

UPDATED
FOUNDATION INVESTIGATION AND DESIGN REPORT
BERNARD CREEK BRIDGE NBL
HIGHWAY 11, BURK'S FALLS TO SOUTH RIVER
G.W.P. 742-93-00, W.P. 755-93-01, SITE 44-99

Geocres Number: 31E-205

Report to

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**FOUNDATION INVESTIGATION REPORT
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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of a proposed bridge to carry the proposed Northbound Lane (NBL) of Highway 11 over the Bernard Creek. No preliminary foundations data was available for this site.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, borehole logs, stratigraphic profile and cross-sections and a written description of the subsurface conditions. A model of the subsurface conditions was developed based on the data obtained in the course of the present investigation. This model describes the geotechnical conditions influencing design and construction of the foundations and approach embankments for the bridge.

Thurber carried out the investigation as a sub-consultant to Marshall Macklin Monaghan, under the Ministry of Transportation Ontario (MTO) Agreement Number 5005-A-000188.

2 SITE DESCRIPTION

The proposed bridge will replace the existing bridge that carries Highway 11 over Bernard Creek and will lie approximately on the same horizontal alignment. A twin structure will be built approximately 40 m to the west to carry the Highway 11 SBL over Bernard Creek. The site lies in Strong Township, approximately 500 m south of the intersection of Highway 11 with Robins Road and Black Creek Road, and approximately 8km southwest of Sundridge.

The site is drained by the Bernard Creek, which locally flows westward towards Stirling Creek, approximately 500m west of the bridge site. The creek channel at the bridge site is shallow, apparently less than 1m deep, approximately 8 m wide and the flood plain is broad and indistinct, merging into a wide, low-lying plain.

The general site area is located within the physiographic region known as the Canadian Shield, characterized by Pre-Cambrian bedrock typically occurring as rounded knobs and ridges where exposed. Locally, however, the site lies in a comparatively flat area, with land rising, and bedrock

outcropping, a few hundred metres to the east. The site is mapped by Chapman and Putnam¹ as lying at the south end of an area of clay plain, but generally within an area of discontinuous outwash plain that characterizes much of the Highway 11 corridor south of North Bay.

The site is mostly surrounded by farmland. Wooded areas, typically consisting of spruce and cedar, are present to the south of Bernard Creek. Near Bernard Creek the land is covered by thick bush.

There is a seasonal trailer park and associated buildings approximately 50m south of the creek and east of existing Highway 11.

3 SITE INVESTIGATION AND FIELD TESTING

Site investigation and field testing was carried out for the NBL structure between February 17 and March 10, 2004, and for the adjacent SBL structure between January 8 and 12, 2004. Due to the proximity of the structures, all boreholes drilled for these two structures have been taken into account in the preparation of this report.

The positions of the boreholes most relevant to this structure site, their depths and elevations are as shown in Table 3.1.

Table 3.1 – Borehole Locations Relative to Structure

Location on Structure	Relevant Boreholes	Borehole Depth (m)	Elevation	
			OG	EOH
South Approach	BH 99N-1	8.2	312.2	304.0
South Abutment	BH 99N-2	40.2	312.0	271.8
North Abutment	BH 99N-5	27.4	312.5	285.1
	BH 99S-5	38.2	312.2	274.0
North Approach	BH 99N-6	8.2	312.2	304.0

OG=Original Ground

EOH=End of Hole

Site access is restricted by the presence of constantly inundated ditches, fences overhead, buried utility lines and by the widespread and poorly defined creek margins. On account of these difficulties encountered in accessing the originally planned drilling sites, several of the boreholes were relocated further from the anticipated structure than originally planned. The locations of the boreholes, as actually drilled, are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix F.

Surveyors from Marshall Macklin Monaghan Ltd. marked the borehole locations in the field and utility clearances were obtained by Thurber prior to any drilling being carried out.

George Downing Estate Drilling and All-Terrain Drilling both supplied and operated the drilling and sampling equipment used for investigation program at Bernard Creek. Both companies

¹ L.J. Chapman and D.F. Putnam, "The Physiography of Southern Ontario, Third Edition", Ontario Geological Survey Special Volume 2, 1984.

supplied track mounted CME-75 drill rigs equipped with the necessary drilling and sampling tools. A combination of hollow stem auger, wash boring and rotary drilling techniques were used to advance the boreholes. Samples were obtained using a split spoon sampler in conjunction with Standard Penetration Testing (SPT), thin wall Shelby tubes and diamond core barrels. The consistency of cohesive soils was inferred from a combination of SPT and in situ MTO vane tests. Bedrock was encountered under both abutments of the NBL structure and was proved by diamond coring for depths of 3m. Bedrock was not encountered under the adjacent SBL structure site within the depths investigated and the deeper boreholes were extended to a minimum depth of 3m below refusal as defined by material for which Standard Penetration Tests exceeding 100 blows per 0.3m.

The coordinates and elevations of the boreholes are given on the Borehole Locations and Soil Strata Drawing in Appendix E and on the individual Record of Borehole Sheets in Appendix A.

Standpipe piezometers, consisting of 19 mm PVC pipe with slotted tips, were installed in selected boreholes to monitor the groundwater levels. The locations and completion details for the piezometers are shown in Table 3.2.

Table 3.2 – Piezometer Details

Piezometer Location	Piezometer Details	
	Tip Depth/ Elevation	Completion Details
BH99N-2	38.2m/274.0	Bentonite seal from 40.2m (bottom of borehole) to 32.0m. 19mm standpipe piezometer with 1.5m slotted tip installed at 32.0m; filter sand from 32.0m to 26.8m; bentonite seal from 26.8m to 25.9m and grout to ground surface.
BH99N-5	27.4m/284.8	19mm standpipe piezometer with 1.5 m slotted tip installed at 27.4m. Sand backfill from 27.4m to 25.6m; bentonite plug from 25.6m to 23.8m (top of bedrock) and grout to ground surface.
BH99S-2	38.2m/274.0	19mm standpipe piezometer with 1.5m slotted tip installed at bottom of borehole; filter sand from 38.2m to 36.3m; the borehole walls collapsed around the standpipe to 10m depth prior to placement of bentonite seal above the sand filter. Bentonite seal from 10m to 8.2m and from 4.3 to ground surface. Clay cuttings from 8.2m to 4.3m.
BH99S-5	4.3m/307.9	19mm standpipe piezometer with 1.5 m slotted tip installed at 4.3m. Sand backfill from 4.3m to 2.4m; bentonite plug from 2.4m to 1.8m and from 0.6m to ground surface; clay cuttings from 1.8m to 0.6m.

Members of Thurber's technical staff supervised the drilling and sampling operations on a full time basis. The supervisors logged the boreholes and the recovered samples and processed them for transport to Thurber's Oakville office.

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification and to natural moisture content

determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A.

Selected samples were subjected to gradation analysis (sieve and hydrometer) and Atterberg Limit testing. A total of thirteen samples were selected for these tests. In addition, two samples were selected for oedometer testing from boreholes drilled at the adjacent SBL structure and the results extrapolated to the NBL structure.

The test results are shown on the Record of Borehole sheets in Appendix A and on the charts in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

5.1 General

Reference is made to the Record of Borehole sheets in Appendix A and to the Borehole Locations and Soil Strata Drawings in Appendix E. Details of the encountered soil stratigraphy are presented in these appendices. An overall description of the stratigraphy is given in the following paragraphs however the factual data presented in the borehole logs governs any interpretation of the site conditions.

The site of the proposed NBL structure lies in the Bernard Creek flood plain with original ground elevations ranging from 312.0 to 312.5 and encompasses the footprint of the existing Highway 11 structure.

The subsoil stratigraphy at the site is governed by the physiography and glacial history of the area. The recent geological history of the site is interpreted to consist of:

- Glaciation and scouring of the bedrock
- Glacial deposition of a sand (till) with cobbles and boulders
- Deposition of a layer of sandy silt to clayey silt, and a thick layer of silty clay, possibly in a glacial lake environment
- Subsequent deposition of silt and sand, possibly in a glacial/post-glacial outwash environment, with recent modification of the near surface soils by Bernard Creek

In general terms, the site was found to be mantled by a layer of topsoil up to 0.8m thick which is underlain by sand, silt, a thick deposit of silty clay and a lower layer of silt. This sequence is, in turn, underlain by sand (till) and then by bedrock. Although not penetrated by a borehole, asphalt paving and granular base materials occur within the existing Highway 11 pavement.

More detailed descriptions of the individual strata are presented below.

5.2 Topsoil

Topsoil was encountered only at the south approach to a depth of 0.1 m. The remainder of the boreholes for the NBL structure did not encounter topsoil. The thickness of topsoil may vary across the site and the data is not intended for the purpose of estimating quantities. In particular, the boreholes at the north abutment and north approach were drilled in the existing ditch where topsoil had been removed. Topsoil should be expected across the site, beyond the limits of the present road construction.

The topsoil was generally soft, brown and visually assessed to have a high organic content.

5.3 Fill and Existing Structure

Although not penetrated by a borehole, the existing highway pavement exists across the site. Reference should be made to the Pavement Design Report prepared for the project by others for possible information regarding the existing pavement.

An existing bridge structure, possibly including concrete approach slabs, spans Bernard Creek at the site. Reference should be made to any as-built drawings that may exist for details of the construction.

5.4 Silt and Sand

A geologically-recent deposit consisting of silt and sand lies immediately below the topsoil layer and was identified across the entire site. This soil was probably deposited in a glacial lake environment and is characterized by the interbedding of sand and silt layers and intermittent clay seams. The variability of the sequence is consistent with the changing depositional environments understood to occur in glacial and post-glacial lakes. The clay-size content, and plasticity, is generally greater toward the base of the layer and the sand content and occurrences of sand seams increases towards the top. The base of the silt and sand layer extended to depths of 2.8 m (Elevation 309.2) at the south abutment to 3.1 m (Elevation 309.4) at the north abutment.

The silt and sand is classified as very loose to loose, based on SPT values ranging from generally from 2 to 9 blows for 0.3 m of penetration. One SPT value of 17 blows for 0.3 m of penetration near about 1 m depth at the north abutment is attributed to frozen ground. Where cohesive seams were observed within the silt during visual identification, they were described as soft.

The deposit is brown at the top, tending to grey with increasing depth. The measured natural moisture contents ranged from 17 to 30% and the soil is described as saturated.

Typical grain size distributions for this soil are shown in Figure B1-N in Appendix B.

5.5 Silty Clay

A thick silty clay deposit was encountered below the silt and sand. The clay was grey in colour and contained occasional silt and sand lenses. The underside of the silty clay dips from northeast to southwest, based on review of data, including that from the adjacent structure site. Within the footprint of the NBL structure the base of the silty clay varies from a depth of 28.3 m (Elevation 283.7) at the south abutment to 20.5 m (Elevation 292.0) at the north abutment. The thickness of the clay layer varies from 21.1 m close to the south abutment to 14.4 m at the north abutment. The silty clay was not fully penetrated at the approaches.

Hydrometer test results showed that the silty clay consists typically of 35% to 80% of silt and 15% to 60% clay. Typical grain size distributions for this silty clay are shown in Figures B3-S and B3-N in Appendix B.

The natural water content in the top portion of the silty clay, above approximate Elevation 299, was in the order of 30%. Below Elevation 299 the water content ranged from 45% to 75%, with values lower than 45% encountered in sand and silt lenses.

Liquid Limit values ranged from 18% to 43% and were typically lower than the natural moisture content possibly reflecting a structured deposit. Plastic Limit values ranged from 16% to 22%. According to the Modified Unified Soil Classification System and the Plasticity Chart in Figure BN-4 and B6-S, Appendix B, the silty clay is classified as CL to CI with occasional OL seams.

Attached Figure 1 shows a detailed summary of the SPT and undrained shear strength (S_u) values obtained in the silty clay at the NBL structure. Figure 2 shows the same information for the silty clay at the adjacent SBL structure. An analysis of Figure 1 indicates that the S_u values ranged from 30 kPa to 75 kPa with average value in the order of 50 kPa. Figure 2 shows values of S_u ranging from 30 to 60 kPa, with some higher values at greater depth. The higher values of S_u are probably related to sand and silt lenses. Based on the undrained shear strength values from in situ vane tests, the silty clay can be described as firm to stiff.

Oedometer tests carried out on silty clay samples collected from Elevation 293.5.3m and Elevation 295, at Boreholes 99S-2 and 99S-5, respectively, at the SBL structure are summarized in Appendix B and in the Table 5.1.

The oedometer test results indicate that the samples collected from Boreholes BH 99S-2 and BH 99S-5, at 18.2m (EL.294.0) and 17.1m (EL.295.1) depth, respectively, were normally to slightly under-consolidated.

Table 5.1 – Oedometer Test Results

Borehole log/Sample Depth	BH 99S-2 EL.294.0 / 18.2m depth South Abutment	BH 99S-5 EL.295.1/ 17.1m depth North Abutment
Initial Void Ratio	1.573	1.165
Preconsolidation Pressure (p' - kPa)	117	103
In situ vertical Effective stress (σ_v' - kPa)	117	118
Over Consolidation Ratio (OCR)	1	<1
Compression Index – Virgin Curve (C_c)	0.74	0.33
Compression Index – Rebound Curve (C_r)	0.15	0.05

5.6 Clayey Silt

A stratum of clayey silt underlies the silty clay. The silt contains clay content ranging from clayey to some clay. The thickness of this layer diminishes 3.4 m at the south abutment to 1.7 m at the north abutment. The underside of the clayey silt layer occurred at depths of 32.4 m (Elevation 279.8) at the south abutment and 27.9 m (Elevation 284.1) at the north abutment.

Based on SPT values ranging from 3 to 9, the clayey silt is described as soft to stiff.

The clayey silt is grey in colour and the measured natural moisture contents ranged from 26% to 43%.

A grain size distribution curve for this soil is shown in Figure B4-S, in Appendix B.

5.7 Sand Till

A layer of glacial till described as sand, trace gravel, trace silt and occasional cobbles and boulders underlies the silt deposit. Boulders were encountered near the top of the sand layer at Elevation 278.3 at the south abutment and Elevation 284.3 at the north. The thicknesses of these boulders, measured along the axis of the borehole, were 0.8 and 0.3m, respectively.

Additional cobbles and boulders were inferred in this deposit from the behaviour of the response of the drill string as it advanced. Boulders and boulders were penetrated by coring between depths of 37.1 and 38.0m (Elevation 275.1 to 274.0) at the south abutment.

This layer should be assumed to contain many more cobbles and boulders than what was positively identified in the course of drilling.

Augering was not possible in this stratum due to the presence of boulders and the boreholes

were advanced using tri-cone and diamond coring techniques in order to penetrate the boulders.

The SPT values in the sand till were in excess of 50 for 100 mm penetration, with the exception of one SPT value of 21 at the north abutment at Elevation 281.5. This indicates that the sand is generally very dense.

The measured moisture contents ranged from 11% to 28%.

The deep boreholes drilled along or to the west of centreline of structure terminated in this deposit after proving 3 m of soil with SPT values exceeding 100 blows for 0.3 m of penetration.

5.8 Bedrock

Bedrock was encountered underlying the sand till only in one borehole drilled about the centreline of the proposed highway close to the south abutment but east of the SBL structure. The top of the bedrock lies at depth 36.0 m (Elevation 276.1) at the south abutment and depth 23.5 m (Elevation 289.0) at the north abutment.

The bedrock is described as slightly weathered, granitic gneiss, laminated to thinly banded, pale pink with sub-vertical dark banding. Rough, sub-planar joints were noted, generally with a sub-horizontal orientation. Clayey infilling was noted in some joints.

Both total core recovery (TCR) and solid core recovery (SCR) were 100% at this site. The rock quality designation (RQD) ranged from 74% to 100%.

The compressive strength of the bedrock, assessed by means of point load testing, ranged from 87 to 153 MPa. Based on this range of strengths, the rock is classified as strong to very strong.

5.9 Depths to Refusal

Effective refusal, defined as the equivalent of SPT values exceeding 100 blows for 0.3 m of penetration, was first encountered in the abutment boreholes at a depth of 35.0 m (Elevation 277.0) at the south abutment but not until bedrock was encountered at a depth of 23.5 m (Elevation 289.0) at the north abutment.

5.10 Water Levels

Groundwater levels at this site were measured within standpipe piezometers in the winter of 2003/2004 and 2004/2005 as summarized in Table 5.2. The piezometers were sealed in the sand till at the south abutment and in the silty sand layer at the north abutment.

Table 5.2 – Groundwater Depths (in metres) and Elevations

Date	99N-2		99N-5		99S-2		99S-5	
	Depth	Elev.	Depth	Elev.	Depth	Elev.	Depth	Elev.
Feb 26, 2004	-	-	2.2	310.3	-	-	-	-
Mar 4, 2004	-	-	+0.9	313.4	+1.5	313.7	0.1	312.1
Mar 10, 2004	+1.2	313.2	-	-	-	-	-	-
Mar 11, 2004	+1.2	313.2	+1.1	313.6	+1.5	313.7	0.1	312.1
Feb 23, 2005	+0.8	312.8	+1.1	313.6	+0.45	312.65	-0.45	311.55

The deep piezometers sealed in the sand till layer indicate the presence of artesian piezometric head up to 1.2m above ground surface at the NBL and up to 1.5 m at the SBL. This measurement confirmed the observations of artesian conditions encountered during drilling, when the boreholes were advanced into the sand till. The water level in the shallower piezometer (in the silty sand) was 0.1m below ground surface.

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall. The level of Bernard Creek will also vary seasonally and after severe weather events, leading to flooding of the area around the proposed structure.

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FOUNDATION INVESTIGATION AND DESIGN REPORT**BERNARD CREEK BRIDGE NBL****HIGHWAY 11, BURK'S FALLS TO SOUTH RIVER****G.W.P. 742-93-00, W.P. 755-93-01, SITE 44-99****Geocres Number: 31E-205****PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS****6 INTRODUCTION**

This part of the report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approach fills for the proposed structure.

The proposed structure will consist of a single-span structure, approximately 26m long, with two foundation elements: one at each abutment. The approaches to the bridge will consist of embankments up to 4m high to the top of pavement, founded on loose non-plastic sediments underlain by compressible silty clay soils. The new approaches will be formed, in part, by constructing a grade raise of approximately 2 m on top of the existing Highway 11 embankment. The new approach embankment will, however, be wider than the existing, particularly to the north of the structure. The widened portion of the embankment will be constructed on soil that has not previously been loaded by an embankment, leading to the possibility of differential settlement across the width of the embankment.

The stability and time-dependent settlements are also analysed in this section of the report and an assessment of the impact of the construction schedule is presented.

The discussion and recommendations presented in this report are based on our understanding of the project and on the factual data obtained in the course of the investigation.

There were no visible signs of excessive settlement of the bridge and its approach embankments during the investigation program described in this report.

7 STRUCTURE FOUNDATIONS**7.1 Foundation Alternatives**

A comparison of foundation alternatives is presented in Table 7.1. As shown in this comparison, analysis of the subsurface conditions indicates that the bridge support should be provided by deep foundation elements.

Spread footings founded at shallow depths are not considered a suitable alternative due to



the presence of deep compressible cohesive soils and associated potential for long-term time-dependent settlements. Drilled shafts (also referred to as cast-in-place piles or caissons) are also not considered feasible due to the difficulties associated with installation through relatively soft soils into water bearing cohesionless soils containing cobbles and boulders. Therefore it is recommended that the bridge abutments be supported on steel H-piles driven to bedrock.

7.2 Pile Resistance

Steel piles should be founded on the bedrock. The piles should be designed on the basis of the concentric, axial geotechnical resistances given in Table 7.2.

Table 7.2 – Pile Factored Geotechnical Axial Resistance (ULS)

Pile	Factored Geotechnical Axial Resistance (ULS)
HP 310 X 110	2,000 kN
HP 310 X 132	2,400 kN
HP 310 X 152	2,750 kN
HP 360 X 132	2,400 kN

The geotechnical resistance is limited by the structural resistance.

The pile tip elevations shown in Table 7.3 should be used for cost estimating purposes.

Table 7.3 Anticipated Pile Tip Elevations

Foundation Element	Anticipated Pile Tip Elevation (m)
South Abutment	276.1
North Abutment	289.0

7.3 Pile Tips

Due to the presence of cobbles and boulders in the sand till layer the tips of all piles should be fitted with H-section rock points from approved manufacturers such as Titus Steel (Standard H-Point) or Associated Pile & Fitting Corp. (APF Hard Bite).

Table 7.1 – Comparison of Foundation Types

	Spread Footings			Driven Steel Piles		Drilled Shafts	
	Ease of construction			Easy to install and commonly available and used in Ontario Permits integral abutments.		High capacity	
Advantages							
Disadvantages							
	High potential for long term settlements. Precludes use of integral abutments.			Relatively high downdrag forces. Piles can only be installed after most lateral ground displacement due to consolidation has been completed. Artesian conditions pose a risk of erosion around the shafts.		Very difficult installation through soft soils into cohesionless, waterbearing soils under artesian conditions. Very difficult to effect quality control for verification of end bearing capacity. Relatively high downdrag forces. Drilled shafts can only be installed after most lateral ground displacement due to consolidation has been completed. Precludes use of integral abutments.	
Risks	High			Medium		High	
Costs	Low			Low to medium		Medium to high	

7.4 Pile Installation

Pile should be driven to bedrock in accordance with Special Provision No. 903S01.

The Contract Documents should contain a NSSP alerting the Contractor to the presence of cobbles and boulders, particularly in the sand till stratum at this site and to the possibility of piles reaching apparent refusal at higher elevations than anticipated. The NSSP should require the QVE to terminate driving before the pile is damaged by overdriving.

7.5 Downdrag

As discussed later in this report, within 60m of the bridge abutments the approach embankments will consist of EPS lightweight fill. Therefore the abutment piles will not be subjected to downdrag forces.

7.6 Shaft Erosion

The presence of water under artesian pressure in the sand till overlying bedrock presents a risk of erosion of fine soil particles around the pile shaft if flow develops.

At this specific site, very loose to loose silt and sand strata near the original ground surface will collapse around the pile shaft and provide a filtering effect that should be sufficient to prevent loss of fines. The filtering will be further augmented if an integral abutment design is selected and the upper 3 m length of the pile shaft is surrounded by sand inside a CSP.

7.7 Lateral Resistance of Piles

The lateral resistance of the piles should be calculated based on the following horizontal coefficients of subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}):

- Sand and silt deposit between EL.312 and EL309:

- $k_s = f \cdot z / D \text{ (MN/m}^3 \text{)}$

Where $f = 2.5 \text{ MN/m}^3$

z = depth below ground surface

D = pile diameter or width in a direction perpendicular to the pile movement

- $p_{ult} = \gamma \cdot z^3 \cdot K_p \text{ (kPa)}$

Where $\gamma = 20 \text{ kN/m}^2$

z = depth below ground surface (m)

$K_p = 3.5$ (passive earth pressure coefficient)

- Silty Clay between El. 309 and El. 292:

- $k_s = 5/D \text{ (MN/m}^3\text{)}$

Where D = pile diameter or width in a direction perpendicular to the pile movement (m)

- $p_{ult} = 9 \cdot C_u = 9 \cdot 40 \text{ kPa} = 360 \text{ kPa}$

- Silty Clay between El. 292 and El. 282:

- $k_s = 8.8/D \text{ (MN/m}^3\text{)}$

Where D = pile diameter or width in a direction perpendicular to the pile movement (m)

- $p_{ult} = 9 \cdot C_u = 9 \cdot 70 \text{ kPa} = 630 \text{ kPa}$

- Assume pile fixity for rotation below EL.282

Spring constant (K_s) and ultimate spring load (P_{ult}) values for numerical analysis of the integral abutment piles can be obtained by multiplying the k_s and p_{ult} values above, respectively, by the pile diameter or width (in a direction perpendicular to the pile movement) and the vertical distance between nodal points of the numerical model mesh along the pile.

7.8 Frost Protection

The depth of earth cover required to provide frost protection for footings and pile caps at this site is 1.8 m.

It is recommended that the full depth of soil cover be provided (and not be reduced) at this site, due to the nature of the native soils and the high groundwater table.

It should be noted that rock fill does not provide the insulation value of soil cover. Where rock fill is used as backfill or for the construction of forward slopes in frost sensitive locations, consideration should be given to incorporating synthetic insulation.

Rigid Extruded Polystyrene insulation may be used and it may be assumed that 25 mm of this insulation provide protection equivalent to 600 mm of soil cover.

7.9 Abutment Type

From a geotechnical perspective, the subsurface conditions at this site are considered to be suitable for the construction of conventional, semi-integral or integral abutments. However, the recommended foundation system consists of H-piles driven to refusal, making integral abutments a feasible option from the geotechnical viewpoint.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. At this site, the upper 3 m of the pile length will lie in the very loose to

loose sand. Therefore a 600mm diameter CSP filled with sand, which is typically installed with integral abutment piles, are not required at this site.

As discussed later in this report the approach embankments to this bridge will consist of EPS lightweight fill. It is understood from conversations with MTO that the use of EPS may preclude the use of integral abutments.

8 EXCAVATION

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils at this site and any earth fill are classed as Type 4 soils. If the soils are effectively unwatered and the free groundwater surface is drawn down and maintained at least 1 m below the level of excavation, the soils may be treated as Type 3.

Unwatering and/or groundwater control will be required prior to excavation below the existing ground surface. The design of the unwatering or groundwater control systems is the responsibility of the Contractor. A NSSP addressing the dewatering requirements prior to excavation at this site should be included in the Contract Documents.

For the purposes of assessing constructability, it is considered feasible to unwater the sand layer near the surface using vacuum well-points. The underlying silt or silty clay is not likely to respond to well-points and groundwater control in these layers, if required, will probably have to be achieved through the use of steel sheet pile cut-offs.

9 APPROACH EMBANKMENTS

9.1 General

The approach embankments to the proposed structure are up to 4m high and will be founded on loose cohesionless deposits underlain by silty clay. The following materials were considered for the construction of the approach embankments:

- Earth Fill
- Rock Fill
- Lightweight Fill (EPS)

The geotechnical issues associated with approach embankments constructed on soft to firm, compressible foundation soils are:

- Stability of the embankments, particularly during construction and during a seismic event
- Long-term settlements due to primary and secondary consolidation

- Lateral deflections or spreading of the underlying soils in the area of the abutment piles associated with settlements due to consolidation and during a seismic event

The stability, settlement and lateral displacement associated with seismic events are addressed in a separate section of this report.

Engineering analyses were carried out in order to address the issues above associated with static loading conditions as follows:

- Stability analysis of embankment cross sections behind the abutments and through the forward slope
- Consolidation settlement of the approach fills

9.2 Stability Analysis

9.2.1 General

Simplified stratigraphic profiles were selected for the subsurface conditions and embankment geometries behind the abutment locations in order to analyse the embankment stability. Table 9.1 presents simplified subsurface conditions for the south and north approach embankments.

The stability analysis was carried out with the following initial assumptions:

- Embankment Side Slopes:
 - Earth Fill: 2H:1V
 - Rock Fill: 1.25H:1V
- Embankment Forward Slopes
 - Earth Fill: 2H:1V
 - Rock Fill: 1.5H:1V
 - Vertical abutment wall for long-term stability
- Site Preparation:
 - All organic soils will be removed within the footprint of the embankment
- Maximum height of embankment = 4 m
- Rate of Construction
 - Not restricted, full embankment loading assumed to be applied “instantaneously”
- Limit Equilibrium Analysis:

- Bishop Modified using G-Slope, developed by Mitre Software
- Shear Strength of Native Soils:
 - Undrained shear strength (S_u) for cohesive soils as shown in Table 9.1. For vertical effective stresses (σ'_v) larger than the pre-consolidation pressure (p'), S_u was assumed equal to $0.25 \cdot \sigma'_v$.
 - Drained shear strength (ϕ') as shown in Table 9.1
- Pore pressure generation:
 - Generation of excess pore pressures (EPP) upon undrained loading of the compressible and cohesive deposits is calculated assuming a B_{bar} (ratio of EPP over vertical total stress) of 0.9
- In Situ Piezometric Head:
 - 1.5 m artesian at the underside of the silty clay

9.2.2 Stability Analysis Results

The global stability of the embankment is governed by the sand and silt layer and not by the deeper deposit of silty clay. Since the thickness and characteristics of the sand and silt deposit are similar for both approaches, the stability analysis was carried out only for the subsurface model associated with the north approach embankment, as shown on Figures C1 through C6 in Appendix C.

The following presents a summary of the results of the stability analysis.

Earth Fill

The results of the analysis showed that an earth fill embankment with maximum height of 4 m and constructed with 2H:1V slopes is stable and has a factor of safety of 1.39, short term and long term.

An analysis was run for a forward slope constructed at 1.5H:1V and produced a factor of safety of 1.23, which is not considered to be adequate.

Rock Fill

The results of the analysis showed that a rock fill embankment with maximum height of 4 m and constructed with 1.25H:1V slopes is stable and has a factor of safety of 1.42, short term and long term.

Table 9.1 – Soil properties Used in Analysis

Highway 11 – Bernard Creek – SBL Structure

Soil Layer		Rock or Earth Fill	Sand and Silt	Silty Clay	Silt	Sand Till
Depth Interval South Approach (m)	From To	Top of fill 1	1 6	6 29	29 34	34 38
Depth Interval North Approach (m)	From To	Top of fill 1	1 6	6 21.5	Absent	21.5 23.5
Unit Weight	γ (kN/m ³)	20/22	20	18	18	21
Undrained Shear Strength	C_u (kPa)	-	-	40	-	-
Drained Shear Strength	c' (kPa)	0	0	0	0	0
	ϕ' (degrees)	42/30	29	28	29	32
Shaft adhesion/friction	α	-	-	0.8	-	-
	β	1	1	0.25	0.4	0.5
Poisson's Ratio	μ	0.3	0.35	0.45	0.35	0.3
Young's Modulus	E (MPa)	150	15	12	15	30
Compression Ratio	$C_c/(1+e_o)$	-	-	0.25	-	-
	$C_r/(1+e_o)$	-	-	0.025	-	-
Pre-Consolidation Pressure	p'_o (kPa)	-	-	100	-	-
Coefficient of Consolidation	C_v (m ² /y) N/C	-	-	31.1	-	-
Coefficient of Secondary Consolidation	$C_{\alpha'}/(1+e_o)$	-	-	0.02	-	-

Note: N/C = Normally Consolidated Soil

O/C = Over consolidated Soil

9.3 Settlement Analysis

A summary of the results of the analysis of settlements due to primary consolidation, secondary consolidation and required depth of fill replacement with EPS is presented in Tables 3.22, 3.30 and 3.38, respectively. These tables were extracted from a report ² prepared for the Robins Road/Black Creek Interchange embankments, including the approaches to the Bernard Creek Bridges.

Settlements due to primary consolidation in the order of 600mm (without surcharge) over a period of more than six years were predicted for the NBL Bridge approach embankments. It is anticipated that the magnitude of settlement adjacent to the structure and time for stabilization will not permit construction to proceed on an acceptable schedule.. The steps required to reduce post-construction settlement and construction time to acceptable levels include:

- Construction of the embankment higher than the proposed top-of-pavement elevation (surcharge) and installation of wick drains to over-consolidate foundation soils and to accelerate the rate of settlement, respectively. Depending on the amount of surcharge and the anticipated settlements due to secondary consolidation, the surcharge and a portion of the fill below the top-of-pavement may be replaced with lightweight fill. In this case a geotechnical monitoring program is required to track the performance of the embankment and control the construction schedule.
- Construction the embankment using EPS, without the use of surcharge or wick drains

Based on the results of the settlement analysis and on discussions with MTO, a decision was made by MTO to construct the approach embankments using EPS lightweight fill within 20m of the bridge abutments and a combination of wick drains, surcharge and fill replacement with EPS beyond 20m. However, due to the fact that the required depth of fill replacement with EPS beyond 20m from the abutments is very close to the embankment height to the top of the pavement, it is more economical to extend the use of EPS from 20m to 60m behind the abutments, eliminating the need for wick drains and surcharging within this area. With the use of EPS, stability, settlements and lateral displacement of the piles associated with the embankment static and dynamic loading of the foundation soils are not an issue in the proximity of the bridge.

² Supplementary Design Report – Embankments along Highway 11 – Sta.13+260 to Sta.14+600 – Robins Road/Black Creek Road I/C Underpass Structure – Approach Embankments and Access Ramps – Highway 11, Burk's Falls to South River, Ontario. G.W.P. 5079-06-00; W.P. 742-93-00 – Geocress Number: 31E-234 – January 3, 2007

10 RETAINED SOIL SYSTEMS

Retained Soil System (RSS) walls are not considered suitable for abutments at this site due to the anticipated large settlements during construction and long-term settlements due to secondary consolidation. Furthermore, RSS systems in the DSM are based on conventional granular backfill and are not compatible with lightweight fill.

11 ABUTMENT DRAINAGE

A geosynthetic sheet drain, not natural aggregate, should be installed behind the abutment wall, at the contact between the concrete and the EPS fill to reduce the vertical and lateral earth pressure on the abutment and to facilitate construction. The sheet drain should be able to maintain its drainage capacity under the horizontal pressures discussed below.

12 STATIC LATERAL PRESSURES AND RESISTANCE

EPS blocks should be continued up to the drainage sheet that is placed along the back of the abutment wall.

The gravity load of the pavement structure and of EPS blocks will result in negligible horizontal active horizontal loading of the abutment wall.

The horizontal coefficient of subgrade reaction of the EPS fill should be calculated based on the following equation:

$$K'_{\text{EPS}} = 0.14 * E_{\text{EPS}} / \{H * (1 - \nu_{\text{EPS}}^2)\} \text{ (units: kN/m}^3\text{)}$$

Where: E_{EPS} = Young's Modulus of EPS Blocks

ν_{EPS} = Poisson's Ratio of EPS Blocks

H = Abutment Wall or Abutment Stem height

The horizontal pressure applied by the abutment wall to the EPS fill must be smaller than the Elastic Limit Stress of the EPS. Typical design parameters and methodology for the design of bridge abutments and EPS backfill are provided in NCHRP's Report 529³.

13 SEISMIC CONSIDERATIONS

13.1 Seismic Hazard Design Values

For design purposes, the site is treated as lying in Seismic Zone 2.

³ NCHRP Report 529 - Guideline and Recommended Standard for Geofoam Applications in Highway Embankments – Transportation Research Board on the National Academies - 2004

13.2 Seismic Design Parameters

The following seismic parameters should be used for design:

- | | |
|--|--------------|
| • Velocity Related Seismic Zone (Zv) | 1 |
| • Zonal Velocity Ratio (V) | 0.05 |
| • Acceleration Related Seismic Zone (Za) | 2 |
| • Zonal Acceleration Ratio (A) | 0.10 |
| • Peak Horizontal Acceleration (PHA) | 0.08 to 0.11 |

The Soil Profile Type at this site has been classified as Type IV. Thus, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient “S” of 2.0 should be used in seismic design.

13.3 Liquefaction Potential

The potential for liquefaction of the cohesionless foundation soils has been assessed using the method recommended by Youd et al (2001)⁴. Using this method, it was determined that the sand and silt deposits up to 3.1m depth at the bridge abutments are in danger of liquefaction under earthquake loading under in situ conditions.

The cohesionless deposit consists of interbedded sand and silt with intermittent clay seams. and the base of the deposit was encountered at 2.8 m to 3.1 m depth near the NBL structure location.

A review of the particle size analysis on selected samples from the sand and silt unit indicates that some portions of the deposit have a high fines content which may reduce the potential for liquefaction in these zones. The particle size data were compared to the Chinese criteria (Marcuson et al , 1990)⁵ to evaluate the liquefaction potential of the deposit. A comparison of the particle size data and the criteria are included in the table below. The available subsurface investigation data indicates that the silt and clay layers which comprise approximately 70% of the deposit are not considered susceptible to liquefaction. Although liquefaction in the silt layers is not likely to occur, loss of strength and decreasing stiffness is expected because of cyclic mobility. However, the sand layers which comprise the remaining 30% are expected to be susceptible to liquefaction.

⁴ Youd, T.L. et al. (2001). “Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils”. *J. Geotech. Geoenv Eng.* 127(1), 817-833.

⁵ Marcuson III, W.E., Hynes, M.E. and Franklin, A.G. 1990. “Evaluation and Use of Residual Shear Strength in Seismic Safety Analysis of Dams.” *Earthquake Spectra*, Vol. 6, No. 3 August.

TABLE 13.1: COMPARISON OF PARTICLE SIZE DATA AND CHINESE CRITERIA

Borehole	Depth (m)	portion finer than 0.005 mm size	Chinese Criteria Indicating Potential Liquefaction
99N-2	2.29	<13%*	<15%
99S-1	6.4	41%	<15%
99S-2	4.88	22%	<15%
99S-2	7.77	<8%*	<15%
99S-5	3.35	16%	<15%
99S-5	6.4	26%	<15%
99S-6	1.83	31%	<15%
99S-6	4.88	23%	<15%

* Deposit may be susceptible to liquefaction

The analysis indicates that the majority of the sand layers will liquefy under the design ground accelerations. Considering the interbedded nature of the deposit, the entire sand and silt unit is therefore expected to exhibit significantly reduced shear strength and stiffness following the design seismic event. Consequently, lateral support of the abutment piles within the sand and silt layer during seismic loading should be disregarded or reduced significantly for the structural analysis of the piles. Alternately, ground improvement may be required to mitigate the potential for liquefaction and allow higher resistance to be utilized

Reduced strength parameters for foundation design under seismic conditions and options for soil improvement are provided in the following section.

13.3.1 Pile Reaction

If soil improvement is not carried out, the available soil resistance following liquefaction for the design seismic event can be calculated based on the method of Olson and Stark (2002). This method results in an average liquefied soil shear strength of 5.5 kPa within the sand and silt deposit over a depth of 0 to 3.1 m.

The available horizontal subgrade reaction for vertical piles under these conditions should be calculated using the following values for the sand and silt deposit:

- $K_s = 1250/D$ (kN/ m³)
- $P_{ult} = 50$ kPa
- $\gamma = 20$ kN/m³

If the available lateral capacity is not sufficient, changes in the structural design such as pile batter and abutment type should be considered. If structural options are not considered feasible then soil improvement will be required to reduce the likelihood of liquefaction in the soil surrounding the foundation.

13.3.2 Improvement of Silt and Sand Unit

To increase the soil resistance under seismic loading, ground improvement can be carried out in a zone surrounding the proposed foundation to provide higher resistance to liquefaction in the sand and silt unit. Treatment methods that increase the density of the deposit (reducing the void ratio) or improve drainage (to reduce high pore water pressures) are considered effective for reducing the likelihood of liquefaction.

The following design parameters are recommended for the zone of treatment:

- Horizontal extent of treatment should extend a minimum 6 m in front and behind the structure foundation.
- The horizontal extent transverse to the bridge axis for treatment should extend a minimum 3 m beyond the foundation.
- Depth of treatment should extend to base of silt and sand unit or a minimum 6 m depth.
- Grout or stone columns should be installed no closer than 1.5 m from the abutment piles.

The options for treatment and the feasibility of each option are presented below:

Wick Drains

Installation of wick drains is considered a viable option for improving drainage and reducing the potential for liquefaction at this site. Equipment for installation of wicks will be available onsite because of requirements at the adjacent approach fills. A maximum horizontal spacing of 1.0 m in a triangular array is recommended between wicks for this application.

Compaction Grouting:

Compaction grouting is considered a viable option for the soil conditions at this site. Grouting contractors with suitable experience in southern Ontario are readily available near the project area. The soil improvement criteria should be developed by the grouting contractor to provide adequate resistance to liquefaction under the design loading conditions.

Rammed aggregate columns

It is expected that compacted stone columns such as a GeoPier or an equivalent rod installed system may be feasible provided a liner is utilized to control sloughing and water inflow. However, the practical depth of treatment for this method is limited to about 6 m depth. Considering the potential construction difficulties and limited depth of treatment this method is not recommended.

Vibro-stone columns:

Stone columns installed using vibro-flotation equipment are used to improved fine-grained cohesionless deposits. However, the fines content at the site is higher than that required for effective vibro-flotation installation. Also, this method is not commonly employed in the local construction market and experienced contractors and equipment for this method may not be available at reasonable cost.

Removal and Replacement

Excavation of the very loose silt and sand and replacement with compacted granular backfill may be feasible at this site. This method would require a temporary sheet pile enclosure for lateral support of the excavation and seepage control during excavation. The maximum practical depth of treatment will depend on the design of the retaining system and is expected be limited to about 4 m for cantilevered systems. Backfill material shall be OPS Gran B compacted as per OPS 501.

13.3.3 Pile Design for Improved Soil

Installation of wick drains or removal and replacement are considered the most feasible options for soil improvement at the Hwy 11 NBL structure.

For piles installed within the treated zones described above, the horizontal soil reaction parameters for pile design will be as provided in Section 7.7 of this report.

13.4 Dynamic Lateral Pressures and Resistance

Inertia forces from seismic excitation of the pavement system and the EPS blocks are very small and should be disregarded in the structural design of the abutment walls. Horizontal reaction to the lateral movements of the abutment wall should be calculated according to Section 12 above.

14 CONSTRUCTION CONCERNS

During construction, the Contract Administrator should employ experienced geotechnical staff to implement the geotechnical monitoring program and to observe construction activities related to foundation construction.

Thurber Engineering Ltd.

Fieldwork supervision, analysis and report preparation by:

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Seismic Design by:

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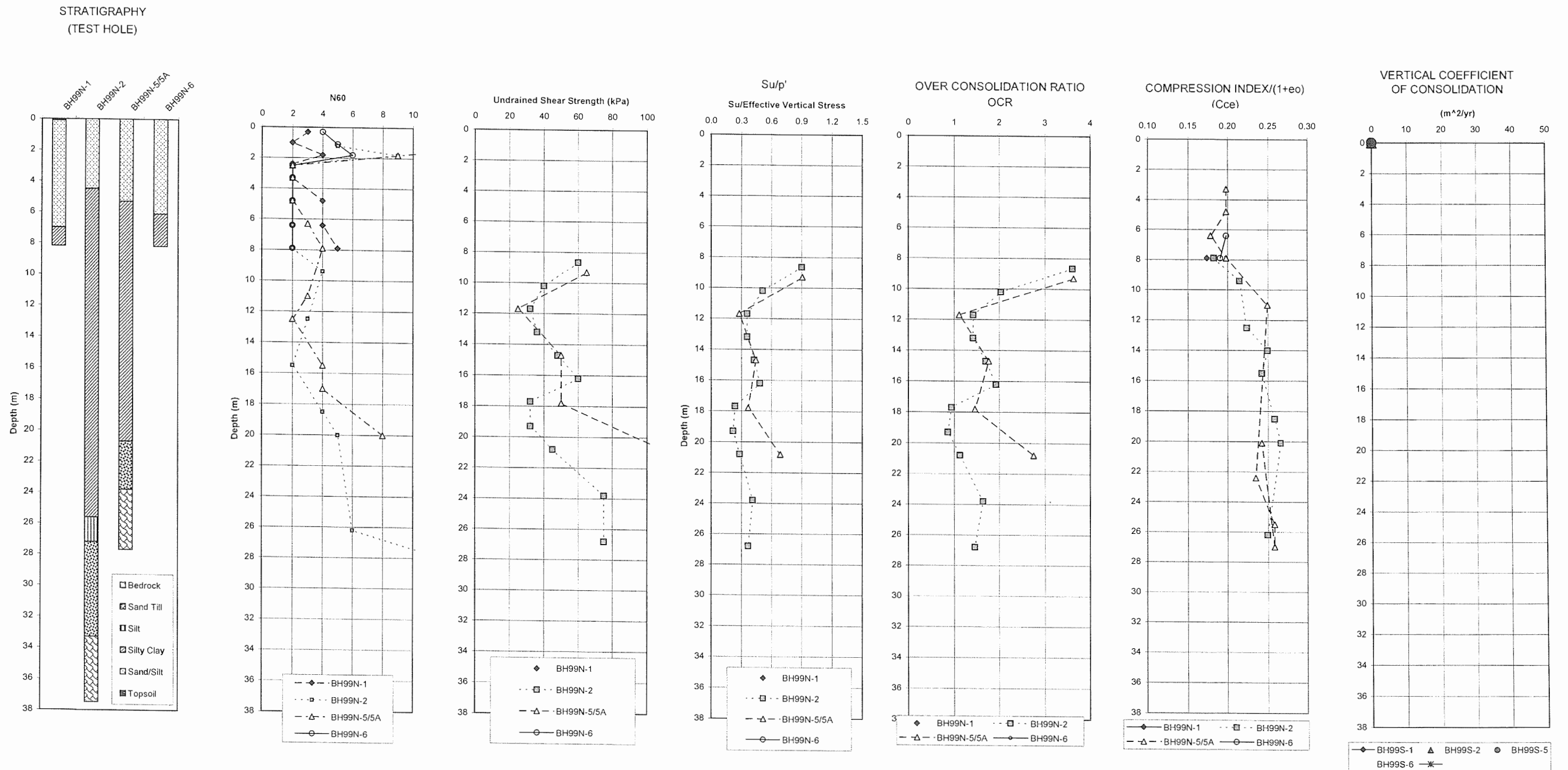
Report reviewed by:

P.J. Branco, P.Eng., Ph.D.

Review Principal



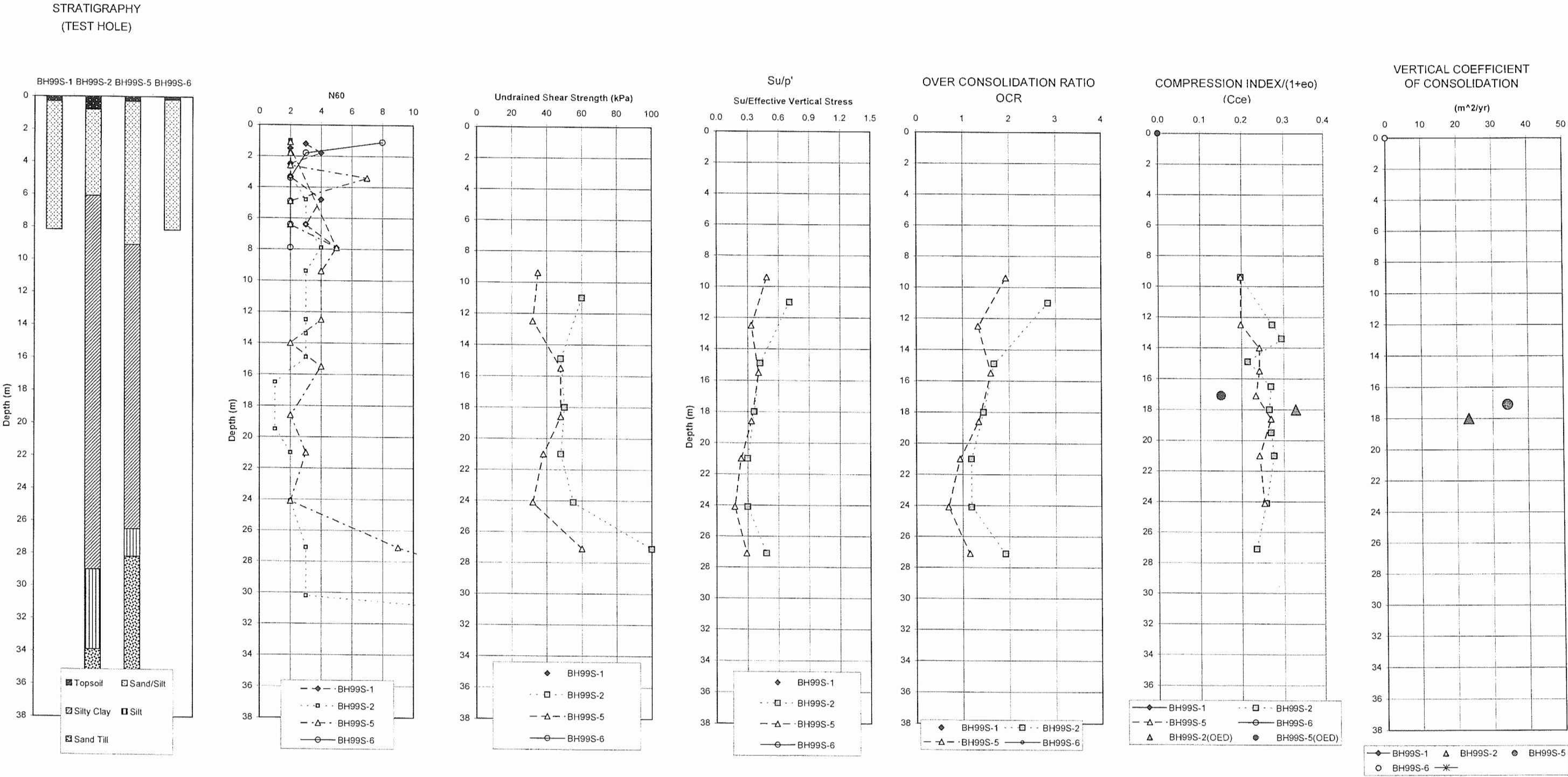
HIGHWAY 11 - BERNARD CREEK - NBL BRIDGE SUMMARY OF SUBSURFACE CONDITIONS



MASTER PLOT

FIGURE 1

HIGHWAY 11 - BERNARD CREEK - SBL BRIDGE
SUMMARY OF SUBSURFACE CONDITIONS



MASTER PLOT

FIGURE 2

Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$



Water Level

C_{pen}





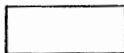
Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
	HIGHLY ORGANIC SOILS		Pt
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

<u>ROCK WEATHERING CLASSIFICATION</u>		<u>SYMBOLS</u>
Fresh (FR)	No visible signs of weathering.	
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.	 CLAYSTONE
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.	 SILTSTONE
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.	 SANDSTONE
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.	 COAL
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.	 BENTONITE

<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 200	Greater than 29,200	Requires many blows of geological hammer to break.
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-200	14,600 to 29,200	Requires a few blows of geological hammer to break.
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,300 to 14,600	Breaks under single blow of geological hammer.
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Moderately Strong	12.5 to 50.0	1,825 to 7,300	¼" indentations with sharp end of geological pick.
<u>TERMS</u>					
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Moderately Weak	5.0 to 12.5	730 to 1,825	Too hard to cut by hand into triaxial specimen.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Weak	1.25 to 5.0	182 to 730	Crumbles under firm blows of geological pick.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.	Very Weak (Rock)	0.60 to 1.25	85 to 182	May be broken in the hand with difficulty.
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

RECORD OF BOREHOLE No 99N-1

1 OF 1

METRIC

W.P. 755-93-01 LOCATION N 5 062 517.2 E 309 452.6 (Bernard Creek Bridge NBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 10.03.04 - 10.03.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL			× LAB VANE	W _p	W
312.2							20	40	60	80	100				
318.6	TOPSOIL														
0.1	SILT and SAND, trace organics near top of layer Very Loose to Loose Brown Wet (Glacial lake deposit characterized by interbedding of silt and sand and intermittent clay seams. Partially reworked at surface by present stream.)		1	SS	3										
			2	SS	2										
			3	SS	4										
			4	SS	2										
309.4	Silty CLAY, occasional sand Soft to Firm Grey Wet (CL)		5	SS	2										
			6	SS	4										
			7	SS	4										
			8	SS	5										
304.0	END OF BOREHOLE AT 8.23m. WATER LEVEL IN OPEN BOREHOLE AT 0.76m DEPTH UPON COMPLETION. BOREHOLE BACKFILLED WITH DRILL CUTTINGS.														
8.2															

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

ONTMT4 99NBERNARD-I.GPJ 13/04/04

RECORD OF BOREHOLE No 99N-2

1 OF 5

METRIC

W.P. 755-93-01 LOCATION N 5 062 536.0 E 309 447.4 (Bernard Creek Bridge NBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers, Tricone COMPILED BY SS
 DATUM Geodetic DATE 04.03.04 - 05.03.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60		
312.0												
0.0	Snow to 0.3m											ground water at 1.2m above ground surface
311.7												
0.3	SILT and SAND , trace organics near top of layer Very Loose to Loose Dark Brown to Brown Wet (Glacial lake deposit characterized by interbedding of silt and sand and intermittent clay seams. Partially reworked at surface by present stream.)		1	SS	5							
			2	SS	9							
	occasional gravel Brown to Grey		3	SS	2							5 82 13 (SI+CL)
309.2												
2.8	Silty CLAY , trace sand Soft to Firm Grey Wet (CL)		4	SS	2							
			5	SS	2							
			6	SS	2							0 1 81 18
			7	SS	2							0 4 66 30
			8	SS	4							
								</				

Continued Next Page

+ 3 x 3: Numbers refer to
Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 99N-2

3 OF 5

METRIC

W.P. 755-93-01 LOCATION N 5 062 536.0 E 309 447.4 (Bernard Creek Bridge NBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers, Tricone COMPILED BY SS
 DATUM Geodetic DATE 04.03.04 - 05.03.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _P	W	W _L		
312.0								20 40 60 80 100						
			12	SS	5		292							0 1 39 60
							291							
							290							
			3	TW	PH		289							
							288							
							287							
			13	SS	6		286							
							285							
							284							
283.7														
28.3	Clayey SILT, occasional gravel Very Stiff Grey Wet (OL-ML)													
			14	SS	16		283							
282.2														

Continued Next Page

Mud rotary from

+ 3, x 3, Numbers refer to
Sensitivity

20
15
10


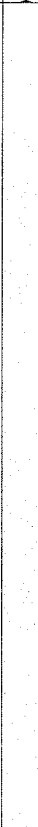

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 99N-2

4 OF 5

METRIC

W.P. 755-93-01 LOCATION N 5 062 536.0 E 309 447.4 (Bernard Creek Bridge NBL) ORIGINATED BY GA
HWY 11 BOREHOLE TYPE Hollow Stem Augers, Tricone COMPILED BY SS
DATUM Geodetic DATE 04.03.04 - 05.03.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100					PLASTIC LIMIT w _p NATURAL MOISTURE CONTENT w LIQUID LIMIT w _L				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%) 20 40 60				
312.0							282					GR SA SI CL					
29.9	BOULDER											29.87m					
281.5																	
30.5	SAND, trace to some gravel, occasional silt, occasional cobbles or boulders Very Dense Brown Wet (SP)											Tricone casing from 30.48m to 32m					
			15	SS	67		280			○							
							279										
							278										
							277			○							
275.1			16	SS	105/ .254		276										
35.0	BEDROCK Slightly weathered GRANITIC GNEISS, laminated to thinly banded, pale pink with sub-vertical dark banding, rough sub-planar joints, some clay infilling, strong to very strong						275					RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=135.4MPa					
			1	RUN			274					RUN 2# TCR=100%, SCR=100%, RQD=90%, UCS=117.2MPa					
			2	RUN			273					RUN 3# TCR=100%, SCR=100%, RQD=74%, UCS=141.2MPa					
			3	RUN													

Continued Next Page

+ ³ , × ³ : Numbers refer to
Sensitivity 15 20 5 10 (%) STRAIN AT FAILURE

ONTMT4 99NBERNARD-I.GPJ 10/04/04

RECORD OF BOREHOLE No 99N-2

5 OF 5

METRIC

W.P. 755-93-01 LOCATION N 5 062 536.0 E 309 447.4 (Bernard Creek Bridge NBL) ORIGINATED BY GA
HWY 11 BOREHOLE TYPE Hollow Stem Augers, Tricone COMPILED BY SS
DATUM Geodetic DATE 04.03.04 - 05.03.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	SHEAR STRENGTH kPa						
312.0								○ UNCONFINED + FIELD VANE							
271.9								● QUICK TRIAXIAL × LAB VANE							
40.2	END OF BOREHOLE AT 40.16m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) Mar. 10/ 2004 1.2 above ground surface Mar. 11/ 2004 1.2 above ground surface														

ONTMT4 99BERNARD-I.GPJ 10/04/04

METRIC

ONTMT4 99NBERNARD-I.GPJ 13/04/04

[illegible]

+ 3, × 3; Numbers refer to Sensitivity

RECORD OF BOREHOLE No 99N-5

2 OF 3

METRIC

W.P. 755-93-01 LOCATION N 5 062 580.6 E 309 470.0 (Bernard Creek Bridge NBL) ORIGINATED BY GA
HWY 11 BOREHOLE TYPE Hollow Stem Augers, NW Casing COMPILED BY SS
DATUM Geodetic DATE 17.02.04 - 18.02.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
312.5															
	Silty CLAY , occasional sand seams Soft Grey Wet (CL)		8	SS	3		302								
							301	3 +							
			9	SS	2		300								0 5 78 17
							299								
			1	TW	PH		298								
							297								
			10	SS	4		296								
							295								
			11	SS	4		294								
							293								
			2	TW	PH										

Continued Next Page

+ 3, × 3: Numbers refer to
Sensitivity 20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 99N-5

3 OF 3

METRIC

W.P. 755-93-01 LOCATION N 5 062 580.6 E 309 470.0 (Bernard Creek Bridge NBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers, NW Casing COMPILED BY SS
 DATUM Geodetic DATE 17.02.04 - 18.02.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
312.5														
292.0			12	SS	8									
20.5	SAND, some gravel or cobbles Loose Brown Wet (TILL) boulder from 20.57m to 20.88m		13	SS	50/ .000		292					2.6 +		
							291							
			14	SS	8		290							
			1	GS										
289.0														
23.5	BEDROCK Slightly weathered GRANITIC GNEISS, laminated to thinly banded, pale pink with sub-vertical dark banding, rough sub-planar joints, some clay infilling, strong to very strong		1	RUN			289							
							288							
			2	RUN										
							287							
			3	RUN			286							
285.1														
27.4	END OF BOREHOLE AT 27.38m. WATER LEVEL IN OPEN BOREHOLE AT 0.3m DEPTH UPON COMPLETION. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. BOREHOLE BACKFILLED AS FOLLOWS: 0 - 19.8m Grout 19.8m - 22.9m Holeplug WATER LEVEL READINGS: DATE DEPTH (m) 26/02/04 2.2 04/03/04 0.9 above ground surface 11/03/04 1.1 above ground surface													

ONTMT4 99NBERNARD-I.GPJ 13/04/04

RECORD OF BOREHOLE No 99N-6

1 OF 1

METRIC

W.P. 755-93-01 LOCATION N 5 062 595.7 E 309 466.7 (Bernard Creek Bridge NBL) ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 18.02.04 - 18.02.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
312.2 0.0	SILT and SAND, trace organics near top of layer Very Loose to Loose Brown to Grey Wet (Glacial lake deposit characterized by interbedding of silt and sand and intermittent clay seams. Partially reworked at surface by present stream.)		1	SS	4		312							
			2	SS	5		311							
			3	SS	6		310							
			4	SS	2		309							
309.3 2.9	Silty CLAY, occasional sand seams Very Soft Grey Wet (CL)		5	SS	2		308							
			6	SS	2		307							
			7	SS	2		306							
			8	SS	2		305							
304.0 8.2	END OF BOREHOLE AT 8.23m. WATER LEVEL IN OPEN BOREHOLE AT 0.1m ABOVE SURFACE. BOREHOLE BACKFILLED WITH DRILL CUTTINGS TO SURFACE.													

ONTMT4 99NBERNARD-I.GPJ 13/04/04

RECORD OF BOREHOLE No 99S-1

1 OF 1

METRIC

W.P. 756-93-01 LOCATION N 5 062 507.3 E 309 419.1 (Bernard Creek Bridge SBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers, HQ Coring COMPILED BY SS
 DATUM Geodetic DATE 10.01.04 - 10.01.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
312.6 0.0 312.4	TOPSOIL (250mm)											
0.3	SILT and SAND, trace organics near top of layer Very Loose to Loose Brown Wet (Glacial lake deposit characterized by interbedding of silt and sand and intermittent clay seams. Partially reworked at surface by present stream.)		1	SS	3		312					
			2	SS	4		311					
			3	SS	2		310					
			4	SS	2		309					0 44 56 (SI+CL)
			5	SS	4		308					
			6	SS	3		306					0 4 72 24
			7	SS	5		305					
304.4 8.2	END OF BOREHOLE AT 8.23m. WATER LEVEL IN OPEN BOREHOLE AT 5.33m DEPTH UPON COMPLETION.											

ONTMT4 99SBERNARD.GPJ 05/04/04

+³, x³: Numbers refer to Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 99S-2

1 OF 4

METRIC

W.P. 756-93-01 LOCATION N 5 062 526.2 E 309 414.7 (Bernard Creek Bridge SBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers, HW Casing COMPILED BY SS
 DATUM Geodetic DATE 11.01.04 - 12.01.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	
312.2 0.0	TOPSOIL (750mm)						312					ground water at 1.45m above ground surface
311.4 0.8	SILT and SAND, trace organics near top of layer Very Loose to Loose Brown Wet (Glacial lake deposit characterized by interbedding of silt and sand and intermittent clay seams. Partially reworked at surface by present stream.)		1	SS	2		311					
			2	SS	2		310					
			3	SS	2		309					
			4	SS	2		308					
			5	SS	3		307					0 26 60 14
			6	SS	3		306					
			7	SS	4		305					0 93 7 (SI+CL)
304.3 7.9	Silty CLAY, trace sand Soft to Firm Grey Wet		8	SS	3		304					
	occasional silt layers, thin sand seams Soft						303					

Continued Next Page

+³ ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

ONTMT4 99SBERNARD.GPJ 05/04/04

RECORD OF BOREHOLE No 99S-2

3 OF 4

METRIC

W.P. 756-93-01 LOCATION N 5 062 526.2 E 309 414.7 (Bernard Creek Bridge SBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers, HW Casing COMPILED BY SS
 DATUM Geodetic DATE 11.01.04 - 12.01.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
312.2								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	WATER CONTENT (%) 20 40 60			
	(Cl)		14	SS	2		292					
							291	1.3				0 0 35 65
							290					
							289					
			15	SS	2		288	3.5				
							287					
							286					
							285					
			16	SS	3		284					
							283					
283.2												
29.0	Clayey SILT, with thin clay seams Firm to Stiff Grey Wet											

ONTMT4 99SBERNARD.GPJ 05/04/04

Continued Next Page

+³, x³: Numbers refer to
Sensitivity 20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 99S-2

4 OF 4

METRIC

W.P. 756-93-01 LOCATION N 5 062 526.2 E 309 414.7 (Bernard Creek Bridge SBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers, HW Casing COMPILED BY SS
 DATUM Geodetic DATE 11.01.04 - 12.01.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
312.2			17	SS	3		282						0 0 84 15
279.8							281						
32.4	SAND, fine grained, with occasional cobbles and boulders Very Dense Grey Wet		18	SS	41		279						
278.3							278						Tricone from 33.91m to 34.44m
33.9	BOULDER		1	RUN			277						HQ casing to 35.05m 0.5m of sand in HQ casing cleaned out with tricone
277.5							276						
34.7	SAND, fine grained, with occasional cobbles and boulders Very Dense Grey Wet (TILL)		19	SS	50/.078		275						NQ barrel from 37.13m to 38m broke through boulders
			20	SS	50/.102								
	cobbles from 37.13m to 37.36m boulder from 37.36m to 38m		2	RUN									
274.0			21	SS	50/.078								No sample in spoon
38.2	END OF BOREHOLE AT 38.18m. STANDPIPE PLACED IN HOLE. BOREHOLE CAVED TO 10.06m BEFORE SEALING THE STANDPIPE TIP. DATE DEPTH(m) Mar. 4/ 2004 1.5 above ground surface Mar. 11/ 2004 1.45 above ground surface												

ONTMT4 99SBERNARD.GPJ 05/04/04

RECORD OF BOREHOLE No 99S-5

1 OF 4

METRIC

W.P. 756-93-01 LOCATION N 5 062 564.3 E 309 429.0 (Bernard Creek Bridge SBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers, HW Casing COMPILED BY SS
 DATUM Geodetic DATE 08.01.04 - 08.01.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
312.2								20 40 60 80 100						
0.0	TOPSOIL						312							
312.0								20 40 60 80 100						
0.3	SILT and SAND, trace organics near top of layer Very Loose to Loose Grey Wet (Glacial lake deposit characterized by interbedding of silt and sand and intermittent clay seams. Partially reworked at surface by present stream.)		1	SS	2		311							
			2	SS	2		310							
			3	SS	2		309							0 46 42 12
			4	SS	7		308							
			5	SS	2		307							
			6	SS	2		306							0 2 83 16
			7	SS	5		305							
			8	SS	4		304							
303.4	Silty CLAY, trace sand Soft to Firm Grey						303							

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+ 3, x 3: Numbers refer to Sensitivity 20 15 10 (% STRAIN AT FAILURE

ONTMT4 99SBERNARD.GPJ 05/04/04

RECORD OF BOREHOLE No 99S-5

3 OF 4

METRIC

W.P. 756-93-01 LOCATION N 5 062 564.3 E 309 429.0 (Bernard Creek Bridge SBL) ORIGINATED BY SL
HWY 11 BOREHOLE TYPE Hollow Stem Augers, HW Casing COMPILED BY SS
DATUM Geodetic DATE 08.01.04 - 08.01.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE						
								● QUICK TRIAXIAL	× LAB VANE						
312.2							20 40 60 80 100	20 40 60							
			13	SS	3		292								
							291								
							290								
							289								
			14	SS	2		288								
							287								
							286								
							285								
							284								
							283								
286.0							286								
26.2	Clayey SILT, with thin clay seams Firm to Stiff Grey Wet		15	SS	9		285								
							284								
284.3							283								
27.9	Boulder						284								
284.1	SAND, trace silt, trace gravel, occasional cobbles Compact to Very Dense Brown Moist (TILL)		1	RUN			283								
28.1			2	RUN			284								

Continued Next Page

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

ONTMT4 99SBERNARD.GPJ 05/04/04

RECORD OF BOREHOLE No 99S-5

4 OF 4

METRIC

W.P. 756-93-01 LOCATION N 5 062 564.3 E 309 429.0 (Bernard Creek Bridge SBL) ORIGINATED BY SL
HWY 11 BOREHOLE TYPE Hollow Stem Augers, HW Casing COMPILED BY SS
DATUM Geodetic DATE 08.01.04 - 08.01.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)				
								○ UNCONFINED	+ FIELD VANE											
								● QUICK TRIAXIAL	× LAB VANE											
312.2								20 40 60 80 100												
			16	SS	21		282													
							281													
							280													
			3	RUN			279													
							278								Sand heaving into NQ casing to 30.48m					
			4	RUN			277								Sand coming into NQ casing to 32.92m Tricone method to get SPT values at 35.06m					
			17	SS	50/ .076		276													
							275								No sand found inside HW casing at 36.58m					
			18	SS	50/ .076		274													
274.0			19	SS	50/ .127		274													
38.2	END OF BOREHOLE AT 38.23m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) Mar. 4/ 2004 0.1 Mar. 11/ 2004 0.12																			

ONTMT4 99SBERNARD.GPJ 05/04/04

RECORD OF BOREHOLE No 99S-6

1 OF 1

METRIC

W.P. 756-93-01 LOCATION N 5 062 582.6 E 309 418.3 (Bernard Creek Bridge SBL) ORIGINATED BY SL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 09.01.04 - 09.01.04 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								WATER CONTENT (%)				
312.9							20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L		
312.7	TOPSOIL						20 40 60 80 100					
0.2	SILT and SAND, trace organics near top of layer Very Loose to Loose Brown Wet (Glacial lake deposit characterized by interbedding of silt and sand and intermittent clay seams. Partially reworked at surface by present stream.)		1	SS	8		312					
311.5			2	SS	3		311					0 8 72 20
1.5			3	SS			310					
			4	SS	2		309					
			5	SS	2		308					0 22 63 15
							307	2.5				
			6	SS	2		306					
			7	SS	2		305					
	END OF BOREHOLE AT 8.23m. WATER LEVEL IN OPEN BOREHOLE AT 3.48m DEPTH UPON COMPLETION. BOREHOLE CAVED TO 5.56m.											

ONTM14 99SBERNARD.GPJ 05/04/04

Appendix B

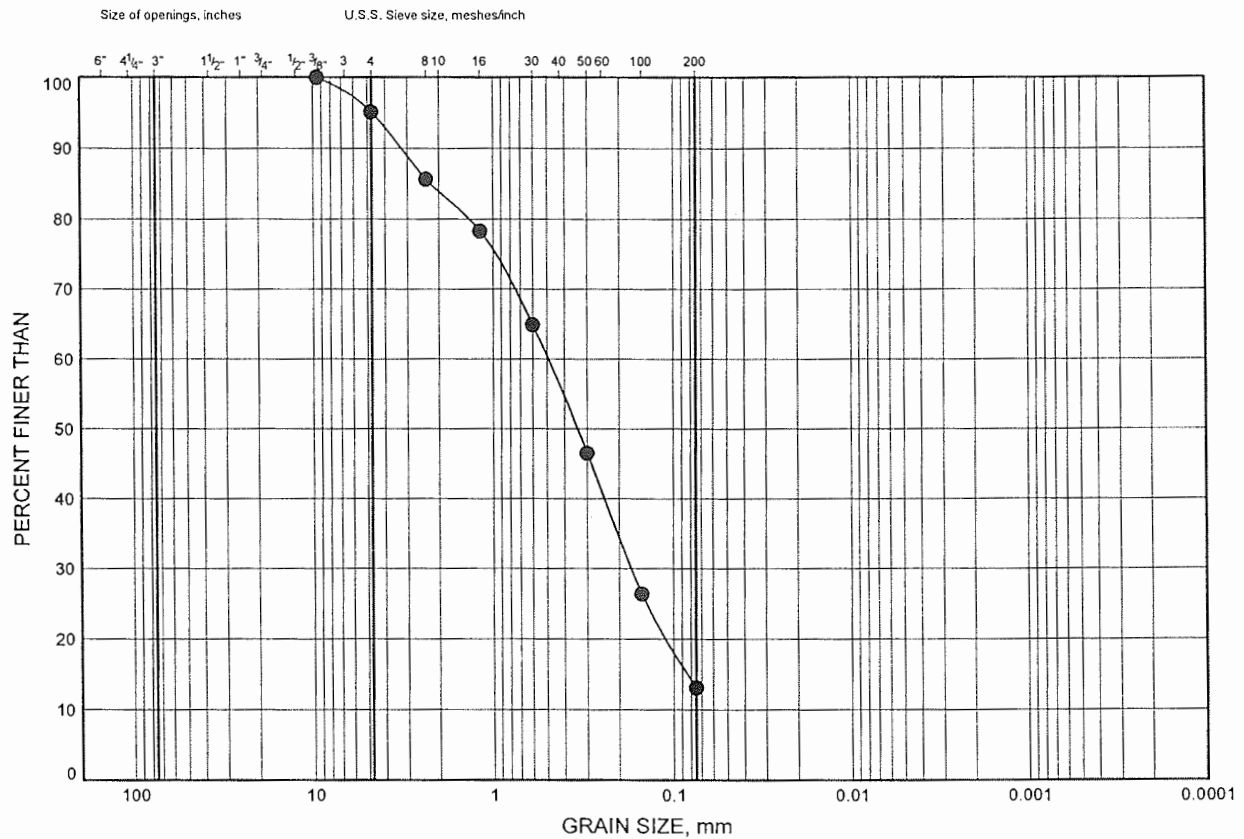
Laboratory Test Results

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B1-N

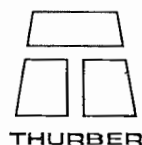
SAND



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	99N-2	2.59	309.44

Date April 2004
Project 755-93-01



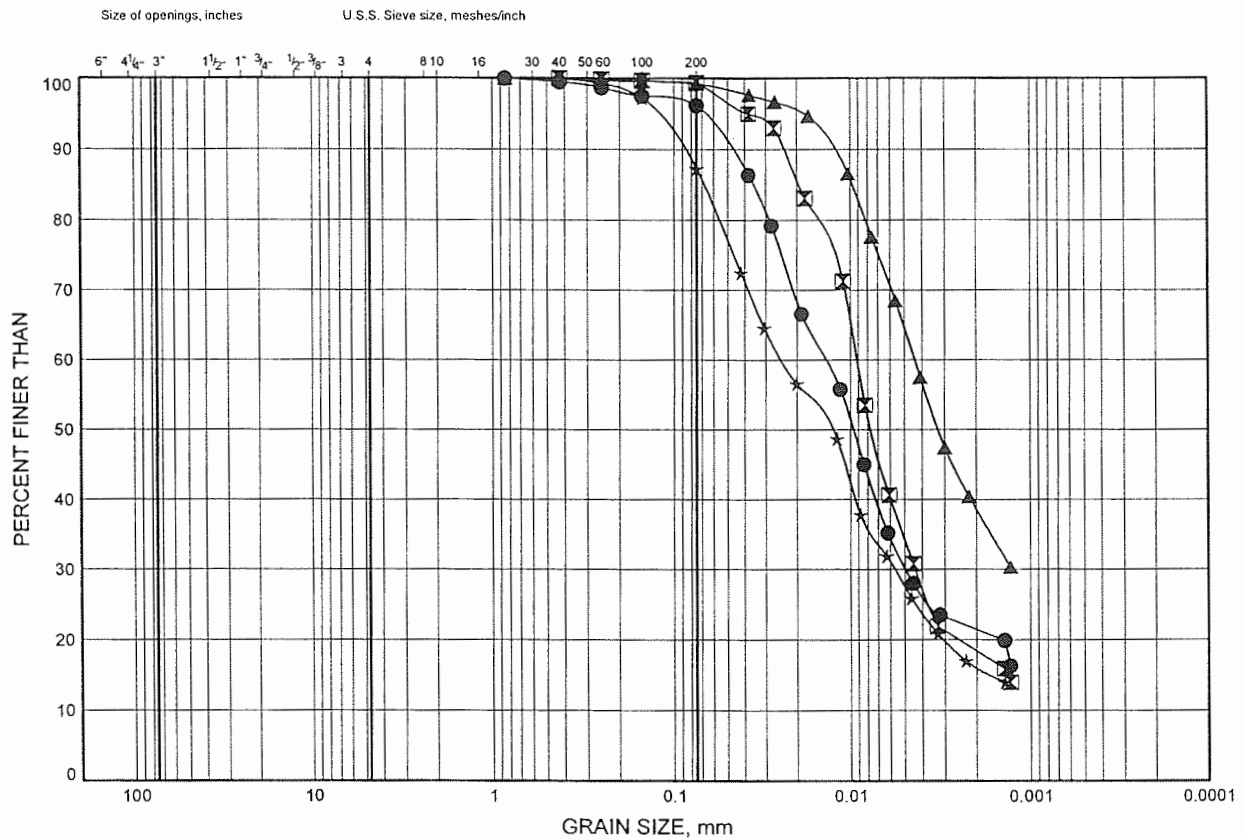
Prep'd SS
Chkd. AEG

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B2-N

SILT AND SAND

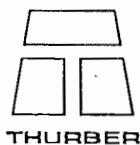


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	99N-1	3.35	308.86
⊠	99N-2	6.40	305.63
▲	99N-5	3.35	309.13
★	99N-6	3.35	308.84

Date April 2004

Project 755-93-01



Prep'd SS

Chkd. AEG

TABLE B1 Silty Clay

SUMMARY OF WATER CONTENT DETERMINATIONS

PROJECT NUMBER 04-1116-019					
PROJECT NAME Thurber / Lab Testing / 19-1423-12					
DATE TESTED March, 2004					
Borehole No.	Sample No.	Depth (ft)	Depth (m)	Water Content (%)	Atterberg Limits LL, PL, PI
99S-2	ST#2	58.0-60.0	17.68-18.29	55.9%	LL=45.1, PL=25.0, PI=20.1
99S-5	ST#2	55.0-57.0	16.76-17.37	41.6%	LL=33.7, PL=23.5, PI=10.2

TABLE B2

Silty Clay

SPECIFIC GRAVITY TEST RESULTS**ASTM D 854-00 TEST METHOD A**

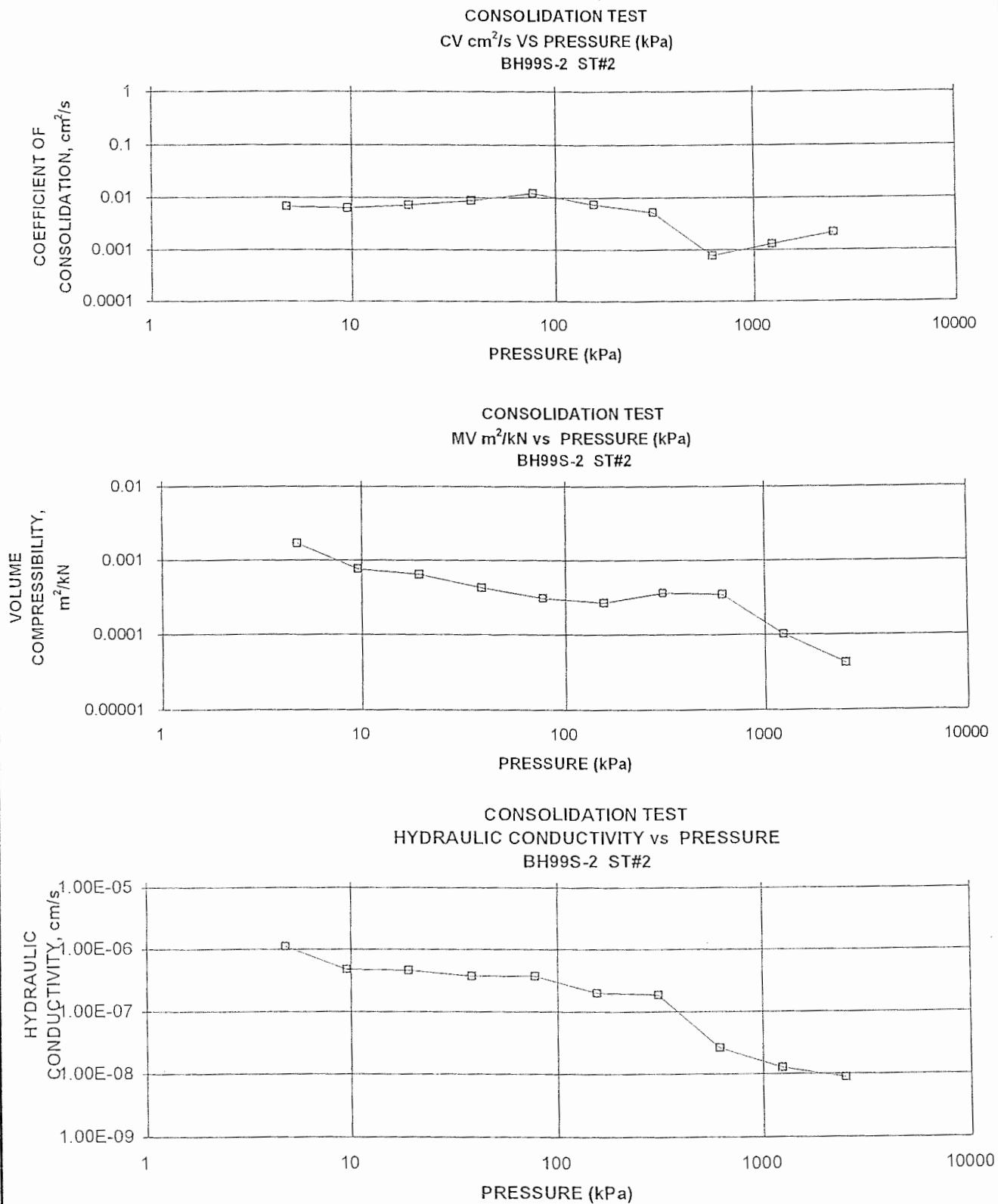
PROJECT NUMBER	04-1116-019		
PROJECT NAME	Thurber / Lab Testing / 19-1423-12		
DATE TESTED	March, 2004		
Borehole	Sample	Specific	
No.	No.	Gravity	
99S-2	ST#2	2.81	
99S-5	ST#2	2.77	

Note: Test carried out on soil particles <4.75mm using distilled water.

TABLE B3

OEDOMETER CONSOLIDATION SUMMARY SILTY CLAY

SAMPLE IDENTIFICATION							
Project Number	04-1116-019			Sample Number	ST#2		
Borehole Number	99S-2			Sample Depth, m	17.7-18.3		
TEST CONDITIONS							
Test Type	Standard			Load Duration, hr	24-48		
Oedometer Number	5						
Date Started	3/04/2004						
Date Completed	3/17/2004						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	1.91			Unit Weight, kN/m ³	16.70		
Sample Diameter, cm	6.35			Dry Unit Weight, kN/m ³	10.71		
Area, cm ²	31.65			Specific Gravity, measured	2.81		
Volume, cm ³	60.45			Solids Height, cm	0.742		
Water Content, %	55.92			Volume of Solids, cm ³	23.49		
Wet Mass, g	102.92			Volume of Voids, cm ³	36.96		
Dry Mass, g	66.01			Degree of Saturation, %	99.9		
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	1.573	1.910				
4.70	1.895	1.553	1.903	113	6.79E-03	1.67E-03	1.11E-06
9.54	1.888	1.544	1.892	119	6.37E-03	7.58E-04	4.73E-07
19.29	1.876	1.528	1.882	103	7.29E-03	6.44E-04	4.60E-07
38.71	1.860	1.506	1.868	85	8.70E-03	4.31E-04	3.68E-07
77.44	1.837	1.475	1.849	60	1.21E-02	3.11E-04	3.68E-07
154.52	1.797	1.421	1.817	94	7.45E-03	2.72E-04	1.98E-07
309.92	1.689	1.276	1.743	124	5.19E-03	3.64E-04	1.85E-07
619.04	1.483	0.998	1.586	696	7.66E-04	3.49E-04	2.62E-08
1236.79	1.363	0.836	1.423	338	1.27E-03	1.02E-04	1.27E-08
2474.94	1.264	0.703	1.314	171	2.14E-03	4.19E-05	8.78E-09
1236.79	1.273	0.715	1.269				
309.92	1.315	0.771	1.294				
154.52	1.340	0.805	1.327				
38.71	1.401	0.888	1.371				
9.54	1.475	0.987	1.438				
4.70	1.486	1.002	1.481				
Notes:							
k calculated using cv based on t ₉₀ values.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	1.49			Unit Weight, kN/m ³	19.43		
Sample Diameter, cm	6.35			Dry Unit Weight, kN/m ³	13.76		
Area, cm ²	31.65			Specific Gravity, measured	2.81		
Volume, cm ³	47.03			Solids Height, cm	0.742		
Water Content, %	41.18			Volume of Solids, cm ³	23.49		
Wet Mass, g	93.19			Volume of Voids, cm ³	23.54		
Dry Mass, g	66.01						



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE B2-S

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH99S-2 ST#2

Silty Clay

Silty Clay

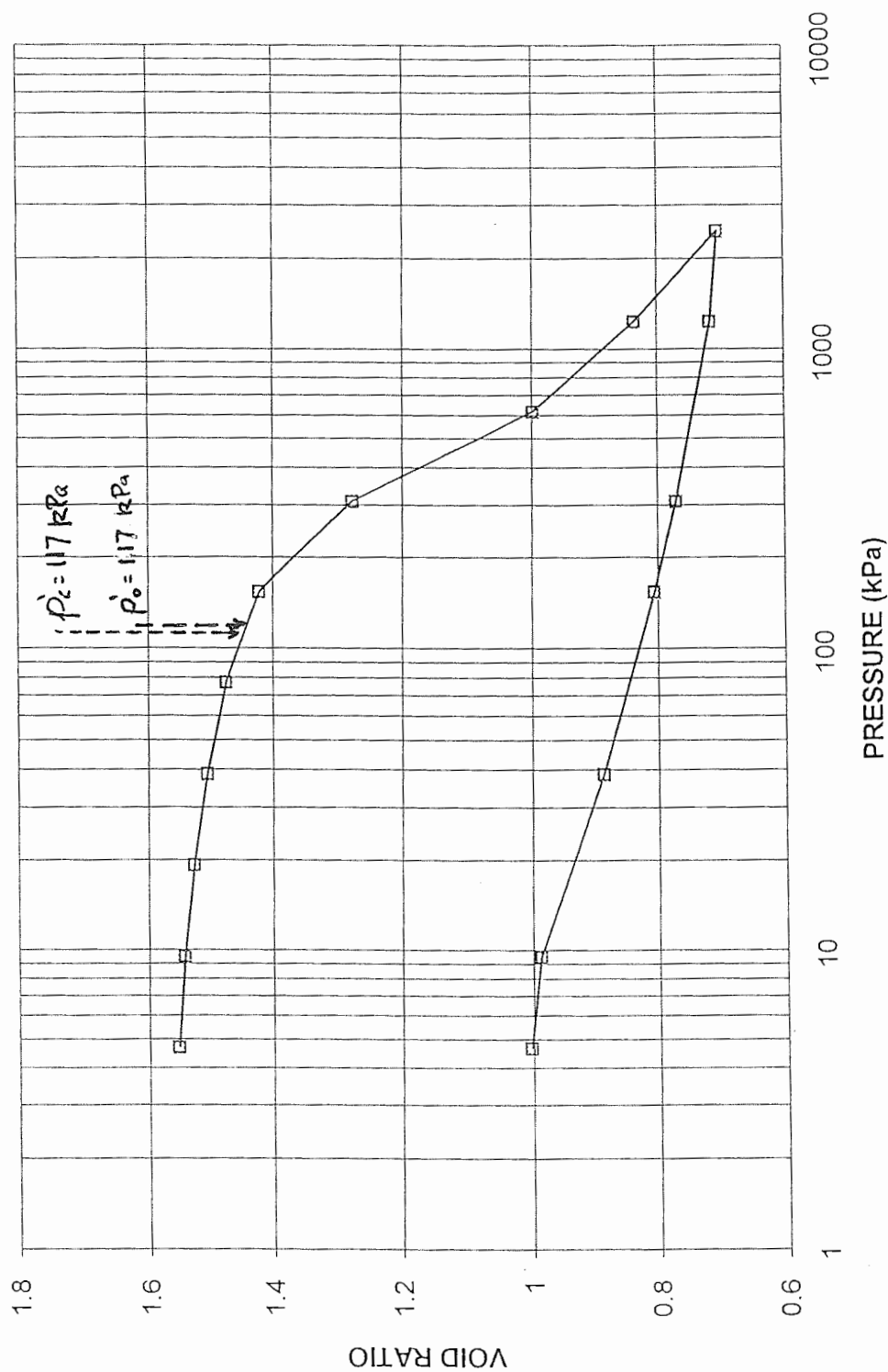


FIGURE B3-S

BOREHOLE 99S-2 ST#2

APPLIED PRESSURE = 154.52 kPa

Silty Clay

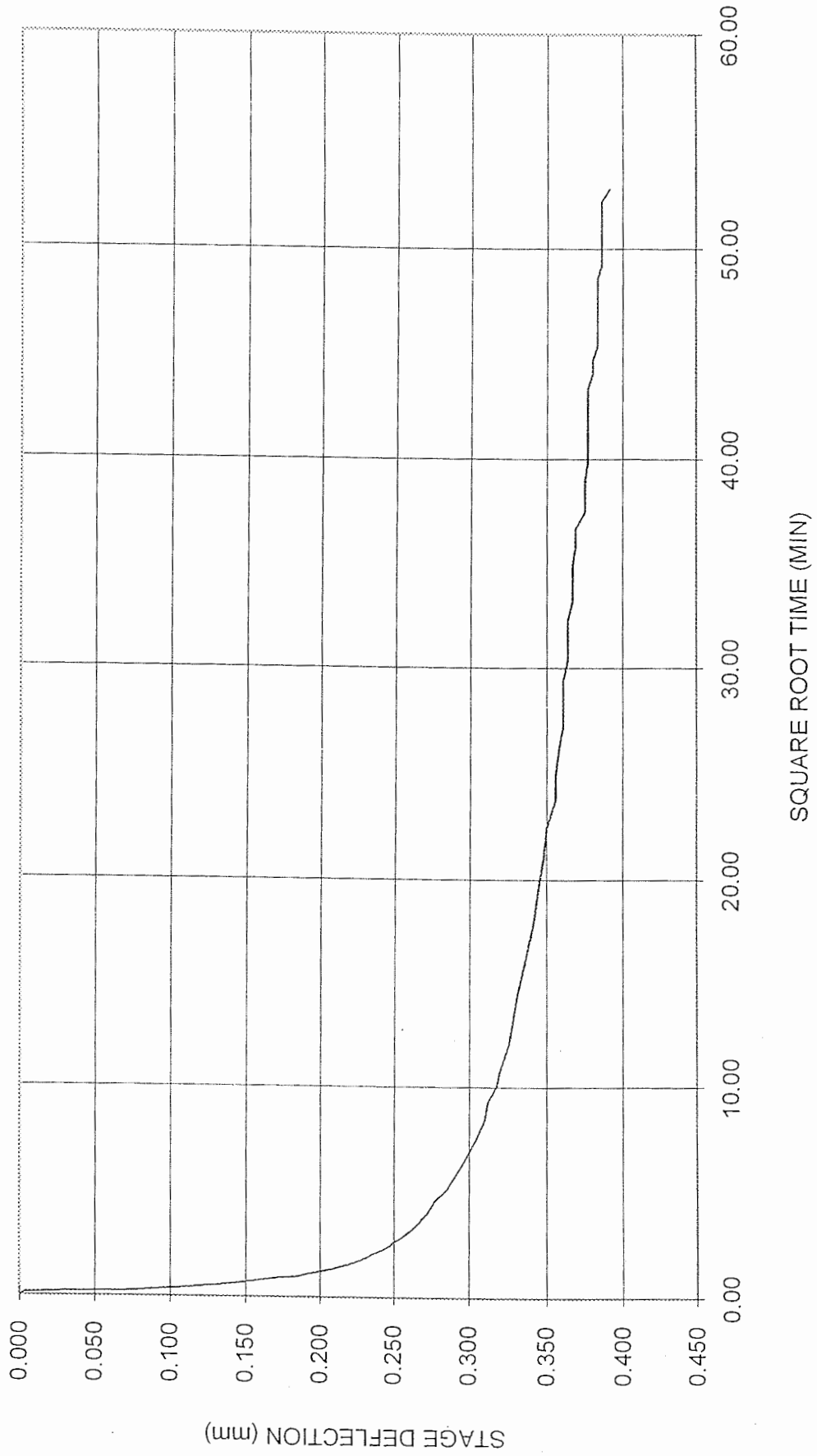


FIGURE B4-S

Silty Clay

BOREHOLE 99S-2 ST#2

APPLIED PRESSURE = 154.52 kPa

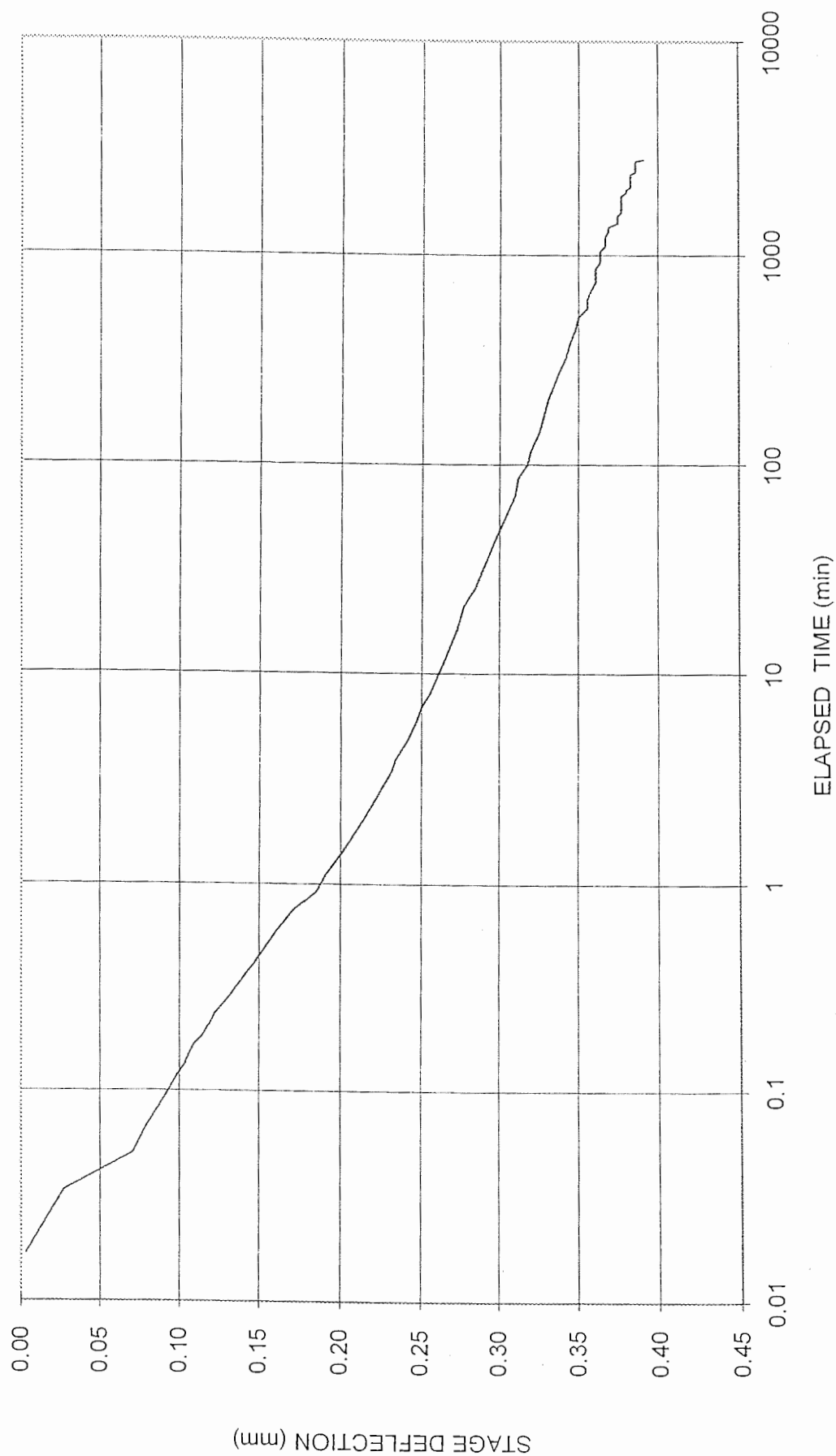


TABLE B4

OEDOMETER CONSOLIDATION SUMMARY

Silty Clay

SAMPLE IDENTIFICATION

Project Number	04-1116-019	Sample Number	ST#2
Borehole Number	99S-5	Sample Depth, m	16.8-17.4

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24-48
Oedometer Number	6		
Date Started	3/04/2004		
Date Completed	3/17/2004		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m ³	17.77
Sample Diameter, cm	6.36	Dry Unit Weight, kN/m ³	12.54
Area, cm ²	31.79	Specific Gravity, measured	2.77
Volume, cm ³	60.72	Solids Height, cm	0.882
Water Content, %	41.62	Volume of Solids, cm ³	28.04
Wet Mass, g	110.00	Volume of Voids, cm ³	32.68
Dry Mass, g	77.67	Degree of Saturation, %	98.9

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	1.165	1.910				
4.70	1.902	1.156	1.906	124	6.21E-03	8.91E-04	5.42E-07
9.55	1.894	1.147	1.898	60	1.27E-02	8.64E-04	1.08E-06
19.30	1.882	1.134	1.888	76	9.94E-03	6.44E-04	6.28E-07
38.70	1.865	1.114	1.874	53	1.40E-02	4.59E-04	6.31E-07
77.43	1.841	1.087	1.853	53	1.37E-02	3.24E-04	4.37E-07
154.78	1.799	1.040	1.820	60	1.17E-02	2.84E-04	3.26E-07
309.24	1.718	0.948	1.759	72	9.11E-03	2.75E-04	2.45E-07
618.72	1.637	0.856	1.678	68	8.77E-03	1.37E-04	1.18E-07
1237.12	1.566	0.775	1.602	53	1.03E-02	6.01E-05	6.04E-08
2476.10	1.492	0.692	1.529	49	1.01E-02	3.13E-05	3.10E-08
1237.12	1.506	0.707	1.499				
309.24	1.524	0.728	1.515				
154.78	1.535	0.740	1.530				
38.70	1.561	0.770	1.548				
9.55	1.593	0.806	1.577				
4.76	1.615	0.831	1.604				

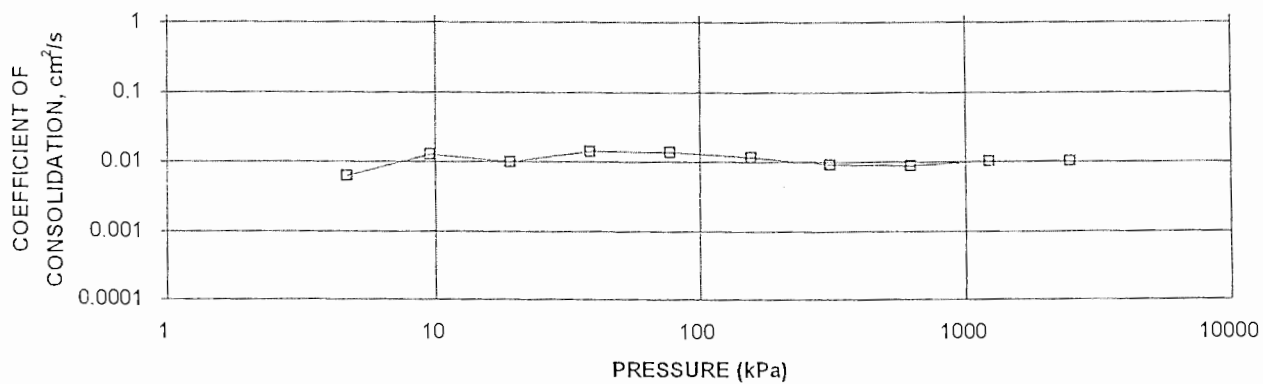
Notes:

k calculated using cv based on λ_0 values.

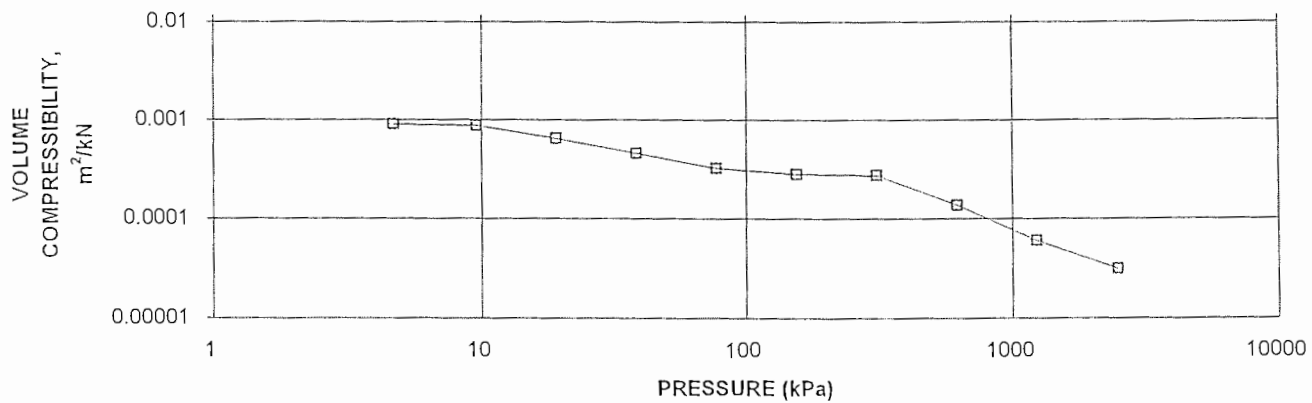
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.62	Unit Weight, kN/m ³	19.49
Sample Diameter, cm	6.36	Dry Unit Weight, kN/m ³	14.84
Area, cm ²	31.79	Specific Gravity, measured	2.77
Volume, cm ³	51.34	Solids Height, cm	0.882
Water Content, %	31.36	Volume of Solids, cm ³	28.04
Wet Mass, g	102.03	Volume of Voids, cm ³	23.30
Dry Mass, g	77.67		

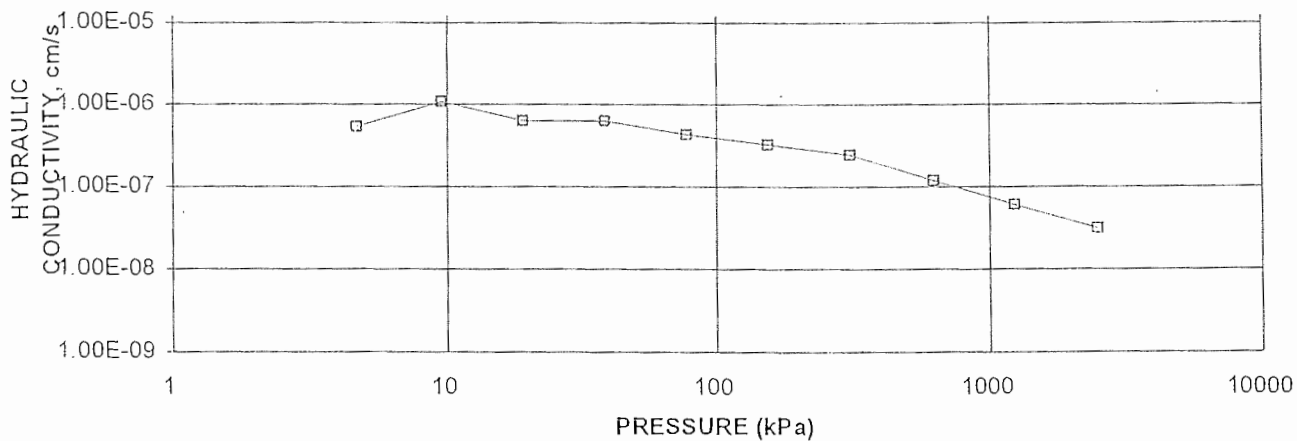
CONSOLIDATION TEST
CV cm^2/s VS PRESSURE (kPa)
BH99S-5 ST#2



CONSOLIDATION TEST
MV m^2/kN vs PRESSURE (kPa)
BH99S-5 ST#2



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH99S-5 ST#2



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE B6-S

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH99S-5 ST#2

Silty Clay

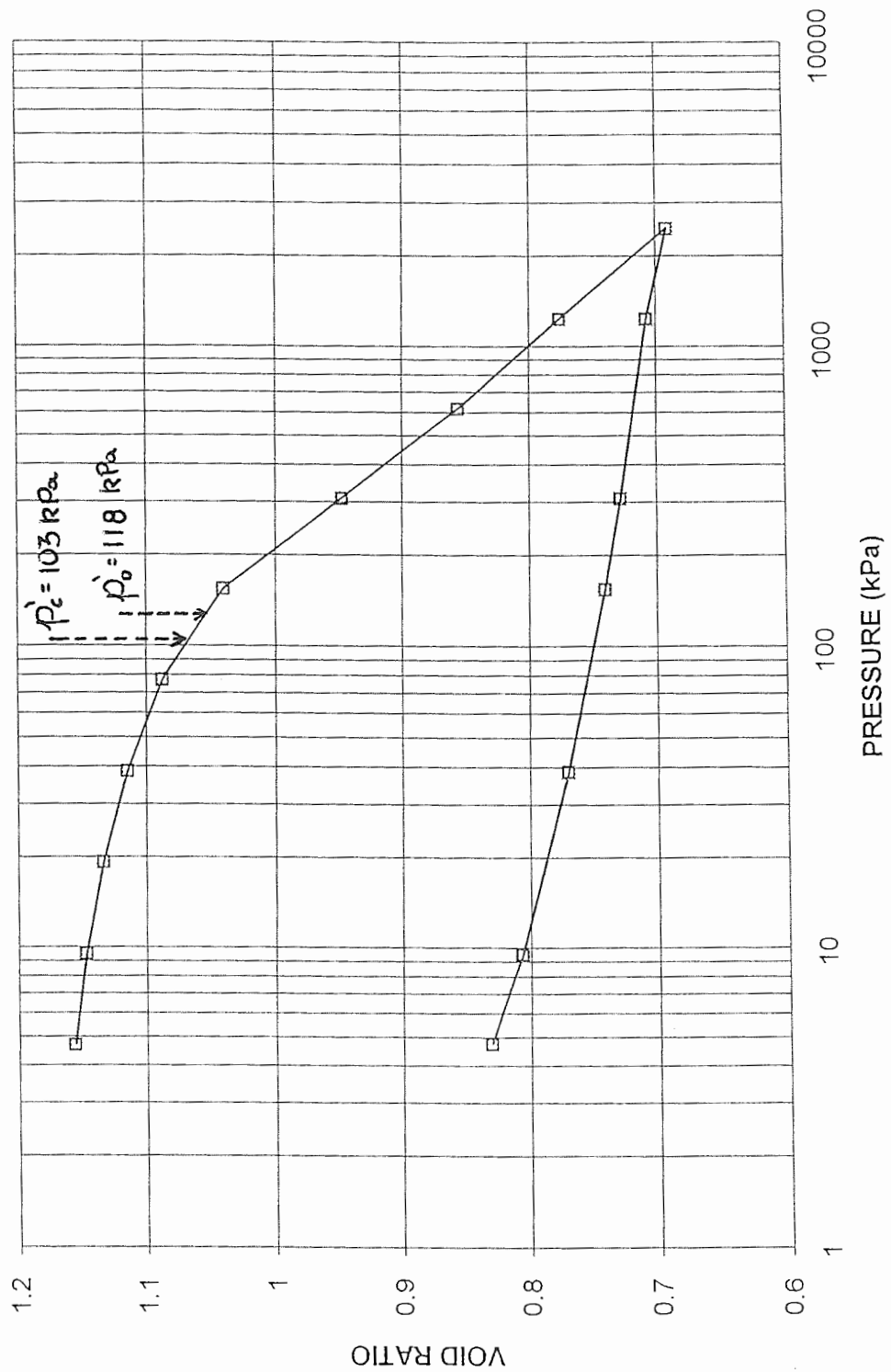


FIGURE B7-S

BOREHOLE 99S-5 ST#2

APPLIED PRESSURE = 154.78 kPa

Silty Clay

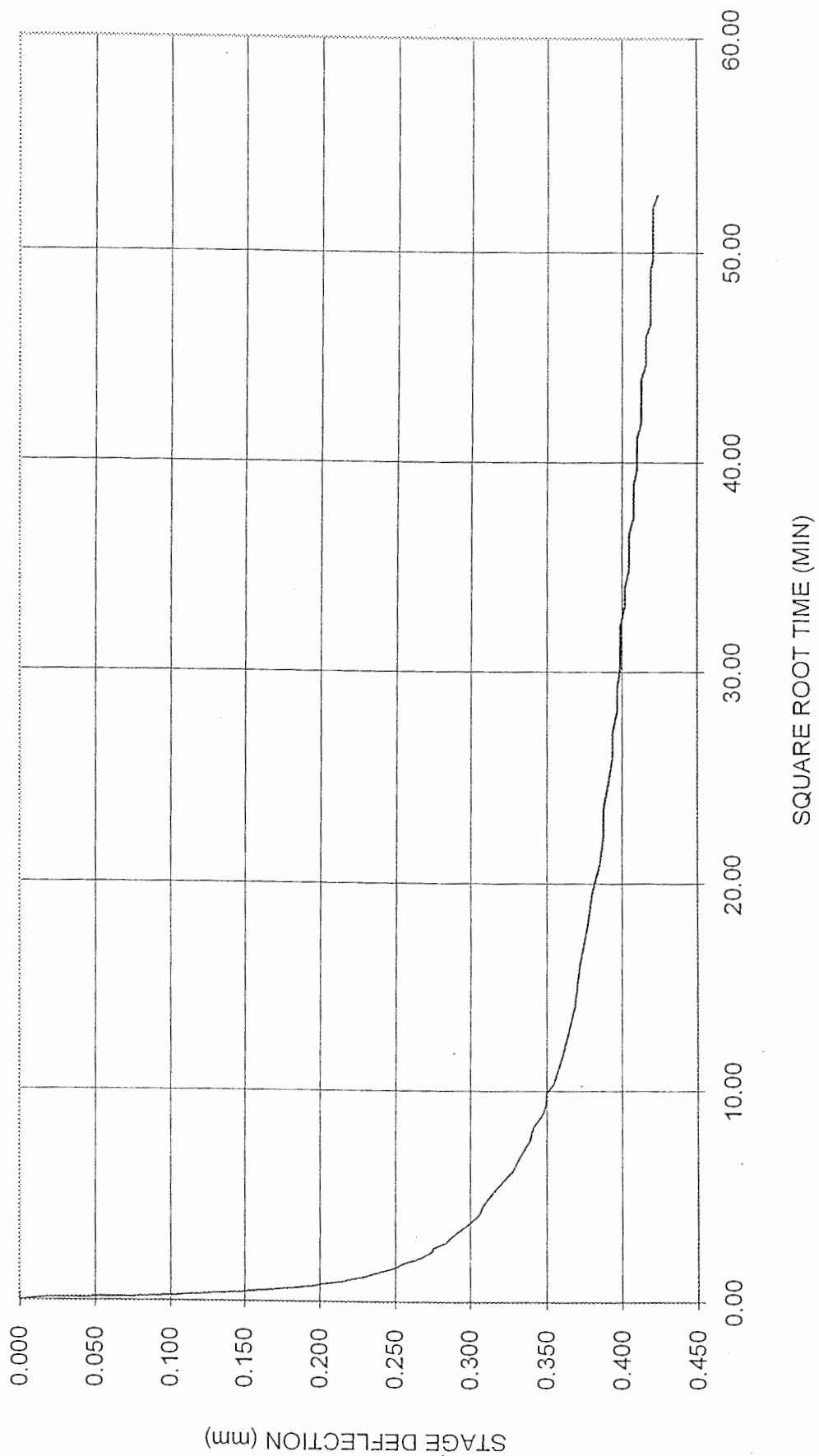
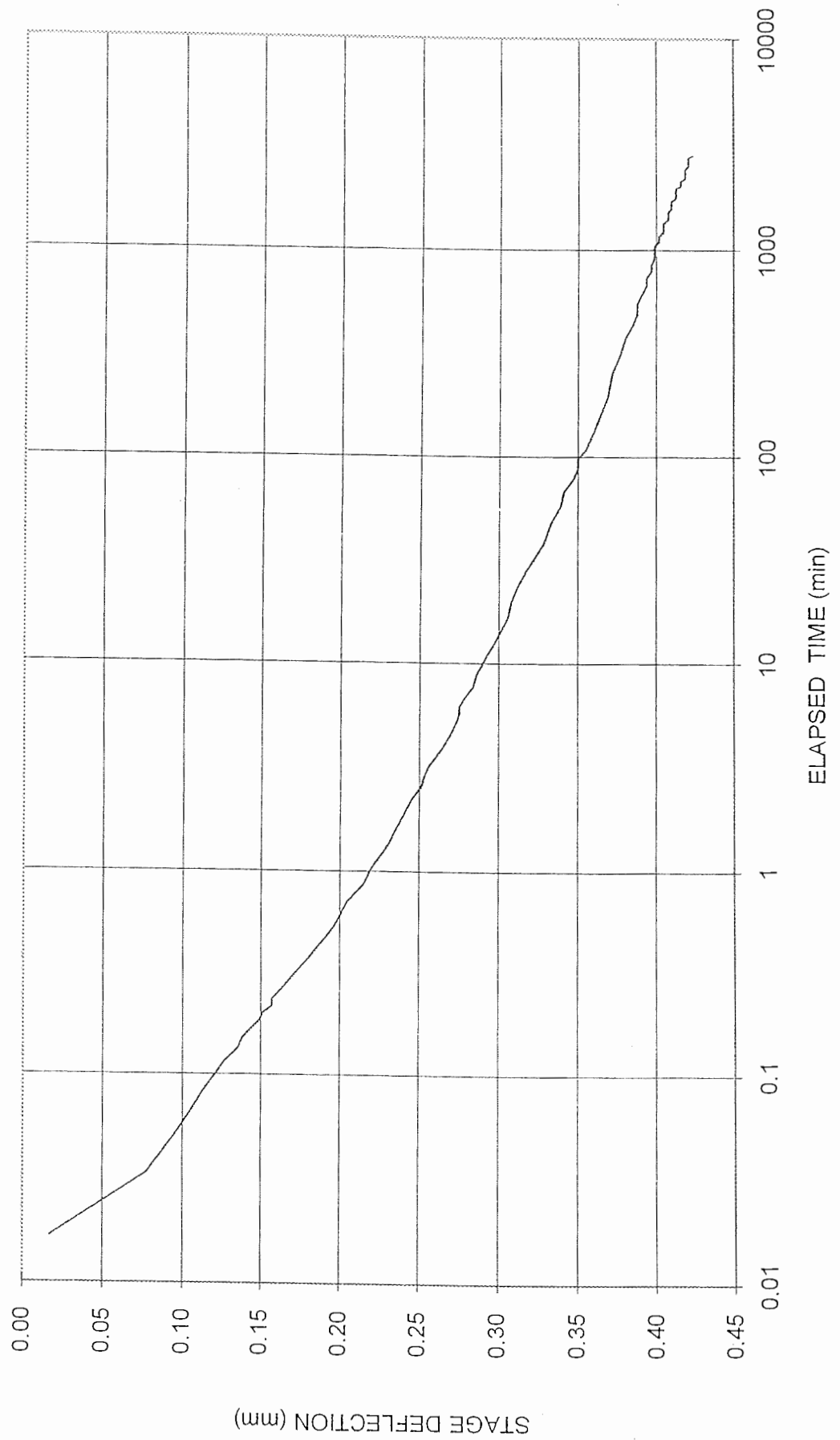


FIGURE B8--S

Silty Clay

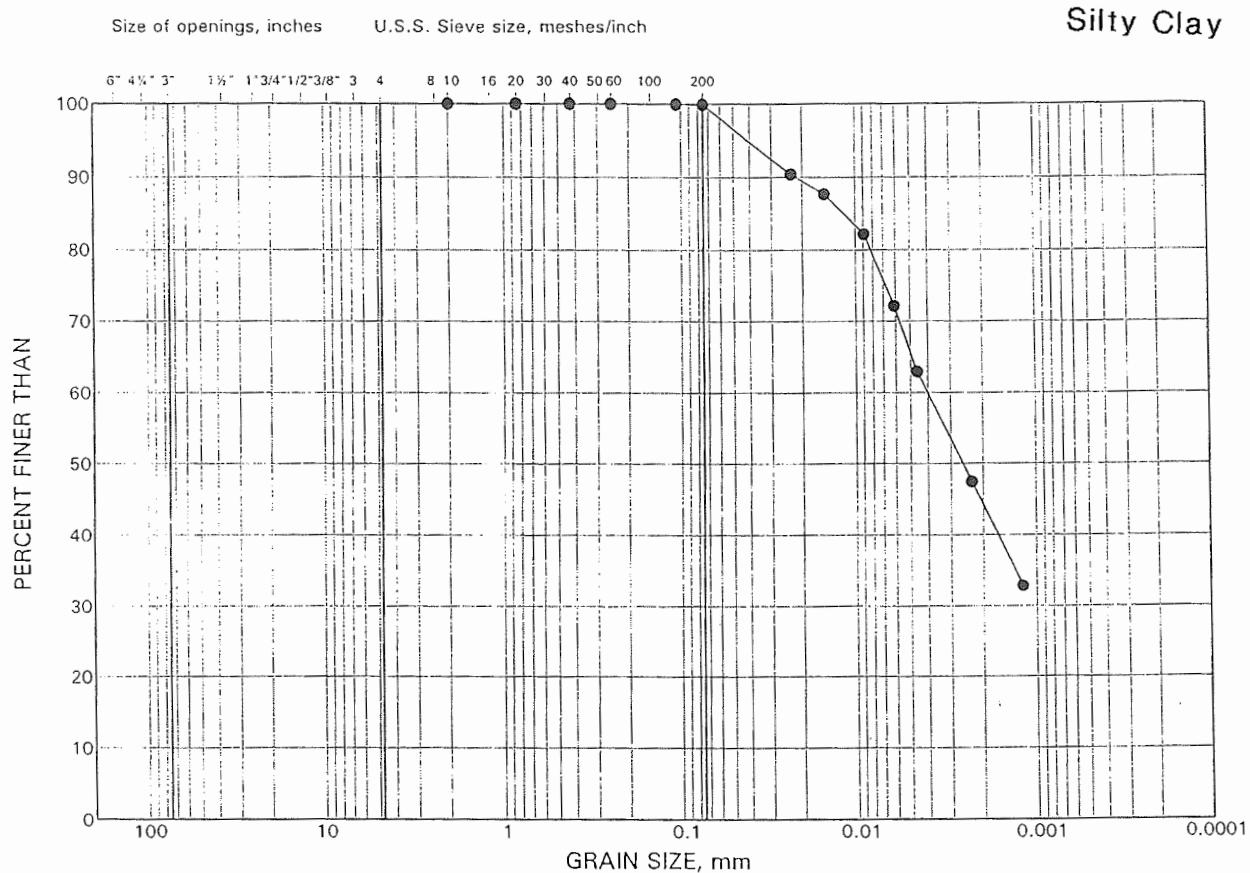
BOREHOLE 99S-5 ST#2

APPLIED PRESSURE = 154.78 kPa



GRAIN SIZE DISTRIBUTION

FIGURE B9-S



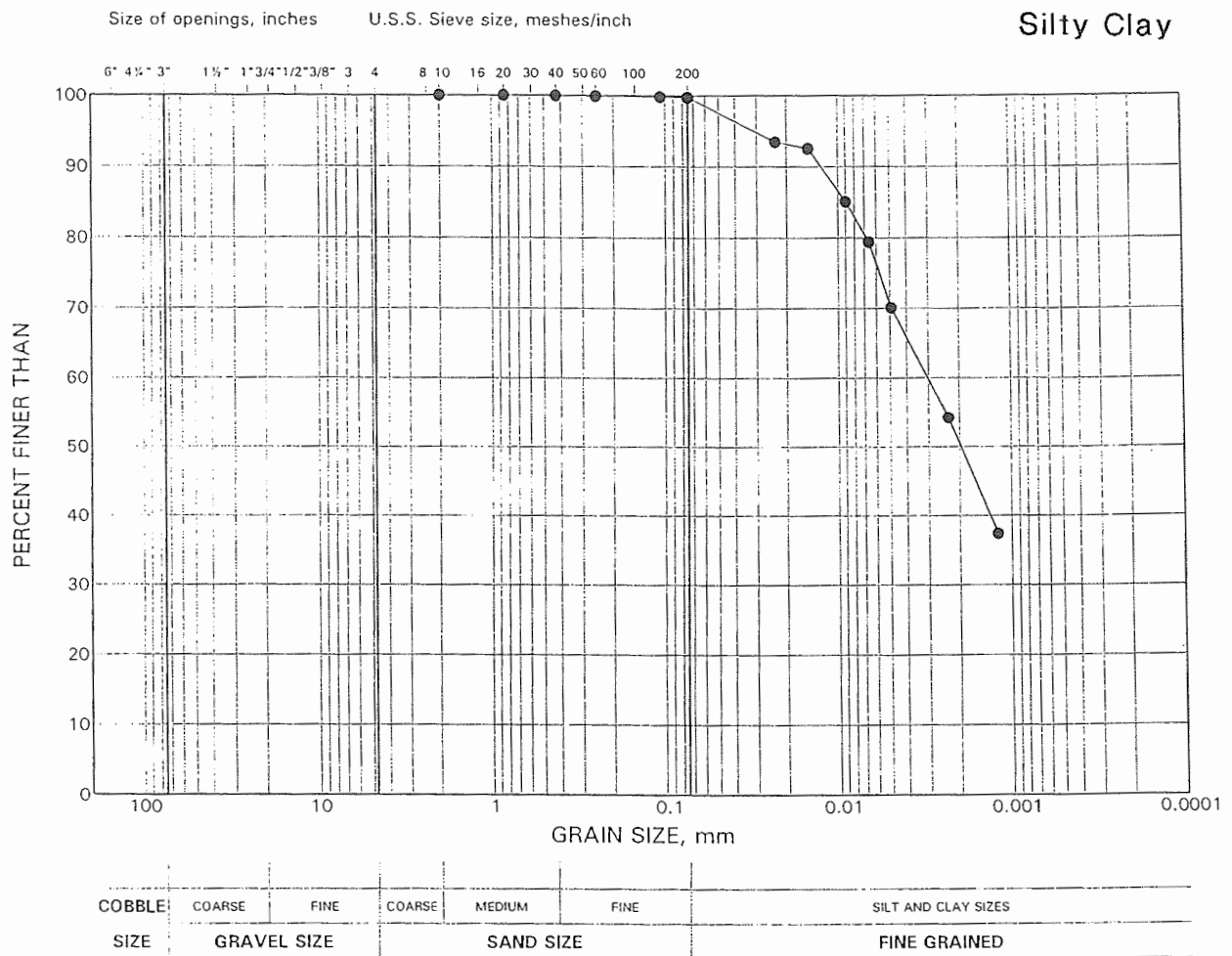
COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	99S-2	ST#2	17.7-18.3

GRAIN SIZE DISTRIBUTION

FIGURE B10-S



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	99S-5	ST#2	16.7-17.4

Appendix C

Limit Equilibrium Analysis

Selected Runs

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy11 - Burk's Falls - Bernard Creek - NBL
 April 13 2004
 Side Slopes
 Short Term - Earth Fill

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Earth Fill	22	0	30	0
Existing Fill	22	0	32	0
Silt and Sand	20	0	29	0
Silty Clay	18	40	0	.25
Sand	21	0	29	0
Hard Bottom	(Infinitely Strong)			

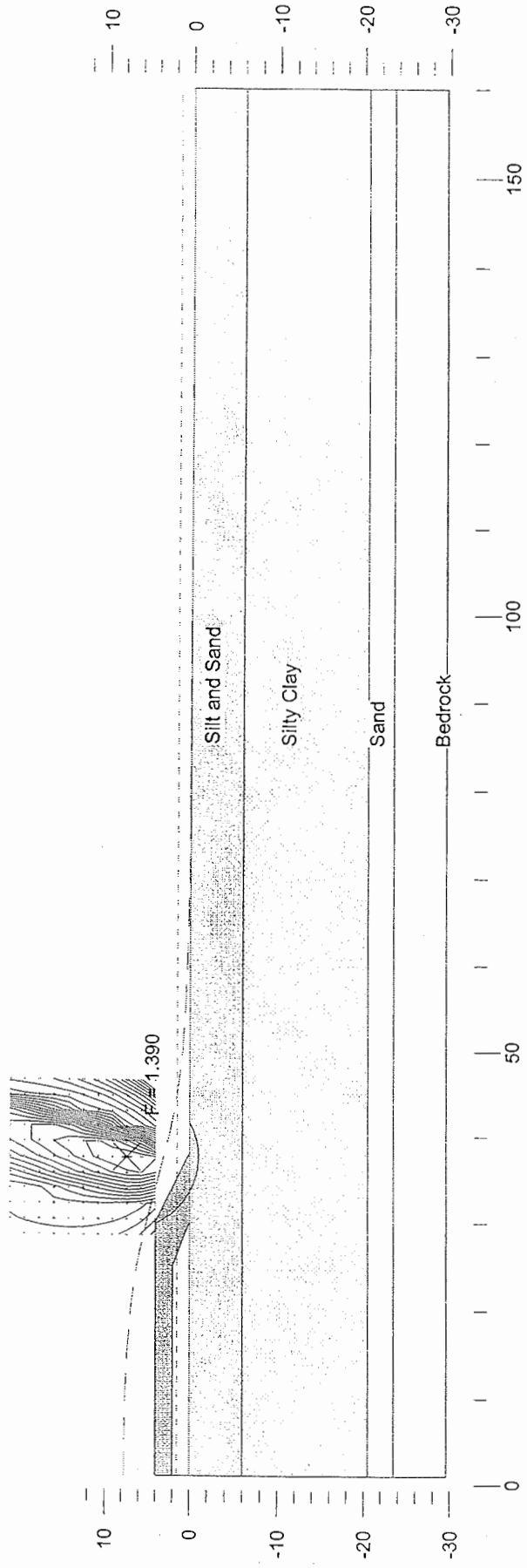


Figure C1

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy11 - Burk's Falls - Bernard Creek - NBL
 April 13, 2004
 Side Slopes
 Long Term - Earth Fill

Earth Fill	Gamma C kN/m3	Phi deg	Piezo Surf.
Existing Fill	22	30	1
Silt and Sand	22	32	1
Silty Clay	20	29	1
Sand	18	28	2
Hard Bottom	21	29	2
(Infinitely Strong)			

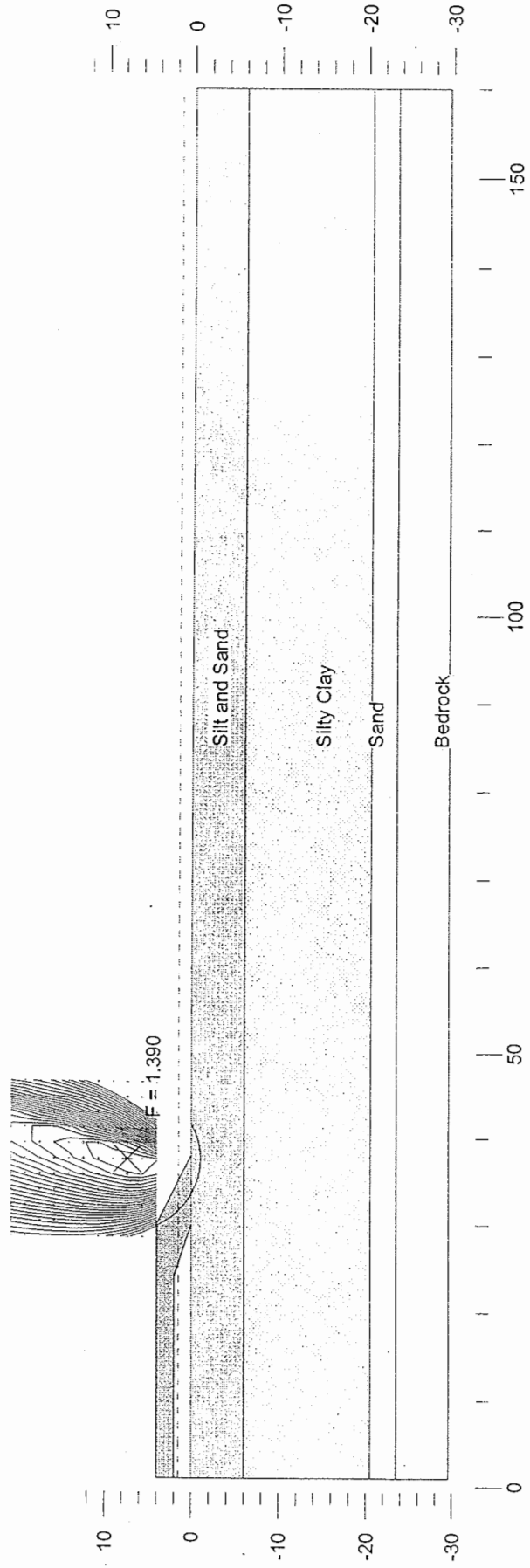


Figure C2

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy11 - Burk's Falls - Bernard Creek - NBL
 April 13, 2004
 Side Slopes
 Long Term - Rock Fill

	Gamma C	Phi	Piezo
	kN/m ³	kPa	deg Surf.
Rock Fill	20	0	42 1
Existing Fill	22	0	32 1
Sand and Silt	20	0	29 1
Silty Clay	18	0	28 2
Silt	21	0	29 2
Hard Bottom	(Infinitely Strong)		

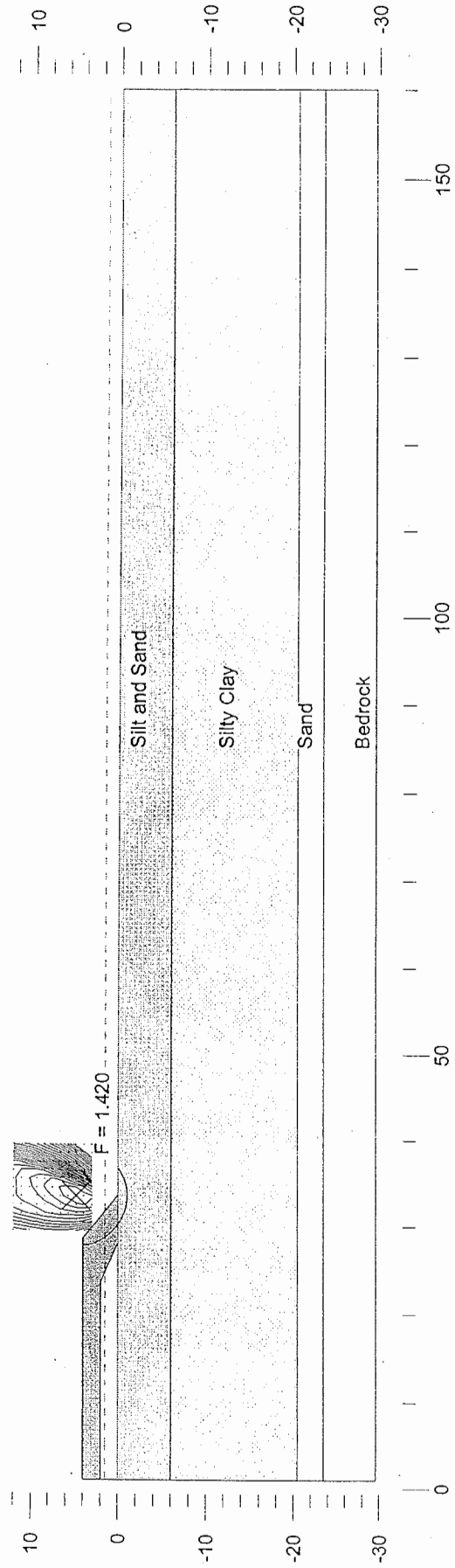


Figure C3

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy11 - Burk's Falls - Bernard Creek - NBL
 April 13, 2004
 Side Slopes
 ShortTerm - Rock Fill

	Gamma C	Phi	Min	Piezo	Ru
	kN/m3	deg	c/p	Surf.	
Rock Fill	20	0	0	1	0
Existing Fill	22	0	0	1	0
Silt and Sand	20	0	0	1	0
Silty Clay	18	40	0	3	.25
Sand	21	0	0	2	0
Hard Bottom	(Infinitely Strong)				

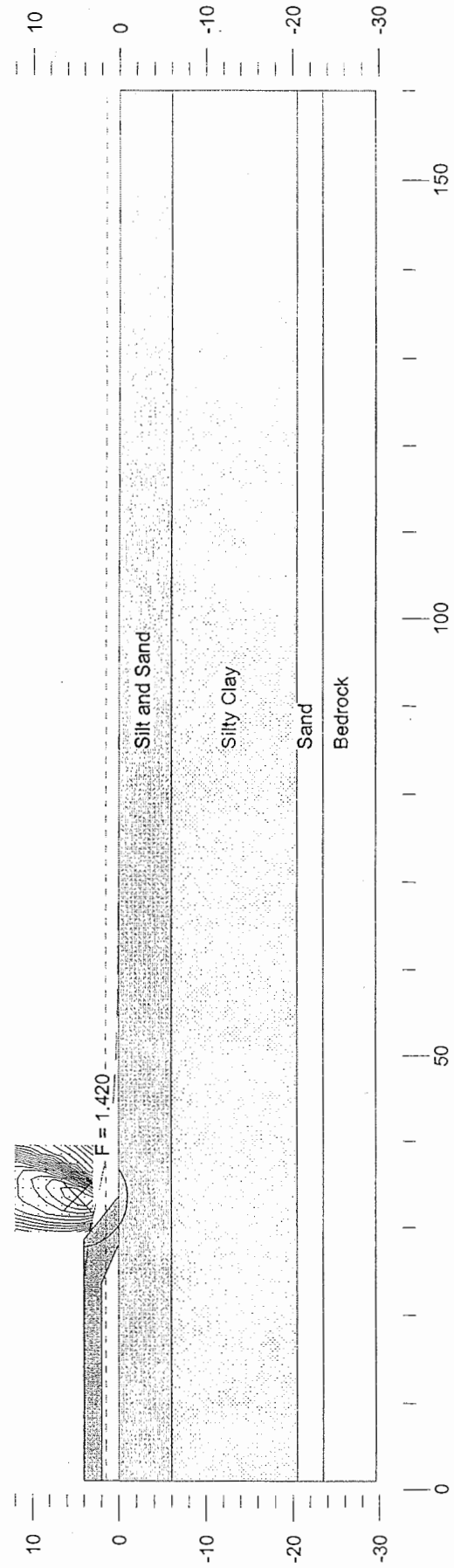
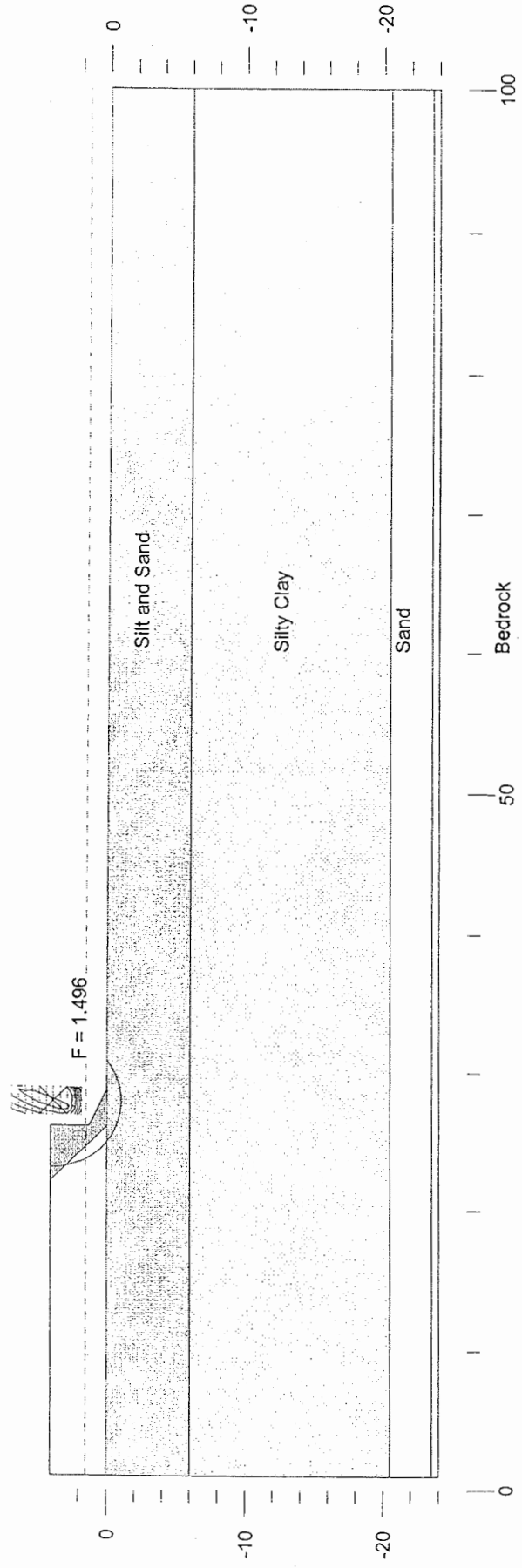


Figure C4

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy11 - Burk's Falls - Bernard Creek - NBL
 April 13 2004
 Abutment
 Long Term - Earth Fill

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Granular A	23	0	35
Earth Fill	22	0	30
Silt and Sand	20	0	30
Silty Clay	18	0	28
Sand	21	0	29
Hard Bottom	(Infinitely Strong)		



Thurber Engineering Ltd. - Toronto
 19-1423-12
 Hwy11 - Burk's Falls - Bernard Creek - NBL
 April 13, 2004
 Temporary Forward Slope
 Short Term - Earth Fill

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Earth Fill	22	0	0	1
Silt and Sand	20	0	0	1
Silty Clay	18	40	0	3
Sand	21	0	0	2
Hard Bottom	(Infinitely Strong)			

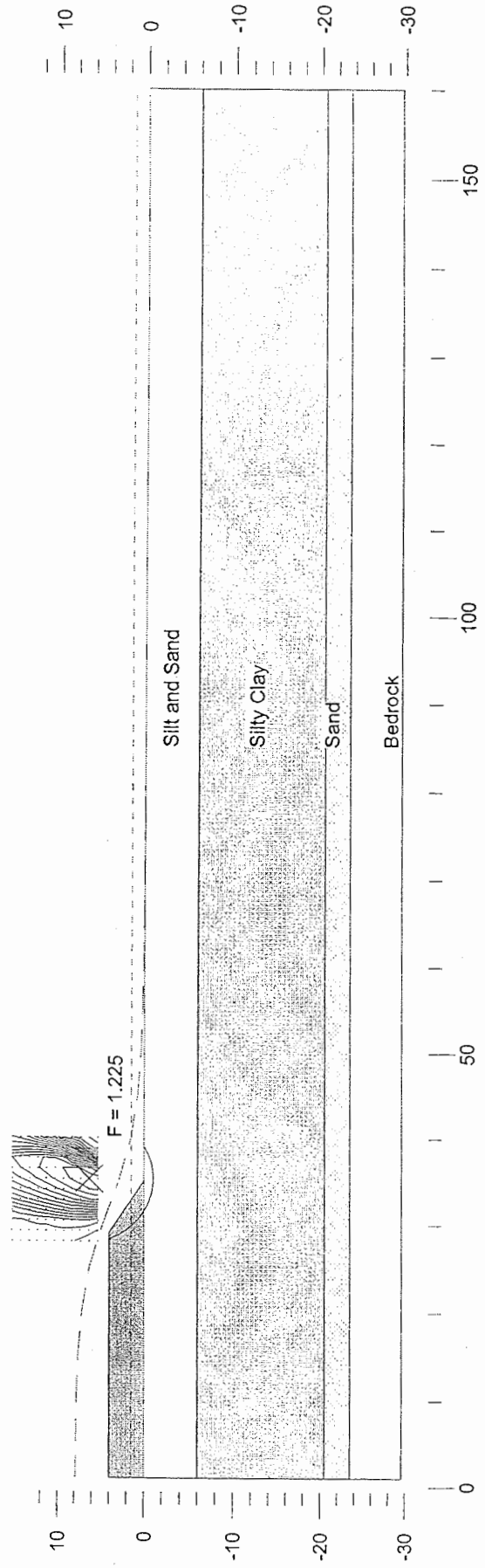


Figure C6

Appendix D

Tables from the Report:

“Supplementary Design Report – Embankments along Highway 11 – Sta.13+260 to Sta.14+600 – Robins Road/Black Creek Road I/C Underpass Structure – Approach Embankments and Access Ramps – Highway 11, Burk’s Falls to South River, Ontario. G.W.P. 5079-06-00; W.P. 742-93-00 – Geocross Number: 31E-234

January 3, 2007”

HIGHWAY 11 - EMBANKMENTS (Station 13+260 to 14+600)
SETTLEMENTS DUE TO PRIMARY CONSOLIDATION - SUMMARY

Location	Fill Material	Station	Main Embankment				Construction Stage	Height at this Stage (m)	Height at Previous Stage (m)	Total Settlement		Surcharge Removal Time for End of Primary Consolidation (years)	No wick drains			With Wicks	
			Maximum Height (m)		Slope Inclination					Due to Primary Consolidation			Primary Consolidation Settlement After Surcharge Removal (mm)			Wick Spacing for 98% Consolidation	
			Top of Pavement	Top of Surcharge	Embankment (H:V)	Surcharge (H:V)				With surcharge (time-independent)	Without surcharge (time-independent)		Surcharge Left in Place for:			During Surcharge	
2m SURCHARGE																	
SBL	Select Subgrade Material (SSM)	13+260 to 13+320	3.0	5.0	4:1	1.5:1	1 of 1	5.0	0.0	730	610	4	400	310	180	2.1	
		13+320 to 13+570	4.0	6.0	4:1	1.5:1	1 of 1	6.0	0.0	890	780	5	520	410	250	2.1	
		13+570 to 13+700 (*)	4.3	6.3	4:1	1.5:1	1 of 1	6.3	0.0	750	650	3	360	250	120	2.1	
		13+700 to 14+250	3.0	5.0	4:1	1.5:1	1 of 1	5.0	0.0	470	390	3	190	120	40	1.8	
		13+700 to 14+250 (**)	2.0	4.0	4:1	1.5:1	1 of 1	4.0	0.0	360	270	2	110	60	0	1.8	
		14+250 to 14+600	3.5	3.5 (4)	4:1	(4)	1 of 1	3.5	0.0	50 (4)	50 (4)	2 months	0 (4)	0 (4)	0 (4)	(6)	
NBL	Select Subgrade Material (SSM)	13+260 to 13+350	1.0/2.0 (3)	3 (3)	4:1	1.5:1	1 of 1	3.0	0.0	330	170	2	70	20	0	2.1	
		13+350 to 13+600	2.5/3.0 (3)	4.5 (3)	4:1	1.5:1	1 of 1	4.5	0.0	670	490	2	220	110	0	2.1	
		13+600 to 13+650 (*)	2.0/5.0 (3)	4 (3)	4:1	1.5:1	1 of 1	4.0	0.0	760	590	3	280	160	20	2.1	
		13+650 to 13+705	1.7/4.0 (3)	3.7 (3)	4:1	1.5:1	1 of 1	3.7	0.0	580	320	2	130	40	0	2.1	
		13+705 to 13+750	2.0/2.0 (3)	4 (3)	4:1	1.5:1	1 of 1	4.0	0.0	270	140	3 months	0	0	0	(6)	
		13+750 to 13+900	1.0/1.0 (3)	(4)	4:1	1.5:1	1 of 1	1.0	0.0	140	50	(4)	(4)	(4)	(4)	(6)	
		13+900 to 14+110	0.0(7) /1.0	(4)	4:1	(4)	(7)	0.0	0.0	(4)	(7)	(4)	(4)	(4)	(4)	(6)	
		14+110 to 14+350 (**)	1.5/(8)	(4)	4:1	(4)	(8)	0.0	0.0	(4)	(8)	(4)	(4)	(4)	(4)	(6)	
		14+350 to 14+470	2.0/2.0	2.0 (4)	4:1	1.5:1	1 of 1	2.0	0.0	(4)	30	(4)	(4)	(4)	(4)	(6)	
		14+470 to 14+510	(8)	(4)	4:1	(4)	(8)	0.0	0.0	(4)	(8)	(4)	(4)	(4)	(4)	(6)	
		14+510 to 14+600	2.2/2.5 (3)	2.2 (4)	4:1	1.5:1	1 of 1	2.2	0.0	(4)	40	(4)	(4)	(4)	(4)	(6)	
3.5m SURCHARGE																	
SBL	Select Subgrade Material (SSM)	13+260 to 13+320	3.0	6.5	4:1	1.5:1	1 of 1	6.5	0.0	790	610	4	430	320	170	2.1	
		13+320 to 13+570	4.0	7.5	4:1	1.5:1	1 of 1	7.5	0.0	940	780	5	560	420	240	2.1	
		13+570 to 13+700 (*)	4.3	7.8	4:1	1.5:1	1 of 1	7.8	0.0	800	650	3	350	230	90	2.1	
		13+700 to 14+250	3.0	6.5	4:1	1.5:1	1 of 1	6.5	0.0	540	390	3	190	120	30	1.8	
		13+700 to 14+250 (**)	2.0	5.5	4:1	1.5:1	1 of 1	5.5	0.0	360	270	2	170	70	0	1.8	
		14+250 to 14+600	3.5	3.5 (4)	4:1	1.5:1	1 of 1	3.5	0.0	50 (5)	50 (4)	2 months	0 (4)	0 (4)	0 (4)	(6)	
NBL	Select Subgrade Material (SSM)	13+260 to 13+350	1.0/2.0 (3)	4.5	4:1	1.5:1	1 of 1	4.5	0.0	390	170	1	40	0	0	2.1	
		13+350 to 13+600	2.5/3.0 (3)	6.0	4:1	1.5:1	1 of 1	6.0	0.0	750	490	2	170	50	0	2.1	
		13+600 to 13+650 (*)	2.0/5.0 (3)	5.5	4:1	1.5:1	1 of 1	5.5	0.0	850	590	2	230	100	0	2.1	
		13+650 to 13+705	1.7/4.0 (3)	5.2	4:1	1.5:1	1 of 1	5.2	0.0	720	320	1	60	0	0	2.1	
		13+705 to 13+750	2.0/2.0 (3)	5.5	4:1	1.5:1	1 of 1	5.5	0.0	310	140	2 months	0	0	0	(6)	
		13+750 to 13+900	1.0/1.0 (3)	(4)	4:1	1.5:1	1 of 1	1.0	0.0	(4)	50	(4)	(4)	(4)	(4)	(6)	
		13+900 to 14+110	0.0(7) /1.0	(4)	4:1	(4)	(7)	0.0	0.0	(4)	(7)	(4)	(4)	(4)	(4)	(6)	
		14+110 to 14+350 (**)	1.5/(8)	(4)	4:1	(4)	(8)	0.0	0.0	(4)	(8)	(4)	(4)	(4)	(4)	(6)	
		14+350 to 14+470	2.0/2.0	2.0 (4)	4:1	(4)	1 of 1	2.0	0.0	(4)	30	(4)	(4)	(4)	(4)	(6)	
		14+470 to 14+510	(8)	(4)	4:1	(4)	(8)	0.0	0.0	(4)	(8)	(4)	(4)	(4)	(4)	(6)	
		14+510 to 14+600	2.2/2.5 (3)	2.2 (4)	4:1	(4)	1 of 1	2.2	0.0	(4)	40	(4)	(4)	(4)	(4)	(6)	

- Notes:
- (1) EPP: Excess Pore Pressure or groundwater pressure in excess of hydrostatic
 - (2) 0.9 / 0.1 / 0.05 - refer to assumed Bbar values used in the analysis and loading associated the Current Stage/Last Stage/Before Last Stage
 - (3) Minimum /Maximum heights above existing Hwy11 or existing ground surface
 - (4) No surcharge in this section
 - (5) Settlement calculated without any surcharge
 - (6) Wick drains not required
 - (7) Proposed NBL elevation is very close to existing Hwy 11
 - (8) Proposed NBL elevation is partly in cut
 - (*) Bernard Creek Bridge South and North Abutments at St. 13+610 and 13+640, respectively
 - (**) Robins Road/Black Creek Underpass at St.14+160

HIGHWAY 11 - MAINLINE EMBANKMENTS (Station 13+260 to 14+600)
DEPTH OF FILL REPLACEMENT WITH EPS

Location	Fill Material	Station	Embankment Height to top of Pavement (m)	Post Construction Settlements (mm) (**) (10)										No Surcharge				2m Surcharge				3.5m Surcharge			
				Without fill replacement with EPS										Depth of Fill Replacement with EPS for Post-Construction Settlement Equal to:				Depth of Fill Replacement with EPS for Post-Construction Settlement Equal to:				Depth of Fill Replacement with EPS for Post-Construction Settlement Equal to:			
				Wick Spacing	No Surcharge	Surcharge = 2.0m	Surcharge = 3.5m	50mm	100mm	25mm	50mm	100mm	25mm	50mm	100mm	25mm	50mm	100mm							
SBL	Select Subgrade Material (SSM)	13+260 to 13+320	3.0	2.1 m	(9)	96	82	-	-	-	1.0 m	0.0 m	-	1.0 m	0.0 m	-	1.0 m	0.0 m							
		13+320 to 13+560	4.0	2.1 m	(9)	102	90	-	-	-	1.5 m	0.0 m	-	-	1.0 m	0.0 m	-	1.0 m	0.0 m						
		13+560 to 13+580	4.3	2.1 m	(9)	166	156	-	-	-	2.5 m	-	-	-	2.0 m	-	-	2.0 m	-						
		13+580 to 13+610 (S.Abut. (*)	4.3	2.1 m	(9)	166	156	-	-	-	-	4.0 m	-	-	-	-	-	-	-						
		13+610 to 13+670 (*)	4.3	2.1 m	(9)	166	156	-	-	-	-	4.0 m	-	-	-	-	-	-	-						
		13+670 to 13+700	4.3	2.1 m	(9)	166	156	-	-	-	2.5 m	-	-	-	-	-	-	-	-						
		13+700 to 14+250	3.0	1.8 m	(9)	81	73	-	-	-	1.0 m	0.0 m	-	-	0.5 m	0.0 m	-	0.5 m	0.0 m						
		13+700 to 14+250 (**)	2.0	1.8 m	(9)	67	56	-	-	-	0.5 m	0.0 m	-	-	0.0 m	0.0 m	-	0.0 m	0.0 m						
		14+250 to 14+600	3.5 (4)	N/R	0	0	0	0.0 m	0.0 m	-	0.0 m	-	-	-	-	0.0 m	0.0 m	-	0.0 m	0.0 m					
		13+260 to 13+350	1.0 (2.0) (3)	N/R	(9)	109	94	-	-	-	1.0 m	0.0 m	-	-	1.0 m	0.0 m	-	1.0 m	0.0 m						
NBL	Select Subgrade Material (SSM)	13+350 to 13+550	2.5 (3.0) (3)	2.1 m	(9)	161	145	-	-	-	2.5 m	1.0 m	-	-	2.0 m	1.0 m	-	2.0 m	1.0 m						
		13+550 to 13+580	2.5 (3.0) (3)	2.1 m	(9)	161	145	-	-	-	-	2.5 m	-	-	-	-	-	-	-						
		13+580 to 13+610 (S.Abut. (*)	2.5 (3.0) (3)	2.1 m	(9)	161	145	-	-	-	-	4.0 m	-	-	-	-	-	-	-						
		13+610 to 13+670 (S.Abut. (*)	2.0 (5.0) (3)	2.1 m	(9)	160	137	-	-	-	-	4.0 m	-	-	-	-	-	-	-						
		13+670 to 13+700	1.7 (4.0) (3)	2.1 m	(9)	138	99	-	-	-	1.5 m	-	-	-	-	-	-	-	-						
		13+700 to 13+750	2.0 (2.0) (3)	N/R	66	85	77	0.5 m	0.0 m	-	0.5 m	0.0 m	-	-	-	-	-	-	-						
		13+750 to 13+900	1.0 (1.0) (3) (4)	N/R	36	-	-	0.0 m	0.0 m	-	-	-	-	-	-	-	-	-	-						
		13+900 to 14+110	0.0 (7) (1.0) (4)	N/R	0	-	-	0.0 m	0.0 m	-	-	-	-	-	-	-	-	-	-						
		14+110 to 14+350 (**)	1.5 (6) (4)	N/R	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-						
		14+350 to 14+470	2.0 (2.0) (4)	N/R	0	-	-	0.0 m	0.0 m	-	0.0 m	-	-	-	-	-	-	-	-						
		14+470 to 14+510	(8) (4)	N/R	-	-	-	-	-	-	-	-	-	-	-	-	-	-							
		14+510 to 14+600	2.2 (2.5) (3) (4)	N/R	0	-	-	0.0 m	0.0 m	-	-	-	-	-	-	-	-	-							

Notes:

- (1) EOP: End of Primary Consolidation
 (3) Minimum Maximum heights above existing Hwy 11 or existing ground surface
 (4) No surcharge in this section
 (7) Proposed NBL elevation is very close to existing Hwy 11
 (8) Proposed NBL elevation is partly in cut
 (9) Surcharge is required to compensate for loss of elevation due to settlement due to primary consolidation
 (10) Surcharge left in place for 3 to 6 months after the end of the embankment construction where wick drains are used
 (*) Bernard Creek Bridge South and North Abutments at 13+610 and 16+640, respectively
 (**) Robins Road/Black Creek Underpass at St. 14+160
 (***) Approximately 20 years after the end of construction

Replacement

TABLE 3.38

**HIGHWAY 11 - EMBANKMENTS (Station 13+260 to 14+600)
SETTLEMENTS DUE TO SECONDARY CONSOLIDATION - SUMMARY**

Location	Fill Material	Station	Embankment		Time for EOP (1) (years)	No Wick Drains					Time for EOP (1) (years)	With Wick Drains				
			Maximum Height (m) Top of Pavement	Top of Surcharge		Time After EOP (1) (years)						Time After EOP (1) (years)				
						1	3	6	10	20		1	3	6	10	20
2m SURCHARGE																
SBL	Select	13+260 to 13+320	3.0	5.0	4	2	12	22	31	45	6 months	27	49	65	78	96
	Subgrade Material (SSM)	13+320 to 13+570	4.0	6.0	5	1	11	21	30	44	6 months	30	53	70	83	102
		13+570 to 13+700 (*)	4.3	6.3	3	10	30	48	65	91	6 months	49	87	115	136	166
		13+700 to 14+250	3.0	5.0	3	2	14	23	31	43	6 months	23	41	55	66	81
		13+700 to 14+250 (**))	2.0	4.0	2	5	15	24	31	42	6 months	19	34	46	55	67
NBL	Select	14+250 to 14+600	3.5	3.5 (4)	6 months	0	0	0	0	0	6 months	0	0	0	0	0
	Subgrade Material (SSM)	13+260 to 13+350	1.0/2.0 (3)	3 (3)	2	14	38	59	76	103	6 months	46	83	111	131	161
		13+350 to 13+600	2.5/3.0 (3)	4.5 (3)	2	14	38	59	77	103	6 months	46	83	111	131	161
		13+600 to 13+650 (*)	2.05/0. (3)	4 (3)	3	8	28	46	62	86	6 months	46	82	109	130	160
		13+650 to 13+705	1.7/4.0 (3)	3.7 (3)	2	8	29	46	61	84	6 months	35	67	91	110	138
	Subgrade	13+705 to 13+750	2.0/2.0 (3)	4 (3)	3 months	29	47	60	70	85	-	-	-	-	-	-
	Material	13+750 to 13+900	1.0/1.0 (3)	(4)	1 month	10	19	25	30	36	-	-	-	-	-	-
	(SSM)	13+900 to 14+110	0.0/7.1/1.0	(4)	-	-	-	-	-	-	-	-	-	-	-	-
		14+110 to 14+350 (**))	1.5/ (8)	(4)	-	-	-	-	-	-	-	-	-	-	-	-
		14+350 to 14+470	2.0/2.0	(4)	0	0	0	0	0	0	-	-	-	-	-	-
3.5m SURCHARGE																
SBL	Select	13+260 to 13+320	3.0	6.5	4	1	8	16	23	35	6 months	20	38	53	64	82
	Subgrade Material (SSM)	13+320 to 13+570	4.0	7.5	5	0	8	16	23	36	6 months	23	44	60	72	90
		13+570 to 13+700 (*)	4.3	7.8	3	7	26	44	59	83	6 months	44	79	106	126	156
		13+700 to 14+250	3.0	6.5	3	0	9	17	24	35	6 months	17	33	46	57	73
		13+700 to 14+250 (**))	2.0	5.5	2	1	10	17	23	32	6 months	12	25	35	43	56
NBL	Select	14+250 to 14+600	3.5	3.5 (4)	6 months	0	0	0	0	0	6 months	0	0	0	0	0
	Subgrade Material (SSM)	13+260 to 13+350	1.0/2.0 (3)	4.5	1	23	50	72	90	117	6 months	39	72	97	117	145
		13+350 to 13+600	2.5/3.0 (3)	6.0	2	9	29	47	62	86	6 months	39	72	97	117	145
		13+600 to 13+650 (*)	2.05/0. (3)	5.5	2	8	28	45	60	83	6 months	35	66	90	109	137
		13+650 to 13+705	1.7/4.0 (3)	5.2	1	9	26	41	54	75	6 months	19	41	60	75	99
	Subgrade	13+705 to 13+750	2.0/2.0 (3)	5.5	2 months	24	40	52	62	77	-	-	-	-	-	-
	Material	13+750 to 13+900	1.0/1.0 (3)	(4)	1 month	10	19	25	30	36	-	-	-	-	-	-
	(SSM)	13+900 to 14+110	0.0/7.1/1.0	(4)	-	-	-	-	-	-	-	-	-	-	-	-
		14+110 to 14+350 (**))	1.5/ (8)	(4)	-	-	-	-	-	-	-	-	-	-	-	-
		14+350 to 14+470	2.0/2.0	2.0 (4)	0	0	0	0	0	0	-	-	-	-	-	-
Secondary Consolidation																
	Subgrade	14+470 to 14+510	(8)	(4)	-	-	-	-	-	-	-	-	-	-	-	-
	Material	14+510 to 14+600	2.2/2.5 (3)	2.2 (4)	0	0	0	0	0	0	-	-	-	-	-	-
	(SSM)															

Notes:

- (1) EOP: End of Primary Consolidation
 (3) Minimum /Maximum heights above existing Hwy11 or existing ground surface
 (4) No surcharge in this section
 (7) Proposed NBL elevation is very close to existing Hwy 11
 (8) Proposed NBL elevation is partly in cut
 (*) Bernard Creek Bridge South and North Abutments at St. 13+610 and 13+640, respectively
 (**) Robins Road/Black Creek Underpass at St.14+160

Secondary Consolidation

TABLE 3.30



Non Standard Special Provisions

Appendix E

SUPPLY EQUIPMENT FOR DRIVING PILES - Item No.

SUPPLY EQUIPMENT FOR INSTALLING CAISSON PILES - Item No.

SUPPLY EQUIPMENT FOR INSTALLING DISPLACEMENT CAISSON PILES - Item No.

SHEET PILES - Item No.

H-PILES - Item No.

TUBE PILES - Item No.

WOOD PILES - Item No.

PRECAST CONCRETE PILES - Item No.

CAISSON PILES - Item No.

DISPLACEMENT CAISSON PILES - Item No.

DRIVING SHOES - Item No.

ROCK POINTS - Item No.

RETAPPING PILES - Item No.

Special Provision No. 903S01

October, 2002

Piling

OPSS 903, December 1983, is deleted and replaced with the following:

903.01 SCOPE

This specification covers the requirements for the supply and installation of deep foundation units comprised of wood, steel, concrete or a combination of these materials.

903.02 REFERENCES

This specification refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, General:

OPSS 180 Management and Disposal of Excess Materials

Ontario Provincial Standard Specifications, Construction:

OPSS 904 Concrete

OPSS 905 Steel Reinforcement

OPSS 909 Prestressed Concrete - Precast

OPSS 911 Coating Structural Steel Construction

Ontario Provincial Standard Specifications, Material:

OPSS 1302 Water

OPSS 1350 Concrete (Materials and Production)

OPSS 1440 Steel Reinforcement for Concrete

Canadian Standards Association Standards:

CAN/CSA 3-G40.20/G40.21-M92 - General Requirements for Rolled or Welded Structural Quality

Steel/Structural Quality Sheets

CAN3-056-M79 - Round Wood Piles

CSA-080 Series-M97 - Wood Preservation

W47.1-92 - Certification of Companies for Fusion Welding of Steel Structures

W48.1 - M1991 - Carbon Steel Covered Electrodes for Shielded Metal Arc Welding

W59 - M1989 - Welded Steel Construction (Metal Arc Welding)

American Society for Testing and Materials Standards:

ASTM A 252-93 Welded and Seamless Steel Pipe Piles

ASTM A 328/ A 328M-93A Steel Sheet Piling

American Petroleum Institute:

API 13A-86 Oil Well Drilling Fluid Materials

API 13B Standard Procedures for Field Testing Drilling Fluids

903.03

DEFINITIONS

For the purposes of this specification, the following definitions apply:

Anvil: means the component of a diesel hammer that acts as an impact block for the ram

Bedrock: means a natural solid bed of the hard, stable, cemented part of the earth's crust, igneous, metamorphic or sedimentary in origin which may or may not be weathered. The actual surface of the bedrock, weathered or unweathered, exists immediately below the overburden.

Casing: means open ended enclosing cylindrical steel tubing or pipe permanently installed in the ground with caisson piles that is structurally required and can be used to render a stable excavation hole.

Caisson Pile: means a cast in place deep foundation unit with or without an enclosing liner formed by placing concrete in a bored or excavated hole.

Cap Block: means a material placed on top of the helmet to cushion the blow of the hammer and to attenuate the peak impact energy without causing excessive loss of the impact energy.

Deep Foundation Unit: means a structural member, driven or otherwise installed in the ground to transfer the loads from a structure to soil or rock and derives supporting resistance from the surrounding soil or rock or from the soil or rock strata below its tip or a combination of both.

Displacement Caisson Pile: means a pile formed in the ground by driving a casing or liner by means of a concrete plug or an expendable metal plate and replacing the displaced soil with plain or reinforced concrete.

Driving Shoe: means a reinforcement attached to the bottom of the pile and designed to protect the pile during driving or to penetrate into a hard stratum

Driving to a Set: means driving the pile to a penetration that satisfies pile driving criteria correlated to a required pile resistance

Follower: means a removable extension which transmits the hammer blows to the head of the pile.

Helmet: means a formed steel cap that fits over the top of a pile head to retain in position a resilient cap block.

Jetting: means the use of a jet of water at high pressure directed into the ground below the pile tip to assist its penetration

Linier: means open ended enclosing steel tubing or pipe temporarily installed in the ground to facilitate the construction of caisson piles

Pile: means a relatively slender structural element which is installed, wholly or partly in the ground by driving, drilling, auguring, jetting or other means.

Pile Cap: means a footing or some other structural component used to transfer the load to the piles as well as maintaining them in position.

Pile Cushion: means a pad of resilient material placed between the helmet and the top of a reinforced concrete or timber pile to minimize damage to the head during driving.

Pile Group: means the piles supporting a pile cap.

Pumped Concrete: means a method of transporting concrete through hose or pipe by means of positive and continuous pressure.

Quality Verification Engineer (QVE): means an Engineer who has a minimum of five (5) years experience in the field of installation of piling or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and to issue Certificate(s) of Conformance.

Retapping: means verifying that the specified resistance previously attained has been sustained by imparting appropriate hammer energy to the pile and monitoring pile penetration

Rock Points: means a specially designed steel tip, fitted to piles to enable them to be driven into hard, sound sloped bedrock

Sheet Pile: means a pile that is designed to interlock with adjacent piles and form a continuous wall for the purpose of resisting mainly lateral forces and to reduce seepage

Slurry: means a drilling fluid, consisting of water mixed with one or more of various solids or polymers, used to maintain the stability of the side walls and bottom of an excavation

Stamped: means drawings or details that have been reviewed and stamped "Conforms to Contract Documents". The stamp shall include the date and signature of the Quality Verification Engineer

Tremie: means a hopper with a vertical pipe leading out of the bottom of it, used for placing concrete under water. The foot of the pipe is always submerged in concrete except during commencement of concreting and the upper level of the concrete is always above water level

903.04 SUBMISSION AND DESIGN REQUIREMENTS

All submissions shall bear the seal and signature of an Engineer experienced in this field. This Engineer, under this section, will not be permitted to carry out the work of the Quality Verification Engineer.

The Contractor shall submit to the Quality Verification Engineer for review and stamping, the equipment and installation procedure and the procedure for monitoring installation.

903.04.01 Site Survey

Prior to commencing the work, the Contractor shall submit to the CA, a condition survey of property and structures that may be affected by the work. The survey shall include, but not be limited to, the locations and conditions of adjacent properties, buildings, underground structures, utility services and structures such as walls abutting the site.

903.04.02 Materials

903.04.02.01 Mill Certificates

The Contractor shall submit to the Contract Administrator at the time of delivery one copy of the mill certificate, indicating that the steel meets the requirements for the appropriate standards for H-piles, tube piles, casings and sheet piles.

Where mill test certificates originate from a mill outside Canada or the United States of America the Contractor shall have the information on the mill certificate verified by testing by a Canadian laboratory. The laboratory shall be accredited by a Canadian National Accreditation Body to comply with the requirements of ISO/IEC Guide 25 for the specific tests or type of tests required by the material standard specified on the mill test certificate. The mill test certificates shall be stamped with the name of the Canadian testing laboratory and appropriate wording stating that the material conforms to the specified material requirements. The stamp shall include the appropriate material specification number, the date and the signature of an authorized officer of the Canadian testing laboratory.

903.04.02.02 Concrete

Concrete and concrete work shall conform to OPSS 1350 and OPSS 904. The Contractor shall submit a suitable, site specific concrete mix design that meets the requirements of the hardened concrete specified. The Contractor is responsible for providing plastic concrete with suitable characteristics for installation. The concrete shall be flowable, non segregating concrete that does not exhibit rapid slump loss. The concrete mix design shall be submitted to the Contract Administrator for information purposes only, one (1) week prior to construction.

Wood piles shall not be spliced.

Wood piles shall be according to CSA CAN3-056 and shall be clean and peeled. Treated piles shall be pressure treated with creosote according to CSA 080.

903.05.01 Wood Piles

903.05 MATERIAL

Upon completion of the deep foundation work, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer. The certificate shall state that the work has been carried out in general conformance with the contract documents, specifications and stamped working drawings.

903.04.03.04 Certificate of Conformance

1. Equipment to be used for installation.
2. Procedure for installation
3. Procedure for monitoring installation.

The Contractor shall submit, for information purposes only, one (1) week prior to construction:

903.04.03.03 Displacement Caisson Piles

1. Shop drawings that describe and illustrate equipment, materials
2. Procedure for caisson excavation and construction
3. Procedure for monitoring installation and caisson inspection

The Contractor shall submit, for information purposes only, one (1) week prior to construction:

903.04.03.02 Caisson Piles

1. Type of equipment and hammer details, including Contractors stated potential energy (rated energy) of the hammer, operating efficiency, weight of ram, anvil and helmet
2. Procedure including sequence for pile installation
3. Procedure for monitoring installation

The Contractor shall submit, for information purposes only, one (1) week prior to construction:

903.04.03.01 Driven Piles

903.04.03 Installation

1. The type, source, physical and chemical properties of the bentonite or polymer
2. Slurry mix proportions and procedure
3. Quality Control Plan to control properties of slurry mix
4. Method of disposal

The Contractor shall submit, for information purposes only, one (1) week prior to construction:

903.04.02.03 Slurry

903.05.02	Steel Piles	
903.05.02.01	Steel H Piles	Steel H piles shall be according to CSA G40.20/G40.21 and shall be 350 W grade.
903.05.02.02	Steel Tube Piles	
903.05.02.03	Steel Sheet Piles	Steel tube piles shall be according to ASTM A252 minimum Grade 2.
903.05.02.04	Straightness Tolerance for Steel Piles	Steel sheet piles shall be according to ASTM A328. Steel sheet piles shall not be spliced.
903.05.03	Driving Shoes and Rock Points	All steel piles shall conform to a straightness tolerance of 1.5 mm maximum per metre of length.
903.05.04	Casing for Caissons	Where the contract shows details of "Splice and Driving Shoe Details for Steel 'H' Piles, the Contractor may substitute the Titus "H" Bearing Pile Point, Standard model, in place of the driving shoe details shown. Where the contract shows details of "Oslo Points for HP310 H-Piles" the Contractor may substitute the Titus "H" Bearing Pile Point, Rock Injector model in place of the pile point details shown. Welding of Titus Points shall conform to the manufacturer's specifications. Where the Contractor elects to use any of the above substitutions, the cost shall be deemed to be included in the contract price for the appropriate item.
903.05.05	Steel Reinforcement	Casings shall be according to ASTM A252 Grade 2. If welded they shall be welded by the electric arc method according to CSA W59. The wall thickness specified is the minimum that shall be supplied. The wall thickness shall be increased as required to ensure the casing is not damaged during handling and installation.
903.05.06	Concrete	
903.05.06.01	General	Steel reinforcement shall be according to OPSS 1440.

Concrete shall be according to OPSS 1350.

903.05.06.02 Tube Piles

Concrete shall have a slump of 150 to 180 mm.

903.05.08.03 Caisson Piles

Concrete shall have a slump of 150 to 180 mm. When approved by the Contract Administrator in writing, admixtures may be used. Where the liner is to be withdrawn, sufficient retarder shall be added to prevent arching of concrete during liner withdrawal, and to prevent setting of concrete until after the liner is withdrawn.

903.05.07 Slurry

903.05.07.01 Solids

Bentonite and polymers shall be according to API 13A.

903.05.07.02 Slurry Composition

Slurry shall be according to API 13B

903.05.08 Helmets and Striker Plates

The head of piles shall be protected by a striker plate or a helmet. Helmets shall have adequate and suitable cushioning material. Helmets and striker plates shall distribute the blow of the hammer evenly throughout the cross-section of the pile head.

903.06 EQUIPMENT

The hammers shall be capable of driving the piles and liners/casings to the prescribed depth or to the specified resistance without damage to portions that are not cut off.

903.07 CONSTRUCTION

903.07.01 Subsurface Conditions

A Foundation Investigation Report that describes the subsurface conditions for the project is available, as specified elsewhere in the Contract. The Ministry warrants that the information provided in the Foundation Investigation Report can be relied upon with the following limitations and exceptions:

1. Any interpretation of data or opinions expressed in the report is not warranted.
2. Regarding the data presented in the report, although the raw measured data presented is warranted, the Contractor must satisfy itself as to the sufficiency of the information presented for the intended construction purpose and obtain any updating or additional information as required to facilitate the deep foundation works.

903.07.02 Transportation, Handling, Storage

Piles, casings and reinforcing steel cages shall be transported, stored and handled in such a manner that damage and distortion is prevented and that the strength and integrity are maintained.

903.07.03 Driven Piles

903.07.03.01 Pile Driving Requirements and Restrictions

Piles shall be installed at the locations indicated and to the set or depth specified without being damaged.

Damage to adjacent structures, utilities and fresh concrete shall be prevented during pile installation. Piles shall not be driven within a radius of 7.5 m of concrete which has been in place for less than 72 hours.

The tops of all piles shall be either square to the longitudinal axis of the pile or horizontal as indicated on the Contract Drawings.

Piles shall not be forced into their proper position by the use of excessive manipulation. Pile damage due to excessive driving shall be avoided.

903.07.03.02 Splicing

903.07.03.02.01 General

Splices within 6 m of the pile cut-off shall be certified by the Quality Verification Engineer as being equal to the full strength of the pile. Any damaged material shall be cut-off prior to splicing. The certificate shall be sealed and signed by the Quality Verification Engineer and shall be submitted to the Contract Administrator.

903.07.03.02.02 H Piles, Tube Piles and Sheet Piles

Welding shall be according to CSA W59 and shall be done by a qualified welder employed by a firm certified according to CSA W47.1, Division 1 or Division 2.1.

Steel H piles and steel tube piles may be spliced providing the pieces being spliced are not less than 3 m long. Splices in marine structures shall be located below the low water level unless otherwise encased in concrete.

Sheet piles shall not be spliced without approval by the Contract Administrator.

903.07.03.02.03 Precast Piles

Precast piles shall only be spliced when specified and the splices shall only be made with approved mechanical splicing devices.

903.07.03.03 Concrete in Steel Tube Piles

Concrete in steel tube piles shall be placed according to the OPSS 904 requirements.

903.07.03.04 Cutting Off Piles

903.07.03.04.01 General

903.07.03.04.02	Wood Piles	<p>The length of pile supplied shall be sufficient to ensure there is no damaged material below the cut off. Damaged material at the pile head shall be cut off.</p>
903.07.03.05	Protective Coating for Steel H and Steel Tube Piles	<p>Where wood piles are broomed, splintered or otherwise damaged below the cut-off elevation, the pile shall be considered defective and shall be replaced.</p>
903.07.03.06	Reinforcing Steel	<p>Reinforcing Steel shall be installed according to OPSS 905.</p> <p>The reinforcing steel cage shall be fabricated in one piece.</p> <p>Welding of reinforcing steel and use of splices shall not be done unless specified in the contract.</p>
903.07.04	Caisson Piles	
903.07.04.01	Installation - General	<p>Caissons shall be constructed as specified in the contract.</p> <p>The final bearing elevation shall be as specified in the contract or shall be an elevation determined by the Contract Administrator. When permanent casings are not specified the caisson shall be constructed in a drilled hole with or without the use of a temporary liner or slurry as determined by the Contractor.</p>
903.07.04.02	Excavation	<p>Sidewall stability shall be maintained throughout the excavation and concrete placement operation. Soil cave-in into the excavation hole shall be prevented.</p> <p>Excavation methods shall be such that the sides and bottoms of the hole are straight and free of loose material.</p> <p>Except when founded on sloping rock, the caisson bottom shall be level. On sloping rock, the caisson bottom may be stepped with each step not greater than 3 the diameter of the bearing area.</p>
903.07.04.03	Unwatering	

Where unwatering is required, the Contractor shall effect a dewatering scheme in such a manner as to prevent any disturbance to the base founding material, or prevent subsidence or ground loss that may adversely affect the work of adjacent structures.

903.07.04.04 Backfilling Liners Left in Place

The annular space between a liner permanently left in place and shaft excavation shall be filled with concrete or fluid grout.

903.07.04.06 Concrete

903.07.04.06.01 General

Concrete shall be placed in the caisson according to OPSS 904. Concrete shall be placed immediately following acceptance of the caisson hole by the QVE.

The reinforcement shall not be displaced or distorted during the construction of the caisson. Arching of concrete during casing withdrawal shall be prevented.

The QVE shall provide inspection throughout the concreting operation.

903.07.04.06.02 Concrete Placed in the Dry

The concrete may be placed free fall provided the fall is vertically down the centre of the opening and transverse ties, spacers or other do not impede the free fall. In the event of interference with the concrete free fall, an elephant trunk or other means shall be used to prevent concrete segregation.

Concrete shall be placed in a continuous operation from the bottom to the top of the caisson, or where columns are cast integral with the caisson, to the elevation of the bottom of the column reinforcing cage. The concrete shall be vibrated for the last 1.5 m of the pour.

903.07.04.06.03 Concrete Placed Under Water or Under Slurry

tremie or pumped concrete shall be carried out in one continuous operation. The Contractor shall carry out the tremie or pumping operation to ensure a continuous flow of concrete that prevents the inflow of water or slurry.

903.07.04.07 Reinforcing Steel

The reinforcing steel cage shall be checked to ensure conformance to the approved shop drawings prior to installation and during concrete placement.

903.07.05 Displacement Caisson Piles

903.07.05.01 General

Work shall be carried out in accordance with displacement caisson pile suppliers installation procedures. A permanent liner shall be used when specified.

The pile shall not be extended below the specified pile tip elevation without approval in writing from the Contract Administrator.

903.07.06 Tolerances

903.07.06.01 Driven Piles

1. Cut off 25 mm
2. Deviation from vertical not more than 1 in 50, except in the case of a pile cap or footing supporting only a single row of piles the deviation shall not be more than 1 in 75 in the direction of the span
3. The deviation from the specified inclination for battered piles shall not exceed 1 in 25
4. The centre of the pile at the junction with the pile cap shall be within 150 mm of that specified (measured horizontally) except in the case of a pile cap or footing supported on a single row of piles the deviation shall not be more than 75 mm (measured horizontally) in the direction of the span.

903.07.06.02 Caissons

1. Cut off elevation 25 mm
2. Horizontal location at cut-off not more than 5% of shaft diameter nor 75 mm
3. Vertical alignment not more than 2% of the caisson length from vertical for vertical caissons, nor 2% of the caisson length from the specified inclination for battered caissons

903.08 QUALITY CONTROL

903.08.01 Monitoring Driven Piles

903.08.01.01 General

The driving of piles shall be carefully monitored and controlled and pile driving records produced for each pile. All driving records shall be certified by the Quality Verification Engineer and submitted to the Contract Administrator.

903.08.01.02 Driving to a Set

The founding elevation shall be established by driving to a set determined in accordance with the dynamic formula specified or by the application of the wave equation analysis procedure that verifies the pile resistance. This set shall be established on the first pile of every ten piles driven in a pile group. The other piles shall be controlled by the pile penetration rate in blows per mm that correlates to the set.

When new conditions such as change in hammer size, change in pile size or change in soil material occur, new sets shall be determined.

903.08.01.03 Driving to Bedrock

When driving piles to bedrock, the Contractor shall adequately seat the pile on bedrock without damaging the pile.

Where rock points are used the rock points shall penetrate into the rock. Piles driven using rock points shall be driven to ensure adequate seating on the bedrock without damaging the pile.

When requested by the Contract Administrator, the Contractor shall verify the hammer performance using the Pile Driving Analyzer or other approved equivalent. The Contractor shall provide all instrumentation, related access and assistance for the testing and monitoring as directed by the Contract Administrator.

Hammer performance shall be verified to ensure that the actual potential energy is not less than 90% of the stated potential energy.

903.08.01.04 Hammer Performance

In each pile group, 10% of the piles (actual number of piles to be rounded off to higher number) but no fewer than two piles shall be retapped no sooner than 24 hours after installation of the individual pile to confirm the bearing resistance has been sustained.

Retapping of piles driven to bedrock is not required.

Where the retapping tests indicate the bearing resistance has not been sustained, all piles in the group shall be retapped.

Where the retapping reveals that the bearing resistance of the piles has not been achieved, the piles shall be redriven to the specified resistance. Where piles have risen, the piles shall be redriven to the original depth.

903.08.02 Inspection of Caisson Holes

The caisson holes shall be inspected and approved by the QVE.

903.09 MEASUREMENT FOR PAYMENT

903.09.01 H Piles, Tube Piles, Wood Piles and Precast Concrete Piles

Measurement is in metres of the piling left in place after cut-off.

903.09.02 Sheet Piles

Measurement is in square metres based on the driving lines specified and the length of piling left in place after cut-off.

903.09.03 Driving Shoes and Rock Points

Measurement is for each driving shoe and rock point specified and used.

903.09.04 Caissons and Displacement Caisson Piles

Measurement is in metres of the depth along the centre line between the approved bearing surface at the bottom and the specified elevation at the top.

903.09.05 Retapping Piles

Measurement is lump sum for retapping the piles above and beyond the minimum 10% but no fewer than two piles requirement for the pile group.

For measurement purposes a count will made of the number of piles retapped above and beyond the minimum 10% but no fewer than two piles requirement and the number of piles in the pile group and a ratio will be determined.

Where retapping is not required above and beyond the minimum, no measurement for payment will be made for this item.

903.10 BASIS FOR PAYMENT

**903.10.01 Supply Equipment for Installing Driven Piles - Item
Supply Equipment for Installing Caisson Piles - Item
Supply Equipment for Installing Displacement Caisson Piles - Item**

Payment at the contract price for the above items shall be full compensation for all labour, testing, equipment and material required to do the work.

It will be assumed, for payment purposes, that 50% of the work under this item has been completed when the satisfactory performance of the equipment has been demonstrated to the Contractor Administrator by the installation of one (1) pile. The remaining 50% will be paid on the satisfactory completion of the installation. When the hammer performance is requested to be verified, all costs associated with this work will be included in the contract price when the energy delivered is less than 90% of the stated potential energy (rated energy) specified in the submission.

When the energy is greater than 90% of the stated potential energy (rated energy) stated in the required submission, the cost will be paid as extra work.

**903.10.02 H-Piles - Item
Tube Piles - Item
Precast Concrete Piles - Item
Wood Piles - Item
Displacement Caisson Pile - Item
Caisson Piles - Item
Driving Shoes - Item
Rock Points - Item
Sheet Piles - Item**

Payment at the contract price for the above items shall be full compensation for all labour, equipment and material to do the work

Payment for redriving piles shall be at the contract price for the applicable item(s) above.

903.10.03 Retapping Piles – Item

Payment for retapping the minimum specified number of piles is included in the Pile Item. Where additional retapping is required, payment will be made based on the ratio of the number of piles retapped in a pile group above the minimum requirement, to the total number of piles in that pile group, times the tender price for retapping all piles for that pile group.

WARRANT: Always with these tender items.

Special Provision

The item Concrete Base shall refer to the Concrete Pad as shown in the Contract drawings.

1.0 Scope

This special provision covers the requirements for the construction of the concrete pad associated with the rigid expanded polystyrene approach embankment.

2.0 References

This special provision refers to the following standards, specifications or publications.

Ontario Provincial Standard Specifications, Construction:

OPSS 904 Concrete Structures

OPSS 905 Concrete Reinforcement

OPSS 919 Formwork and Falsework

Ontario Provincial Standard Specifications, Material:

OPSS 1002 Aggregates – Concrete

OPSS 1305 Moisture Vapour Barriers

OPSS 1306 Burlap

OPSS 1308 Joint Filler (Concrete)

OPSS 1315 White Pigmented Membrane Curing Compounds for Concrete

OPSS 1350 Concrete (Materials and Production)

3.0

Submission and Design Requirements

3.1

Submission of Shop Drawings

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of shop drawings and method statement that provides full details of materials and construction procedure. Construction of the concrete pad shall not commence until the submission is returned to the Contractor with the words "permission to construct".

4.0

Materials

4.01 Concrete and concrete materials shall conform to OPSS 1350 with the following exceptions and/or additions

Class of Concrete	30 MPa at 28 days
Coarse Aggregate	19 mm nominal maximum size
Air Content	7% ± 1.5%
Maximum Slump	60 mm

4.02 Burlap

Burlap shall conform to OPSS 1306.

4.03 Moisture Vapour Barrier for Curing

Moisture vapour barrier for curing shall conform to OPSS 1305.

4.04 Curing Compound

White pigmented membrane curing compounds for concrete shall conform to OPSS 1315.

4.05 Water for Curing

Water for curing shall be free of any impurities which would adversely affect the concrete.

4.06 Joint Materials

Expansion joint filler shall conform to OPSS 1308.

The joint sealing compound shall be hot poured rubberized asphalt conforming to OPSS1212.

4.07 Reinforcement

The steel reinforcement shall conform to the requirements of OPSS 1440 and shall be placed in accordance with OPSS 905.

5.0 Construction

5.01 General

The work required includes the construction of the concrete pad as detailed in the Contract Drawings in accordance with the requirements of OPSS 904 unless otherwise noted.

5.02 Preparation Work

5.02.01 Setting Forms

Throughout their entire length, forms shall be set true to line and grade and directly in contact with the polyethylene sheeting over the rigid expanded polystyrene. Forms shall be anchored in such a manner as not to damage the polyethylene or polystyrene.

5.02 Joints

5.02.01 General

Joints shall be of the type and at the locations detailed in the contract.

The saw cutting of the joints shall be performed within sufficient time to prevent cracking.

5.02.02 Transverse Joints – Construction

Transverse construction joints shall be made at the end of each day's run or when interruptions occur in the concreting operation. Transverse construction joints shall be formed at a contraction or expansion joint, except in exceptional cases of plant breakdown or adverse weather conditions. In these exceptional cases, a construction joint may be formed in the mid slab area subject to the provision that the portion of the slab placed, and the portion of the slab to be placed, is not less than 3 m in length.

5.03 Surface Tolerance

The surface of the concrete is to be such that when tested with a 3 m long straightedge placed anywhere, in any direction on the surface, except across the crown or drainage gutters, there shall not be a gap greater than 10 mm between the bottom of the straightedge and the surface of the pavement.

5.04 Traffic

Equipment other than rubber-tire sawing equipment shall not be permitted on the concrete until it has attained a minimum compressive strength of 24 MPa.

A lift of granular no less than 550 mm shall be placed on the concrete pad before traffic is permitted.

Equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

5.05 Sampling and/or Testing

Field sampling and testing of concrete shall conform to the requirements OPSS 904 for construction category 1 in Table 2.

5.06	Measurement for Payment	
5.06.01	Measurement – Concrete Pad	
Measurement is by Plan Quantity as may be revised by Adjusted Plan Quantity of the area of concrete pad placed in squared metres.		
5.07	Basis of Payment	
5.07.01	Concrete Pad	
Payment at the contract price for the above item(s) shall be full compensation for all labour, equipment and material required to do the work.		

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

Special Provision

1.

Scope

This special provision covers the requirements for the supply and construction of the rigid expanded polystyrene embankment fill and associated works as shown on the contract drawings.

2.

References

This special provision refers to the following standards, specifications or publications.

National Standards of Canada

CAN/CGSB - 51.20 M87

ASTM

ASTM D1621 Test Method for Compressive Properties of Rigid Cellular Plastics

ASTM C203 Test Method for Breaking Load and Flexural Properties of Block Type Thermal Insulation

ASTM C177 Test Method for Steady State Heat Flux Measurements and Thermal Transmission Properties by Means of the Heat Flow Apparatus

ASTM D2842 Test Method for Water Absorption by Rigid Cellular Plastics

ASTM D2863 Test Method for Measuring the Minimum Oxygen Content

ASTM D2126 Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

OPSS - Ontario Provincial Standard Specification

OPSS 212 Borrow

OPSS 501 Compaction

OPSS 517 Dewatering

OPSS 1010 Aggregates – Granular A,B,M, and Selected Subgrade Material

OPSS 1605 Expanded Extruded Polystyrene Pavement Insulation

OPSS 1860 Geotextiles

3. Subsurface Conditions

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

4. Definitions

For the purpose of this special provision, the following definitions apply:

Rigid Expanded Polystyrene

Molded rigid blocks produced by a process of pre-expansion, aging and forming of petroleum based raw material.

Rigid Extruded Expanded Polystyrene

Rigid boards made by extrusion of expanded polystyrene beads

Production lot

The quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

Quality Verification Engineer

Means an Engineer with a minimum of five (5) years experience related to the design and/or construction of expanded polystyrene systems of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue of certificate(s) of conformance.

5. Qualification

The Contractor shall have on site, at the commencement of the work, a representative of the rigid expanded polystyrene supplier to give advice on recommended construction procedures.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

6. Submission and Design Requirements

6.1 Submission of Shop Drawings

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the shop drawings and method statement signed and sealed by the Quality Verification Engineer that provides full details of materials and construction procedure.

6.2 Delivery, Storage, Handling and Protection

The Contractor shall submit the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the rigid expanded polystyrene manufacturer's requirement.

6.3 Construction

The contractor shall submit full details of the following:

- a) The method of foundation excavation and preparation.
- b) Construction of levelling pad
- c) The method of placement of expanded polystyrene blocks including temporary ballasting and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer by layer basis
- d) The method and limits of placement of polyethylene sheeting
- e) The method of placement of 125 mm reinforced concrete base pad (or equivalent)
- f) The method of placement of subbase material
- g) The method of placement of side slope cover

Materials

- 7.1 Granular Levelling Pad
- The levelling pad shall consist of a Granular "A" or Granular "B" material with gradation and physical requirements as specified in OPS 1010.

7.2 Rigid Expanded Polystyrene

7.2.1 General

7.2.1.1 The Contractor shall submit:

1. A general statement as to the type, composition, and method of production of the material
2. The manufacturer's name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the rigid expanded polystyrene
3. Certification of compliance of physical and mechanical properties
4. An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the rigid expanded polystyrene
5. The physical and mechanical properties of the rigid expanded polystyrene including:

- a) Geometry
- b) Nominal Density

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry	mm	1200 x 600 x 300	
- Linear		with tolerances \pm	
- Flatness		1%	
- Squareness		10 mm in 3 m \pm	
- Thickness		0.5%	
Compressive Strength	kPa (min)	110	ASTM D1621
Flexural Strength	kPa (min)	240	ASTM C203 (Procedure A)
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Thermal Resistance	m ² °C/W (min for 25 mm thickness)	0.7	ASTM C177
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863 or C518

Table 1 – Material Properties

Requirements shall be as shown in Table 1 and as described below.

7.2.2

Detail Requirements

Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The polystyrene shall be free from defects affecting serviceability.

7.2.1.2

Block Identification

6. Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
7. A sample of the expanded polystyrene material to the Quality Verification Engineer for review.
8. To the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents.

- c) Compressive Strength
- d) Flexural Strength
- e) Thermal Resistance
- f) Dimensional Stability
- g) Flammability
- h) Water Absorption

Water Absorption	% by Volume (max)	4	ASTM D2842
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7.2.2.1

Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel blocks of minimum acceptable dimensions of 1200 mm x 600 mm x 300 mm.

The maximum deviation from the specified linear dimensions shall be $\pm 1\%$. The flatness of the block faces shall be within ± 10 mm of a line formed by a 3 m straight edge.

The maximum difference in corner to corner dimensions (squareness) shall be 0.5%. The thickness shall be within -3 to $+5$ mm.

7.2.2.2

Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 140 kPa at a strain of not more than 5%. The maximum permissible permanent stress level should not exceed 30% of the compressive strength of the material at 5% strain.

7.2.2.3

Flexural Strength

The minimum flexural strength of the polystyrene shall be 240 kPa. The flexural strength shall be determined in accordance to ASTM C203, method 1, Procedure B.2.7.4 Dimensional Stability.

7.2.2.4

Dimensional Stability

Dimensional Stability shall be determined in accordance with ASTM D2126, Procedure G. A tolerance of 1.5% shall be satisfied.

7.2.2.5

Thermal Resistance

The thermal resistance shall be $0.7 \text{ m}^2 \cdot ^\circ\text{C}/\text{W}$ for a 25 mm thickness using the following equation and using the average value from three specimens:

$$R_{25\text{mm}} = R_{\text{measured}} \frac{\text{Thickness (mm)}}{25}$$

The thermal resistance shall be measured in accordance with ASTM C177 or C518.

7.2.2.6

Flammability

The expanded polystyrene shall be classified as to surface burning characteristics in accordance with CAN/ULC - S1022 having a flame spread rating less than 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863

Place, level and compact a layer of Granular 'A' or Granular 'B' material in accordance with OPSS 501 to within ± 30 mm of the design elevation. The levelling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks. The levelling pad shall not be placed on frozen ground.

9.2 Levelling Pad

Foundation excavation shall be carried out to the design elevations shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with Granular 'A' or Granular 'B' material.

9.1 Foundation Excavation

9.0 Construction

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation. Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

8.0 Delivery, Storage and Handling

The expanded polystyrene shall be inert, non-nutritive and highly stable and shall not produce undesirable gases or leachate.

7.2.2.10 Environmental

The expanded polystyrene shall be resistant to biological degradation caused by organisms or enzymes.

7.2.2.9 Biological Resistance

The expanded polystyrene shall be resistant to common inorganic acids and alkalis. A table identifying the chemical resistance as either resistant limited or not resistant shall be submitted.

7.2.2.8 Chemical Resistance

The water absorption as measured by ASTM D2842 shall be limited to 4% by volume.

7.2.2.7 Water Absorption

Installation of Blocks

1. The individually marked blocks shall be placed on the prepared levelling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.
2. Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers. A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.
3. Sloping end adjustments at the abutments shall be accomplished by levelling terraces in the subsoil in accordance with the block thickness.
4. Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.
5. The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.
6. The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction. The proposed method of protection during construction shall be submitted to the Contractor's Quality Verification Engineer for review and to the Contract Administrator for information purposes.
7. Exposed blocks shall be covered immediately to avoid possible burrowing by animals.
8. Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.
9. The top surface and side surfaces of the expanded polystyrene shall be covered with .30 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.
10. The contractor shall install the concrete base pad as detailed elsewhere in the contract.
11. The side slope of the rigid expanded polystyrene embankment shall be covered with granular fill as detailed elsewhere in this contract.
12. The Contractor shall submit details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted at least three weeks prior to the installation of the rigid expanded polystyrene embankments the Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.
13. The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation. Upon completion of the Expanded Polystyrene Embankment the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer.

stating that the Expanded Polystyrene Embankment has been constructed in conformance with the installation procedures and specifications of the contract documents.

10.0 Equipment

All cutting of polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

11.0 Quality Assurance

11.1 Sampling and Testing

11.1.1 General

The Contract Administrator may undertake an independent testing program of the expanded polystyrene. Sampling and testing will be carried out in conformance with the relevant test procedure. The physical and thermal property testing identified in Table 1 will be conducted. The testing shall be conducted by a recognized testing laboratory accredited by the Standards Council of Canada.

11.1.2 Sampling Frequency

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. As a minimum, three blocks shall be tested.

11.1.3 Acceptance/Rejection

Failure of any one of the sample blocks to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the blocks shall be at the Contractor's expense.

12. Measurement for Payment

12.1 Actual Measurement

Measurement will be by volume in cubic metres measured in its original position and based on cross-sections.

13. Payment

13.1 Basis of Payment

The Concrete Base pad and granular levelling pad shall be paid for with the appropriate tender items as detailed elsewhere in the contract.

Payment at the contract price for the above tender item shall be full compensation for all labour, materials and equipment to do the work as described above and no extra payments will be made.

14.0 SHEETING

14.1 Scope of Work

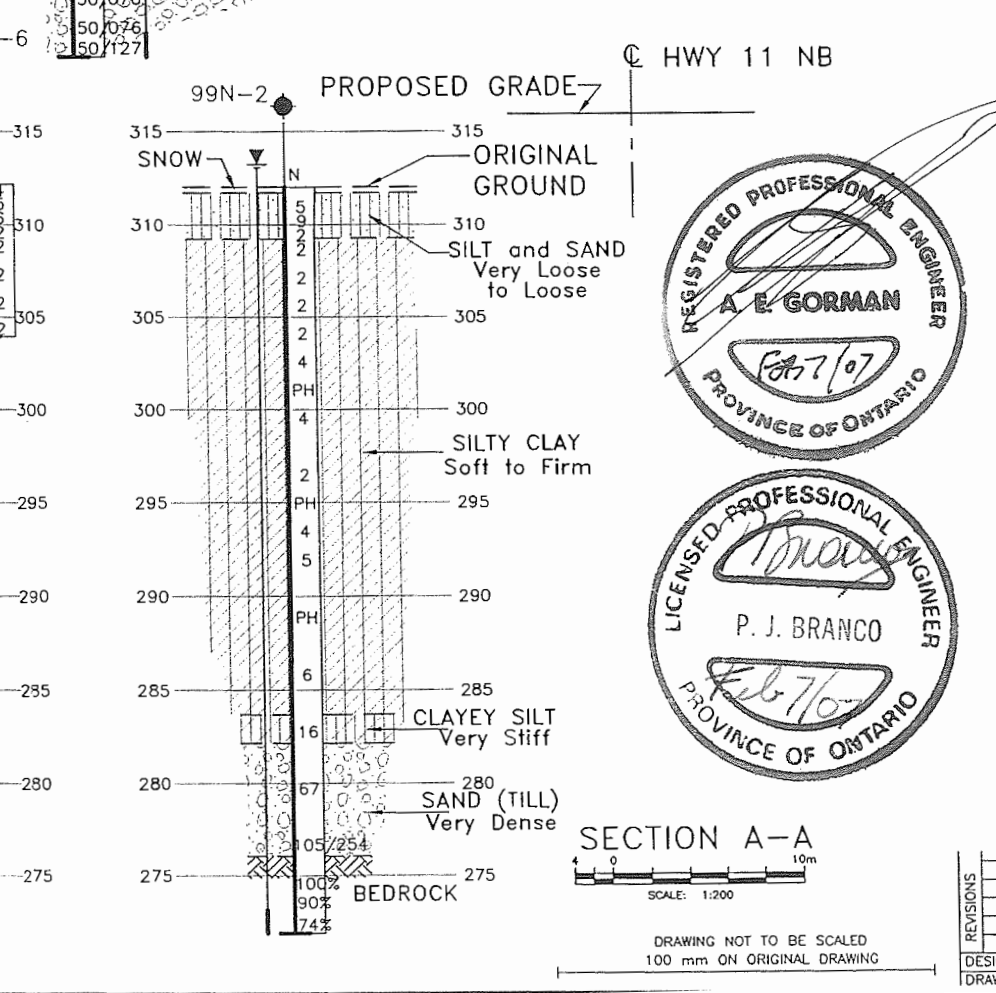
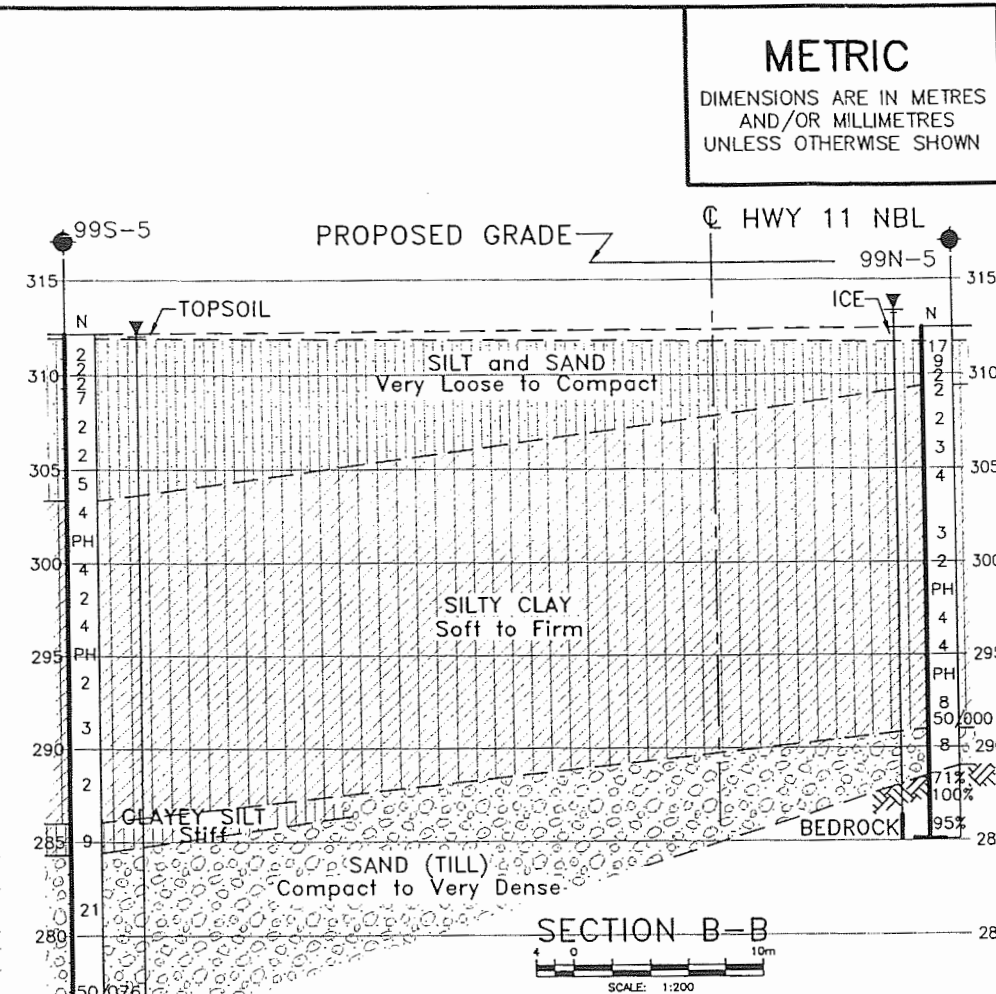
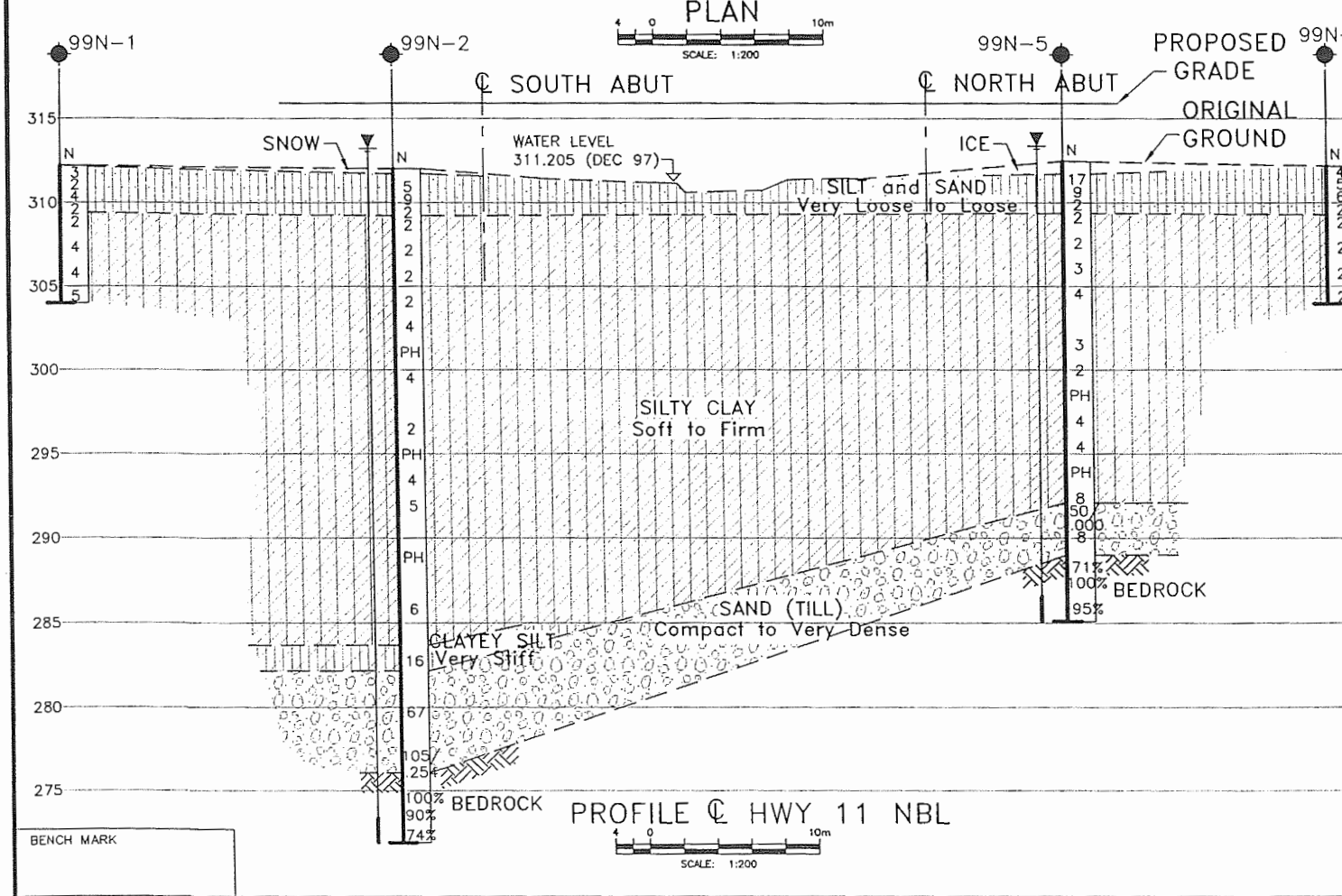
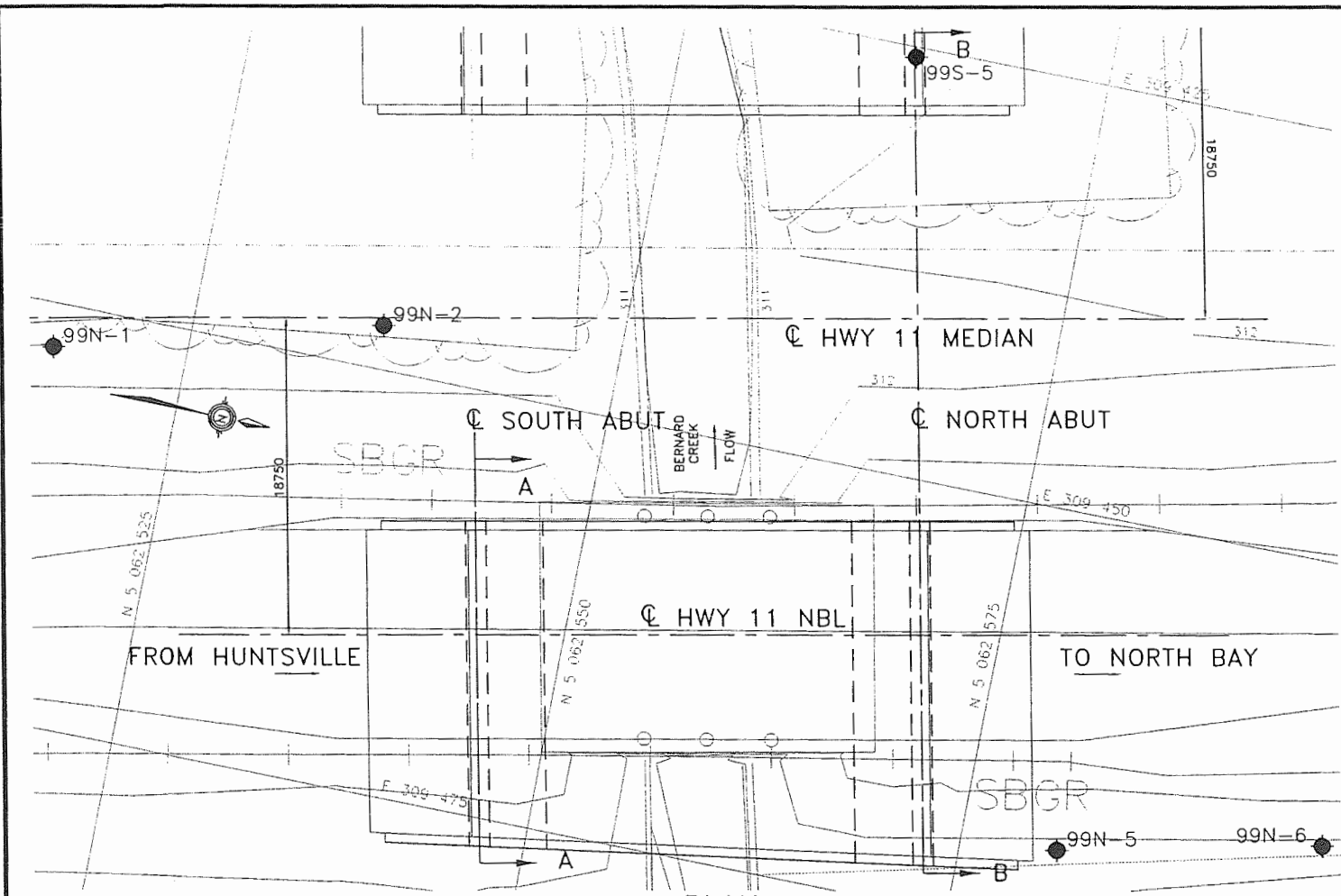
As part of the work of the above noted tender item the Contractor shall supply and install Polyethylene Sheeting as detailed elsewhere in the contract.

14.2 Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all labour, equipment and materials to install the Polyethylene Sheeting as detailed elsewhere in the contract and no extra payment will be made.

Borehole Locations and Soil Strata Drawing

Appendix F



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

HWY 11
CONT No
WP No 755-93-01

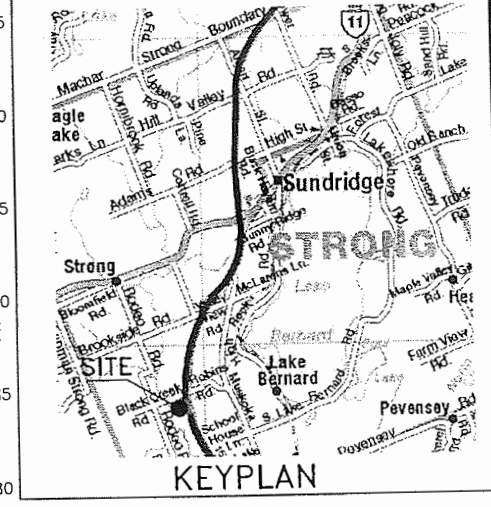
BERNARD CREEK BRIDGE
(NBL)

BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Marshall Macklin Monaghan
CONSULTING ENGINEERS • SURVEYORS • PLANNERS

THURBER ENGINEERING LTD.



LEGEND				
	Borehole by THURBER			
	Dynamic Cone Penetration Test (cone)			
N	Blows /0.3m (Std Pen Test, 475J/blow)			
CONE	Blows /0.3m (60° Cone, 475J/blow)			
PH	Pressure, Hydraulic			
	WL at March, 04, 2004			
	Piezometer			
90%	Rock Quality Designation (RQD)			
NO	ELEVATION	NORTHING	EASTING	
99N-1	312.2	5 062 517.2	309 452.6	
99N-2	312.0	5 062 536.0	309 447.4	
99N-5	312.5	5 062 580.6	309 470.0	
99N-6	312.2	5 062 595.7	309 466.7	
99S-5	312.2	5 062 563.7	309 425.6	
<p align="center">— NOTE —</p> <p>The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.</p>				



REVISIONS		DATE				DESCRIPTION			
DATE	BY	DESIGN	AEG	CHK	CODE	LOAD	DATE	APR. 2004	
DRAWN	SS	CHK	AEG	SITE	ISTRUCT	ISCHFMF	IDWG		

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING