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REPORT ON

FOUNDATION INVESTIGATION AND DESIGN
G.W.P. 167-98-00, HIGHWAY 11
HIGH FILL AND CULVERT
REALIGNED HIGHWAY 11
DISTRICT 53, NEW LISKEARD

Submitted to:

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TABLE OF CONTENTS

TABLE OF CONTENTS (cont'd)

LIST OF DRAWINGS

LIST OF FIGURES

LIST OF APPENDICES

PART A

FOUNDATION INVESTIGATION REPORT
G.W.P. 167-98-00, HIGHWAY 11
HIGH FILL AND CULVERT
REALIGNED HIGHWAY 11
DISTRICT 53, NEW LISKEARD

Golder Associates

TABLE OF CONTENTS

SECTION PAGE

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (McCormick Rankin) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services associated with the Poplar Rapids River bridge rehabilitation project on Highway 11 near Cochrane, Ontario. Foundation engineering services are required for the area to the east of the existing Poplar Rapids Bridge where it is proposed to realign the existing horizontal curve along Highway 11.

This report addresses the proposed high fill embankment section and the proposed concrete box culvert as part of the realignment of Highway 11. A foundation investigation was carried out to determine the subsurface conditions at the high fill and culvert locations by drilling a limited number of boreholes, and carrying out in-situ testing and laboratory testing on selected samples.

The terms of reference for the scope of work are outlined in our Proposal No. P21-1274, dated August 22, 2002 and in our subsequent transmittal dated September 06, 2002.

2.0 SITE DESCRIPTION

The site is located in the Township of Haggart in the District of Cochrane, and is situated about 500 m to the east of the existing Poplar Rapids River Bridge site (Site 39W-001) and immediately to the north of the existing Highway 11 (see Figure 1). In this area, Highway 11 is presently a two lane, rural King's highway with a posted speed limit of 90 km/hr. The existing embankment is about 5.5 m high in this area with side slopes formed at about 2 horizontal to 1 vertical (2H:1V).

A small tributary creek channel runs through the site oriented in a general north-south direction. The channel is located within a 'valley' up to about 4.5 m deep with the valley slopes inclined at up to about 6.5H:1V. The creek was dry at the time of the geotechnical investigation carried out in August 2002 and contained a few inches of water at the time of the recent foundation investigation. Water in the creek is carried beneath the existing Highway 11 through an existing 33.6 m long reinforced concrete box culvert that is 1800 mm wide and 1220 mm high. The existing ground surface is relatively flat with a slight downward trend towards Poplar Rapids River to the west of the site.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out on October 2 and 3, 2002. At that time, a total of three (3) boreholes, labelled Boreholes 02-1 to 02-3, were advanced at locations along the proposed culvert extension and within the proposed high fill area. Boreholes 02-1 and 02-3 were advanced near the limits of the proposed culvert to depths of about 10 m each. Borehole 02-2 was extended to about 20 m depth near the midpoint of the culvert, which coincides with the centreline of the proposed Highway 11 realignment in the area where the embankment will be highest.

The boreholes were advanced with a track-mounted CME-75 drill rig using 108 mm inside diameter hollow stem augers supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. In the boreholes, overburden samples were obtained at 0.75 m to 1.5 m intervals of depth using 73 mm outside diameter Shelby tubes, and 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. In addition, in-situ vane testing was carried out to measure the undrained shear strength of the clay using an 'N' size vane. The groundwater conditions in the open boreholes were observed throughout the drilling operations, and piezometers were installed in two of the boreholes to permit monitoring of the groundwater levels at these locations. The piezometers consist of 25 mm outside diameter pipes with a 0.3 m long slotted tip that are sealed at selected depths within the boreholes.

The field work was supervised on a full-time basis by a member of our engineering staff who cleared the area of buried utilities, located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers (split spoon samples) or sealed in the Shelby tubes, and transported to our laboratory in Mississauga for further examination and testing. Index and classification tests consisting of grain size analyses, Atterberg limits tests and water content determinations were carried out on selected soil samples. In addition, standard, one-dimensional consolidation (oedometer) tests were carried out on two selected samples of the cohesive deposit.

The borehole locations were established relative to the proposed Highway 11 realignment stations staked in the field. Using the stations and offsets, the borehole locations were plotted on the drawing provided by McCormick Rankin and the northing and eastng coordinates were obtained. The elevations of the boreholes were referenced to the top of the north limit of the existing culvert (Elevation 227.04 m). The borehole locations, together with elevations, northings, and eastings, are shown on the attached Drawing 1.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Site Geology

This section of Highway 11 is located in the physiographic region of Ontario known as the Abitibi Upland, which is part of the central portion of the Canadian Shield (Geology of Ontario, Ontario Geological Survey, Special Volume 4, 1991). The Abitibi Upland generally extends northwest from Sudbury to Lake Superior, and transitions to the Severn Upland north of the lake. Physiographic mapping in the vicinity of this project indicates the subsoils consist of fine grained deposits, predominantly with a silty clay to clayey silt matrix (Map 2555, Quaternary Geology, Barnett, Henry and Babuin, 1991). These conditions are generally consistent with the results of the current geotechnical investigation at the bridge sites.

Geological mapping (Map 2543, Ministry of Northern Development and Mines, 1991) indicates that the subject site is located within the Superior Province (Precambrian). The bedrock in this area consists of Metasedimentary rocks (wacke, conglomerate, slate, chert and siltstone). However, bedrock was not encountered at the site during the course of the foundation investigation.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets and Figures 2 to 7 following the text of this report. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoils at the site consist of a surficial layer of topsoil underlain by an extensive deposit of silty clay to clayey silt. A layer of silty sand underlain by silty sand till was encountered below the silt /clay deposit in the deepest boring. The results of the groundwater monitoring indicate that sub-artesian pressures exist within the underlying sand deposits where the water level measured in the lower piezometer was at about 3.3 m depth about 5 weeks after installation. The water level measured in the piezometer installed in the overlying silty clay deposit was at about 6.7 m depth about 5 weeks after its installation.

A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections. The locations and elevations of the boreholes, and a stratigraphic section along the proposed culvert are shown on the attached Drawing 1. The locations of the relevant Test Pits that were put down as part of the geotechnical investigation for this project (Golder No.

021-8014, dated October 2002) are also shown on Drawing 1; the information for these test pits is included in Appendix A.

4.2.1 Topsoil

A layer of black organic silty topsoil was encountered surficially in all boreholes. The topsoil ranges in thickness from 0.3 m to 0.4 m at the borehole locations. The relevant test pits put down as part of the geotechnical investigation indicate that the topsoil ranges in thickness from 0.5 m to 0.8 m at the test pit locations. It should be noted that the test pits were advanced within the existing streambed. The water content of two selected samples of the topsoil was measured at about 30 percent.

4.2.2 Silty Clay to Clayey Silt

A deposit of grey silty clay to clayey silt, trace sand and fine gravel was encountered below the topsoil. The deposit contains occasional silt seams in the lower portion of the deposit. On average, the silt seams are approximately 25 mm thick. A grain size distribution curve of a selected sample of the silty clay is shown on Figure 2.

Standard Penetration Testing (SPT) measured 'N' values ranging from 5 blows to 6 blows per 0.3 m of penetration within the silt / clay deposit. The results of the field vane tests are summarized on Figure 3. The measured undrained shear strength of the silt / clay deposit varies from about 40 kPa to 80 kPa indicating a firm to stiff consistency. A weaker zone, with an undrained shear strength of about 30 kPa, was encountered between 8 m and 10 m depth. The sensitivity of the deposit, as estimated from the field vane tests, ranges from about 1.5 to 4.5 as shown on Figure 3. The majority of the test results range from 1.5 to 2.5, implying that the clayey stratum in this area is of low to medium sensitivity based on the classification system provided in CFEM (1992). It should be noted that the highest sensitivity (about 4.5) was measured in the zone where silt seams were encountered.

Atterberg Limits testing conducted on samples obtained from this stratum show liquid limits (w_L) ranging from about 30 percent to 35 percent and plasticity indices (I_P) ranging from about 16 percent to 20 percent. The results of the Limits testing classify the soil in this stratum as an inorganic silty clay to clayey silt of low to intermediate plasticity. The results of the Atterberg Limits testing are shown plotted on the plasticity charts on Figures 4 and 5. The natural water content measured on selected samples from this stratum range from about 11 percent to 30 percent. In general, the water contents are between the measured plastic and liquid limits, corresponding to a liquidity index of less than 1.

The measurements of mass and dimensions conducted to estimate the natural bulk unit weight of two carefully trimmed samples from this stratum resulted in values of about 19 and 19.5 kN/m^3. The specific gravity measured on these two samples produced an average value of 2.72.

Consolidation tests (oedometer) were performed on two samples from this stratum. The results are summarized below.

Borehole (Sample)	Elevation (Depth) (m)	σ_{vo}' (kPa)	σ_p' (kPa)	OCR	e_o	C_r	C_c	c_v (cm^2/s)
02-2 (3)	223.3 (3.3)	40	170	4.2	0.748	0.032	0.200	1.5×10^{-3}
02-2 (7)	217.2 (9.4)	96	140	1.5	0.852	0.038	0.278	1.5×10^{-3}

where : σ_{vo}' is the effective overburden pressure in kPa
σ_p' is the pre-consolidation pressure in kPa
OCR is the overconsolidation ratio
e_o is the initial void ratio
C_r is the recompression index
C_c is the compression index
c_v is the estimated coefficient of consolidation for anticipated stress range at sample depth in cm^2/s

The results of the consolidation tests are provided in Appendix B. Plots showing the Void Ratio versus Pressure for Samples 3 and 7 are shown on Figures 6 and 7, respectively. Figure 7A is a plot of the OCR values for the two oedometer tests and the interpretation of OCR based on correlations with the results of the in-situ vane tests. The correlation, $s_u = 0.22\sigma_p'$, relating in-situ shear strength to preconsolidation pressure (Mesri, 1975) was employed. The shear strengths calculated from the results of the two consolidation tests are also shown on Figure 3.

Boreholes 02-1 and 02-3 were both terminated within the silty clay to clayey silt deposit (about 10 m depth each). The deposit was found to be about 15.5 m thick at the location of Borehole 02-2, where it was penetrated.

4.2.3 Sand

Beneath the silt / clay stratum in Borehole 02-2, a deposit of grey silty sand to sand, trace silt was encountered. Trace gravel and clay were also noted within the deposit. Cobbles and/or boulders were inferred about 1 m above the base of the deposit in Borehole 02-2 based on auger resistance/grinding. The top of this layer is at about Elevation 211 m and the layer is about 3.7 m thick at the borehole location.

Standard Penetration Testing (SPT) measured an 'N' values of 7 blows and 69 blows per 0.3 m of penetration. A blow back of sand in the augers was encountered upon penetration of the deposit. The blow back caused disturbance of the deposit and likely resulted in the low 'N' value measured near the top of the deposit. The sand was washed out of the augers prior to taking the second sample ('N'=69). Therefore, it is inferred that the deposit has a very dense state of packing. The natural water content measured on a selected sample from this layer is about 19 percent.

4.2.4 Silty Sand Till

Underlying the sand layer in Borehole 02-2 exists a deposit of grey silty sand till containing trace gravel. Occasional cobbles were noted within the deposit based on auger resistance / grinding. The elevation of the top of this layer is at about 207.6 m. Borehole 02-2 was terminated at 0.4 m within this deposit (i.e. the deposit is at least 0.4 m thick at this location).

Standard Penetration Testing (SPT) measured 'N' values of 70 blows per 0.3 m of penetration, indicating a very dense state of packing. The natural water content measured on a selected sample from this layer is about 10 percent.

4.2.5 Groundwater Conditions

Boreholes 02-1 and 02-3 were both open and dry upon completion of drilling. A piezometer was sealed into the lower portion Borehole 02-2 (within the sand layers and the silt / clay containing silt seams) and within the silt / clay deposit in Borehole 02-3. The water level in the piezometer in Borehole 02-2 was measured at 2.5 m depth (Elevation 224.1 m) one day after its installation and at 3.3 m depth (Elevation 223.3 m) about 5 weeks after its installation. The piezometer installed in Borehole 02-3 was dry one day after completion of the drilling; the water level was measured in the piezometer at 6.7 m depth (Elevation 219.2 m) about 5 weeks after its installation. Assuming that the water level in the upper piezometer has stabilized, the readings indicate that sub-artesian pressures exist within the sand deposit underlying the silt / clay deposit and that there is upward seepage within the silt / clay deposit. It should also be noted that the water levels in the area are subject to seasonal variations.

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PART B

FOUNDATION DESIGN REPORT
G.W.P. 167-98-00, HIGHWAY 11
HIGH FILL AND CULVERT
REALIGNED HIGHWAY 11
DISTRICT 53, NEW LISKEARD

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides recommendations on the foundation aspects of design of the proposed culvert based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

The overall project involves the rehabilitation and widening of the Missinabi River and Poplar Rapids Bridges along Highway 11 in the District of Cochrane. As part of this work, the existing horizontal curve of Highway 11 about 500 m east of the Poplar Rapids Bridge will be realigned to the north of the existing alignment. A tributary creek flows from north to south within a shallow valley at this location and is carried beneath the existing Highway 11 via a 1,800 mm wide, 1,220 mm high, and 33.6 m long reinforced concrete box culvert. Based on the proposed grade of the realigned Highway 11, the fill embankment between Stations 16+565 and 16+605 (within the creek valley area) will be up to about 6 m high. It is understood that a new culvert will be placed across the realigned Highway at Station 16+575. The culvert will be 43 m in length and will have the same section dimensions as the existing culvert. The existing culvert will be removed as part of the works.

It is understood that the embankment slopes for the new Highway 11 alignment will have side slopes of 2 horizontal to 1 vertical (2H:1V) beyond the 3H:1V slopes as part of the pavement structure.

5.2 Culvert Foundations

The subsoils encountered in the boreholes at the site generally consist of a thin layer of topsoil underlain by firm to stiff silty clay to clayey silt that in turn is underlain by very dense sand and silty sand till. The silty clay / clayey silt deposit is about 15.5 m thick, based on the results of the deepest borehole (Borehole 02-2) put down at the centre of the proposed culvert. The groundwater level is expected to be within about 1 m of the existing ground surface.

Based on the subsurface information available at the culvert location it is considered feasible to found the reinforced concrete box culvert on the native soils providing the subgrade preparation at grade and allowance for settlements is provided as per the following subsections of this report. The culvert should be designed to withstand the appropriate weight of fill and traffic loadings, and frost pressures (where adequate frost cover is not provided).

5.2.1 Axial Geotechnical Resistance

Based on the cross section of the culvert provided by McCormick Rankin dated February 2003, the invert of the proposed box culvert will be at Elevations 225.6 m and 226.1 m at the north and south ends, respectively. This will place the culvert about 0.5 m below the existing ground surface. The factored geotechnical resistance at Ultimate Limit States (ULS) that may be used for design of the culvert where founded on the properly prepared native soil at or below the invert elevations is 200 kPa. The design value for the geotechnical resistance at Serviceability Limit States (based on 25 mm of settlement) does not apply since the culvert settlement will be governed by the embankment loading under which greater settlement will occur as discussed in Section 5.2.3.

It should be noted that the ULS axial capacity given above is based on a box width equal to 1.8 m.

5.2.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the base of the concrete box culvert and the granular bedding should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction and the corresponding coefficient of friction that may be used for design are 24 degrees and 0.45, respectively (assuming intimate contact between the base and granular bedding along the length of the culvert).

5.2.3 Estimated Settlement

A settlement analysis was carried out to assess the impact of the construction of the high embankment along the alignment of the proposed culvert. The analysis was based on the existing subsurface information, including the oedometer and in-situ test data collected.

The settlement analysis was performed using the commercially available program UNISETTLE (v3.0) produced by Unisoft Limited. A section representing the highest anticipated embankment area, which also corresponds to the location of the proposed culvert, was used for the analysis.

The overconsolidation ratio (OCR) profile required for the analysis was established using the results of the oedometer tests as well as correlations with the results of the in-situ vane tests. The following correlation relating in-situ vane shear strength to pre-consolidation pressure (Mesri, 1975) was employed.

$$s_u = 0.22\sigma_p'$$

where: s_u is the in-situ measured undrained shear strength in kPa

A summary plot showing the estimated values of OCR and the design profile of OCR versus depth used in the analyses for the clayey stratum at the site is shown on Figure 7A. The design profile of OCR was chosen giving the most weighting to the results of the oedometer tests conducted on the specimens obtained from Borehole 02-2, Samples 3 and 7. The OCR results from these tests were consistent with the lower values estimated from the correlation with the in-situ vane shear strengths. Therefore, the OCR profile line follows the lower bound of the trend provided by the scatter of all vane strength correlations.

The following additional parameters were used in the analyses of the settlement along the culvert:

Borehole (Sample)	*Initial Void Ratio* e_o	*Recompression Index* C_r	*Compression Index* C_c	*Thickness of Clayey Stratum (m)*	*Unit Weight* γ *(kN/m³)*
02-2 (3)	0.748	0.032	0.200	6.5	19.5
02-2 (7)	0.852	0.038	0.278	9	19.0

The results of the settlement analysis is presented on Figure 8 showing the estimated settlements along the proposed embankment and the proposed 52.5 m long culvert. The calculated settlements presented in this figure are due to the highest anticipated loading imposed on the compressible silt / clay foundation soils by the construction of the new roadway embankment at the site. As shown on Figure 8, the maximum settlement along the culvert is estimated to be about 300 mm (including secondary consolidation).

Based on an estimated coefficient of consolidation (c_v) of 1.5×10^{-3} cm²/s as determined from the oedometer tests at the stress range anticipated below the embankment, it is calculated that about 50 percent of the primary consolidation component of the settlement will be complete in about 3 years and about 95 percent will be complete in about 20 years. The presence of silty seams and the higher horizontal permeability will reduce this time probably by a factor of about 5 to 10. For the secondary (creep) consolidation component of the settlements, it is estimated [based on an estimated coefficient of secondary consolidation $\{c_\alpha\}$ of 0.009 for the upper 6.5 m and $\{c_\alpha\}$ of

0.013 for the lower 9 m of the deposit] that approximately 100 mm of settlement will occur over each log cycle of time. Therefore, following completion of primary consolidation, about 100 mm of creep settlement is expected to occur.

5.3 Culvert Construction

5.3.1 Placement

The following placement procedure may be utilized for the culvert in order to maintain flow immediately after construction and after long-term settlement due to consolidation:

- Place culvert at a minimum grade of 0.5 percent (or as required by stream hydraulic requirements) from the intake to the embankment centreline.
- Place culvert from centreline to outlet at minimum grade of 0.5 percent positive grade, plus an allowance for maximum centreline settlement. (i.e. primary and secondary consolidation) e.g. grade = 0.5 + (0.30/21.5) = 1.90 percent.

The magnitude of settlement that the culvert will have to withstand implies that construction joints will be required. It is recommended, based on the settlement pattern, that at least five joints are used; one located at the centreline, and the remaining four spaced 10 m and 15 m from the centreline (i.e. two north and two south of the centreline). It should be noted that spreading of the joints will occur due to the horizontal strain component associated with the settlement.

5.3.2 Bedding and Backfill

The bedding, backfill and levelling pad requirements for the culvert extension should be in accordance with OPSD 803.020 for concrete box culverts. The culvert extension should be designed for the full overburden pressure and live load, assuming an embankment fill unit weight of 21 kN/m^3.

The box culvert should be provided with at least 400 mm of OPSS Granular 'A' material for bedding purposes and for partial frost protection. The bedding should be compacted to at least 95 per cent of the Standard Proctor maximum dry density. Groundwater and surface water control will be required to avoid "pumping" of water into the fill during compaction.

Backfill to the culvert walls should consist of granular fill meeting the specifications for OPSS Granular 'A' or Granular 'B', Type II (but with less than 5 per cent passing the 200 sieve). The backfill should be placed in lifts not exceeding 200 mm loose thickness and compacted to 95 per cent Standard Proctor dry density. The fill depth during placement should be maintained equal on both sides of the culvert with one side not exceeding the other by more than 400 mm as per Section 5.3.3 of this report.

5.3.3 Lateral Earth Pressures

The lateral pressures acting on the walls of the culvert extension will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill and on the tolerable lateral movement of the structure. It is assumed that the box culvert is a rigid structure that will not tolerate lateral movement and therefore, at rest pressures should be used. The following recommendations are made concerning the design of the culverts in accordance with the OHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' Type II but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the culvert granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 803.010 and 803.02.
- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the culvert, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.
- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the walls (Case I from OHBDC Figure 6-7.4.1) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the footing (Case II from OHBDC Figure 6-7.4.3).
- For Case I, the pressures are based on the existing and proposed embankment fill materials and the following parameters (unfactored) may be assumed:

Soil unit weight (assuming clean earth fill)	21kN/m^3
Coefficients of lateral earth pressure:	
'at rest'	0.50

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

	Granular 'A'	**Granular 'B' Type II**
Soil Unit Weight	22 kN/m^3	21 kN/m^3
Coefficients of Lateral Earth Pressure		
'at rest'	0.43	0.47

5.3.4 Erosion Protection

Typically, the subsoils at the invert level of the culverts consists of a firm to stiff silty clay / clayey silt deposit. This cohesive material would be classified as having low scourability.

In order to prevent creek water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a clay seal or cut-off wall should be provided at the upstream end of the box culverts. The clay seal should have a minimum thickness of 0.3 m. It should be keyed into the native subsoil, and extended a minimum horizontal distance of 2 m on either side of the culvert inlet openings and a minimum vertical height equal to the high water level. The material for the clay seal should be in accordance with the requirements of OPSS 1205.

Erosion protection should be provided upstream and downstream of the culvert as appropriate. Consideration could be given to the use of suitable non-woven geotextiles and rip-rap to provide erosion protection based on hydraulic requirements.

In addition, sediment control such as silt fences and / or erosion control blankets may be required during construction and diversion of creek to mitigate migration of fine soil particles into the water course.

5.3.5 Groundwater and Surface Water Control

The founding soils for the culvert are susceptible to disturbance due to upward seepage, water ponding and / or construction traffic. Groundwater seepage into the excavations should be expected, particularly where water-bearing fill or native soils are present. In general, pumping from properly filtered sumps or a filtered drain placed at the base of the excavation should provide sufficient groundwater control during foundation works. The creek / ditch waters will generally have to be diverted in order to permit construction in the dry.

5.3.6 Subgrade Preparation

It is noted that the soils in which the excavations will be formed are susceptible to disturbance from ponded water and construction traffic. For protection of the founding soils, a working mat of lean concrete should be placed as soon as practical after reaching the base of the excavation and following inspection by qualified geotechnical personnel. The granular bedding could then be placed on the mat. Where drainage or levelling is required additional granular bedding should be provided.

5.3.7 Excavations

Excavations works should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The native silt / clay is classified as a Type 3 soil and therefore temporary open-cut slopes can be carried out at 1 horizontal to 1 vertical (1H:1V)

5.4 High Embankment between Stations 16+565 and 16+605

5.4.1 Embankment Design

The profile of the proposed grade of Highway 11 is shown at about Elevations 233 m to 234 m from the west to east within the high fill section. This indicates the embankment will be up to about 6 m in height.

The permanent soil slopes of the embankment should be maintained not steeper than 2 horizontal to 1 vertical (2H:1V). Vegetation cover should be established on all soil slopes to protect embankment fill against surficial erosion, as per OPSS 572.

The embankment subgrade soil consists of a firm to stiff silty clay to clayey silt. The undrained shear strength profile of the subsoil is shown on Figure 3. Providing that the embankment subgrade is properly prepared with side slopes maintained at 2 horizontal to 1 vertical, the calculated factor of safety for the highest (i.e. 6 m) embankment section will be about 1.6, which indicates the embankment is stable against deep-seated failure (see Figure 9). Settlement of the embankment fills, properly placed and compacted, is estimated to be up to 300 mm, as described above.

5.4.2 Embankment Construction

The topsoil should be stripped from below the embankment areas and the subgrade should be inspected by qualified geotechnical personnel. Construction of the embankment above the prepared subgrade may be carried out using clean earth fill meeting specifications OPSS 212 or Select Subgrade Material meeting specifications with OPSS 1010, depending on material availability. All embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be compacted to 100 percent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

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DKB/ASP/JW/FJH/mmh
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LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (con't.)

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

(c) Hydraulic Properties

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

(d) Consolidation (one-dimensional)

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

(e) Shear Strength

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

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

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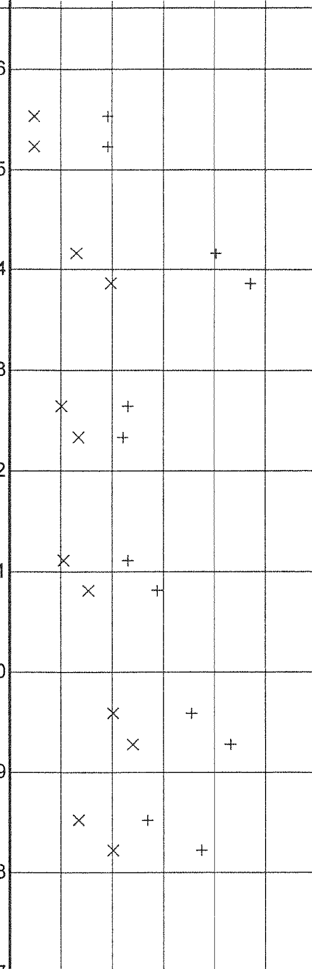
RECORD OF BOREHOLE No 02-1

METRIC

PROJECT	021-1153				
W.P.	167-98-00	LOCATION	5461542.7 N, 248153.5 E	ORIGINATED BY	ES
DIST	53 HWY 11	BOREHOLE TYPE	108mm I.D. Hollow Stem Augers	COMPILED BY	DKB
DATUM	Geodetic	DATE	Oct.2,2002	CHECKED BY	ASP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
226.6	GROUND SURFACE			
226.3 / 0.0	Topsoil	1	SS	6
0.3	Silty Clay to Clayey Silt, trace sand and fine gravel Firm to Stiff Grey Moist			
		2	TO	
		3	TO	
		4	TO	
		5	TO	
		6		
217.0 / 9.6	END OF BOREHOLE Note: Open borehole dry upon completion of drilling.			

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (226 to 217); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL



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

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RECORD OF BOREHOLE No 02-2

METRIC

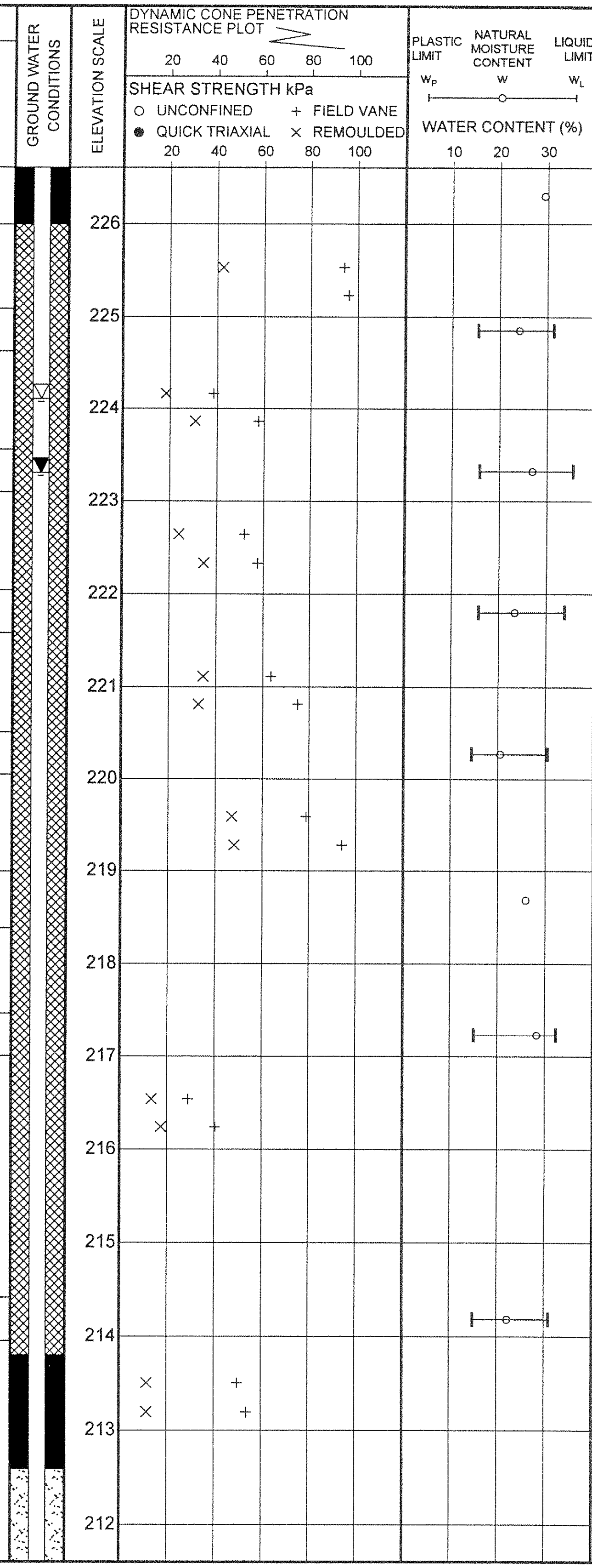
PROJECT 021-1153

W.P. 167-98-00 — LOCATION 5461517.4 N, 248150.4 E — ORIGINATED BY ES

DIST 53 HWY 11 — BOREHOLE TYPE 108mm I.D. Hollow Stem Augers — COMPILED BY DKB

DATUM Geodetic — DATE Oct.3,2002 — CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
226.6	GROUND SURFACE								
0.0	Topsoil	1	TO	6					
226.2 / 0.4	Silty Clay to Clayey Silt, trace sand and fine gravel Firm to Stiff Grey Moist	2	TO						
		3	TO		19.5	1	9	41	49
		4	TO						
		5	TO						
		6	TO						
		7	TO						
214.4 / 12.2	Silty Clay to Clayey Silt, trace sand and fine gravel, occ. silt seams (25mm thick)	8	TO		18.9				



Continued Next Page

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

RECORD OF BOREHOLE No 02-2

METRIC

PROJECT 021-1153

W.P. 167-98-00 LOCATION 5461517.4 N, 248150.4 E ORIGINATED BY ES

DIST 53 HWY 11 BOREHOLE TYPE 108mm I.D. Hollow Stem Augers COMPILED BY DKB

DATUM Geodetic DATE Oct.3,2002 CHECKED BY ASP

— CONTINUED FROM PREVIOUS PAGE —

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
211.4 / 15.2	Silty Sand to Sand, trace silt, trace gravel and clay Loose to very dense Grey Wet Note: blow-back in augers likely disturbed deposit and resulted in a low 'N' value for Sample 9. Cobbles and/or boulders inferred between 17.1m and 18.0m depth.	9	SS	7	211	~12
					210	
					209	
		10	SS	69	208	~18
207.5	Silty Sand, trace gravel, occ. cobbles Very dense Grey Wet (Till)	11	SS	70		~9
207.1 / 19.5	END OF BOREHOLE					

Notes:
1. Sand blow-back in augers to 11.9m depth (El.214.7m) upon penetration of sand deposit at 15.2m depth (El.211.4m). Sand washed out of augers prior to taking Samples 10 and 11.
2. Water level measured in piezometer at 2.5m depth (El. 224.1m) on October 4, 2002.
3. Water level measured in piezometer at 3.3m depth (El. 223.3m) on November 15, 2002.

Column headings: SOIL PROFILE (ELEV/DEPTH, DESCRIPTION, 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 × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

RECORD OF BOREHOLE No 02-3

METRIC

PROJECT	021-1153		
W.P.	167-98-00	LOCATION	5461493.8 N, 248148.1 E
DIST	53 HWY 11	BOREHOLE TYPE	108mm I.D. Hollow Stem Augers
DATUM	Geodetic	DATE	Oct.2,2002

ORIGINATED BY ES
COMPILED BY DKB
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
225.9	GROUND SURFACE			
0.0	Topsoil	1	SS	5
225.5 0.4	Silty Clay to Clay Silt, trace sand and fine gravel Firm to Stiff Grey Moist	2	SS	
		3	SS	
		4	SS	
		5	SS	
		6	SS	
		7	SS	
215.5 10.4	END OF BOREHOLE			

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (225, 224, 223, 222, 221, 220, 219, 218, 217, 216); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Notes:
1. Open borehole dry upon completion of drilling.
2. Piezometer dry on October 3, 2002.
3. Water level measured in piezometer at 6.7m depth (El. 219.2m) on November 15, 2002.

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

SITE LOCATION MAP
PROPOSED HIGH FILL AND CULVERT
HIGHWAY 11 - EAST OF POPLAR RAPIDS BRIDGE
NEW LISKEARD, ONTARIO

FIGURE 1

REFERENCE:
THIS FIGURE WAS CREATED FROM MAPART MAP TITLED "TIMMIINS" 1991/92 EDITION.

Date: OCTOBER, 2002

Project: 021-1153

Golder Associates

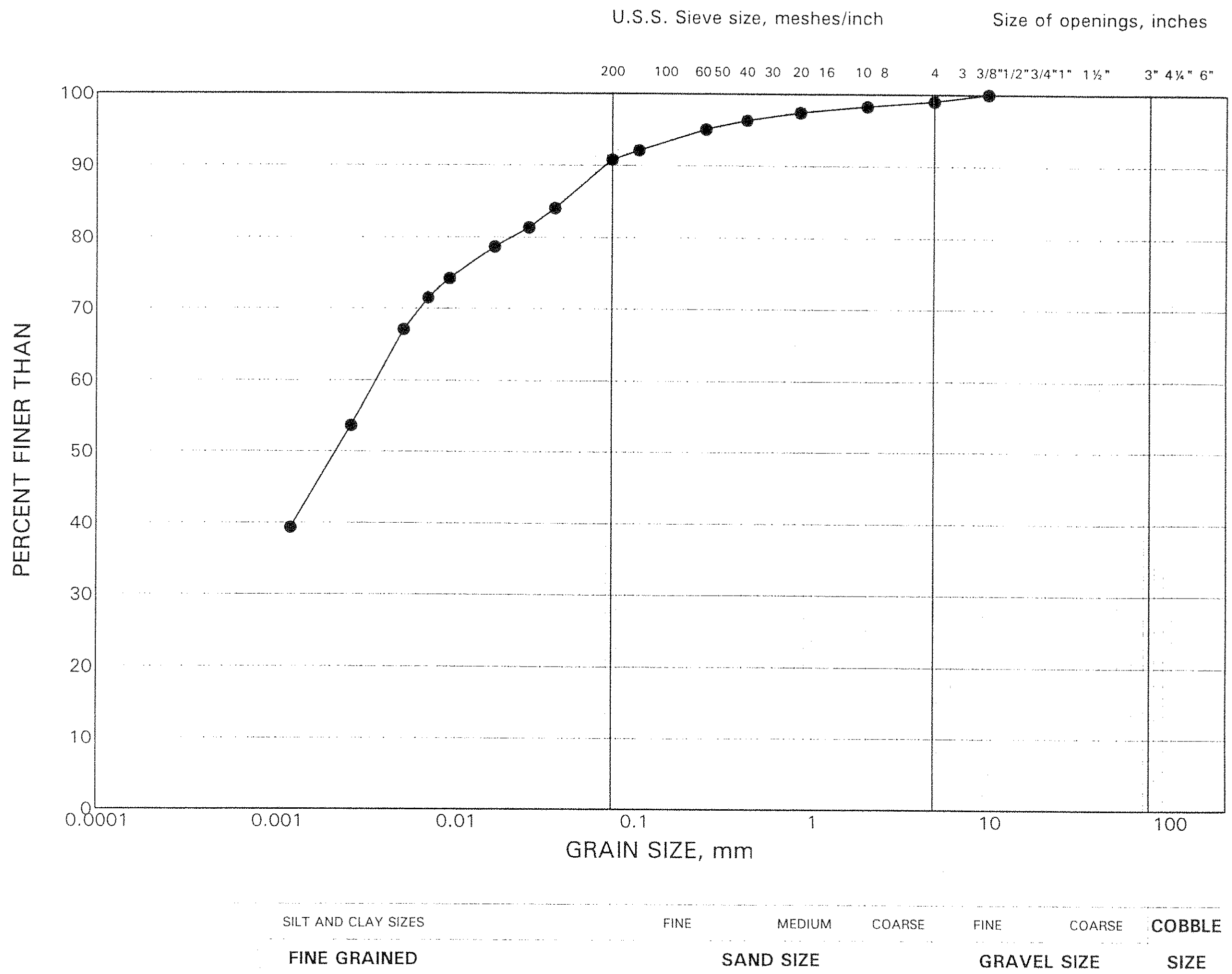
Drawn: RJ

Chkd: DKB

0211153FIG01.cdr

GRAIN SIZE DISTRIBUTION
Silty Clay to Clayey Silt

FIGURE 2



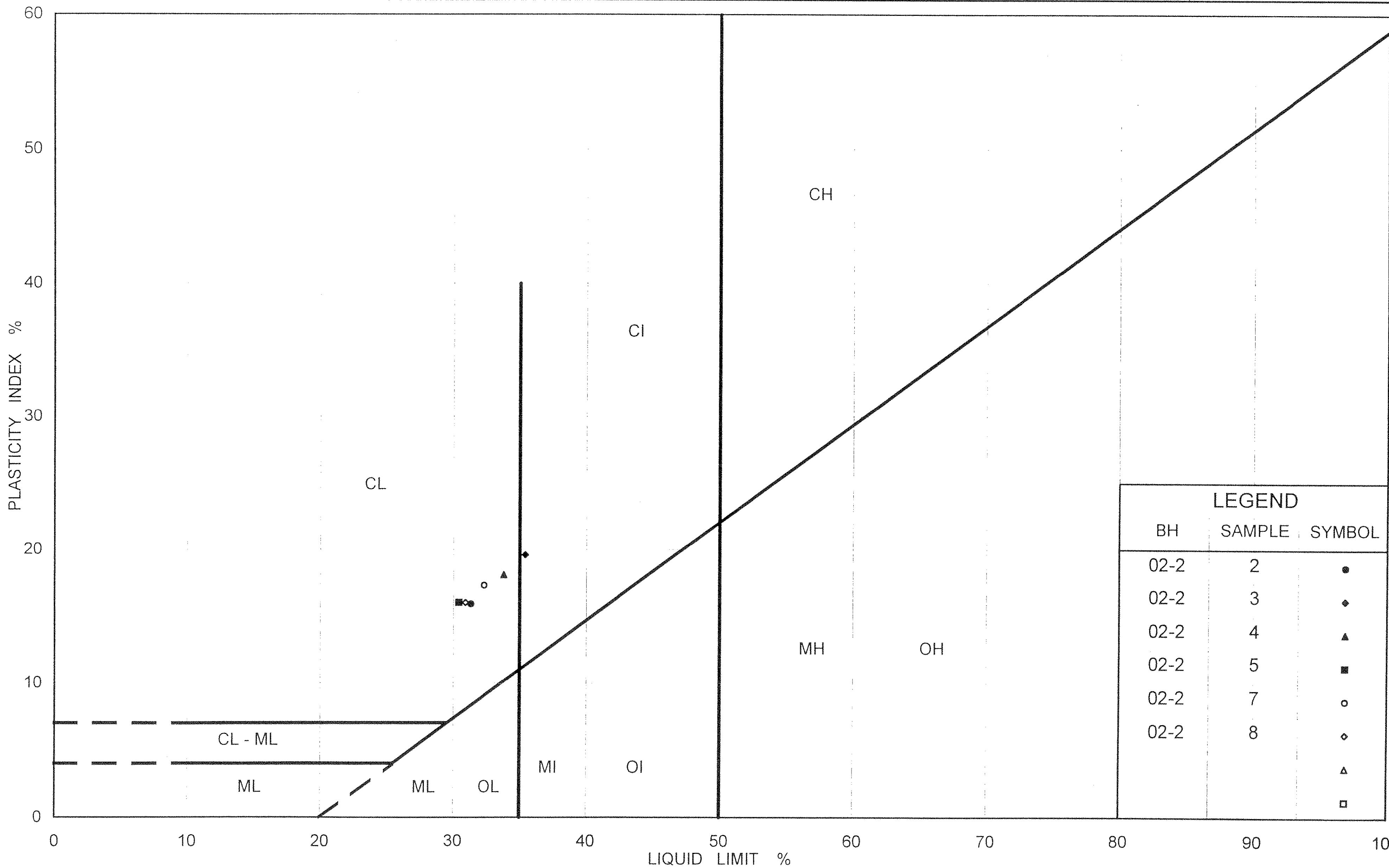
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	02-2	3	223.1

Project 021-1153

Golder Associates

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
02-2	2	●
02-2	3	◆
02-2	4	▲
02-2	5	■
02-2	7	○
02-2	8	◇
		△
		□

Ministry of Transportation

Ontario

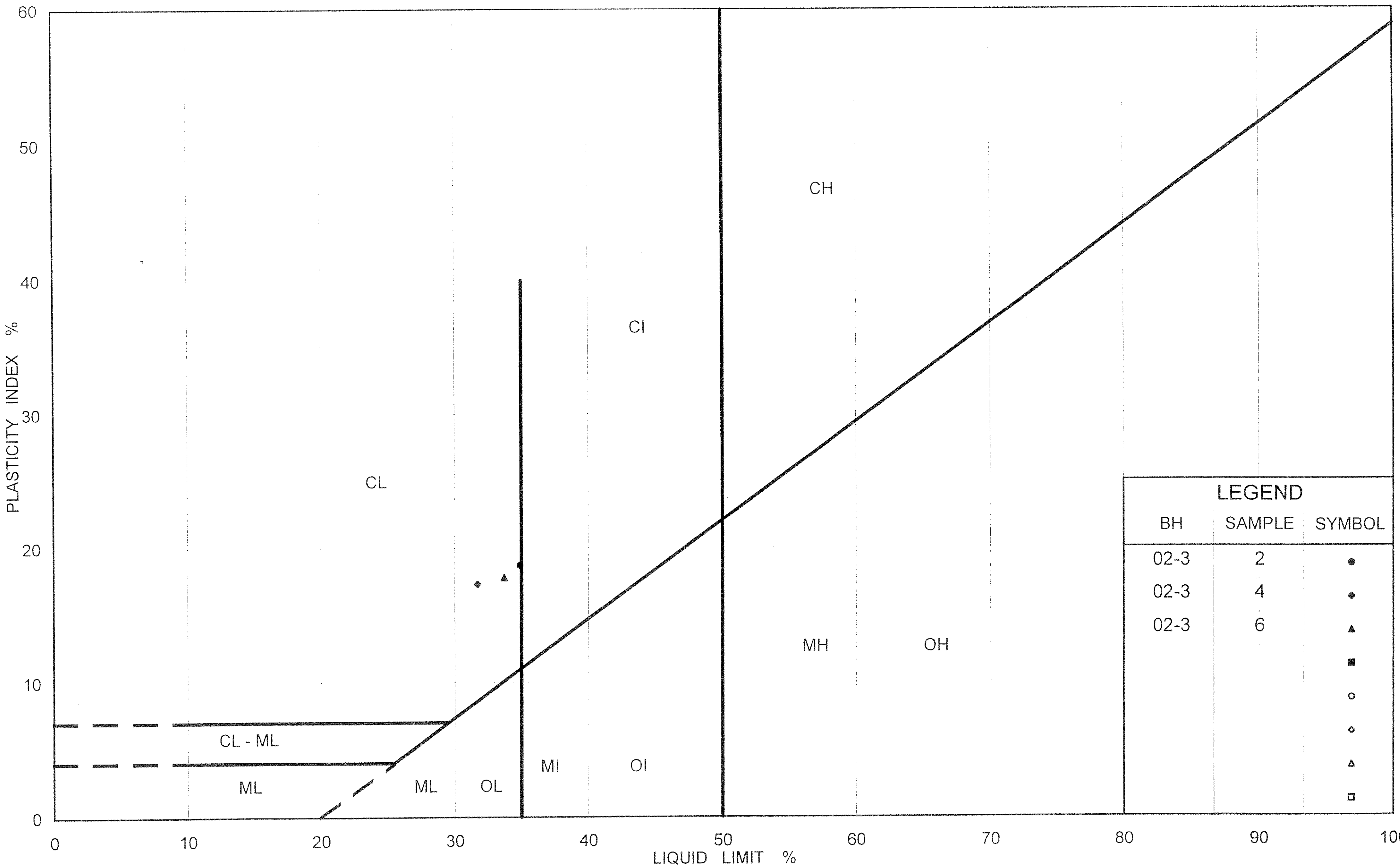
PLASTICITY CHART

Silty Clay to Clayey Silt

FIG No. 4

Project No. 021-1153

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

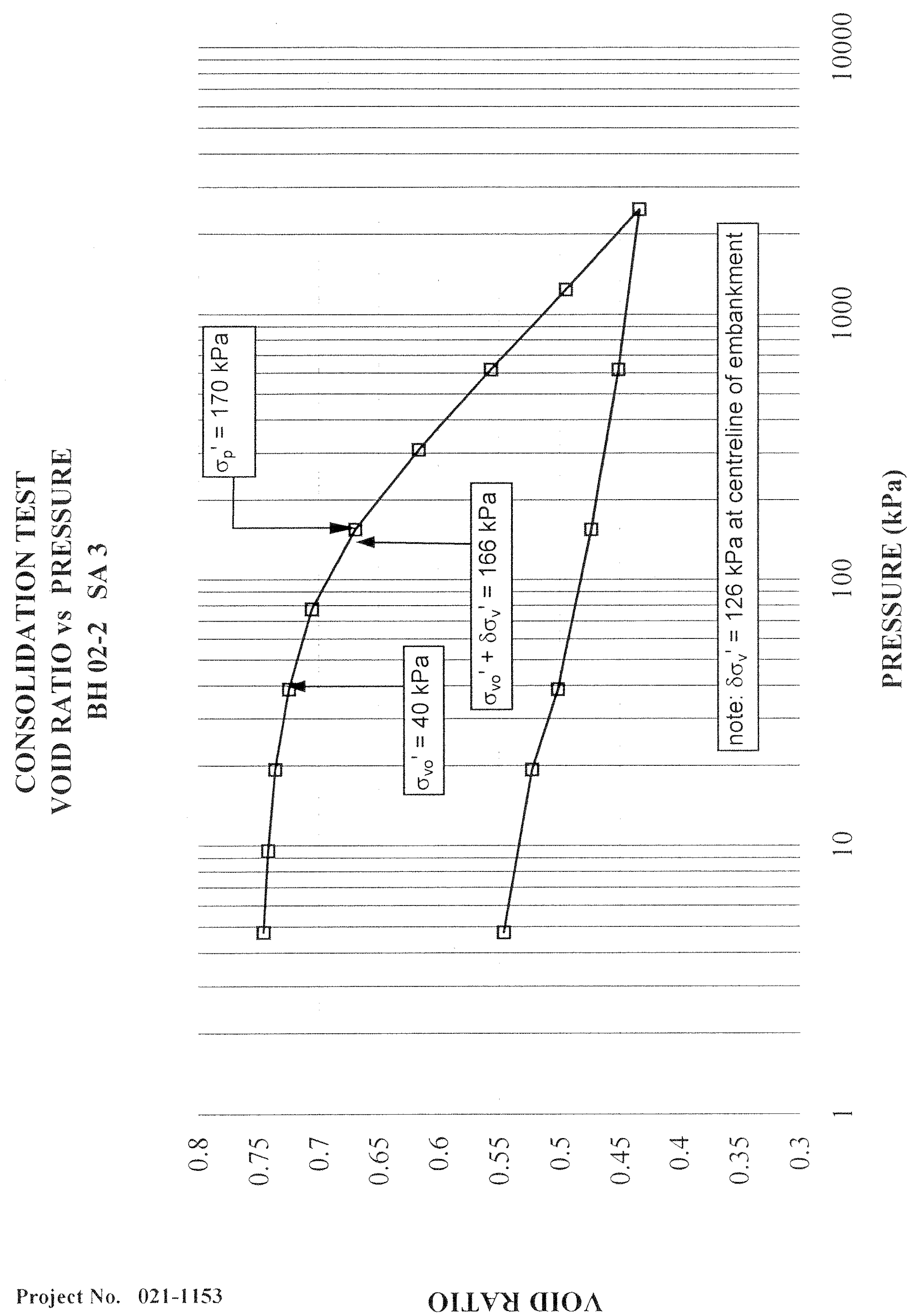
Silty Clay to Clayey Silt

FIG No. 5

Project No. 021-1153

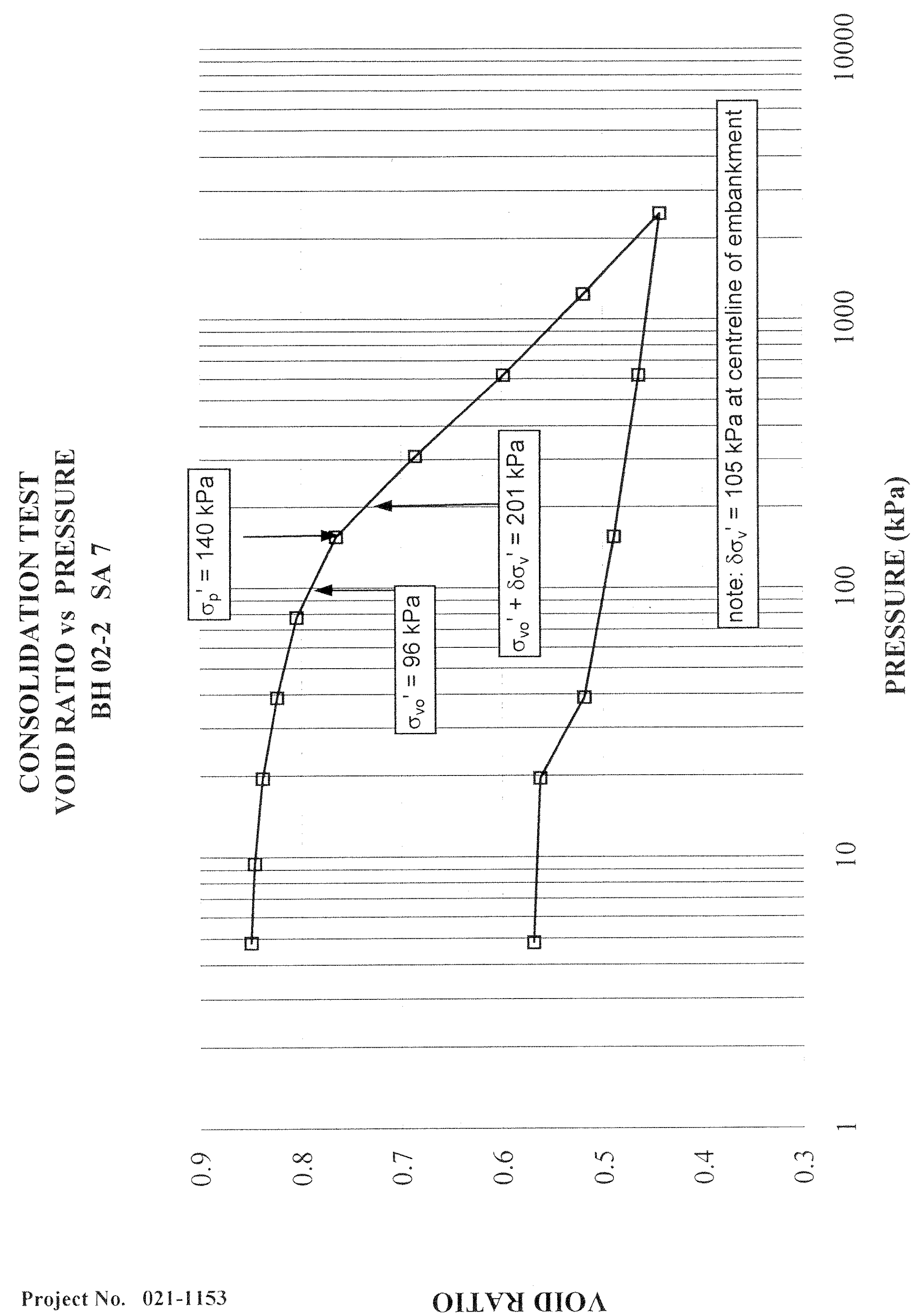
CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE 6



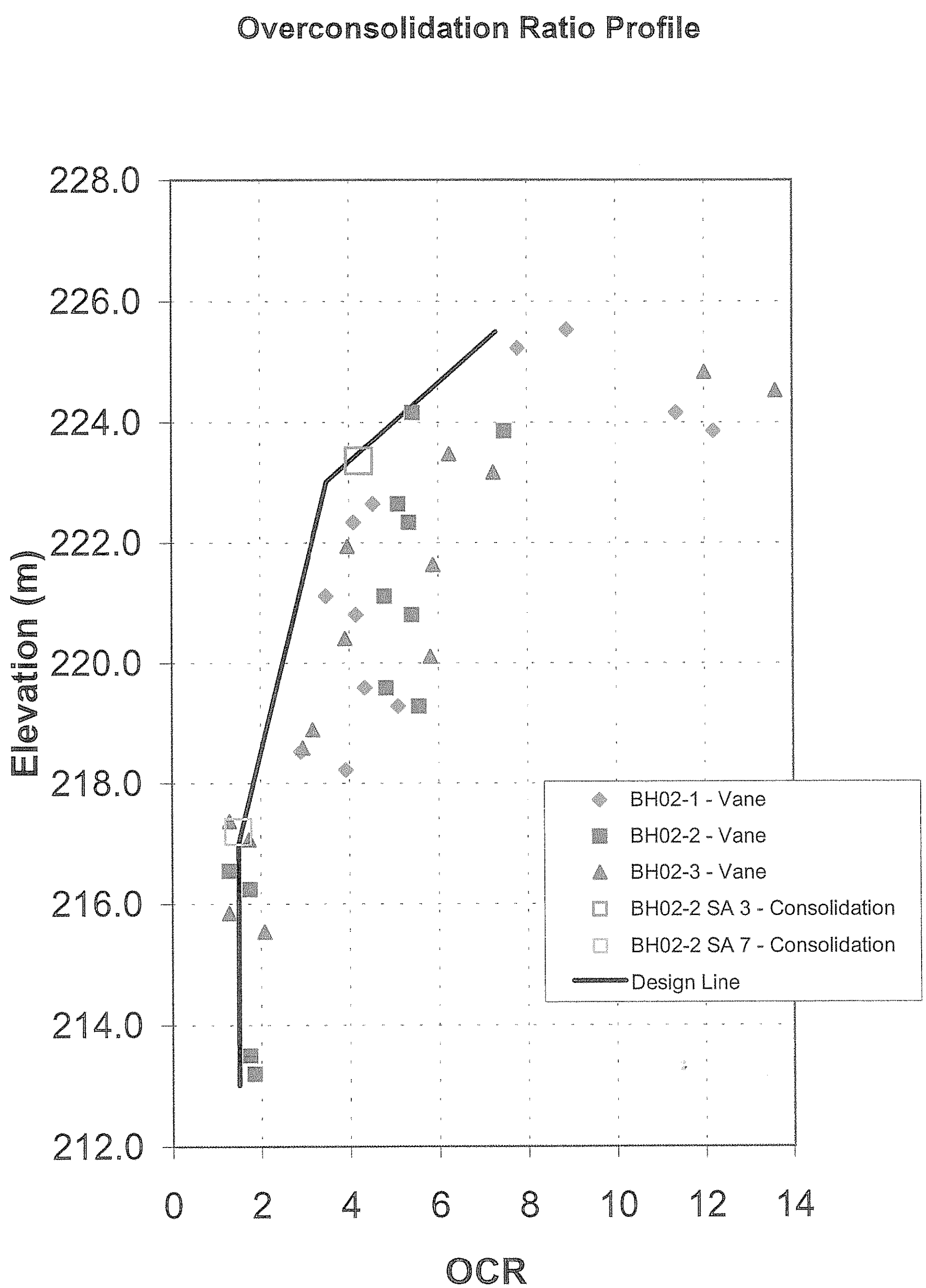
Project No. 021-1153

CONSOLIDATION TEST VOID RATIO VS. LOG PRESSURE	FIGURE 7



Project No. 021-1153

Overconsolidation Ratio Stations 15+565 to 16+605	Figure 7A

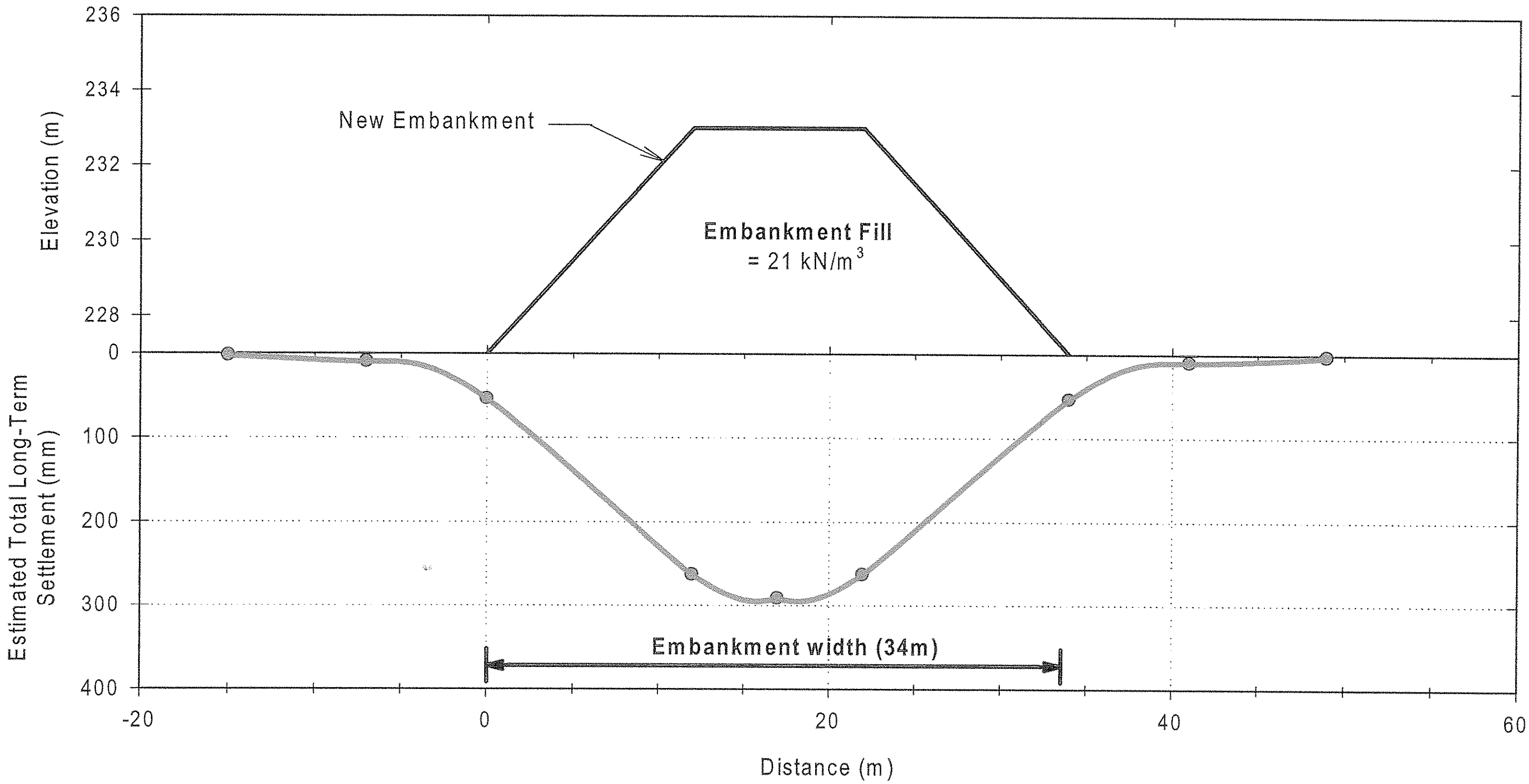


Date: November 2002
Project: 021-1153

Drawn: DKB
Checked: ASP

Golder Associates Ltd.

Estimated Settlement Along Proposed Embankment Stations 16+565 to 16+605	Figure 8



Project: 021-1153
Date: November 2002

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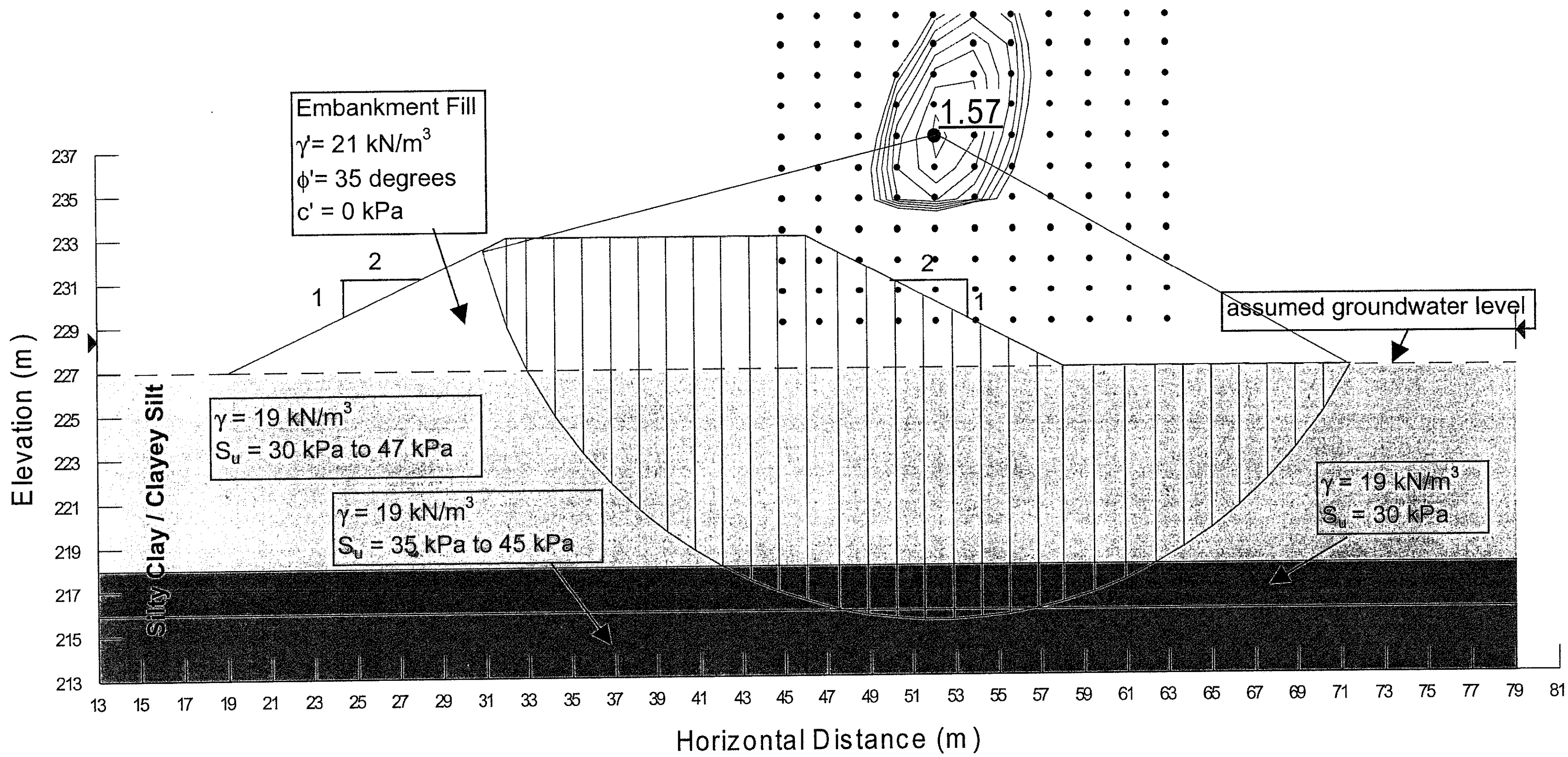
Drawn by: CN
Checked by: DKB

Stability Assessment
Proposed Embankment
Stations 16+565 to 16+605

Figure 9



Project: 021-1153
Date: November 2002

Golder Associates Ltd.

Drawn by: DKB
Checked by: ASP

APPENDIX A

RELEVANT RECORD OF TEST PITS
(GOLDER NO. 021-8014, DATED OCTOBER 2002)

Highway 11 Upgrading, Poplar Rapids River Bridge, Site 39W-001
Station 15+100 to 17+300, Referenced to C/L

16+575 26.50 Lt of C/L D+200 TP

0	- 900	Blk Org
900	- 3.00	Gry Br Si Cl Tr Sa Tr Gr , Tr Sea Shells, Moist, Soft to Firm

16+579 C/L D-0 TP

0	- 800	Blk Org
800	- 1.70	Br Si Cl W Sa Tr Gr, Moist, Firm
1.70	- 6.30	Gry Si Cl Tr Sa Tr Gr , Tr Sea Shells, Moist, Soft to Firm

16+584 27.00 Rt of C/L D-400 TP

0	- 500	Blk Org
500	- 1.40	Br Si Cl W Sa Tr Gr, Moist, Firm
1.40	- 3.00	Gry Si Cl Tr Sa Tr Gr , Tr Sea Shells, Moist, Soft to Firm

Note: Dimensions of Test Pits = 1.2 m by 3 m

APPENDIX B

CONSOLIDATION TEST DATA

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	021-1153	Sample Number	3
Borehole Number	02-2	Sample Depth, m	3.0-3.5

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	6		
Date Started	02-10-09		
Date Completed	02-10-19		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m^3	19.52
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	15.34
Area, cm^2	31.61	Specific Gravity, measured	2.74
Volume, cm^3	60.37	Solids Height, cm	1.092
Water Content, %	27.23	Volume of Solids, cm^3	34.53
Wet Mass, g	120.16	Volume of Voids, cm^3	25.84
Dry Mass, g	94.44	Degree of Saturation, %	99.5

TEST COMPUTATIONS

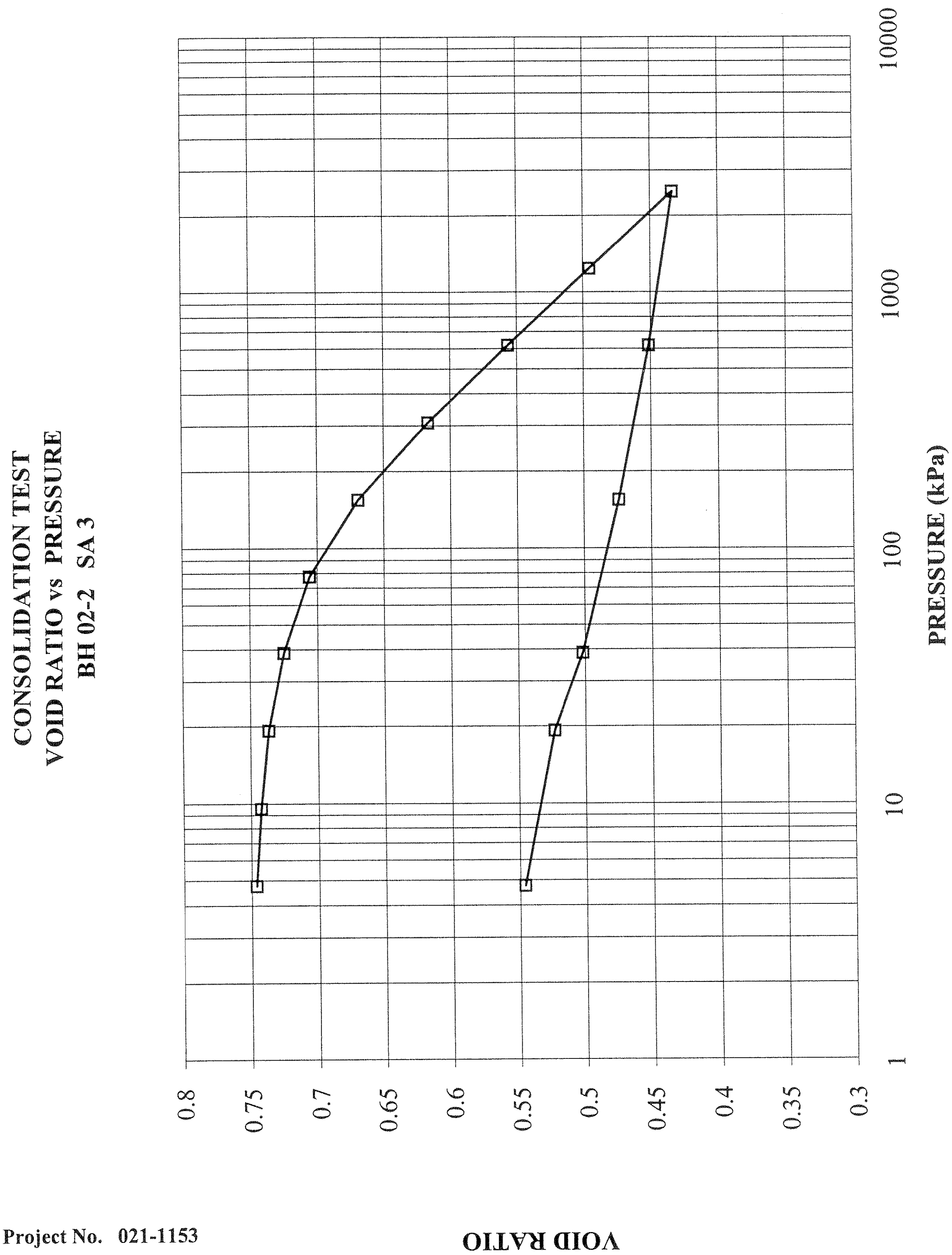
Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.910	0.748	1.910				
4.76	1.908	0.746	1.909	15	5.15E-02	2.64E-04	1.33E-06
9.56	1.904	0.742	1.906	76	1.01E-02	4.47E-04	4.44E-07
19.29	1.897	0.737	1.900	243	3.15E-03	3.50E-04	1.08E-07
38.75	1.884	0.725	1.891	529	1.43E-03	3.42E-04	4.80E-08
77.53	1.863	0.706	1.874	540	1.38E-03	2.84E-04	3.83E-08
154.98	1.824	0.669	1.843	454	1.59E-03	2.68E-04	4.17E-08
308.78	1.766	0.617	1.795	844	8.09E-04	1.96E-04	1.55E-08
618.53	1.700	0.557	1.733	735	8.66E-04	1.11E-04	9.42E-09
1238.65	1.633	0.495	1.667	356	1.65E-03	5.67E-05	9.20E-09
2477.57	1.566	0.434	1.600	240	2.26E-03	2.84E-05	6.28E-09
618.53	1.585	0.451	1.576				
154.98	1.610	0.474	1.598				
38.75	1.641	0.502	1.625				
19.29	1.664	0.523	1.652				
4.76	1.689	0.546	1.677				

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

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.69	Unit Weight, kN/m^3	20.94
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	17.35
Area, cm^2	31.61	Specific Gravity, measured	2.74
Volume, cm^3	53.39	Solids Height, cm	1.092
Water Content, %	20.71	Volume of Solids, cm^3	34.53
Wet Mass, g	114.00	Volume of Voids, cm^3	18.86
Dry Mass, g	94.44		

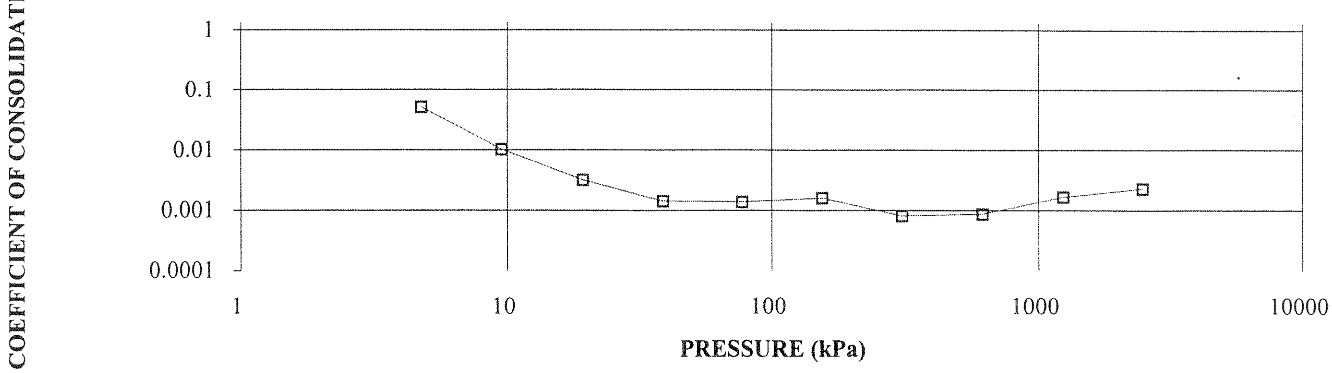
CONSOLIDATION TEST VOID RATIO VS. LOG PRESSURE	FIGURE



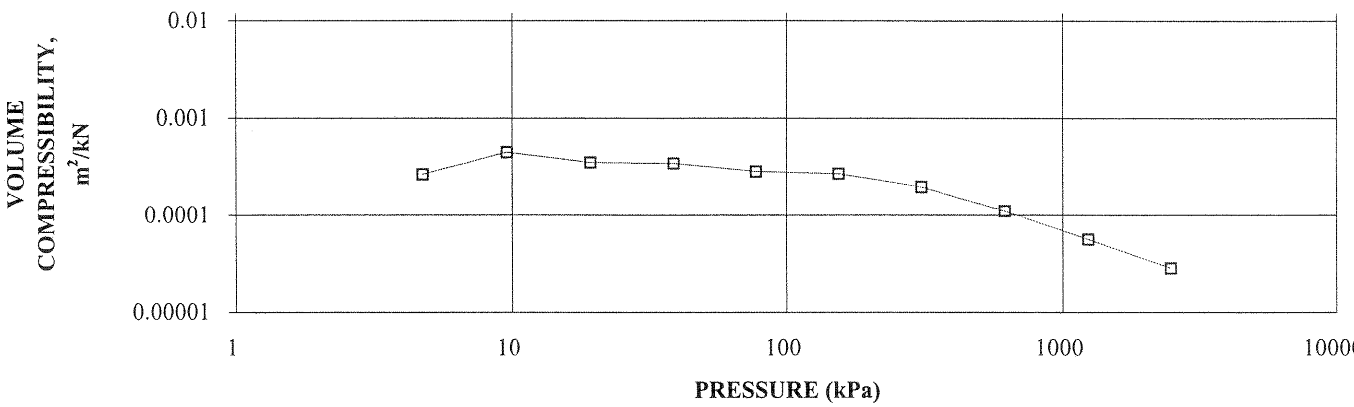
Project No. 021-1153

OEDOMETER CONSOLIDATION SUMMARY

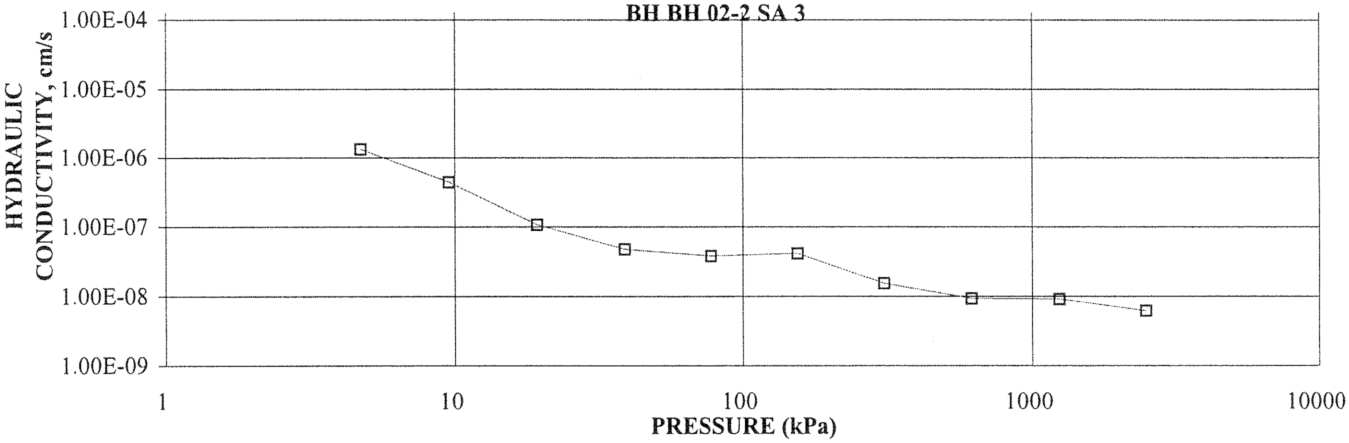
CONSOLIDATION TEST
cv cm^2/s vs PRESSURE (kPa)
BH 02-2 SA 3



CONSOLIDATION TEST
mv, m^2/kN vs PRESSURE (kPa)
BH 02-2 SA 3

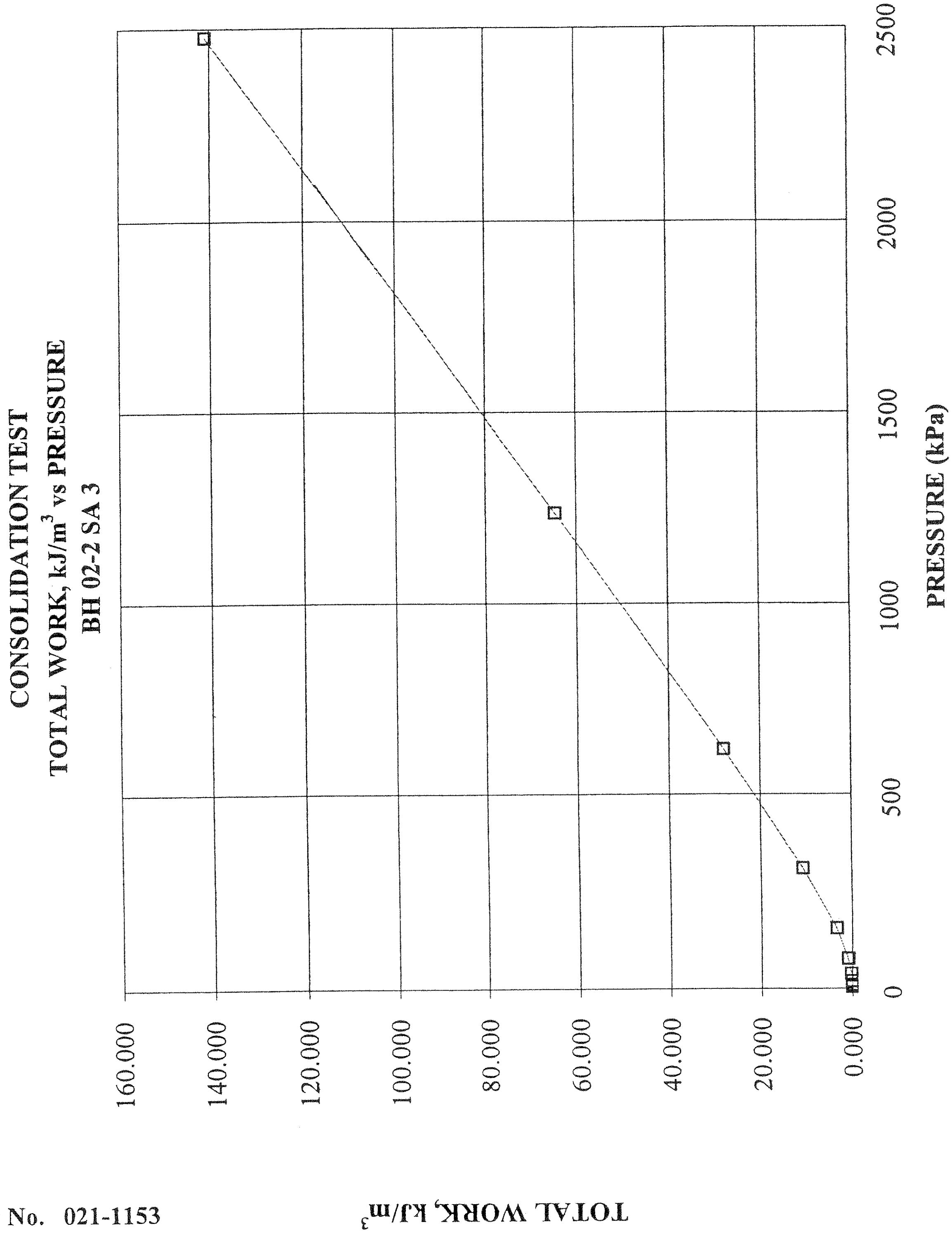


CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH BH 02-2 SA 3



Project No. **021-1153**

FIGURE



Project No. 021-1153

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	021-1153	Sample Number	7
Borehole Number	02-2	Sample Depth, m	9.1-9.6

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	02-10-09		
Date Completed	02-10-20		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m^3	18.90
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	14.35
Area, cm^2	31.67	Specific Gravity, measured	2.71
Volume, cm^3	60.49	Solids Height, cm	1.031
Water Content, %	31.72	Volume of Solids, cm^3	32.67
Wet Mass, g	116.56	Volume of Voids, cm^3	27.82
Dry Mass, g	88.49	Degree of Saturation, %	100.9

TEST COMPUTATIONS

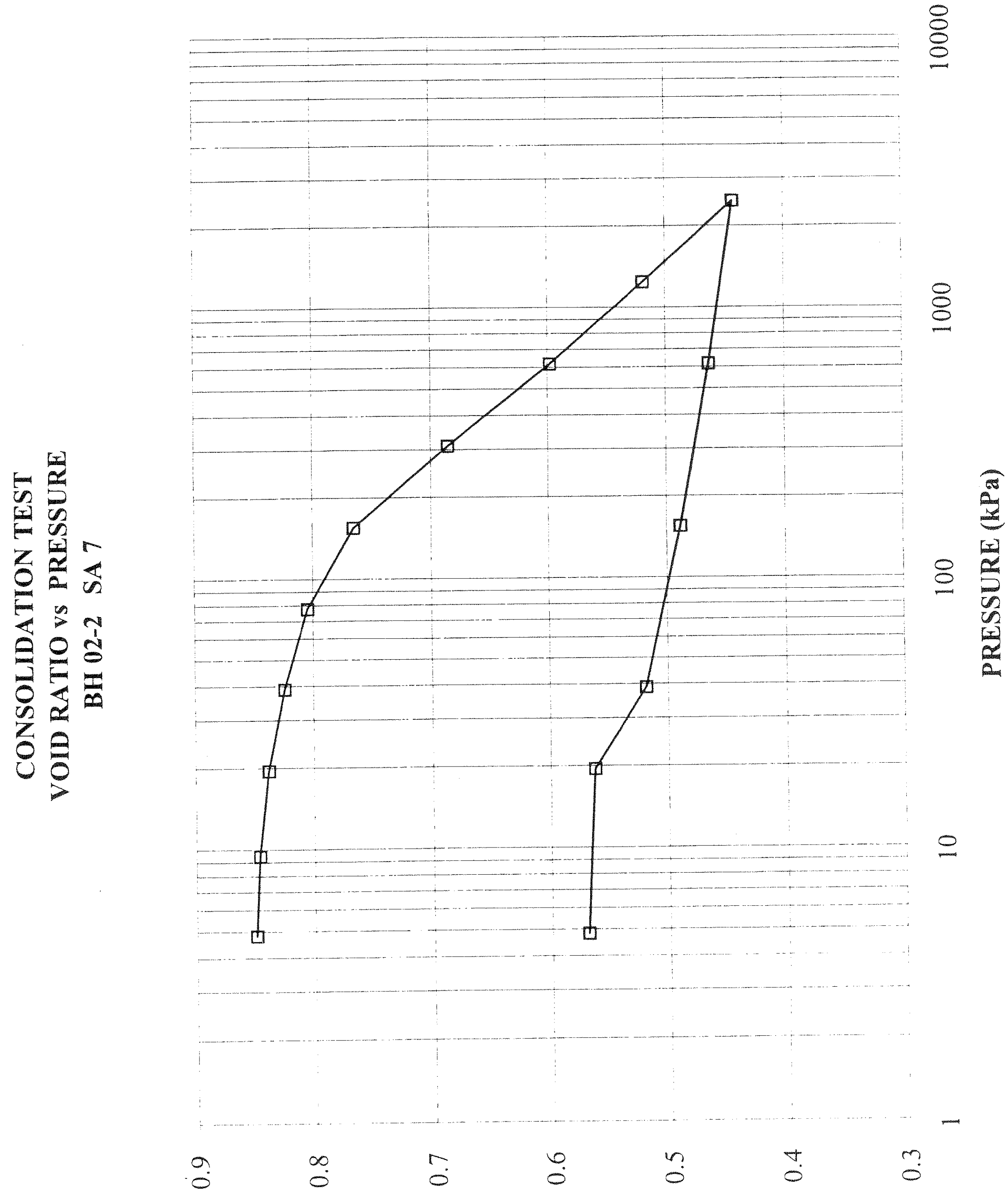
Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.910	0.852	1.910				
4.82	1.908	0.850	1.909	85	9.09E-03	2.28E-04	2.03E-07
9.45	1.905	0.847	1.906	135	5.71E-03	3.73E-04	2.09E-07
19.49	1.896	0.838	1.900	394	1.94E-03	4.38E-04	8.34E-08
38.88	1.882	0.824	1.889	475	1.59E-03	3.89E-04	6.07E-08
77.52	1.861	0.805	1.872	496	1.50E-03	2.76E-04	4.06E-08
154.78	1.821	0.766	1.841	375	1.92E-03	2.72E-04	5.12E-08
309.02	1.739	0.686	1.780	844	7.96E-04	2.79E-04	2.18E-08
618.06	1.649	0.598	1.694	1052	5.78E-04	1.53E-04	8.68E-09
1238.65	1.567	0.519	1.608	518	1.06E-03	6.90E-05	7.15E-09
2473.03	1.488	0.443	1.528	394	1.26E-03	3.33E-05	4.09E-09
618.06	1.510	0.464	1.499				
154.78	1.536	0.489	1.523				
38.88	1.567	0.519	1.551				
19.49	1.612	0.563	1.589				
4.82	1.619	0.569	1.615				

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

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.62	Unit Weight, kN/m^3	20.72
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	16.93
Area, cm^2	31.67	Specific Gravity, measured	2.71
Volume, cm^3	51.26	Solids Height, cm	1.031
Water Content, %	22.40	Volume of Solids, cm^3	32.67
Wet Mass, g	108.31	Volume of Voids, cm^3	18.59
Dry Mass, g	88.49		

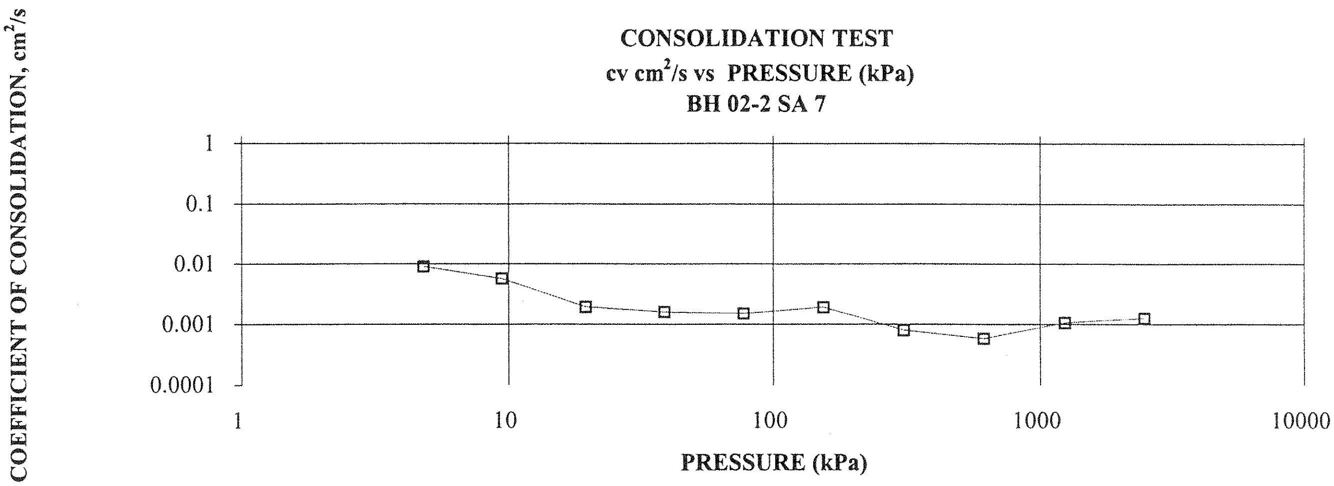
CONSOLIDATION TEST VOID RATIO VS. LOG PRESSURE	FIGURE



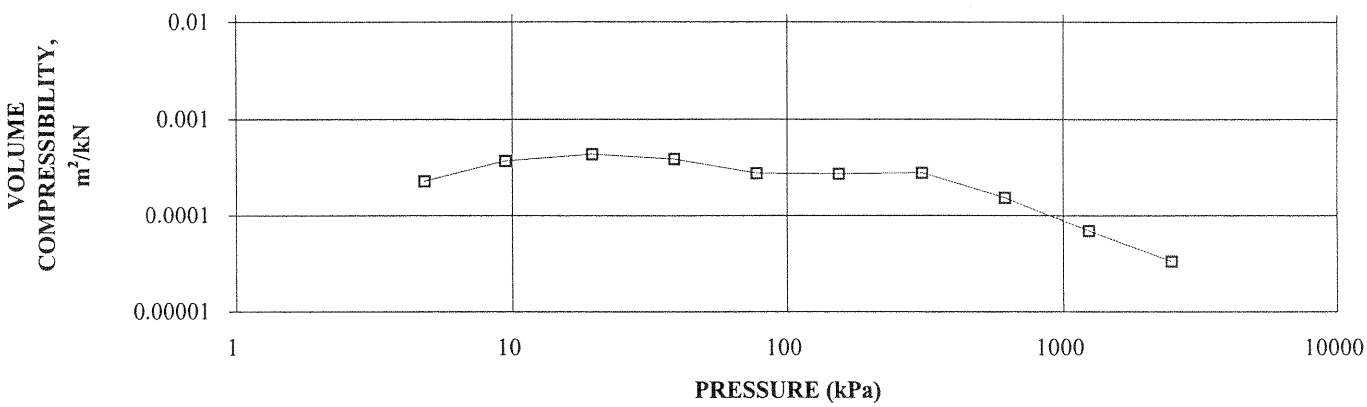
Project No. 021-1153

OEDOMETER CONSOLIDATION SUMMARY

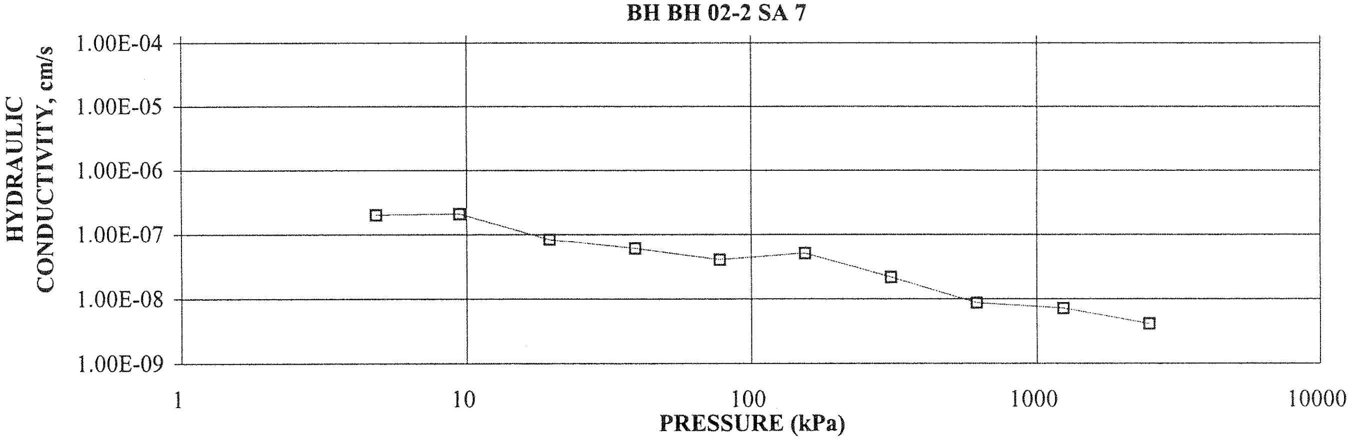
CONSOLIDATION TEST
cv cm^2/s vs PRESSURE (kPa)
BH 02-2 SA 7



CONSOLIDATION TEST
mv, m^2/kN vs PRESSURE (kPa)
BH 02-2 SA 7

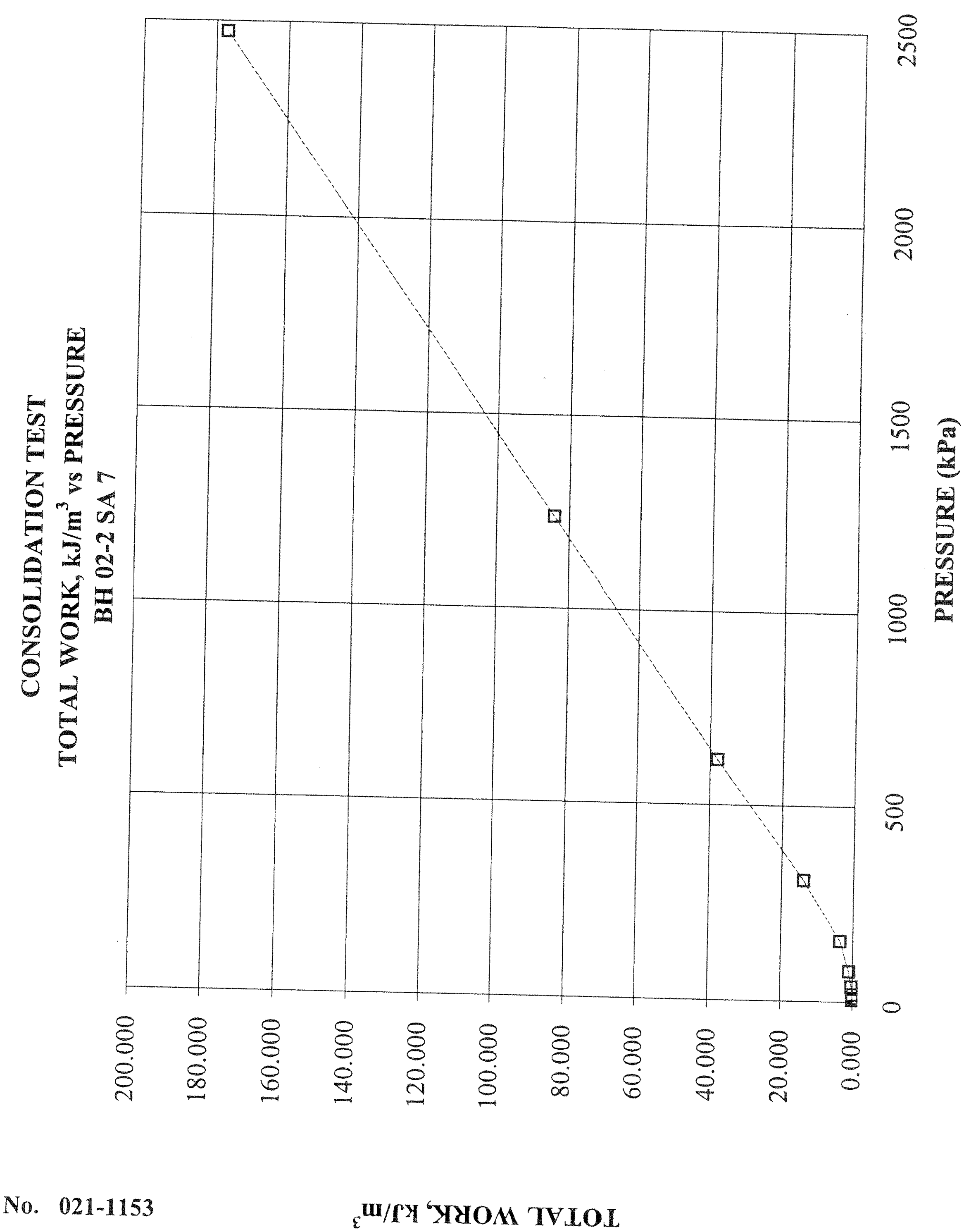


CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH BH 02-2 SA 7

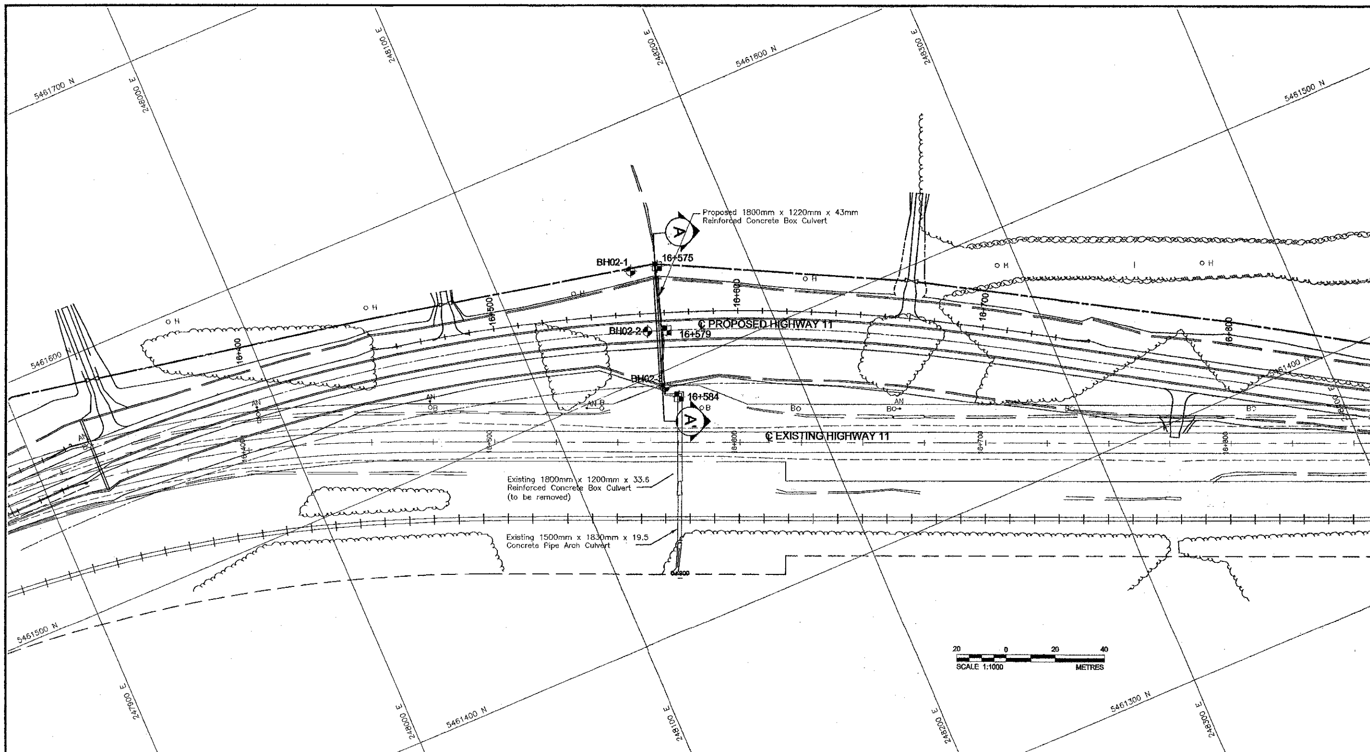


Project No. **021-1153**

FIGURE

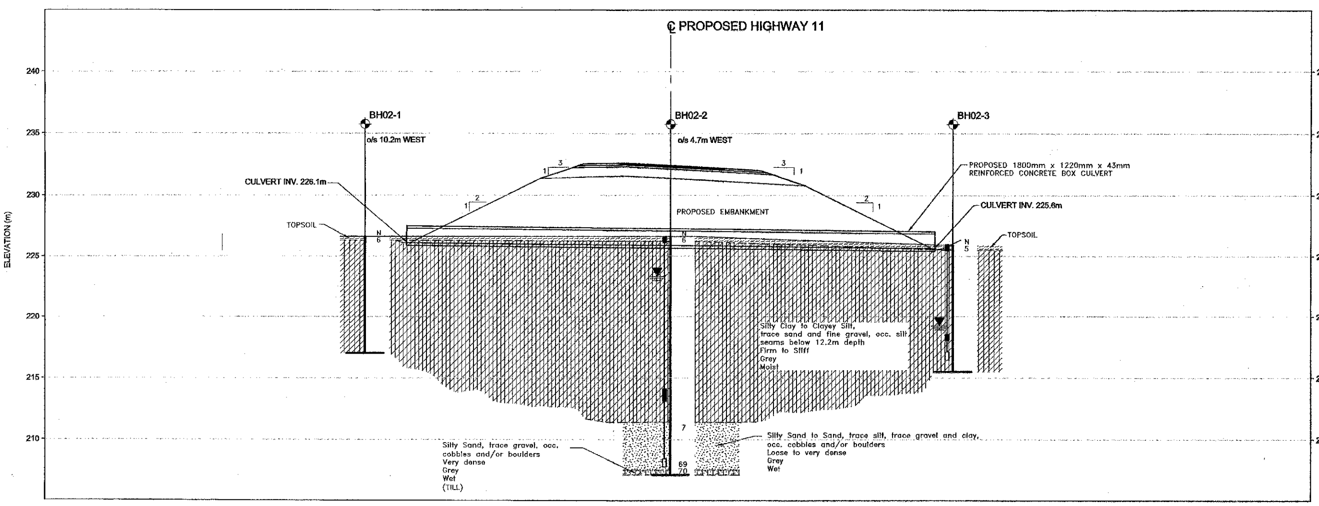


Project No. 021-1153



METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



A-A' CROSS SECTION AT STATION 16+575

SCALE 1:200 METRES

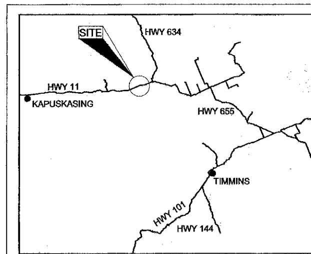
DIST. 53 HWY. 11
CONT No.
WP No. 167-98-00

PROP. HIGH FILL & CULVERT
STATIONS 16+565 TO 16+605
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

LEGEND

- Borehole
- Test Pit, Golder Associates Ltd. Geotechnical Investigation, Report No. 021-8014, dated October 2002.
- Seal
- Piezometer
- N Standard Penetration Test value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, November 15, 2002
- WL upon completion of drilling

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
BH02-1	226.6	5461542.7	248153.6
BH02-2	226.6	5461517.4	248150.4
BH02-3	225.9	5461493.8	248148.1
		STATION	OFFSET
16+575	–	16+575	26.5m Lt
16+579	–	16+579	0m
16+584	–	16+584	27m Rt

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

REFERENCE
Drawing provided by McCormick Rankin drawing file No. al112-feb07-03.dwg received February 7, 2003.

NO.	DATE	BY	REVISION

Geocres No.

HWY. 11		PROJECT NO. 021-1153	DIST. 53
SUBM'D. DKB	CHKD. DKB	DATE: FEBRUARY 2003	SITE:
DRAWN: JFC	CHKD. ASP	APPD. FJH	DWG. 1