Submitted To AECOM Canada Ltd.
189 Wyld Street Suite 103, North Bay, Ontario P1B 1Z2
On Behalf of the Ontario Ministry of Transportation

Highway 535 Rehabilitation
Approach Fill Settlement – Nepewassi River Bridge
Site No. 46-130, TWP. of Dunnet
GWP 5573-04-00

Highway 535
From 20.2 km north of Highway 17;
Including Highway 17/Highway 535(Hagar) Intersection;
and OVR to 0.01 km north of Roy Lumber Co.
District of Sudbury

FINAL FOUNDATION INVESTIGATION AND DESIGN REPORT

Date: December, 2011
Ref. Nº: 11/04/11046-F4 R2

Geocres No. 41I-278

LVM | MERLEX

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1.0 INTRODUCTION

LVM | MERLEX has been retained by AECOM Canada Ltd., on behalf of the Ministry of Transportation of Ontario (MTO), to carry out a foundation investigation to provide information for the rectification of ongoing approach fill settlement at the Nepewassi River Bridge (Site No. 46-130) as well as for the design of a roadway protection system. The Nepewassi River Bridge is located on Highway 535, some 3.5 km south of Hwy 17, in the Township of Dunnet.

The foundation investigation location was specified by the MTO in the RFP/TPM documentation Agreement No. 5010-E-0015. The terms of reference for the scope of work are outlined in LVM | MERLEX's proposal P-10-169, dated December 2010. The purpose of this investigation was to determine the subsurface conditions in the area of the bridge approaches. LVM | MERLEX investigated the foundation areas by the drilling of boreholes, carrying out in-situ tests, and performing laboratory testing on select samples.

2.0 SITE DESCRIPTION

The foundation investigation at the bridge approaches is located between Stations 18+843 and 18+928, Township of Dunnet (Site No. 46-130). The topography at the site is a low wide valley in which the Nepewassi River meanders in a west to east direction. The river valley is some 300 m in width, at the bridge site, with outcropping bedrock defining the south and north valley walls. The low flood plain area, adjacent to the river, supports bulrushes/grasses and low shrub vegetation. As the grade rises up the valley walls, to the north and south, mature deciduous and coniferous trees are present. Developed lands, to the north and south of the bridge site, support farming activities. The approaches, for the 1998 detour bridge, are still partially present to the west of the alignment. The existing highway embankment, at the two lane bridge structure, currently supports two undivided lanes of highway, running in a north south direction. The road centerline elevation ranges from 207.8 m, at the south abutment, to 208.9 m at the north abutment. The existing bridge, at this location, is a three span steel plate girder bridge with concrete deck, supported on steel H piles driven to practical refusal. The river water level at the time of investigation was at elevation 202.5 m

The existing bridge is the third structure that has been built on this horizontal alignment. It is understood that the original structure, prior to 1951, was a timber structure on timber pile bents. A second, 11 span, timber trestle type structure on new timber pile bents was constructed in 1951. The existing 3 span bridge was constructed in 1999 under Contract No. 98-215. During design of the present bridge (in 1995) the MTO had reported that the timber trestle structure was in poor condition and that it had undergone considerable settlement at the abutments. The timber structure was described on Sheet 45 of Contract Drawing No.83-227 as having a 52.5 m span.

The vertical alignment, at the existing structure, was adjusted upward from 1.0 to 1.8 m at the south and north abutments respectively under the 1998 contract. It is unknown if the vertical alignment was altered at the time of the 1951 construction.

2.1 Site Physiography and Surficial Geology

This project is located in the Geomorphic Sub-province known as the North Shore - Sudbury Ridges and Pockets. The topography on this section of Highway 535 is generally rolling. There are frequent exposed bedrock ridges. At many locations, significant layers of earth overlay the bedrock. Organic terrain was also observed. Within the project area overburden consists primarily of silty clay, overlying silts and sands. The grade drops some 25 m into the valley where the Nepawassi Bridge is located.

Bedrock in the area, as indicated on OGS Map 2506, is of the Late to Middle Precambrian Era. At the location of this foundation investigation, the bedrock comprises of biotite gneiss.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between June 14^{th} and June 21^{st}, 2011, during which time four (4) sampled boreholes were advanced in the area of the approach slabs at the Nepewassi Bridge. A deep borehole and dynamic cone penetration test was advanced through the existing approach slabs, a short distance back from the rear of both the south and north abutments (Borehole Nos. LVM-2 and LVM-3, respectively). In addition two borings were advanced a short distance beyond the ends of both the south and north approach slabs (Borehole Nos. LVM-1 and LVM-4, respectively).

The field investigation was carried out using a CME drilling rig equipped with hollow stem augers, standard augers, N size coring and casing, and all routine geotechnical sampling equipment. Soil samples were obtained at the borehole locations at regular intervals of depth using the standard 50 mm O.D. split spoon sampler advanced in accordance with the Standard Penetration Test (SPT) procedures (ASTM D-1586). The SPT method involves advancing a 50 mm O.D. split spoon sampler with the force of a 63.5 kg hammer freely dropping 760 mm mounted in a trip (automatic) hammer. The number of blows per 300 mm penetration was recorded as the “N” value. At the boreholes, a Dynamic Cone Penetration Test (DCPT) was carried out to give a continuous plot of the soil resistance with depth. When cohesive deposits were encountered, the in-situ strength was measured using an “N” size field vane, vane collar, and calibrated torque meter. Relatively undisturbed samples of the cohesive deposits were retrieved using a 73 mm O.D. thin walled sampler in accordance with ASTM D-1587. All samples taken during this investigation were stored in labeled airtight containers for transport to our North Bay laboratory for visual examination and select laboratory testing.

Groundwater conditions in the open boreholes were observed during the advancement of and immediately following completion of the individual boreholes. Peizometers were installed in Borehole No. LVM-3 to monitor groundwater conditions, in the lower clay stratum and upper fills. All open boreholes were backfilled upon completion with compacted auger cuttings and bentonite backfill. At Borehole No. LVM-3 bentonite combined with filter sand packs and bentonite pellets were employed to filter and isolate the piezometers. At the boreholes through the embankment, the upper portion of the hole, where necessary, was backfilled with an asphalt cold patch to seal the existing asphalt surface. The field work for this investigation was under the full time direction of a senior member of our engineering staff, who was responsible for locating the boreholes, clearing the borehole locations of underground services, in-situ sampling and testing operations, logging of the boreholes, labeling and preparation of samples for transport to our North Bay laboratory, plus overall drill supervision. All samples received a visual confirmatory inspection in our laboratory. Laboratory testing of select samples included routine testing for natural moisture content determination and particle size analysis as well as Atterberg Limits testing. Detailed consolidation testing on four samples was carried out at a MTO approved high capacity soil testing laboratory (Golder Associates, Mississauga). The results of the laboratory testing are presented on the individual Record of Borehole Sheets (Appendix B), with a summary of results presented on the laboratory sheets in Appendix C (Figures Nos. L-1 to L-9).

The location of the individual boreholes were determined in the field using highway chainage (established by others) and offset relative to highway centerline. The MTO co-ordinates, northing and easting, were then established for the boring locations. Elevations contained in this report are referenced to a geodetic datum.

4.0 SUBSURFACE CONDITIONS

Details of the subsurface conditions revealed by the investigation program are presented on the enclosed Record of Borehole Logs (Appendix B) and on Figure No. 2 (Appendix C). Please note that stratigraphic delineation presented on the borehole logs and soil strata plot are the results of non-continuous sampling, response to drilling progress, the results of SPT and Dynamic Cone Penetration Test (DCPT) plus field observations. Typically such boundaries represent transitions from one zone to another and are not an exact demarcation of specific geological unit. Additional consideration should be given to the fact that subsurface conditions may vary markedly between adjacent boreholes and beyond any specific boring location, and are shown on the drawings for illustration purposes only.

4.1 Background Data

Following award of this project LVM | MERLEX was supplied with a copy of a 1995 Foundation Investigation and Design Report carried out for construction of the existing bridge, under Contract No. 98-215. This earlier investigation, by Shaheen and Peaker Limited (S&P), revealed an extensive deposit of soft clay up to 20 m in depth, (south abutment), underlain by cohesionless silts and fine sands to sands and gravels overlying bedrock, which was encountered at depths varying from 45 to 27 m, (south and north abutment respectively). Groundwater, in the lower aquifer, was under an artesian condition, which indicated an elevated hydraulic gradient of 1 to 2 m above the water level in the river. This factual data consisted of six (6) sampled boreholes advanced by Shaheen and Peaker in 1995 and numbered D-1 to D-6 inclusive. In addition, the 1995 report contained the borehole logs of a single borehole advanced by the MTO in 1992 and two boreholes (Borehole Nos. 1A and 2A) advanced by Morton and Partners Ltd in 1981. For completeness the location of these previous borings are shown on the LVM | MERLEX Borehole Location Plan (in grey scale) and copies of the Borehole logs, and factual laboratory data have been included in Appendix D.

4.2 Nepewassi River Bridge – Site No. 46-130

A plan and profile showing the borehole locations and stratigraphic sequences is shown on Figure No. 2, Appendix C. During the course of this exploration program, four (4) sampled boreholes were put down at this site, with Borehole Nos. LVM-1 and LVM-4 advanced through the existing embankment to the south and north of the 10 m long approach slabs, respectively, for the purpose of obtaining information for possible design of a roadway protection system. Borehole Nos. LVM-2 and LVM-3 were advanced through the approach slab at the south and north abutments, respectively, to address the ongoing settlement of the approach embankments. At the time of the subsurface investigation, the ground surface elevations at Boreholes Nos. LVM-1 to LVM-4 inclusive were recorded at elevations 207.7, 207.8, 208.9, and 209.1 m, respectively.

This study was undertaken to define the characteristics of the deep clay stratum and, as such, the depth of borehole was limited to 25 m. For completeness the stratigraphic column has been supplemented with previous data which is shown on the stratigraphic profile and plan in grey scale.

The following is a description of the subsurface conditions encountered in the boreholes advanced by LVM | MERLEX during the 2011 investigation.

4.2.1 Pavement Structure

At the boreholes advanced beyond the approach slab a surficial pavement structure consisting of 200 and 275 mm of asphalt and 175 and 250 mm of crushed gravel was encountered at Borehole Nos. LVM-1 and LVM-4, respectively. At the two borings, advanced through the approach slab a short distance behind the rear of the abutments, a surficial pavement structure consisting of 240 and 200 mm asphalt and 280 and 350 mm concrete approach slab was encountered at Borehole Nos. LVM - 2 and LVM-3, respectively. At these two boreholes a void of 125 to 130 mm was encountered below the approach slab

4.2.2 Embankment - Lightweight Fill

At each borehole, underlying the above described surficial pavement structures, a deposit of granular fill was penetrated. The original contract drawings (Contract No. 98-215) indicate that lightweight fill was to be used to re-construct the upper part of the approach embankments and identifies this material as 3/8 structural coarse lightweight blast furnace slag (LBFS) fill. The encountered deposit consisted of brown to reddish brown medium to fine sand size particles trace silt size particles. The natural moisture content measured from samples of the LBFS fill was in the order of 5 to 20%. Gradation analyses were carried out on twelve (12) samples of this deposit, the results of which indicated 0 to 2% gravel size particles, 93 to 97% sand size particles, and 2 to 5% silt and clay size particles (Figure Nos. L-1a and L-1b, Appendix C). Based on mass/volume measurements taken on retrieved samples of the lightweight fill deposit, the unit weight of the in place fill was estimated to be on average 15.5 kN/m^3. At Station 18+843, a bulk sample of the lightweight fill material was retrieved to allow a Standard Proctor Dry Density (SPDD) Test to be undertaken. This testing, as per ASTM D-698 returned a SPDD value of 1580 kg/m^3 at an optimum moisture content of 20.5%. Gradation analysis was carried out on the sample used for the SPDD test, before and after testing. This data is shown on

Figure No. L-2, Appendix C. Based on SPT ‘N’ values of 16 to 81 (average 39) blows per 300 mm penetration, the compactness of this deposit was described as compact to very dense, generally dense. This deposit was encountered to depths of 3.7 and 4.3 m below ground surface at Borehole Nos. LVM-2 and LVM-3, respectively (elevations 204.1 and 204.6 m, respectively). A geotextile was encountered at a 4.3 m depth at Borehole No. LVM-3 at the transition from the lightweight fill to the original embankment fill materials.

Initially auger refusal was encountered at the surface of the lower fill deposit, at depths of 3.3 and 4.5 m below grade at Borehole Nos. LVM-1 and LVM-4, respectively (elevations 204.4 and 204.6 m, respectively). At Borehole No. LVM-1 two additional attempts were made, with both standard and hollow stem augers, to penetrate below the refusal depth, however, refusal was met on cobble/boulder size rock obstructions at a 3.1 m depth. These additional borings were within a horizontal distance of 1 m of the original boring. At the location of Borehole No. LVM-4 initial auger refusal was met at a 4.5 m depth, however, a second boring within 1 m of the original, penetrated the cobble/boulder size rock obstruction to allow advance of Borehole No. LVM-4a to a 16.1 m depth (elevation 193.0). DCPT refusal was encountered in this deposit at depths of 3.0, 3.7, 4.3 and 0.2 m below grade at Borehole Nos. LVM-1 to LVM-4, respectively (elevations 204.7, 204.1, 204.6 and 208.9 m, respectively). This refusal was interpreted to be due to obstructions in the underlying original embankment fill material, at approximately the interface between the upper LBFS fill and the original embankment fill, at all locations except at Borehole No. LVM-3 and at the dynamic cone penetration test at Borehole No. LVM-4.

4.2.3 Embankment – Fill

Underlying the LBFS fill, at Borehole Nos. LVM-2 and LVM-4a, a deposit of embankment fill consisting of sands and gravels with frequent cobble and boulder size rock was penetrated. Core drilling using an N size diamond core bits was required in this deposit to penetrate the cobble/boulder size rock obstructions at Borehole No. LVM-2. The natural moisture content measured on the limited samples obtained from the granular portion of this deposit obtained using the SPT method was in the order of 6 to 8%. This deposit was encountered to a depth of 10.6 m below grade (elevation 197.2 m), at Borehole No. LVM-2 and to a 5.3 m depth (elevation 203.8 m) at Borehole No. LVM-4a. Although not sampled (cored) at Borehole No. LVM-1, it is anticipated that auger refusal, which was encountered at a depth of 3.3 m, was due to the presence of cobble/boulder size rock in this embankment fill.

4.2.4 Silty Clay

Underlying the LBFS fill at Borehole No. LVM-3 (north abutment), and underlying the fill at Borehole No. LVM-4a, a deposit of black to dark grey silty clay trace sand trace organics and trace asphalt was penetrated. At Borehole No. LVM-3 this deposit contained a sand and gravel layer, approximately 150 mm thick, at a depth of 5.5 m (elevation 203.5 m). The natural moisture content measured on samples of this deposit was in the order of 4 to 24%. Atterberg Limits testing was carried out on two (2) samples of the cohesive portion of this deposit, the results of which indicated a Liquid Limit in the order of 28 to 32% and a Plastic Limit in the order of 20 to 22%. Based on the results of Atterberg Limits testing, this deposit was classified under USCS as silty clay of low plasticity (CL) (Figure No. L-3, Appendix C). Based on SPT ‘N’ values of 8 to 11 blows per 300 mm penetration the consistency of this deposit was described as stiff to very stiff. This deposit was encountered to depths of 8.8 and 9.1 m below grade at Borehole Nos. LVM-3 and LVM-4a, respectively (elevations 200.1 and 200.0 m, respectively).

4.2.5 Clay

Underlying the original embankment fill, at Borehole No. LVM-2, and underlying the silty clay at Borehole Nos. LVM-3 and LVM-4a, a deposit of grey clay was penetrated. The natural moisture content measured on samples of this deposit was in the order of 38 to 70%. Atterberg Limits testing was carried out on twelve (12) samples of this deposit, the results of which indicated a Liquid Limit in the order of 39% to 66% and a Plastic Limit in the order of 18 to 25%. Based on the results of Atterberg Limits testing, this deposit was classified under USCS as clay of high to medium plasticity (CH to CI) (Figure No. L-4, Appendix C). Based in-situ shear strength testing, returning values of 24 to 60 kPa, the consistency of this deposit was described as firm to stiff (Figure No. L-5, Appendix D). The sensitivity of the clay deposit is defined as the ratio of the in-situ shear strength over that of the remolded shear strength of the material. Based on the results, as obtained from in-situ field vane tests, the sensitivity of this clay deposit ranged from 2 to 6, indicating a low sensitivity. This deposit was encountered to a depth of 21.6 m below grade at Borehole No. LVM-3 (elevation 187.3 m). Sampling was terminated in this deposit at a depth of 25.0 m below grade at Borehole No. LVM-2 (elevation 182.8 m) and 15.5 m depth at Borehole No. LVM-4a (elevation 193.0 m).

Four (4) one-dimensional oedometer (consolidation) tests were carried out on samples of the clay deposit (Borehole Nos. LVM-2 Sample 11 (12.5 m depth), LVM-2 Sample 15 (18.6 m depth), LVM-2 Sample 17 (21.8 m depth), and LVM-3 Sample 16 (18.6 m depth)). The preconsolidation pressure was estimated (using the Casagrande method) to be in the order of 130 to 160 kPa. The over-consolidation ratio, which is the ratio of the preconsolidation pressure to the existing effective overburden pressure, was in the order of 0.81 to 1.23. Based on the results of the oedometer (consolidation) tests, vane shear strength data, and the relationship of the moisture content to liquid limit, this deposit is considered to be underconsolidated to just slightly overconsolidated, relative to the existing overburden pressure. The sample taken from the higher elevation of 195.5 m, has been described as being slightly overconsolidated. Results from the consolidation tests are shown on enclosed Figure Nos. L-6 to L-9 (Appendix C).

4.2.6 Silty Sand

Underlying the clay, at Borehole No. LVM- 3, a deposit of grey silty sand trace gravel trace clay was penetrated. Auger refusal was encountered on a boulder in this deposit at a depth of 21.9 m below grade (elevation 187.0 m).

4.2.7 Compilation of Information From Previous Investigations

Following award of this assignment MTO supplied LVM | MERLEX with a copy of the Foundation Investigation and Design Report, as produced in 1995 by Shaheen and Peaker Limited (S&P), for the construction of the existing bridge, constructed under Contract No. 98-215. This data, as outlined previously, indicated that the extensive clay deposit was underlain by cohesionless deposits of silts and sand over bedrock. The bedrock was encountered at depths varying from 45 to 27 m, (south and north abutment respectively). Groundwater, in the lower aquifer, was under an artesian condition, which indicated a hydraulic gradient of 1 to 2 m higher than river water level. Information from this previous work has been used to supplement the lower portion of the stratigraphic column and has been generally confirmed by LVM | MERLEX where our borings have been advanced at a similar location.

4.3 Groundwater Conditions

It was known from the onset of this investigation, based on historical site data, that an artesian condition was present in the cohesionless deposits underlying the extensive clay stratum. A peizometer was installed in Borehole No. LVM-3 at an elevation of 194.0 m, which was approximately mid height of the clay deposit. The intent of this piezometer was to measure the pore water pressure in the clay stratum. This piezometer returned a stabilized water level reading at elevation 204.1 m, which is some 1.6 m above the river water level measured at elevation 202.5 m, at the time of this investigation. In the 1995 investigation the water level was measured at 202.5 m whereas on Contract Drawing No. 83-227 the river water level was shown at 202.11 m .The river water level directly reflects the groundwater level in the fill deposits along the adjacent riverbanks and approach embankments. The groundwater levels will fluctuate seasonally. The 1995 report indicates that the river water level had increased 0.3 to 0.4 m by the end of the field work.

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5.0 DESIGN COMMENTS AND RECOMMENDATIONS

5.1 General

This foundation investigation has been carried out to provide information for the rectification of ongoing approach fill settlement at the Nepewassi River Bridge on Highway 535. The requirements of the foundation investigation were outlined by the MTO in the RFP/TPM documentation Agreement No. 5010-E-0015 and addressed in LVM | MEL's proposal P-10-169, dated December 2010. The Nepewassi River Bridge is identified as Site No. 46-130, located in the Township of Dunnet on Hwy 535 some 3.5 km south of the intersection with Highway 17.

The existing bridge is a three span steel plate girder structure, supported on end bearing H piles and currently supports two undivided lanes, running in a north south direction. At the time of the investigation the transition from the approach fills to the bridge deck was smooth with no noticeable recent settlement at the approaches. However, it was visually evident that both approach fills had been recently patched. At the time of writing this report, no information was available as to when the most recent or historical patching had been carried out at the approaches.

5.2 Site History

At the Nepewassi River Bridge, the present road centerline elevation ranges from 207.8 m, at the south abutment, to 208.9 m at the north abutment. The existing bridge is a three span steel plate girder bridge with concrete deck, supported on steel end bearing H piles driven to practical refusal. The river flows west to east and the river water level, at the time of investigation, was measured at elevation 202.5 m. The existing bridge is the third structure that has been built on this horizontal alignment.

It is understood that the original structure, prior to 1951, was a timber structure on timber pile bents. A second, 11 span, timber trestle type structure on new timber pile bents was constructed in 1951. The profile drawing in Contract Drawing No. 83-227, surveyed in 1980, indicates the 1951 trestle has a T/O of 207.00 m at Station 18+885, with a river water level at elevation 202.1 m. It is unknown if the vertical alignment was altered at the time of constructing this structure.

The 1951 trestle bridge was supported on 11 timber pile bents. It was noted in the previous investigation that the original timber structure was also supported on timber piles since the tops of the old piles could be seen midway between the 1951 pile bents. The river width (distance between abutments) is 50.8 m wide at the existing 3 span concrete bridge. The drawings for contract No. 83-227 indicated the timber trestle had a 52.5 m clear span. During the design of the present bridge in 1995, the MTO had reported that the 1951 timber trestle “structure is in poor condition and that it has undergone considerable settlement at the abutments”.

The 1995 design of the existing bridge required that the approach grade, at the south and north approaches, be increased in elevation by some 1.0 to 1.8 m, respectively. It was concluded that if the grade raise was carried out with conventional embankment fill (assumed unit weight of 20.5 kN/m^3) a short term slope stability issue would develop and that the increase in load on the underlying silty clay would give rise to large settlements (in the order of 200 to 300 mm).

The results of consolidation testing carried out in 1995, and data quoted by the MTO from their 1992 preliminary alignment study, indicated that the clay stratum was very slightly overconsolidated in three of the tests and more normally consolidated in the lower basal portion of the clay deposit.

It is noted that, in 1999, a slope failure is reported to have occurred during construction of the south temporary detour embankment. It is estimated that the height of the embankment fill was some 5 m when the failure occurred. This confirms the susceptibility of the clay deposit to short term slope instability under load increases.

In order to carry out the proposed grade raise, at the approaches, it was concluded that sub-excavation and replacement with light weight blast furnace slag (LBFS) fill, with a design unit weight of 11.5 kN/m^3, would be required. Employing LBFS fill would accommodate the new grade raises and would satisfy the requirement that no additional load was applied to the underlying clay (i.e. a zero net load increase design). As such, the approaches for the new bridge were designed with sub excavation of 2 m of existing embankment fill and replacement with some 3.0 and 3.8 m of LBFS fill, at the south and north approaches respectively, based on Contract Drawing No. 98-215. Since the completion of the approach grade raise in 1999 ongoing settlement, requiring periodic maintenance, has occurred.

5.3 Discussion

Based on the above historical information, and current data, it is apparent that the approach fill embankments, to the 1951 timber trestle and existing 1999 bridge, have been settling for a period of 60 years. It is also likely that the approaches to the bridge, prior to the 1951 construction of the second trestle type bridge, were also settling. Establishing an estimate of the magnitude of this past settlement, prior to 1999 was not possible since records were not available as to the time of placement of patching or on the thickness of patching required to correct approach fill settlement at the bridge abutments. The available borehole logs from 1981, 1992 and 1995 were reviewed for an indication of settlement based on thickness of patching;

however, only two of the nine original boreholes (Borehole D-1 and D-5) contained information on the hot mix layer thickness which was recorded at 40 mm thick, overlying granular fill.

It is generally accepted, that, when dealing with settlement associated with loading of clay deposits, provided the increase in effective vertical stress is maintained at a level less than the preconsolidation pressure (i.e. loading in the recompression range), the clay soil structure accommodates the increased effective stress without significant settlement. As such settlements are generally within normally accepted tolerable limits. However, if the increase in effective vertical stress exceeds the preconsolidation pressure (i.e. enters the virgin portion of the consolidation curve) the interparticle support structure that has been established begins to rearrange into a more stable configuration of interparticle contact to accommodate the stress increase as the pore water pressure dissipates. This leads to large decreases in the void ratio with large settlements developing.

5.3.1 Stress History

To illustrate the development of the embankment settlement at this site, the stress history (i.e. effective vertical stresses, along with the available preconsolidation pressure data) has been plotted on Figure Nos. 3a and 3b in Appendix E. For discussion purposes, refer to Figure No. 3a (south abutment) where the clay stratum is thickest and the majority of consolidation tests were carried out. The vertical effective stress history, relative to elevation, is identified with the dashed and solid lines and the preconsolidation pressure values from the consolidation testing (estimated using the Casagrande method) are plotted as points.

The “Prior to Highway Construction” line was set at elevation 199.0 m which is the bottom of the existing stream bed. As can be seen this line falls to the left of all the preconsolidation points.

The two points, reproduced from the 1995 consolidation tests are located just to the right of this "prior to construction" line. These samples were taken from Borehole Nos. D-3 and D-4 at elevations 194.5 and 193.0 m respectively and have estimated preconsolidation values of 85 and 60 kPa, respectively. The vertical effective stress at the specific borehole locations, associated with a thin layer of submerged fill (varied between 1.5 to 3 m) and native silty clay, was calculated at 54.9 and 45.7 kPa at Borehole Nos. D-3 and D-4, respectively. These samples showed a slight overconsolidation. It is understood that consolidation testing on samples from the MTO 1992 Preliminary Investigation were not formally reported however values of preconsolidation at "30 to 40 kPa" were quoted.

When the approach fills and bridge were constructed at this site in 1951, which is represented by the solid (red) line, the effective stress increased (shifted to the right) and exceeds the historical and all but one of the preconsolidation data points. As such, large settlements developed and continued to develop, due to major interparticle rearrangement in the natural clay structure as the increased stress levels followed the virgin portion of the consolidation curve. With time, and provided no new additional load is applied, the clay structure will rearrange and eventually carry the entire vertical effective stress load, as the pore water pressure decreases. If the consolidation process were left to continue, under the existing loads, the preconsolidation pressure values would increase to the present day effective vertical stress, at the elevation of sampling, as represented by the dashed (light green) 1999 to 2011 vertical stress line (i.e. the deposit would become normally consolidated). It is noted that the slope of the 1999-2011 effective stress line varies slightly, (relative to the 1951-1999 line) increasing through the depth of the clay deposit. This variation is due to an increase in the average unit weight of the clay from the initial values in the consolidation test results in the 1995 data, where the average unit weight was 15.5 kN/m^3 and that in the 2011 data where the average unit weight was 16.5 kN/m^3 This increase in unit weight is due to ongoing consolidation of the clay deposit.

In order to reduce the magnitude and ongoing nature of future settlement of the embankment fill at the approaches the vertical effective stress would have to be reduced to a level less than the existing preconsolidation stress levels in the clay stratum, as represented by the 2011 preconsolidation points on Figure Nos. 3a and 3b, Appendix E.

5.3.2 Approach Fill Settlement

Settlements of the approach fills have been of a relatively large magnitude and ongoing in nature probably since the first structure was built at the Nepewassi Bridge site. The most accurate data on settlement is from the current study where it has been recorded that settlement of the approach fills has been in the order of 275 and 240 mm at the south and north abutments, respectively. This estimate has been based on patch thickness and void dimension and has occurred over a relatively short period of some 12 years, since construction of the new bridge.

The majority of this current settlement is, most likely, associated with ongoing consolidation of the clay stratum. However, some settlement may also be associated with consolidation of the LBFS fill, which, based on Borehole Nos. LVM-2 and LVM-3, varied in thickness from 3.3 and 3.7 m at the south and north abutments respectively. The NSP-263 for Type II LBFS fill specifies that backfill to structures shall be placed in lifts of 300 mm and compacted with three passes of a manually guided tamper such as a Bomag BPR 30/38D or equivalent. This unit has an operating weight of 175 kg and mass per square metre of base plate of 1439 kg/m^2. Compaction records for the Contract No. 98-215 are not available. Based on the above described compaction procedure, a compacted density somewhat less than 100% Standard Proctor Dry Density would have probably been achieved. The sampling and testing carried out during this investigation on the LBFS fill indicates that on average the fill is in a compacted state of approximately 100% SPDD (i.e. average unit weight of 15.5 kN/m^3 compacted to SPDD of 1580 kg/m^3). Based on the Standard Penetration test values, average 39 blows per 300 mm, the material has an average compactness described as dense. As such, it is possible that the thickness of the LBFS fill could have reduced in thickness by say up to 1 to 2%. This would result in possible settlement associated with consolidation of the LBFS fill of some 35 to 70 mm. The remaining settlement is associated with consolidation of the underlying clay.

As history, at this site, has shown this consolidation process is very slow due partially to the thickness of the relatively uniform clay deposit but also as a result of the variations in the underlying artesian groundwater pressure resulting in a slightly elevated and fluctuating positive pore water pressures in the clay stratum. Based on the current consolidation data, and present day loading, it is estimated that future settlement, due to consolidation of the clay stratum, could be some 500 and 300 mm and would take from 45 and 30 years to develop, at the south and north abutments respectively, if a “do nothing “ approach is taken.

In addition to vertical settlement a lateral displacement (spreading) may have developed in the underlying clay stratum. This lateral displacement, which generally develops shortly after loading (i.e. within say five years) and is of lower magnitude than the amount of vertical settlement, could result in an unaccounted for lateral load on the abutment piles. A preliminary review of the current centerline survey elevation of the south and north abutments relative to the centerline elevations at the two pier locations has been carried out and this data indicates the vertical alignment is of a continuous concave geometry, with no downward displacement at the abutment ends. This preliminary survey information indicates that there is no apparent excessive settlement of the abutments relative to the piers.

5.3.3 Approach Fill Remediation

As noted in the previous section, in order to reduce the magnitude and ongoing nature of future settlement of the embankment fill at the abutments, the vertical effective stress on the consolidating clay stratum will have to be reduced to a level less than the existing preconsolidation stress values in the clay stratum, as represented by the points on Figure Nos. 3a and 3b, Appendix D.

Alternate approaches to rectifying the impact of the settlement of the approach embankment fill at the abutments, or a “Do Nothing” approach have been considered. These are as follows:

a.) Remove approach embankment fill and increase the length of the bridge to span overtop of the sections of the approach embankment which are settling excessively.

b.) Remove approach embankment fill and replace with a lightweight fill material such that the vertical effective stress is reduced to a value less than the existing preconsolidation values of the clay. Two types of lightweight fill material being considered are Rigid Expanded Polystyrene (EPS) with a design unit weight of 0.5 kN/m^3 and Cellular Concrete with a design unit weight of 4.0 kN/m^3

c.) A “Do Nothing” approach would involve correcting the differential settlement, at both approaches, with patching. Based on the anticipated settlement it is likely that corrective patching would be required every three years initially. Every sixth year the guide rails would have to be refurbished along with the repaving to correct the settlement. Since the magnitude of settlement would decreases logarithmically over time, the frequency of repairs would also decrease with time. After 25 years the frequency of repair would probably increase to a five year interval.

The first alternative comes at a very high cost, lengthy construction duration and would require a detour to be constructed. As such it has not, at this time, been considered further since the method of using EPS lightweight fill to reconstruct an embankment is a proven method in Northeastern Region and Cellular Concrete has also been used by the Ministry. Both types of lightweight fill can be used and at a substantially lower cost than extending the bridge.

5.3.4 Lightweight Fill Embankment

In order to minimize the amount of future embankment fill settlement and lateral spread of the clay stratum to tolerable levels, specifically at the abutments, the effective vertical stress in the underlying supporting clay stratum must be reduced to a level less than the preconsolidation values of the clay. At the south embankment, where the clay deposit is thicker, it will be necessary to reduce the vertical effective stress by 50 kN/m^2 to achieve a stress level less than the preconsolidation level of the clay stratum, based on test results from this investigation. At the north embankment a reduction of 40 kN/m^2 will be required to achieve the same goal. This is represented on Figure Nos. 3a and 3b by the dashed line to the left of the current preconsolidation points.

Construction of an embankment using rigid expanded polystyrene (EPS) requires preparation of the subgrade with a 300 mm bed of Granular B Type I followed by placement of the EPS blocks with staggered joints, as the cross-section is stepped in to follow the 2:1 (H:V) foreslope of the embankment. The EPS blocks cannot be exposed to sun light for an extended period of time and will dissolve if in contact with hydrocarbons (gas, diesel, other solvents, etc.). As such, they have to be fully covered with a 10 mil polyethylene and the top blocks protected with 125 mm of reinforced concrete. To prevent differential freezing and to act as ballast the MTO requires an 875 mm thick pavement structure on top, as shown on Sketch SK-1, Appendix E.

Construction of an embankment using cellular concrete (CC) is of a similar cross-section but differs from EPS in the following ways:

1) The Cellular Concrete (CC) slurry can be poured directly on the freshly excavated subgrade without preparation. The CC acts as a mud slab and sets up overnight, depending upon temperature.
2) The sides are supported, during placement/pouring, with light plywood forms varying from 500 mm to 1.2 m in height. The unit weight of the CC is 4.0 kN/m^3 therefore the forms do not have to be as robust as with normal weight concrete.
3) The CC slurry, when hardened is inert to hydrocarbons deterioration and not susceptible to UV rays, therefore does not have to be covered with a protective covering.
4) The final lift of cellular concrete requires only a layer of 150 mm of granular base with a surface course of 50 mm asphalt to act as a wearing surface or concrete approach slab with asphalt surface. A typical cross-section is shown in SK-2 Appendix E.

Since the Cellular Concrete requires a thinner surface covering of granular base and asphalt, relative to the EPS cross-section, achieving a reduction of 50 kPa in vertical effective stress, at the south approach, would require an excavation of existing embankment fill to a depth of 4.5 m (elevation 203.3 m). If EPS is used, an excavation to a depth of 4.7 m (elevation 203.1 m) would be required. A similar situation occurs at the north embankment where a 40 kPa reduction in vertical effective stress is required. At the north approach embankment an excavation to a depth of 3.7 m (elevation 205.2 m) is required if Cellular Concrete is used to construct the lightweight fill cross-section and an excavation to a depth of 4.0 m (elevation 204.9 m) is required if EPS is used as the lightweight fill .

A comparison of the advantages and disadvantages, and associated cost, of the two lightweight fill products is shown on Table B Appendix E. As can be seen from this comparison the Cellular Concrete requires less subgrade preparation, is not as susceptible to damage during placement and fills voids without leaving gaps which are definite advantages in this rehabilitation case. In consideration of cost the Cellular Concrete is some 27% less than the Expanded Polystyrene. Based on this comparison we recommend that Cellular Concrete be used as a lightweight fill at the Nepewassi Bridge embankment approaches. Since the thickness of the granular layer directly over the Cellular Concrete is less than 300 mm, beyond the approach slab, it is recommended that a Granular O as per SSP 110S13 be used to promote lateral drainage.

The longitudinal extent to which this treatment, with lightweight fill, should be extended to is dependent upon the presence of the clay stratum and level of preconsolidation of the clay deposit under the embankment. Investigation and testing, as per the RFP, has concentrated on the embankment fills directly behind the abutments. No information is available as to movements of the embankment fills at various distances up and down chainage from the abutments. It is quite possible that the clay deposit has a higher level of preconsolidation closer to the valley walls where the historical overburden thickness may have been greater, considering the meandering nature of the river and possible erosion of past preload. To minimize the amount of future embankment settlement, which is predominately associated with the past grade raise, the treatment with lightweight fill could be extend for the full length of the previous treatment with LBFS fill, which is some 170 m at the south and some 50 m at the north.

This would be extremely costly, estimated at some $ 3.3 million using EPS and some $ 2.3 million using cellular concrete, plus roadway protection, estimated at $ 650,000.

The area where the future development of settlement is most critical is in the localized area directly behind the south and north abutments. If this treatment is localized (i.e. extended only 10 m back from each abutment), the development of future detrimental lateral forces on the abutment foundation piles would be reduced to a negligible magnitude. Additionally, any future settlement associated with consolidation of the clays would be outside of the areas treated with EPS or Cellular Concrete lightweight fill and would not adversely affect the bridge structure and could be corrected under the highway maintenance program with occasional patching of the surface. As such, limiting the treatment to a distance extending back from the north and south abutments 10 m, and then tapering down on a 2H:1V slope, longitudinally, would protect the structure (see Sketch Nos. SK-1 to SK-3, Appendix E). The estimated costs for this treatment at the abutments would be some $ 376,000 using EPS and some $ 272,000 using cellular concrete (CC), plus roadway protection, estimated at approximately $ 131,000.

It is necessary to maintain a uniform cross-section over the full width of the embankment when applying the lightweight fill treatment. Maintaining staggered joints would be more difficult with the EPS blocks than with the cellular concrete slurry if carried out in two separate operations with a longitudinal vertical separator (i.e. roadway protection).

A summary and comparison of the advantages and disadvantages of Rigid Expanded Polystyrene and Cellular Concrete as well as a cost estimate for the treatment noted above is presented on Table B, Appendix E.

The last alternate of "Do Nothing" will require patching the asphalt at the abutments, probably every third year, as the approach fill settles. Assuming that the bridge structure has a 50 year life expectancy these patching operations would have to continue for essentially the remaining life expectancy of the structure. If the "Do Nothing" alternate is used, it is then recommended that a detailed survey monitoring program be initiated to record if the abutments are moving laterally since the vertical settlement of the clay will also results in a lateral spread. This could apply an unaccounted for lateral load on the abutment(s) and steel H pile foundations which may result in lateral movement of the abutment(s). It is estimated that the cost to repave the approaches say every three years and repave and repair the guide rails every six years for the next 25 years, and then on a five year interval until 40 years (estimated 50 year bridge life) would have a total cumulative cost of $ 200,000.

5.3.5 Roadway Protection

The areas, either side of the Nepewassi Bridge, can be accessed via Highway 64 and Highway 17, or a shorter distance by township roads. As such a temporary, short term, closure of the bridge should be considered to allow excavation and uniform installation of the EPS treatment and save the roadway protection costs.

If the bridge must remain open during re-construction of the lightweight fill embankment a longitudinal system of roadway protection must be employed to maintain one lane of traffic with temporary traffic lights. This temporary flexible retaining type structure must be sufficiently robust to support the 4.7 m excavation and loading from the adjacent traffic lane and comply with a minimum performance level 2, as per OPSS 539. Considering the depth of excavation required and, more importantly, the requirement to maintain a uniform cross-section throughout the lightweight fill treatment, a relatively thin wall section, such as a steel sheet pile wall or soldier-pile and wood lagging system, should be considered. More elaborate retaining structures (roadway protection systems) such as pre-cast concrete panels, tangent, secant or staggered piles or concrete diaphragm walls have not been considered at this time due to their higher cost, greater thickness of wall (300 mm to 1200 mm) and more permanent nature of the structure which would make them more difficult to remove and subsequently effectively develop a uniform lightweight fill cross-section. A comparison of different Roadway Protection systems and their advantages and disadvantages is contained on Table A, Appendix E.

At Borehole No. LVM-2, located south of the south abutment, diamond core drilling was required to penetrate cobble and boulder size rock in the original embankment fill starting at elevation 203.2 m. Auger refusal was also met in the upper part of the original embankment fill, immediately below the deposit of LBFS, at Borehole Nos. LVM-1 and LVM-4. However, at adjacent Borehole No. LVM-4a (north side) auger drilling penetrated a 0.3 m thick obstruction layer allowing advance into the underlying silty clay. As such difficulties may be encountered in driving some of the steel sheet piles a sufficient depth below the excavation. As such a system of tie back anchors, with suitable sized whaler or soil nailing will have to be considered to resist the lateral earth pressure forces against the shoring. The LBFS fill is essentially a cohesionless material with a unit weight of 15.5 kN/m^3 and an estimated internal angle of friction of 35^o. A rectangular apparent earth pressure equivalent to the height of the excavation time 0.65K_a times the effective vertical stress at the base of the excavation can be used for temporary shoring design. Lateral earth pressures acting on a temporary support system should be computed in accordance with the Canadian Highway Bridge Design Code. The active earth condition (K_a) may be assumed to apply if the structure is designed to be yielding. For unyielding structures, the at rest condition (K_o) may be assumed to apply. The soil parameters for shoring design are tabulated below for the south and north approaches:

South Approach

Elevation From – To (m)	Water Elev. (m)	Soil Type	Unit Weight (KN/m^3)	Internal Angle of Friction (Deg)	In-situ Shear Strength s_u (kPa)	Anchorage Coefficient	Reduction Factor
207.8-204.1	202.5*	Lightweight Fill (Dense)	15.5	35		1.5	
204.1-198.2		Embankment Fill-frequent cobble/boulder rock sizes (Loose to Compact)	20.0	30		1.0	
198.2-182.8		Clay-(firm)	16.7		25		N/A **

* Note: 1) * Water level based on river water level at time of investigation. Water level will fluctuate seasonally.
2) ** Anchors not to be formed in firm or soft clay

North Approach

Elevation From – To (m)	Water Elev. (m)	Soil Type	Unit Weight (KN/m^3)	Internal Angle of Friction (Deg)	In-situ shear strength s_u (kPa)	Anchorage Coefficient	Reduction Factor
208.9-204.6	202.5*	Lightweight Fill (Dense)	15.5	35		1.5	
204.6-200.1		Silty Clay (stiff-very stiff)	17.0		100		0.45
200.1-187.3		Clay-(firm)	16.7		25		N/A**

* Note: 1)*Water level based on river water level at time of investigation. Water level will fluctuate seasonally.
2) ** Anchors not to be formed in firm or soft clay

Surcharge loads must be included in the lateral pressure calculations. Depending on the section properties of the support system, walers with rakers and bracing or a tieback system will be required. If a tieback system is considered, the pull-out resistance (R) for tremie-grouted anchors in **cohesionless** soils can be estimated from the following equation as supplied in the Canadian Foundation Engineering Manual (4rd Edition):

$\mathbf{R = \sigma_z' A_s L_s K_f}$ Where: $\boldsymbol{\sigma_z'}$= effective vertical stress at the midpoint of the load carrying length

$\mathbf{A_s}$ = effective unit surface area of the anchor

$\mathbf{L_s}$ = effective embedment length of the anchor

$\mathbf{K_f}$ = anchorage coefficient dependent on the soil type and conditions as given in the above Tables

The pull-out resistance (R) for grouted anchors in **clay** soils can be estimated from the following equation:

$\mathbf{R = \alpha_c A_s L_s s_u}$ Where $\boldsymbol{\alpha_c}$ =reduction factor in above table

$\mathbf{A_s}$ =effective unit surface area of the anchor bond zone

$\mathbf{L_s}$ = effective length of anchor bond zone

$\mathbf{S_u}$=average undrained shear strength of clay

Unless the pull-out resistance (capacity) of the anchor is proven with a load test program the allowable anchor load, as suggested by the Canadian Foundation Engineering Manual (4rd Edition), is commonly obtained by dividing the computed capacity of the anchor by a factor of safety of 3. Alternatively, proprietary anchor systems can be used.

5.3.6 Excavation and Dewatering

To prevent unbalanced lateral loading on the bridge and bridge foundations, due to fill removal from behind the abutments, excavation should be carried out in a balanced manner at the north and south abutments. As such, the excavation should be carried out simultaneously at both ends. The approaches must also be checked to ensure that unloading one side of the abutment does not result in an unbalanced torsional load on that abutment.

The LBFS backfill, below the pavement structure and approach slabs, and the original sands and gravel embankment fill as sampled in the south embankment at Borehole No. LVM-2 to a 9.6 m depth (elevation 198.2 m) can be considered a Type 3 soil in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. As such, side walls of temporary open excavations, above the water table, would have to be cut back to a angle of 1H:1V to remain temporarily stable. Replacement of the existing embankment fills with either EPS or CC lightweight fill will require an excavation to a depth of some 4.0 m, at the north abutment (elevation 204.9 m) and to a depth of some 4.7 m (elevation 203.1 m) at the south abutment. The ground water levels along the river bank at the abutments was at elevation 202.5 m as such the depth of excavation will be from 0.6 to 2.4 m above the anticipated groundwater level. These water levels will fluctuate seasonally and the groundwater surface may rise gradually as the excavation moves up grade away from the bridge proper therefore the contractor must be prepared to maintain his excavation in an unwatered state at all times during construction of the lightweight fill embankment.

5.3.7 Slope Stability

To construct the lightweight fill embankment an excavation of half the embankment width, with temporary support using roadway protection will be required. To assess the stability of the temporary support system a stability analysis has been undertaken with the Slope/W Software. This analysis indicates that the excavation and temporary shoring with tie back anchors would have a Factor of Safety in the order of 1.7 against a shallow failure as indicated on Figure No. S-1 Appendix E.

6.0 CLOSURE

Information provided in this report is valid only at the locations described above. Any assumptions of continuity of soil stratigraphy between boreholes, as shown on the enclosed cross-sections, is intended as an aid for design purposes only and does not constitute a statement of existing conditions for contractual or construction purposes. The report was prepared by Mr. J. R. Berghamer, P. Eng and reviewed by the firm's principal and MTO designate Mr. M. A. Merleau, P. Eng.

Details of the investigation, the material analysis and recommendation in this report are considered to be complete. However, should any questions arise, please do not hesitate to contact the undersigned.

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M. A. Merleau, P. Eng.
Principal Engineer
MTO Designate

J. R. Berghamer, P. Eng.
Regional Manager

Z:\PROJECT FILES\2011\11046 - PAVE & FDN, Hwy 535 Noelville to Hagar (AECOM)\`FOUNDATION\Reports\FINAL\Area 4 - GWP 5573-04-00 - Site 46-130 - Nepewassi River Bridge\' Rev 2\11046 - FINAL FIDR, Hwy 535 GWP 5573-04-00 - Nep River - R2.doc

Appendix A Key Plan

Figure No. 1: Key Plan

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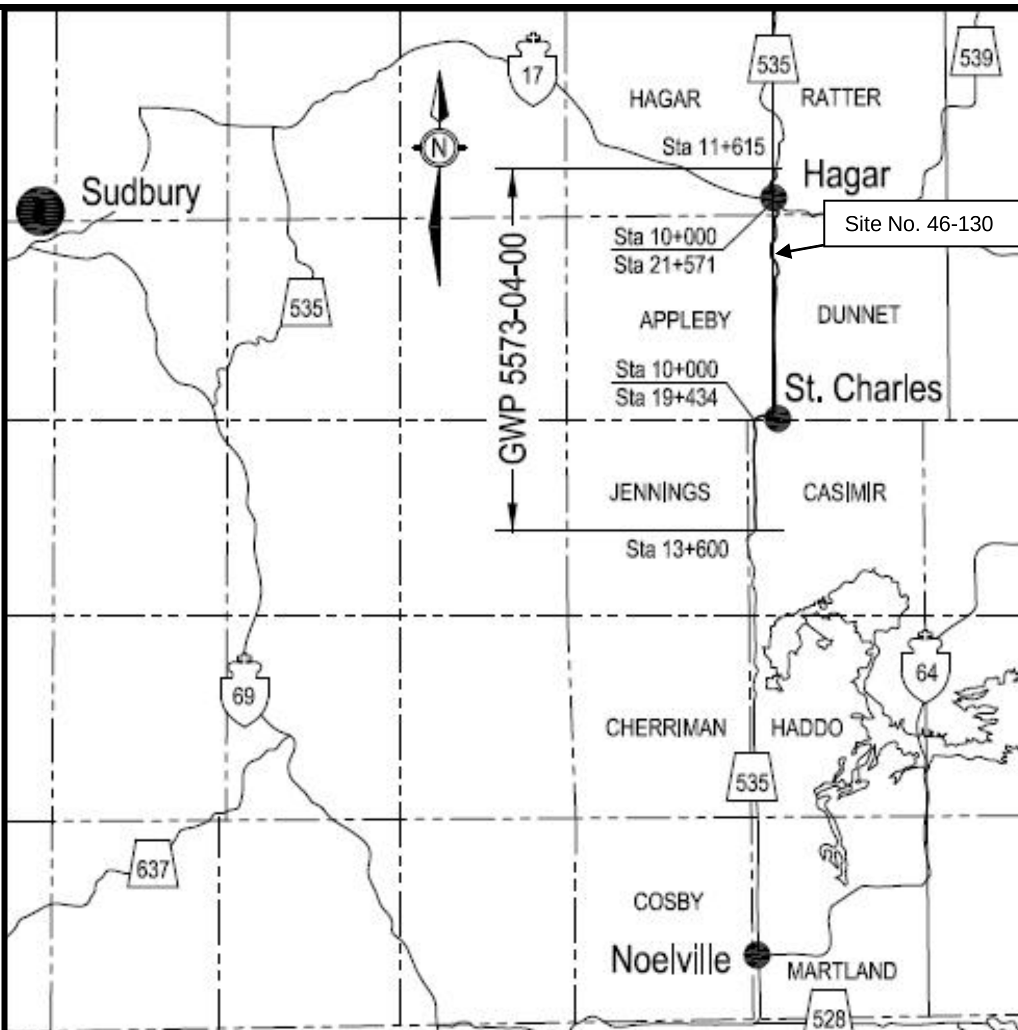
KEY PLAN

Figure No. 1

NOT TO SCALE



FINAL
FOUNDATON INVESTIGATION
AND DESIGN REPORT
GWP 5573-04-00
Highway 535
From 20.2 km North of
Highway 64, Northerly to Highway 17:
Including Highway 17/Highway 535 (Hagar)
Intersection;
and
OVR to 0.1 km North of Roy Lumber Co.
District of Sudbury

Ref. No.: 11/04/11046-F4 R2 December, 2011

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Appendix B Abbreviations
Record of Borehole Sheets

Enclosure No. 1: List of Abbreviations and Symbols
Enclosure Nos. 2 to 6: Record of Borehole Sheets

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LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

The abbreviations and terms, used to describe retrieved samples and commonly employed on the borehole logs, on the figures and in the report are as follows:

1. ABBREVIATIONS

AS Auger Sample
CS Chunk Sample
DS Denison type sample
FS Foil Sample
NP Non Plastic
PH Sampler advanced by hydraulic pressure
PM Sampler advanced by manual pressure
RC Rock core with size & percentage of recovery
SS Split Spoon
ST Slotted Tube
TO Thin-walled, open
TP Thin-walled, piston
WS Wash Sample

2. PENETRATION RESISTANCE/"N"

Dynamic Cone Penetration Test (DCPT):

A continuous profile showing the number of blows for each 300 mm of penetration of a 50 mm diameter 60° cone attached to AW rod driven by a 63 kg hammer falling 760 mm.

Plotted as ——•——•——•——•

Standard Penetration Test (SPT) or "N" Values

The number of blows of a 63 kg hammer falling 760 mm required to advance a 50 mm O.D. drive open sampler 300 mm.

3. SOIL DESCRIPTION

a) *Cohesionless Soils:*

"N" (blows/0.3 m)	*Relative Density*
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

3. SOIL DESCRIPTION (Cont'd)

b) *Cohesive Soils:*

Undrained Shear Strength (kPa)	*Consistency*
Less than 12	very soft
12 to 25	soft
25 to 50	firm
50 to 100	stiff
100 to 200	very stiff
over 200	hard

c) *Method of Determination of Undrained Shear Strength of Cohesive Soils:*

+ 3.2 - Field Vane test in borehole. The number denotes the sensitivity to remoulding.

D - Laboratory Vane Test

.. - Compression test in laboratory

For a saturated cohesive soil the undrained shear strength is taken as one-half of the undrained compressive strength.

4. TERMINOLOGY

Terminology used for describing soil strata is based on the proportion of individual particle sizes present in the samples (please note that, with the exception of those samples subject to a grain-size analysis, all samples were classified visually and the accuracy of visual examination is not sufficient to determine exact grain sizing):

Trace, or occasional	Less than 10%
Some	10 to 20%
With	20 to 30%
Adjective (i.e. silty or sandy)	30 to 40%
And (i.e. sand and gravel)	40 to 60%

5. LABORATORY TESTS

P Standard Proctor Test
A Atterberg Limit Test
GS Grain Size Analysis
H Hydrometer Analysis
C Consolidation

LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

SAMPLE DESCRIPTION NOTES:

1. **FILL:** The term fill is used to designate all man-made deposits of natural soil and/or waste materials. The reader is cautioned that fill materials can be very heterogeneous in nature and variable in depth, density and degree of compaction. Fill materials can be expected to contain organics, waste materials, construction materials, shot rock, rip-rap, and/or larger obstructions such as boulders, concrete foundations, slabs, abandoned tanks, etc.; none of which may have been encountered in the borehole. The description of the material penetrated in the borehole therefore may not be applicable as a general description of the fill material on the site as boreholes cannot accurately define the nature of fill material. During the boring and sampling process, retrieved samples may have certain characteristics that identify them as 'fill'. Fill materials (or possible fill materials) will be designated on the Borehole Logs. If fill material is identified on the site, it is highly recommended that testpits be put down to delineate the nature of the fill material. However, even through the use of testpits defining the true nature and composition of the fill material cannot be guaranteed. Fill deposits often contain pockets or seams of organics, organically contaminated soils or other deleterious material that can cause settlement or result in the production of methane gas. It should be noted that the origins and history of fill material is frequently very vague or non-existent. Often fill material may be contaminated beyond environmental guidelines and the material will have to be disposed of at a designated site (i.e. registered landfill). Unless requested or stated otherwise in this report, fill material on this site has not been tested for contaminants however, environmental testing of the fill material can be carried out at your request. Detection of underground storage tanks cannot be determined with conventional geotechnical procedures.

2. **TILL:** The term till indicates a material that is an unstratified, glacial deposit, heterogeneous in nature and, as such, may consist of mixtures and pockets of clay, silt, sand, gravel, cobbles and/or boulders. These heterogeneous deposits originate from a geological process associated with glaciation. It must be noted that due to the highly heterogeneous nature of till deposits, the description of the deposit on the borehole log may only be applicable to a very limited area and therefore, caution must be exercised when dealing with a till deposit. When excavating in till, contractors may encounter cobbles/boulders or possibly bedrock even if they are not indicated on the borehole logs. It must be appreciated that conventional geotechnical sampling equipment does not identify the nature or size of any obstruction.

3. **BEDROCK:** Auger refusal may be due to the presence of bedrock, but possibly could also be due to the presence of very dense underlying deposits, boulders or other large obstructions. Auger refusal is defined as the point at which an auger can no longer be practically advanced. It must be appreciated that conventional geotechnical sampling equipment does not differentiate between nature and size of obstructions that prevent further penetration of the boring below grade. Bedrock indicated on the borehole logs will be labeled 'possibly' or 'probable' etc. based on the response of the boring and sampling equipment, surrounding topography, etc. Bedrock can be proven at individual borehole locations, at your request, by diamond core drilling operations or, possibly, by testpits. It must also be appreciated that bedrock surfaces can be, and most times are, very erratic in nature (i.e. sheer drops, isolated rock knobs, etc.) and caution must be used when interpreting subsurface conditions between boreholes. A bedrock profile can be more accurately estimated, at the clients' request, through a series of closely positioned unsampled auger probes combined with core drilling.

4. **GROUNDWATER:** Although the groundwater table may have been encountered during this investigation and the elevation noted in the report and/or on the record of boreholes, it must be appreciated that the elevation of the groundwater table will fluctuate based upon seasonal conditions, localized changes, erratic changes in the underlying soil profile between boreholes, underlying soil layers with highly variable permeabilities, etc. These conditions may affect the design and type and nature of dewatering procedures. Cave-in levels recorded in borings give a general indication of the groundwater level in cohesionless soils however, it must be noted that cave-in levels may also be due to the relative density of the deposit, drilling operations etc.

ENCLOSURE NO.:2 (Pg. 1 of 1)

METRIC

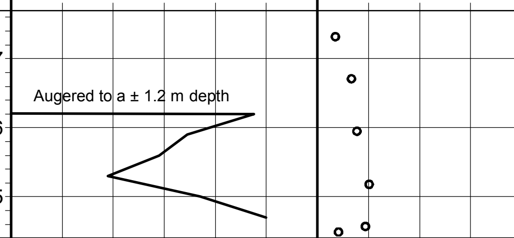
RECORD OF BOREHOLE NO. LVM-1

REFERENCE 11/04/11046-F4 R2 DATUM Geodetic
LOCATION N5331504.4 E372595.0 - Dunnet Township - Nepewassi River Bridge
ORIGINATED BY JL

PROJECT GWP 5573-04-00, Highway 535 - Site No. 46-130
BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers
COMPILED BY AT

CLIENT AECOM Inc.
DATE (Started) June 20, 2011
DATE (Completed) June 20, 2011
TIME 5:00:00 PM
CHECKED BY MAM

ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
207.7	Asphalt Surface										
0.0	± 200 mm Asphalt ± 175 mm Crushed Gravel		1	AS	N/A						
	FILL - brown to reddish brown lightweight blast furnace slag fill		2	SS	47		207				0 97 (3)
	(dense/compact)		3	SS	41		206	Augered to a ± 1.2 m depth			
			4	SS	22		205				
204.7 203.0 3.0	DCPT Refusal		5	SS	49/ 200mm						0 96 (4)
3.3	Auger Refusal Cobble/boulder size rock End of Borehole See Comments										



COMMENTS
Two additional borings (LVM-1a and LVM-1b) were advanced at Station 18+850, 2.0 m Rt of CL and Station 18+851, 1.8 m Rt of CL. Refusal was encountered at both borings at a depth of 3.1 m.

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3, \times^3$: Numbers on right refer to Sensitivity
Numbers on left refer to values greater than 120 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (yy/mm/dd)/Time	Water Depth (m)	Cave In (m)
1) 6/20/11 5:00:00 PM	DRY	2.9
2)	-	-
3)	-	-

MEL-GEO 11046 - BH LOGS - NEPEWASSI RIVER BRIDGE.GPJ MEL-GEO.GDT 12/8/11

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ENCLOSURE NO.:3 (Pg. 1 of 2)

METRIC

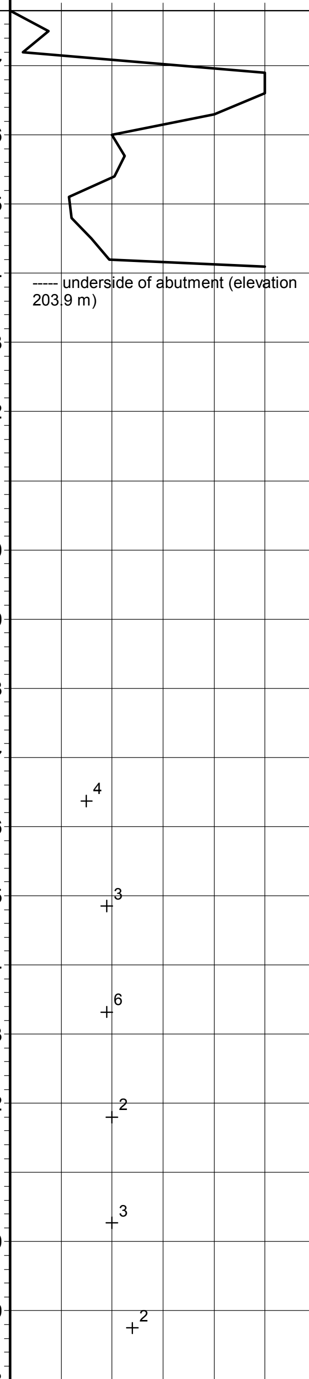
RECORD OF BOREHOLE NO. LVM-2

REFERENCE 11/04/11046-F4 R2 DATUM Geodetic LOCATION N5331508.1 E372600.4 - Dunnet Township - Nepewassi River Bridge ORIGINATED BY AT

PROJECT GWP 5573-04-00, Highway 535 - Site No. 46-130 BOREHOLE TYPE Track Mounted CME 55 - Hollow Stem Augers & N Casing COMPILED BY AT

CLIENT AECOM Inc. DATE (Started) June 16, 2011 DATE (Completed) June 16, 2011 TIME 5:20:00 PM CHECKED BY MAM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
207.8	Asphalt Surface							
0.0	± 240 mm Asphalt ± 280 mm Concrete ± 125 mm Void							
	FILL - brown to reddish brown lightweight blast furnace slag fill	1	SS	85	207	o		0 96 (4)
		2	SS	26	206	o		0 96 (4)
	(very dense/compact)	3	SS	16	205	o		0 95 (5)
204.1		4	SS	19		o		0 97 (3)
3.7	DCPT Refusal	5	SS	89/175 mm	204	o		
	FILL - sands and gravels frequent cobble/boulder size rock pieces in granular matrix	6	RC	N/A	203			
	core through 150 mm piece of rock at 4.6 m depth	7	SS	12	202	o		
		8	SS	2	201			
	core through blloulder size rocks from 7.6 to 9 m recovered 325 mm core	9	RC	N/A	200 / 199			
198.2								
9.6	CLAY - grey silty clay (medium to high plasticity) (firm)	10	SS	WH	198 / 197	o		
		11	TO	WH	196 / 195	├o┤	17.9	Consolidation Test
		12	SS	WH	194	├o┤		
		13	TO	WH	193 / 192	o		
		14	SS	WH	191	├o┤		
		15	TO	WH	190 / 189	├┤ o	16.6	Consolidation Test
	Continued Next Page				188			



COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

+3, ×3 : Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 120 kPa

O 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (yy/mm/dd)/Time	Water Depth (m)	Cave In (m)
1) 6/16/11 6:05:00 PM	DRY	4.6
2)	-	-
3)	-	-

MEL-GEO 11046 - BH LOGS - NEPEWASSI RIVER BRIDGE.GPJ MEL-GEO.GDT 12/8/11

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METRIC

RECORD OF BOREHOLE NO. LVM-2

REFERENCE 11/04/11046-F4 R2 DATUM Geodetic
LOCATION N5331508.1 E372600.4 - Dunnet Township - Nepewassi River Bridge
ORIGINATED BY AT
PROJECT GWP 5573-04-00, Highway 535 - Site No. 46-130
BOREHOLE TYPE Track Mounted CME 55 - Hollow Stem Augers & N Casing
COMPILED BY AT
CLIENT AECOM Inc.
DATE (Started) June 16, 2011
DATE (Completed) June 16, 2011
TIME 5:20:00 PM
CHECKED BY MAM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa (FIELD VANE +)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
	Continued from Previous Page									
	CLAY - grey silty clay (medium to high plasticity) (firm)	16	SS	WH		187	+ 3			
		17	TO	WH		186	+ 2		16.4	Consolidation Test
						185				
						184				
182.8		18	SS	WH		183				
25.0	End of Sampling End of Borehole									

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60)

ENCLOSURE NO.:4 (Pg. 1 of 2)

METRIC

RECORD OF BOREHOLE NO. LVM-3

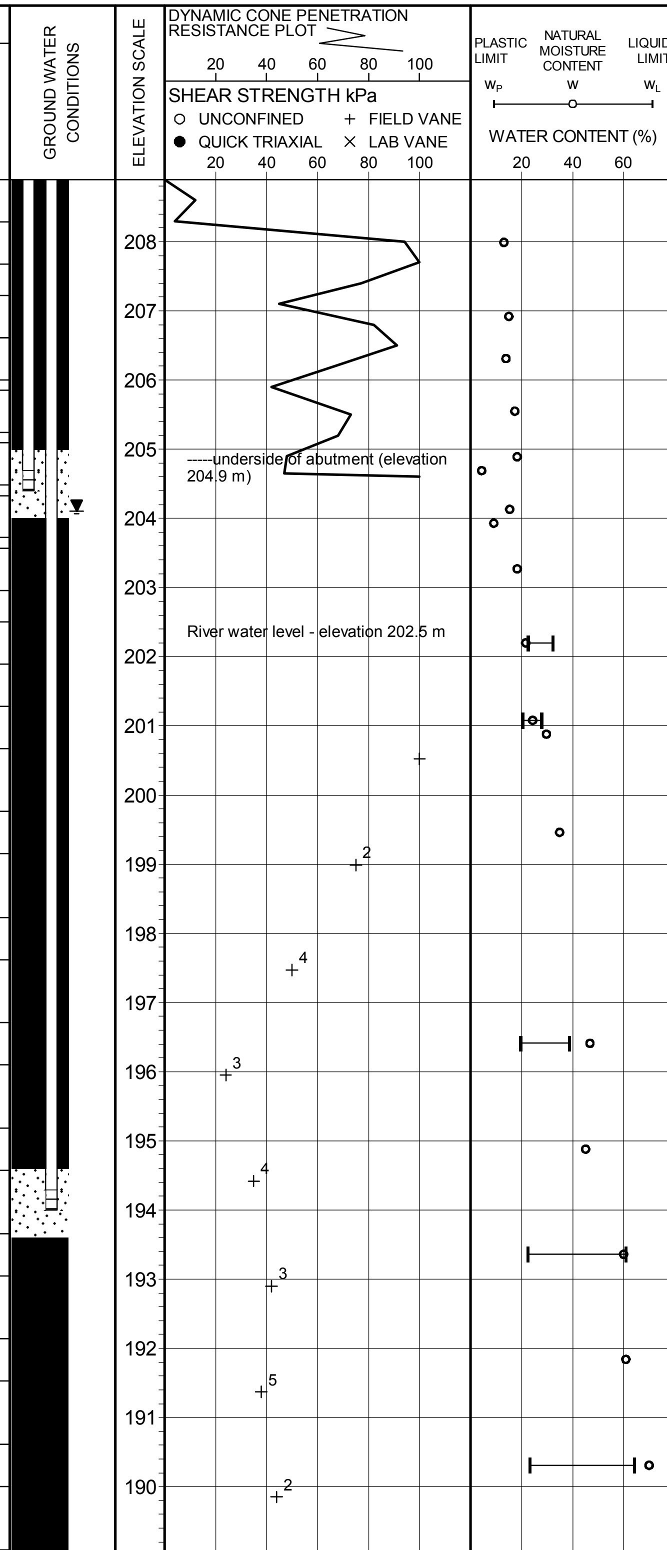
REFERENCE 11/04/11046-F4 R2	DATUM Geodetic	LOCATION N5331525.3 E372618.2 - Dunnet Township - Nepewassi River Bridge	ORIGINATED BY AT
PROJECT GWP 5573-04-00, Highway 535 - Site No. 46-130		BOREHOLE TYPE Track Mounted CME 55 - Hollow Stem Augers & N Casing	COMPILED BY AT
CLIENT AECOM Inc.		DATE (Started) June 14, 2011 / DATE (Completed) June 15, 2011 / TIME 4:20:00 AM	CHECKED BY MAM

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
208.9	Asphalt Surface
0.0	± 200 mm Asphalt ± 350 mm Concrete ± 130 mm Void FILL - brown to reddish brown lightweight blast furnace slag fill (very dense/ compact)
204.6 4.3	DCPT Refusal Geotextile at 4.3 m depth SILTY CLAY - grey to black silty clay trace sand trace organics trace asphalt sand and gravel layer 150 mm thick at 5.5 m depth (very stiff)
200.1 8.8	CLAY - grey silty clay and clay (high/medium plasticity) (stiff/very stiff) (firm) (stiff)
	Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
1	SS	81	2 93 (5)
2	SS	22	1 97 (2)
3	SS	33	1 96 (3)
4	SS	45	
5	SS	33	0 96 (4)
6	SS	11	
7	SS	7	
8	SS	8	
9	SS	6	
10	SS	4	
11	TO	WH	
12	SS	WH	
13	SS	WH	
14	TO	WH	
15	SS	WH	
16	TO	WH	Unit weight 15.9 kN/m³; Consolidation Test



COMMENTS
Water levels shown on log refer to the 15 m deep piezometer (i.e. tip ay elevation 194 m).
Water levels in the 4.7 m deep well was measured at 4.5 m depth on June 20, and was dry on August 16, 2011.

The stratification lines represent approximate boundaries. The transition may be gradual.

+3, ×3 : Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 120 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (yy/mm/dd)/Time	Water Depth (m)	Cave In (m)
1) 8/16/11	4.8	-
2)	-	-
3)	-	-

MEL-GEO 11046 - BH LOGS - NEPEWASSI RIVER BRIDGE.GPJ MEL-GEO.GDT 12/8/11

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ENCLOSURE NO.:4 (Pg. 2 of 2)

METRIC

RECORD OF BOREHOLE NO. LVM-3

REFERENCE 11/04/11046-F4 R2 DATUM Geodetic
LOCATION N5331525.3 E372618.2 - Dunnet Township - Nepewassi River Bridge ORIGINATED BY AT
PROJECT GWP 5573-04-00, Highway 535 - Site No. 46-130
BOREHOLE TYPE Track Mounted CME 55 - Hollow Stem Augers & N Casing COMPILED BY AT
CLIENT AECOM Inc.
DATE (Started) June 14, 2011
DATE (Completed) June 15, 2011 TIME 4:20:00 AM CHECKED BY MAM

ELEV DEPTH	DESCRIPTION (Continued from Previous Page)	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
	CLAY - grey silty clay and clay (high/medium plasticity)	17	SS	WH	188	4			
187.3									
187.0 / 187.0 21.9	SILTY SAND - grey silty sand trace gravel trace clay Auger Refusal End of Borehole	18	TO	WH	187				

Column headings: SOIL PROFILE; STRATA PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60.

ENCLOSURE NO.:5 (Pg. 1 of 1)

METRIC

RECORD OF BOREHOLE NO. LVM-4

REFERENCE 11/04/11046-F4 R2 DATUM Geodetic LOCATION N5331529.3 E372624.0 - Dunnet Township - Nepewassi River Bridge ORIGINATED BY JL

PROJECT GWP 5573-04-00, Highway 535 - Site No. 46-130 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY RG

CLIENT AECOM Inc. DATE (Started) June 21, 2011 DATE (Completed) June 21, 2011 TIME 9:10:00 AM CHECKED BY MAM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
209.1	Asphalt Surface					
0.0	± 275 mm Asphalt ± 250 mm Crushed Gravel	1	AS	N/A		
	FILL - brown to reddish brown lightweight blast furnace slag fill	2	SS	50/ 125 mm	208	
	(compact/very dense)	3	SS	44	207	0 97 (3)
		4	SS	29		
		5	SS	39	206	
		6	SS	58	205	0 96 (4)
204.6 4.5	Auger Refusal End of Borehole See Borehole No. LVM-4a					

SOIL PROFILE | SAMPLES | STRATA PLOT | GROUND WATER CONDITIONS | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100 | PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60 | UNIT WEIGHT γ kN/m³

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

+3, ×3 : Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 120 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS Date (yy/mm/dd)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

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ENCLOSURE NO.:6 (Pg. 1 of 1)

METRIC

RECORD OF BOREHOLE NO. LVM-4a

REFERENCE 11/04/11046-F4 R2 DATUM Geodetic

LOCATION N5331529.3 E372624.0 - Dunnet Township - Nepewassi River Bridge

ORIGINATED BY JL

PROJECT GWP 5573-04-00, Highway 535 - Site No. 46-130

BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers

COMPILED BY AT

CLIENT AECOM Inc.

DATE (Started) September 8, 2011

DATE (Completed) September 8, 2011

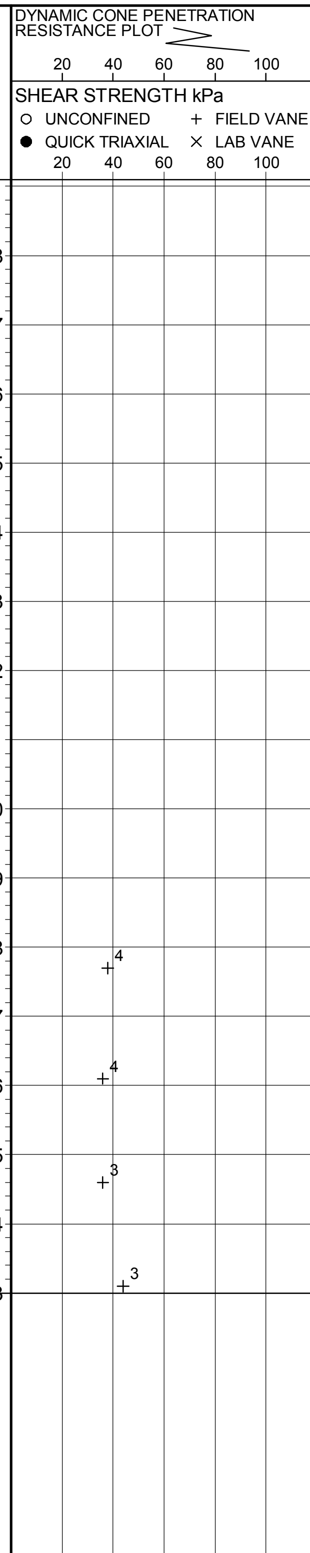
TIME

CHECKED BY MAM

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES
209.1	Asphalt Surface			
0.0	± 225 mm Asphalt FILL - brown to reddish brown lightweight blast furnace slag fill based on auger cuttings			
204.8 4.3	FILL - sands and gravels some silt frequent cobble/boulder size rock pieces in granular matrix	1	SS	50/100mm
203.8 5.3	SILTY CLAY - dark grey silty clay trace organics trace sand (very stiff)	2	SS	9
		3	SS	9
		4	SS	8
200.0 9.1	CLAY - grey clay sand w/ silt seam at 9.2 m (firm)	5	SS	7
		6	SS	PM
		7	SS	PM
		8	SS	PM
		9	SS	PM
193.0 16.1	End of Sampling End of Borehole			

GROUND WATER CONDITIONS

ELEVATION SCALE: 208, 207, 206, 205, 204, 203, 202, 201, 200, 199, 198, 197, 196, 195, 194, 193



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

WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

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

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3$, $\times^3$: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 120 kPa

O 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (yy/mm/dd)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

MEL-GEO 11046 - BH LOGS - NEPEWASSI RIVER BRIDGE.GPJ MEL-GEO.GDT 12/8/11

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Appendix C Borehole Location Plan Labwork

Figure No. 2: Borehole Location and Soil Strata

Figure Nos. L-1a, L-1b and L-2: Grain Size Analysis Graph

Figure Nos. L-3 and L-4: Plasticity Chart

Figure No. L-5: In-Situ Shear Strengths vs. Elevation

Figure Nos. L-6a to L9c: Consolidation Test Results and Summary

Figure No. L-10: Laboratory Tests- Summary Sheet

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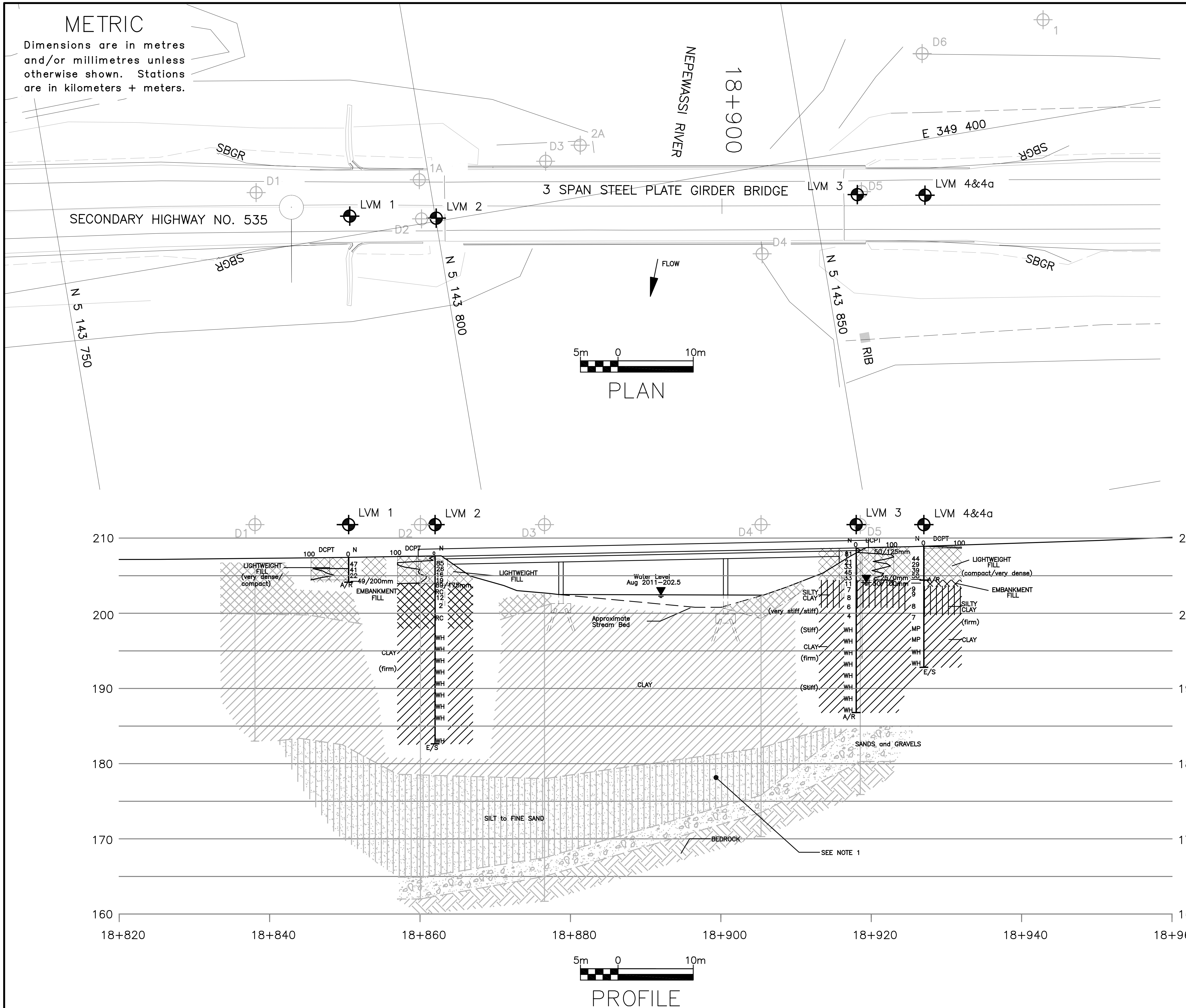
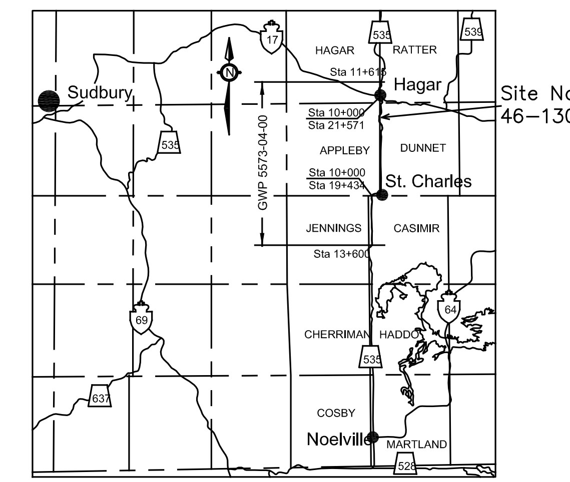


SITE No 46–130
WP No 5573–04–00
GEOCRES No 41I–278

HWY NO. 535 – Township of Dunnet
Nepewassi River Bridge – Site No. 46–130
Approach Fill Settlement
BOREHOLE LOCATIONS & SOIL STRATA

Figure 2

L|V|M | MERLEX



KEY PLAN – NOT TO SCALE

LEGEND

LVM Borehole — Boreholes by Others
"N" Blows/0.3 m (Std Pen Test, 475 J/blow)
DCPT Blows/0.3 m (60° Cone, 475 J/blow)
Water Level at Time of Investigation
A/R Auger Refusal at Elevation
E/S End of Sampling

Borehole No.	Elev.	O/S	Station	Co–ordinates Northerly	Co–ordinates Easterly
Borehole No. LVM1	207.7	1.2m Rt	18+851	5143789	349398
Borehole No. LVM2	207.8	1.5m Rt	18+862	5143800	349400
Borehole No. LVM3	208.9	1.5m Lt	18+918	5143856	349406
Borehole No. LVM4	209.1	1.1m Lt	18+927	5143865	349408
Borehole No. LVM4a	209.1	1.1m Lt	18+928	5143866	349408

NOTE 1:
Borehole Nos. D1, D2, D3, D4, D5, D6, 1A, 2A and 1 (in greyscale) were advanced under three previous investigations by others. Reproduced for general information and completeness of stratigraphy.

NOTE 2:
The boundaries between soil strata have been established at the borehole locations only. The boundaries illustrated and stratigraphy between boreholes on this drawing are assumed based on borehole data and may vary. They are intended for design purposes only.

REVISIONS DATE	BY	DESCRIPTION
Oct 2011	MCM	Final

HWY No. 535 – Dunnet Twp – Nepewassi River Bridge		REF: 11046
SUBM'D		SITE 46–130
DRAWN RG	CHK MAM	DATE June 2011 / FIG 2

Reference No.: 11/04/11046-F4 R2
Date: December 2011

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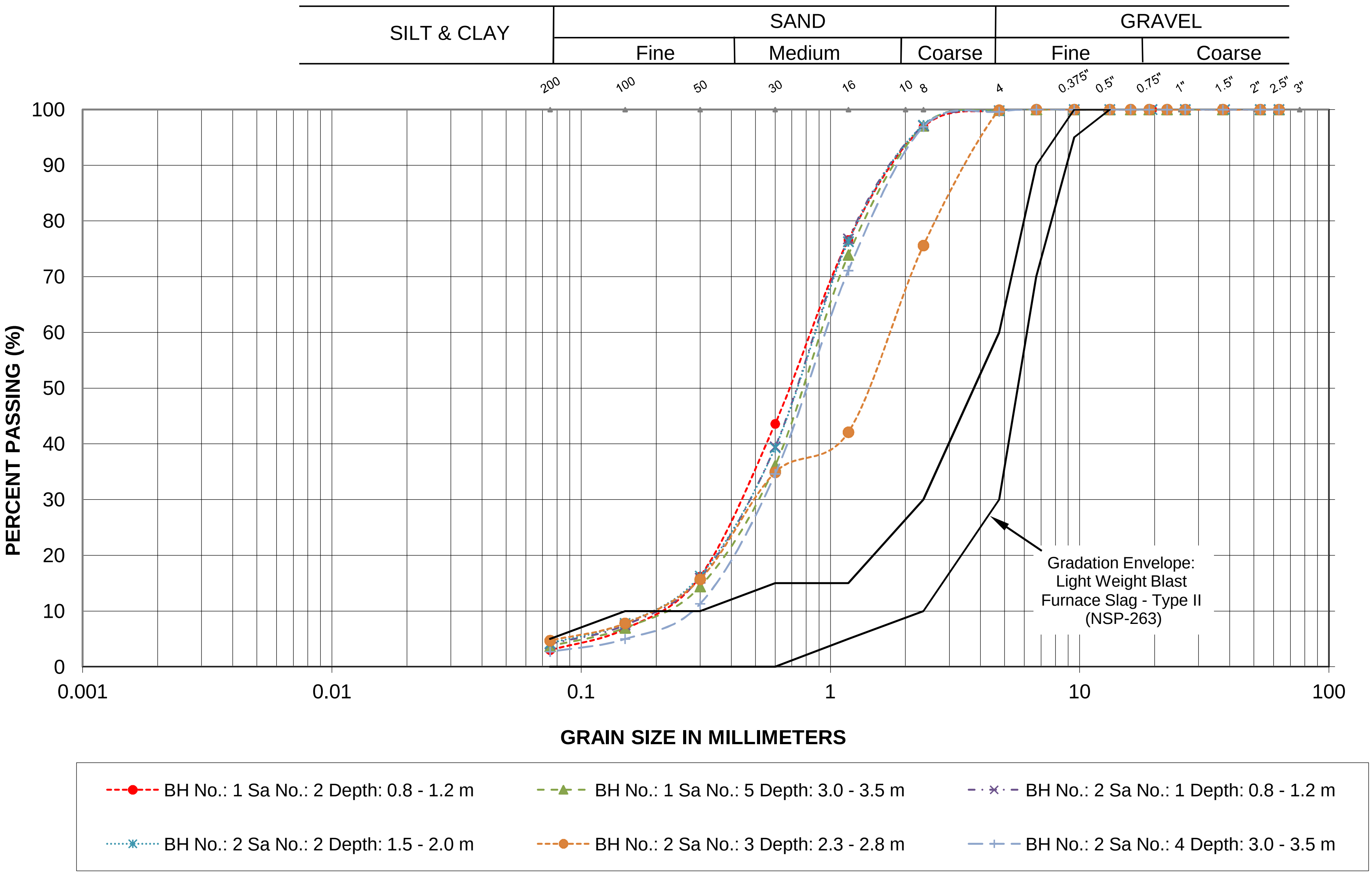
GRAIN SIZE ANALYSIS



PROJECT: Hwy 535 - Nepewassi Bridge
LOCATION: Site No. 46-130

LBFS FILL

Reference No.: 11/04/11046-F4 R2
Date: December 2011

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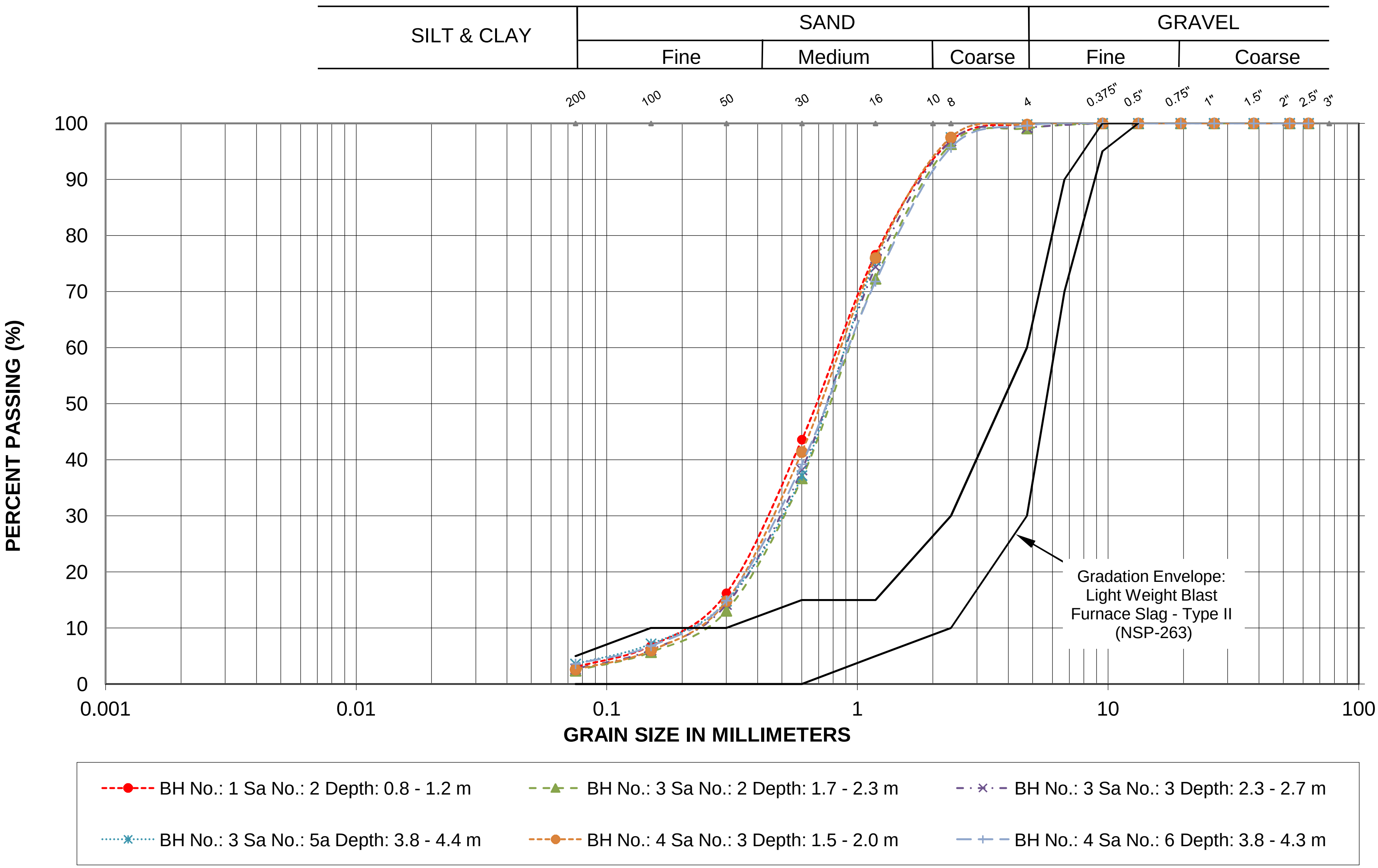
GRAIN SIZE ANALYSIS



LBFS FILL

PROJECT: Hwy 535 - Nepewassi Bridge
LOCATION: Site No. 46-130

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FIGURE L-1b

Reference No.: 11/04/11046-F4 R2
Date: December 2011

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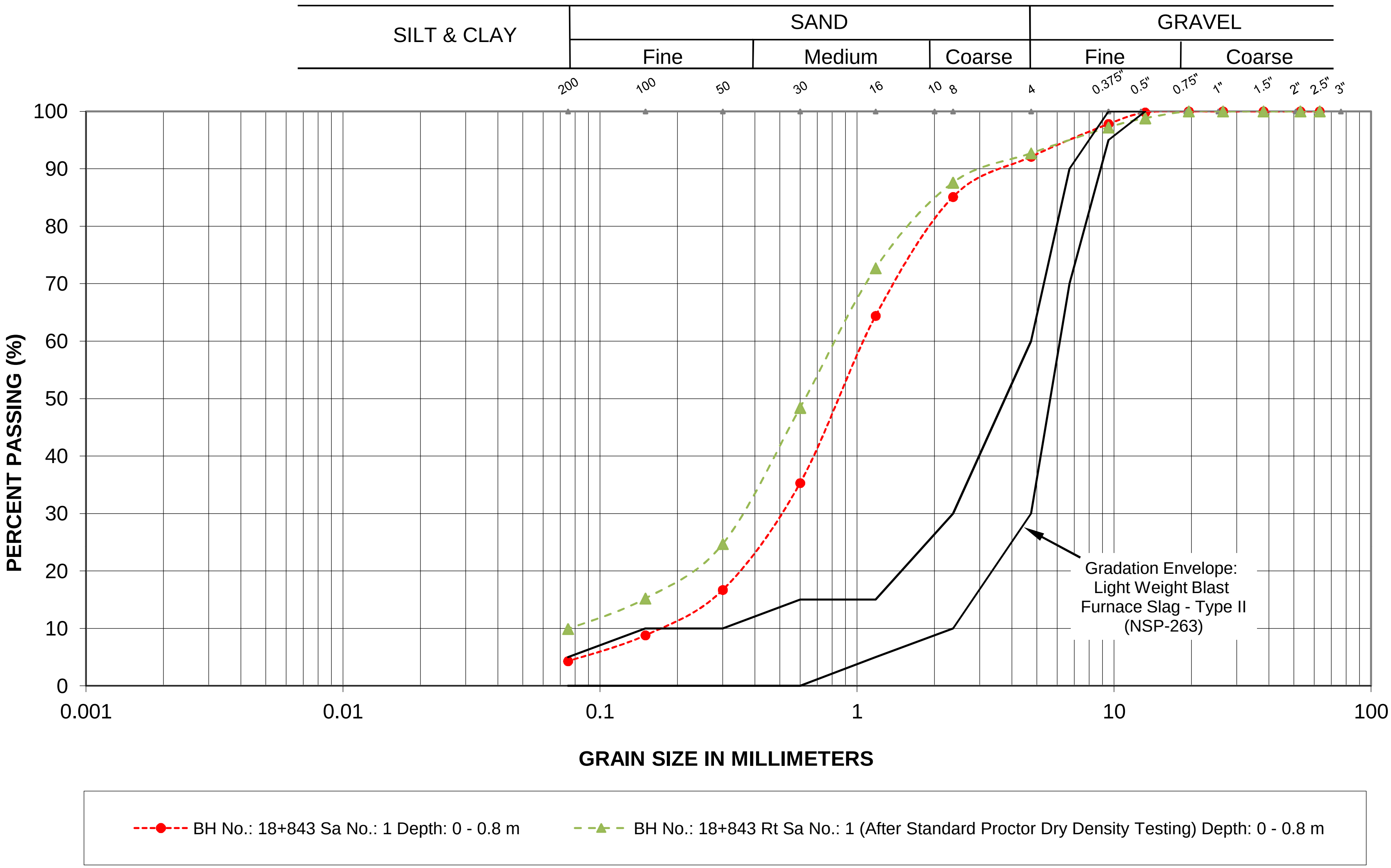
GRAIN SIZE ANALYSIS



PROJECT: Hwy 535 - Nepewassi Bridge
LOCATION: Site No. 46-130

LBFS Fill
Before and After SPDD Test
Standard Proctor Density 1580kg/m^3
Optimum Moisture 20.5%
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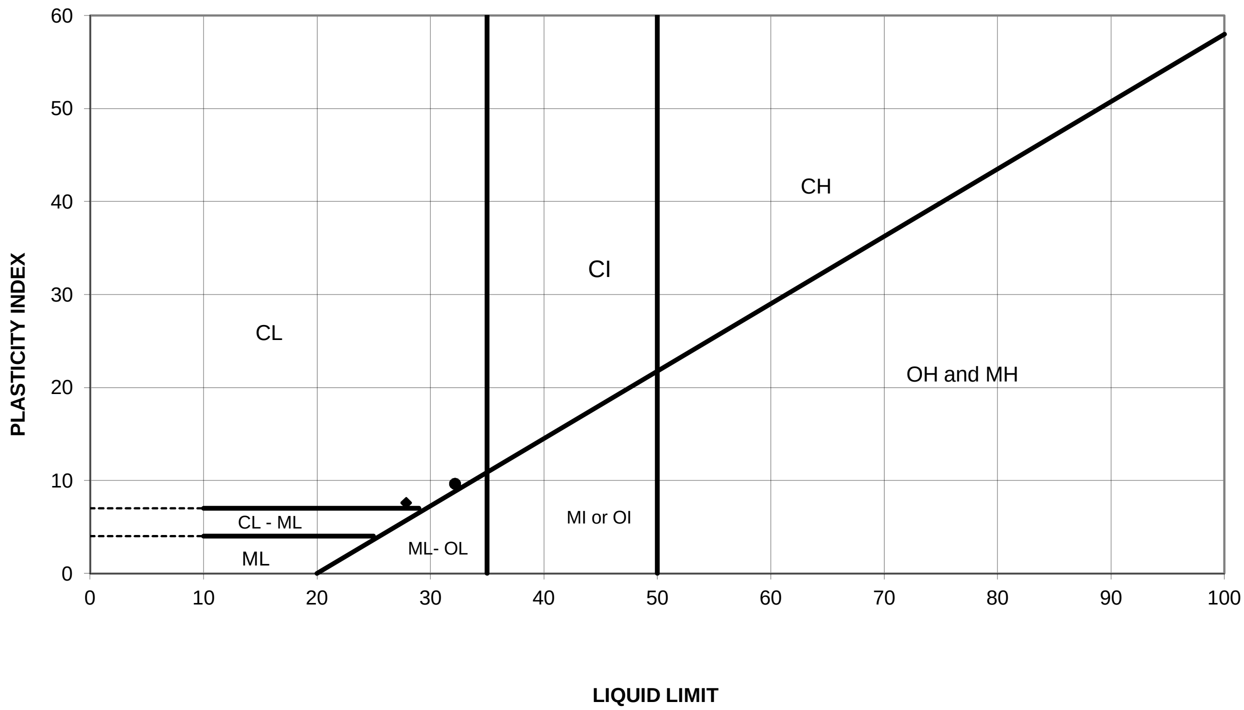
FIGURE L-2

ATTERBERG LIMITS TEST RESULTS	FIGURE L-3

ATTERBERG INDICES



SYMBOL	BH	Sa. No.	Depth(m)	Elev.(m)	Liquid Limit	Plastic Limit	Plasticity Index	NMC %
●	3	8	6.1	202.8	32.2	22.5	9.7	21.6
◆	3	9a	7.6	201.3	27.8	20.2	7.6	24.3

Date: Dec-11
Project: Hwy 535 - Site 46-130
G.W.P: 5573-04-00

Prep'd: AT
Chkd: RG
Ref. No.: 11/04/11046-F4 R2

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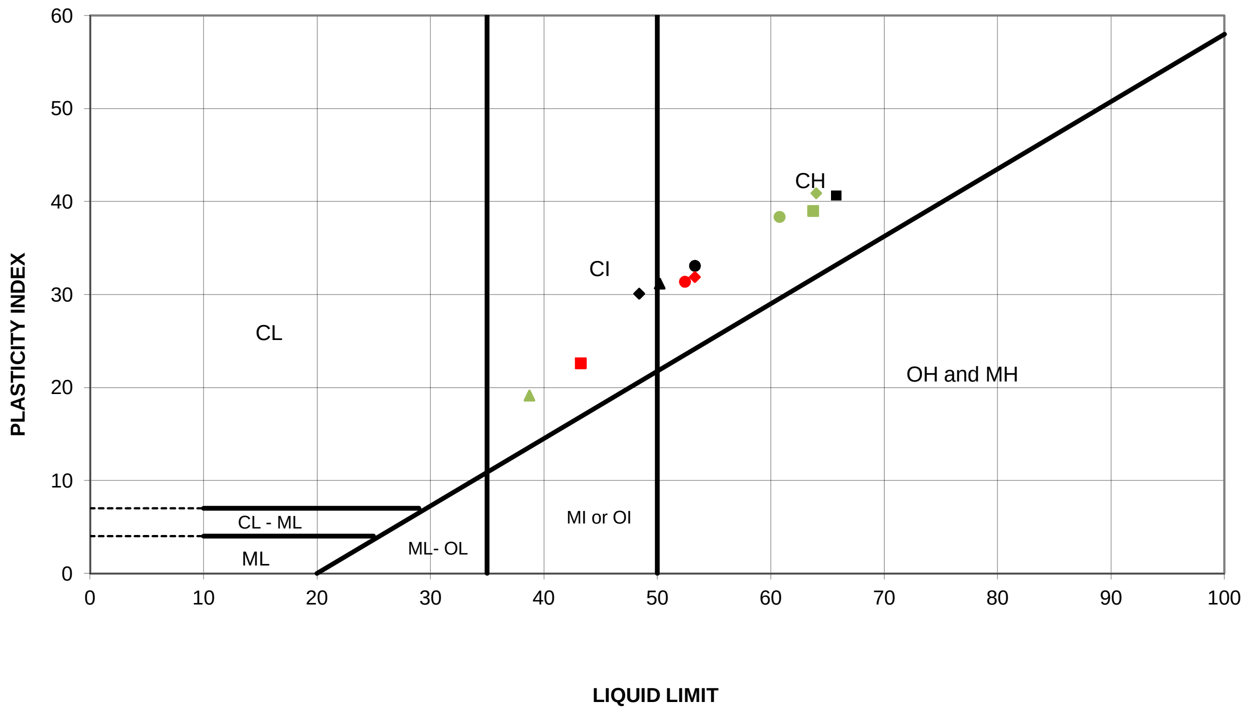
ATTERBERG LIMITS TEST RESULTS	FIGURE L-4

ATTERBERG INDICES



SYMBOL	BH	Sa. No.	Depth(m)	Elev.(m)	Liquid Limit	Plastic Limit	Plasticity Index	NMC %
● (black)	2	11	12.2	195.6	53.3	20.2	33.1	40.3
◆ (black)	2	12	13.7	194.1	48.4	18.3	30.1	41.8
■ (black)	2	14	16.8	191.0	65.8	25.1	40.7	58.1
▲ (black)	2	15	18.3	189.5	50.2	19.0	31.2	63.5
● (red)	2	16	19.8	188.0	52.4	21.0	31.4	55.4
◆ (red)	2	17	21.3	186.5	53.3	21.4	31.9	60.6
■ (red)	2	18	24.4	183.4	43.2	20.6	22.6	38.4
▲ (green)	3	12	12.2	196.7	38.7	19.6	19.2	46.8
● (green)	3	14	15.2	193.7	60.8	22.4	38.4	59.9
◆ (green)	3	16	18.3	190.6	64.0	23.1	40.9	69.9
■ (green)	3	17	19.8	189.1	63.7	24.7	39.0	62.6

Date: Dec-11
Project: Hwy 535 - Site 46-130
G.W.P: 5573-04-00

Prep'd: AT
Chkd: RG
Ref. No.: 11/04/11046-F4 R2

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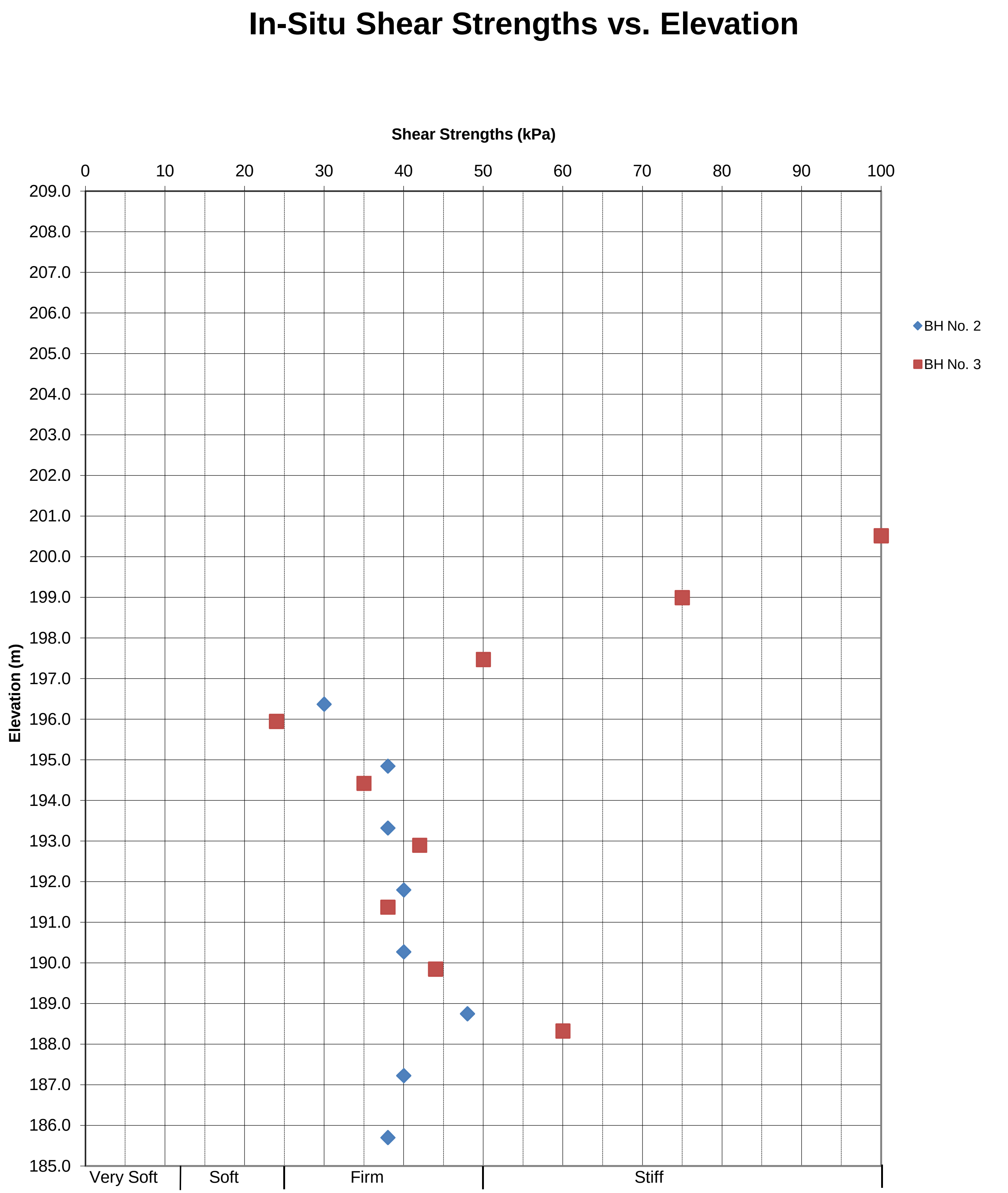
IN-SITU SHEAR STRENGTHS	Figure No. L-5



Date: December 2011
Project: 11/04/11046-F4 R2
Location: Site No. 46-130

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Checked:MAM

CONSOLIDATION TEST SUMMARY

FIGURE L-6a

SAMPLE IDENTIFICATION

Project Number	11-1183-0039	Sample Number	11
Borehole Number	2	Sample Depth, m	12.2

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	10		
Date Started	6/24/2011		
Date Completed	7/08/2011		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m^3	17.92
Sample Diameter, cm	6.30	Dry Unit Weight, kN/m^3	12.69
Area, cm^2	31.18	Specific Gravity, measured	2.75
Volume, cm^3	79.14	Solids Height, cm	1.195
Water Content, %	41.20	Volume of Solids, cm^3	37.25
Wet Mass, g	144.65	Volume of Voids, cm^3	41.89
Dry Mass, g	102.44	Degree of Saturation, %	100.8

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	c_v cm^2/s	m_v m^2/kN	k cm/s
0.00	2.538	1.125	2.538				
5.03	2.525	1.114	2.532	1	1.36E+00	1.00E-03	1.34E-04
9.03	2.522	1.111	2.524	83	1.63E-02	2.96E-04	4.71E-07
20.02	2.515	1.106	2.519	235	5.72E-03	2.47E-04	1.39E-07
40.46	2.502	1.094	2.509	485	2.75E-03	2.60E-04	7.01E-08
79.83	2.475	1.072	2.488	485	2.71E-03	2.68E-04	7.11E-08
160.23	2.420	1.026	2.448	576	2.20E-03	2.69E-04	5.81E-08
315.19	2.240	0.875	2.330	2907	3.96E-04	4.59E-04	1.78E-08
629.29	2.077	0.739	2.158	1500	6.58E-04	2.04E-04	1.32E-08
1259.30	1.950	0.632	2.014	853	1.01E-03	7.97E-05	7.87E-09
2470.11	1.841	0.541	1.895	628	1.21E-03	3.54E-05	4.21E-09
1259.30	1.845	0.544	1.843				
315.19	1.883	0.576	1.864				
79.83	1.932	0.617	1.908				
20.02	1.976	0.654	1.954				
5.03	2.014	0.686	1.995				

Note:
k calculated using cv based on t_{90} values.
Specimen swelled under 5kPa
Specimen taken 12cm from the bottom of the tube

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	2.01	Unit Weight, kN/m^3	20.40
Sample Diameter, cm	6.30	Dry Unit Weight, kN/m^3	16.00
Area, cm^2	31.18	Specific Gravity, measured	2.75
Volume, cm^3	62.80	Solids Height, cm	1.195
Water Content, %	27.54	Volume of Solids, cm^3	37.25
Wet Mass, g	130.65	Volume of Voids, cm^3	25.55
Dry Mass, g	102.44		

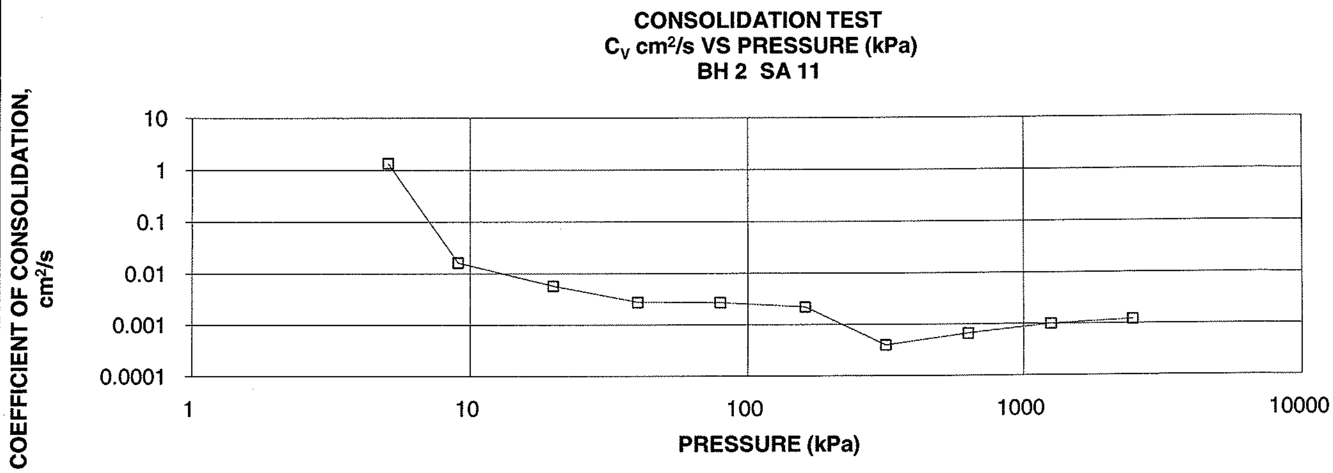
Prepared By: LFG

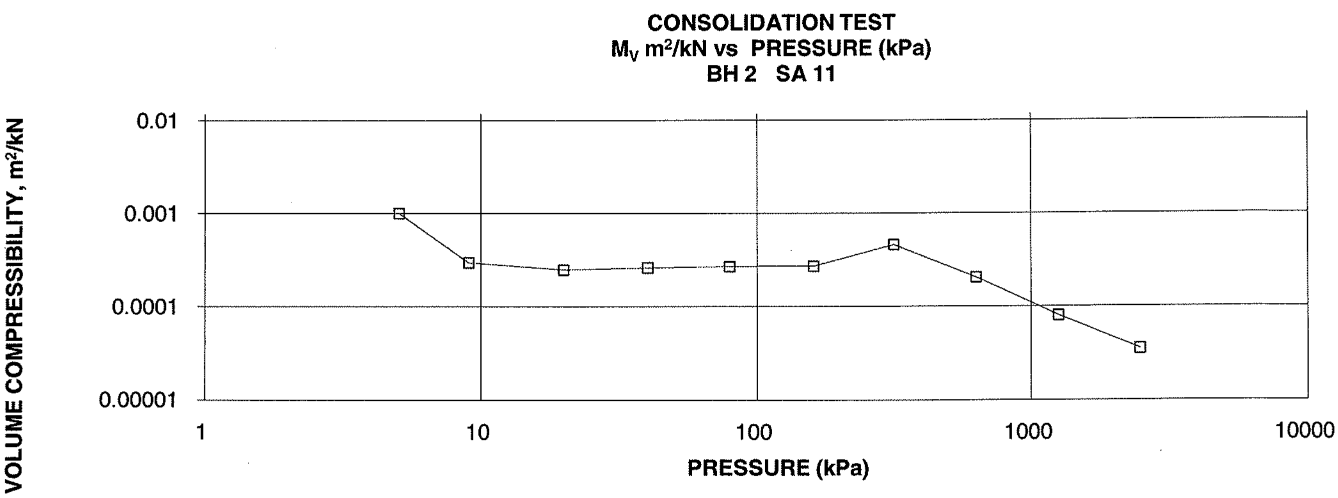
Golder Associates

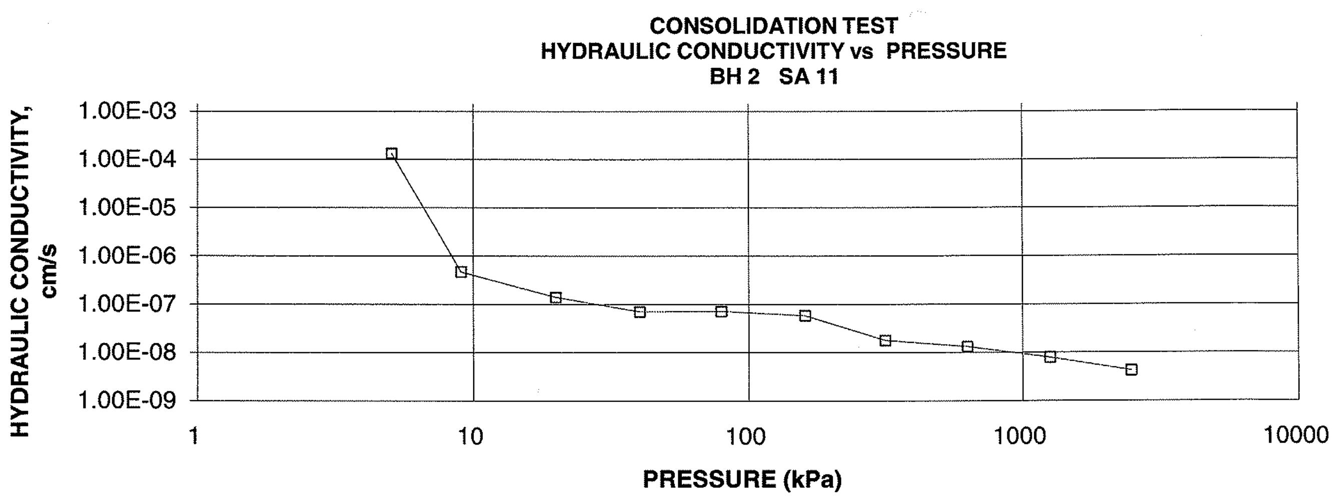
Checked By: [signature]

CONSOLIDATION TEST SUMMARY

FIGURE L-6b





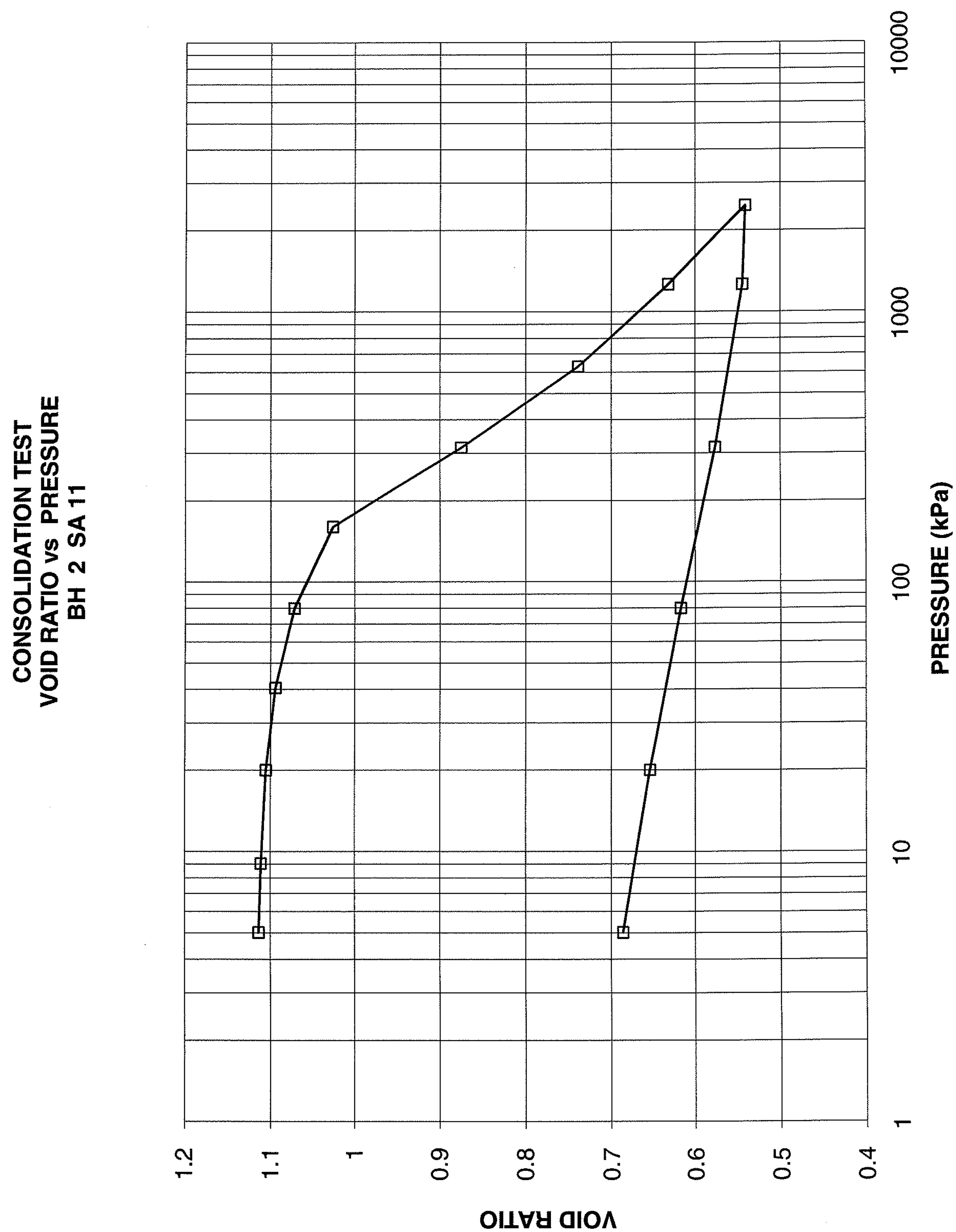


Project No. 11-1183-0039
Prepared By: LFG

Golder Associates

Checked By:

CONSOLIDATION TEST VOID RATIO VS LOG PRESSURE	FIGURE L-6c



Project No. 11-1183-0039

Prepared By: LFG

Golder Associates

Checked By: [illegible]

CONSOLIDATION TEST SUMMARY

FIGURE L-7a

SAMPLE IDENTIFICATION

Project Number	11-1183-0039	Sample Number	15
Borehole Number	2	Sample Depth, m	18.3

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	7/01/2011		
Date Completed	7/17/2011		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m^3	16.59
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	10.87
Area, cm^2	31.55	Specific Gravity, measured	2.69
Volume, cm^3	80.07	Solids Height, cm	1.045
Water Content, %	52.71	Volume of Solids, cm^3	32.99
Wet Mass, g	135.50	Volume of Voids, cm^3	47.09
Dry Mass, g	88.73	Degree of Saturation, %	99.3

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	c_v cm^2/s	m_v m^2/kN	k cm/s
0.00	2.538	1.428	2.538				
4.98	2.538	1.427	2.538	2	6.83E-01	7.91E-06	5.29E-07
10.00	2.534	1.424	2.536	208	6.55E-03	3.14E-04	2.02E-07
19.96	2.529	1.419	2.531	652	2.08E-03	2.02E-04	4.12E-08
39.99	2.507	1.398	2.518	591	2.27E-03	4.33E-04	9.64E-08
80.00	2.473	1.365	2.490	620	2.12E-03	3.36E-04	6.98E-08
159.80	2.389	1.285	2.431	1500	8.35E-04	4.14E-04	3.39E-08
313.28	2.170	1.075	2.279	2746	4.01E-04	5.62E-04	2.21E-08
621.99	1.986	0.899	2.078	1771	5.17E-04	2.35E-04	1.19E-08
1242.10	1.843	0.763	1.914	1017	7.64E-04	9.09E-05	6.81E-09
2484.23	1.714	0.639	1.778	709	9.45E-04	4.09E-05	3.79E-09
1242.10	1.726	0.651	1.720				
313.28	1.770	0.693	1.748				
80.00	1.840	0.760	1.805				
19.96	1.904	0.821	1.872				
4.99	1.960	0.875	1.932				

Note:
k calculated using cv based on t_{90} values.
Specimen swelled under 10kPa
Specimen taken 12cm from the bottom of the tube

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.96	Unit Weight, kN/m^3	18.92
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	14.07
Area, cm^2	31.55	Specific Gravity, measured	2.69
Volume, cm^3	61.83	Solids Height, cm	1.045
Water Content, %	34.45	Volume of Solids, cm^3	32.99
Wet Mass, g	119.30	Volume of Voids, cm^3	28.85
Dry Mass, g	88.73		

Prepared By: LFG

Golder Associates

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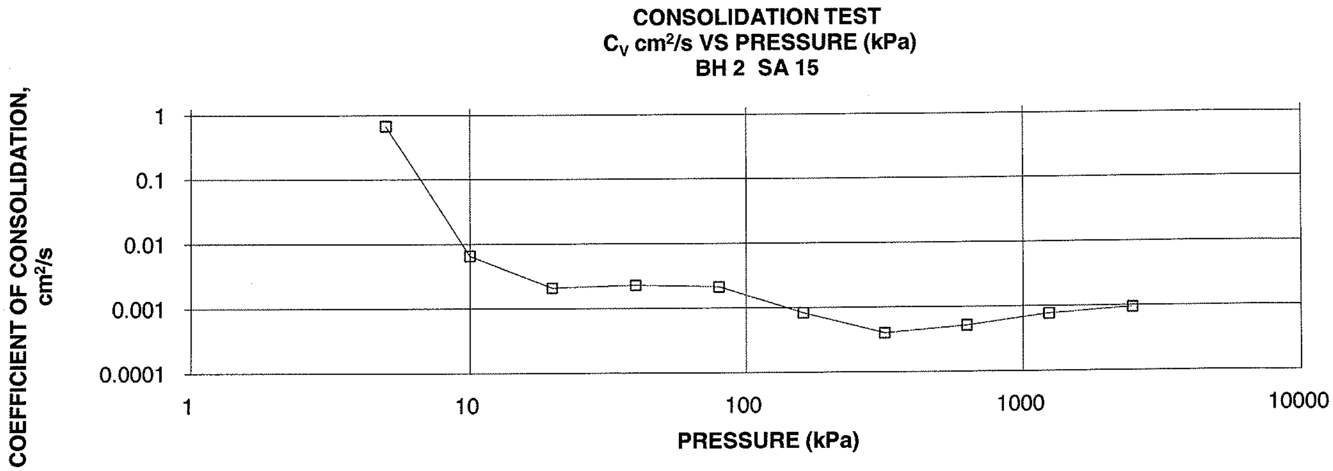
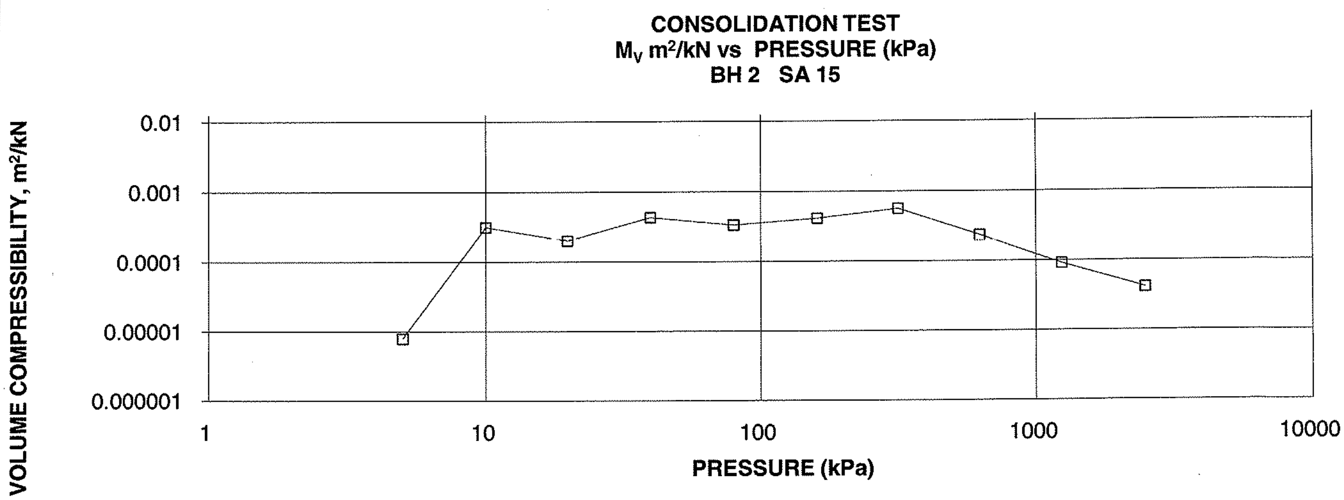
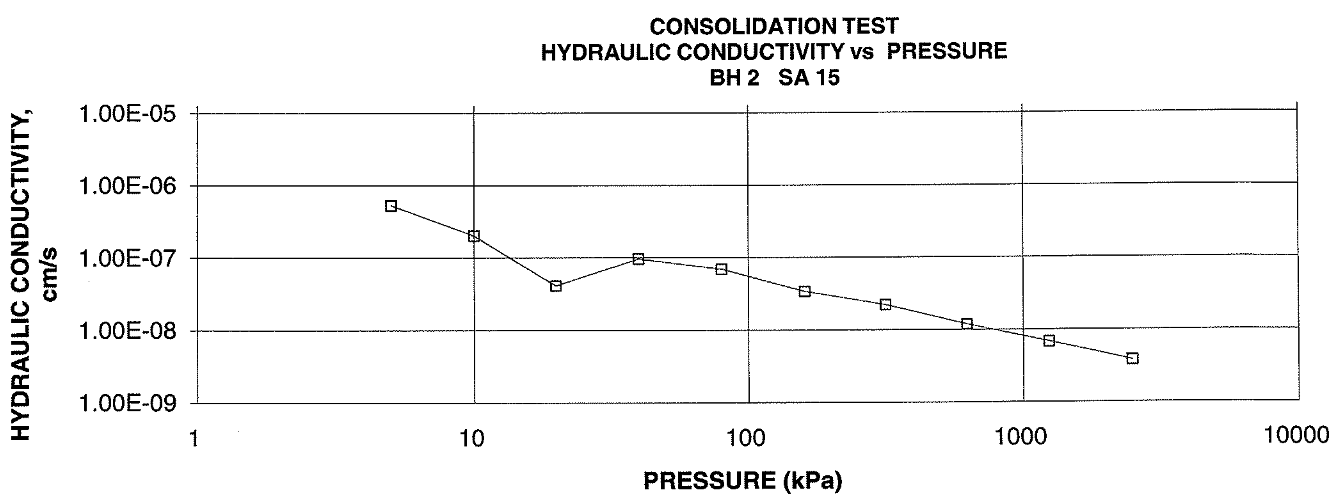
CONSOLIDATION TEST SUMMARY	FIGURE L-7b





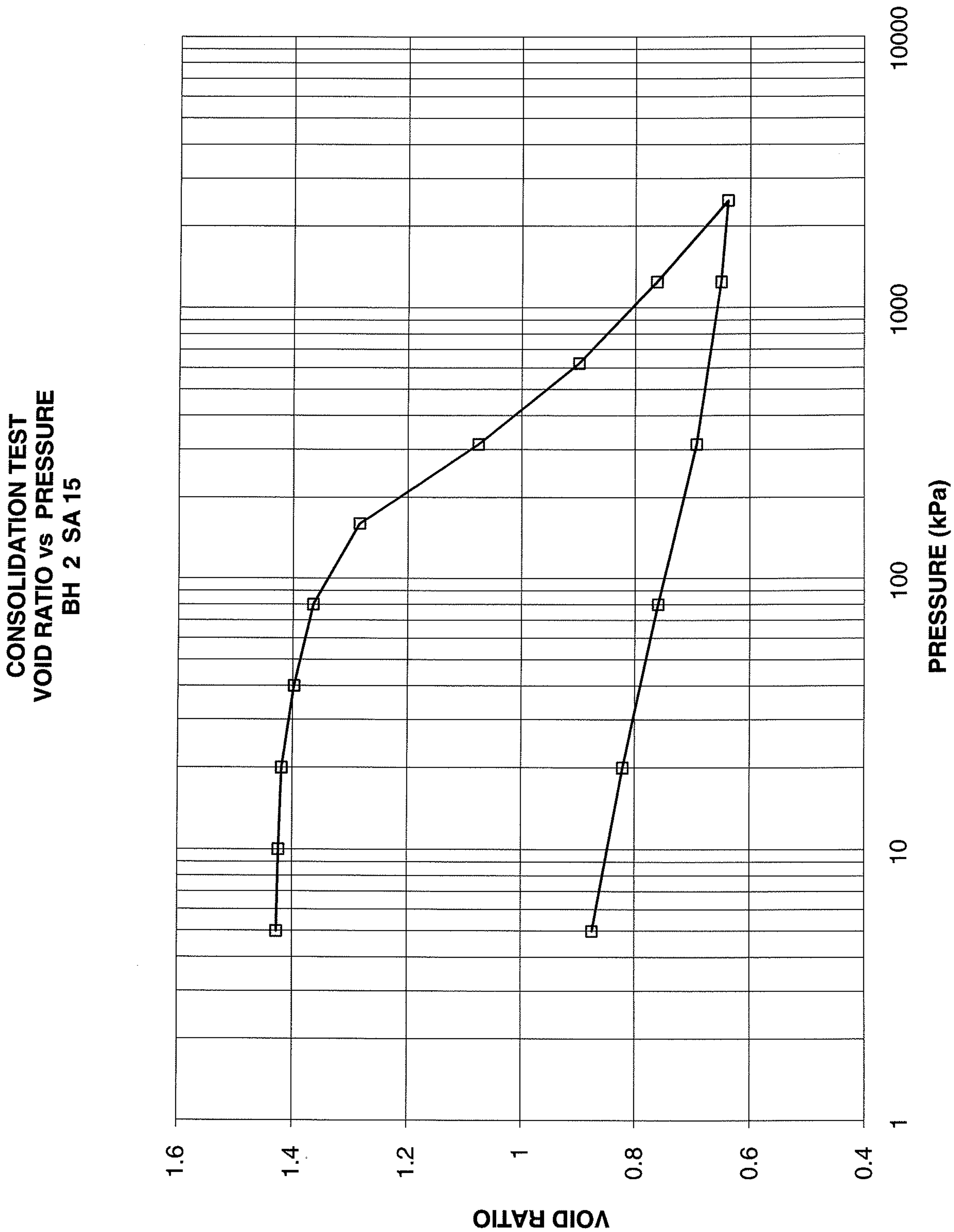


Project No. 11-1183-0039
Prepared By: LFG

Golder Associates

Checked By: Ul

CONSOLIDATION TEST VOID RATIO VS LOG PRESSURE	FIGURE L-7c



Project No. 11-1183-0039

Prepared By: LFG

Golder Associates

Checked By:

CONSOLIDATION TEST SUMMARY

FIGURE L-8a

SAMPLE IDENTIFICATION

Project Number	11-1183-0039	Sample Number	17
Borehole Number	2	Sample Depth, m	21.3

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	12		
Date Started	6/24/2011		
Date Completed	7/07/2011		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.55	Unit Weight, kN/m^3	16.37
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	10.31
Area, cm^2	31.58	Specific Gravity, measured	2.76
Volume, cm^3	80.46	Solids Height, cm	0.970
Water Content, %	58.86	Volume of Solids, cm^3	30.64
Wet Mass, g	134.33	Volume of Voids, cm^3	49.83
Dry Mass, g	84.56	Degree of Saturation, %	99.9

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	c_v cm^2/s	m_v m^2/kN	k cm/s
0.00	2.548	1.626	2.548				
5.01	2.549	1.627	2.548	1	1.38E+00	-7.05E-05	-9.51E-06
9.84	2.549	1.627	2.549	4	3.44E-01	2.44E-05	8.22E-07
20.52	2.541	1.619	2.545	305	4.50E-03	2.98E-04	1.31E-07
39.98	2.521	1.598	2.531	652	2.08E-03	3.97E-04	8.11E-08
80.00	2.485	1.562	2.503	1116	1.19E-03	3.48E-04	4.06E-08
156.44	2.389	1.462	2.437	1744	7.22E-04	4.94E-04	3.50E-08
314.93	2.097	1.161	2.243	2469	4.32E-04	7.24E-04	3.06E-08
624.36	1.932	0.991	2.014	1500	5.74E-04	2.09E-04	1.17E-08
1246.31	1.796	0.851	1.864	923	7.98E-04	8.61E-05	6.73E-09
2438.45	1.680	0.732	1.738	667	9.60E-04	3.81E-05	3.58E-09
1246.31	1.688	0.739	1.684				
314.93	1.739	0.792	1.713				
79.37	1.800	0.855	1.769				
20.52	1.849	0.906	1.824				
5.01	1.894	0.952	1.871				

Note:
k calculated using cv based on t_{90} values.
Specimen swelled under 10kPa
Specimen taken 10cm from bottom

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.89	Unit Weight, kN/m^3	18.77
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	13.87
Area, cm^2	31.58	Specific Gravity, measured	2.76
Volume, cm^3	59.80	Solids Height, cm	0.970
Water Content, %	35.35	Volume of Solids, cm^3	30.64
Wet Mass, g	114.45	Volume of Voids, cm^3	29.16
Dry Mass, g	84.56		

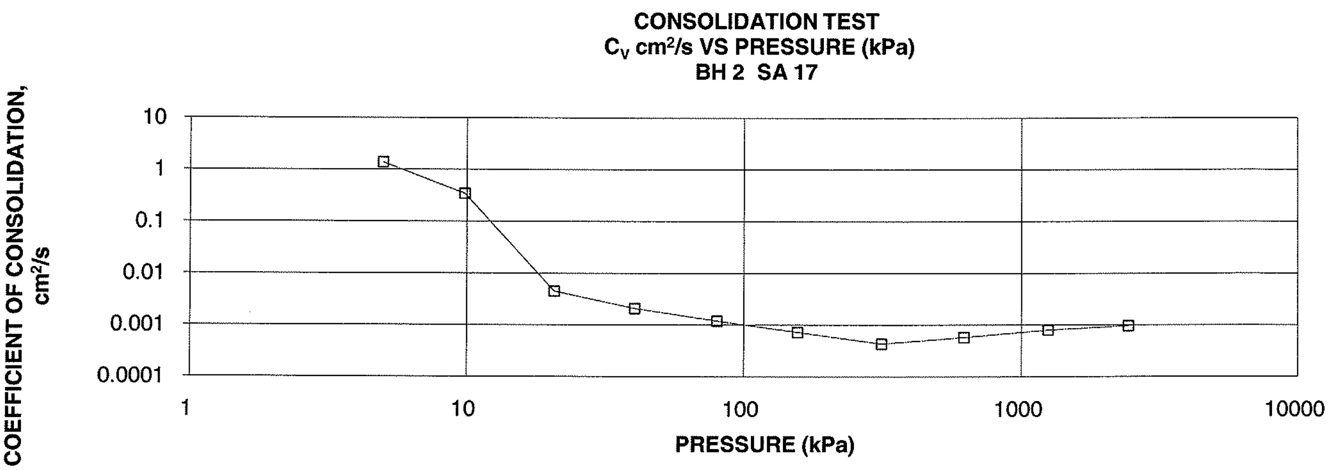
Prepared By: LFG

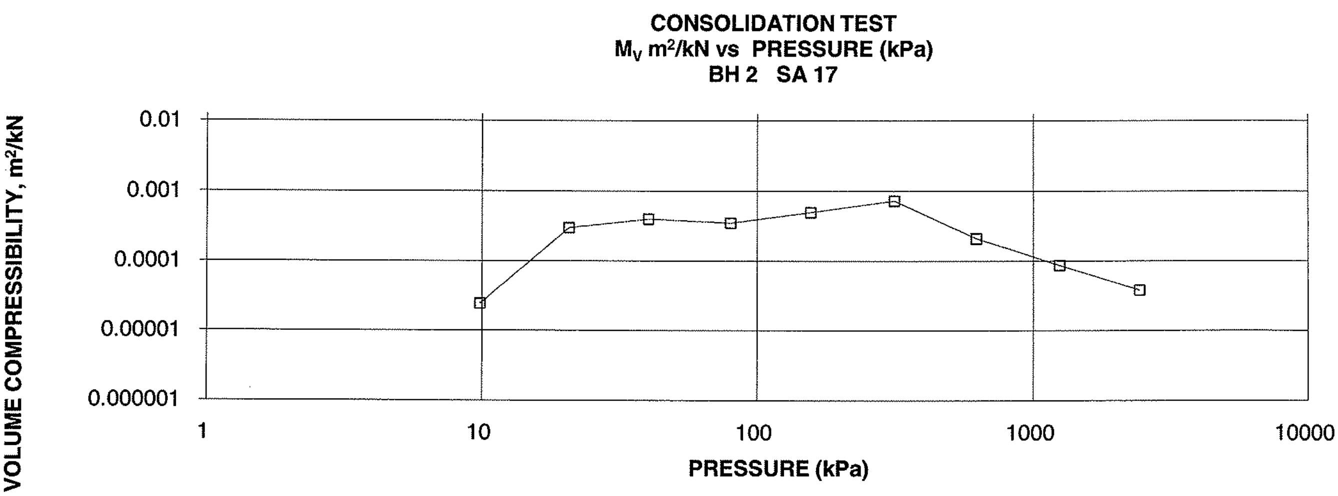
Golder Associates

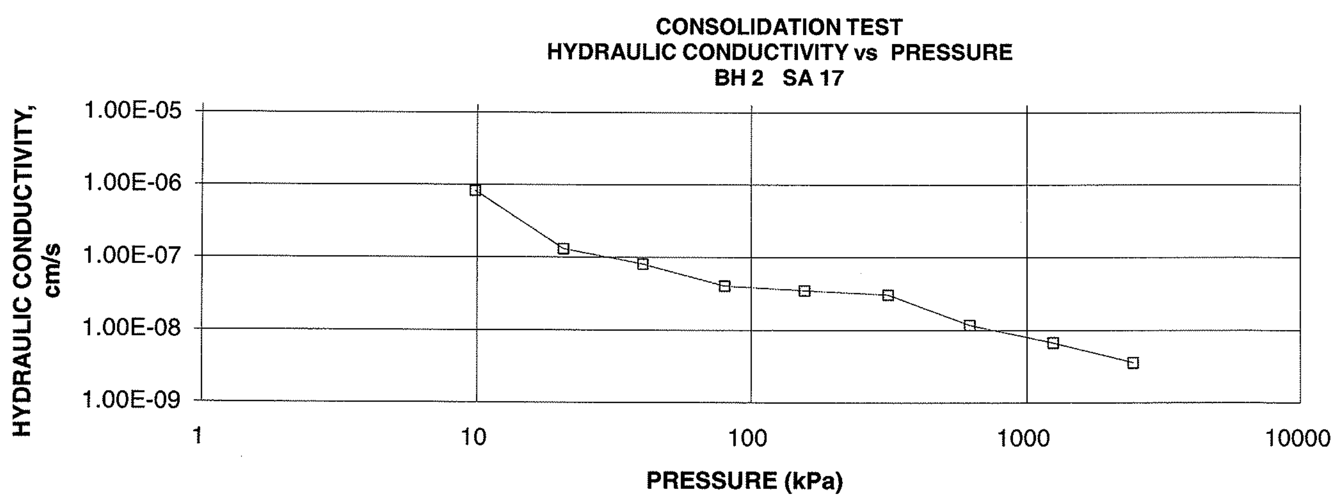
Checked By: [signature]

CONSOLIDATION TEST SUMMARY

FIGURE L-8b





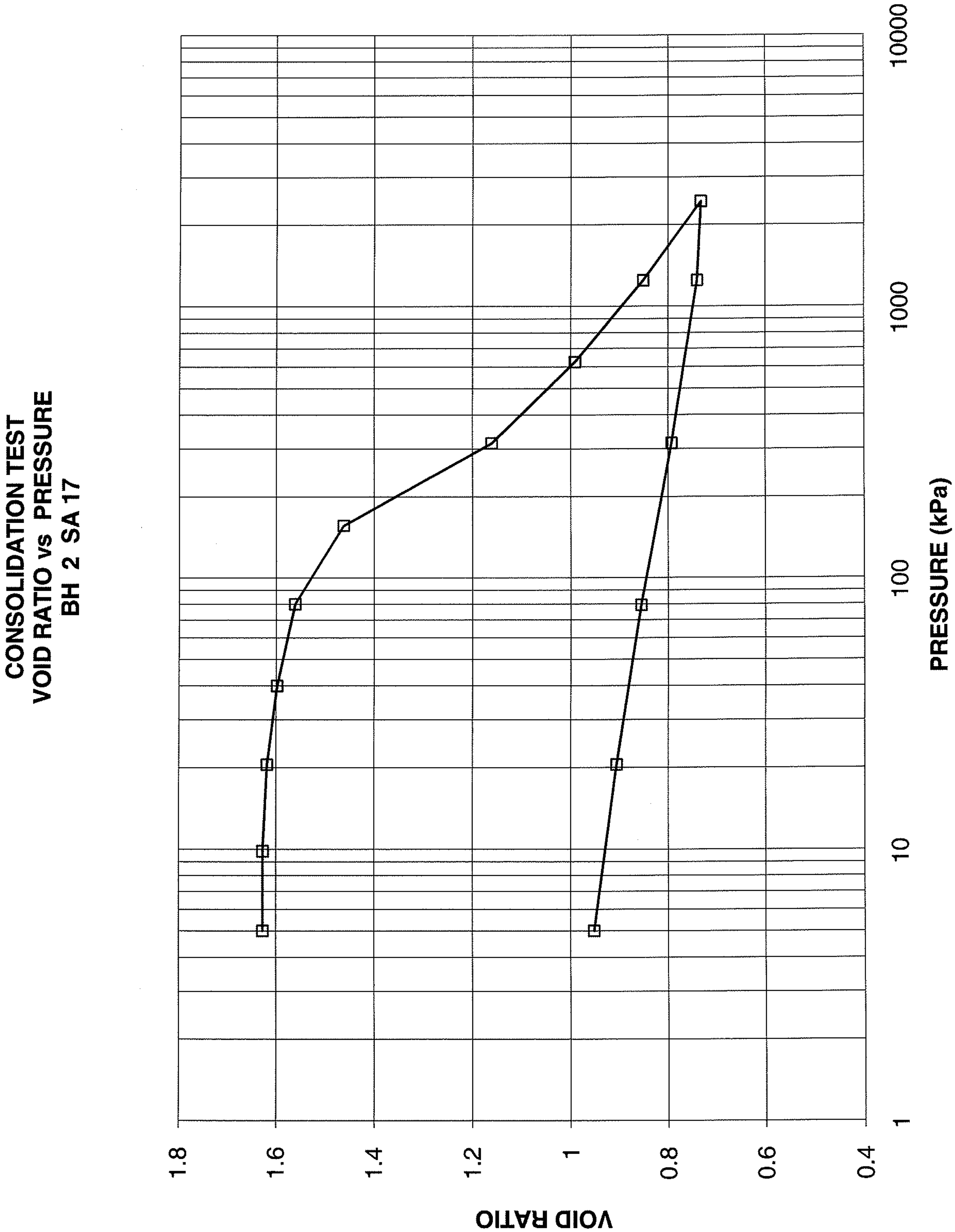


Project No. 11-1183-0039
Prepared By: LFG

Golder Associates

Checked By: [signature]

CONSOLIDATION TEST VOID RATIO VS LOG PRESSURE	FIGURE L-8c



Project No. 11-1183-0039

Prepared By: LFG

Golder Associates

Checked By: [signature]

CONSOLIDATION TEST SUMMARY

FIGURE L-9a

SAMPLE IDENTIFICATION

Project Number	11-1183-0039	Sample Number	16
Borehole Number	3	Sample Depth, m	18.3

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	9		
Date Started	6/24/2011		
Date Completed	7/05/2011		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	15.92
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	9.83
Area, cm^2	31.47	Specific Gravity, measured	2.77
Volume, cm^3	59.79	Solids Height, cm	0.688
Water Content, %	61.86	Volume of Solids, cm^3	21.65
Wet Mass, g	97.05	Volume of Voids, cm^3	38.15
Dry Mass, g	59.96	Degree of Saturation, %	97.2

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	c_v cm^2/s	m_v m^2/kN	k cm/s
0.00	1.900	1.762	1.900				
4.96	1.900	1.762	1.900	1	7.65E-01	0.00E+00	0.00E+00
9.94	1.903	1.767	1.902	3	2.56E-01	-3.17E-04	-7.94E-06
19.42	1.897	1.758	1.900	167	4.58E-03	3.33E-04	1.50E-07
40.00	1.888	1.745	1.893	239	3.18E-03	2.30E-04	7.17E-08
79.47	1.863	1.708	1.876	279	2.67E-03	3.33E-04	8.73E-08
160.63	1.757	1.554	1.810	1500	4.63E-04	6.87E-04	3.12E-08
312.72	1.553	1.258	1.655	2192	2.65E-04	7.06E-04	1.83E-08
628.31	1.401	1.037	1.477	1301	3.55E-04	2.53E-04	8.82E-09
1250.77	1.286	0.870	1.344	667	5.74E-04	9.72E-05	5.47E-09
2444.12	1.186	0.724	1.236	305	1.06E-03	4.41E-05	4.59E-09
1250.77	1.199	0.744	1.193				
312.72	1.241	0.805	1.220				
79.74	1.298	0.886	1.269				
21.11	1.341	0.950	1.319				
4.96	1.375	0.999	1.358				

Note:
k calculated using cv based on t_{90} values.
Specimen swelled under 10kPa
Specimen taken 12cm from the bottom of the tube

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.38	Unit Weight, kN/m^3	18.55
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	13.59
Area, cm^2	31.47	Specific Gravity, measured	2.77
Volume, cm^3	43.27	Solids Height, cm	0.688
Water Content, %	36.47	Volume of Solids, cm^3	21.65
Wet Mass, g	81.83	Volume of Voids, cm^3	21.63
Dry Mass, g	59.96		

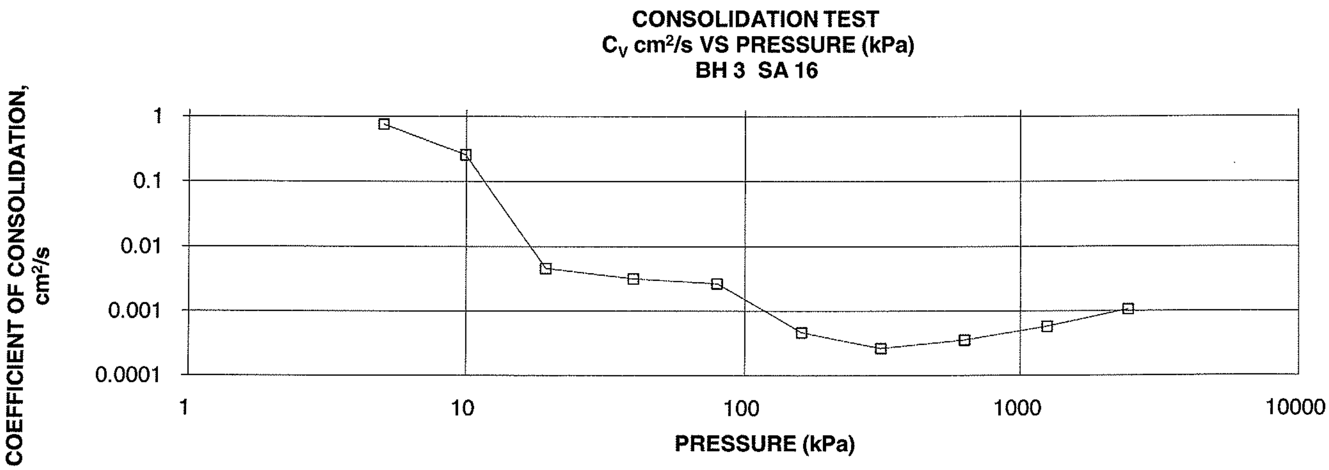
Prepared By: LFG

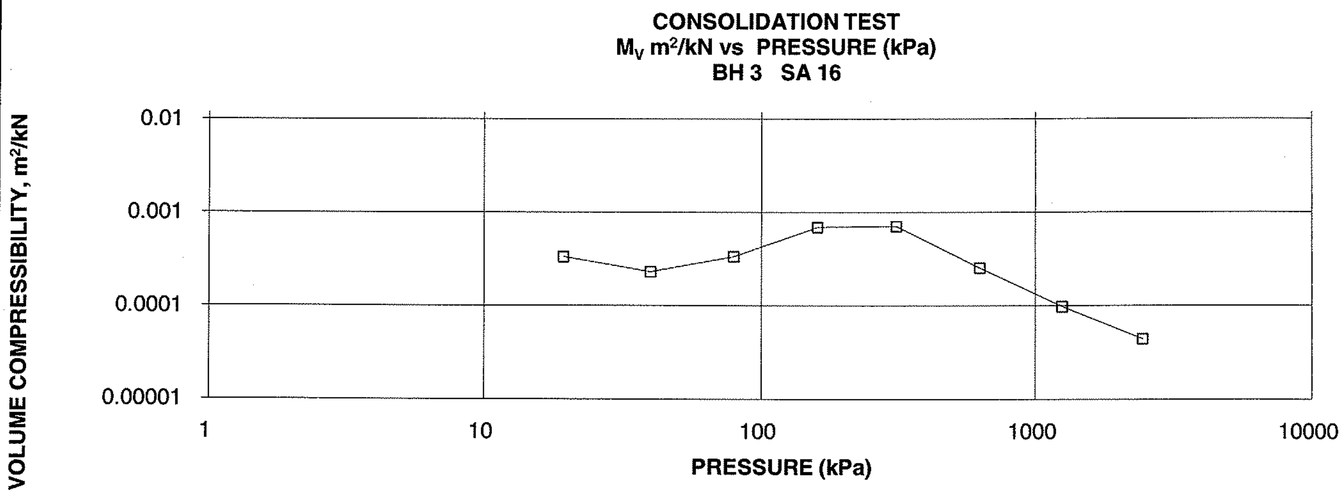
Golder Associates

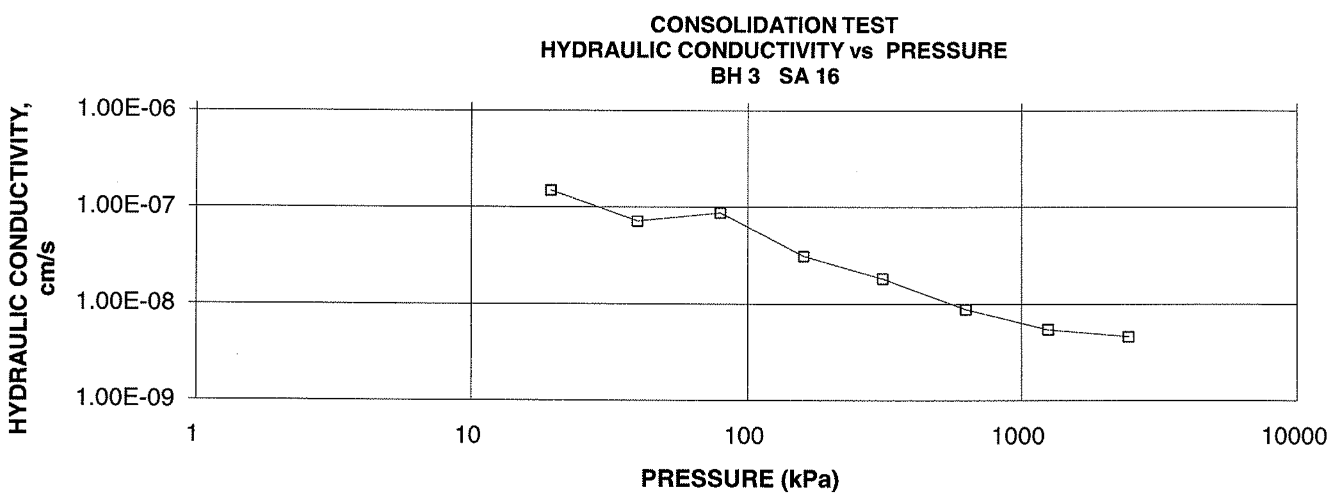
Checked By: [signature]

CONSOLIDATION TEST SUMMARY

FIGURE L-9b







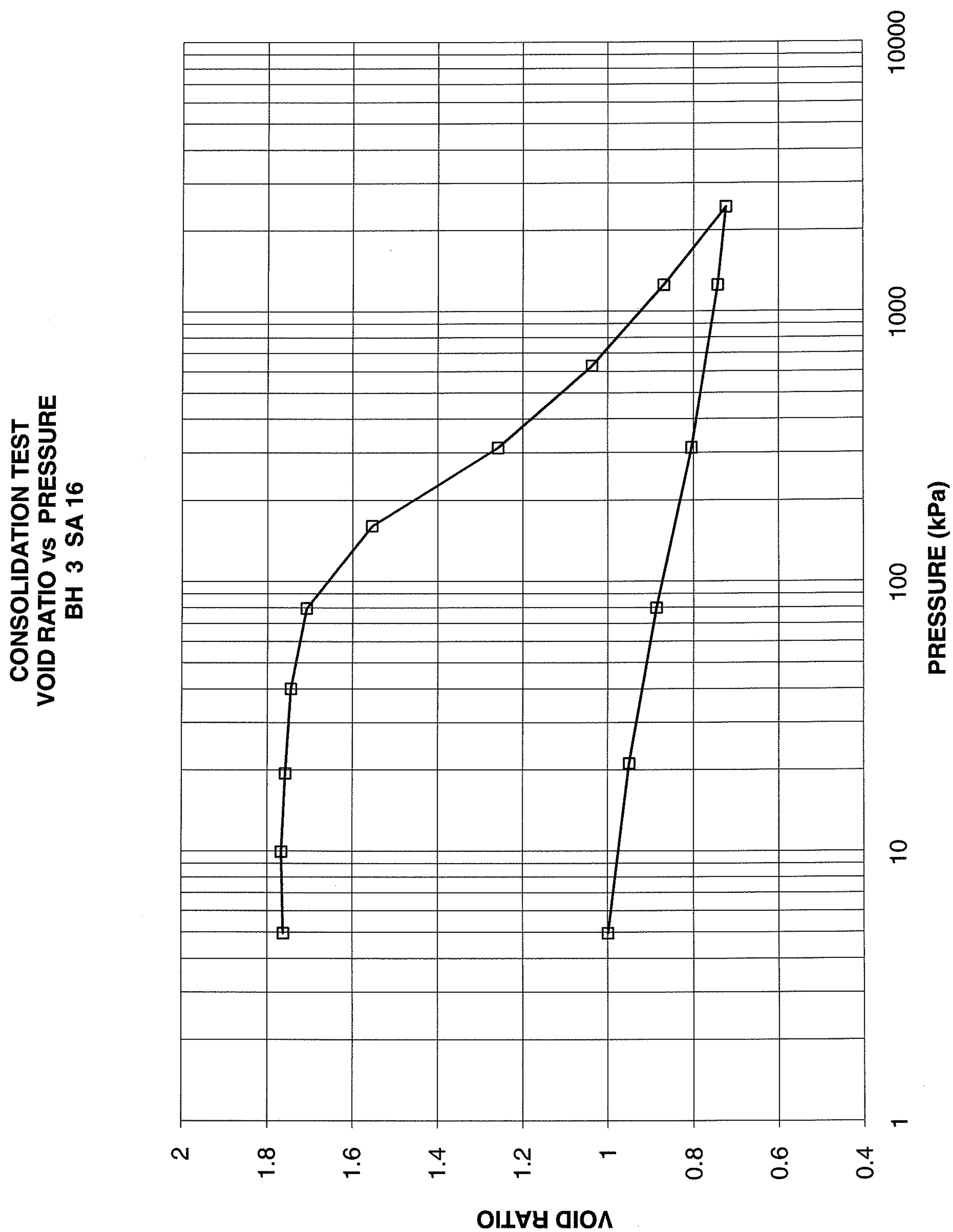
Project No. 11-1183-0039
Prepared By: LFG

Golder Associates

Checked By:

CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE

FIGURE L-9c



Project No. 11-1183-0039

Prepared By: LFG

Golder Associates

Checked By:

LVM | MERLEX

Laboratory Tests - Summary Sheet

Borehole No.	Sample No.	Depth	Particle Size Analysis				NMC	Atterberg Limits			SPT 'N'	USCS	Unit Weight (kN/m3)	Remarks
			Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)		LL (%)	PL (%)	IP (%)				
1	1	0.0					7.0				N/A			
	2	0.8		97.0	3.0		13.3				47	SP		
	3	1.5					15.4				41			
	4	2.3					20.2				22			
	5a	3.0		96.0	4.0		18.8				49/200mm	SP		
	5b	3.0					8.2				49/200mm			
2	1	0.8		96.0	4.0		15.8				85	SP	13.0	Field unit weight Estimate
	2	1.5		96.0	4.0		14.7				26	SP	14.6	Field unit weight Estimate
	3	2.3		95.0	5.0		16.9				16	SP	13.8	Field unit weight Estimate
	4	3.0		97.0	3.0		16.1				19	SP	14.6	Field unit weight Estimate
	5	3.8					5.9				89/175mm			
	6	4.6									N/A			NQ Core - 25% Recovery
	7	5.3					8.4				12			
	8	6.4									2			<1" Recovery
	9	7.6									N/A			NQ Core - 13% Recovery
	10	10.7					41.3				WH			
	11	12.2					40.3	53.3	20.2	33.1	WH	CH	17.9	Consolidation
	12	13.7					41.8	48.4	18.3	30.1	WH			
	13	15.2					55.2				WH			
	14	16.8					58.1	65.8	25.1	40.7	WH	CH		
	15	18.3					63.5	50.2	19.0	31.2	WH	CH	16.6	Consolidation
	16	19.8					55.4	52.4	21.0	31.4	WH	CH		
	17	21.3					60.6	53.3	21.4	31.9	WH	CH	16.4	Consolidation
	18	24.4					38.8	43.2	20.6	22.6	WH	CI		
3	1	0.6	2.0	93.0	5.0		13.1				81	SP	19.0	Field unit weight Estimate
	2	1.7	1.0	97.0	2.0		14.9				22	SP	11.5	Field unit weight Estimate
	3	2.3	1.0	96.0	3.0		13.8				33	SP	13.8	Field unit weight Estimate

Laboratory Tests - Summary Sheet

Borehole No.	Sample No.	Depth	Particle Size Analysis				NMC	Atterberg Limits			SPT 'N'	USCS	Unit Weight (kN/m3)	Remarks
			Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)		LL (%)	PL (%)	IP (%)				
	4	3.0					17.3				45		15.5	Field unit weight Estimate
	5a	3.8	0.0	96.0	4.0		18.2				33	SP	17.1	Field unit weight Estimate
	5b	3.8					4.3				33			
	6a	4.6					15.3				11			
	6b	4.6					9.1				11			
	7	5.3					18.2				7			
	8	6.1					21.6	32.2	22.5	9.7	8	CL		
	9a	7.6					24.3	27.8	20.2	7.6	6	CL		
	9b	7.6					29.7				6			
	10	9.1					34.9				4			
	11	10.7									WH			
	12	12.2					46.8	38.7	19.6	19.2	WH	CI		
	13	13.7					45.1				WH			
	14	15.2					59.9	60.8	22.4	38.4	WH	CH		
	15	16.8					60.8				WH			
	16	18.3					69.9	64.0	23.1	40.9	WH	CH	15.9	Consolidation
	17	19.8					62.6	63.5	24.7	38.8	WH	CH		
	18	21.3					29.8				WH			
4	1	0.0					4.9				N/A			
	2	0.8					8.9				50/125mm			
	3	1.5		97.0	3.0		12.9				44	SP		
	4	2.3					13.6				29			
	5	3.0					18.6				39			
	6	3.8		96.0	4.0		16.6				58	SP		

Appendix D Boreholes by Others

Borehole Logs by Others

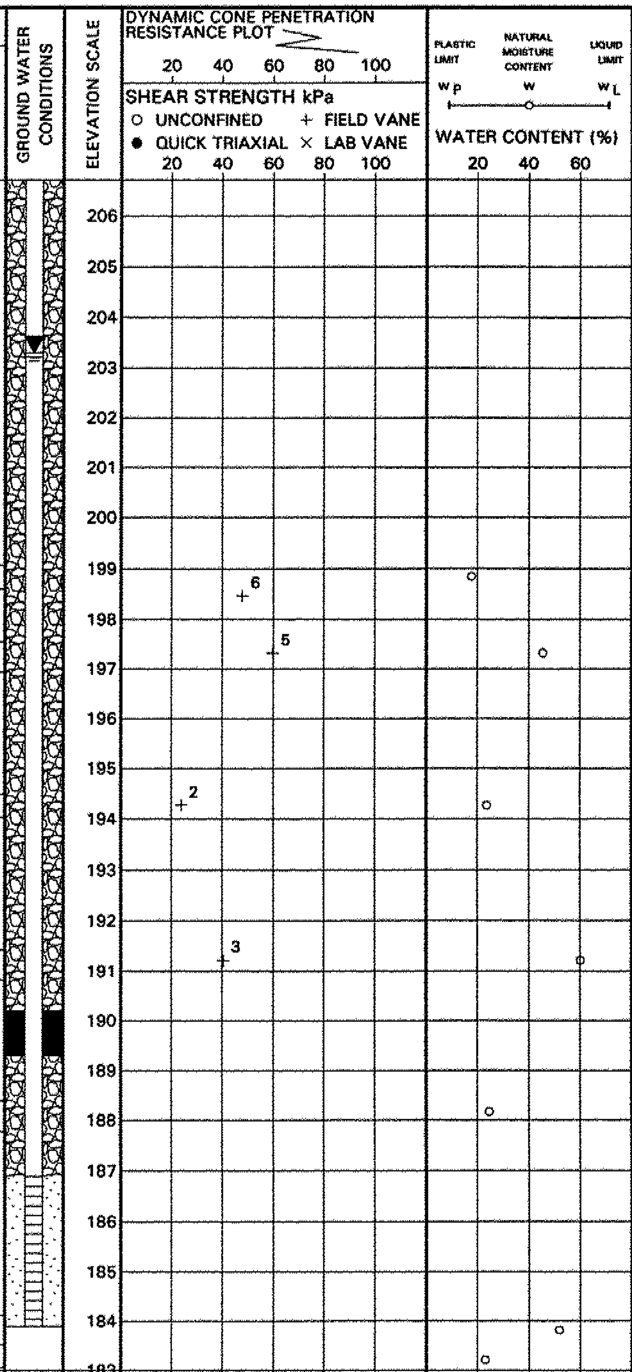
LVM | MERLEX

RECORD OF BOREHOLE No D1

1 OF 1 **METRIC**

W.P. 128-88-01	LOCATION Station 18+816.3, 1.9 m left of centreline		ORIGINATED BY S.C.
DIST 54 HWY 535	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY B.B.
DATUM Geodetic	DATE 11.01.95 & 11.02.95		CHECKED BY K.P.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
206.7	Ground Surface				
0.0	~ 40 mm of asphalt over; Fill: sand and coarse gravel				206, 205
204.0 / 2.7	Fill: rock fill with sand and gravel, rock sizes at least to 0.3 m				204, 203, 202, 201, 200
199.9 / 6.8	Clayey Silt: soft, with organics	1	SS	5	199
198.8 / 8.0	Silty Clay to Clay: soft to stiff, grey, laminated, fissured	2	TW	PM	198, 197, 196, 195
	sand and gravel layer	3	SS	1	194
193.3 / 13.4	Silty Clay: contains sand seams and gravel sizes, soft to firm, laminated, fissured	4	TW	PM	193, 192, 191, 190, 189
		5	SS	1	188, 187, 186, 185
		6	TW	PM	184
183.0		7	SS	1	183
23.7	End of Borehole Stabilized water level measured one day after completion of drilling. Standpipe slotted from 22.9 m to 19.8 m, sealed from 17.4 m to 16.5 m.				



UNIT WEIGHT γ kN/m³

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

+³, ×³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No D2

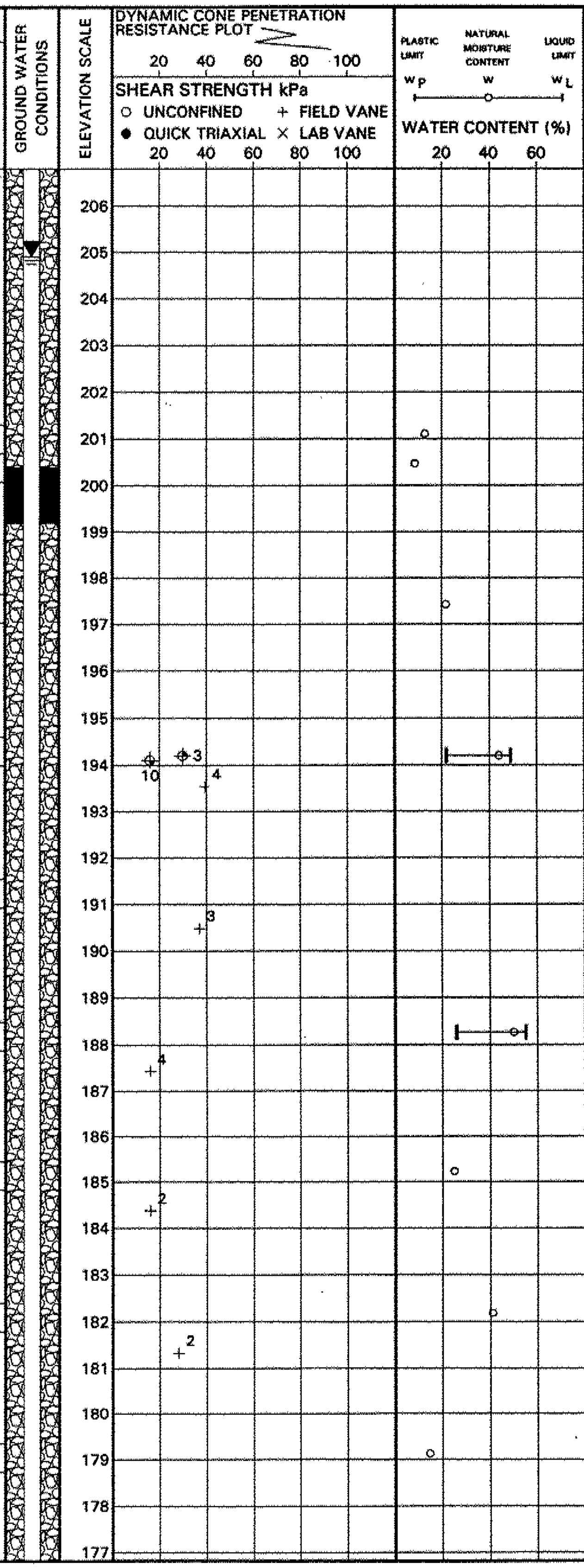
METRIC

W.P. 128-88-01 LOCATION Station 18+838.7, 1.6 m right of centreline ORIGINATED BY S.C.

DIST 54 HWY 535 BOREHOLE TYPE Hollow Stem Augers COMPILED BY B.B.

DATUM Geodetic DATE 11.03.95 & 11.04.95 CHECKED BY K.P.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
206.8	Ground Surface								
0.0	Fill: mostly sand and gravel, compact, wet below 5 m, some organics at 6 m, some blast rock should be anticipated	1	SS	5					
		2	SS	18					
198.9 / 8.0	Silt: clayey, with some sand, loose, grey, wet	3	SS	5					
		4	TW	PM	14.5				
194.0 / 12.8	Silty Clay: becoming clayey silt or silt at depth, soft to firm, fissured, laminated, grey	5	SS	1					
		6	TW	PM	15.7	0	0	27	73
		7	SS	0					
		8	TW	PM	17.3				
		9	SS	16					
178.8 / 28.0	Silt: changing to fine sand, some boulders near base of this horizon, compact to very dense								



Continued Next Page

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No D2

METRIC

W.P. 128-88-01 | LOCATION Station 18+838.7, 1.6 m right of centreline | ORIGINATED BY S.C.

DIST 54 HWY 535 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY B.B.

DATUM Geodetic | DATE 11.03.95 & 11.04.95 | CHECKED BY K.P.

ELEV DEPTH	DESCRIPTION (Continued)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silt: changing to fine sand, some boulders near base of this horizon, compact to very dense	10	SS	27	176		0 0 92 8
		11	SS	20	173		
		12	SS	26	170		
		13	SS	78	167		
164.9 / 41.9	Boulders and broken rock, poor recovery, some sand						
162.0 / 44.8	Bedrock: Biotite Gneiss, vertically jointed, highly broken (see attached core log for detailed description)						
158.2 / 48.6	End of Borehole						

Water rose (artesian) to 0.5 m above ground level when drilling at 30.5 m depth

Stabilized water level measured one day after completion of drilling. Standpipe slotted from 43.0 m to 40.0 m, sealed from 7.6 m to 6.4 m.

Continued Next Page

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

(%) STRAIN AT FAILURE

Shaheen & Peaker 1995

CORE LOG OF BOREHOLE D2

S & P

Project : WP 128-88-01
Location : Station 18+838.7, 1.6 m right of centreline
Started : November 4, 1995
Completed : November 5, 1995

Inclination: Vertical Azimuth: Vertical Datum: Geodetic

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN; F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED; SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR; FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION
28				178.80 28.00				
41–42		Refer to Page 1 of Borehole D2 for soil description		164.90 41.90				
42	CORE	Boulders and Broken Rock: argillite and gneiss			1	0.07	grey 100	
44	CORE				2	0.12	100	
		Clayey Silt Till layer at 44.6 m		161.98 44.82				
45	NQ Core Barrel, CORE	**Biotite Gneiss:** medium grained, biotite-quartz-feldspar, brown to grey, moderately to slightly weathered, friable in upper levels, medium to high strength	+ + +		3	0.05	grey-rd.brown 100	3 sets of joints, flat, dipping and vertical joint surfaces are rough planar to rough undulating sandy along joints in Run 3 where gneiss is friable occasional healed joints
46	CORE				4	0.03	100	
47	CORE				5	0.07	milky-brown 100	
48	CORE				6	0.11	100	
		End of Borehole		158.18 48.62	7	0.04	100	

RECOVERY: TOTAL CORE %, SOLID CORE %; R.Q.D. %; FRACT. INDEX PER .3 m; DIP wrt Core Axis; HYDRAULIC CONDUCTIVITY k, cm/sec; DIAMETRAL POINT LOAD INDEX (MPa); NOTES WATER LEVELS INSTRUMENTATION

Groundwater Elevations

Shallow/Single Installation — Water Level (date)

Deep/Dual Installation — Water Level (date)

Logged : H.Lohse
Checked : H.Lohse

Ministry of Transportation Ontario

Shaheen & Peaker 1995

Foundation Design

RECORD OF BOREHOLE No D3

METRIC

W.P. 128-88-01 | LOCATION Station 18+855.5, 5.5 m left of centerline | ORIGINATED BY R.O.

DIST 54 HWY 535 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY B.B.

DATUM Geodetic | DATE 10.06.95 & 10.11.95 | CHECKED BY K.P.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
202.5	Ground Surface
0.0	Fill: clay, silt, some sand
201.0 / 1.5	Silty Clay to Clay: soft to firm, grey, laminated structure, fissured, with gravel sizes
190.0 / 12.5	Silty Clay: with sand and gravel seams
186.7 / 15.8	Silty Clay to Clay: soft to firm, grey, laminated, fissured, gravel sizes Silt and Sand rose in casing
178.0 / 24.5	Silt: layered with sand, some clay, compact, grey
172.5	

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
1	SS	2					
2	SS	1					
3	TW	PM	16.4	0	5	41	54
4	SS	0					
5	TW	PM	15.0	0	5	41	54
6	SS	1	14.5				
7	TW	PM	15.3				
8	SS	1	14.5				
9	TW	PM		0	4	23	73
10	SS	1	15.7				
11	TW	PM	16.2	0	2	43	55
12	SS	0					
13	SS	6					
14	SS	20		0	2	81	17

GROUND WATER CONDITIONS | ELEVATION SCALE

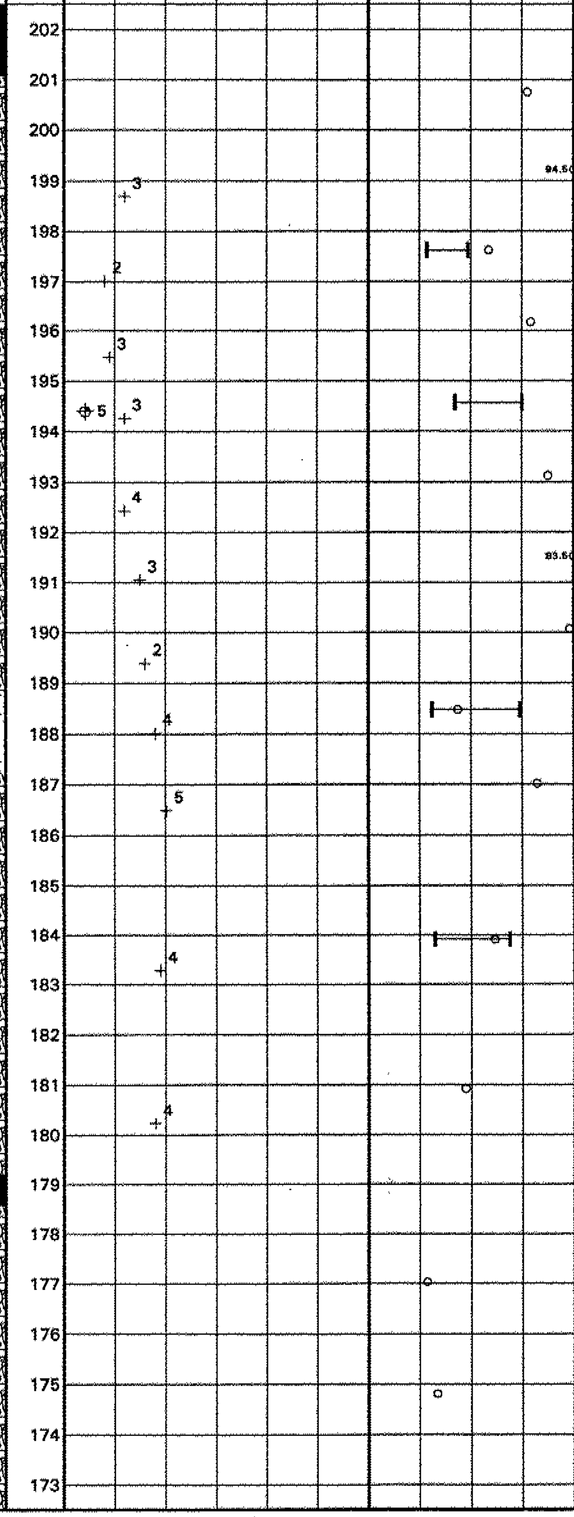
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) 20 40 60

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



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

15 ⌖ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Shaheen & Peaker 1995

Foundation Design

RECORD OF BOREHOLE No D3

METRIC

W.P. 128-88-01	LOCATION Station 18+855.5, 5.5 m left of centerline	ORIGINATED BY R.O.
DIST 54 HWY 535	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY B.B.
DATUM Geodetic	DATE 10.06.95 & 10.11.95	CHECKED BY K.P.

ELEV DEPTH	DESCRIPTION (Continued)	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
30.0	Silty Sand to Sandy Silt: fine, compact to dense	15	SS	25	172, 171	
		16	SS	34	170, 169, 168	
167.5 / 35.0	Cobbles, Boulders and Sand: possible broken bedrock	17	SS	31	167, 166	
165.0 / 37.5	Bedrock: Biotite Gneiss (see attached core log for detailed description)	18	SS	60/0.1	165, 164, 163, 162	
161.7 / 40.8	End of Borehole					

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

End of Borehole

Stabilized water levels measured in #1 and #2 seven days after completion of drilling. Standpipe #1 slotted from 40.8 m to 39.3 m, sealed from 39.2 m to 37.0 m. Standpipe #2 slotted from 33.5 m to 30.5 m, sealed from 23.9 m to 23.3 m, artesian water level at +1.5 m. Standpipe #3 slotted from 15.3 m to 13.7 m, sealed from 1.4 m to surface, water level 10.3 m below grade upon completion.

Water Level Records

Piez No./Tip Depth/Water Level

1/40.8 m/+1.4 m

2/33.5 m/+1.5 m

3/15.2 m/-10.3 m on completion

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity   15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Shaheen & Peaker 1995

CORE LOG OF BOREHOLE D3

S & P

Project : WP 128-88-01
Location : Station 18+855.5, 5.5 m left of centerline
Started : October 11, 1995
Completed : October 11, 1995

Inclination: Vertical Azimuth: Vertical

Sheet 3 of 3
Datum: Geodetic

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION
30			172.50 30.00				
35		Refer to page 2 of Borehole D3 for soil description	167.50 35.00				
37.55	NQ Core Barrel, CORE	**Biotite Gneiss:** medium grained, biotite-quartz-feldspar, grey with occasional brown seams, unweathered, high to very high strength	164.95 37.55	1	0.12	grey 100	3 sets of joints, flat, dipping and vertical
	CORE			2	0.1	grey 100	surfaces are rough planar
	CORE			3	0.1	grey 100	
40.84		End of Borehole	161.66 40.84				

Groundwater Elevations

Shallow/Single Installation — Water Level (date)

Deep/Dual Installation — Water Level (date)

Logged : H. Lohse
Checked : H. Lohse

CORELOG C1100 03/08/96

RECORD OF BOREHOLE No D4

METRIC

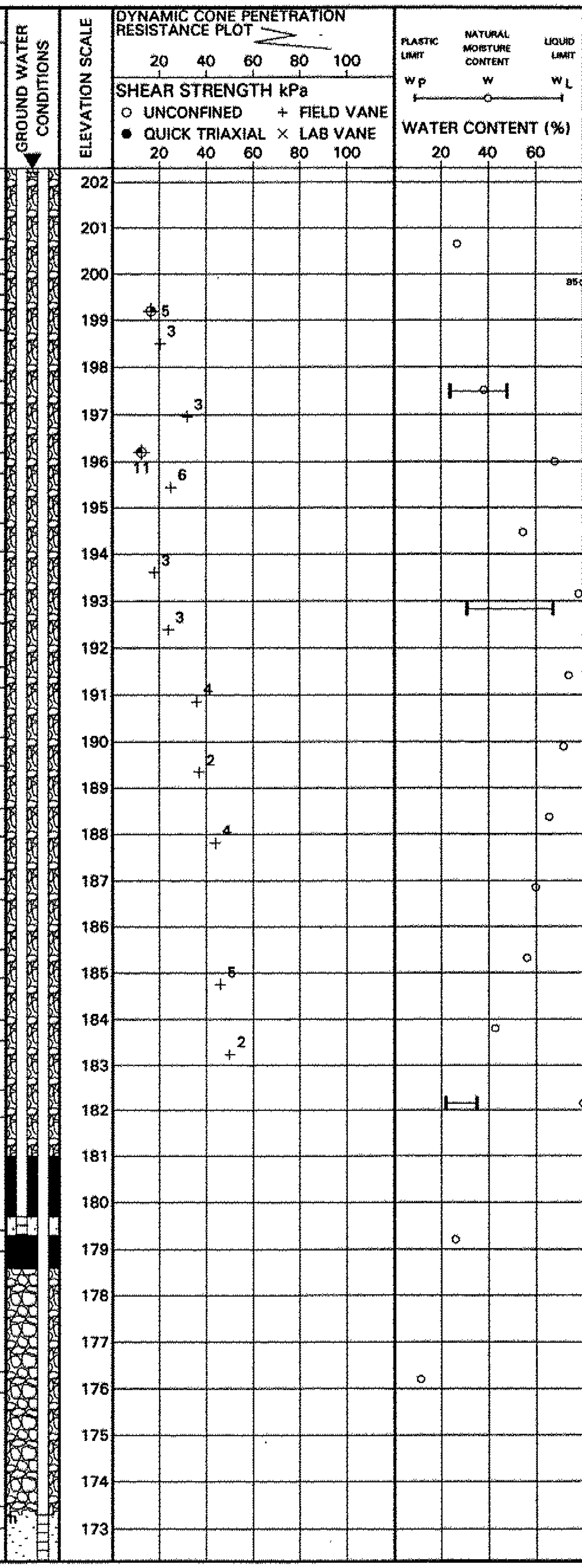
W.P. 128-88-01	LOCATION Station 18+883.5, 6.4 m right of centrline	ORIGINATED BY R.O.
DIST 54 HWY 535	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY B.B.
DATUM Geodetic	DATE 10.04.95 & 10.05.95	CHECKED BY K.P.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
202.3	Ground Surface
0.0	Fill: sand and blast rock
200.8 / 1.5	Fill: sand and organics
199.3 / 3.0	Silty Clay to Clay: soft to firm, grey, laminated, fissured, gravel sizes, trace sand
187.3 / 15.0	Gravel and Cobbles: some sand and clay
186.3 / 16.0	Silty Clay to Silt: soft to firm, grey, some gravel sizes, probable seams of sand and gravel
182.1 / 20.2	Silt, Sand, Gravel and Clay: artesian conditions
175.8 / 26.6	Cobbles, Boulders and Gravel: broken bedrock?
173.5 / 28.8	Bedrock: Biotite Gneiss

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
1	SS	3					
2	SS	1	14.7				
3	TW	PM	14.5				
4	SS	1		0	2	33	66
5	TW	PM	14.3				
6	SS	1					
7	TW	PM	14.6	0	2	47	52
8	SS	1	14.6				
9	TW	PM	15.0				
10	TW	PM	15.3				
11	SS	2	15.3				
12	TW	PM	14.5				
13	SS	8					
14	TW	PM	17.2	0	1	64	35
15	SS	10		0	4	84	13
16	SS	59					
17	SS	60/ 0.0					



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+³, ×³: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No D4

METRIC

W.P. 128-88-01	LOCATION Station 18+883.5, 6.4 m right of centrline		ORIGINATED BY R.O.
DIST 54	HWY 535	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY B.B.
DATUM Geodetic	DATE 10.04.95 & 10.05.95		CHECKED BY K.P.

ELEV DEPTH	SOIL PROFILE: DESCRIPTION (Continued)	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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Bedrock: Biotite Gneiss (see attached core log for detailed description)						172				
170.3							171				
32.0	End of Borehole Stabilized water levels measured 5 days after completion of drilling. Standpipe #1 slotted from 32.0 m to 29.0 m, sealed from 23.7 m to 23.0 m, artesian water level at +1.8 m. Standpipe #2 slotted from 23.0 m to 22.6 m, sealed from 22.6 m to 21.3 m, artesian water level +1.8										

Continued Next Page

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

CORE LOG OF BOREHOLE D4

S & P

Project : WP 128-88-01
Location : Station 18+883.5, 6.4 m right of centrline
Started : October 4, 1995
Completed : October 5, 1995

Sheet 3 of 3

Inclination: Vertical Azimuth: Vertical Datum: Geodetic

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER .3 m	DISCONTINUITY DATA DIP wrt Core Axis	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
28		**Refer to Page 1 of Borehole D4 for soil description**		173.50 28.80												
30	NQ Core Barrel; CORE	**Biotite Gneiss:** medium grained, biotite-quartz-feldspar, banded, grey to brown, occasional pegmatite layers consisting of quartz and feldspar, unweathered, high strength to very high strength			1	0.14	grey 100						3 sets of joints, flat, dipping and vertical some joints are partly healed joint surfaces are rough planar			
	CORE				2	0.07	grey 100									
	CORE			170.32	3	0.17	grey 100									
32		End of Borehole		31.98												

Depth scale continues: 34, 36, 38, 40, 42, 44, 46, 48, 50, 52

Groundwater Elevations

Shallow/Single Installation — Water Level (date)

Deep/Dual Installation — Water Level (date)

Logged : H.Lohse
Checked : H.Lohse

RECORD OF BOREHOLE No D5&D5A

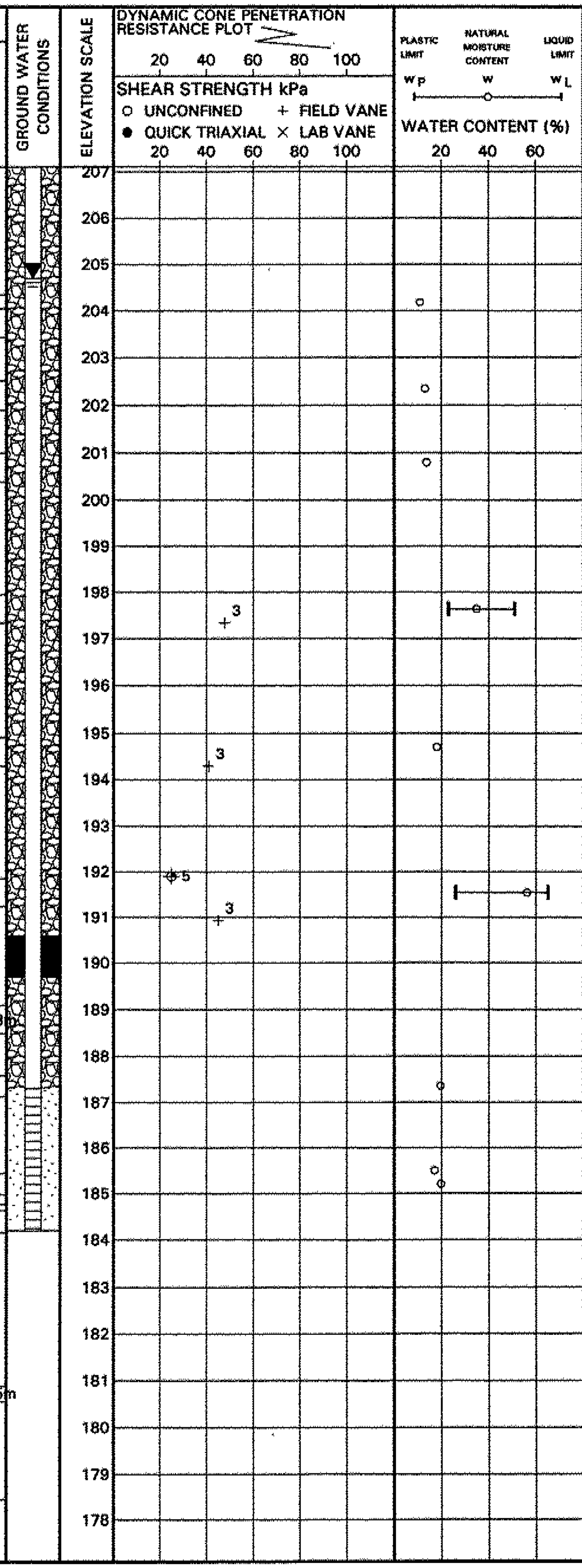
METRIC

W.P. 128-88-01 LOCATION Station 18+898.8, 1.9 m left of centreline ORIGINATED BY R.O.

DIST 54 HWY 535 BOREHOLE TYPE Hollow Stem Augers COMPILED BY B.B.

DATUM Geodetic DATE 10.30.95 & 11.06.95 CHECKED BY K.P.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	UNIT WEIGHT γ kN/m³
207.1	Ground Surface					
0.0	~40 mm asphalt over; Fill: sand and gravel over rock fill with some sand and gravel	1	AS			
		2	SS	22		
202.1		3	SS	32		
5.0	Sand: fine with silt, some organics	4	SS	18		
199.2						
8.0	Clay: silty, soft to firm, stratified, fissured	5	TW	PM	0 0 35 65	
	sand seams and layers	6	SS	1		
	Clay: silty, soft to firm, stratified, fissured	7	TW	PM		15.6
	layer of boulders at least 0.3 m diameter	8	SS	50/0.03m		
	End of Borehole 5, moved 4 m east	9	SS	22		
185.6						
21.5	Boulders, Sand, Gravel	10	SS	26		
			CORE			
			CORE			
		11	SS	90/0.25m		
179.7						
27.4	Bedrock: Biotite Gneiss (see attached core log for detailed description)		CORE			



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+³, ×³: Numbers refer to Sensitivity

15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No D5&D5A

METRIC

W.P. 128-88-01 | LOCATION Station 18+898.8, 1.9 m left of centreline | ORIGINATED BY R.O.

DIST 54 HWY 535 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY B.B.

DATUM Geodetic | DATE 10.30.95 & 11.06.95 | CHECKED BY K.P.

ELEV DEPTH	DESCRIPTION (Continued)	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Bedrock: Biotite Gneiss	+ + +		CORE			177				
175.5							176				
31.6	End of Borehole 5A Stabilized water levels measured upon completion of drilling. Standpipe slotted from 22.9 m to 19.8 m, sealed from 17.4 m to 16.5 m.										

Continued Next Page

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

15-⌖-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Shaheen & Peaker 1995

CORE LOG OF BOREHOLE D5A

S & P

Project : WP 128-88-01
Location : Station 18+898.8, 1.9 m left of centreline
Started : October 31, 1995
Completed : November 6, 1995

Sheet 3 of 3

Inclination: Vertical Azimuth: Vertical Datum: Geodetic

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED

Depth Scale (metres)	Drilling Record	Description	Elev. / Depth (m)	Run No.	Penetration Rate (m/min)	Flush Colour / % Return	Type and Surface Description (Discontinuity Data)
21.50–27.40	NQ Core Barrel / CORE	Refer to page 1 of Borehole D5&D5A for soil description	185.60 / 21.50	1, 2			
27.40–31.62	NQ Core Barrel / CORE	**Biotite Gneiss:** medium grained, biotite-quartz-feldspar, brown to grey, unweathered, high to very high strength	179.70 / 27.40	3	0.11	beige 100	3 set of joints, flat, dipping and vertical join surfaces are rough planar some healed joints and partial 2 mm wide open joints in Run 4
	NQ Core Barrel / CORE			4	0.48	beige 100	
31.62		End of Borehole	175.48 / 31.62				

Depth scale: 22, 24, 26, 28, 30, 32, 34, 36, 38, 40

Groundwater Elevations

Shallow/Single Installation — Water Level (date)

Deep/Dual Installation — Water Level (date)

Logged : H.Lohse

Checked : H.Lohse

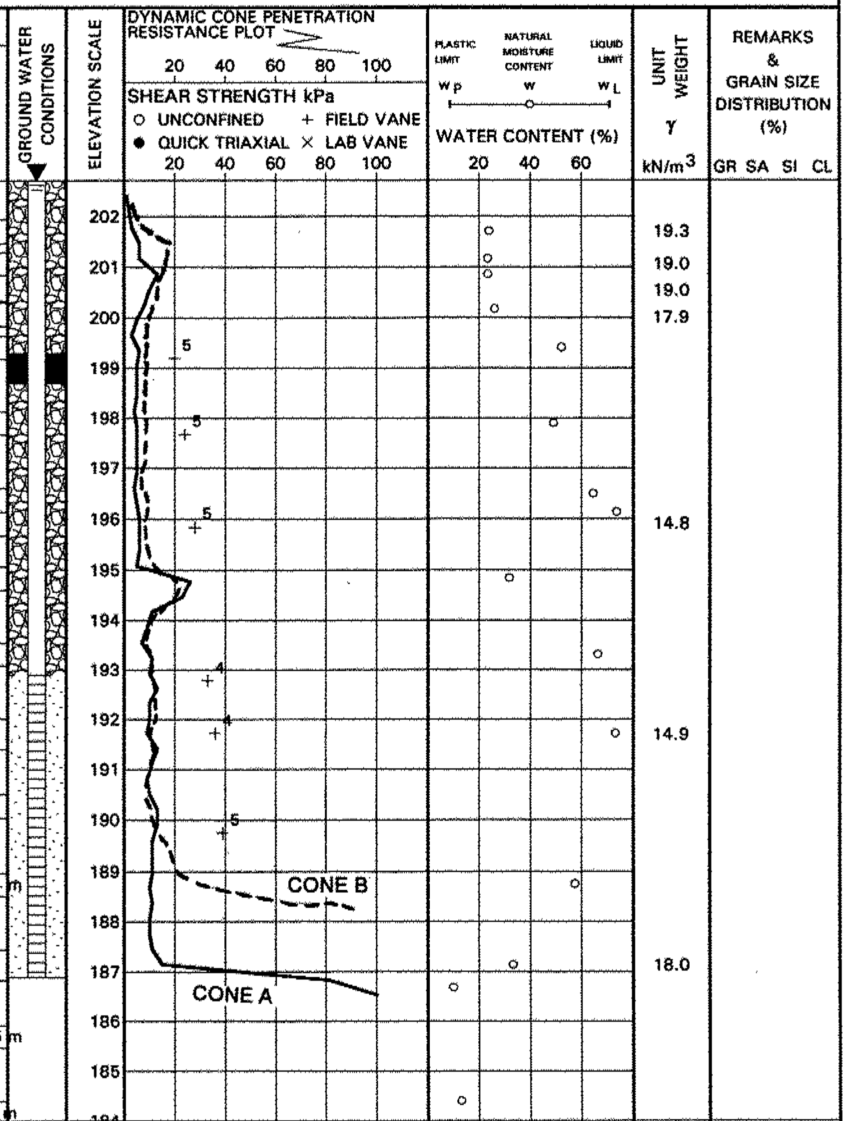
CORELOG C1100 03/09/96

RECORD OF BOREHOLE No D6

METRIC

W.P. 128-88-01	LOCATION Station 18+905.0, 20.3 m left of centrline		ORIGINATED BY R.O.
DIST 54	HWY 535	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY B.B.
DATUM Geodetic		DATE 10.02.95 & 10.03.95	CHECKED BY K.P.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³
202.7	Ground Surface				
0.0	Clay to Silty Clay: sand layers, organics, brown/grey, stiff	1	SS	3	19.3
		2	SS	6	19.0
		2A	TW	PM	19.0
199.7		3	SS	10	17.9
3.0	Silty Clay to Clay: soft to firm with some gravel sizes, fissured, laminated structure	4	SS	2	
		5	SS	1	
		6	SS	1	
195.2		6A	TW	PM	14.8
7.5	Silty Clay: silt, sand and gravel layers, also with cobbles and boulders, soft or loose	7	SS	27	
		8	SS	1	
		9	TW	PM	14.9
		10	SS	1	
		11	SS	60/0.04 m	
		12	TW	PM	18.0
		13	SS	60/0.06 m	
184.0		14	SS	60/0.1 m	
18.7	End of Borehole. Stabilized water level measured nine days after completion of drilling. Standpipe slotted from 15.8 m to 9.8 m, sealed from 4.0 m to 3.4 m.				



+³, ×³: Numbers refer to Sensitivity

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Shaheen & Peaker 1995

Foundation Design

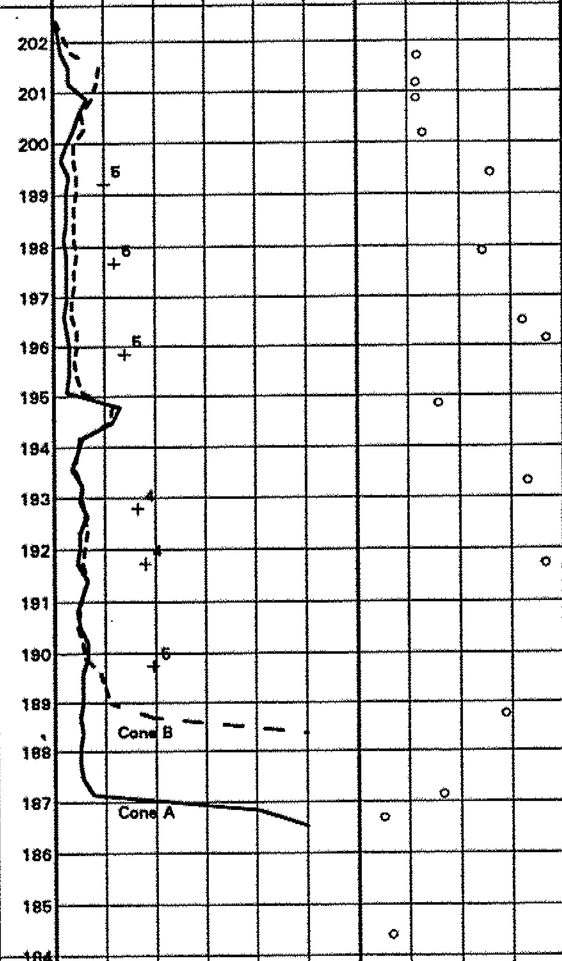
RECORD OF BOREHOLE No D6

METRIC

W.P. 128-88-00	LOCATION Station 18+905.5, 20.7 m west of centreline		ORIGINATED BY R.O.
DIST 13	HWY 535	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY B.B.
DATUM Geodetic	DATE 10.02.95 & 10.03.95		CHECKED BY K.P.

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³
202.7 / 0.0	**Clay to Silty Clay:** sand layers, organics, brown/grey, stiff	1	SS	3	19.3
		2	SS	6	19.0
		2A	TW		19.0
		3	SS	10	17.9
199.7 / 3.0	**Silty Clay to Clay:** soft to firm with some gravel sizes, fissured, laminated structure	4	SS	2	
		5	SS	1	
		6	SS	1	14.8
		6A	TW		
195.2 / 7.5	**Silty Clay:** silt, sand and gravel layers, also with cobbles and boulders, soft or loose	7	SS	27	
		8	SS	1	
		9	TW		14.9
		10	SS	1	
		11	SS	60/0.04 m	
		12	TW		18.0
		13	SS	60/0.05 m	
		14	SS	60/0.1 m	
184.0 / 18.7	End of Borehole. Stabilized water level measured nine days after completion of drilling. Standpipe slotted from 15.8 m to 9.8 m, sealed from 4.0 m to 3.4 m.				

Columns also shown: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL



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

15 20 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 1

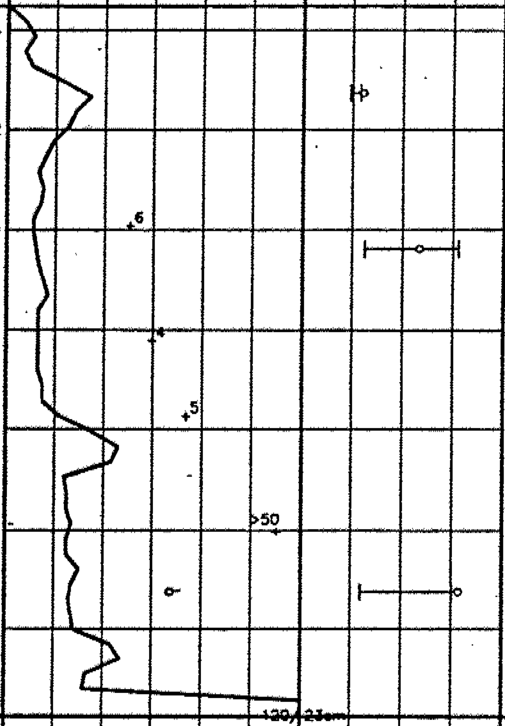
METRIC

W.P. 128-88-01 LOCATION Sta 18+920.6 Lt 24.7 m ORIGINATED BY DK

DIST 54 HWY 535 BOREHOLE TYPE Hollow Stem Auger, Cone COMPILED BY DK

DATUM Geodetic DATE 92 03 06 CHECKED BY BI

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
204.5	Ground Surface									
0.0 / 203.4	Clayey Silt, Some Sand Occasional Organic Inclusions Brown, (Possible Fill)	1	SS	8		204				
1.1 / 201.5	Clayey Silt, Layered Occasional Rootlets Light Grey and Brown Stiff to Very Stiff	2	SS	18						0 0 (100)
		3	SS	8		202				
3.0	Silty Clay to Clay Firm Grey	4	SS	0						
		5	TW	PH	▼*	200	+6		16.8	
						198	+4			
						196	+5			
	Some Gravel	6	SS	3						
						194	>50			
		7	SS	1						
191.7		8	TW	PH		192			16.0	0 0 (100)
12.8 / 190.2	Silty Sand Grey, Very Loose	9	SS	0						
14.3	End of Borehole Probable Bedrock						120/ 23cm			



* Water level in the borehole one hour after completion

Reproduced from W.P 50-78-02

+³, ×⁵: Numbers refer to Sensitivity   15 ⊙ 5 (%) STRAIN AT FAILURE (20)

MORTON & PARTNERS LIMITED, OCTOBER 1981 **

RECORD OF BOREHOLE No 1A

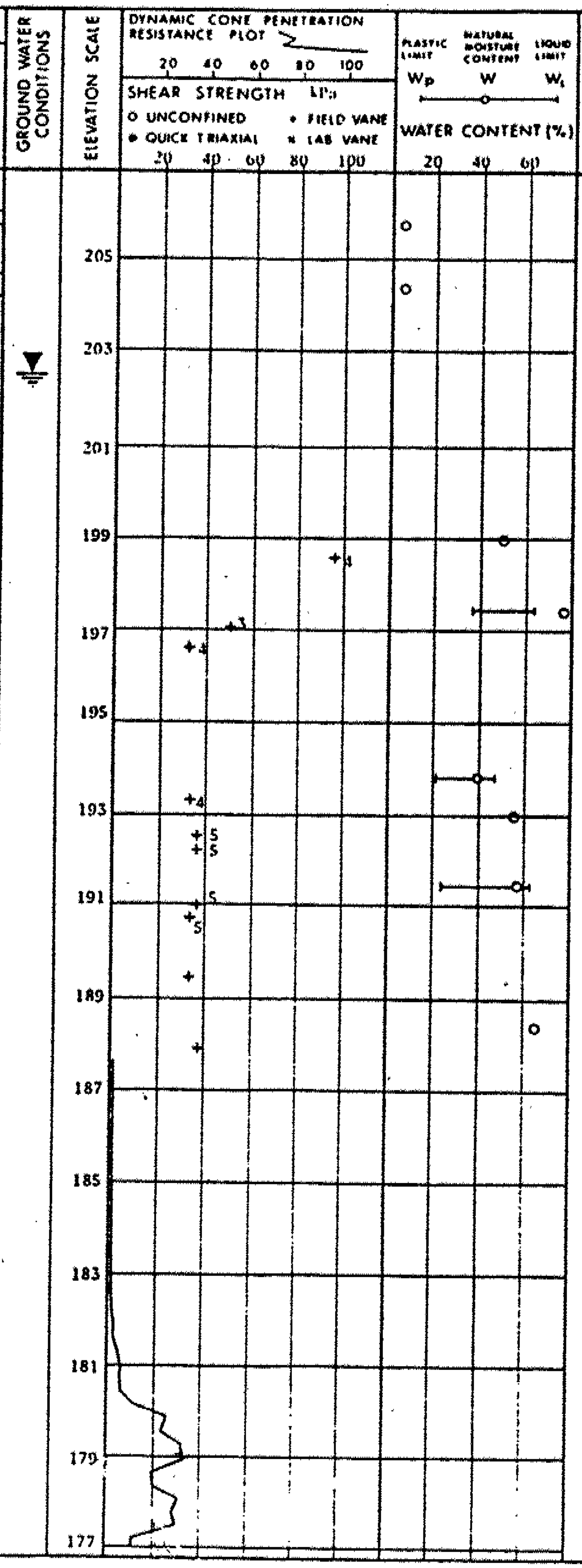
METRIC

W.P. 128-88-01 LOCATION Sta 18+838.4 Lt 3.6 m ORIGINATED BY

DIST 54 HWY 535 BOREHOLE TYPE NW Casing, Tricone, Washboring, Cone Test COMPILED BY

DATUM Geodetic DATE 81-09-10, 11, 12 and 13 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
206.5						
0.0	Sand and gravel, Compact, Granular "B" fill, Brown	1	SS	-		32 65 3
		2	SS	32		
204.1 / 2.7	Rockfill, Red and grey	3	SS	27		
199.2 / [illegible]	Clay, stiff, Green and grey	4	SS	9		
197.9 / 8.9	Clay, silty, trace of sand and gravel, Firm, Grey	5	SS	7	87%	Organic
		6	TW	PH		
		7	TW	PH		
		8	TW	PH	18.0	8 53 39
		9	SS	<1		
		10	SS	<1		
		11	TW	PH		
		12	SS	1		
187.6	End of sampling					
19.2	Start cone test					
	Probably clay					
182.4 / 24.4	Probably granular material, Compact					



+3, ×3 : Numbers refer to Sensitivity

20
15 ◇ 4 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

Ontario Transportation and Communications

RECORD OF BOREHOLE No 1A (continued)

METRIC

W.P. 128-88-01 LOCATION Sta 18+838.4 Lt 3.6 m ORIGINATED BY

DIST 54 HWY 535 BOREHOLE TYPE NW Casing, Tricone, Washboring Cone Test COMPILED BY

DATUM Geodetic DATE 81-09-10, 11, 12 and 13 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
177.0 / 29.8	Probably granular materials. Compact to dense, changing to very dense at ± elev. 168.4						177, 175, 173, 171, 169, 167				
166.0 / 40.8	End of cone test										

Reproduced from W.P. 50-78-02

OFFICE REPORT ON SOIL EXPLORATION

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 (%) STRAIN AT FAILURE / 10

RECORD OF BOREHOLE No 2A

METRIC

W.P. 128-88-01 LOCATION Sta 18+860.0 Lt 8.2 m ORIGINATED BY

DIST 54 HWY 535 HOLE TYPE Wash Boring COMPILED BY

DATUM Geodetic DATE 81-09-13, 14 and 15 CHECKED BY

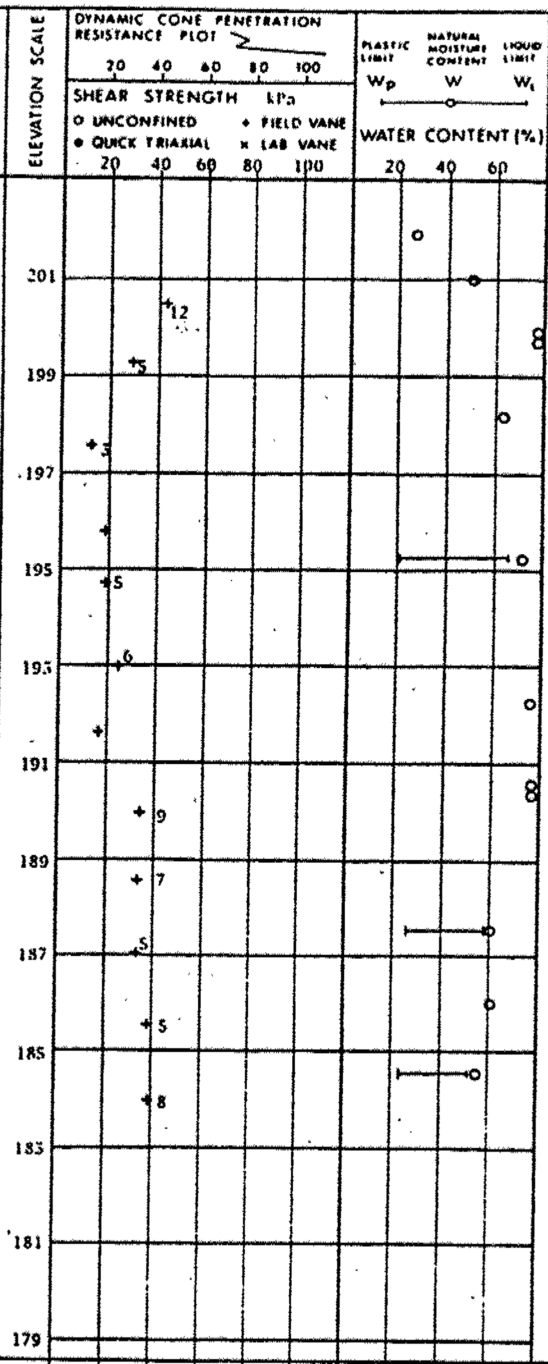
SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
203.0 / 0.0	Rockfill to 0.5 m Sand, silty, clayey. Wood pieces
201.2 / 1.8	Clay Firm Green and grey
198.7 / 4.3	Clay, silty, trace of sand and gravel. Soft to firm. Grey Washed only below elev. 184.3
178.6 / 24.4	End of Borehole

Reproduced from W.P. 50-78-02

SAMPLES

NUMBER	TYPE	'N' VALUES
1	SS	2
2	SS	7
3	SS	1
4	SS	1
5	TW	PM
6	TW	PM
7	TW	PM
8	SS	<1
9	SS	<1
10	TW	PM
11	TW	PM
12	SS	<1
13	SS	<1



UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
111%	
97%	
16.0	2 50 48
81%	

$^{+3}$, $^{\times 5}$: Numbers refer to Sensitivity

15 ⟡ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Appendix E Design Data

Figure Nos. 3a and 3b: Stress History
Sketch No. SK-1: Rigid Expanded Polystyrene
Sketch Nos. SK-2 and SK-3: Cellular Concrete
Table A: Roadway Protection
Table B: Comparison – Lightweight Fill Products
Figure No. S-1: Slope Stability

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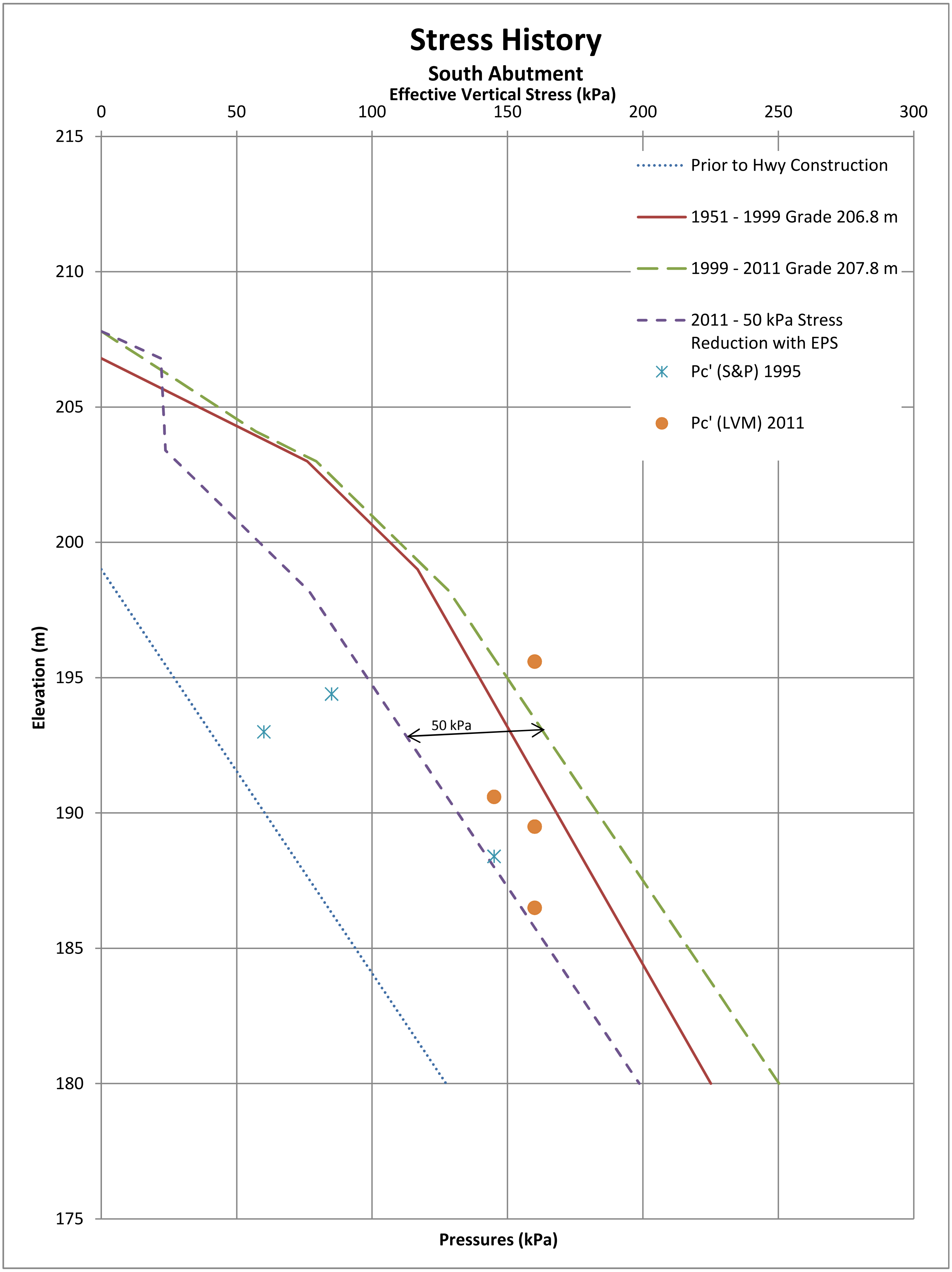
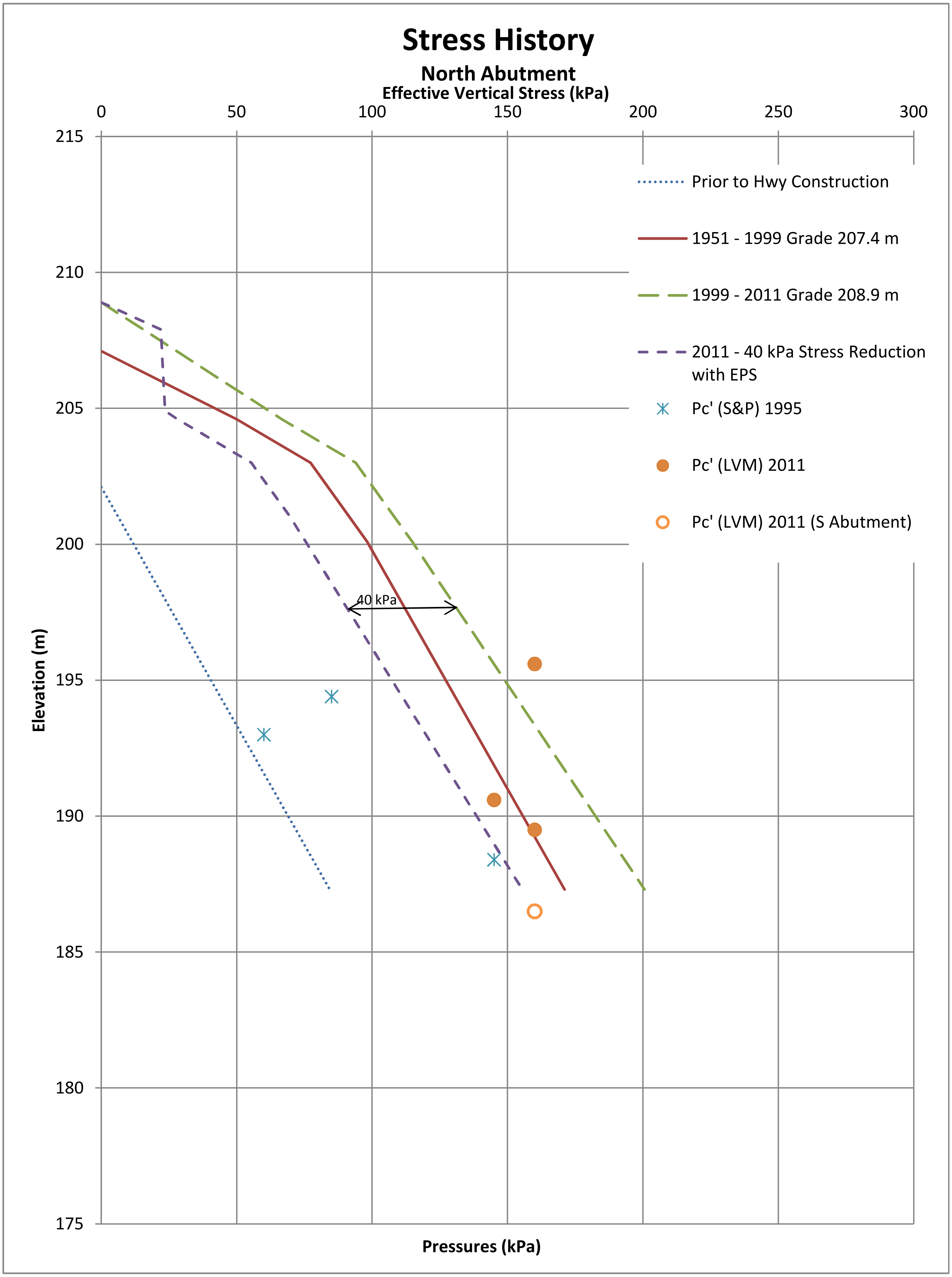

Stress History
South Abutment
Effective Vertical Stress (kPa)
0
50
100
150
200
250
300
215
210
205
200
195
190
185
180
175
Elevation (m)
Prior to Hwy Construction
1951 - 1999 Grade 206.8 m
1999 - 2011 Grade 207.8 m
2011 - 50 kPa Stress Reduction with EPS
Pc' (S&P) 1995
Pc' (LVM) 2011
50 kPa
Pressures (kPa)


Stress History
North Abutment
Effective Vertical Stress (kPa)
0
50
100
150
200
250
300
215
210
205
200
195
190
185
180
175
Elevation (m)
Prior to Hwy Construction
1951 - 1999 Grade 207.4 m
1999 - 2011 Grade 208.9 m
2011 - 40 kPa Stress Reduction with EPS
Pc' (S&P) 1995
Pc' (LVM) 2011
Pc' (LVM) 2011 (S Abutment)
40 kPa
Pressures (kPa)

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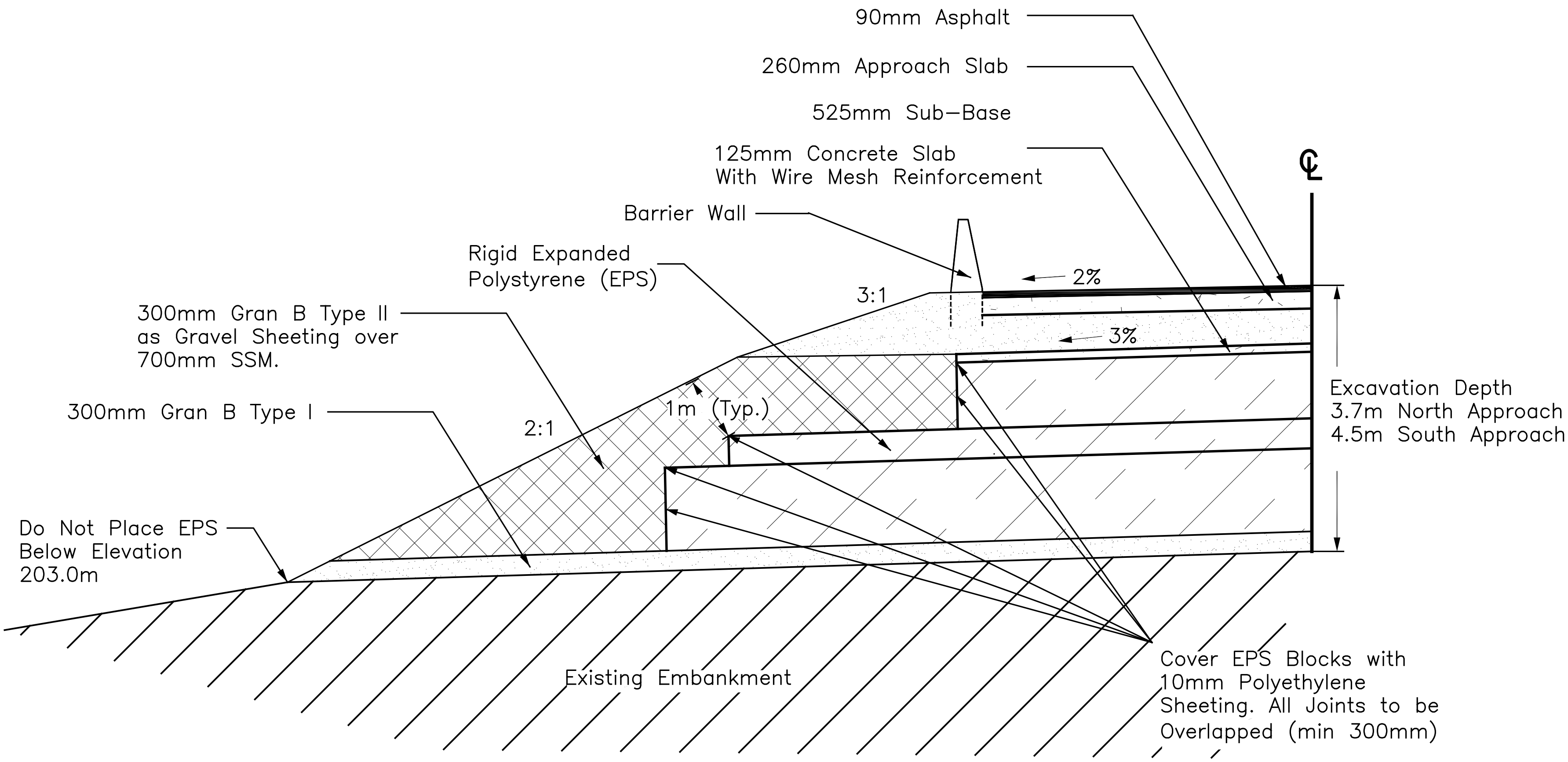
Rigid Expanded Polystyrene (EPS) Lightweight Fill

Typical Half Cross Section
North and South Abutments



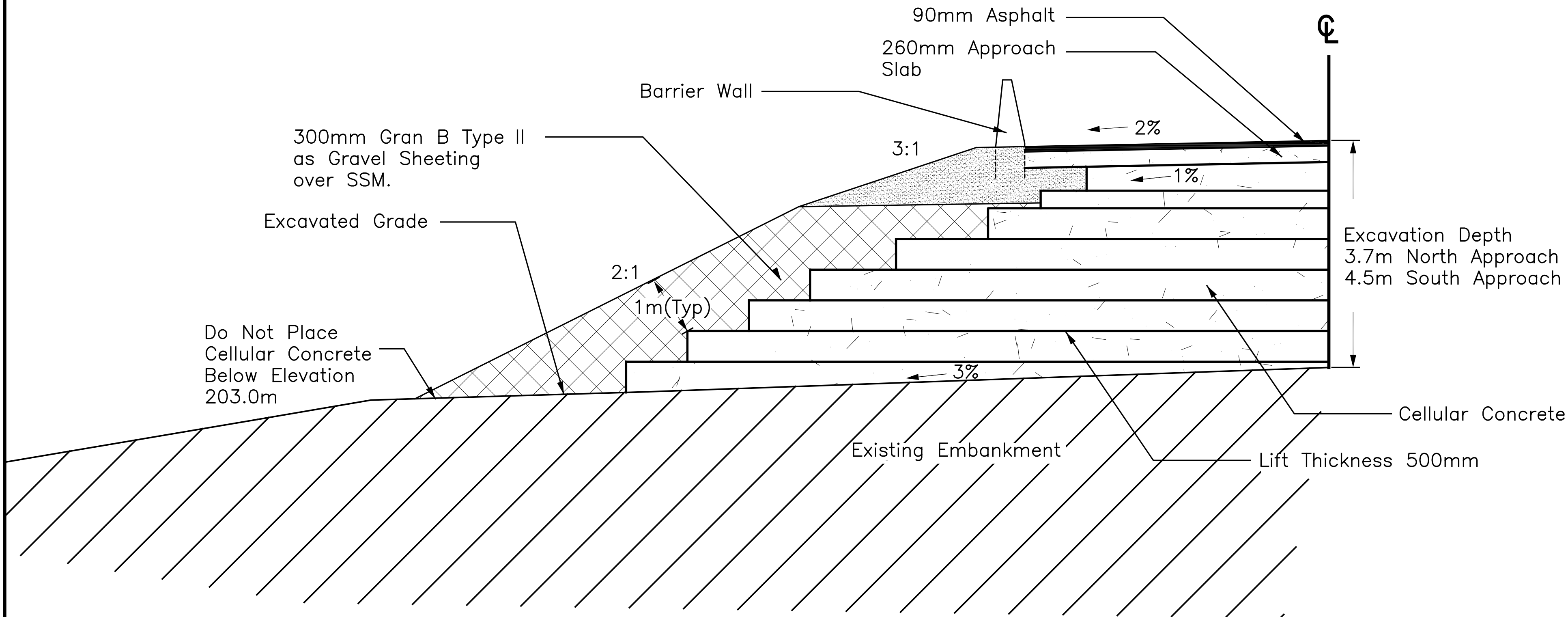
Scale: 1:125

HWY 535 - Nepewassi River Bridge - Site 46-130 Rigid Expanded Polystyrene (EPS) Lighweight Fill Township of Dunnet	FIGURE SK-1

LVM | MERLEX

Cellular Concrete Lightweight Fill

Typical Half Cross Section
North and South Abutments



Scale: 1:125

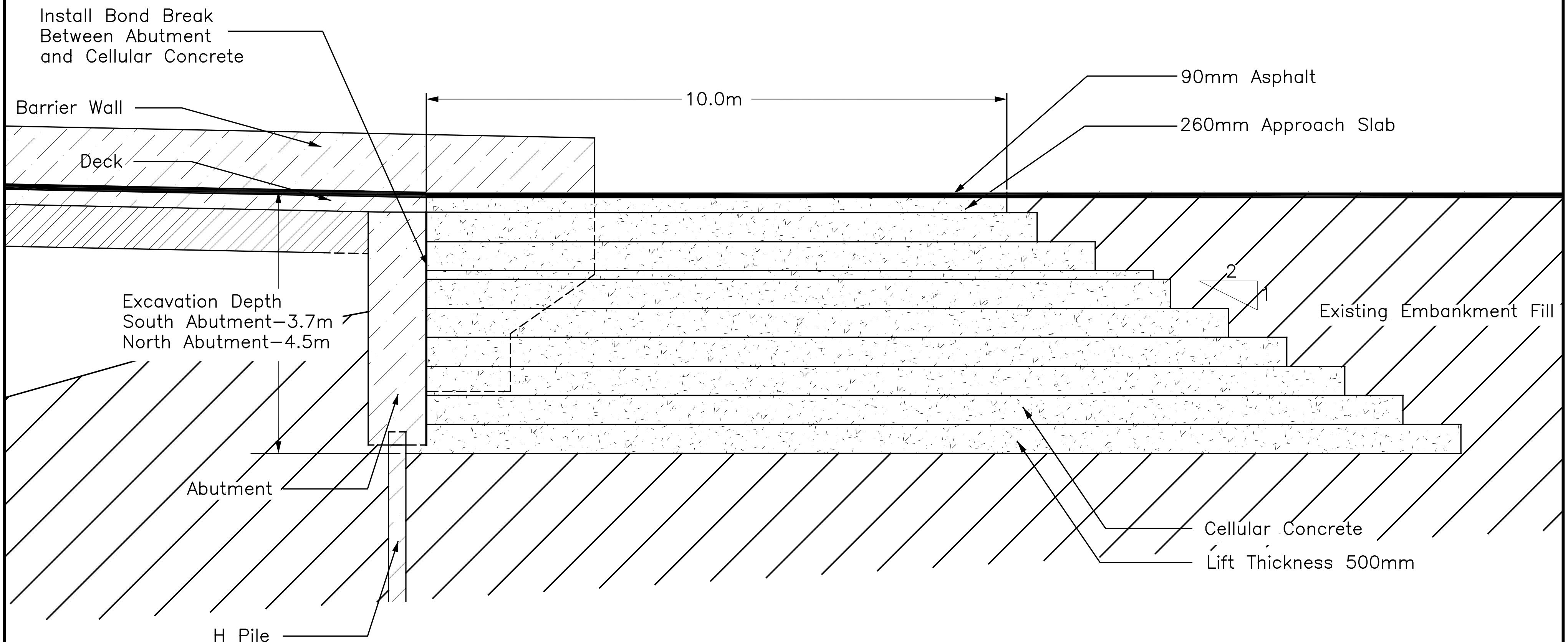
HWY 535 - Nepewassi River Bridge - Site 46-130 Cellular Concrete Embankment - Township of Dunnet	FIGURE SK-2

LVM | MERLEX

Cellular Concrete Lightweight Fill

Typical Centreline Longitudinal Section

North and South Abutments



Scale: 1:125

HWY 535 - Nepewassi River Bridge - Site 46-130 Cellular Concrete Embankment - Township of Dunnet	FIGURE SK-3

LVM | MERLEX

Table A – Roadway Protection Systems

Method	Depth Range (m)	Advantages	Disadvantages	Remarks	Estimated Costs
Wood Sheeting	1.5 – 5	-Low cost, -Easily installed in good ground conditions	-Limited by soil conditions, -Limited depth of installation, -Low strength, -discontinuous	Not considered due to ground conditions and height and type of cut	
Steel Sheet Piles	5 – 21	-High strength, continuous, -Readily available	-Limited by soil conditions (i.e. obstructions)	Considered as roadway protection at this site	$650/m^2
Pre-cast concrete panels	3 – 10	-Durable -Assists in minimizing seepage	-Limited depths -Can be damaged during installation -Limited by soil conditions (i.e. obstructions)	Not considered due to ground conditions and higher cost	
Soldier piles	5 – 25	-Easy installation -Readily available -Adaptable to various ground conditions	-Pre-drilling may be required -Possible ground loss	Considered as roadway protection at this site	$650/m^2
Tangent/ Secant/ Staggered Drilled Piles	10 – 18	-Readily available -Adaptable to various ground conditions	-Possible ground loss and/or seepage -Poor alignment tolerance	Not Considered due to limited depths required, size(width) and higher cost	
Concrete Diaphragm	10 – 30	-High Strength -Durable -Can be permanent	-High cost -Requires specialized equipment/control	Not Considered due to limited depths required and higher costs	
Micropiles with reinforced shotcrete face		-Can be installed in various ground conditions -High strength -Good tolerance	-High Cost -Requires specialized equipment	Not considered due to cost	

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Table B – Comparison Lightweight Fill Products

Fill	Unit Weight (kN/m^3)	Advantages	Disadvantages	Remarks
Rigid Expanded Polystyrene (EPS)	0.5	-1/40th of the unit weight of conventional granular fill -Frequently used by MTO	-Greater buoyancy compared to Cellular Concrete -Susceptible to break down due to hydrocarbon spills -Breaks down if exposed to flame (i.e. torch cutting during removal of RWP) -Time consuming in placing blocks	Cost – Excavation, bedding prep, placement, protective cover to profile grade $245/$m^3$, 10 m long excavation plus tapering down over 8 m length @ 2H:1V. S. Approach - $231,900 (4.7 m depth) N. Approach - 144,100 (4.0 m depth) Total $376,000 Cost – Roadway Protection $ 650/m^2 S. Approach – $ 73,600 N. Approach – 57,200 Total $130,800
Cellular Concrete	4.0	- Can be rapidly placed by pumping (up to 800 m^3/day) -Can be pumped from up to 1000 m away -Can be poured directly on excavated unwatered subgrades (no special preparations required) -Aids in stabilizing subgrade immediately after excavations -Fills voids without leaving gaps -Less buoyant than EPS -Not susceptible to hydrocarbons -Not susceptible to low temperature flame -Permits a thinner pavement structure (100-200 mm thick) -Provides greater pull out resistance -Provides greater resistance to lateral movements from seismic loading	-Higher unit weight than EPS, however requires only 250 mm pavement structure compared to 1 m thickness for EPS	Cost – Excavation, form and placement, cover to profile grade $175/$m^3$, 15 m long excavation plus tapering down over 8 m length @ 2H:1V. S. Approach - $156,800 (4.5 m depth) N. Approach - 115,200 (3.7 m depth) Total $272,000 Cost – Roadway Protection S. Approach – $ 68,700 N. Approach – 50,700 Total $117,400

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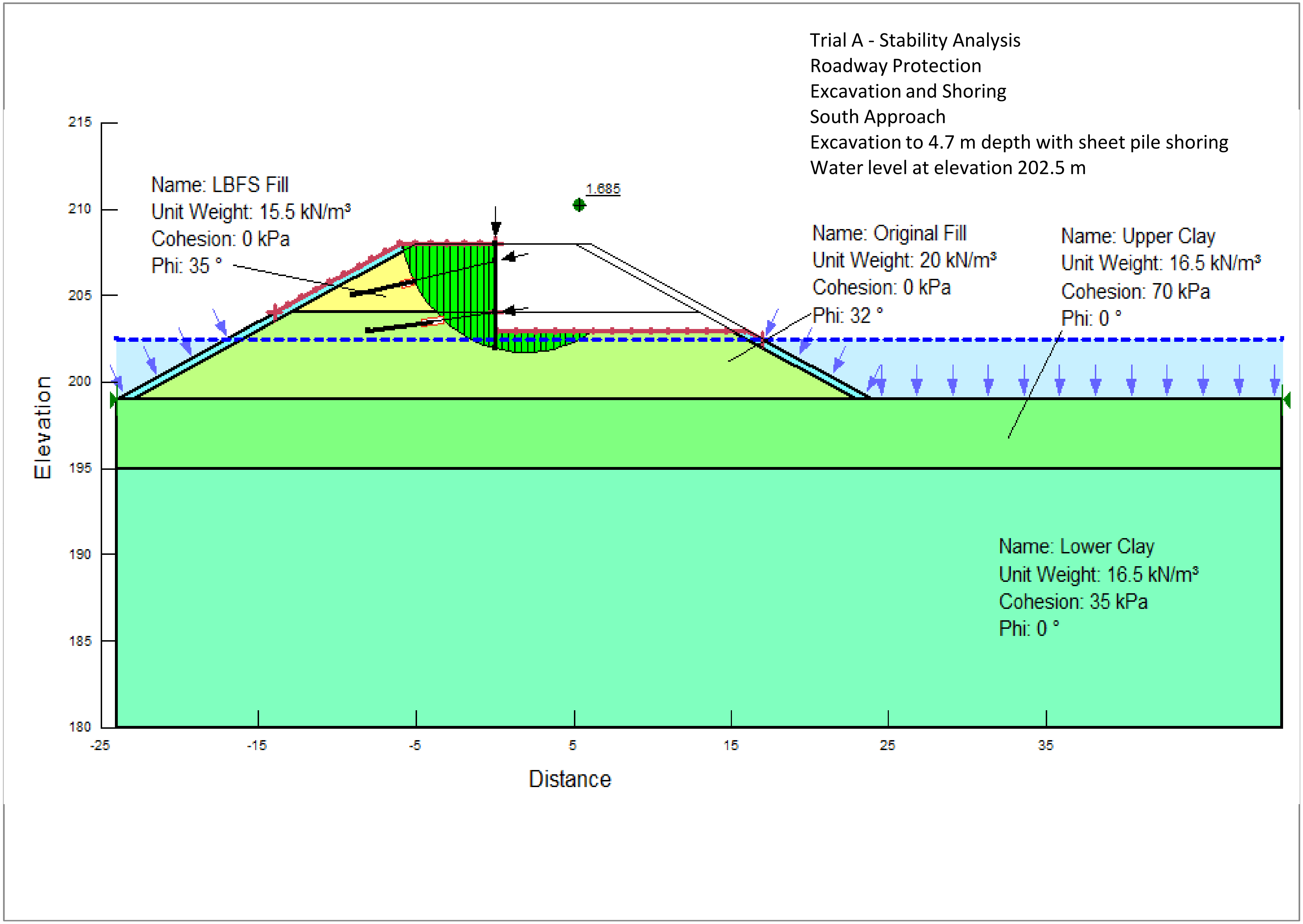

Stability Analysis
Station 18+840 TWP of Dunnet

Project: Hwy 535, Nepewassi River Bridge
Location: Site No. 46-130

Figure No. S-1