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REPORT ON

FOUNDATION INVESTIGATION AND DESIGN
DETAIL DESIGN
VERNON LAKE NARROWS
REHABILITATION AND WIDENING OF NORTHBOUND STRUCTURE
HIGHWAY 11
G.W.P 5189-05-00, SITE NO. 42-018
MINISTRY OF TRANSPORTATION, ONTARIO
HUNTSVILLE, ONTARIO

Submitted to:

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GEOCRES NO.   31E-268

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December 15, 2006 06-1191-001-N

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PART A

FOUNDATION INVESTIGATION REPORT
VERNON LAKE NARROWS
REHABILITATION AND WIDENING OF NORTHBOUND STRUCTURE
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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 widening of the existing northbound lane 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. 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 northbound lane (NBL) bridge erected in 1977 will be widened by about 9 m towards the median (west) side. The existing structure has seven spans, with a total of six in-water piers. The existing bridge is founded on piles driven to bedrock. At the piers, each pile was driven within a 1:8 battered steel casing installed inside a pre-cast concrete cofferdam.

The highway grade is at about Elevation 294 m and Elevation 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

3.1 Foundation Investigation

The fieldwork at the bridge site was carried out in two stages: four boreholes (BH06-1, BH06-2, BH06-9 and BH06-10) were drilled on land; and six boreholes (BH06-3 to BH06-8) were drilled over-water. The land-based work was carried out between June 20 and July 4, 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. A single Dynamic Cone Penetration Test was advanced in close proximity to BH06-1 as shown in the Record of Borehole sheet for BH06-1.

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. Tri-cone methods were used to advance the boreholes at some locations. Soil samples were obtained at intervals ranging from 0.75 m to 1.5 m in 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. Soil samples were obtained at intervals ranging from 0.75 m to 1.5 m in depth, from the tip of the casing.

The land-based boreholes were advanced to depths ranging from 10.1 m to 19.7 m below the existing ground surface. The water-based boreholes were advanced to depths ranging from 18.1 m to 26.1 m below the water surface at the time of drilling. Approximately 3 m of rock core was obtained from eight of the ten boreholes drilled at this site.

The groundwater conditions in the open boreholes were observed during the drilling operations and piezometers were installed in two land-based boreholes, BH06-2 and BH06-9 on 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 the gravelly sand stratum at the south and north abutment boreholes, respectively. The boreholes and piezometers (after the last water level was obtained) were backfilled with bentonite and/or cement-bentonite grout as per Ontario Reg. 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. One one-dimensional consolidation (oedometer) and three unconfined compression tests were carried out on Shelby Tube samples from two 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 water main, which crosses the bridge alignment under the lake channel, was coordinated by LEA.

The locations of the widened 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 Drawing 1.

In general, the subsoils in the land-based boreholes consisted of embankment fill underlain by silty clay to clayey silt, silt, 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 0.9 m and 4.6 m. The subsoils generally consisted of a clayey silt alluvium layer underlain by deposits of clayey silt, silt, and gravelly sand overlying bedrock. The gravelly sand deposit contained cobbles and boulders and, at one location, was underlain by a layer of cobble and boulders in a sand matrix immediately overlying the bedrock. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

Boreholes BH06-1 and BH06-10 were advanced at the approaches to the bridge structure within the existing northbound lanes. Asphalt, approximately 150 mm thick was encountered overlying a 100 mm to 150 mm thick layer of sand and gravel road base fill. Boreholes BH06-2 and BH06-9 were advanced in the median adjacent to the existing abutments. A 100 mm thick layer of sandy topsoil was encountered at the ground surface in BH06-2. About 1.4 m of cobbles and boulders (riprap type of material) were encountered from the ground surface in BH06-9.

Underlying the road base materials and the topsoil and riprap, the land-based boreholes penetrated a layer of granular fill consisting of sand, silty sand, and/or silt. The fill thickness ranged from 2.2 m to 3.3 m on the south side of the bridge and between 4.8 m and 7.6 m on the north side of the bridge. Occasional cobbles were noted within the fill materials in boreholes BH06-1 and BH06-10.

SPT ‘N’ values measured within the fill ranged between 2 and 53 blows per 0.3 m of penetration, indicating a very loose to very dense relative density. In general, based on the average ‘N’ values, the fill is considered to be compact to dense.

The natural water content measured on samples of the fill ranged between 4 percent and 22 percent.

In Borehole BH06-8, a 3.2 m thick deposit of brown sand fill was encountered at the lake bed at Elevation 283.2 m. One SPT ‘N’ value measured within this fill was 5 blow per 0.3 m of penetration indicating that this material is loose. It is postulated that this material comprises the granular backfill after the alluvium was removed near the toe of the slope. The natural water content measured on one sample of this fill was 24 percent.

4.2.2 Alluvium

In boreholes BH06-3, and BH06-5 to BH06-7, alluvium was encountered at the bottom of the lake bed. In boreholes BH06-3, BH06-5 and BH06-6, the alluvium consisted of silty sand and sandy gravel or organic silt and was between 0.1 m and 0.4 m thick. In boreholes BH06-6 underlying the surficial silty sand alluvium and in borehole BH06-7, the alluvium consisted of clayey silt containing trace to some organics and trace shells and was 3.5 m thick. The surface of the alluvium varied between Elevation 279.5 m and 282.6 m.

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

An Atterberg limits test carried out on one sample of the clayey silt alluvium deposit yielded a liquid limit of about 31 percent and a plastic limit of about 8 percent (plasticity index of about 23 percent). The results of the Atterberg limits test are shown on the plasticity chart on

Figure A-1 in Appendix A and classify the deposit as a clayey silt of low plasticity. A grain size distribution of the same sample of the alluvium is shown on Figure A-2.

The natural water content measured on samples of the alluvium ranged between 39 and 72 percent. The higher water contents are likely attributed to the presence of organics.

Directly underlying the fill in borehole BH06-9, a 0.3 thick layer of clayey silt with organics was encountered at Elevation 283.2 m, underlain by a 0.6 m thick layer of silt containing trace to some sand. One measured 'N' value in the clayey silt and silt layers in borehole BH06-9 was 19 blows per 0.3 m of penetration indicating a very stiff consistency and compact relative density.

4.2.3 Clayey Silt

A deposit of grey clayey silt to silty clay was encountered below the surficial fill and alluvium deposits in all boreholes except BH06-1. In borehole BH06-4, the clayey silt deposit was encountered at the lake bed. The top of this deposit varied between Elevation 282.3 m and 288.5 m in the land based boreholes and between Elevation 277.2 m and 282.5 m in the water based boreholes. The thickness of the deposit ranged from 3.2 m to 11.5 m.

Land-Based Boreholes

The samples of this deposit from the land-based boreholes consisted of clayey silt to silty clay containing trace sand and occasional organics near the surface of the deposit. Measured SPT 'N' values ranged from 3 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 63 kPa to greater than 100 kPa. In general, the field vane and SPT 'N' values suggest the clayey silt to silty clay stratum has a soft to very stiff consistency.

Atterberg limits testing carried out on three samples of the deposit from the land-based boreholes indicate liquid limits ranging from about 30 percent to 36 percent and the plastic limit ranging from about 22 percent to 25 percent yielding plasticity indices ranging from about 9 percent to 12 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure A-3 in Appendix A and indicate that the material is classified as a clayey silt of low plasticity to a silty clay of intermediate plasticity; all three test results plot at or just below the A-line indicating this material has a significant silt content.

The natural water content measured on select samples of this deposit ranged between 21 percent and 36 percent.

Three unconfined compression tests were carried out on specimens of the clayey silt to silty clay obtained from boreholes BH06-9 and BH06-10. Details of the test results are shown on Figures A-4 to A-6 in Appendix A. The following table summarizes the unconfined compression test results.

Borehole and Sample Number	*Elevation (m)*	*Compressive Stress (kPa)*	*Undrained Shear Strength (kPa)*
BH06-9 SA 8a	279.8	102	51
BH06-9 SA 8b	279.8	191	95
BH06-10 SA 9	283.1	224	112

One laboratory consolidation (oedometer) test was carried out on a specimen of the silty clay obtained from Borehole BH06-10 and the test results are shown on Figure A-7 in Appendix A. A pre-consolidation pressure of approximately 260 kPa was estimated from the voids ratio versus logarithmic pressure plots using the Casagrande method. 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)
BH06-10 SA 9	283.1	195	260	65	1.33	0.89	0.033	0.179	0.386

Note: *For stress range of $20 \leq \sigma_v' \leq 300$ 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.

SPT ‘N’ values measured on samples of this deposit from the water based boreholes ranged from 0 blows (weight of hammer) to 17 blows per 0.3 m of penetration. In situ field vane testing carried out in these boreholes measured undrained shear strengths ranging from 10 kPa to 48 kPa. In general, the field vane and SPT ‘N’ values suggest the clayey silt stratum has a very soft to very stiff consistency, becoming stiffer with depth.

Grain size distribution and Atterberg limits testing was carried out on six samples of the clayey silt deposit and the results are shown on Figures A-8 and A-9. The liquid limit ranged from about 22 percent to 33 percent and the plastic limit ranged from about 5 percent to 12 percent yielding plasticity indices ranging from about 17 percent to 24 percent, indicating that the material is classified as a clayey silt of low plasticity.

The natural water content measured on select samples of this deposit ranged between 29 percent and 38 percent. The natural water content of a sample of the clayey silt in each of boreholes BH06-4 and BH06-5 was greater than the corresponding liquid limit, resulting in liquidity indices of 1.2 and 1.6, respectively.

4.2.4 Silt

A deposit of silt was encountered below the fill in borehole BH06-1, and below the clayey silt in boreholes BH06-6 to BH06-10. The surface of the deposit was generally encountered between Elevation 269.3 and Elevation 275.3 m in the water-based boreholes and between Elevation 279.1 m and Elevation 290.1 m in the land-based boreholes. The thickness of the silt deposit ranged between 2.3 m and 5.4 m.

Measured SPT ‘N’ values in the silt deposit ranged from 10 to 24 blows per 0.3 m of penetration, indicating a compact relative density.

Grain size distributions for four samples from the silt are shown on Figure A-10 (containing trace sand and trace clay). The natural water content measured on samples of the silt deposit ranged from 17 percent to 29 percent, typically between 4 percent and 15 percent.

4.2.5 Silty Sand to Sandy Silt

A deposit of silty sand to sandy silt to sand was encountered below the clayey silt or silt deposits in boreholes BH06-1, BH06-2 and BH06-3. The deposit contained trace to some gravel and trace clay. The surface of the deposit was encountered between Elevation 278.5 m and Elevation 284.7 m and ranged between 1.0 m and 5.5 m in thickness. In borehole BH06-2, instances of augers grinding were noted below Elevation 282.3 m indicating the presence of cobbles and/or boulders within this deposit.

Auger refusal was encountered at a depth of about 9.8 m in borehole BH06-1, corresponding to Elevation 284.0 m. The dynamic cone penetration test advanced about 2 m north of borehole BH06-1 encountered refusal at a depth of about 10.1 m.

Measured SPT 'N' values in the silty sand to sandy silt deposit ranged from 8 to 28 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 to sand deposit ranged from 12 percent to 26 percent.

4.2.6 Gravelly Sand

Underlying the deposits of clayey silt, silt or silty sand to sandy silt, a deposit of gravelly sand containing trace to some silt was encountered in all the boreholes except BH06-1. In borehole BH06-9, the lower 1.3 m portion of the deposit is described as cobbles and boulders in a sand and gravel matrix. Frequent cobbles and boulders, inferred by 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 266.3 m to Elevation 279.2 m and the thickness varied between 3.7 m and 7.5 m.

At the borehole locations, measured SPT 'N' values ranged between 6 and greater than 100 blows per 0.3 m of penetration, indicating a very loose to very dense relative density. Typically, the deposit is compact to dense with 'N' values ranging between about 15 and 30 blows per 0.3 m of penetration.

In boreholes BH06-2 and BH06-9, gravelly sand was noted to heave into the hollow-stem augers upon obtaining the first sample of this deposit (i.e. about Elevation 278.7 m and Elevation 276.9 m, respectively).

A grain size distribution for one sample from the gravelly sand deposit is shown on Figure A-11.

The natural water content measured on samples of the gravelly sand deposit range from 10 percent to 21 percent.

As indicated above, cobbles and boulders were encountered within the gravelly sand deposit. In borehole BH06-2, NW casing was used to advance the borehole below 13.4 m of depth due to difficulty advancing the hollow-stem augers.

4.2.7 Bedrock

Gneiss bedrock was encountered in boreholes BH06-2 to BH06-9 and confirmed by coring between 3.1 m and 4.8 m of the bedrock. The surface of the bedrock was encountered between Elevation 262.0 m and Elevation 274.6 m, corresponding to depths of 16.2 m and 16.6 m below existing ground surface in the land-based boreholes and corresponding to depths of between

14.2 m and 22.1 m below the water surface in the water-based boreholes. The rock core is described as a gneiss, grey, fined to medium grained and fresh to slightly weathered. A sand layer was penetrated within the bedrock in borehole BH06-6 from 25.7 m to 25.9 m of depth. The Rock Quality Designation (RQD) measured on the core samples ranged from about 0 percent to 100 percent. This indicates rock mass variable in quality, ranging from very poor to excellent. In general, the RQD values are between 50 percent and 100 percent, and are considered to be fair to excellent quality. Generally, the RQD values increase with depth.

Uniaxial compression strength (UCS) testing was carried out on three samples of the gneiss bedrock from boreholes BH06-2, BH06-5 and BH06-9. The UCS results were between 64 MPa and 113 MPa. The depths and corresponding elevations of the samples and results of the UCS testing are shown in Table A-1. Diametral (i.e. horizontal or perpendicular to the core axis) point load strength tests were performed on twelve samples of the gneiss bedrock from the boreholes. Diametral point load index values ranged from about 2.5 MPa to 5.6 MPa which correspond to estimated UCS values between 50 MPa and 110 MPa with an average strength of about 89 MPa, as presented in Table A-2. Using the Intact Rock Strength Classification table, these results indicate that the gneiss rock is classified as 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 BH06-2 and BH06-9 with screened zones sealed within the sandy silt and gravelly sand deposits, respectively. 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	06-1	Open Borehole	8.5	285.3	Upon Completion of Drilling
South Abutment	06-2	Piezometer	6.2 6.4	284.6 284.4	July 4, 2006 August 24, 2006
In-Water Piers*	06-3 to 06-8	N/A	0	284.1	July and August, 2006
North Abutment	06-9	Piezometer	4.1 4.3	285.2 285.0	July 4, 2006 August 24, 2006
North Approach	06-10	Open Borehole	n/a	Dry	Upon Completion of Drilling

* Lake water level/elevation

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 boreholes BH06-2 and BH06-9, gravelly sand was noted to flow into the hollow-stem augers and a constant head of water was required to prevent heave during drilling.

The above groundwater levels are consistent with the adjacent river 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 of Golder's Sudbury office. This report was prepared by Mr. André Bom, P.Eng., a geotechnical engineer; the technical aspects were reviewed by Miss Sarah Poot. 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
REHABILITATION AND WIDENING OF NORTHBOUND STRUCTURE
HIGHWAY 11
G.W.P 5189-05-00, SITE NO. 42-018
MINISTRY OF TRANSPORTATION, ONTARIO
HUNTSVILLE, ONTARIO

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides design recommendations on the foundation aspects of the proposed widening of Highway 11 Northbound Lane 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 seven-span structure, about 250 m in overall length. The proposed works involve widening the existing NBL bridge towards the median by about 9 m. The foundation abutments and piers will be widened at the same location as the existing abutments and piers. The recommendations also discuss issues related to the interaction of the existing and proposed foundations. The existing approach embankments are between about 9 m and 10 m in height above the existing lake level. The proposed grade of the northbound lanes will not change significantly; however, widening towards the median will result in an effective grade raise of about 3.5 m in the median area at the abutment locations. The recommendations provided also address settlement and stability of the widened embankments.

It should be noted that the existing southbound lane structure, built in the 1950s, is located approximately 13 m to 18 m west of the NBL structure. This bridge is anticipated to be replaced as part of the overall project and any implications of the existing and/or proposed foundations of the SBL structure on the widened foundations for the NBL structure will be addressed in the SBL report. Further, after the widening, the two bridges will be approximately 4 m to 8 m apart.

5.1 General Bridge Foundation Options

Shallow foundations are not recommended for support of the NBL bridge widening 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 widened 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 H-piles be used as the preferred founding alternative. The following paragraphs describe the methodology for pile installation and pile cap construction for the driven pile alternative at the piers. Pile installation at the abutments should be carried out using standard construction techniques.

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 pile installation allowed for relatively straightforward in-water construction. 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 and lateral support of the pile lengths through water. 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 widening of the NBL 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 using wash boring methods to flush the casings, in regards to disposal of cuttings into a sediment basin/tank rather than into the lake.

In the case of the widened NBL piers, this construction technique should be compatible with existing footings. In order to reduce the potential for interference/disturbance to the existing footings, a construction joint should be provided between the existing pile cap and the new cofferdam (i.e. new pile cap) at each pier, in order to articulate any possible differential settlement between the two elements. From a vibration standpoint, it is our opinion that the vibrations during pile driving will typically be low and should not be a significant issue at this site. However, vibration monitoring should also be carried out during construction.

The designer should check that the new piles (batter and orientation) do not interfere with the existing piles. This should be checked to the full extent of the pile length to the bedrock surface.

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 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 near the banks may not have an adequate depth of water and, therefore, consideration should be given to building an access road in the water to these near-shore locations.

5.3 Shallow Foundations

Due to the presence of compressible clayey silt 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 widened section of the NBL would not be consistent with the existing foundations.

5.4 Steel H-Pile Foundations

Based on the previous borehole information obtained at this site, piles driven to bedrock are recommended for support of the foundations of the widened 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 suitable for this site. Also, this foundation method would match the existing foundations consisting of piles end-bearing on bedrock.

5.4.1 Axial Geotechnical Resistance

5.4.1.1 Existing Piles

Based on the summary report made available to us (Stantec, 2005), the factored axial resistance at Ultimate Limit State (ULS) for the existing pier piles was given as 1450 kN for HP 310x110 piles. For HP310x79 piles at the abutments, the factored axial resistance at ULS is 1,000 kN. However, as stated in the report, these values were based on several assumptions including that the steel used in 1977 did not have as high a yield strength as modern steel.

The existing NBL pile foundations installed in 1977 have been subjected to the bridge loadings for over 25 years. The design of the bridge for the widening must take into account the potential for compression of the new piles associated with the new construction and potential interaction between the existing bridge deck and the new bridge deck.

For the widening, the design should accommodate the potential elastic compression shortening of the new piles since the existing piles have already been compressed. In addition, the axial resistance of the existing piles should be checked to verify that they can support the additional load from the widened structure.

Since the majority of the settlement related to the 3.5 m high embankment widening section is expected to occur during construction, downdrag on the existing piles need not be considered.

5.4.1.2 New Piles

The bedrock was encountered between Elevation 262.0 m and Elevation 274.5 m at the proposed widened abutment and pier locations. These elevations correspond to the design pile tip level. The current lake bed is between Elevation 279.5 m and Elevation 283.2 m at the pier borehole locations, resulting in piles up to about 19.1 m in length at the piers (about 22.1 m in length relative to the July/August 2006 water level) and up to about 16 m in length at the abutments. The design pile tip elevations and the ground surface/lake bed elevations are given in Table 2.

Also presented in Table 2 is the elevation at which the surface of the very dense gravelly sand and/or a layer of cobbles and boulders were encountered, as these deposits may impact the final tip elevation. For design, 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 granular layer at the estimated elevations given in Table 2.

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 m and 11 m. 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 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 State (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 A for reference. The driving procedures to enable pile seating depend on the type of pile driving rig used; 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 Contract Drawings:

- “Piles to be driven to bedrock.”

Steel casings installed below the lake bed through the very soft to firm clayey silt to 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, as given below (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 both the existing piles (HP 310x110 at the piers and HP 310x79 at the abutments) and 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's 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's 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 settlement of the soils is expected to occur during construction, 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. Details of insulation placement can be given if this is to be used as an alternative to a 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). It should be noted that caissons may not be the most practical alternative for the abutments and piers since the existing bridge is founded on piles. In addition, there could be disturbance to the existing piles during caisson advance and socketting.

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 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, with the upper limit as determined through the use of 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 OPSD 3501.00 and 3504.00.
- 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 3505.00.
- 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 culverts 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 0.05g 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; however, based on the current topographic plan of the site, we understand that the widening in the median area will result in an effective grade raise of between about 3 m and 3.5 m above the current grade. Since the new widening portion of the NBL bridge will be between 4 m and 8 m away from the southbound bridge, there will essentially be only a small median ditch area. The following sections present the results of settlement and stability analysis and subsequent recommendations in the widening area (west side). No comment is made on the slopes on the east side of the bridge.

It should be noted that sub-excavation of the soft organic soil at the toe of the existing NBL structure north abutment, as well as construction of a 3 m wide stabilizing berm at the toes of the front slopes, was recommended in 1977. The geometry of the existing abutment front slopes indicate that the upper portion of the slope is at a configuration of about 1.5 horizontal to 1 vertical (1.5H:1V) while the lower portion of the slope, adjacent to the water, is typically at about 3.5H:1V. The overall height of the embankments above the water level is between about 9 m and 10 m.

At all areas, the stability and settlement analyses assume that organic soils have been removed from the median prior to construction of the widened approach embankments. 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 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 used 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; rather, the whole deposit is layered.

5.8.1.2 Parameter Selection

The subsoils encountered at the site are composed of granular 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 for the soils. The effective stress parameters (effective friction angle and cohesion) for the granular soils were estimated from empirical correlations using the results of in situ Standard Penetration Tests (SPT), in conjunction with engineering judgement considering experience in similar soil conditions.

For cohesive deposits, total-stress parameters were employed in the analyses assuming undrained (i.e. short-term or during construction) for soil layers beyond the toes of the existing embankments and effective stress (i.e. drained conditions) for the soil layers below the existing embankment. Embankment stability was also checked for the total stress conditions under the existing embankment where widening will take place. 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).

For the purposes of analyses, it assumed that granular fill (earth fill) would be used for construction of the widened approach embankments. The parameters used in the analysis are given below:

Soil Type	*Effective Unit Weight* (kN/m^3)*	*Undrained Shear Strength (kPa)*	*Angle of Internal Friction*
New Earth Fill (Assume Granular Material)	21	n/a	35°
Existing Granular Fill	21	n/a	32°
Alluvium	15	10	n/a
Clayey Silt to Silty Clay (below north embankment)	18	30 (upper layer near toe of widening) 60 (lower)	26° (upper) 28° (lower)
Clayey Silt (toe of north slope)	18	20 to 40	n/a
Clayey Silt to Silty Clay (below south embankment)	18	60	n/a
Clayey Silt (toe of south slope)	18	20 to 40	n/a
Silt	19	n/a	30°
Silty Sand to Sandy Silt	19	n/a	30°
Gravelly Sand	21	n/a	35°
Granular Fill for Sub-excavated Area Under Toe of Widening	20	n/a	30°

* Groundwater level assumed between Elevation 284 m and 285 m.

5.8.1.3 Results of Analysis

For the south abutment, there is sufficient space between the abutment face and the shoreline to construct a 3.8H:1V slope. Based on this slope geometry, undrained conditions and a maximum grade raise of 3.5 m from the lowest point of the median, a factor of safety of greater than 1.3 is obtained for the south abutment, as shown on Figure 1. Therefore, no special mitigation measures are required for this slope, provided it is constructed at no steeper than 3.8H:1V.

For the north abutment, there is sufficient space to construct a 3.4H:1V slope for the embankment widening area between the abutment and the existing shoreline. Assuming this slope geometry, undrained or drained conditions and a maximum 3.5 m grade raise from the lowest point of the median, a factor of safety of less than 1.2 is obtained for the north abutment front slope. The results of the analysis are shown on Figure 2. Therefore, ground improvement methods will be required to mitigate stability issues at the north abutment widening as discussed below.

5.8.1.4 Mitigation of Stability (North Slope)

In order to achieve a factor of safety greater than 1.3 for the north abutment widening slope, consideration could be given to construction of a stability berm (slope geometry), sub-excavation of soft materials at the toe of the slope, or the use of lightweight fill to reduce embankment loading.

Based on the existing embankment geometry and the widened bridge abutment, there will be space for an overall slope geometry of approximately 3.4H:1V from the front face of the abutment (approximate Elevation 289.0 m) and the water's edge (approximate Elevation 284 m). For the alternative of sub-excavating the very soft to firm clayey silt alluvium at the toe of the slope and replacement with granular fill, the factor of safety for stability is greater than 1.3, as shown on Figures 3A and 3B, for the drained and undrained conditions, respectively.

For the sub-excavation alternative, the sub-excavation should be in general accordance with OPSD 203.02 except where noted in an Operational Constraint (OC). The sub-excavated material should be replaced with granular material consisting of Granular 'B' Type II. The sub-excavation should be made at no steeper than 1H:1V slope about 5 m out from the crest of the median adjacent to the existing north abutment and extend out from the shore towards Pier No. 6. The base of the sub-excavation should be approximately 4.5 m below the lake bottom to Elevation 279.5 m.

We understand that sub-excavation of the alluvium was recommended at the toe of the slope for the northbound structure (north side) to Elevation 279.5 m according to the 1977 contract drawings. Based on the results of borehole BH06-8, located about 3 m west of the existing pier, this sub-excavation was carried out and backfilled with sand. However, it is not known whether that sub-excavation extended laterally to the limit of the new widened structure. Therefore, additional sub-excavation of alluvium will be required, west of this borehole to approximately 5 m beyond the west edge of the widened pier.

Care must be exercised during sub-excavation of the alluvium to ensure that the stability of the existing abutment slope and pier are not affected by new construction operations.

Alternatively, consideration could be given to the use of lightweight expanded polystyrene fill (EPS) to construct the embankment at the north abutment. This would reduce the loading and, therefore, the embankment geometry would be unchanged from the existing geometry which is considered to be stable. However, the cost of using EPS fill is typically an order of magnitude greater than other options.

Based on the alternatives considered above, we recommend that the overall slope geometry be constructed no steeper than 3.4H:1V in front of the abutment face and that sub-excavation of the clayey silt alluvium at the toe of the slope in the widened area be carried out. This alternative will provide the best technical solution in terms of the stability and long-term performance of the roadway, compatibility with the adjacent existing approach embankments, as well as anticipated construction schedule and overall costs. A sample of the OC required for sub-excavation of the alluvium below the lake level is included in Appendix B.

5.8.2 Settlement

5.8.2.1 Methodology

Settlement analyses were performed on the critical sections (i.e. maximum grade raise) of the widened approach embankments. The settlement analysis was performed using standard equations from literature. Embankment settlement results from primary time-dependent consolidation and secondary creep settlement (where applicable) of the cohesive deposits; immediate settlement of the native granular soils and existing fill material; and self-weight compression of the new embankment fill materials.

5.8.2.2 Parameter Selection

The immediate compression of the native granular soils was modelled by estimating an elastic modulus of deformation based on the SPT 'N' values and correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990).

Settlement analyses were carried out using the results of borehole information, in situ field test data and laboratory results of water content and Atterberg limits determinations.

The cohesive deposits over-consolidation ratio (OCR) profile required in the settlement analyses was established using correlations with the results of the in situ vane shear tests. The following correlation relating in situ undrained shear strength to preconsolidation pressure (Mesri, 1975) was employed:

$$s_u = 0.22\sigma_p'$$

where : s_u = average mobilized undrained shear strength (kPa)
σ_p' = preconsolidation pressure (kPa)

The compression and recompression index profiles required in the analysis were established using correlations with laboratory test data such as Atterberg limits and oedometer tests. The following published correlation (Kulhawy and Mayne, 1990) relating the Plasticity Index (I_p) to the compression and recompression indices was used:

$C_c = I_p/74$

$C_r = C_c/10$

where : C_c = compression index

C_r = recompression index

The coefficient of consolidation, c_v, required in the analysis was established using the results of the correlation with liquid limit (NAVFAX 1971).

The parameters used in the settlement analysis are given below.

Soil Type	***γ (kN/m^3)***	***ϕ'***	***E (MPa)***	***s_u (kPa)***	***e_o***	***C_c***	***C_r***	***c_v (cm^2/s)***
New Earth Fill (Granular)	21	35°	40	n/a	n/a	n/a	n/a	n/a
Existing Granular Fill	21	32°	40	n/a	n/a	n/a	n/a	n/a
Clayey Silt (below embankment)	18	30°	n/a	75	0.89	0.179	0.033	0.192
Silt	9*	30°	30	n/a	n/a	n/a	n/a	n/a
Silty Sand to Sandy Silt	10*	30°	30	n/a	n/a	n/a	n/a	n/a
Gravelly Sand	11*	28°	35	n/a	n/a	n/a	n/a	n/a

where:

γ	unit weight (*effective unit weight used below the water table)
ϕ'	angle of internal friction
E	elastic modulus
s_u	undrained shear strength
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 for the tress range of $20 \leq \sigma_v' \leq 300$ kPa

5.8.2.3 Results of Analysis

Given the proposed grade raise of about 3.5 m, the over-consolidated nature of the 3.2 m to 3.5 m thick clayey silt to silty clay deposit underlain by cohesionless materials, it is estimated that the total magnitude of settlement will be in the order of about 40 mm. It is expected that the majority of this settlement will occur during construction. This settlement will be differential with respect to the existing highway (NBL).

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

Settlement of the clayey silt deposit under the existing NBL as a result of the new embankment fill loading is expected to be less than 15 mm and should occur during construction. Some minor cracking of the asphalt may occur and thus regrading/repaving of the approaches, particularly the west side nearest the widening, may be required.

Granular earth fill should be used for the embankment widening 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 widening, the settlement could be up to about 25 mm and this settlement would occur after construction and be differential with respect to the existing embankment. If cohesive earth fill is used, it is recommended that paving be delayed for at least 6 months.

5.9 Subgrade Preparation and Embankment Construction

The existing fill and native subsoils are considered to be an appropriate subgrade for the proposed widened approach embankments; however, all softened/loosened soils should be stripped from below the approach embankment areas, and all subgrade soils should be proof-rolled prior to placement of new fill. Topsoil was noted in the boreholes advanced in the median and it should be noted that vegetation exists in the median on both sides of the lake. We recommend stripping of the topsoil/vegetation and any other organic materials that may be encountered as part of subgrade preparation for construction of the embankment widening.

The effective embankment heights at this site are less than 6 m and therefore do not require a mid-height berm (in accordance with Northern Region Directives).

Embankment fill materials and placement should be carried out in accordance with the requirements as outlined in Special Provision SP206S03. Side slopes for rock fill embankments should be no steeper than 1.25H:1V and earth fill embankments should be no steeper than 2H:1V. Special requirements with respect to the abutment front slope geometry are given in Section 5.8.1.

All subgrade soils should be proof-rolled prior to fill placement and embankment fill should be placed in accordance with SP206S03. The final lift prior to placement of the granular subbase and base courses should be compacted to 100 per cent 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 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 loose to compact sand to silty sand to silt fill at both abutments and possibly into the native very stiff clayey silt to silty clay at the south abutment. Excavations for abutment pile cap construction should be above the groundwater level which was recorded at between Elevation 284.4 m and Elevation 285.3 m at the south abutment (rising towards the south) and between Elevation 284.7 m and Elevation 285.0 m at the north abutment (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 loose to compact sand to silty sand to silt fill and is classified as Type 3 soil and the native very stiff clayey silt to silty clay is classified as a Type 2 soil, according to the OHSA.

It is expected that groundwater inflow into the excavations for the abutment pile caps will be minimal 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.

Excavation support for protection of the existing roadway at the abutments will be required at this site. The temporary excavation support system 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.

Excavation for the new abutment pile caps will likely expose the existing footings. Extra care should be taken by the contractor to ensure that damage to the existing footing does not occur. An NSSP should be included in the contract documents for this purpose (refer to Appendix B for an example).

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. An NSSP will be required to inform the contractor that the pile cap construction must be carried out in the dry; a sample is included in Appendix B for reference.

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 immediately adjacent to the existing bridge footings. Vibration monitoring should be carried out during construction (particularly pile/caisson installation) to ensure that vibration levels of the existing structure are maintained below tolerable levels. We recommend limiting ground vibration levels to 50 mm/s for the abutment and pier footings. Continuous monitoring of all pile/caisson installation operations would dictate when changes to the installation procedures become necessary to meet these limits. An NSSP should be included in the contract for this purpose and an example has been included in Appendix B for reference. The NSSP includes qualifications, positioning of the monitoring equipment, tolerances, and sequence of monitoring.

5.11 Closure

This report was prepared by Mr. André Bom, P.Eng. and Sarah Poot, 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
REHABILITATION AND WIDENING OF NORTHBOUND STRUCTURE
G.W.P 5189-05-00, 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 difficult 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 5.1 of the Foundation Investigation and Design Report.

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TABLE 2
PROPOSED PILE TIP ELEVATION
REHABILITATION AND WIDENING OF NORTHBOUND STRUCTURE
G.W.P 5189-05-00, SITE NO. 42-018
HIGHWAY 11, HUNTSVILLE

Foundation Element	*Relevant Borehole*	*Ground Surface/Lakebed 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	06-2	290.8	275.5	274.5	14.9
Pier #1	06-3	282.6	272.0	270.0	12.6
Pier #2	06-4	279.7	268.0	266.5	13.2
Pier #3	06-5	279.5	264.0	262.5	17.0
Pier #4	06-6	281.1	n/a	262.0	19.1
Pier #5	06-7	282.3	266.0	265.5	16.8
Pier #6	06-8	283.2	n/a	269.0	14.2
North Abutment	06-9	289.3	n/a	272.5	16.0

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 at the abutments, assumed to be at the same elevation as the existing abutment at about Elev. 288.5 m N and Elev. 289.4 S. Approximate pile length below the lakebed at the piers.

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TABLE 3
PROPOSED BASE OF CASING ELEVATION
REHABILITATION AND WIDENING OF NORTHBOUND STRUCTURE
G.W.P 5189-05-00, SITE NO. 42-018
HIGHWAY 11, HUNTSVILLE

Foundation Element	*Relevant Borehole*	*Proposed Base of Casing Elevation (m)*
Pier #1	06-3	277.0
Pier #2	06-4	273.0
Pier #3	06-5	271.5
Pier #4	06-6	271.5
Pier #5	06-7	271.5
Pier #6	06-8	273.5

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
HORIZONTAL SUBGRADE REACTION VAULES
REHABILITATION AND WIDENING OF NORTHBOUND STRUCTURE
G.W.P 5189-05-00, SITE NO. 42-018
HIGHWAY 11, HUNTSVILLE

Foundation Element	*Relevant Borehole*	*Soil Unit*	*Elevation (m)*	n_h *(MPa/m)*	s_u *(kPa)*
South Abutment	06-2	Loose to compact sand/silt (fill)	Ground surface to 288.5	1.3	--
		Very stiff to firm clayey silt to silty clay	288.5 to 284.7	--	60
		Loose to compact sandy silt	284.7 to 279.2	1.3	--
		Compact to very dense sand	279.2 to 274.6	4.4	--
Pier #1	06-3	Very soft to stiff clayey silt	Lake bed to 278.5	--	30
		Compact to very dense sand to gravelly sand	278.5 to 269.9	4.4	--
Pier #2	06-4	Very soft to stiff clayey silt	Lake bed to 273.9	--	30
		Compact to dense gravelly sand	273.9 to 266.4	4.4	--
Pier #3	06-5	Very soft to stiff clayey silt	Lake bed to 267.9	--	30
		Dense to very dense gravelly sand	267.9 to 262.6	11	--
Pier #4	06-6	Very soft clayey silt (alluvium)	Lake bed to 277.2	--	10
		Very soft to stiff clayey silt	277.2 to 269.3	--	30
		Compact silt	269.3 to 266.3	4.4	--
		Compact gravelly sand	266.3 to 262.0	4.4	--
Pier #5	06-7	Very soft clayey silt (alluvium)	Lake bed to 278.8	--	10
		Very soft to stiff clayey silt	278.8 to 272.4	--	30
		Compact silt	272.4 to 269.3	4.4	--
		Compact to loose gravelly sand	269.3 to 265.3	4.4	--
Pier #6	06-8	Very soft to firm clayey silt (alluvium)	Lake bed to 280.0	--	10
		Very soft to soft clayey silt	280.0 to 275.3	--	30
		Compact silt	275.3 to 272.4	4.4	--
		Compact to loose gravelly sand	272.4 to 268.7	4.4	--
North Abutment	06-9	Cobbles and Boulders riprap (fill)	Ground surface to 288.0	1.3	--
		Very loose to compact silty sand to sand (fill)	288.0 to 283.2	1.3	--
		Very stiff/compact clayey silt to silt with organics	283.2 to 282.3	--	60
		Stiff to firm silty clay	282.3 to 279.1	--	30
		Compact silt	279.1 to 276.8	4.4	--
		Compact gravelly sand	276.8 to 274.0	4.4	--
		Cobbles and Boulders in sand matrix	274.0 – 272.7	4.4	--

NOTES:

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

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LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

2 Shear strength = (Compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

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

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

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

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

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

BEDDING THICKNESS

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
‖ - 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 BH06-1

1 OF 1 **METRIC**

PROJECT 06-1191-001

W.P. 5189-05-01

LOCATION N 5020314; E325113

ORIGINATED BY EHS

DIST 52 HWY 11

BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers

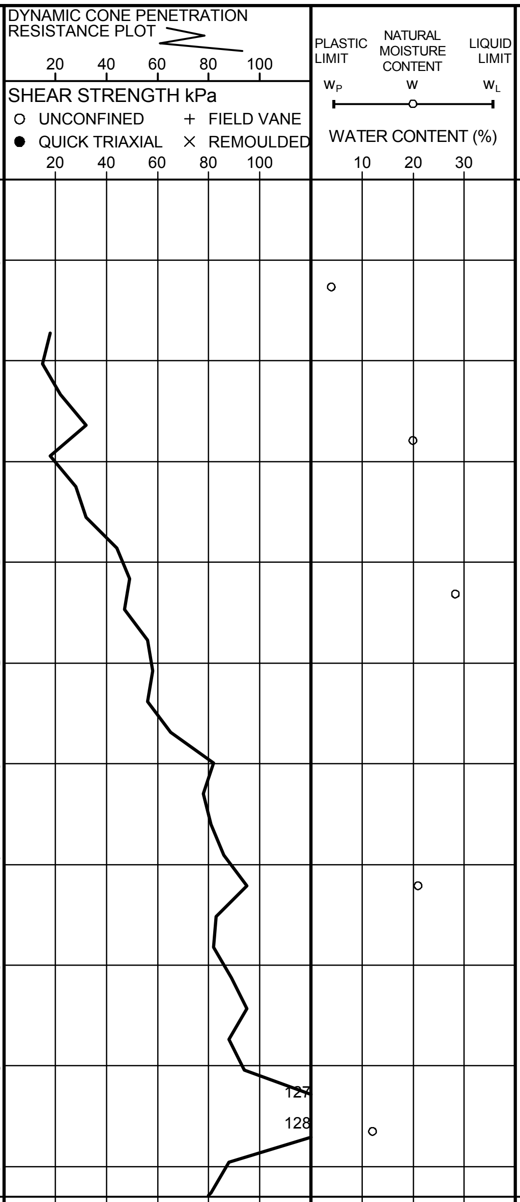
COMPILED BY AB

DATUM Geodetic

DATE 07/06/06

CHECKED BY SEP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES				
293.8	GROUND SURFACE							
0.0	ASPHALT							
0.4	Sand and gravel (FILL) Brown							
	Sand, some gravel, trace silt, occasional cobbles (FILL) Compact Brown Moist	1	SS	29		293		
	Start of Dynamic Cone Penetration Test	2	SS	18		292		
291.2 2.6	Silt, trace to some clay, trace sand, trace organics (FILL) Compact Brown/Grey Moist	3	SS	13		291		
		4	SS	28				
290.1 3.7	SILT, trace to some clay, trace sand, occasional clay and sand seams Compact Brown to grey Moist to wet					290		
		5	SS	24				
		6	SS	14		289		
						288		
		7	TO	PH				
		8	SS	16		287		
		9	SS	12		286		0 2 93 5
					▽			
						285		
284.7 9.1	SILTY SAND, some gravel, occasional cobbles Compact Brown	10	SS	28				
284.0	Wet					284		
283.7 10.1	End of Borehole Refusal to further penetration							



Notes:
1. Augers grinding between 9.5m and 9.8m depth.
2. Auger refusal at 9.8m depth. Borehole moved 2m north and Dynamic Cone Penetration Test advanced from 1.5m to 10.1m depth.
3. Water level measured in open hole at 8.5m depth (Elev. 285.3m) upon completion of drilling.

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

RECORD OF BOREHOLE No BH06-2

1 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-01
DIST 52 HWY 11
DATUM Geodetic
LOCATION N 5020339; E325113
BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers
DATE 06/20/06
ORIGINATED BY EHS
COMPILED BY AB
CHECKED BY SEP

SOIL PROFILE

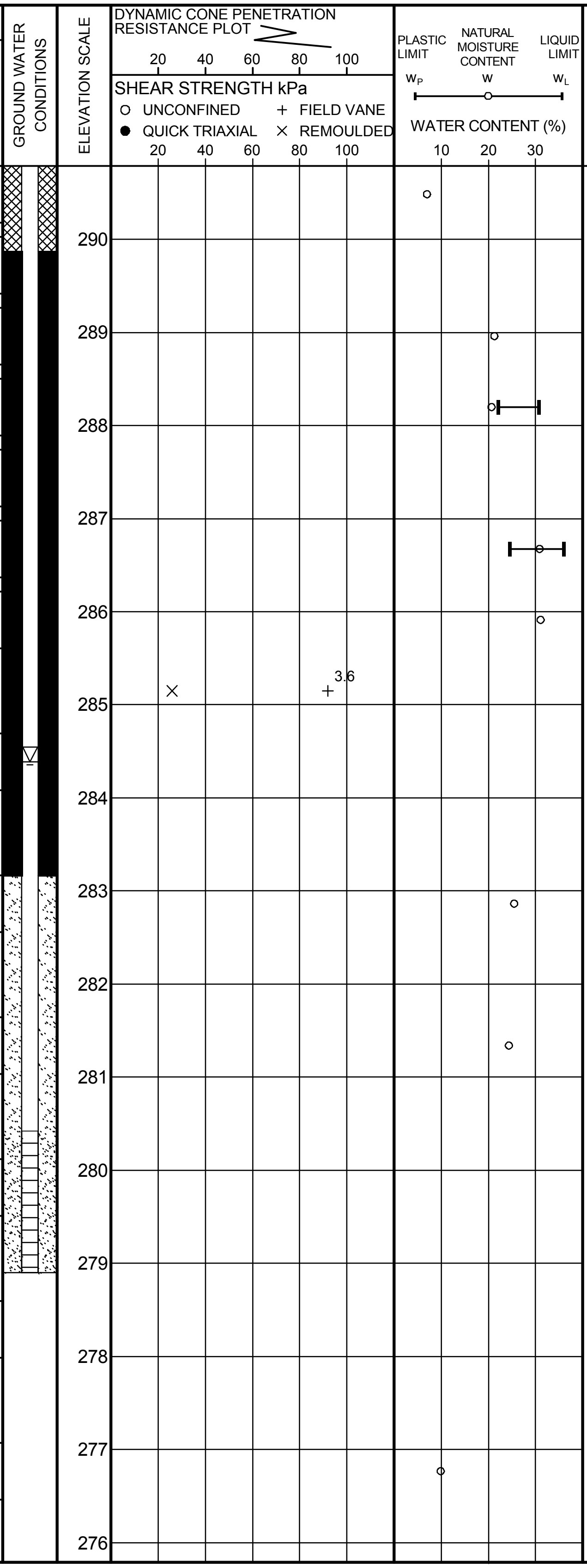
ELEV DEPTH	DESCRIPTION
290.8	GROUND SURFACE
0.0 0.1	Sandy TOPSOIL Brown Moist Sand, trace gravel, trace silt (FILL)
290.0 0.8	Loose Brown Moist Silt, some clay, trace sand, trace gravel, trace asphalt (FILL) Compact Brown Moist
288.5 2.3	CLAYEY SILT to SILTY CLAY , trace sand, layered, containing sand seams Very stiff to firm Brown Moist Trace organics above 4.3m depth
284.7 6.1	SANDY SILT Loose to compact Brown to grey Wet Augers grinding at 8.5m depth Augers grinding below 9.1m depth
279.2 11.6	GRAVELLY SAND, trace to some silt, containing cobbles and boulders Compact to very dense Grey Wet Augers grinding from 11.6m to 13.4m depth

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	5
2	SS	25
3	SS	12
4	SS	24
5	SS	18
6	SS	12
7	SS	6
8	SS	8
9	SS	10
10	SS	27
11	SS	26
12	SS	17
13	SS	48

STRAT PLOT | GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL



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

RECORD OF BOREHOLE No BH06-2

2 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-01
DIST 52 HWY 11
DATUM Geodetic
LOCATION N 5020339; E325113
BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers
DATE 06/20/06
ORIGINATED BY EHS
COMPILED BY AB
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, trace to some silt, containing cobbles and boulders Compact to very dense Grey Wet		14	SS	57		275				
274.6 16.2	GNEISS (BEDROCK) Bedrock cored from 16.2m to 19.7m depth For coring details see Record of Drillhole BH06-02						274 273 272				
271.1 19.7	End of Borehole Notes: 1. Heave in augers to 11.0m depth upon reaching Sample 12. 2. Switch to NW Casing at 13.4m depth. 3. Water level measured in piezometer at 6.4m depth (Elev. 284.4m) on August 24, 2006.										

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

PROJECT: 06-1191-001

RECORD OF DRILLHOLE: BH06-2

SHEET 1 OF 1

LOCATION: N 5020339; E325113

DRILLING DATE: 06/20/06

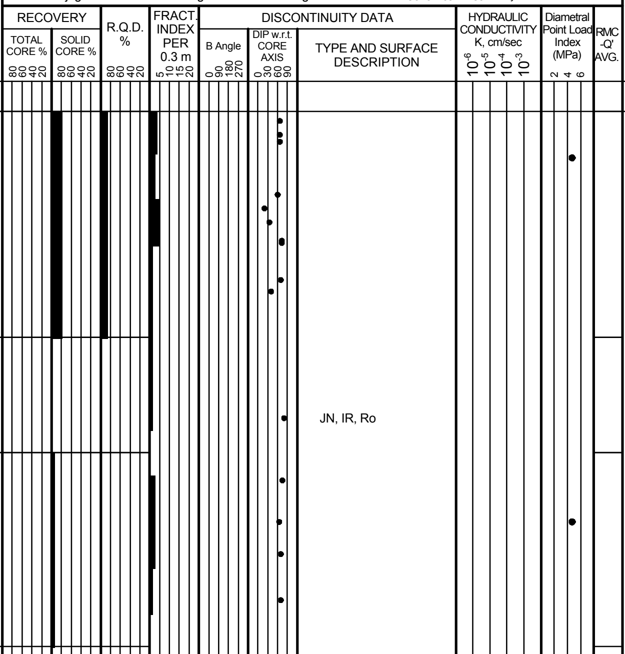
DATUM: Geodetic

DRILL RIG: CME 55

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Walker Drilling Ltd.

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)	COLOUR % RETURN FLUSH	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page		274.6					
17	NQ Coring	GNEISS Fine to medium grained Fresh to slightly weathered Strong Grey		16.2	1				UCS=78MPa
18		All joints and foliations are planar and smooth except where noted			2			JN, IR, Ro	
19				271.1 19.7	3				
20		End of Drillhole							
21									
22									
23									
24									
25									
26									

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: EHS

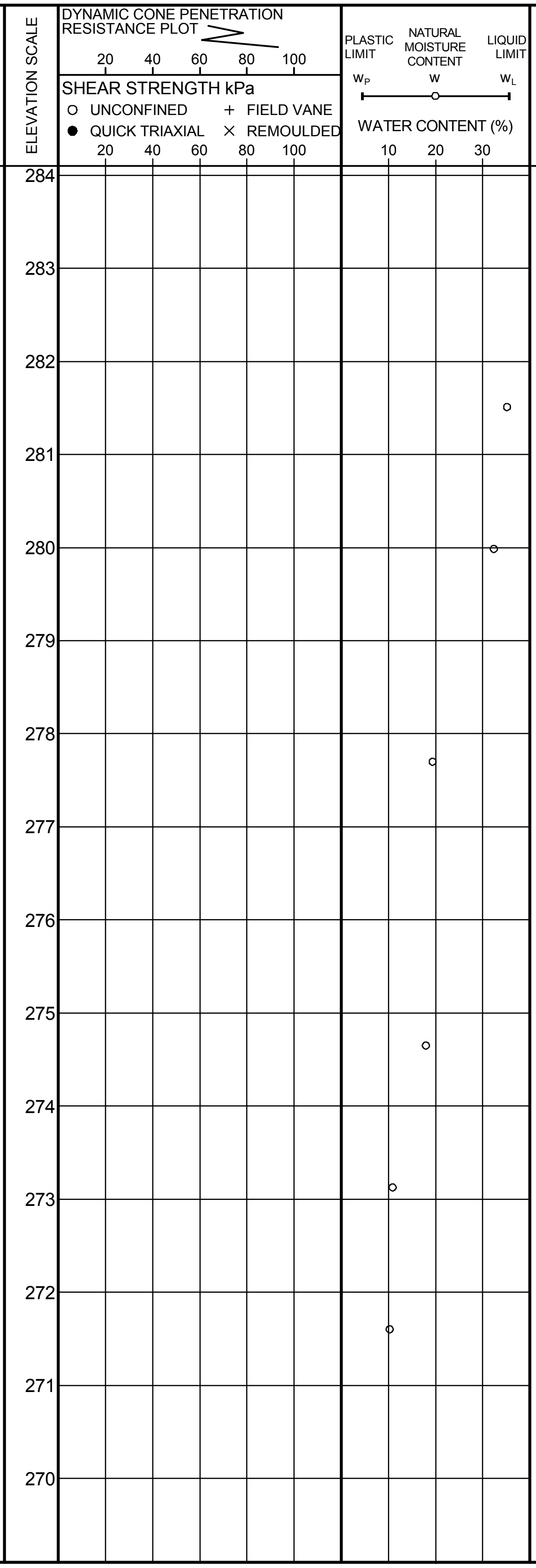
CHECKED: AB

RECORD OF BOREHOLE No BH06-3

1 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-01
DIST 52 HWY 11
DATUM Geodetic
LOCATION N 5020365; E325124
BOREHOLE TYPE NW Casing, Wash Boring
DATE 07/24/06
ORIGINATED BY ID
COMPILED BY AB
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							
282.6 1.6	SANDY fine GRAVEL (ALLUVIUM) CLAYEY SILT, layered Very soft to stiff Grey Wet	1	SS	1				
		2	SS	3				
		3	SS	6				
		4	SS	6				
		5	SS	10				
278.5 5.6	Fine SAND, trace silt, trace clay Compact Grey Wet	6	SS	11				
277.1 7.0	GRAVELLY SAND, trace to some silt Compact to very dense Grey Wet	7	SS	24				
	Containing cobbles and boulders below 9.1m depth	8	SS	53				
		9	SS	38	21	59	20	0
		10	SS	128				
	Spoon bouncing at 14.0m depth	11	SS	104/0.15				
269.9 14.2	GNEISS (BEDROCK)							



UNIT WEIGHT γ kN/m³

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH06-3

2 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-01
LOCATION N 5020365; E325124
ORIGINATED BY ID
DIST 52 HWY 11
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY AB
DATUM Geodetic
DATE 07/24/06
CHECKED BY SEP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 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						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	GNEISS (BEDROCK) Bedrock cored from 14.2m to 18.1m depth For coring details see Record of Drillhole BH06-3						269 268 267				
266.0 18.1	End of Borehole										

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

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

PROJECT: 06-1191-001

RECORD OF DRILLHOLE: BH06-3

SHEET 1 OF 1

LOCATION: N 5020365; E325124

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: 07/25/06

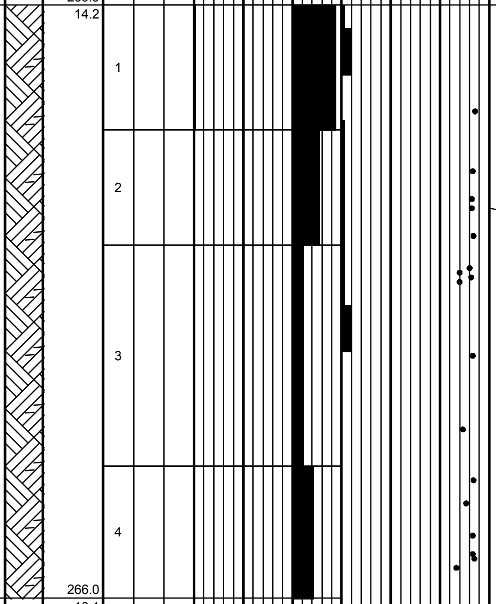
DRILL RIG: D90

DRILLING CONTRACTOR: Walker Drilling Ltd.

DATUM: Geodetic

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)	COLOUR % RETURN FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.3 m 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page		269.9														
	NQ Coring	GNEISS Fine to medium grained Fresh to slightly weathered Strong Grey Broken core between 14.8m and 15.2m depth		14.2	1													
15					2									JN, UN, Ro JN, UN, Ro				
16					3									JN, UN, Ro				
17		All joints and foliations are planar and smooth except where noted.			4													
18				266.0 18.1														
		End of Drillhole																
19		Note: 1. Solid Core Recovery not recorded.																
20																		
21																		
22																		
23																		
24																		



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: ID

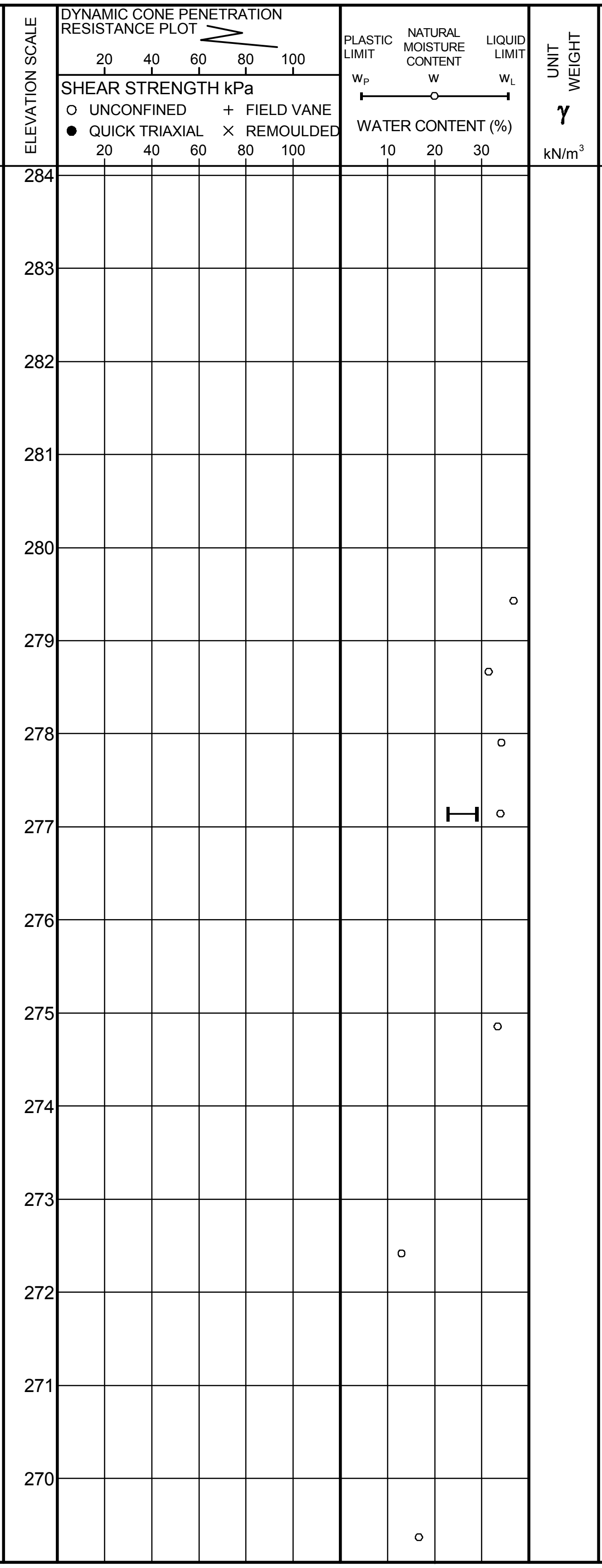
CHECKED: AB

RECORD OF BOREHOLE No BH06-4

1 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-01
LOCATION N 5020401; E325134
ORIGINATED BY ID
DIST 52 HWY 11
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY AB
DATUM Geodetic
DATE 07/26/06
CHECKED BY SEP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
284.1	WATER SURFACE									
0.0	WATER									
279.7 4.4	CLAYEY SILT, layered Very soft to stiff Grey Wet		1	SS	1					
			2	SS	8					
			3	SS	11		0	2	82	16
			4	SS	6					
			5	SS	9					
			6	SS	14					
273.9 10.2	GRAVELLY SAND, trace to some silt Compact to dense Grey Wet		7	SS	18					
			8	SS	22					
			9	SS	40					
			10	SS	26					



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+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

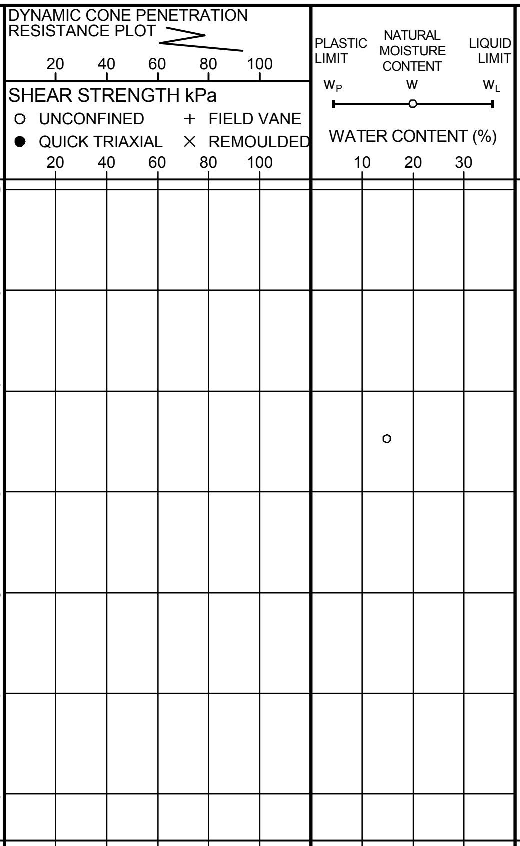
RECORD OF BOREHOLE No BH06-4

2 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-01
DIST 52 HWY 11
DATUM Geodetic
LOCATION N 5020401; E325134
BOREHOLE TYPE NW Casing, Wash Boring
DATE 07/26/06
ORIGINATED BY ID
COMPILED BY AB
CHECKED BY SEP

--- CONTINUED FROM PREVIOUS PAGE ---

SOIL PROFILE			SAMPLES				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
	GRAVELLY SAND, trace to some silt Compact to dense Grey Wet						269
	Containing cobbles and boulders below 16.0m depth Spoon bouncing at 16.3m depth		11	SS	105/0.15		268
							267
266.4	Spoon bouncing at 17.6m depth		12	SS	107/0.2		
17.7	GNEISS (BEDROCK)						266
	Bedrock cored from 17.7m to 21.6m depth						265
							264
	For coring details see Record of Drillhole BH06-4						263
262.5 21.6	End of Borehole						



UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
	GR	SA	SI	CL

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

PROJECT: 06-1191-001

RECORD OF DRILLHOLE: BH06-4

SHEET 1 OF 1

LOCATION: N 5020401; E325134

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: 07/26/06

DRILL RIG: D90

DRILLING CONTRACTOR: Walker Drilling Ltd.

DATUM: Geodetic

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

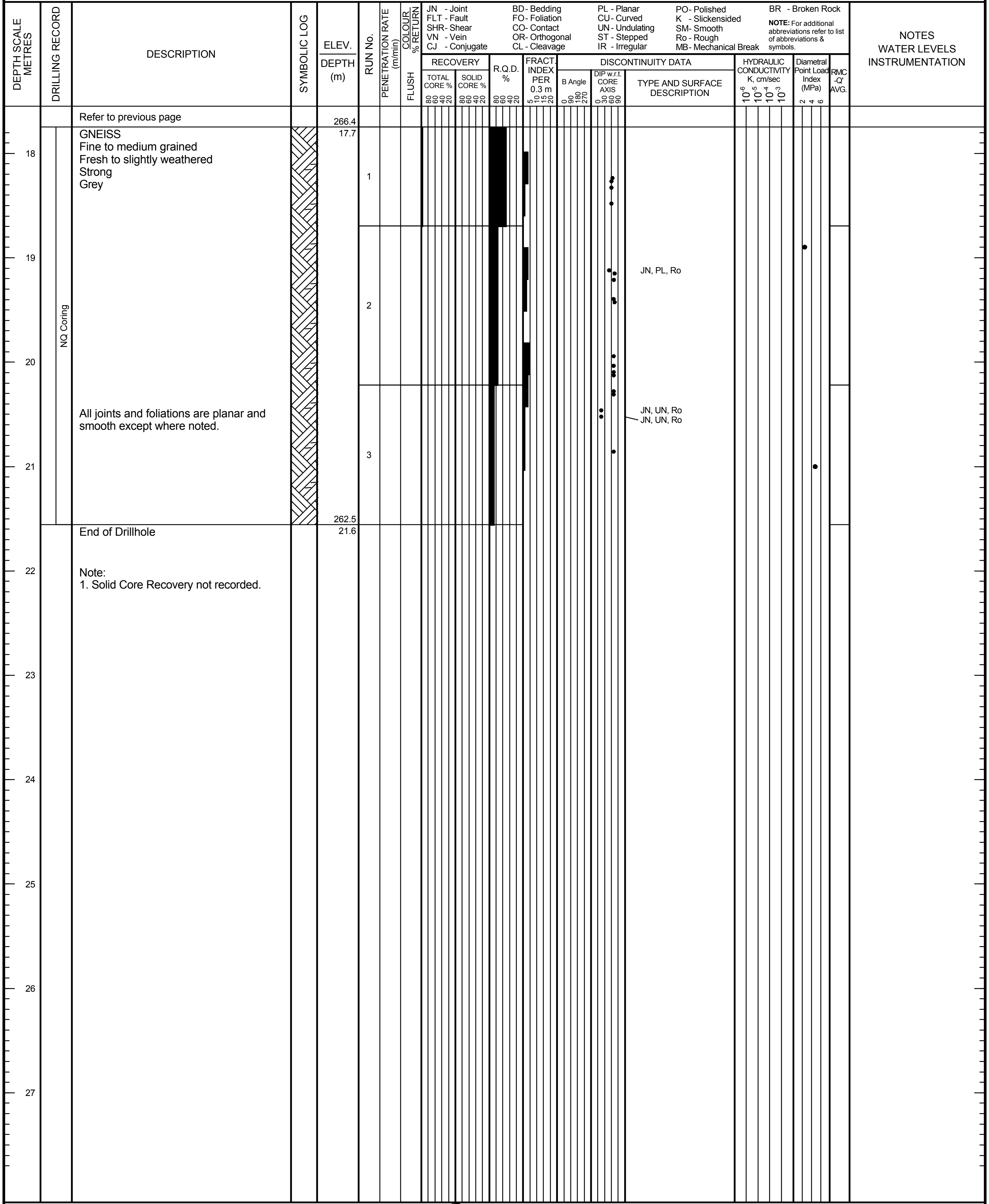
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

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.



DESCRIPTION	ELEV. DEPTH (m)
Refer to previous page	266.4
GNEISS Fine to medium grained Fresh to slightly weathered Strong Grey	17.7
All joints and foliations are planar and smooth except where noted.	
End of Drillhole	262.5 21.6
Note: 1. Solid Core Recovery not recorded.	

TYPE AND SURFACE DESCRIPTION:

JN, PL, Ro

JN, UN, Ro

JN, UN, Ro

DEPTH SCALE

1 : 50

LOGGED: ID

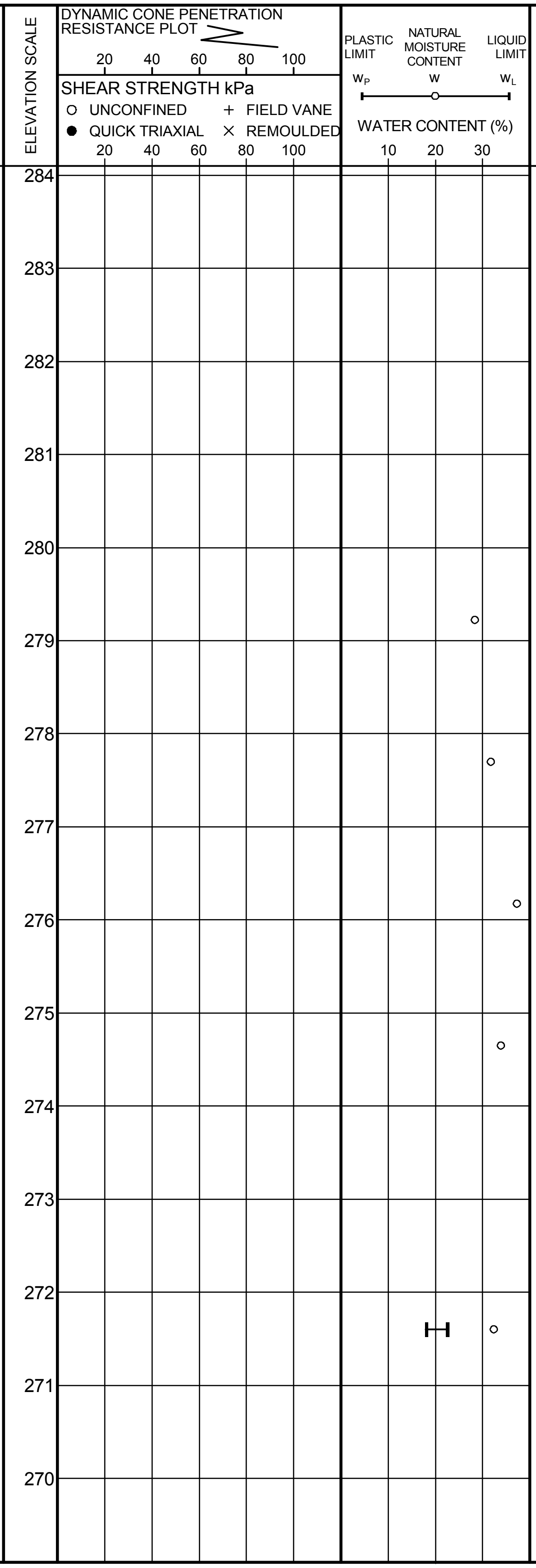
CHECKED: AB

RECORD OF BOREHOLE No BH06-5

1 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-01
LOCATION N 5020442; E325146
ORIGINATED BY ID
DIST 52 HWY 11
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY AB
DATUM Geodetic
DATE 07/27/06
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							
279.5 4.7	Organic SILT (ALLUVIUM) Black CLAYEY SILT, layered Very soft to stiff Grey Wet	1	SS	1				
		2	SS	2				
		3	SS	3				
		4	SS	3				
		5	SS	4				
	Silty clay layers above 9.1m depth	6	SS	4	0	0	77	23
		7	SS	6				
		8	SS	12				
		9	SS	13				



UNIT WEIGHT γ kN/m³

GROUND WATER CONDITIONS

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

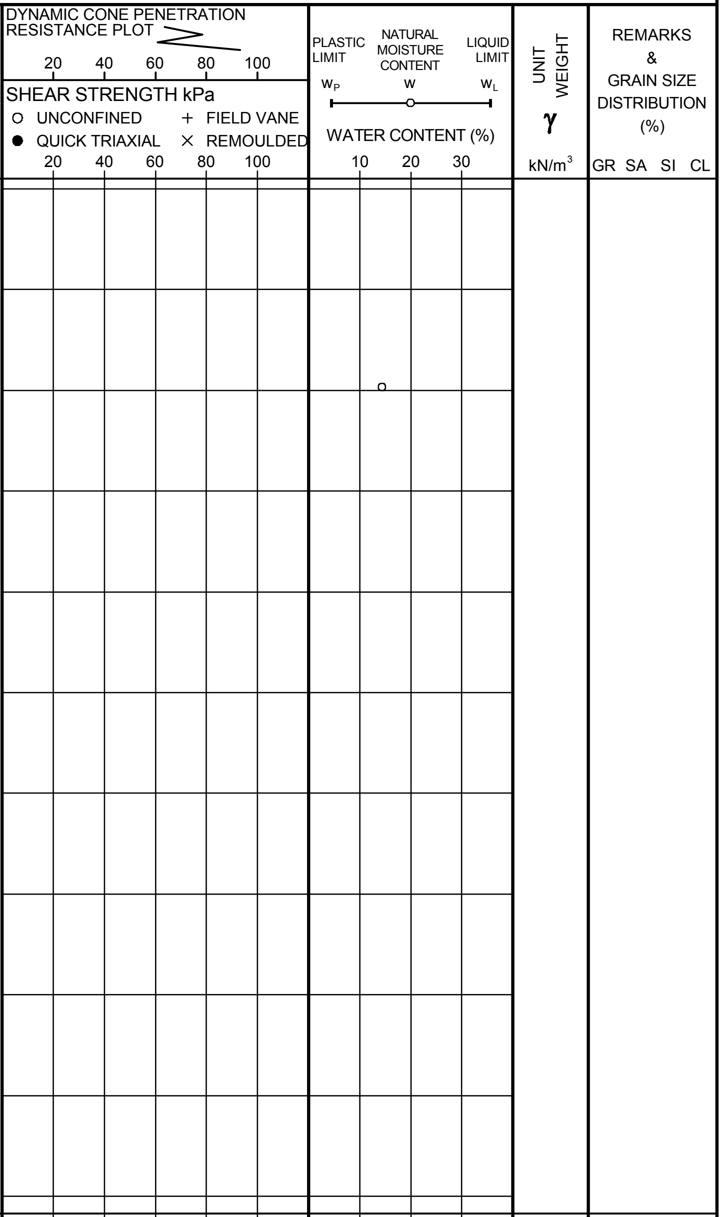
RECORD OF BOREHOLE No BH06-5

2 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-01
LOCATION N 5020442; E325146
ORIGINATED BY ID
DIST 52 HWY 11
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY AB
DATUM Geodetic
DATE 07/27/06
CHECKED BY SEP

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	CLAYEY SILT, layered Very soft to stiff Grey Wet	10	SS	14	269
267.9 16.2	GRAVELLY SAND, trace to some silt, containing cobbles and boulders Compact to very dense Grey Wet	11	SS	30	268 267
		12	SS	27	266 265
		13	SS	143	264 263
		14	SS	106/0.15	
262.6 21.5	GNEISS (BEDROCK) Bedrock cored from 21.5m to 25.3m depth For coring details see Record of Drillhole BH06-5				262 261 260 259
258.8 25.3	End of Borehole				



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

PROJECT: 06-1191-001

RECORD OF DRILLHOLE: BH06-5

SHEET 1 OF 1

LOCATION: N 5020442; E325146

INCLINATION: -90° AZIMUTH: ---

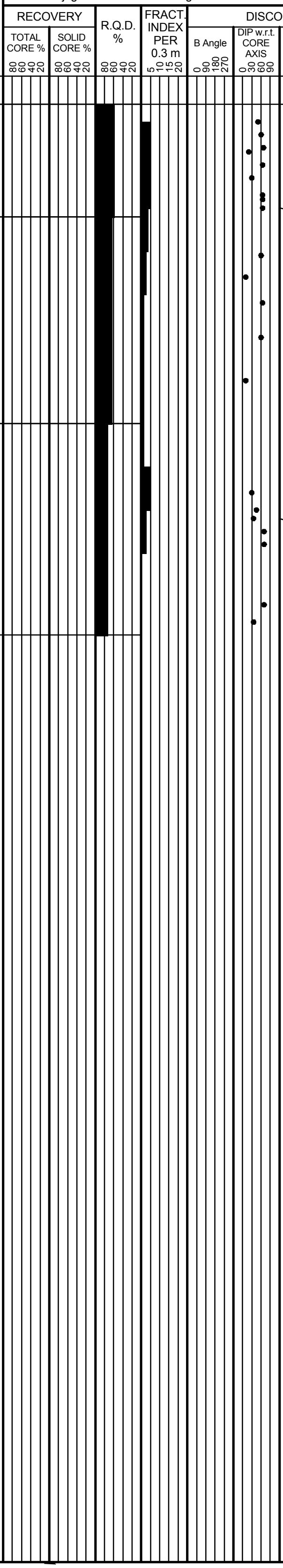
DRILLING DATE: 07/27/06

DRILL RIG: D90

DRILLING CONTRACTOR: Walker Drilling Ltd.

DATUM: Geodetic

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)	COLOUR % RETURN FLUSH	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page									
22–25	NQ Coring	GNEISS Fine to medium grained Fresh to slightly weathered Strong Grey		262.6 21.5	1						
					2						UCS=64MPa
					3						
		End of Drillhole Note: 1. Solid Core Recovery not recorded.		258.8 25.3							

RUN No.	TYPE AND SURFACE DESCRIPTION
1	FO, Ro, PL JN, Ro, UN JN, Ro, UN FO, Ro, PL FO, Ro, PL
2	JN, SM, PL JN, Ro, IR FO, Ro, PL FO, Ro, PL JN, Ro, IR
3	JN, Ro, UN JN, Ro, ST JN, Ro, ST FO, Ro, PL JN, SM, PL JN, SM, PL

DEPTH SCALE 22 23 24 25 26 27 28 29 30 31

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: ID

CHECKED: AB

Golder Associates

RECORD OF BOREHOLE No BH06-6

1 OF 2 **METRIC**

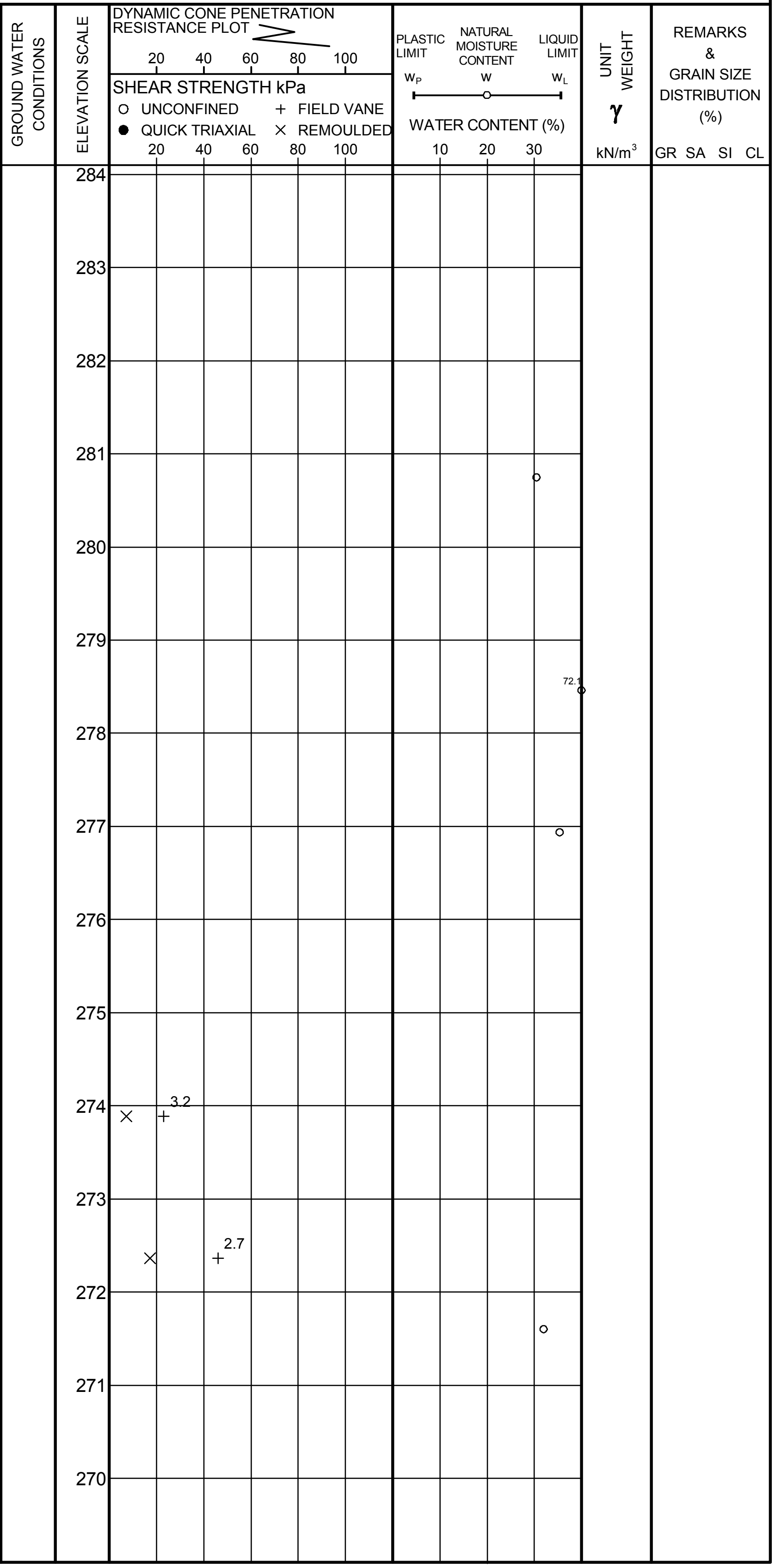
PROJECT 06-1191-001			
W.P. 5189-05-01	LOCATION N 5020484; E325159		ORIGINATED BY EHS
DIST 52	HWY 11	BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers	COMPILED BY AB
DATUM Geodetic	DATE 08/17/06		CHECKED BY SEP

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
284.1	WATER SURFACE
0.0	WATER
281.1	SANDY SILT, trace gravel, trace organics (ALLUVIUM)
280.7 3.4	CLAYEY SILT, some to trace organics, trace sand and gravel, trace shells (ALLUVIUM) Very soft Grey Wet
277.2 6.9	CLAYEY SILT, layered Very soft to very stiff Grey Wet
269.3 14.8	

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	WH
2	SS	WH
2A	-	-
3	SS	WH
4	SS	WH
5	SS	WH
5A	-	-
6	SS	WH
7	SS	4
8	SS	10
9	SS	18



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+3, ×3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH06-6

2 OF 2 **METRIC**

PROJECT 06-1191-001

W.P. 5189-05-01

LOCATION N 5020484; E325159

ORIGINATED BY EHS

DIST 52 HWY 11

BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers

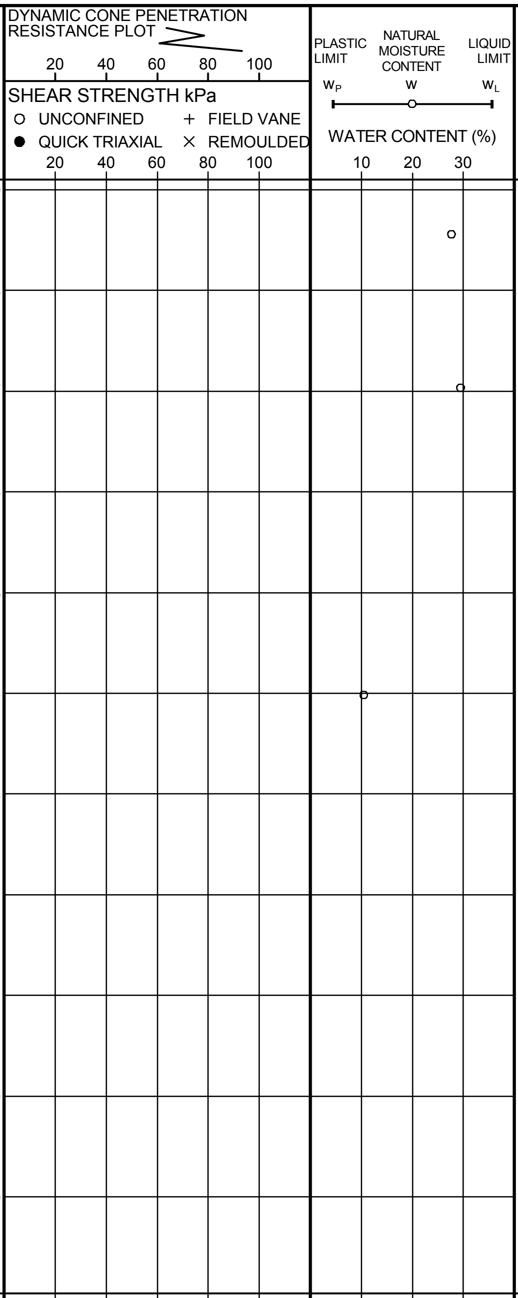
COMPILED BY AB

DATUM Geodetic

DATE 08/17/06

CHECKED BY SEP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES 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 ---											
	SILT, trace clay, trace sand Compact Grey Wet						269					
			10	SS	11				0	4	89	7
							268					
			11	SS	11		267					
266.3 17.8	GRAVELLY SAND, trace to some silt, containing cobbles and boulders Compact Grey Wet						266					
			12	SS	28							
							265					
			13	SS	29		264					
							263					
			14	SS	10							
262.0 22.1	GNEISS (BEDROCK) Bedrock cored from 22.1m to 26.1m depth For coring details see Record of Drillhole BH06-6						262 261 260 259					
258.0 26.1	End of Borehole Notes: 1. Split spoon Samples 2A and 5A were empty after sampling.											



+3, ×3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

PROJECT: 06-1191-001

RECORD OF DRILLHOLE: BH06-6

SHEET 1 OF 1

LOCATION: N 5020484; E325159

DRILLING DATE: 08/17/06

DATUM: Geodetic

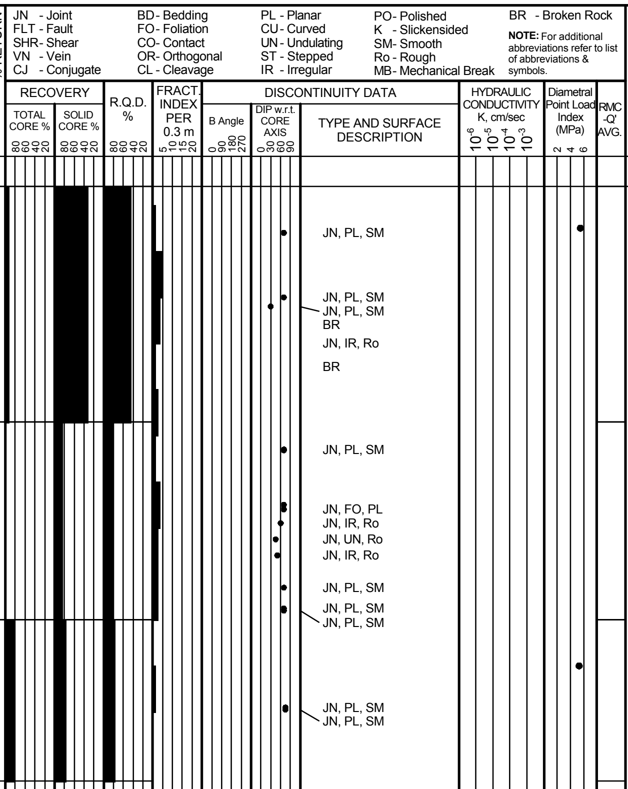
INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D90

DRILLING CONTRACTOR: Walker Drilling Ltd.

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)	COLOUR % RETURN FLUSH	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page		262.0					
	NQ Coring	GNEISS Fine to medium grained Fresh to slightly weathered Strong to very strong Grey		22.1	1			JN, PL, SM JN, PL, SM JN, PL, SM BR JN, IR, Ro BR	
23									
24					2			JN, PL, SM JN, FO, PL JN, IR, Ro JN, UN, Ro JN, IR, Ro JN, PL, SM JN, PL, SM JN, PL, SM	
25		Sand layer from 25.7m to 25.9m depth			3			JN, PL, SM JN, PL, SM	
26		End of Drillhole		258.0 26.1					
27									
28									
29									
30									
31									
32									



DEPTH SCALE

1 : 50

LOGGED: EHS

CHECKED: AB

RECORD OF BOREHOLE No BH06-7

1 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-01
LOCATION N 5020525; E325172
ORIGINATED BY EHS
DIST 52 HWY 11
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY AB
DATUM Geodetic
DATE 08/21/06
CHECKED BY SEP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
284.1	WATER SURFACE							
0.0	WATER							
282.3 1.8	CLAYEY SILT, trace to some organics, trace sand and gravel, trace shells (ALLUVIUM) Very soft Grey Wet	1	SS	WH				
		2	SS	WH	0	4	87	17
		3	SS	2				
		4	SS	WH				
		5	SS	WH				
278.8 5.3	CLAYEY SILT, layered Very soft to very stiff Grey Wet	6	SS	WH	0	2	68	30
		7	SS	WH				
		8	SS	4				
		9	SS	17				
272.4 11.7	SILT, trace clay, trace sand Compact Grey Wet	10	SS	16				
		11	SS	11				
269.3 14.8								

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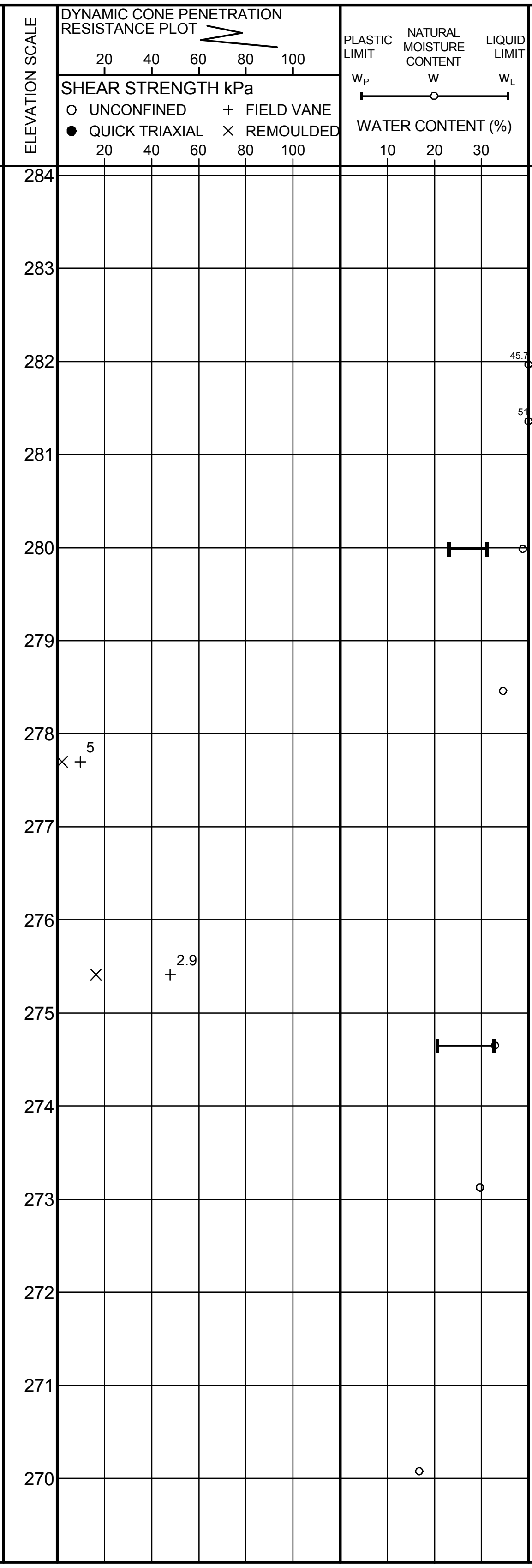


UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

Golder Associates

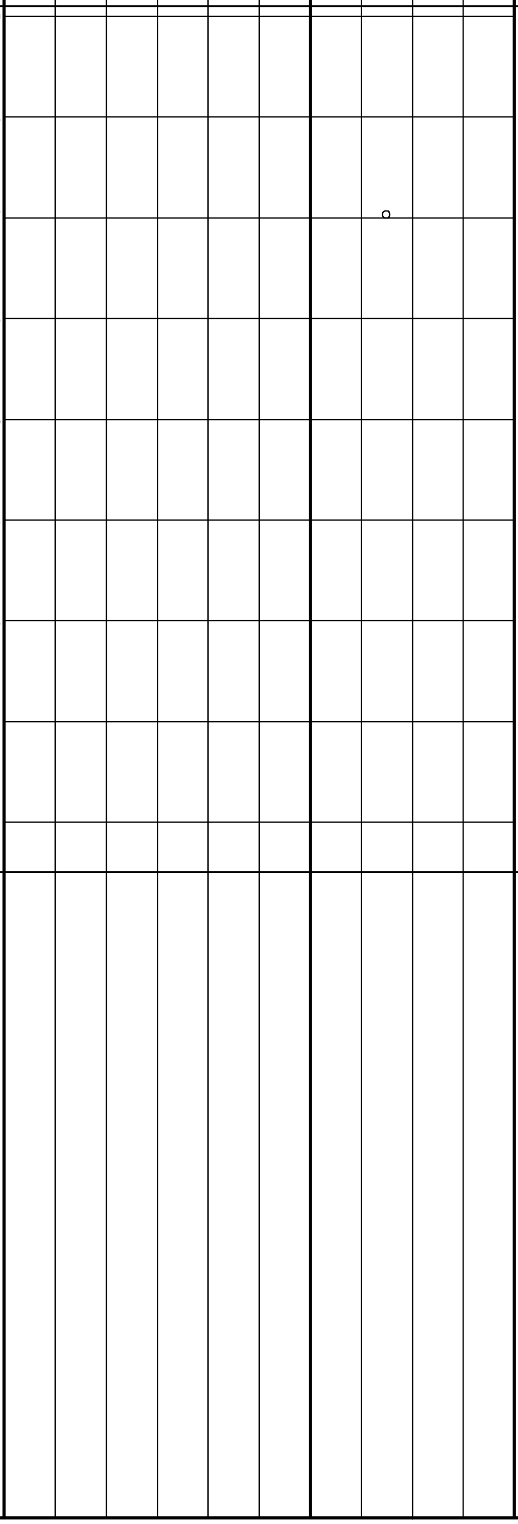
RECORD OF BOREHOLE No BH06-7

2 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-01
LOCATION N 5020525; E325172
ORIGINATED BY EHS
DIST 52 HWY 11
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY AB
DATUM Geodetic
DATE 08/21/06
CHECKED BY SEP

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



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, trace to some silt, containing cobbles and boulders Compact to loose Grey Wet	▨					269					
			12	SS	19							
							268					
			13	SS	6		267					
							266					
			14	SS	11/0.15							
	Spoon bouncing at 18.4m depth											
265.3 18.8	GNEISS (BEDROCK)	▧					265					
	Bedrock cored from 18.8m to 23.6m depth						264					
							263					
							262					
	For coring details see Record of Drillhole BH06-7						261					
260.5 23.6	End of Borehole											

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

MIS-MTO 001 06-1191-001 SOIL WP 5189-05-01.GPJ GAL-MISS.GDT 12/12/06 RN

PROJECT: 06-1191-001

RECORD OF DRILLHOLE: BH06-7

SHEET 1 OF 1

LOCATION: N 5020525; E325172

DRILLING DATE: 08/22/06

DATUM: Geodetic

DRILL RIG: D90

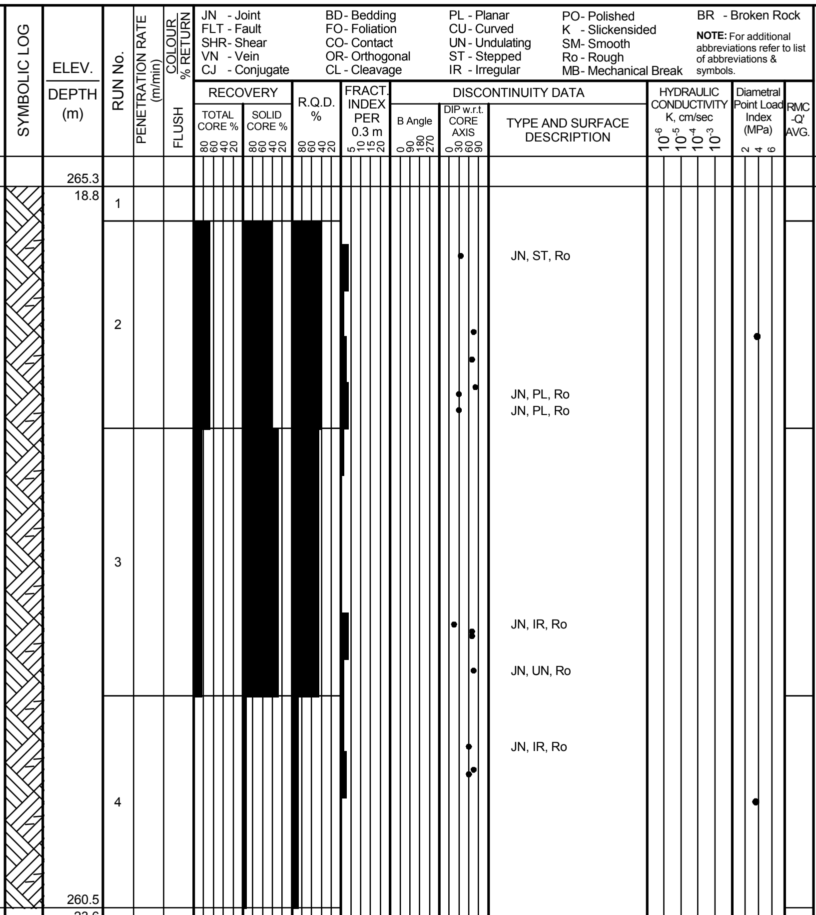
INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Walker Drilling Ltd.

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.	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page	265.3			
	NQ Coring	GNEISS Fine to medium grained Fresh to slightly weathered Strong Grey	18.8	1		
19				2	JN, ST, Ro	
20					JN, PL, Ro JN, PL, Ro	
21				3		
22					JN, IR, Ro JN, UN, Ro	
		All joints and foliations are planar and smooth except where noted.		4	JN, IR, Ro	
23			260.5 23.6			
		End of Drillhole				
24						
25						
26						
27						
28						



DEPTH SCALE
1 : 50

LOGGED: EHS
CHECKED: AB

RECORD OF BOREHOLE No BH06-8

1 OF 2 **METRIC**

PROJECT	06-1191-001		
W.P.	5189-05-01	LOCATION	N 5020560; E325183
DIST	52 HWY 11	BOREHOLE TYPE	NW Casing, Wash Boring
DATUM	Geodetic	DATE	07/31/06

ORIGINATED BY EHS
COMPILED BY AB
CHECKED BY SEP

SOIL PROFILE

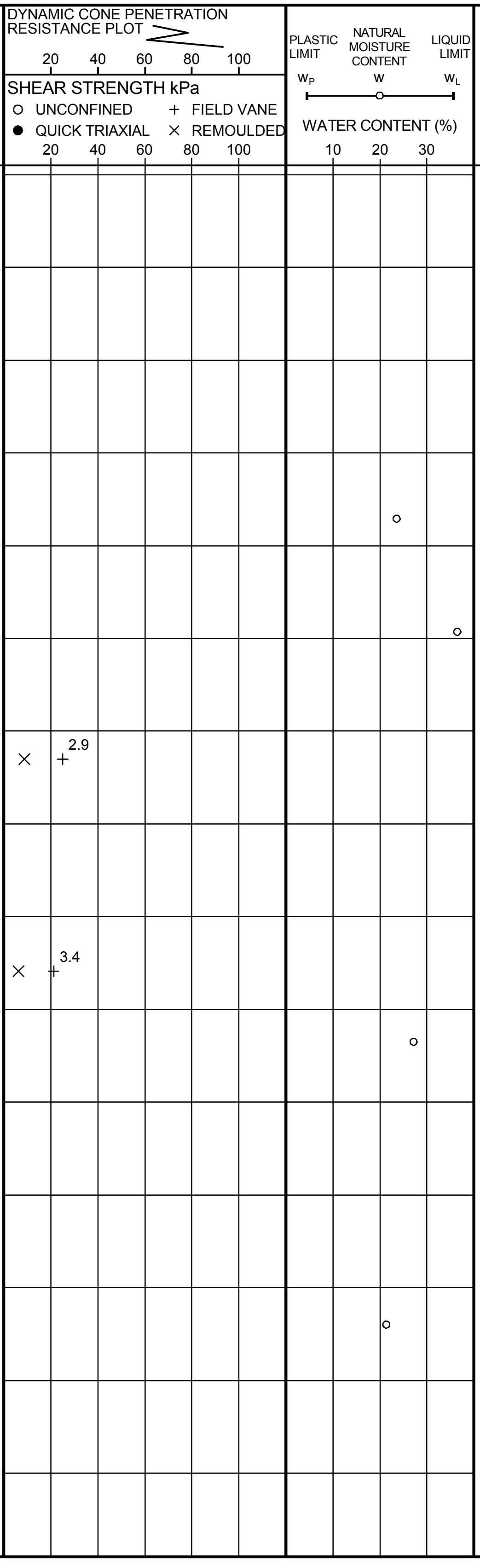
ELEV DEPTH	DESCRIPTION
284.1	WATER SURFACE
0.0	WATER
283.2 0.9	Sand, trace to some silt (FILL) Loose Brown Wet
280.0 4.1	CLAYEY SILT, layered Very soft to soft Grey Wet
275.3 8.8	SILT, trace clay, trace sand Compact Grey Wet
272.4 11.7	GRAVELLY SAND, trace to some silt, containing cobbles and boulders Compact to loose Grey Wet

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	5
2	SS	1
3	SS	4
4	SS	3
5	SS	12
6	SS	14
7	SS	17
8	SS	7

GROUND WATER CONDITIONS

ELEVATION SCALE: 284, 283, 282, 281, 280, 279, 278, 277, 276, 275, 274, 273, 272, 271, 270



UNIT WEIGHT γ kN/m³

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

Continued Next Page

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

RECORD OF BOREHOLE No BH06-8

2 OF 2 **METRIC**

PROJECT 06-1191-001

W.P. 5189-05-01

LOCATION N 5020560; E325183

ORIGINATED BY EHS

DIST 52 HWY 11

BOREHOLE TYPE NW Casing, Wash Boring

COMPILED BY AB

DATUM Geodetic

DATE 07/31/06

CHECKED BY SEP

SOIL PROFILE			SAMPLES					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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
							269				
268.7 15.4	GNEISS (BEDROCK)										
	Bedrock cored from 15.4m to 19.0m depth						268				
							267				
	For coring details see Record of Drillhole BH06-8						266				
265.1 19.0	End of Borehole										

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

PROJECT: 06-1191-001

RECORD OF DRILLHOLE: BH06-8

SHEET 1 OF 1

LOCATION: N 5020560; E325183

DRILLING DATE: 07/31/06

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

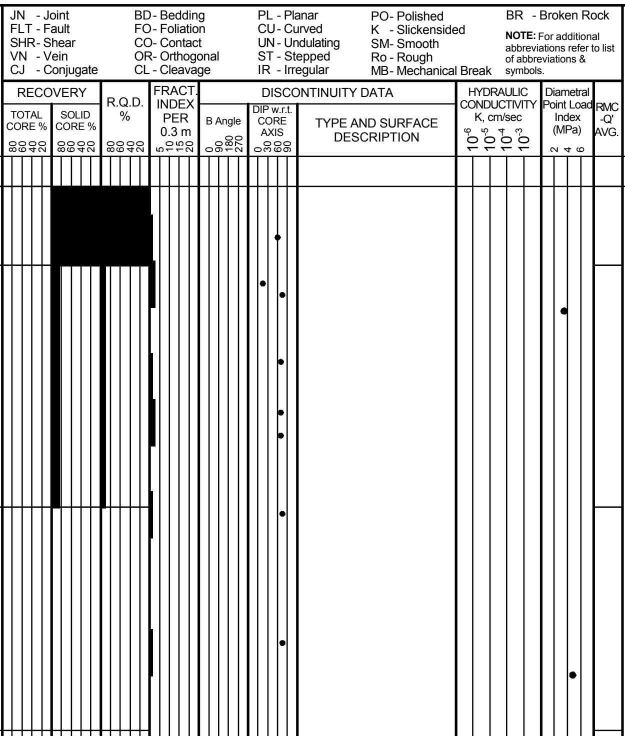
DRILL RIG: D90

DRILLING CONTRACTOR: Walker Drilling Ltd.

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		268.7					
		GNEISS Fine to medium grained Fresh to slightly weathered Strong Grey		15.4	1				
16									
					2				
17									
		All joints and foliations are irregular and rough.							
18					3				
19		End of Drillhole		265.1 19.0					
20									
21									
22									
23									
24									
25									



DEPTH SCALE

1 : 50

Golder Associates

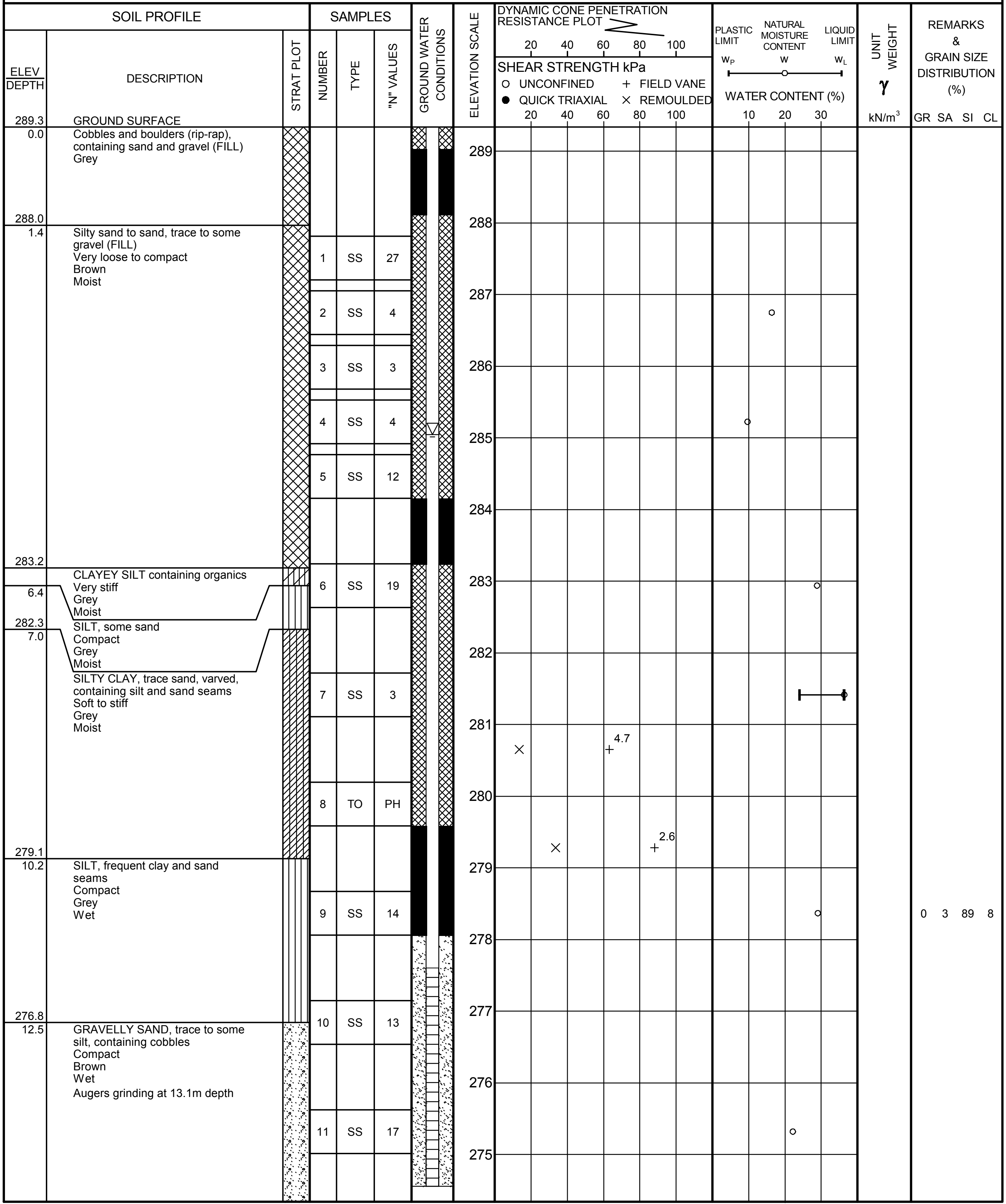
LOGGED: EHS

CHECKED: AB

RECORD OF BOREHOLE No BH06-9

1 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-01
DIST 52 HWY 11
DATUM Geodetic
LOCATION N 5020588; E325187
BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers
DATE 06/28/06
ORIGINATED BY EHS
COMPILED BY AB
CHECKED BY SEP



ELEV DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
289.3	GROUND SURFACE								
0.0	Cobbles and boulders (rip-rap), containing sand and gravel (FILL) Grey								
288.0 1.4	Silty sand to sand, trace to some gravel (FILL) Very loose to compact Brown Moist	1	SS	27					
		2	SS	4					
		3	SS	3					
		4	SS	4					
		5	SS	12					
283.2	CLAYEY SILT containing organics Very stiff Grey Moist	6	SS	19					
6.4									
282.3 7.0	SILT, some sand Compact Grey Moist								
	SILTY CLAY, trace sand, varved, containing silt and sand seams Soft to stiff Grey Moist	7	SS	3					
		8	TO	PH					
279.1 10.2	SILT, frequent clay and sand seams Compact Grey Wet	9	SS	14		0	3	89	8
276.8 12.5	GRAVELLY SAND, trace to some silt, containing cobbles Compact Brown Wet Augers grinding at 13.1m depth	10	SS	13					
		11	SS	17					

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH06-9

2 OF 2 **METRIC**

PROJECT	06-1191-001		
W.P.	5189-05-01	LOCATION	N 5020588; E325187
DIST	52 HWY 11	BOREHOLE TYPE	Power Auger, 108mm ID Hollow Stem Augers
DATUM	Geodetic	DATE	06/28/06

ORIGINATED BY EHS
COMPILED BY AB
CHECKED BY SEP

ELEV DEPTH	SOIL PROFILE DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
274.0 15.3	COBBLES and BOULDERS in a sand and gravel matrix		12	SS	30/0.05		274 273				
272.7 16.6	GNEISS (BEDROCK) Bedrock cored from 16.6m to 19.7m depth For coring details see Record of Drillhole BH06-9						272 271 270				
269.6 19.7	End of Borehole Notes: 1. Heave in augers to 11.3m depth upon reaching Sample 10. 2. Switch to NW Casing at 15.3m depth. 3. Water level measured in piezometer at 4.3m depth (Elev. 285.0m) on August 24, 2006.										

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

PROJECT: 06-1191-001

RECORD OF DRILLHOLE: BH06-9

SHEET 1 OF 1

LOCATION: N 5020588; E325187

DRILLING DATE: 06/29/06

DATUM: Geodetic

DRILL RIG: CME 55

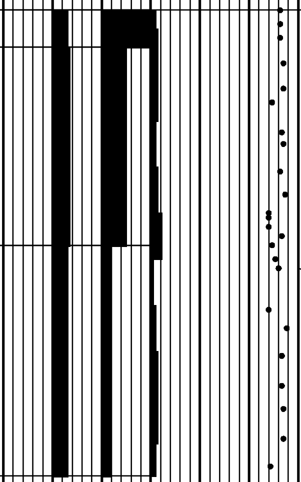
INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Walker Drilling Ltd.

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	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	Refer to previous page				
	GNEISS Fine to medium grained Fresh to slightly weathered Strong to very strong Grey	272.7 16.6	1		
17			2	JN, IR, Ro JN, IR, Ro JN, UN, Ro JN, IR, Ro	
18			3	JN, UN, Ro JN, UN, Ro	UCS=113MPa
19	All joints and foliations are planar and smooth except where noted			JN, IR, Ro JN, PL, Ro JN, IR, Ro	
	End of Drillhole	269.6 19.7			
20					
21					
22					
23					
24					
25					
26					



DEPTH SCALE
1 : 50

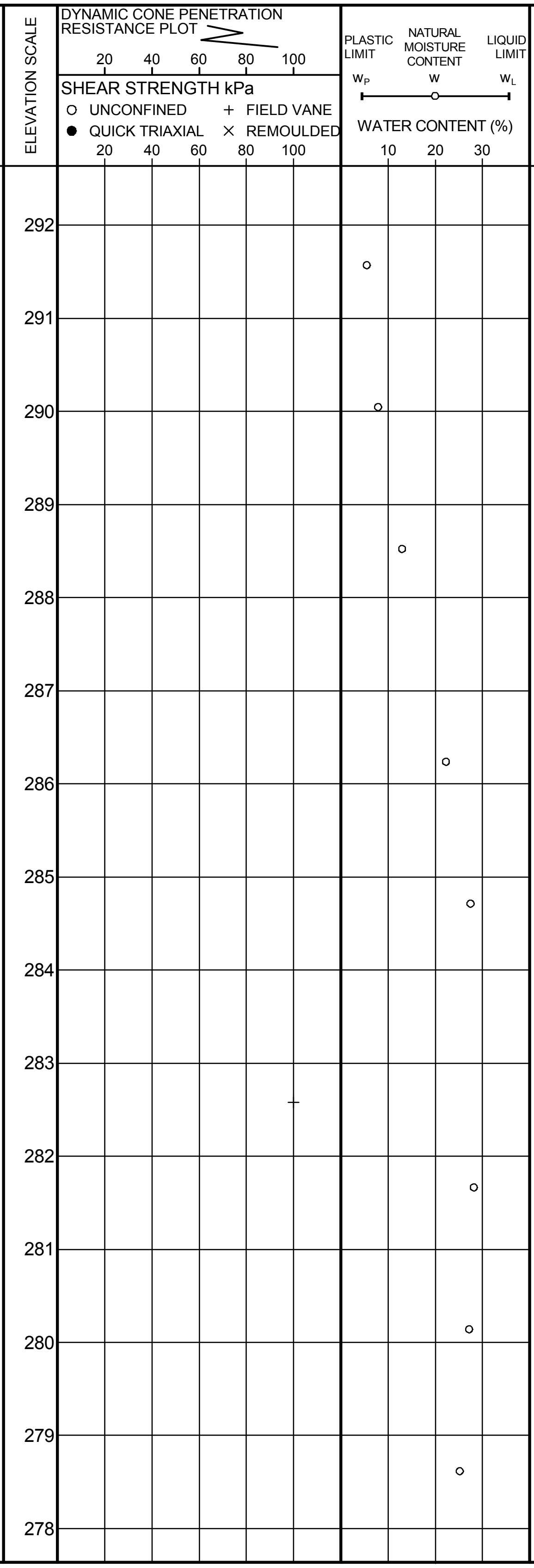
LOGGED: EHS
CHECKED: AB

RECORD OF BOREHOLE No BH06-10

1 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-01
DIST 52 HWY 11
DATUM Geodetic
LOCATION N 5020610; E325199
BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers
DATE 07/07/06
ORIGINATED BY EHS
COMPILED BY AB
CHECKED BY SEP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
292.6	GROUND SURFACE								
0.0	ASPHALT								
0.3	Sand and gravel (FILL) Brown								
	Sand, trace to some silt, trace to some gravel, occasional cobbles (FILL) Compact to very dense Brown Moist	1	SS	31					
		2	SS	30					
		3	SS	23					
	Augers grinding at 3.4m, 4.6m, 4.9m and 5.5m depths	4	SS	28					
		5	SS	53					
		6	SS	45					
	Becoming loose below 5.8m depth								
		7	SS	2					
284.7 7.9	SILTY CLAY, trace sand, trace organics, varved Very stiff Grey Moist to wet	8	SS	13					
		9	TO	PH	18.5				
		10	SS	19					
281.1 11.6	SILT, trace sand, trace clay Compact Brown Wet	11	SS	10		0	3	89	8
		12	SS	12					
278.0 14.6	Augers grinding at 14.6m depth								



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

Golder Associates

RECORD OF BOREHOLE No BH06-10

2 OF 2 **METRIC**

PROJECT 06-1191-001
W.P. 5189-05-01
DIST 52 HWY 11
DATUM Geodetic
LOCATION N 5020610; E325199
BOREHOLE TYPE Power Auger, 108mm ID Hollow Stem Augers
DATE 07/07/06
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	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	GRAVELLY SAND, trace to some silt, containing cobbles Compact Brown Wet		13	SS	20		277				
276.8 15.8	End of Borehole Notes: 1. Borehole dry upon completion of drilling										

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

MIS-MTO 001 06-1191-001 SOIL WP 5189-05-01.GPJ GAL-MISS.GDT 12/12/06 RN

MINISTRY OF TRANSPORTATION, ONTARIO

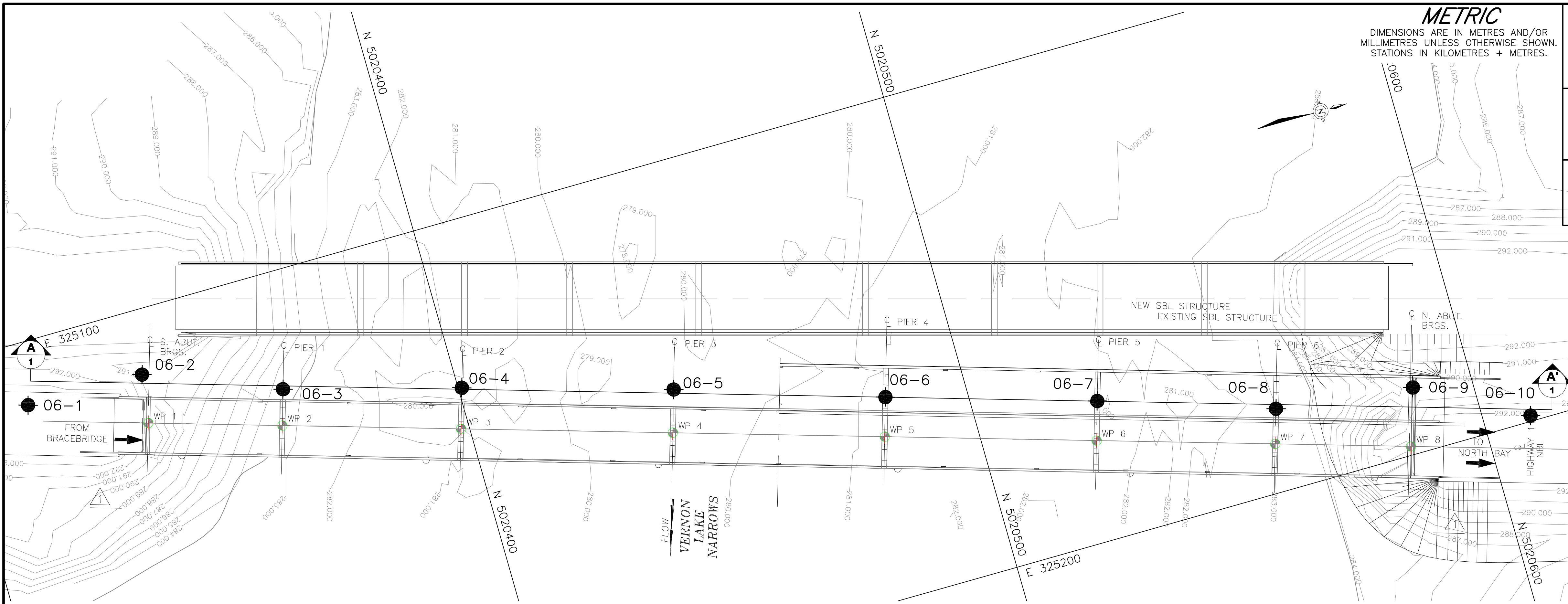
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5189-05-01

VERNON LAKE NARROWS
NBL STRUCTURE WIDENING
BOREHOLE LOCATION AND
SOIL STRATA

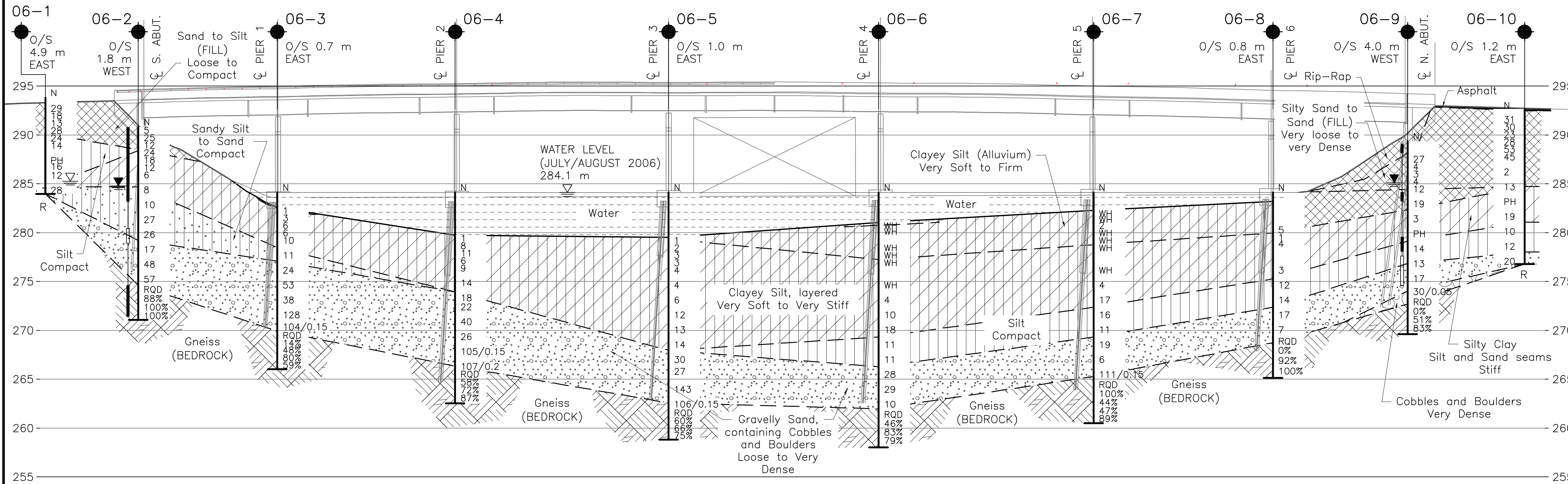
SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

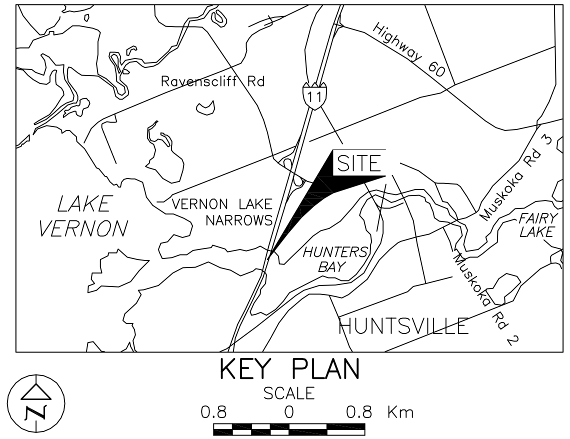
SCALE
10 0 10 20 m



PROFILE A-A'

SCALE
HORIZ. 10 0 10 20 m

SCALE
VERT. 5 0 5 10 m



KEY PLAN
SCALE
0.8 0 0.8 Km

LEGEND

●	Borehole – Current Investigation
▮	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
06-1	293.80	5020314	325113
06-2	290.79	5020339	325113
06-3	284.10	5020365	325124
06-4	284.10	5020401	325134
06-5	284.10	5020442	325146
06-6	284.10	5020484	325159
06-7	284.10	5020525	325172
06-8	284.10	5020560	325183
06-9	289.34	5020588	325187
06-10	292.64	5020610	325199

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.

S.E.M. POOT
DEC. 15, 2006

J.M.A. COSTA
DEC. 15, 2006

NO.	DATE	BY	REVISION

Geocres No. 31E-268

HWY. 11		PROJECT NO. 06-1191-001	DIST. 52
SUBM'D. AB	CHKD. SEP	DATE: DEC 2006	SITE: 42-018
DRAWN: MSM	CHKD. JMAC	APPD.	DWG. 1

PLOT DATE: March 26, 2007
FILENAME: T:\Projects\2006\06-1191-001 (LEA, Huntsville)\-AC- NBL Bridge (Final Report)\061191001AC001.dwg

STABILITY ANALYSIS

South Approach

FIGURE 1

Material #: 1
Water
Wt: 9.807

Material #: 2
New Fill
Wt: 21
Phi: 35

Material #: 3
Existing Fill
Wt: 21
Phi: 32

Material #: 4
Clayey Silt (toe of slope)
Wt: 18
C-Top of Layer: 20
C-Rate of Increase: 4.4
Limiting C: 40

Material #: 5
Clayey Silt to Silty Clay
Wt: 18
Cohesion: 60

Material #: 6
Silty Sand to Sandy Silt
Wt: 19
Phi: 30

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

Material #: 8
Bedrock

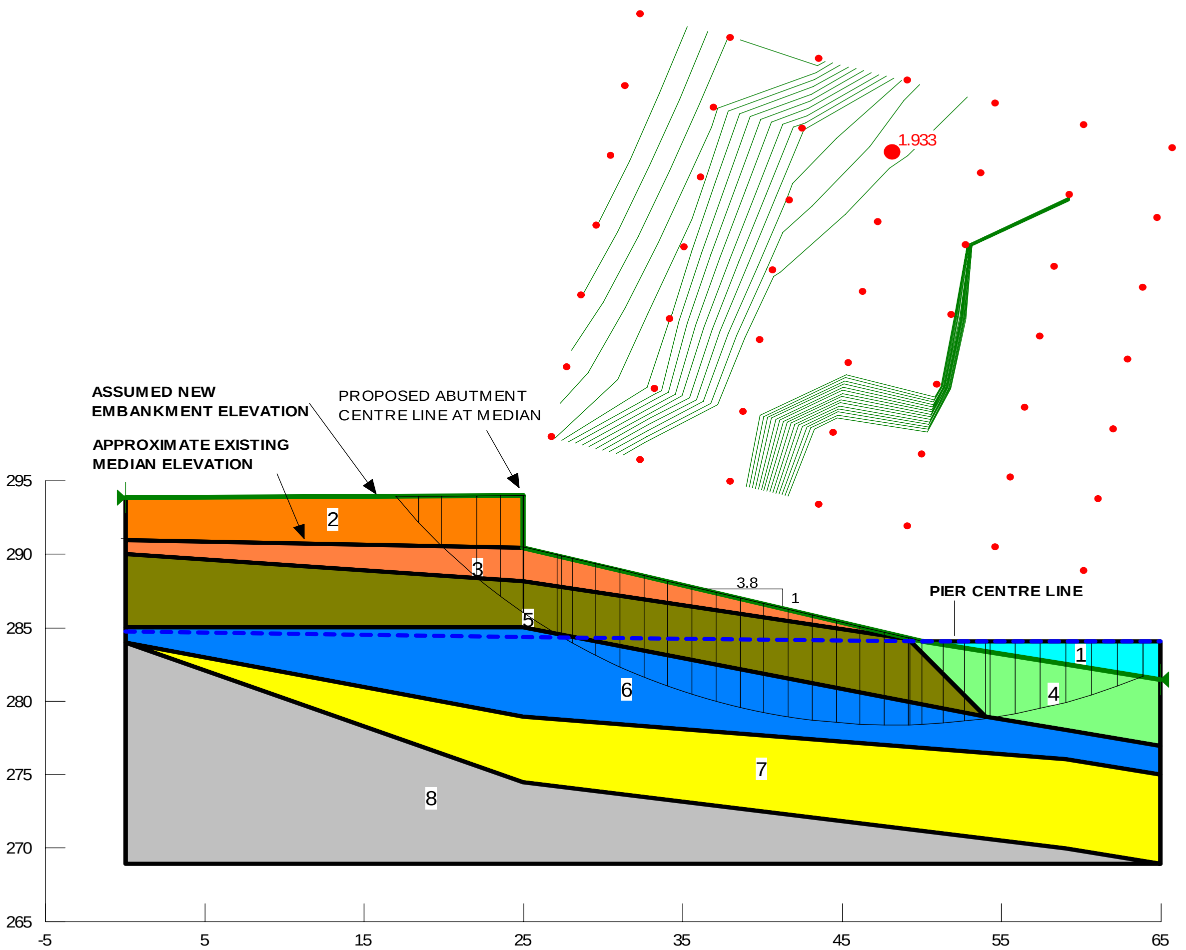


Fig 1 NBL Sslope.gsz

Date: December 2006
Project: 06-1191-001-N

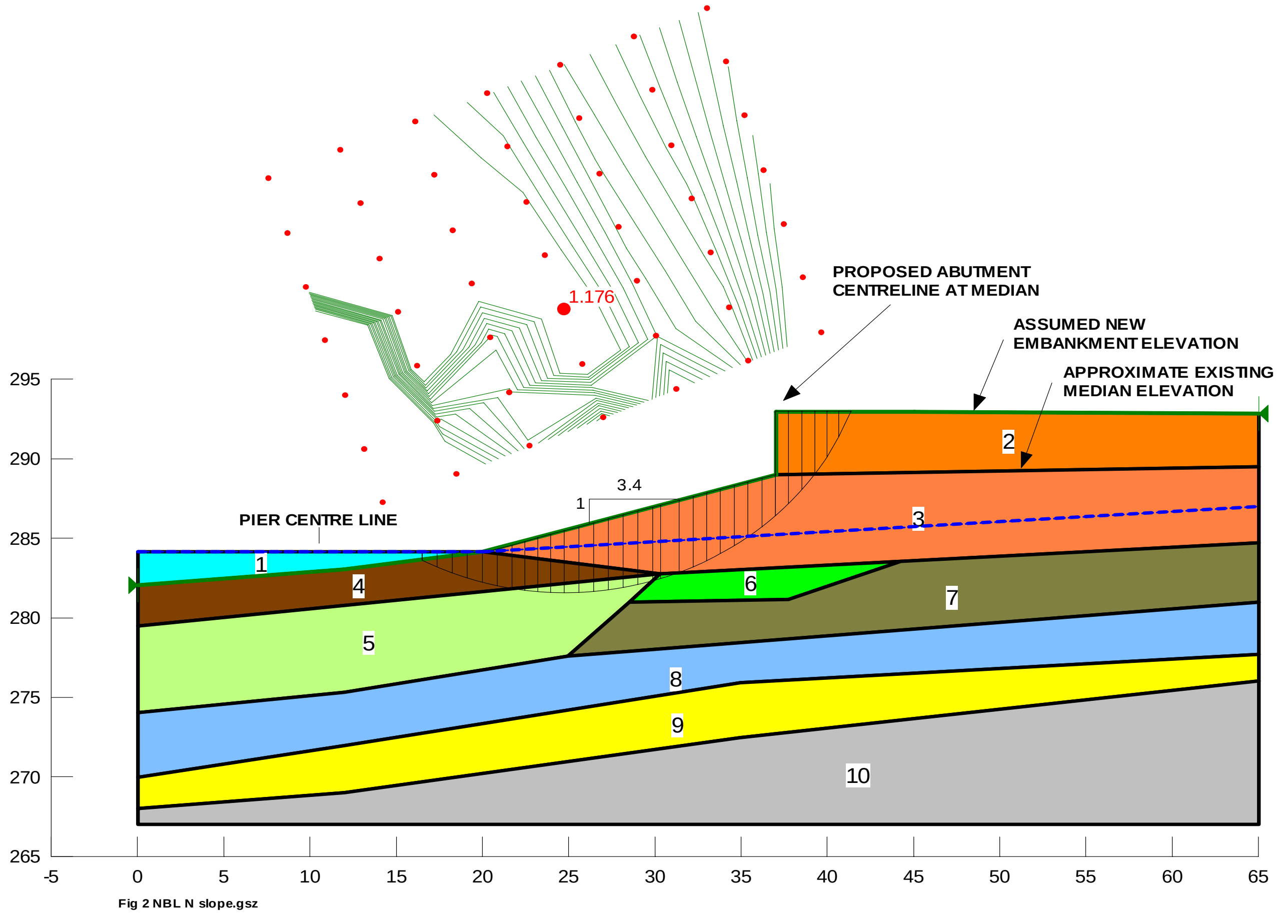
Golder Associates

Drawn: AB
Checked: JMAC

STABILITY ANALYSIS

North Approach - without Sub-excavation (Effective Stress)

FIGURE 2



Material #: 1
Description: Water
Wt: 9.807

Material #: 2
Description: New Earth Fill
Wt: 21
Phi: 35

Material #: 3
Description: Existing Granular Fill
Wt: 21
Phi: 32

Material #: 4
Description: Alluvium
Wt: 15
Cohesion: 10

Material #: 5
Description: Clayey Silt (toe of slope)
Wt: 18
Cohesion: 20

Material #: 6
Description: Clayey Silt to Silty Clay
Wt: 18
Phi: 26

Material #: 7
Description: Clayey Silt to Silty Clay
Wt: 18
Phi: 28

Material #: 8
Description: Silt
Wt: 19
Phi: 30

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

Material #: 10
Description: Bedrock

Date: December 2006
Project: 06-1191-001-N

Golder Associates

Drawn: AB
Checked: JMAC

STABILITY ANALYSIS

North Approach with Sub-excavation (Effective Stress)

FIGURE 3A

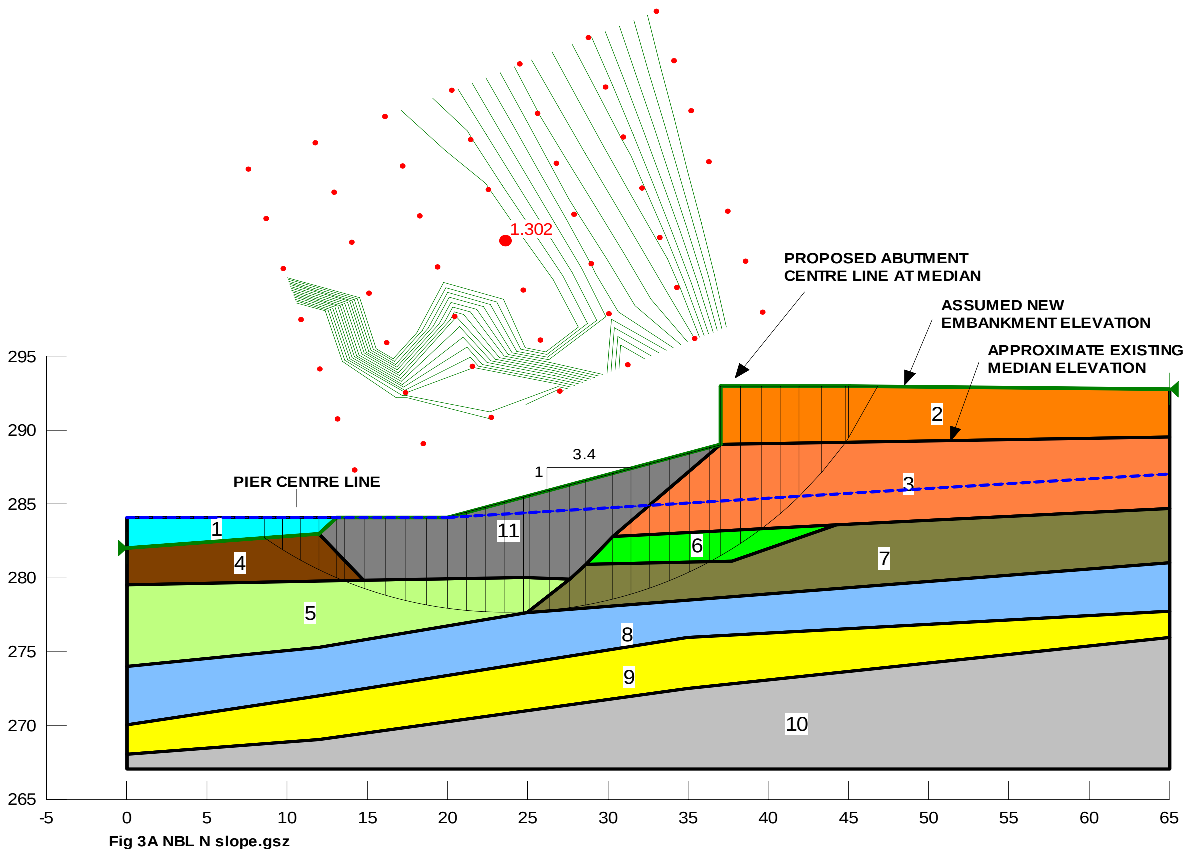


Fig 3A NBL N slope.gsz

Material #: 1
Description: Water
Wt: 9.807

Material #: 2
Description: New Earth Fill
Wt: 21
Phi: 35

Material #: 3
Description: Existing Granular Fill
Wt: 21
Phi: 32

Material #: 4
Description: Alluvium
Wt: 15
Cohesion: 10

Material #: 5
Description: Clayey Silt (toe of slope)
Wt: 18
Cohesion: 20

Material #: 6
Description: Clayey Silt to Silty Clay
Wt: 18
Phi: 26

Material #: 7
Description: Clayey Silt to Silty Clay
Wt: 18
Phi: 28

Material #: 8
Description: Silt
Wt: 19
Phi: 30

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

Material #: 10
Description: Bedrock

Material #: 11
Description: Granular Backfill
Wt: 20
Phi: 30

Date: December 2006
Project: 06-1191-001-N

Golder Associates

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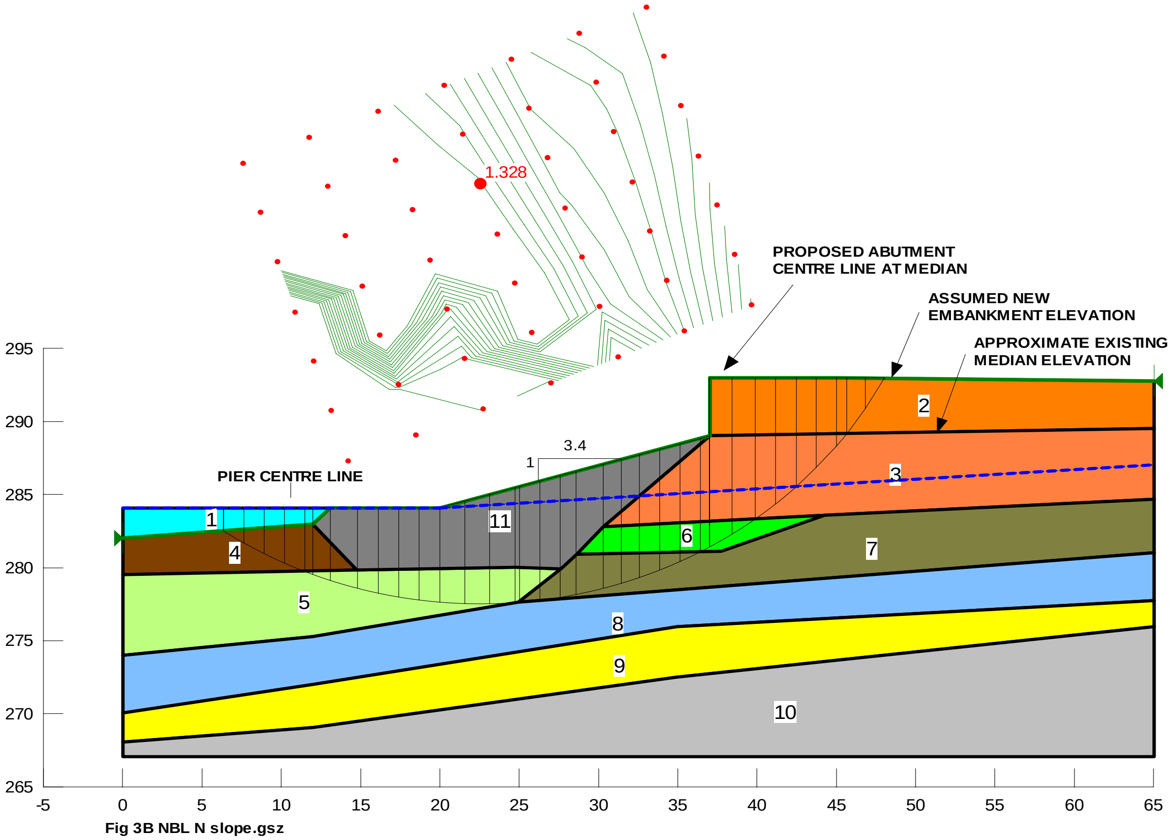
Checked: JMAC

STABILITY ANALYSIS

North Approach with Sub-excavation (Total Stress)

FIGURE 3B



Material #: 1
Description: Water
Wt: 9.807

Material #: 2
Description: New Earth Fill
Wt: 21
Phi: 35

Material #: 3
Description: Existing Granular Fill
Wt: 21
Phi: 32

Material #: 4
Description: Alluvium
Wt: 15
Cohesion: 10

Material #: 5
Description: Clayey Silt (toe of slope)
Wt: 18
Cohesion: 20

Material #: 6
Description: Clayey Silt to Silty Clay
Wt: 18
Cohesion: 30

Material #: 7
Description: Clayey Silt to Silty Clay
Wt: 18
Cohesion: 60

Material #: 8
Description: Silt
Wt: 19
Phi: 30

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

Material #: 10
Description: Bedrock

Material #: 11
Description: Granular Backfill
Wt: 20
Phi: 30

Date: December 2006
Project: 06-1191-001-N

Golder Associates

Drawn: AB
Checked: JMAC

APPENDIX A

LABORATORY TEST RESULTS

TABLE A-1
UNIAXIAL COMPRESSION STRENGTH TEST RESULTS
REHABILITATION AND WIDENING OF NORTHBOUND STRUCTURE
G.W.P 5189-05-00, 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)*
06-2	16.6	274.2	Gneiss	47.0	134.9	78
06-5	23.0	261.1	Gneiss	47.0	111.8	64
06-9	18.4	270.9	Gneiss	47.0	195.5	113

Compiled by: **AB**
Checked by: **JMAC**

TABLE A-2
POINT LOAD STRENGTH TEST RESULTS
REHABILITATION AND WIDENING OF NORTHBOUND STRUCTURE
G.W.P 5189-05-00 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)*
06-2	16.5	274.3	Gneiss	D	47.0	11.2	0.011	4.8	4.7	94
06-2	18.9	271.9	Gneiss	D	47.0	11.3	0.011	4.83	4.7	94
06-4	18.9	265.2	Gneiss	D	47.0	5.9	0.006	2.52	2.5	50
06-4	21.0	263.1	Gneiss	D	47.0	11.6	0.011	4.98	4.8	96
06-6	22.4	261.7	Gneiss	D	47.0	13.1	0.012	5.64	5.5	110
06-6	25.3	258.8	Gneiss	D	47.0	12.6	0.012	5.42	5.3	106
06-7	19.8	264.3	Gneiss	D	47.0	9.1	0.009	3.89	3.8	76
06-7	22.9	261.2	Gneiss	D	47.0	8.5	0.008	3.65	3.6	72
06-8	16.2	267.9	Gneiss	D	47.0	8.3	0.008	3.55	3.5	70
06-8	18.6	265.5	Gneiss	D	47.0	11.4	0.011	4.89	4.8	96
06-9	17.4	271.9	Gneiss	D	47.0	9.1	0.009	3.91	3.8	76
06-9	19.1	270.2	Gneiss	D	47.0	13.0	0.012	5.59	5.4	108

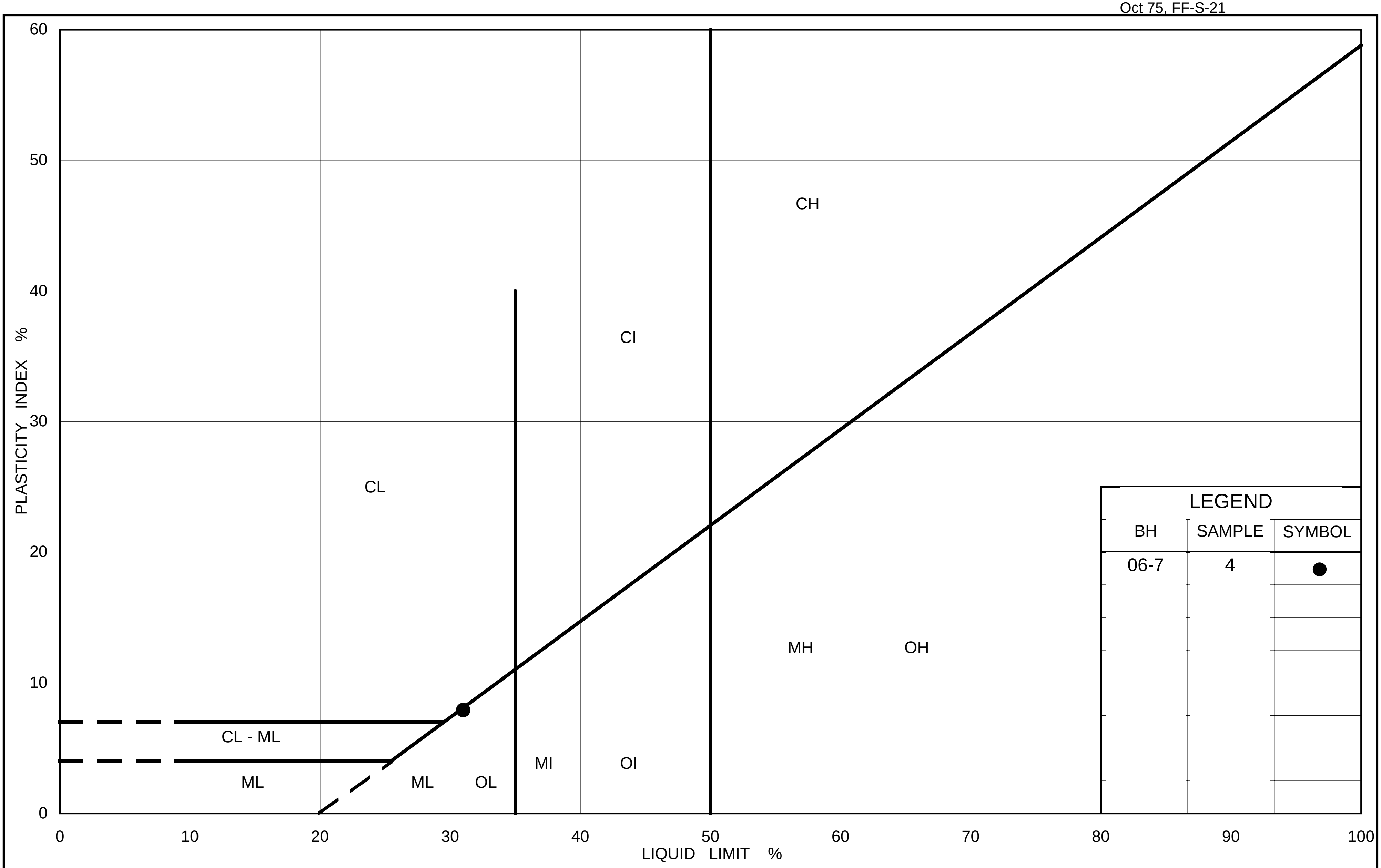
Average[3]	4.4	89

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

Compiled by: **AB**
Checked by: **JMAC**

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
06-7	4	●

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PLASTICITY CHART
Clayey Silt (Alluvium)

FIG No. A-1

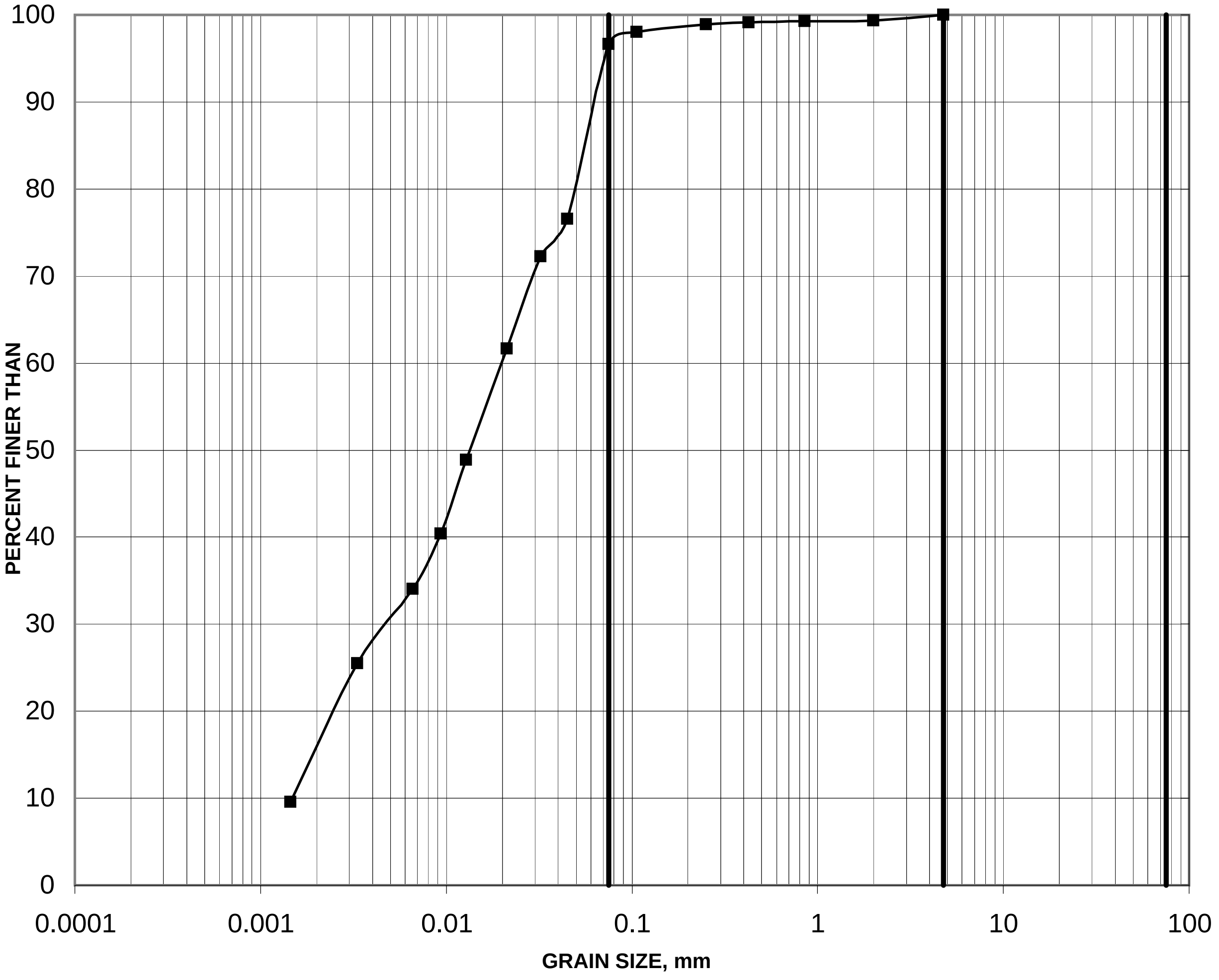
Project No. 06-1191-001-N

Checked By: JMAC

GRAIN SIZE DISTRIBUTION

Clayey Silt (Alluvium)

FIGURE A-2



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

Borehole	Sample	Elevation (m)
■ 06-7	2	281.5

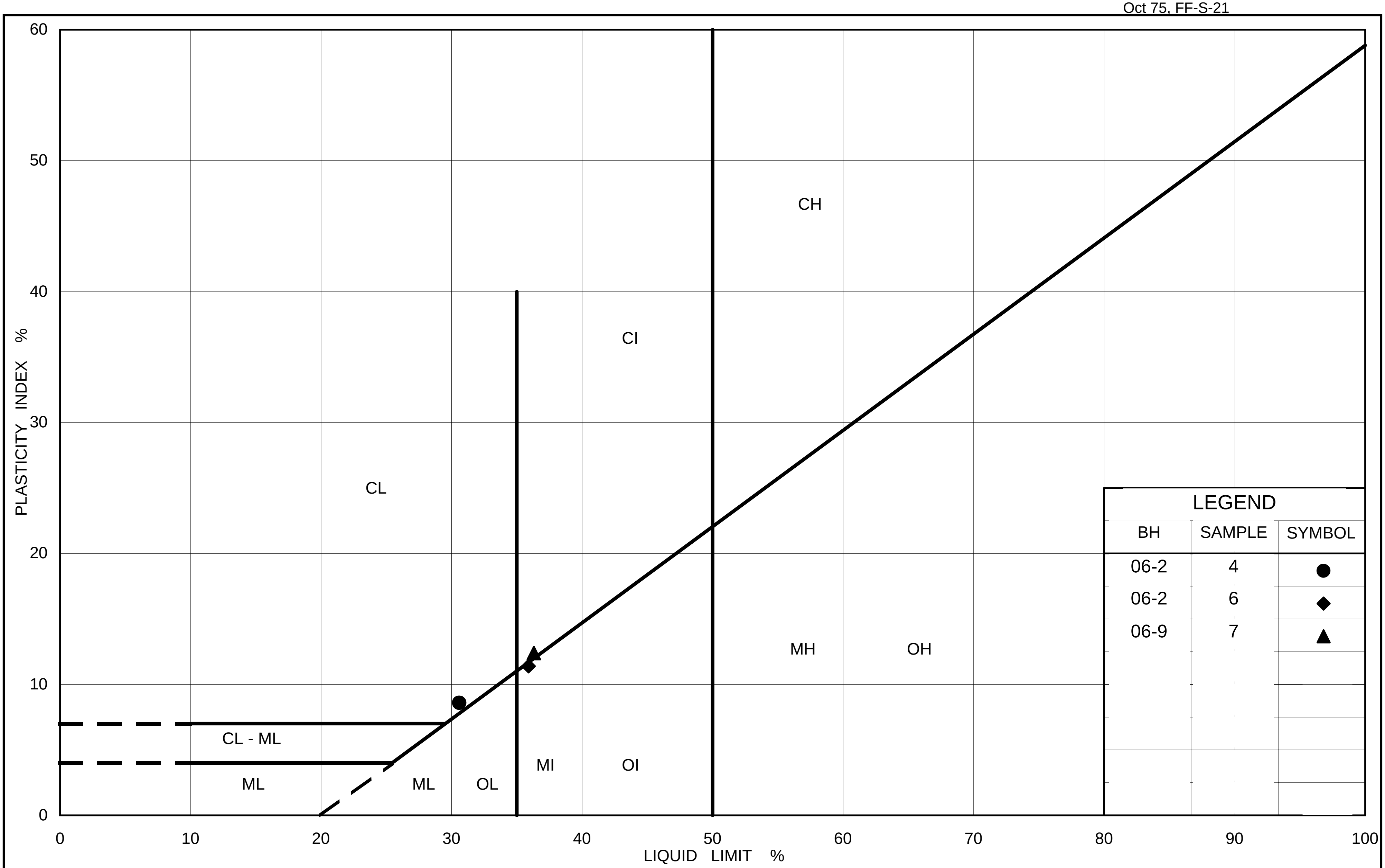
Date: December 2006

Project: 06-1191-001-N

Golder Associates

Created by: SL

Checked by: JMAC



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Ontario

PLASTICITY CHART
Clayey Silt to Silty Clay

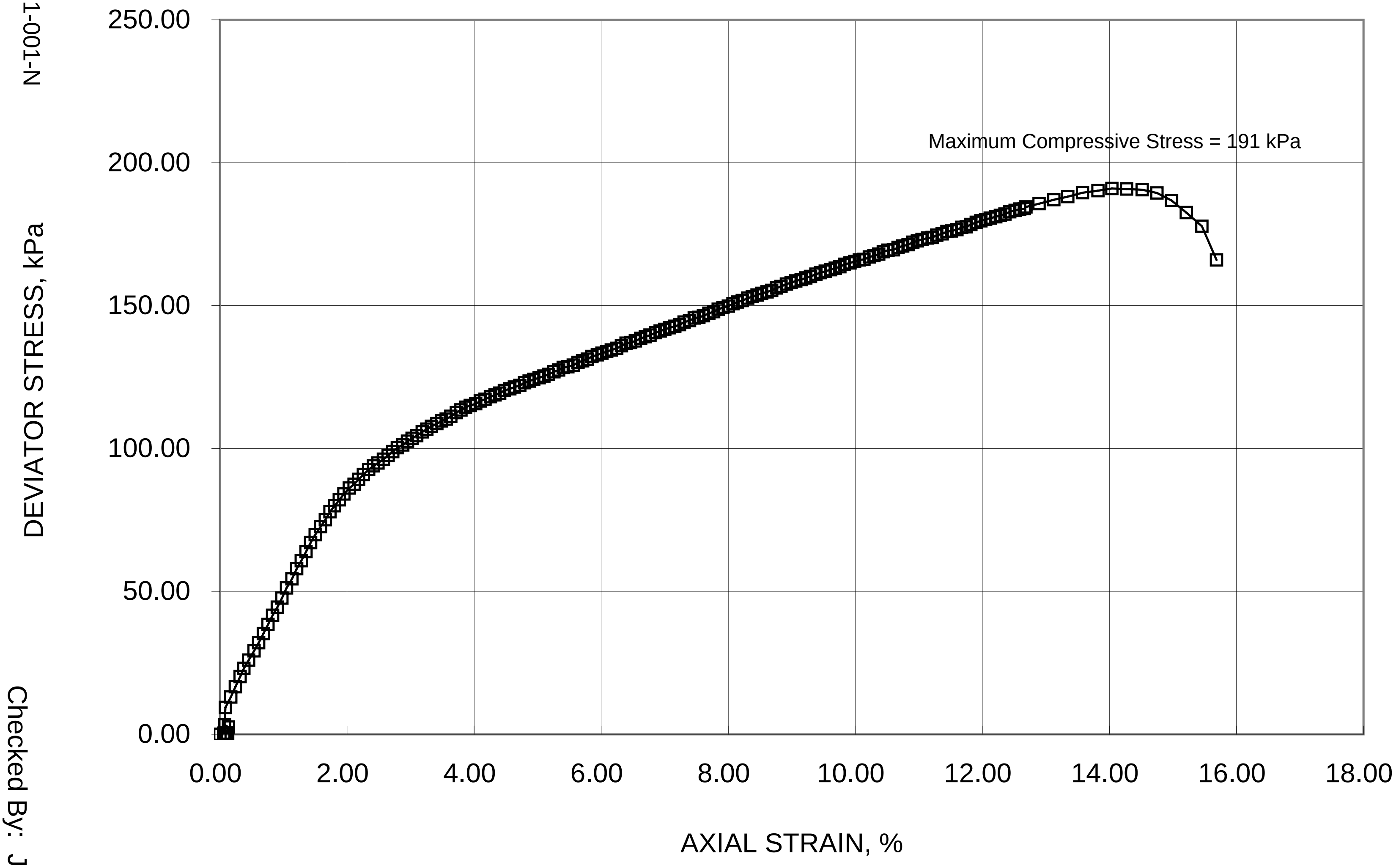
FIG No. A-3

Project No. 06-1191-001-N

Checked By: JMAC

Project No. 06-1191-001-N

BOREHOLE NUMBER 06-9 SAMPLE NUMBER 8a SAMPLE DEPTH, m 9.14-9.75



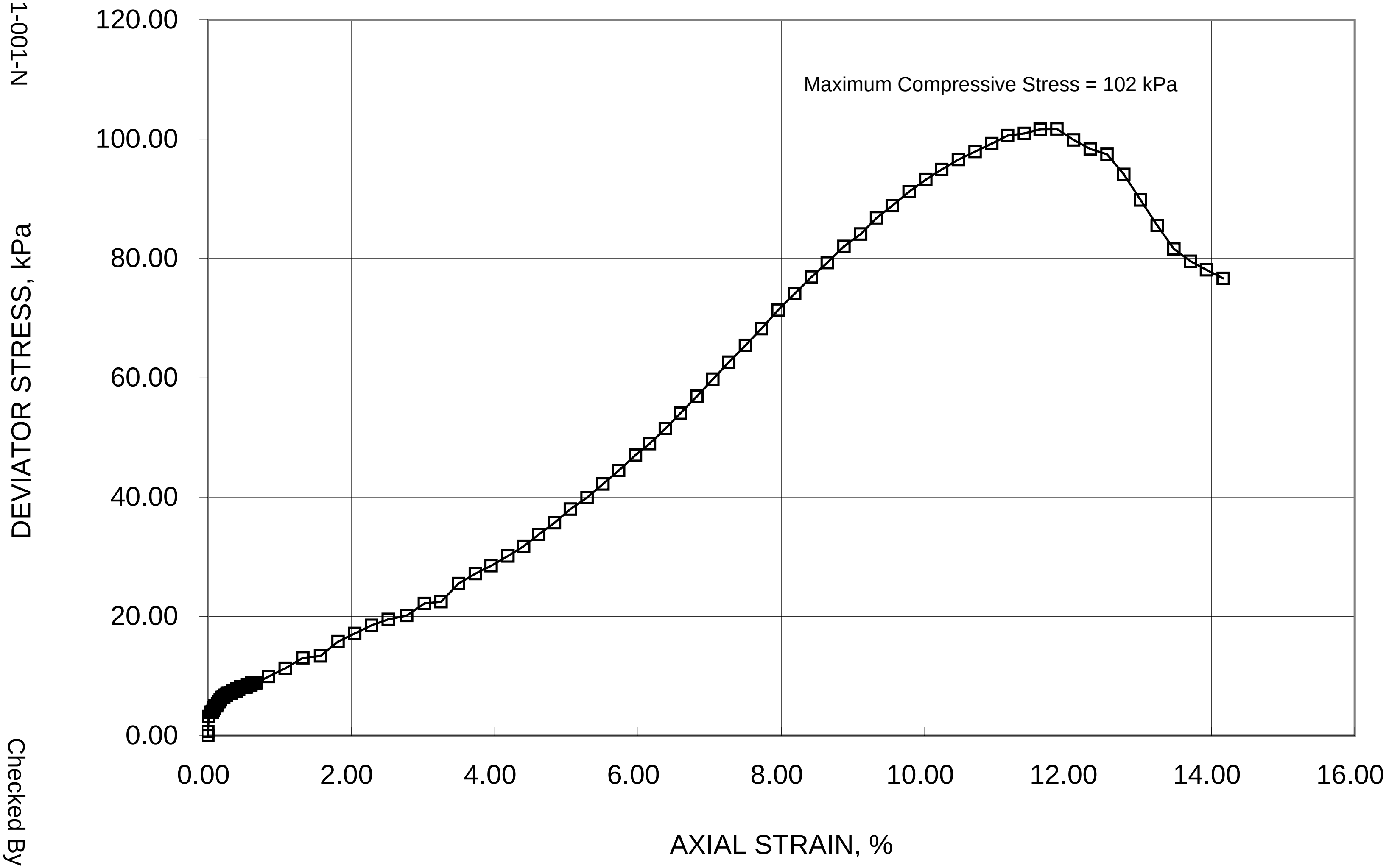
Checked By: JMAC

UNCONFINED COMPRESSION TEST (UC)

FIGURE A-4

Project No. 06-1191-001-N

BOREHOLE NUMBER 06-9 SAMPLE NUMBER 8b SAMPLE DEPTH, m 9.14-9.75



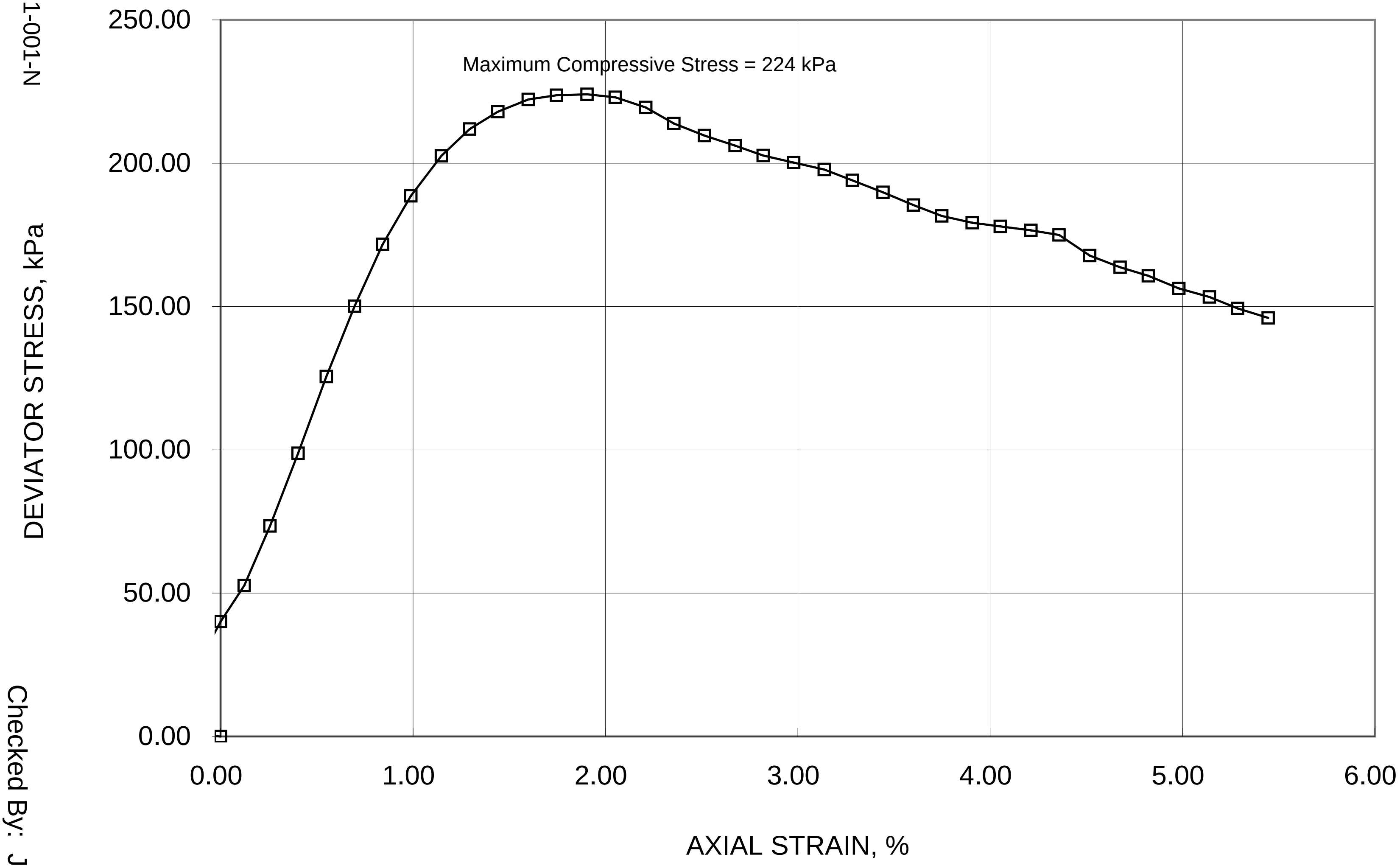
Checked By: JMAC

UNCONFINED COMPRESSION TEST (UC)

FIGURE A-5

Project No. 06-1191-001-N

BOREHOLE NUMBER 06-10 SAMPLE NUMBER 9 SAMPLE DEPTH, m 9.14-9.75



Checked By: JMAC

UNCONFINED COMPRESSION TEST (UC)

FIGURE A-6

OEDOMETER CONSOLIDATION SUMMARY

FIGURE A-7
Page 1 of 4

SAMPLE IDENTIFICATION

Project Number	06-1191-001	Sample Number	9
Borehole Number	06-10	Sample Depth, m	9.1-9.8

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	08/08/2006		
Date Completed	08/19/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m^3	18.51
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	14.04
Area, cm^2	31.65	Specific Gravity, measured	2.71
Volume, cm^3	60.45	Solids Height, cm	1.009
Water Content, %	31.81	Volume of Solids, cm ^{3}	31.93
Wet Mass, g	114.07	Volume of Voids, cm ^{3}	28.52
Dry Mass, g	86.54	Degree of Saturation, %	96.5

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.910	0.893	1.910				
4.70	1.909	0.892	1.910	2	3.86E-01	1.11E-04	4.22E-06
9.54	1.906	0.889	1.908	1	7.71E-01	3.25E-04	2.45E-05
19.26	1.903	0.886	1.905	2	3.84E-01	1.62E-04	6.09E-06
38.71	1.896	0.879	1.900	2	3.82E-01	1.88E-04	7.06E-06
77.44	1.885	0.868	1.891	8	9.47E-02	1.49E-04	1.38E-06
154.67	1.871	0.854	1.878	17	4.40E-02	9.49E-05	4.09E-07
309.36	1.848	0.832	1.860	13	5.64E-02	7.78E-05	4.30E-07
618.73	1.802	0.786	1.825	15	4.71E-02	7.78E-05	3.59E-07
1237.33	1.746	0.730	1.774	23	2.90E-02	4.74E-05	1.35E-07
2479.27	1.690	0.675	1.718	17	3.68E-02	2.36E-05	8.52E-08
1237.33	1.695	0.680	1.693				
309.36	1.709	0.694	1.702				
77.44	1.726	0.711	1.718				
19.26	1.743	0.727	1.735				
4.70	1.764	0.748	1.754				

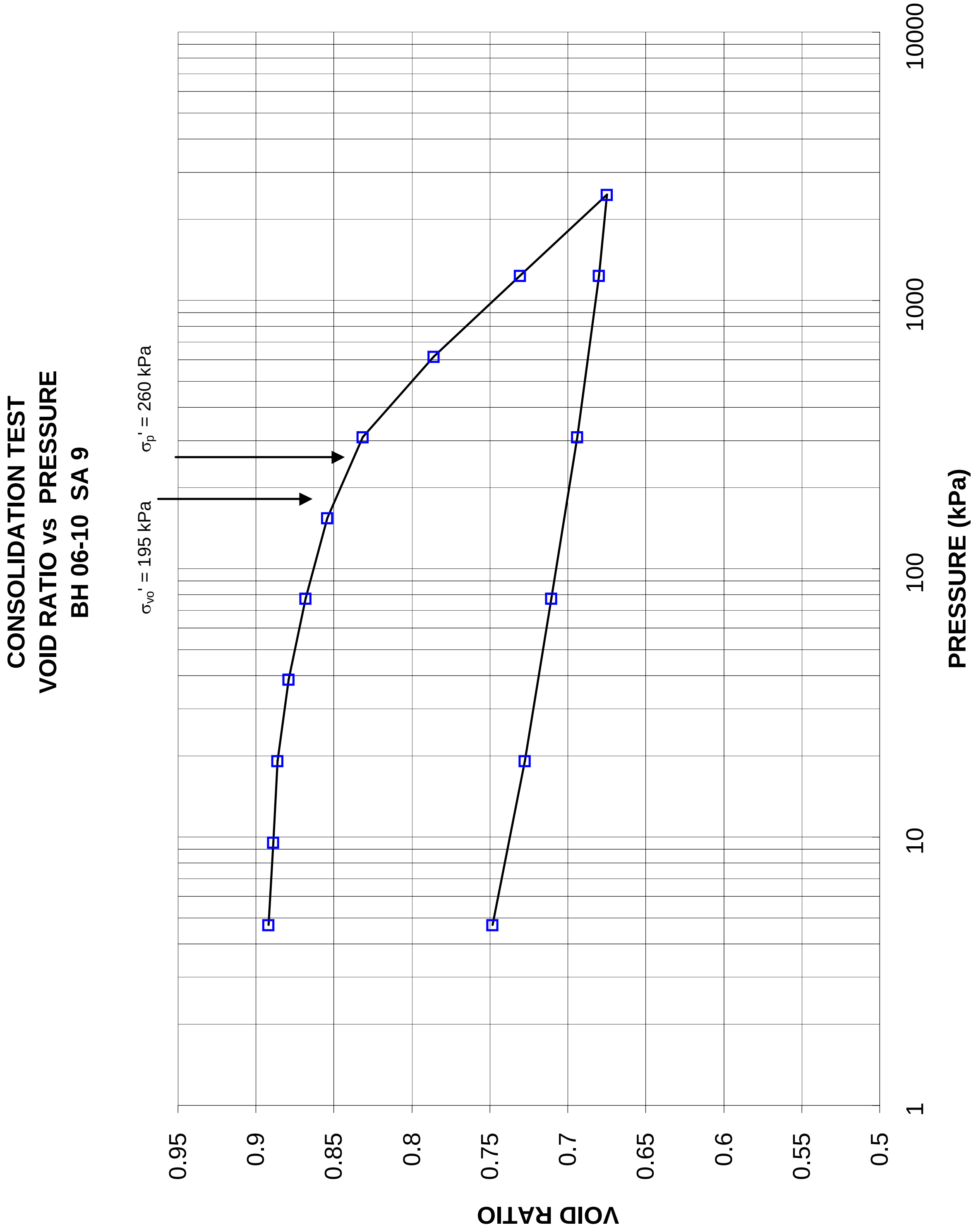
Note:
k calculated using cv based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.76	Unit Weight, kN/m^3	19.56
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	15.20
Area, cm^2	31.65	Specific Gravity, measured	2.71
Volume, cm^3	55.83	Solids Height, cm	1.009
Water Content, %	28.70	Volume of Solids, cm ^{3}	31.93
Wet Mass, g	111.38	Volume of Voids, cm ^{3}	23.90
Dry Mass, g	86.54		

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A-7
Page 2 of 4



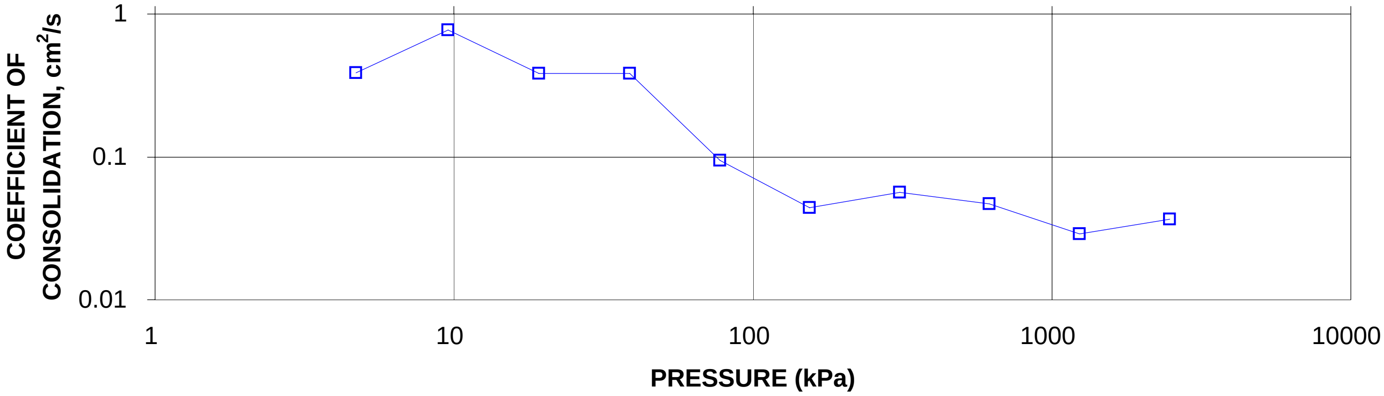
Project No. 06-1191-001

Prepared By: LFG

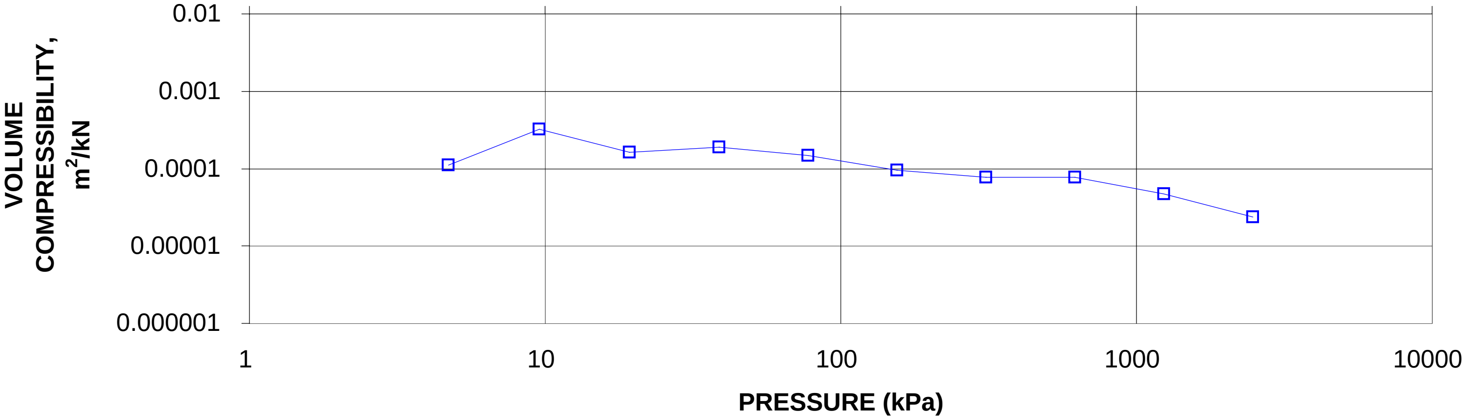
Golder Associates

Checked By: MM/JMAC

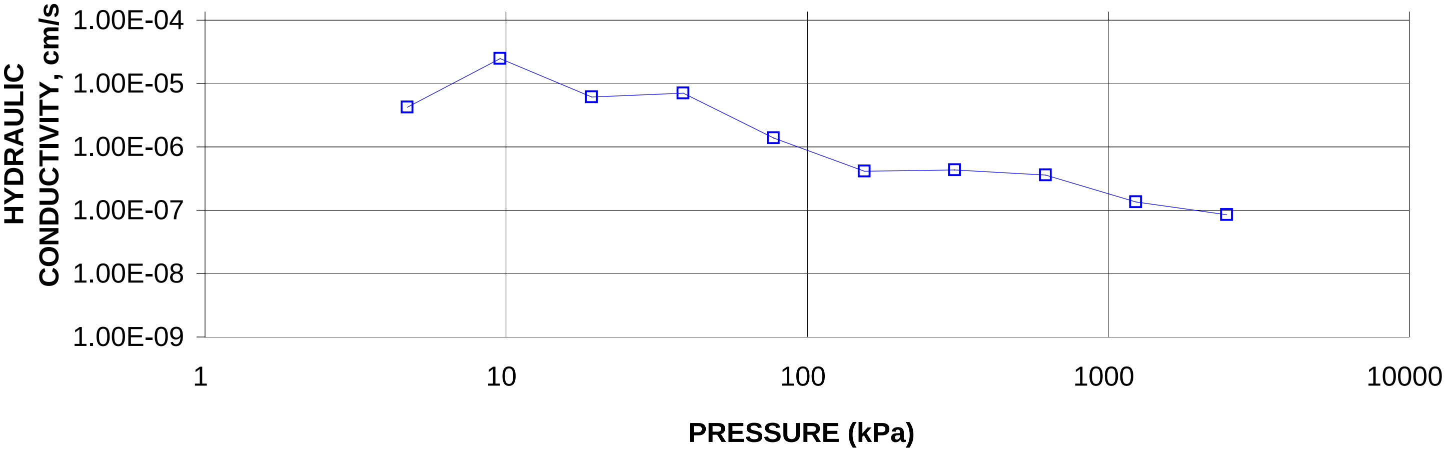
CONSOLIDATION TEST
CV cm^2/s VS PRESSURE (kPa)
BH 06-10 SA 9



CONSOLIDATION TEST
MV m^2/kN vs PRESSURE (kPa)
BH 06-10 SA 9

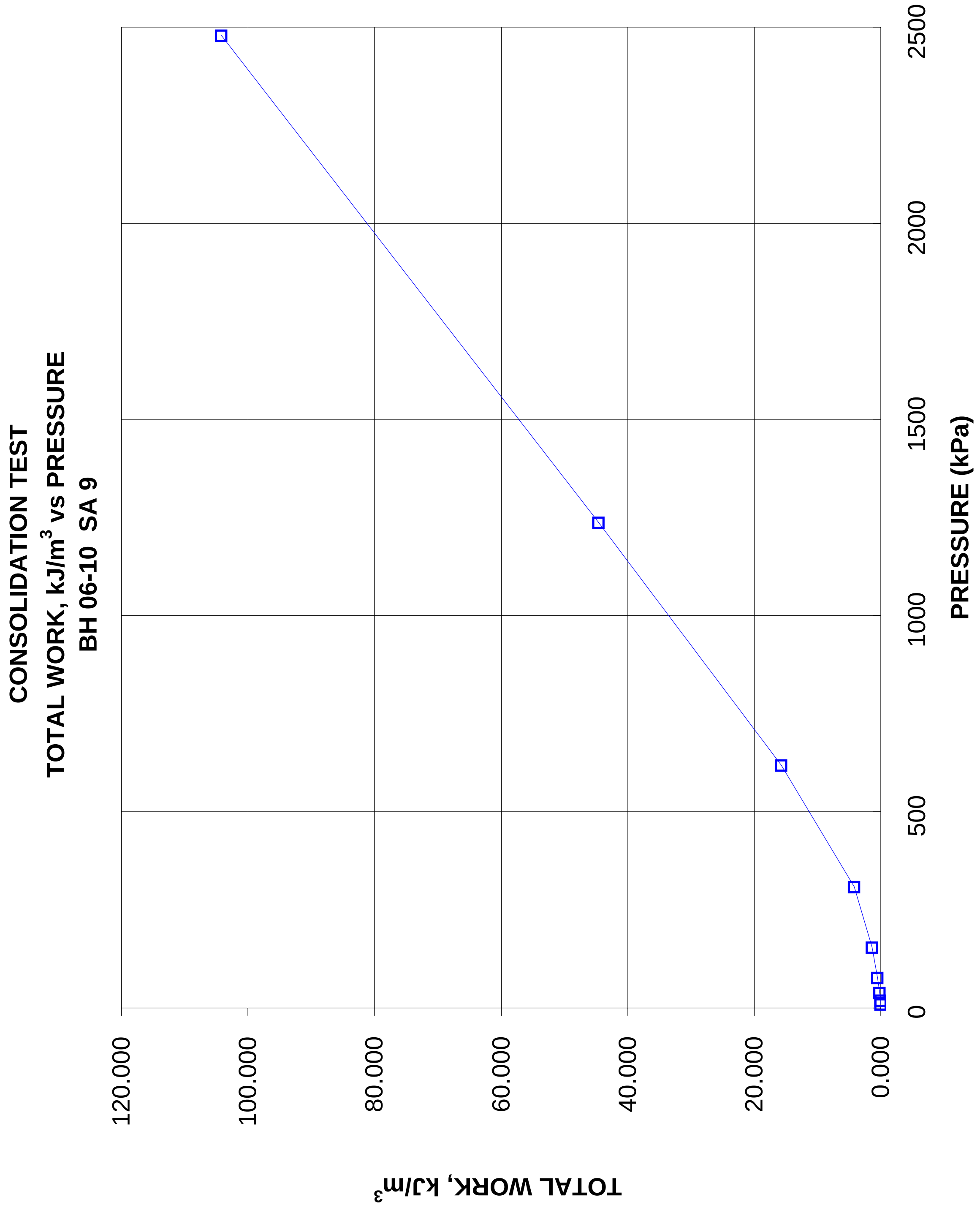


CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 06-10 SA 9



CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE

FIGURE A-7
Page 4 of 4



Project No. 06-1191-001

Prepared By: LFG

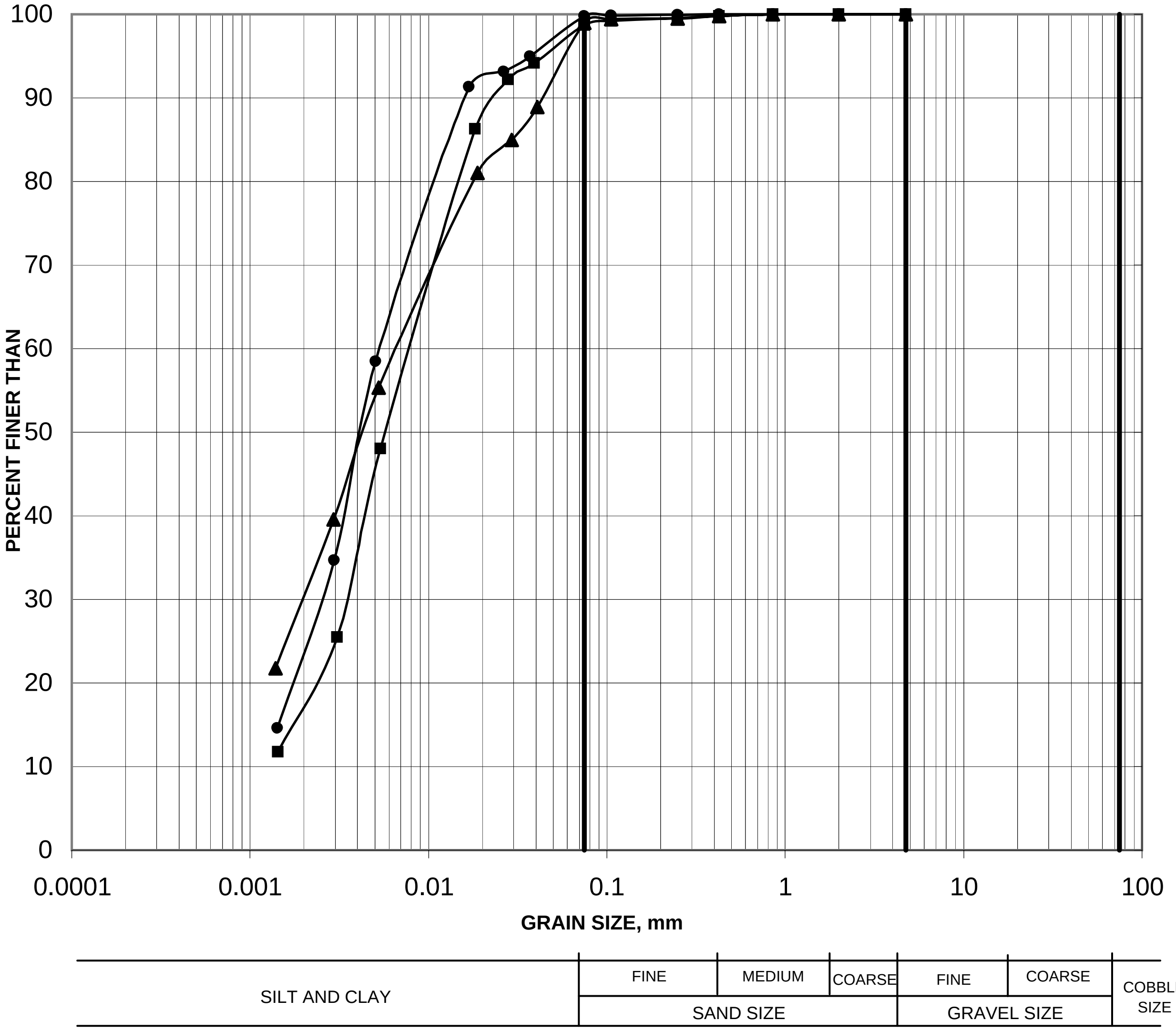
Golder Associates

Checked By: MM/JMAC

GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE A-8



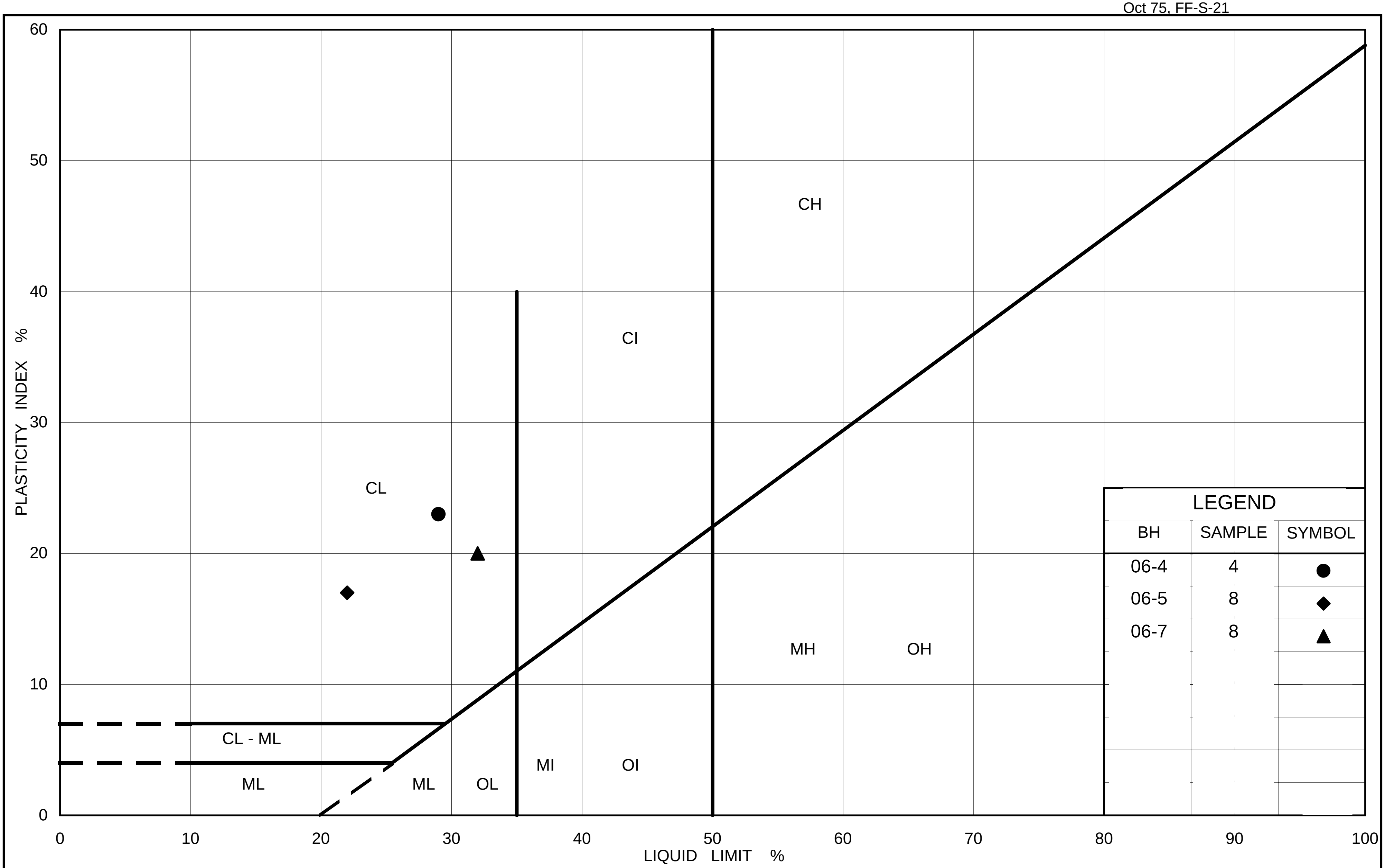
Borehole	Sample	Elevation (m)
—■— 06-4	3	277.9
—●— 06-5	6	274.6
—▲— 06-7	6	278.5

Date: December 2006
Project: 06-1191-001-N

Golder Associates

Created by: SL
Checked by: JMAC

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
06-4	4	●
06-5	8	◆
06-7	8	▲

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PLASTICITY CHART
Clayey Silt

FIG No. A-9

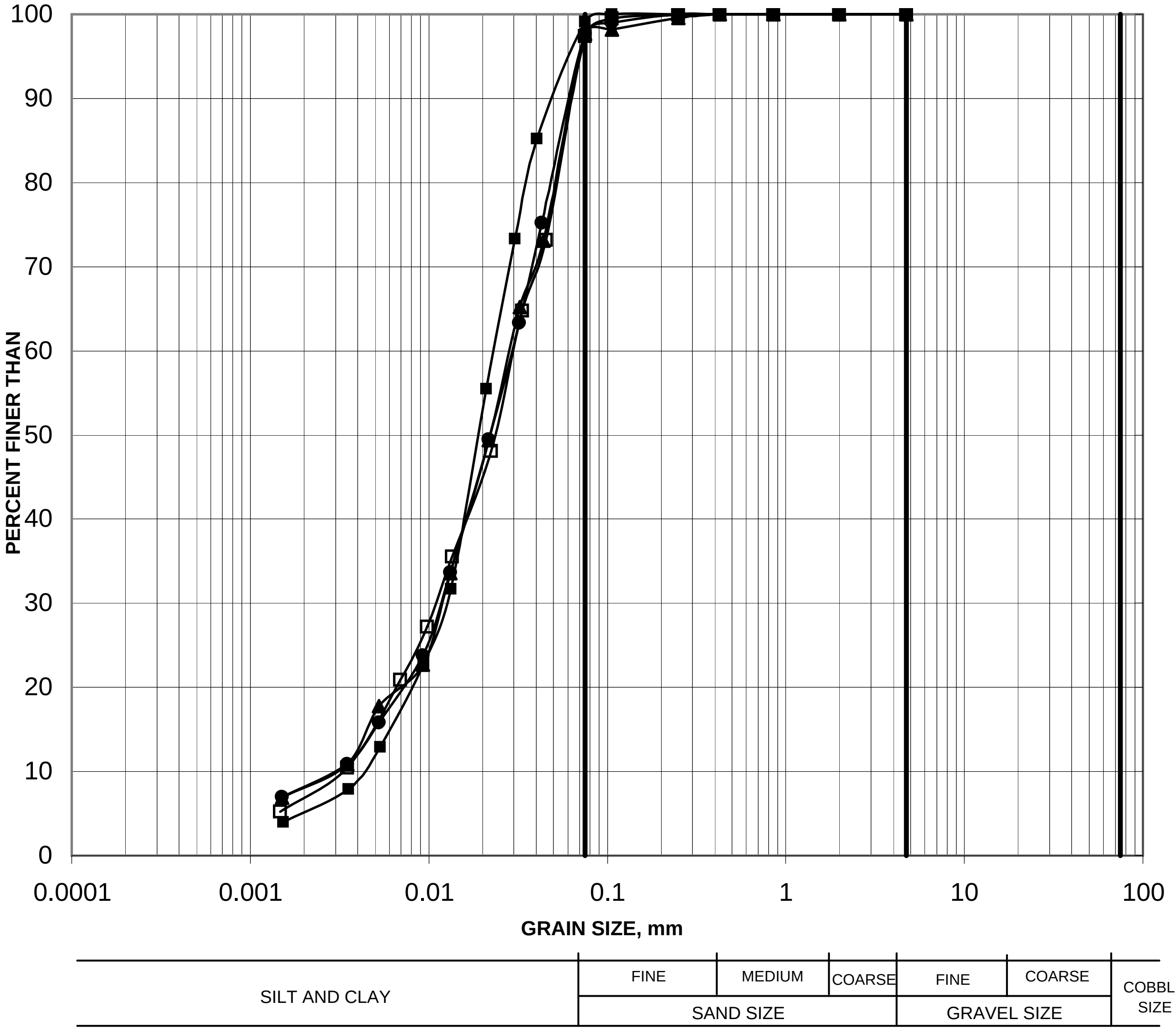
Project No. 06-1191-001-N

Checked By: J MAC

GRAIN SIZE DISTRIBUTION

Silt

FIGURE A-10



Borehole	Sample	Elevation (m)
■ 06-1	9	285.9
□ 06-6	10	268.5
▲ 06-9	9	278.4
● 06-10	11	280.1

Date: December 2006
Project: 06-1191-001

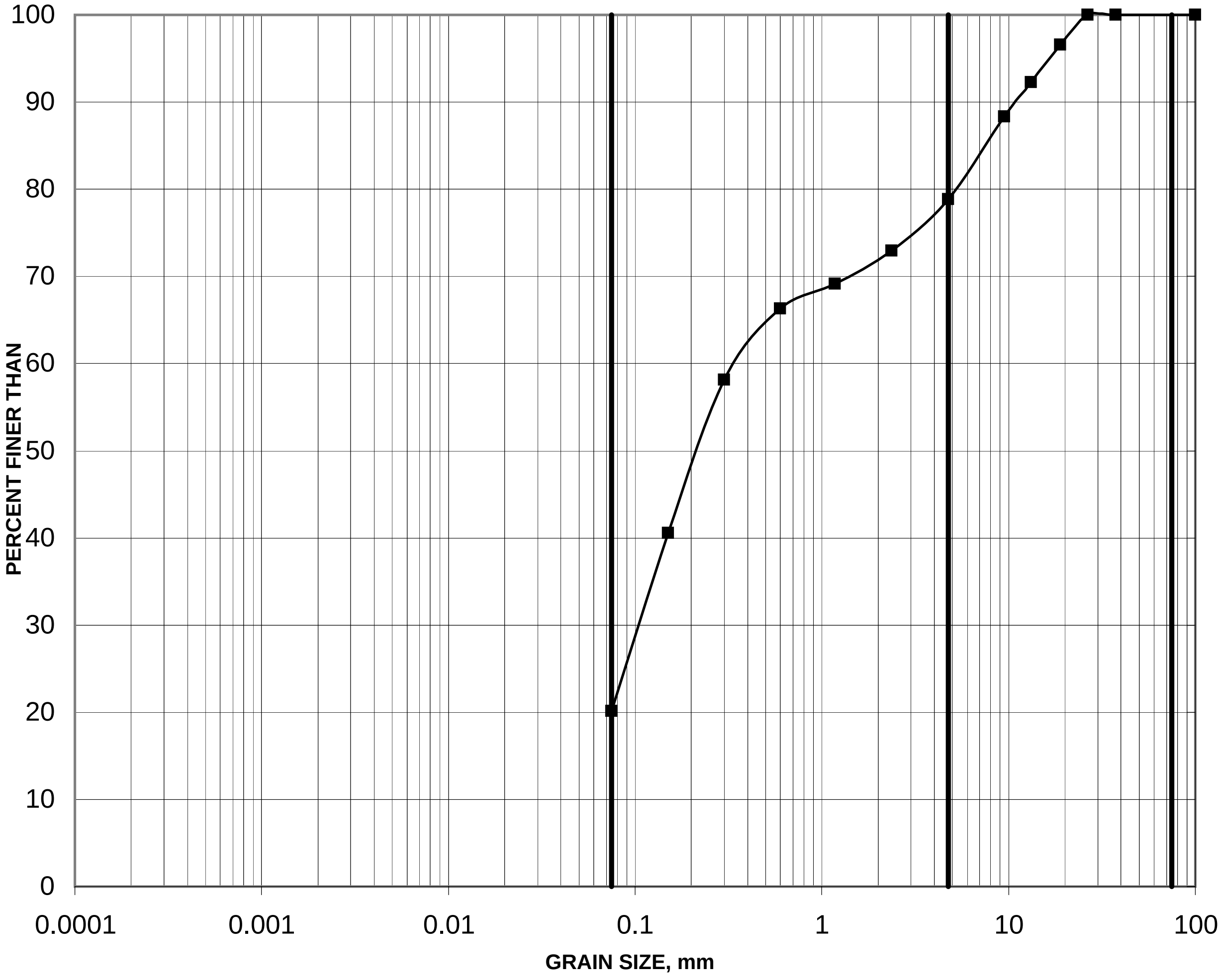
Golder Associates

Created by: SL
Checked by: JMAC

GRAIN SIZE DISTRIBUTION

Gravelly Sand

FIGURE A-11



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

Borehole	Sample	Elevation (m)
—■— 06-3	9	273.1

Date: December 2006

Project: 06-1191-001-N

Golder Associates

Created by: SL

Checked by: JMAC

APPENDIX B

OPERATIONAL CONSTRAINTS (OCs)
NON-STANDARD SPECIAL PROVISIONS (NSSPs)

OPERATIONAL CONSTRAINT

Removal of Alluvium, North Approach

This operational constraint outlines the procedure to be used for excavation of the clayey silt (alluvium) deposit at the toe of the slope at the north approach.

The depth, limits and slopes of the sub-excavation are shown on the contract drawings. The side slopes shall be in accordance with OPSD 203.02.

Work shall be carried out starting from the west limit of the sub-excavation area and proceeding towards the east.

Removal of the alluvium shall be carried out in short sections perpendicular to the slope with the base of the excavation/trench not wider than 3 m at any time and the excavation backfilled with Granular B Type II progressively as the excavation is made.

Temporary excavation side slopes through the organics/clay shall be no steeper than 1H:1V below the water level.

Basis of Payment

Payment for the Contractor to remove the clayey silt alluvium and replace with granular fill, including all equipment, labour and materials shall be deemed to be included in the contract bid price for the various tender items.

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 Northbound 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.

EXCAVATION – Item No.
UNWATERING – Item No.

Non-Standard Special Provision

Scope

The Contractor shall carry out the excavation and construct the widened piers of the Northbound Lane structure such that disturbance to the existing pile caps and piles does not affect the integrity and performance of the existing structure.

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.

<u>UNWATERING FOR STRUCTURE EXCAVATION</u> - Item No.

Non-Standard Special Provision

Scope

The Contractor shall be alerted that the soils at the Vernon Narrows Northbound Lane structure site consist of very soft to firm clayey silt alluvium and very soft to very stiff clayey silt below the water table. Pile caps construction below the groundwater and/or lake water levels must be carried out in the dry. The excavation shall be kept stable during the work.

Basis of Payment

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

VIBRATION MONITORING - Item No.

Non-Standard Special Provision

1.0 GENERAL

1.1 Scope

This special provision describes requirements for vibration monitoring of the existing abutments of the Vernon Narrows Northbound Lane structure during pile driving for the widening of foundation elements.

2.0 REFERENCES

The subsurface conditions at the site are described in the following Foundation Investigation Report for G.W.P 5189-05-00:

Foundation Investigation Report, Vernon Lake Narrows, Rehabilitation and Widening of Northbound Structure, Highway 11, G.W.P. 5189-05-00, Site 42-018, Ministry of Transportation, Ontario, Huntsville, Ontario, dated October 2006.

3.0 DEFINITIONS

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the contract. The Quality Verification Engineer shall be retained by the Contractor to ensure general conformance with the contract documents and issue certificate(s) of conformance.

4.0 SUBMISSION REQUIREMENTS

The Contractor shall submit three (3) copies of the vibration monitoring plan to the Contract Administrator at least 3 weeks prior to the piling operations. The vibration monitoring shall satisfy the specifications and at a minimum contain the following specific information:

Name of Firm/QVE responsible for monitoring including qualifications of vibrations monitoring specialist;
Proposed instrumentation;
Proposed location of instruments on existing structure;
Proposed frequency of readings; and
Proposed methods for adjusting piling methods if readings show excess vibrations.

5.0 PROCEDURES

5.1 Locations of Vibration Monitoring Equipment

The vibration monitoring equipment shall be placed directly on the concrete foundations of the existing bridge abutments as close as possible to the pile driving operations.

6.0 MONITORING

6.1 Vibration Limits

The vibrations on the existing footing shall not exceed 50 mm/s (peak particle velocity).

6.2 Frequency of Readings

6.2.1 The Contractor shall take readings on the first pile in each pile group (i.e. at each corner of the abutment), starting with the pile furthest away from the existing structure. As a minimum, the readings should be taken and recorded during the first 3 m of driving and continuously during driving through the bouldery deposits and during seating of the pile onto the bedrock.

6.3 Submission of Results

6.3.1 The results shall be submitted to the Contract Administrator prior to continuing with the remaining piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results. Additional submissions may be required at the discretion of the Contract Administrator. The results shall be immediately reviewed by the QVE and submitted to the Contract Administrator prior to the Contractor continuing with the remaining piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with the vibration monitoring results.

6.3.2 If the results are acceptable, the Contractor may continue installing the remaining piles with vibration monitoring readings being taken during driving of each pile during bedrock seating. The results of subsequent piles should be submitted to the Contract Administrator at the end of each day.

6.3.3 If the readings are not within the limits stated above, the Contractor must alter his driving procedures until the vibrations on the existing structure are within acceptable levels. The above process must be repeated for each pile.

7.0 CERTIFICATE OF CONFORMANCE (COC)

Upon completion of the work in each area of pile driving, the Contractor shall submit to the Contract Administrator a CoC sealed and signed by the QVE. The certificate shall state that the vibrations on the existing structure were below the limits stated above, and where the levels were exceeded, what procedures were used to reduce the vibrations to below the limits stated above.

8.0 BASIS OF PAYMENT

Payment at the contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

PILES - Item No.
CAISSONS – Item No.
EXCAVATION – 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 Northbound Lane structure for the widened foundation elements, the Contactor shall be alerted that potential obstructions may be encountered in the surficial overburden soils in the existing abutment areas consist of silty sand fill; and below the clayey silt and silty deposits there is a stratum of gravelly sand containing cobbles and boulders. 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.