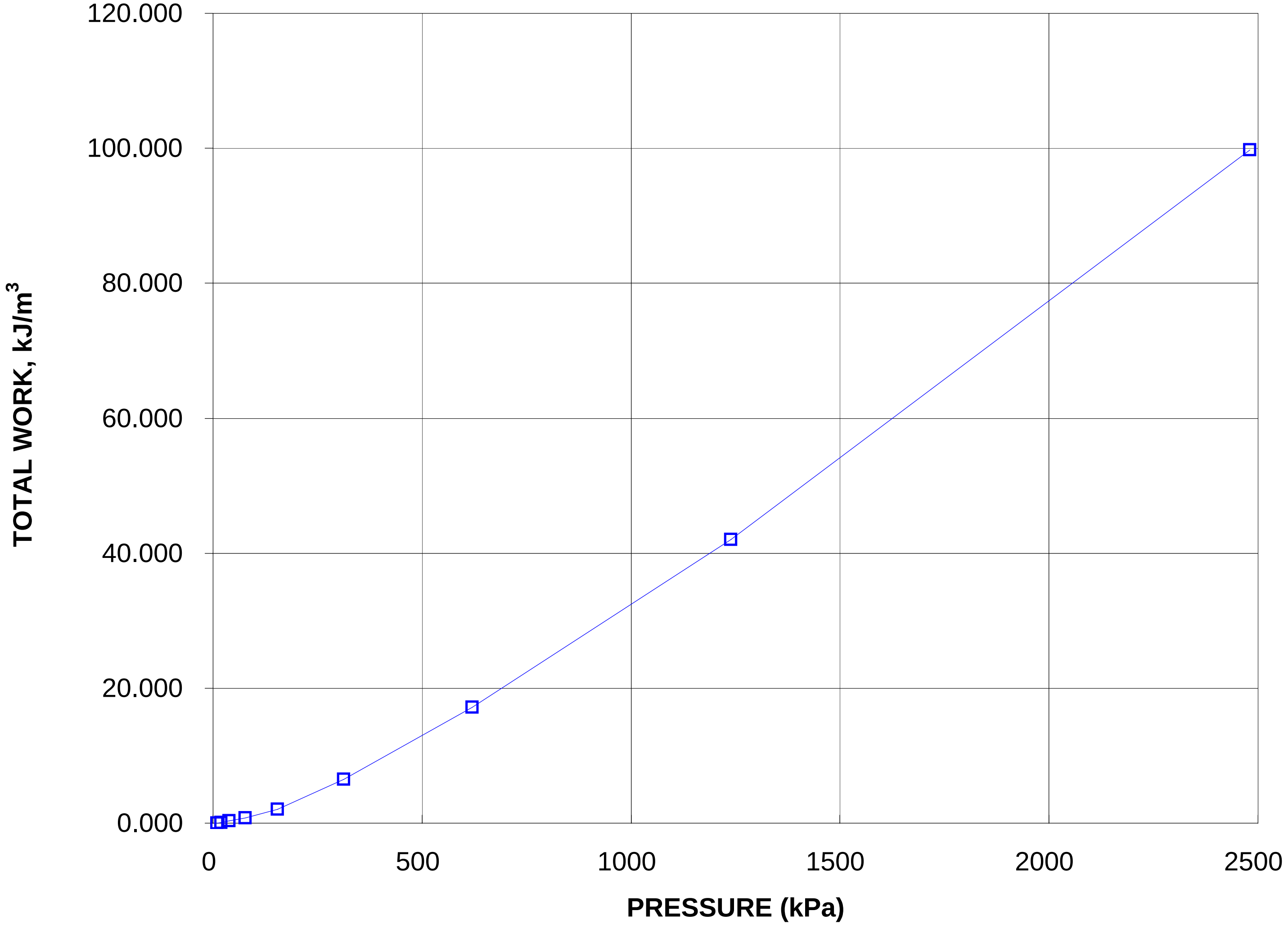


Project No. 06-1191-001-S

Prepared By: LFG Checked By:

Golder Associates

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH 06-11 SA 11



CONSOLIDATION TEST
TOTAL WORK VS. LOG PRESSURE

FIGURE A-9
Page 4 of 4

OEDOMETER CONSOLIDATION SUMMARY

FIGURE A-10
Page 1 of 4

SAMPLE IDENTIFICATION

Project Number	06-1191-001-S	Sample Number	6
Borehole Number	06-19	Sample Depth, m	4.6-5.2

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	8		
Date Started	08/08/2006		
Date Completed	08/19/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.92	Unit Weight, kN/m^3	18.66
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	14.38
Area, cm^2	31.67	Specific Gravity, measured	2.60
Volume, cm^3	60.65	Solids Height, cm	1.080
Water Content, %	29.73	Volume of Solids, cm ^{3}	34.21
Wet Mass, g	115.38	Volume of Voids, cm ^{3}	26.44
Dry Mass, g	88.94	Degree of Saturation, %	100.0

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	1.915	0.773	1.915				
4.85	1.910	0.768	1.913	5	1.55E-01	5.38E-04	8.18E-06
9.53	1.907	0.765	1.909	4	1.93E-01	3.35E-04	6.33E-06
19.40	1.902	0.761	1.905	3	2.56E-01	2.65E-04	6.64E-06
38.88	1.892	0.752	1.897	10	7.63E-02	2.63E-04	1.96E-06
77.52	1.885	0.745	1.889	7	1.08E-01	9.87E-05	1.04E-06
154.57	1.869	0.730	1.877	20	3.73E-02	1.09E-04	3.99E-07
309.24	1.839	0.703	1.854	10	7.29E-02	1.01E-04	7.18E-07
618.44	1.770	0.639	1.805	12	5.75E-02	1.17E-04	6.57E-07
1236.77	1.704	0.578	1.737	21	3.05E-02	5.57E-05	1.66E-07
2474.65	1.640	0.518	1.672	34	1.74E-02	2.70E-05	4.61E-08
1236.77	1.650	0.528	1.645				
309.24	1.668	0.544	1.659				
77.52	1.689	0.564	1.679				
19.40	1.711	0.584	1.700				
4.85	1.732	0.603	1.722				

Note:
k calculated using cv based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.73	Unit Weight, kN/m^3	20.35
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	15.90
Area, cm^2	31.67	Specific Gravity, measured	2.60
Volume, cm^3	54.85	Solids Height, cm	1.080
Water Content, %	28.00	Volume of Solids, cm ^{3}	34.21
Wet Mass, g	113.84	Volume of Voids, cm ^{3}	20.64
Dry Mass, g	88.94		

Prepared By: LFG

Golder Associates

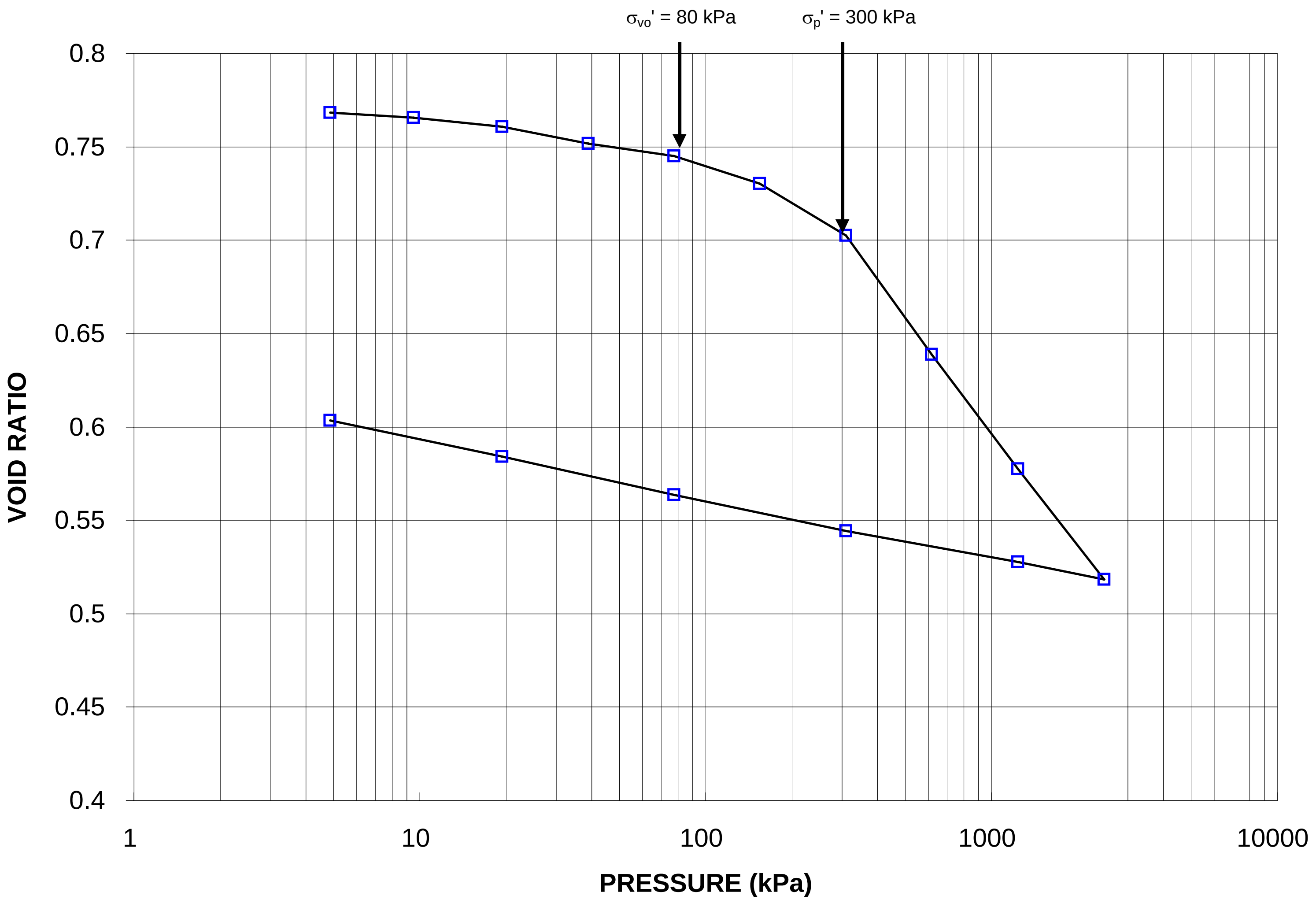
Checked By: MM

Project No. 06-1191-001-S

Prepared By: LFG Checked By: MM

Golder Associates

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 06-19 SA 6



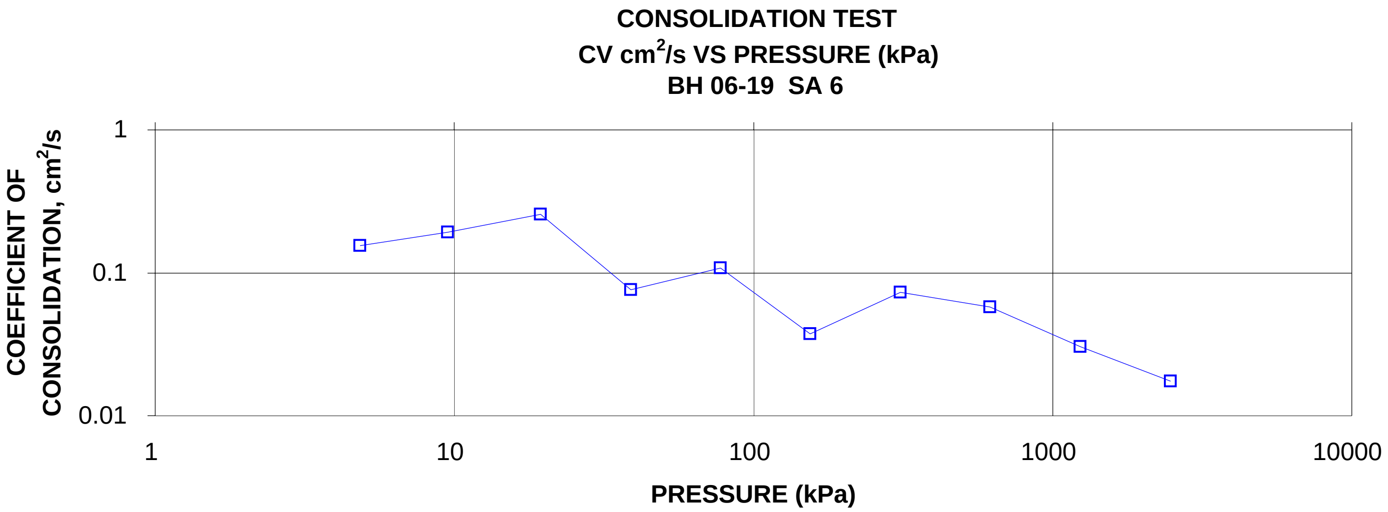
CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

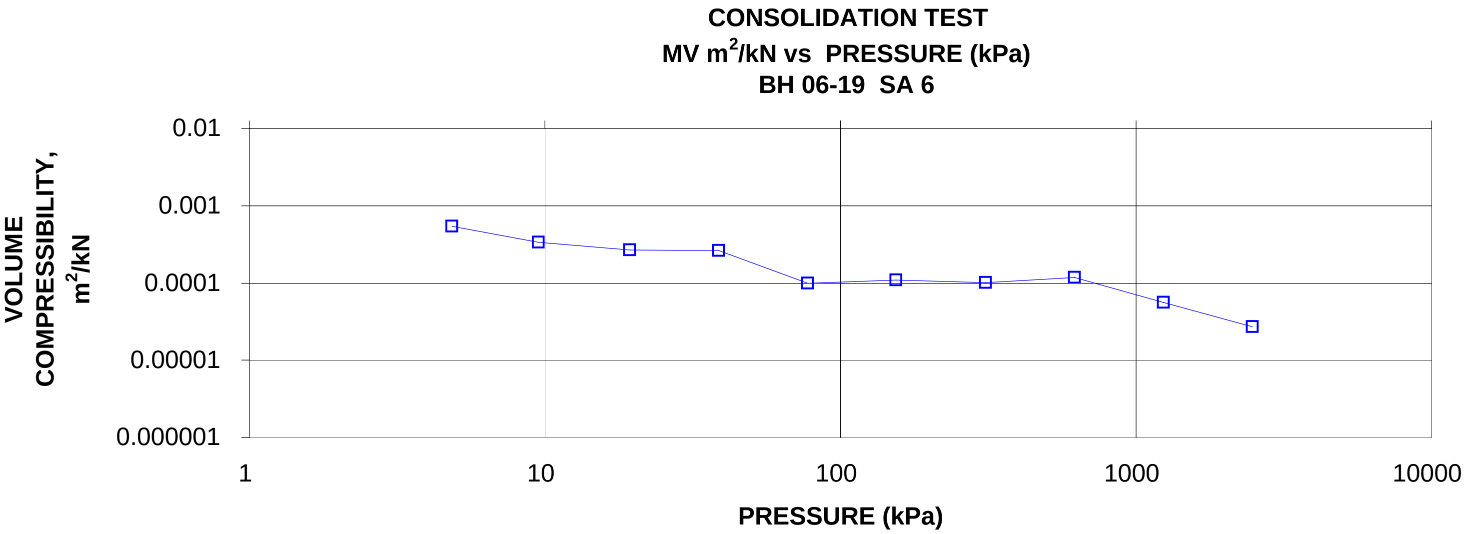
FIGURE A-10
Page 2 of 4

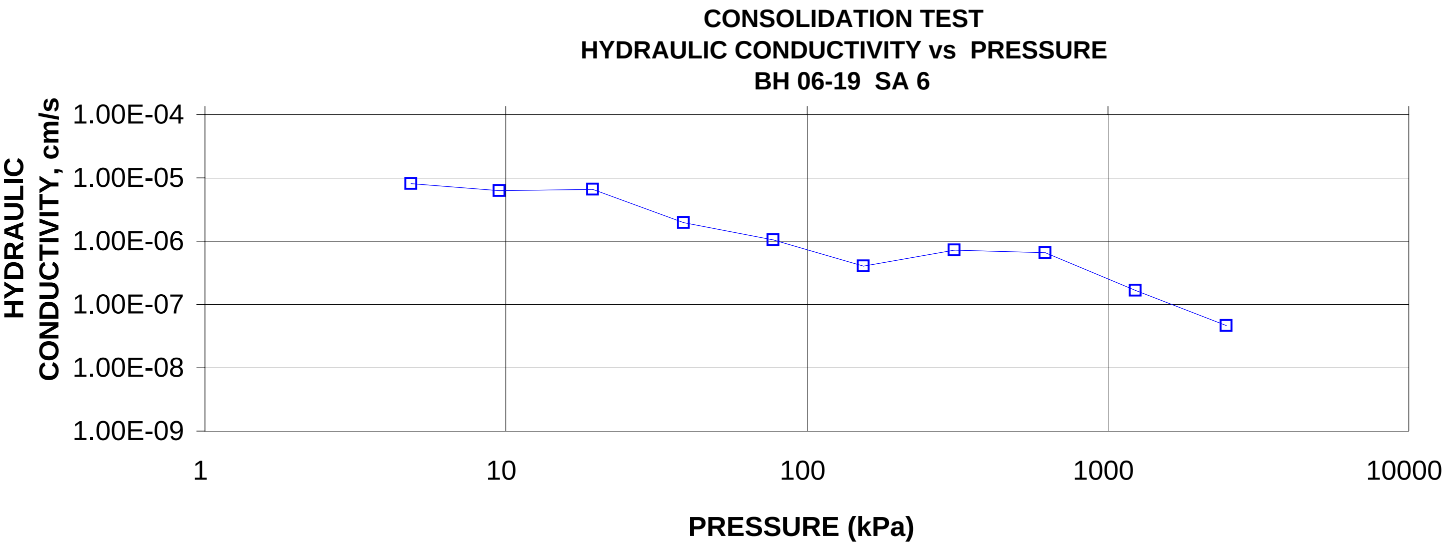
OEDOMETER CONSOLIDATION SUMMARY

FIGURE A-10

Page 3 of 4







Project No. 06-1191-001-S

Prepared By: LFG

Golder Associates

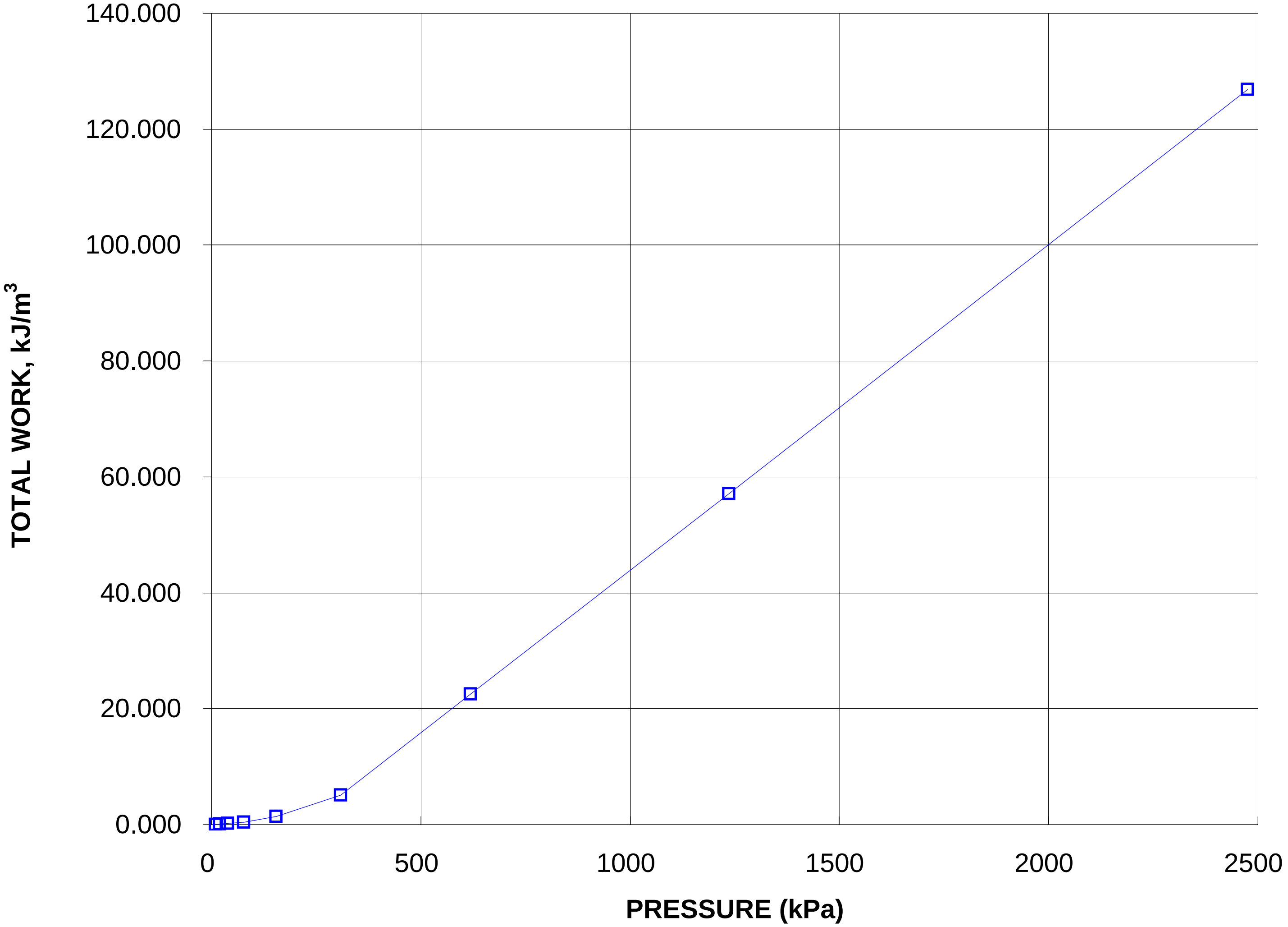
Checkey By: MM

Project No. 06-1191-001-S

Prepared By: LFG Checked By: MM

Golder Associates

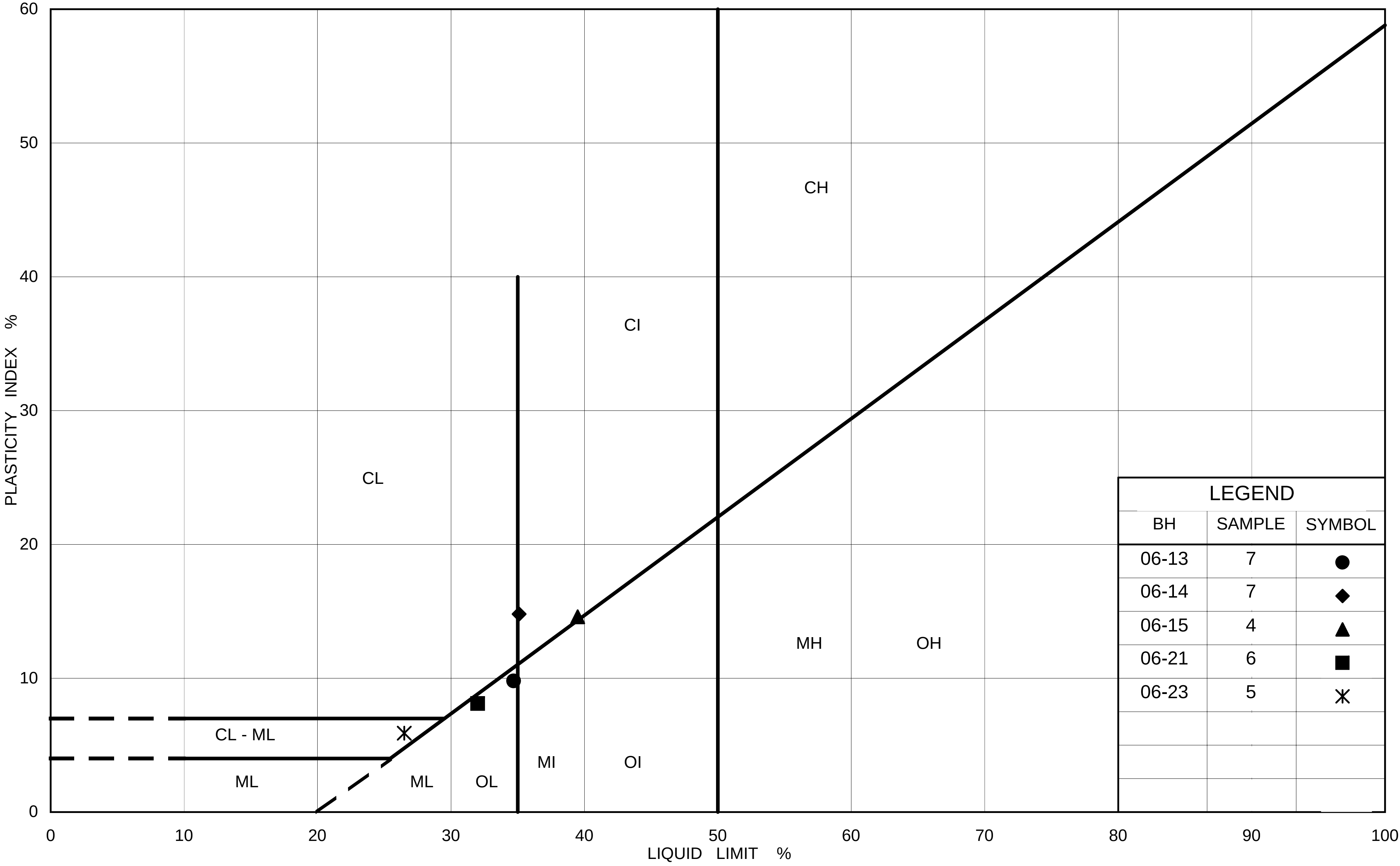
CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH 06-19 SA 6



CONSOLIDATION TEST
TOTAL WORK, kJ/m3 vs PRESSURE

FIGURE A-10
Page 4 of 4

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
06-13	7	●
06-14	7	◆
06-15	4	▲
06-21	6	■
06-23	5	⋇

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt (Water-Based Boreholes)

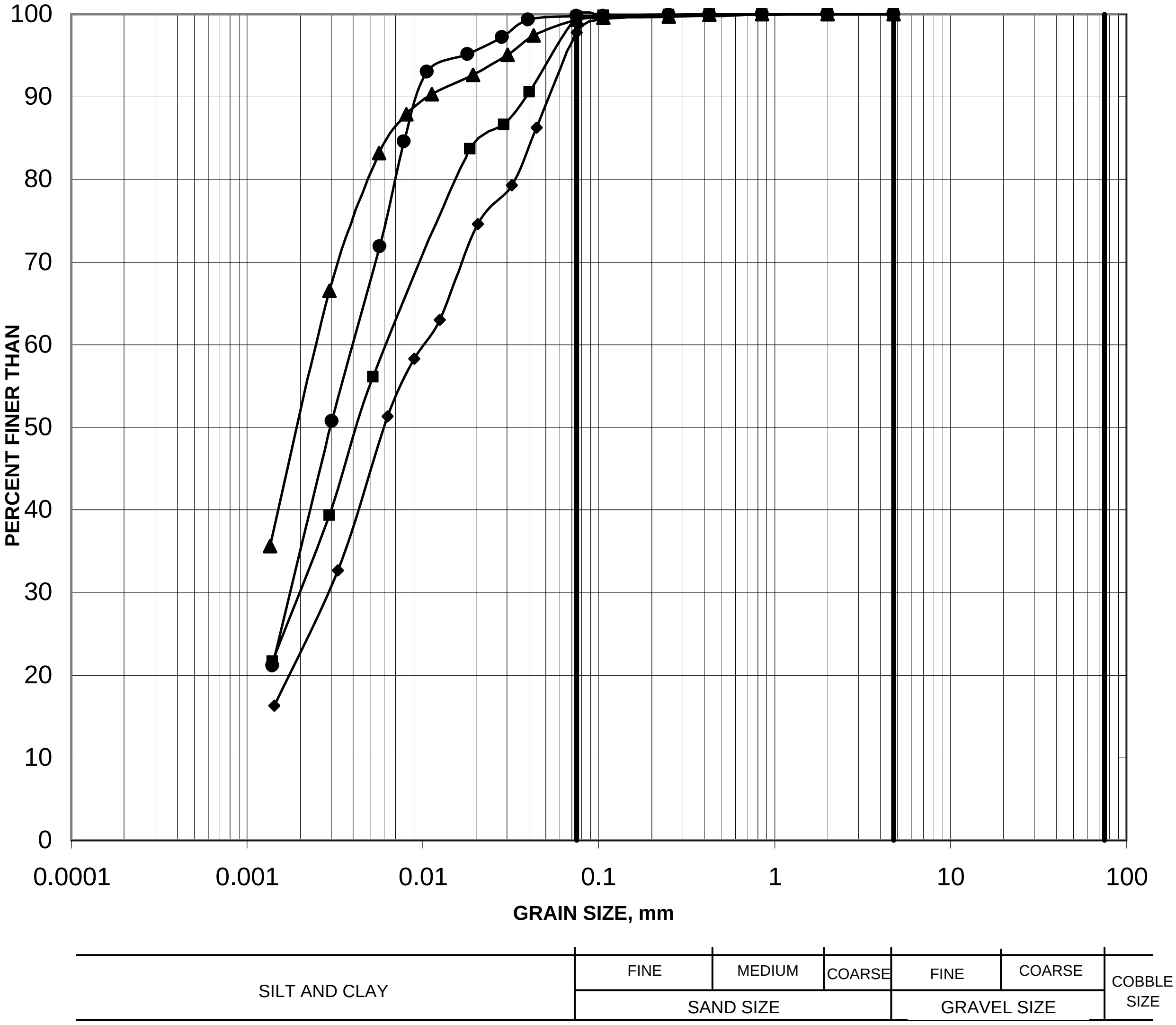
FIGURE No. A-11

Project No. 06-1191-001-S

Checked: AB

GRAIN SIZE DISTRIBUTION
Clayey Silt (Water-Based Boreholes)

FIGURE A-12



Borehole	Sample	Elevation (m)
—■— 06-13	6	277.0
—◆— 06-14	5	277.7
—▲— 06-15	4	276.9
—●— 06-21	4	275.4

Date: July 2007
Project: 06-1191-001-S

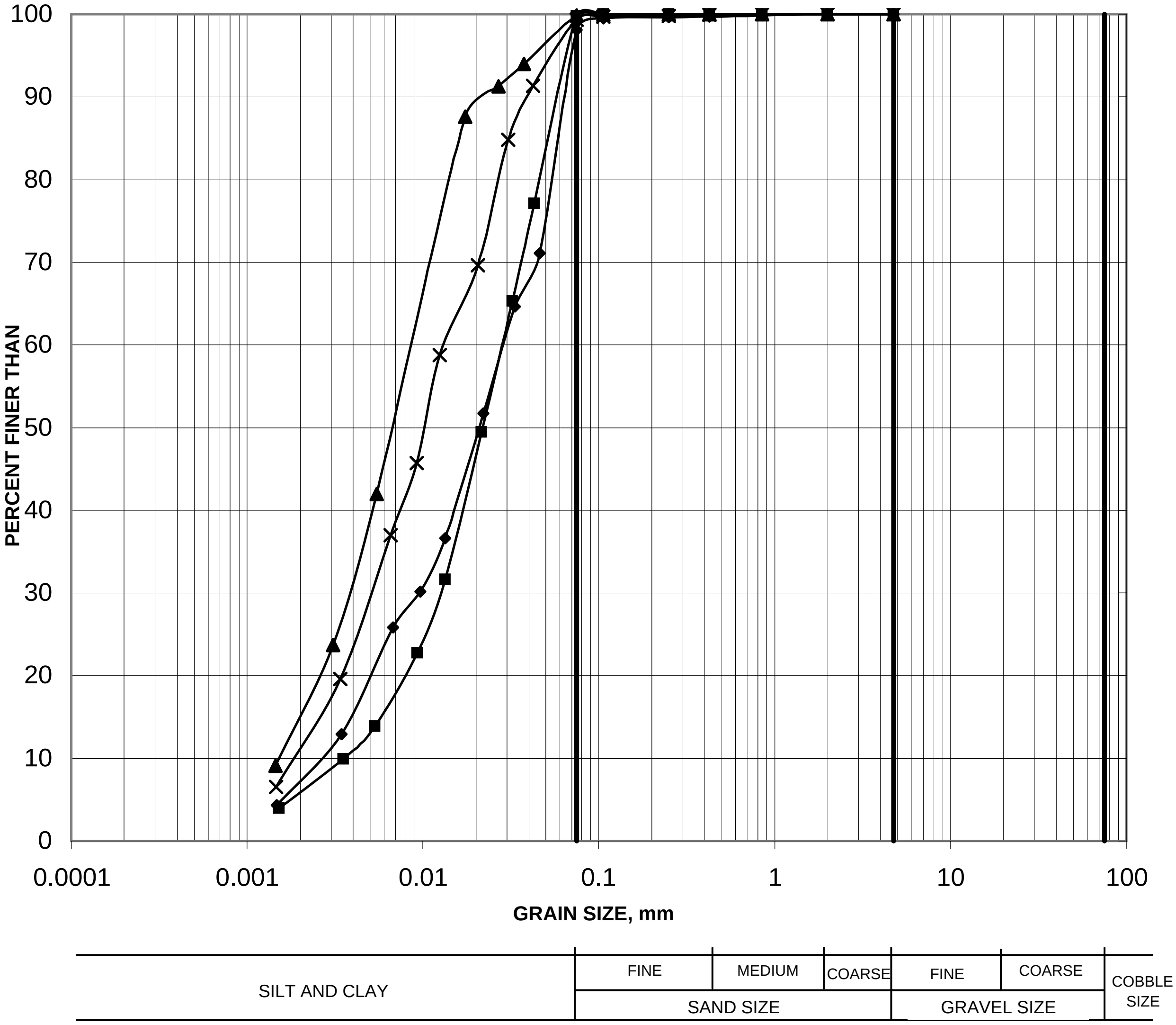
Golder Associates

Created by: SL
Checked by: AB

GRAIN SIZE DISTRIBUTION

Silt

FIGURE A-13



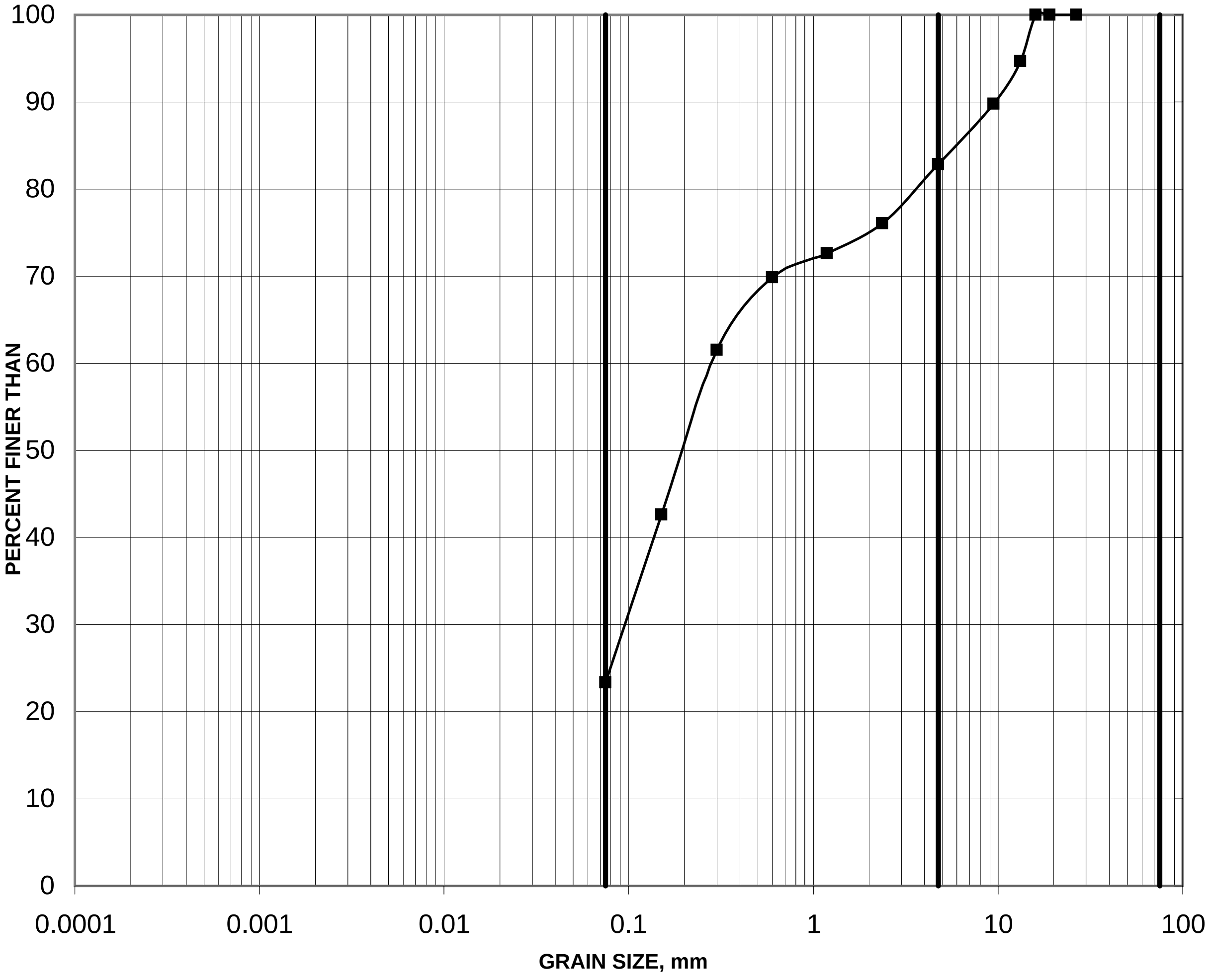
Borehole	Sample	Elevation (m)
■ 06-19	7	281.4
◆ 06-20	6	277.8
▲ 06-22	10	270.1
× 06-23	10	271.6

Date: July 2007
Project: 06-1191-001-S

Golder Associates

Created by: SL
Checked by: AB

GRAIN SIZE DISTRIBUTION Gravelly Sand to Silty Sand	FIGURE A-14



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

Borehole	Sample	Elevation (m)
—■— 06-16	10	273.1

Date: July 2007
Project: 06-1191-001-S

Golder Associates

Created by:	SL
Checked by:	AB

APPENDIX B

NON-STANDARD SPECIAL PROVISIONS

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 for the Vernon Narrows Southbound Lane structure widening. Piles will be driven through cobbles and boulders prior to seating on bedrock.

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.

UNWATERING STRUCTURE EXCAVATION – Item No. 75

Special Provision

902.01 SCOPE

Section OPSS 902.01 of OPSS 902 is amended by the addition of the following:

As part of the work under this item, the Contractor shall unwater pier cofferdams.

902.04 SUBMISSION AND DESIGN REQUIREMENTS

Section OPSS 902.04 of OPSS 902 is amended by the addition of the following:

At least two weeks prior to the commencement of cofferdam construction, the Contractor shall submit to the Contract Administrator, for information purposes only, four (4) sets of drawings of the unwatering system showing the unwatering methodology and measures in place to complete the work in the dry.

902.07 CONSTRUCTION

Section OPSS 902.07.06 of OPSS 902 is modified by the deletion of the second paragraph and replacement with the following:

Control of water shall be according to OPSS 518 and the following. Water from unwatering of pier and abutment cofferdams / excavations, or from drying out tube pile casing excavated materials, for the Vernon Narrows NBL Bridge widening, shall be contained in a settling pond or sediment basin located a minimum of 30 metres from the edge of water. The settling pond or sediment basin shall be of adequate size / design to allow settling out of sediments prior to any water flowing back to the Vernon Narrows watercourse. The discharge channel to the watercourse shall be vegetated.

PRECAST CONCRETE COFFER DAMS – Item No. 102

Special Provision

1.0 SCOPE

As part of the work under this item, the Contractor shall design, supply, and install precast concrete coffer dams for each pier to construct the pier pile caps in the dry inclusive of sealing the joint around the steel tube pile with non-shrink underwater grout. All work as shown on the Contract Drawings.

2.0 DEFINITION

Stamped:	Refers to drawings or details that have been reviewed and stamped "Conforms With Contract Documents". The stamp shall include the date and signature of the Quality Verification Engineer (QVE).
Quality Verification Engineer (QVE):	An Engineer licensed to practice in the Province of Ontario who has a minimum of five (5) years of experience in the field of design and/or construction of cofferdams. The Contractor shall retain the QVE to ensure conformance with the contract document.
Coffer Dam Design Engineer:	An Engineer licensed to practice in the Province of Ontario who has a minimum of five (5) years of experience in the field of design and/or construction of bridges. In addition, the Coffer Dam Design Engineer shall have had responsible experience in the design of at least 5 other coffer dams. The Contractor shall retain the Coffer Dam Design Engineer to ensure conformance with the contract documents and issue certificate(s) of conformance for the design.

3.0 SUBMISSION AND DESIGN REQUIREMENTS

Design of concrete cofferdams shall be in accordance with CAN/CSA-S6-00 and shall be constructed in accordance with OPSS 904.

Submission of Shop Drawings

All shop drawings submissions shall bear the seal and signature of the Coffer Dam Design Engineer. For the purpose of this subsection only, the Coffer Dam Design Engineer shall not be permitted to carry out the work of the Quality Verification Engineer.

The Contractor shall submit to the Quality Verification Engineer shop drawings for review and stamping.

At least one (1) week prior to commencement of fabrication, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of stamped shop drawings. These drawings shall include: coffer dam dimensions, reinforcing steel schedules, method of casting, lifting point locations, temporary support requirements after fabrication and prior to erection, material specifications, embedded hardware and connection details for temporary erection piles and sleeves, and all other pertinent details.

Submission of Installation Procedures

The Contractor shall, at least three (3) weeks prior to the commencement of the coffer dam installation, submit to the QVE for review, four sets of drawings and erection procedures for review and approval. The erection procedures shall:

- Schematically show the location of equipment for the sequence of the installation including location of barges/pontoons, installation of spuds to stabilize the cofferdam, temporary support frames inside the cofferdams for installation of the steel tube casings etc.;
- Provide a step by step procedures for erection;
- Identify the location of each coffer dam to be used;
- Methods to determine correct positioning and control points to be used to monitor the alignment and elevation of the coffer dam during installation.

At least one (1) week prior to commencement of erection, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of QVE stamped and signed installation procedures.

Certificates of Conformance for Fabrication

The QVE shall inspect the reinforcing steel prior to placing concrete to construct the precast concrete cofferdams and submit to the Contract Administrator a Certificate of Conformance sealed and signed by the QVE. The certificate shall state that the work has been carried out in general conformance with the stamped and signed shop drawings.

Certificates of Conformance for Installation

Upon completion of the installation of the cofferdams, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the QVE. The certificate shall state the installation has been carried out in general conformance with the installation procedures and the contract documents and shall state the coffer dam was measured and inspected for defects prior to installing the cofferdam in the water; inspected prior to installing reinforcing steel for the pile cap footing; and the coffer dam is located and positioned in the correct location and elevation.

The Contractor will note that several Certificates of Conformance may be required, each to coincide with each coffer dam installation.

4.0 MATERIALS

The non-shrink grout shall be an approved DSM 9.10.35 non-shrink grout. The anti-washout agent shall be used with the non-shrink grout for the grouting the steel tube piles to the concrete coffer dam. The anti-washout agent shall be one of the following proprietary products:

Sikament 100 SC Anti-washout additive for underwater concrete/grouts
Sika Canada Inc.
970 Verbena Road
Mississauga, Ontario
L5T 1T6, Canada

Mr. Greg Dolenc
Phone: 416-795-3177
Mobile: 416-573-7223

Rheomac UW 450 Liquid anti-washout admixture
Masterbuilder Technologies
1800 Clark Boulevard
Brampton, Ontario

L6T 4M7, Canada

Mr. Eliseo Conciatori
Phone: 905-792-2012
Mobile: 416-567-7665

The Contractor shall provide the following information from the manufacturer for non-shrink grout and anti-washout agent:

- data sheets for the non-shrink grout and anti-washout agent;
- technical information that proves that the non-shrink grout and anti-washout agent are compatible; and
- installation procedures.

5.0 CONSTRUCTION

At least one (1) week prior to commencement of concrete cofferdams, the Contractor shall notify the Contract Administrator of the proposed location to fabricate the coffer dams on site. The Contractor is responsible for ensuring all permits or permissions to enter property are obtained if required. The chosen site must be readily accessible for inspections by the Contract Administrator. All site preparation (levelling, access, security etc.) and the subsequent reinstatement/restoration is the responsibility of the Contractor at no additional cost to the Ministry.

Construction of the precast concrete coffer dams shall be in accordance with the requirements of OPSS 904 and careful consideration shall be given to timing of the work to ensure that the prefabricated concrete coffer dams are properly cured and ready for erection on the scheduled dates.

Minimum dimensions for the inside face of the coffer dam are given on the contract drawings. The Contractor shall verify the dimensions for the inside face of the coffer dam to achieve the minimum offset to drive the steel tube piles and driving steel H-piles.

Grout the gap between the steel tube piles and the precast concrete coffer dam with underwater non-shrink grout. Anti-washout agent for grout shall be used in accordance with the specifications of the manufacturer.

The Contractor shall stabilizing and support precast coffer dam until the concrete in pile cap has attained a compressive strength of 25 MPa.

Footing (pile cap) construction below the water level must be carried out in the dry.

6.0 BASIS OF PAYMENT

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

H-PILES - Item No.
TUBE PILES – 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 Vernon Narrows Southbound Lane structure for the foundation elements, the Contactor shall be alerted that the overburden soils consist of gravelly sand containing cobbles and boulders. In addition, the soils will be susceptible to cave-in, sloughing and heaving due to groundwater pressures.

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.