

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT  
BONNECHERE RIVER CROSSING  
HIGHWAY 17, ARNPRIOR TO RENFREW  
ONTARIO  
W.P. 647-92-00, SITE 29-192/1**

**Geocres Number:  
31F – 136**

**Report to  
National Capital Engineering**

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**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT****BONNECHERE RIVER BRIDGE****HIGHWAY 17, ARNPRIOR TO RENFREW****ONTARIO****W.P. 647-92-00, SITE 29-192/1****Geocres Number:****31F-136****PART 1: FACTUAL INFORMATION****1 INTRODUCTION**

This report presents the factual information obtained from a preliminary foundation investigation at the proposed site of a new bridge over the Bonnechere River located approximately 2.4 km west of County Road 6, near Renfrew, Ontario. The new crossing is part of the 2<sup>nd</sup> phase of the project to twin Highway 17 between Arnprior and Renfrew. The proposed bridge is located about 40 m southwest of the existing Highway 17 six span steel girder structure crossing the Bonnechere River.

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.

The Foundation Investigation Report for the existing Highway 17 Bonnechere River structure, prepared by M.T.O in July 1972, was reviewed in preparing this report.

Thurber carried out the investigation as a sub-consultant to National Capital Engineering, under the Ministry of Transportation Ontario (MTO) Agreement Number 4005-A-000157.

**2 SITE DESCRIPTION**

The proposed bridge site crosses the Bonnechere River valley approximately 40 m southwest of the existing Highway 17 bridge crossing, and near the northwest boundary between the Town of Renfrew and the Township of Horton in south-eastern Ontario.

The site is located in the physiographic area known as the Ottawa Valley Clay plain which is comprised of relatively level terrain interrupted by occasional bedrock or sand upland areas. Locally, the Bonnechere river occupies a broad valley which is incised approximately 30 m below the surface of the clay plain. The slope geometry is benched with gently sloping terrain located at the toe of the slope (EL 85 to 90 m), adjacent to the river. A topographic bench is also present at



the mid-slope position (EL 102 to 103 m) on the west valley slope. The steeper terrain between the topographic benches is generally inclined at 40-50%.

The topographic base mapping and profile indicates that the river bed is at about 80 m elevation and the depth of water on Feb 4/04 was about 3 m. The depth of water will vary with the river level.

Drainage in the surrounding areas is typically poor, but locally is improved by the presence of the river valley slopes.

The proposed bridge site is wooded and development outside of the limits of the Town of Renfrew is predominantly rural.

### **3 SITE INVESTIGATION AND FIELD TESTING**

The site investigation and field testing for this site were carried out between October 14 and November 26, 2003. The site investigation consisted of drilling and sampling a total of six boreholes to depths ranging from 18.5 to 34.4 m. The boreholes were numbered BON1 through BON7. A planned Borehole BON5 was not drilled due to inaccessible terrain on the east valley slope. The surveyed locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing.

The proposed borehole locations were marked in the field by surveyors from J.D. Barnes, and utility clearances were obtained by Thurber prior to any drilling being carried out.

George Downing Estate Drilling Limited supplied a CME 75 drill rig mounted on a Nodwell tracked carrier and conducted the drilling, sampling and in-situ testing operations. Hollow stem auger and wet rotary drilling and wireline coring techniques were used to advance the boreholes and disturbed samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in overburden soils. In-situ vane shear tests and thin-walled tube samples were collected from cohesive layers exhibiting lower strength. Where bedrock was encountered, the borehole was advanced up to 3 m into bedrock by wireline coring using a NQ diamond bit.

A Dynamic Cone Penetration Test (DCPT) was carried out adjacent to each borehole to obtain a continuous profile in the upper portion of the deposit. The DCPT profiles are shown on the borehole logs in Appendix A.

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

Upon completion of drilling and sampling, standpipe piezometers were installed in each borehole and then the boreholes were backfilled using drill cuttings.

#### 4 LABORATORY TESTING

All recovered soil samples were returned to Thurber's laboratory where they were subjected to Visual Identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A.

The bedrock cores were returned to the laboratory where they were subject to detailed geologic logging and point load testing to estimate Unconfined Compressive Strength (UCS) of the samples. The results of the testing are summarized on the borehole logs in Appendix A.

Selected samples were subjected to gradation analysis (sieve test) and Atterberg Limit testing. To obtain consolidation parameters of the foundation clay, a one-dimensional consolidation test was carried out on a clay sample collected with a thin-walled sampler from 12.5 m depth in boreholes BON-7. The tests results are summarized on the Record of Borehole sheets in Appendix A and on the plots in Appendix B.

#### 5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in these appendices and on the "Borehole Locations and Soil Strata" Drawing inserted at the end of the report. A description of the stratigraphy is given in the following sections.

In general terms, the subsurface conditions at the site consist of a sequence of marine clay overlying a sequence of glaciofluvial soils which become coarser with depth. The overburden in turn rests on marble bedrock (Pre-Cambrian). The Bonnechere River valley has been eroded locally to about 30 m depth into this sequence and more recent processes have resulted in deposition of colluvium, fill and topsoil horizons on top of the aforementioned sequence. A general description of these soil units is included below:

##### 5.1 Topsoil

Dark brown to black, moist, topsoil was encountered in the boreholes to depths of 25 to 250 mm as shown below.

**TABLE 1: TOPSOIL THICKNESS**

Borehole	Topsoil Thickness (mm)
BON1	25
BON2	100
BON3	250
BON4	150
BON6	50
BON7	125

Topsoil thickness may vary between boreholes and in other areas of the site.

## 5.2 Clayey Silt and Sand (Fill Material)

Fill was encountered beneath the topsoil only in borehole BON-2 at Pier No. 1, located at the toe of the north valley slope. The fill was comprised of two distinct layers consisting of a 0.6 m thick upper layer of clayey silt with some organics overlying a lower 0.8 m thick layer of sand with some gravel, some silt. The fill is brown in colour and in a moist condition. The SPT N-values range from 16 in the upper layer to 29 in the lower layer, and the moisture content varying from 12 to 9%. Fill deposits are often heterogenous in nature and may vary substantially over short distances.

## 5.3 Clayey Silt to Silty Clay

Marine clay was encountered beneath the topsoil and fill at all of the boreholes. The deposit is mainly comprised of silty clay trace sand, but is somewhat heterogeneous varying from clayey silt with some sand to a mixture of interbedded clay and silt. Numerous silty sand to sand pockets and layers up to 150 mm in thickness are present, becoming more frequent in the lower portion of the deposit.

The thickness and base elevation of the marine clay at each foundation element is summarized in Table 2 below:

**TABLE 2 : CLAY THICKNESS AND BASE ELEVATION**

Foundation Element	Borehole Number	Marine clay thickness (m)	Base elevation of clay
West approach	Bon-6	10.1	92.8 m
	Bon-7	>31.0	-
West abutment	Bon-1	11.9	90.5 m
Pier 1	Bon-2	4.6	81.1 m
Pier 2	Bon-3	1.9	83.9 m
Pier 3	Bon-4	2.7	86.8 m

The clay thickness summarized in the table above is greatest under the proposed west approach and west abutment and is considerably thinner under the piers.

The laboratory tests carried out on the silty clay indicate that the upper portion of the deposit has medium plasticity, while portions of the deposit, at elevations lower than about

95 m, are low plastic. The moisture content of the samples from the deposit varied from 51% to 20 %, generally decreasing at greater depths as shown in Figure 1.

The SPT in the deposit indicate that the upper 4 to 6 m of the deposit is dessicated and over-consolidated with SPT-N values ranging from 10 to 26 indicating stiff to very stiff consistency. Beneath, the dessicated crust, the SPT-N values decrease with depth from 19 to 2, and indicate stiff to soft consistency. Vane shear tests carried out in softer areas indicate undrained shear strength values of 45 to 50 kPa, and sensitivity values of 2.3.

The consolidation test results from BON-7 are summarized in the following table. Undisturbed samples from boreholes BON-1 and BON-6 were also inspected to select appropriate samples for consolidation testing, but were found to contain numerous interbedded sand layers making them unsuitable for consolidation testing.

**TABLE 3: CONSOLIDATION TEST SUMMARY**

Borehole	Sample depth (m)	Insitu $\sigma'$ (kPa)	w (%)	$e_o$	Pc' (kPa)	OCR	Cc	Cr	LL	PL
BON-7	12.5	155	44	1.287	130	0.84	0.37	0.06	42	24

Where

$\sigma'$	insitu overburden pressure
w	moisture content
$e_o$	initial void ratio
Pc'	Preconsolidation pressure
OCR	Overconsolidation ratio
Cc	Compression Index
Cr	Recompression Index
LL	Liquid Limit
PL	Plastic Limit

The laboratory tests indicate that the upper 5 m of the Marine Clay is highly over-consolidated, and below 5 m depth the clay is generally normally consolidated. This is consistent with the measured moisture contents of samples below the over-consolidated crust being near or slightly in excess of the Liquid Limit index, and the decreasing SPT-N values encountered at greater depths within the deposit.

#### 5.4 Glaciofluvial Sand to Silty Sand

A glaciofluvial sand to silty sand deposit was encountered underlying the aforementioned clayey silt to silty clay deposit in each of the boreholes except BON-6 and BON-7. The depth to the lower boundary of this unit varies as shown below



**TABLE 4: LOWER BOUNDARY OF GLACIOFLUVIAL SAND**

BOREHOLE	DEPTH BELOW	ELEVATION
	GROUND SURFACE	(m)
BON-1	27.1 m	75.3
BON-2	27.4 m	59.8
BON-3	18.4 m	67.7
BON-4	17.9 m	71.8

Based on the samples recovered in the SPT samples, this deposit is horizontally stratified, with clay layers of 25 to 50 mm in thickness noted at BON-2. The composition of this unit varies from sand with trace to some silt, to silty sand or to sandy silt (non-plastic). The fines content (portion passing 76  $\mu\text{m}$ ) of the sand and silty sand samples was found to vary from 5 to 20 % by weight. The samples of the sandy silt layers contained up to 80% fines. The deposit typically contains trace to some gravel sizes and occasional cobbles and boulders.

The moisture content of the recovered samples varied from 2% to 38% by weight. The SPT N-values were highly variable ranging from 2 to greater than 100 where gravel or cobbles were encountered. The N-values generally increase with depth except near the east margin of the deposit (BON-4) where more prevalent gravel and cobbles sizes resulted in N-values from 82 to greater than 100 blows per 0.3 m throughout most of the profile. At boreholes BON-2 and BON-3, the upper 5 m of the deposit was loose to compact with N-values ranging from 2 to 12. Below EL 74 m in BON-2 and EL 79 m in BON-3, the N-values generally range from 20 to 56, with several SPT N-values exceeding 100. Lower N-values were encountered near the lower boundary of the unit in boreholes BON-1 and BON-3, where N-values of 8 and 9, respectively, were recorded.

### **5.5 Coarse Glaciofluvial Granular Deposit**

A coarse glaciofluvial deposit generally comprised of sand and gravel with cobbles and boulders was encountered beneath the above sand unit in each of the boreholes except BON-7. The coarse granular deposits vary in texture from a mixture of cobbles and boulders to gravelly sand. A remnant of silty sand till was encountered in this unit at BON-4. The largest boulders encountered in the boreholes were comprised of marble and had a maximum dimension measured along the core axis of 400 mm to 1400 mm.

The SPT N-values encountered in this unit varied from 9 in the silty sand till to more than 50 blows for 50 mm indicating very dense conditions.

Boreholes BON-1 and BON-6 met refusal conditions in the sand and gravel unit. While in boreholes BON-2 through BON-4 which penetrated the unit, bedrock was found to underlying the granular deposits.

## 5.6 Bedrock

The soils described above were found to be underlain by Pre-Cambrian marble bedrock. The bedrock contact was delineated by coring in Boreholes BON-2, BON-3 and BON-4. The rock is described as fresh to slightly weathered, grey, medium to coarse-grained crystalline marble with light and dark banding.

The top of bedrock elevations was in reasonable agreement with that encountered in the MTO investigation for the existing Highway 17 bridge. The elevations of the bedrock encountered during the recent investigation are summarized below:

**TABLE 5: BEDROCK ELEVATION**

Foundation Element	Borehole	Depth to Bedrock (m)	Elevation
			Top of Bedrock (m)
Pier 1	BON-2	31.3	55.9
Pier 2	BON-3	22.3	63.8
Pier 3	BON-4	17.9	71.8

Core recovery in the bedrock was generally 68 to 100%. The RQD values ranged from 68% to 100% and had an average value of 83%, indicating fair to excellent quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, varied from 0 to greater than 5, with a mean value of 2 indicating moderate joint spacing. The condition of the joints was planar and rough.

The unconfined compressive strength of the intact rock, based on point load tests, varies from 79 to 174 MPa and has an average value of 125 MPa , indicating very strong rock.

## 5.7 Water Levels

Observations of groundwater levels and soil moisture conditions during drilling indicate that the water-table was present at about 3 to 4 m depth below ground surface in boreholes drilled within the valley (BON-2 through BON-4). In boreholes BON-1 and BON-7 drilled on the upper portion of the north valley slope, the conditions encountered indicate a perched water-table within the silty clay at about 9 m depth. The upper portion of the sand beneath the silty clay was not saturated at this location.

The piezometric levels recorded at various times are summarized in Table 6 below. The groundwater levels are expected to vary seasonally and with extended precipitation by several metres.

**TABLE 6: PIEZOMETRIC LEVELS**

	Screen Completed in	Depth below ground surface (m)		
		Oct 22/03	Dec 19/03	Feb 4/04
BON-1	Sand	18.59	18.15	17.95
BON-2	Bedrock	3.47	3.42	2.72
BON-3	Bedrock		destroyed	destroyed
BON-4	cobbles & boulders			5.35
BON-6	Sand		18.37	18.28
BON-7	Clay			destroyed

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**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**6 INTRODUCTION**

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

The proposed structure will be located approximately 40 m southwest and parallel to the existing Highway 17 bridge which intersects the Bonnechere river at a skew angle near 60 degrees. The proposed bridge will support a 2-lane roadway. The structural planning report dated December 2003 indicates the preferred option is a 286 m long bridge incorporating 4-spans constructed using steel plate girders.

The location proposed for the northwest abutment is atop the approximately 20 m high north valley slope. The proposed grade requires the south east abutment to be located at mid-slope on the 30 m high southern valley slope. The General Arrangement (GA) for the proposed structure shows a proposed 9 m high fill on the northwest approach and a 5 to 6 m deep cut at the southeast approach.

The discussion and preliminary 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 and from the investigation and construction of the existing bridge.

**7 STRUCTURE FOUNDATIONS**

The proposed structural plan is for a 4-span structure incorporating 5 foundation elements, including 2 abutments and 3 support piers.

The subsurface conditions at the proposed structure generally consist of a sequence of silty clay overlying compact to dense sand to sandy silt, underlain by dense layer of sand and gravel with cobbles and boulders or by silty sand till. This sequence rests on a bedrock surface that slopes

downward from southeast to northwest along the alignment. The elevations at which bedrock was encountered in the boreholes at three foundation elements for the piers are as follows:

**TABLE 7: BEDROCK ELEVATION**

Location	BH Number	Elevations	
		Original Ground	Top of Bedrock
Pier 1	BON-2	87.2	55.9
Pier 2	BON-3	86.1	63.8
Pier 3	BON-4	89.7	71.8

Bedrock was not encountered at the proposed west and east abutment locations since the borehole at the west abutment, BON-1, met the criteria for refusal in a deposit of sand with gravel and cobbles and borehole BON-5 at the proposed east abutment was located on a steep slope that was not accessible to the drill rig was therefore not drilled.

The construction records indicate that the existing Highway 17 bridge is founded on steel H-piles driven to bedrock at elevations varying from 52 m to 70 m.

### **7.1 Foundation Alternatives**

This section discusses the feasible foundation alternatives, provides geotechnical design parameters and recommends a preferred foundation scheme.

Shallow foundations are not considered feasible at the abutments because of potential valley slope movements and settlement associated with soft to firm clay. At the pier locations, loose to compact, saturated, sand to sandy silt was encountered extending to 79m depth at Pier 1 and 75 m depth at Pier 2. These loose saturated sediments do not offer suitable support for the foundation. Initial consideration was therefore given to the following foundation types:

- Driven H-piles
- Caissons (drilled shaft piles)

A comparison of the foundation alternatives based on advantages and disadvantages of each is included in the table below:

**TABLE 8**  
**COMPARISON OF FOUNDATION ALTERNATIVES**  
**FOR EACH FOUNDATION ELEMENT**

Foundation Element	Driven H-Piles	Caisson
<b>West Abutment</b>	<p><b>Advantages:</b> Commonly used deep foundation method, with relatively economic mobilization cost</p> <p><b>Disadvantages:</b> Large depth (estimated &gt;40 m) to reach bedrock and suitable end-bearing conditions Presence of boulders overlying bedrock may result in damage to pile tips</p>	<p><b>Advantages:</b> Can achieve higher capacity than driven piles installed at similar depths</p> <p><b>Disadvantages:</b> i. Poor caisson hole stability in the saturated silty sand encountered beneath the clay will require stabilization using drilling mud or casing and substantially increased costs ii. Costly equipment mobilization</p>
<b>Pier 1</b> <b>Pier 2</b> <b>Pier 3</b>	<p><b>Advantages:</b> Suitable end-bearing can be obtained on bedrock within 18 to 32 m depth below existing surface grade</p> <p><b>Disadvantages:</b> i. Presence of boulders overlying bedrock may result in damage to pile tips</p>	<p><b>Advantages:</b> N/A</p> <p><b>Disadvantages:</b> i. The presence of saturated sand effectively precludes the use of this method at the pier locations</p>
<b>East Abutment</b>	<p><b>Advantages:</b> Commonly used deep foundation method, with relatively economic mobilization cost Suitable end-bearing can be obtained on bedrock expected to be within 30 m depth based on existing bridge pile records</p> <p><b>Disadvantages:</b> Presence of boulders overlying bedrock may result in damage to piles</p>	<p><b>Advantages:</b> N/A</p> <p><b>Disadvantages:</b> i. Relatively expensive equipment mobilization Potential additional costs and difficulty stabilizing the caisson hole in non-cohesive seepage zones</p>

Based on the disadvantages and advantages summarized in the table above, the recommended foundation type at this site is steel H-piles driven to bedrock. Drilled Caissons are not considered a feasible option for the site conditions encountered.

## 7.2 Driven Steel Piles

Driven steel H-piles are considered a feasible option for foundations of the structure. The piles should be driven to bedrock. The expected elevation of bedrock at each of Pier #1, #2 and #3 foundation elements is shown in Table 7 in the preceding section. The

geotechnical report for the existing bridge indicated refusal conditions could be achieved in the granular deposits. However, the available pile driving records for the existing bridge, indicate that the piles were extended to bedrock. It is therefore recommended that the piles for the proposed bridge be extended to bedrockfs.

The top of bedrock was not defined at the proposed west and east abutments during the preliminary field investigation. However, the bedrock profile derived from the 1972 borehole investigation and pile driving records for the existing bridge, offset 40 m northeast of the proposed new alignment, is in reasonable agreement with the conditions encountered along the new alignment, and may be used for preliminary design purposes. Additional drilling will be required during detailed design to establish the bedrock elevation at the final abutment locations and any new pier locations.

The estimated top of bedrock elevation at the various foundation elements are summarized in the following table.

**TABLE 9: BEDROCK ELEVATION FOR PRELIMINARY DESIGN**

Location	BH Number	Elevations	
		Original Ground	Top of Bedrock
Northwest Abutment		102.4	52 (estimated)
Pier 1	BON-2	87.2	55.9
Pier 2	BON-3	86.1	63.8
Pier 3	BON-4	89.7	71.8
Southeast abutment		98- 102 (slope)	68 (estimated)

#### 7.2.1 Geotechnical Resistance of Driven Piles

The geotechnical resistance of steel H-piles driven to bedrock at the site is greater than the axial structural resistance recommended by MTO-Structural Office in their April 15, 1998 memo. It is therefore recommended that the piles be designed based on their structural capacity for the following vertical, factored compressive loads at Ultimate Limit State (ULS) as summarized below:

HP 310 x110                      2000 kN

HP 310 x 132                    2400 kN

The serviceability limit state (SLS) for end-bearing piles on bedrock will not govern.

The group resistance should be in accordance with the recommendations in the Canadian Highway Bridge Design Code(CHBDC), Clause No.6.8.6.2.

The minimum recommended pile spacing, measured centre to centre at the underside of the pile cap, should be 2.5b (where b is the pile width or diameter) or 750 mm, whichever is greater.

To minimize pile loads related to negative skin friction, the piles at the bridge abutments should be driven after the approach fills have been constructed and the associated settlements have occurred. Approximately 90% of the primary settlements are expected to occur within 200 days of completion of the approach fill. If an increased rate of consolidation is required wick drains may be required. Design of these measures should be carried out during detailed design if required.

### 7.2.2 Pile Points

It is recommended that all piles be fitted with Titus Steel Co. "H" bearing pile points or equivalent, to protect the web and flanges from boulders that may be encountered overlying the bedrock.

### 7.2.3 Lateral Pile Resistance

For design of conventional pile groups at the piers, it is recommended that the unbalanced horizontal forces be resisted by battered piles.

Lateral soil resistance against 310 mm wide piles can be calculated based on the CHBDC method. The coefficient of subgrade reaction,  $k_s$ , and the ultimate lateral resistance is given by the following expressions:

#### Silty Clay :

$$k_s = 120 \cdot C_u/D \text{ (kPa/m)}$$

$$P_{ult} = 9 \cdot C_u \text{ (kPa)}$$

#### Cohesion-less deposits:

$$k_s = n_h \cdot z/D \text{ (kPa/m)}$$

$$P_{ult} = 3 \cdot \gamma \cdot z \cdot K_p \text{ (kPa)}$$

where	$C_u$	= average undrained shear strength of clay = 40 kPa at piers = 100 kPa at abutments
	$z$	= depth below pile cap or abutment in metres
	$D$	= pile diameter or width in metres



$$\begin{aligned}
 n_h &= \text{soil density coefficient} \\
 &= 4400 \text{ kPa/m} \quad \text{below water-table} \\
 &= 6600 \text{ kPa/m} \quad \text{above water-table} \\
 \gamma &= \text{Average effective unit weight of sand,} \\
 &= 11 \text{ kN/m}^3 \quad \text{below water-table} \\
 &= 21 \text{ kN/m}^3 \quad \text{above water-table} \\
 K_p &= 3.3 \quad (\text{passive earth pressure coefficient})
 \end{aligned}$$

The above equations and the recommended parameters may be used for numerical analysis of the interaction between a pile and the surrounding soil. The lateral pressure obtained from the numerical analysis should not exceed the ultimate lateral resistance,  $P_{ult}$ .

The spring constant,  $K$ , for analysis may be obtained by the expression,  $K = k_s L D$  (kN/m), where  $k_s$  is the coefficient of horizontal subgrade reaction, (kPa/m),  $D$  is the pile width (m), and  $L$  is the length of pile segment used in the analysis.

To account for reduced resistance resulting from group effects, the lateral reaction values, obtained from the values above, should be modified using the multipliers shown below. The reduction factors will vary with the pile spacing expressed as a multiple of the pile breadth, 'b'.

For a pile group oriented *perpendicular* to the direction of loading:

Pile Spacing	Horizontal Subgrade Reaction Reduction Factor
4b	1.00
3b	0.8

For pile groups orientated *parallel* to the direction of loading the following reduction factors are recommended:

Pile Spacing	Horizontal Subgrade Reaction Reduction Factor
8b	1.00
6b	0.7
4b	0.4
3b	0.25

### **7.3 Pile Caps**

The base of the pile caps should be located below the depth of scour anticipated during the design life. Scour protection is recommended for the pile caps at Pier 1 and Pier 2. The type and extent of the scour protection will be dictated by the hydrologic parameters. Assessment of hydrologic parameters such as, scour rate, rip-rap gradation, river flow rates and elevations and ice forces should be carried out by a qualified professional experienced in this field.

### **7.4 Frost Cover**

Pile caps and footings should be provided with frost protection. This may take the form of a minimum 1.9 m of earth cover from any exposed surface grade.

### **7.5 Integral Abutment Considerations**

The recommended foundation type is steel piles driven to bedrock. The proposed foundation type and the ground conditions at this site are considered suitable for integral abutment design.

To provide the required flexibility in the piles, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP filled with sand in accordance with standard integral abutment design procedures.

## **8 EXCAVATION AND BACKFILL**

### **8.1 Temporary Excavations and Dewatering**

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native clay at this site is classed as Type 2 soil above the water table and Type 3 soil if below the water table.

Temporary excavations for the pile caps adjacent to the river will encounter saturated cohesionless soils overlain by cohesive soils, and will require dewatering measures and construction of coffer dams to maintain excavation stability. The coffer dam will have to be extended a suitable depth below the base of the excavation to maintain basal stability.

### **8.2 Permanent Cutslopes**

For preliminary design purposes, the proposed permanent cut-slopes on the east approach can be inclined at 2H:1V. Adequate drainage and erosion control measures will be required to maintain cut slope stability and reduce maintenance costs. As a minimum, the

drainage measures should incorporate conventional ditching at the toe of the slope. Vegetative covering is recommended for all permanent cut slopes in accordance with SP 572SO1. If continual seepage or surficial instability are evident, remedial measures such as toe drains or gravel sheeting may be required.

## **9 APPROACH EMBANKMENTS**

An approach embankment will be required for the northwest approach to the structure and will be founded on 10 to 12 m thick deposit of firm to very stiff to firm silty clay overlying generally dense glaciofluvial sand unit. The approach fills at this site are not expected to exceed 9 m in height. The General Arrangement for the bridge shows a 2H:1V headslope for the approach fill located on the moderately-steep valley slope consisting of an approximately 5 to 12m thick layer of silty clay overlying loose to very dense sand.

The steeper portions of the natural valley slopes at the proposed bridge site are inclined at 23 to 27 degrees from the horizontal and range from 17 m to 25 m in height. The Bonnechere river surface is at an approximate elevation of 83.5 m and the top of the valley slope is at about 115 m elevation. The northwest valley slope has bench-like morphology with a 20 m wide subhorizontal bench located at about 103 m elevation. The southeast valley slope near the proposed abutment is a relatively uniform slope with the toe of the slope set-back about 60 m from the river on gently sloping terrain. Bench-like slope morphology is often associated with pre-existing slope instability.

An assessment of the stability and estimated settlements associated with the proposed approach fill is provided in the following sections.

### **9.1 Embankment Stability**

The proposed grade at the southeast abutment will result in a cut of approximately 8 m near the top of the valley slope, and placement of about 5 m of fill downslope of the proposed abutment inclined at 2H:1V. These operations will generally improve the stability of the southeast valley slope.

The proposed approach fill at the northwest abutment will require placement of about 9 m of fill near the top of the slope. Retaining structures would be required to prevent the toe of the slope from extending into the river.

A preliminary effective stress stability analysis was carried out for the northwest valley slope using GSlope software (Bishop's Modified Method of slides). The stability analysis was carried out using the soil model developed from the borehole data and using a slope profile located approximately 20 m west of the west bridge abutment. The slope geometry,

shear strength parameters and hydraulic head levels are shown on Figure C-1 in Appendix C.

A parametric analysis was carried out by varying the elevations of the perched groundwater table within the lower portion of the silty clay unit. The elevation of perched groundwater was varied from the levels recorded in the underlying strata (hydrostatic condition at about EL 84 m) and those levels encountered during drilling of the boreholes (perched conditions at about EL 93 m). Average shear strength parameters were selected based on the index properties and the SPT N-values measured during the field investigation. The results of the analysis are summarized in the table below:

**TABLE 10: RESULTS OF STABILITY ANALYSIS – NORTHWEST ABUTMENT**

Geometry	Piezometric levels	FS
Existing Natural slope at northwest abutment location	Water-table EL 84m	1.3
	Perched at EL 93 m	1.2
Fill sloping at 2H:1V extending into river	Perched at EL100m	1.2
Fill sloping at 1.7H:1V extending just to river	Perched at EL 93 m	1.0
Retaining wall constructed at top of slope	Perched at EL 93 m	1.0

The stability analysis indicates that the Factor of Safety is about 1.2 for global stability under the inferred piezometric and shear strength conditions. This low factor of safety is generally consistent with previous observations of slope instability of the valley slopes noted in the Foundation Investigation Report, WP 6-67-02 for the original structure prepared by the Department of Transportation and Communications dated 1972. The earlier design information indicates that the original slope was cut-back to create an acceptable factor of safety for global stability.

The proposed placement of fill on the northwest valley slope or construction of a retaining wall at the top of the slope would decrease the global stability to unsafe levels. The advantages and disadvantages associated with several design options are provided in Table 11 included following the text of the report. Based on this analysis and the current subsurface information the recommended option is to move the northwest abutment back from the steep valley slope, add a bridge span and avoid placement of an approach fill on the valley slope.

Additional drilling, piezometer installation and slope stability analysis should be carried out during detailed design to select the most appropriate option at the northwest abutment. These detailed design studies are beyond the scope of this preliminary investigation.

## 9.2 Embankment Settlement

The estimated settlements associated with construction of the proposed northwest approach embankment were calculated at three locations Sta. 19+075, 20+040 and 20+075. The estimated settlements only apply in the event that the proposed fill can be constructed in a stable configuration as described in the preceding section.

The settlement analysis considered changes in stress beneath the centre of an up to 10 m high embankment with a maximum width of 56 m and sideslopes inclined at 2H:1V. The soil conditions were modelled based on the thickness of clay at each analysis section and included the variation in over-consolidation ratio and compressibility with depth.

The foundation settlement values for the embankment centreline are included in the table below. Long term secondary consolidation in this over-consolidated deposit is calculated to be less than 70 mm in 50 years.

**TABLE 12: ESTIMATED SETTLEMENT**

Location	Approximate Embankment Height (m)	Recompression Settlement (mm)	Primary Settlement (mm)	Total Settlement (mm)
19+075	1	30	120	150
20+040	10	180	50	230
20+075	9	190	100	290

The rate of settlement was estimated based on two-way drainage and the generation of excess pore pressures only in the saturated portion of the deposit. The recompression settlement will occur primarily in the over-consolidated crust and is expected to be over relatively quickly during construction. The estimated rate of settlement for the primary settlement component will see about 90% of the settlement completed by 200 days after construction.

It is recommended that the west approach embankment be constructed prior to bridge construction to preload the foundation soils and reduce the magnitude of primary settlement. The preload should remain in place for at least 200 day, prior to placement of the pavement. If an increased rate of consolidation is required wick drains may be required. Design of these measures if required should be carried out during detailed design if required.

As indicated above, the majority of the foundation settlement is recompression settlement and is expected to occur during construction of the approach embankment. Settlement of

the embankment itself will occur in addition to the foundation settlement, and can be estimated using the values provided below:

Cohesive earthfill (silt and clay)	1%
Granular fill (sand and gravel)	0.5%
Rockfill (excluding shale)	0%

Further settlement analysis should be carried out during detailed design if an approach embankment is included in the design.

## 10 BACKFILL TO ABUTMENTS

In the case of integral or semi-integral abutments, backfill to the abutment should be granular material. In cases where the approach embankment consists of rock fill, the backfill to the abutment wall should consist of OPSS Granular "B" Type II.

In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. If rock backfill is specified, a NSSP is required to set the grading limits for the rock fill. The NSSP should require rock fill used as backfill be limited to fragments no greater than 300 mm and including adequate spalls to fill voids in the rock fill.

In all cases where the approach embankment consists of rock fill and the abutment wall is backfilled with granular material, the granular backfill should consist of OPSS Granular "B" Type II.

The backfill to the abutment walls should be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill should be placed to the extents shown in OPSD 3501.000, and rock backfill should be placed to the extents shown in OPSD 3505.000.

The design of the abutment should incorporate a subdrain as shown in OPSD 3501.000 or OPSD 3505.000, as applicable.

## 11 EARTH PRESSURE

For cases where backfill to the abutment is placed in accordance with OPSD 3501.000 or OPSD 3505.000, as recommended, the lateral earth pressure will be governed by the properties of the material within the backfill limits shown in the respective OPSD, i.e. a line projected up at 1.5H:1V for granular backfill and 1.25H:1V for rock backfill.

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), at-rest horizontal earth pressures should be used. In the case of

an integral or semi-integral abutment design, the pressure on the ballast wall may be interpreted using Figure C6.9.1(a) in the Commentary to the CHBDC.

Earth pressures acting on the structure should be computed in accordance with Clause 6.9 of the CHBDC but generally are given by the expression:

$$P_h = K(\gamma h + q)$$

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient (see below)

$\gamma$  = unit weight of retained soil (typically 21 kN/m<sup>3</sup>)

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or at a depth of 1.7 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in the following table.

**TABLE 13: LATERAL EARTH PRESSURE COEFFICIENTS**

Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II  $\phi = 35^\circ$ $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I  $\phi = 32^\circ$ $\gamma = 21.2 \text{ kN/m}^3$		Rock Fill  $\phi = 42^\circ$ $\gamma = 19.0 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.50*	0.2	.30*
At rest (Restrained Wall)	0.43	-	0.47	-	.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.2	-	5.0	-

\* For wing walls.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral or semi-integral abutments, material with a lower passive pressure coefficient (e.g. Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass. However, the use of Granular "B" Type I may be restricted if the approach embankment consist of rock fill.

The factors in the table above are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the Canadian Highway Bridge Design Code.

## 12 SEISMIC CONSIDERATIONS

The zoning set by the Geological Survey of Canada for the site is summarized below::

▪ Acceleration zone	$Z_a = 4$
▪ Zonal acceleration	0.20 g
▪ Velocity zone	$Z_v = 2,$
▪ Zonal Velocity	0.10 m/s

A site-specific peak horizontal ground acceleration factor  $k_h = 0.184g$  obtained from Geologic Survey of Canada has been used in the seismic analysis. The site profile can be classified as Type 1 based on the CHBDC criteria. A site response factor of 1.0 is applicable to this soil profile.

The granular deposits at depth are partially above the water-table and are in a compact to very dense state. These deposits are not considered susceptible to liquefaction.

For the design of retaining walls, a value of  $k_h$  was set at 67% of the peak value. The vertical acceleration factor  $k_v$  was set as  $0.6 \times k_h$ . The dynamic active pressures generated during seismic events are impulses that act only for a short time. Lower earth pressures can be utilized if the allowable design displacement is increased

The lateral earth pressure coefficients under seismic loading are summarized in the following table:



**TABLE 14: SEISMIC LATERAL EARTH PRESSURE COEFFICIENTS**

Condition	Height of Application From Base as Percentage of Wall Height	Earth Pressure Coefficient ( $K_E$ ) for Earthquake Loading					
		OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \delta = 17.5^\circ$		OPSS Granular B Type I $\phi = 32^\circ, \delta = 16^\circ$		Rock Fill $\phi = 42^\circ$	
		Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active ( $K_{AE}$ )*	40%	0.33	0.70	0.37	0.90	0.26	0.40
Passive ( $K_{PE}$ )*	33%	3.5	-	3.0	-	4.8	-
At Rest ( $K_{OE}$ )**	45%	0.67	-	0.72	-	0.58	-

\* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

\*\* After Woods

### 13 CONSTRUCTION CONCERNS

Additional geotechnical investigation will be required during detailed design to assess several design and construction related issues. These include:

- Global stability will be adversely affected by construction of an approach fill on the northwest valley slope. The design will require changes, such as an additional bridge span or earth retention system, during detailed design phase to address the global stability.
- Potential settlements under the northwest approach fill should be assessed further during the detailed design to arrive at an appropriated design for accommodating the settlements within the required schedule. The piles for the northwest abutment fill should not be driven until the settlements under the approach fill are essentially complete.
- Additional boreholes should be completed to establish the elevation of the bedrock and at the east and west abutments and each pier location. The feasibility of driving the piles through the dense sand and gravel to bedrock should be assessed during the investigation.

Engineering analysis and report preparation by:

S.M. Sather, P.Eng.,  
Senior Geotechnical Engineer



Report reviewed by:  
P.K. Chatterji, P.Eng.,  
Review Engineer



**TABLE 11: DESIGN OPTIONS FOR IMPROVING STABILITY OF WEST VALLEY SLOPE**

<b>Design Option</b>	<b>Advantages</b>	<b>Disadvantages</b>	<b>Hazard</b>
Move abutment west and add bridge span	<ul style="list-style-type: none"> <li>▪ reduces embankment loading on marginally stable slope adjacent to river</li> <li>▪ increases setback of abutment from edge of river</li> </ul>	<ul style="list-style-type: none"> <li>▪ Relatively high cost option</li> <li>▪ Increased bridge length</li> </ul>	<ul style="list-style-type: none"> <li>▪ slope movements have a low likelihood but could potentially impact structure</li> </ul>
Construct retention system to support proposed fill	<ul style="list-style-type: none"> <li>▪ can maintain west abutment at existing location</li> <li>▪ reduced bridge length</li> </ul>	<ul style="list-style-type: none"> <li>▪ large scale of instability would require high capacity support system</li> <li>▪ relatively low strength soil above 20 m depth to provide lateral resistance</li> <li>▪ lightweight fill may be required to reduce the lateral loading and support requirements</li> </ul>	<ul style="list-style-type: none"> <li>▪ Aging of retention system may eventually result in loss of stability</li> </ul>
Construct approach embankment of lightweight fill	<ul style="list-style-type: none"> <li>▪ can maintain west abutment at existing location</li> <li>▪ reduced bridge length</li> </ul>	<ul style="list-style-type: none"> <li>▪ will not improve the existing factor of safety unless some of the existing slope is removed and replaced with lightweight fill</li> <li>▪ ultra lightweight fill may be required to control loading</li> <li>▪ high cost of lightweight fill</li> </ul>	<ul style="list-style-type: none"> <li>▪ probability of instability of the existing natural slope would remain unchanged and is relatively high</li> </ul>
Construct toe berm	<ul style="list-style-type: none"> <li>▪ will improve global stability slightly (&lt;10%)</li> </ul>	<ul style="list-style-type: none"> <li>▪ the berm may require soil reinforcing to prevent local slope instability adjacent to river</li> <li>▪ size of the berm and the improvement in factor of safety is limited by space constraints</li> </ul>	<ul style="list-style-type: none"> <li>▪ likelihood of instability of the existing natural slope would be reduced slightly but would still be higher than recommended</li> <li>▪ increased likelihood of instability of the lower slope adjacent to the river</li> </ul>

## **Appendix A**

### **Record of Borehole Sheets**

# RECORD OF BOREHOLE No BON-1

1 OF 4

METRIC

G.W.P. 647-92-00 LOCATION N 5 038 578.0, E 292 351.3 ( Bonnechere River ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NW Casing, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 16.10.03 - 20.10.03 CHECKED BY SMS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
102.4	TOPSOIL: (25 mm)												
102.0	Silty CLAY, trace rootlets Stiff to Very Stiff Brown Moist (Cl)		1	SS	11								
			2	SS	16								
			3	SS	19								
			4	SS	22								
			5	SS	22								
	with sand seams, trace gravel		6	SS	19								
			7	SS	14								
			8	SS	11								
	silty sand layers, trace gravel		9	SS	12								
	with thin silty sand seams, brown to grey, moist to wet												

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+ 3, x 3: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

## METRIC

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	w <sub>p</sub>	w		
							20 40 60 80 100					
90.5	Firm, grey, wet		10	SS	4							0 6 64 30
11.9	Silty SAND to Sandy SILT Loose to Dense Grey Wet		1	TW			2.27 +					
			11	SS	45							
			12	SS	8							0 20 63 17

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ONTMT4 7450BON.GPJ 11/05/04

## METRIC

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RECORD OF BOREHOLE No BON-1

4 OF 4

METRIC

G.W.P. 647-92-00 LOCATION N 5 038 578.0, E 292 351.3 ( Bonnechere River ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NW Casing, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 16.10.03 - 20.10.03 CHECKED BY SMS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
71.8			16	SS	507		72										
30.5	END OF BOREHOLE AT 30.53m. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.  WATER LEVEL READINGS: DATE DEPTH (m) 22/10/03 18.59 19/12/03 18.15 04/02/04 17.95				.050												

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# RECORD OF BOREHOLE No BON-2

2 OF 4

METRIC

G.W.P. 647-92-00 LOCATION N 5 038 548.1, E 292 414.8 ( Bonnechere River ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, HQ Casing, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 14.10.03 - 16.10.03 CHECKED BY SMS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT Y kN/m <sup>3</sup>	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		
74.7	with clay layers to 50 mm thick		10	SS	10		77					
12.5	SAND, medium to coarse grained, trace gravel, occasional cobbles Compact to Very Dense Grey Wet clay layers to 25 mm thick		11	SS	20		76					
			12	SS	39		75					
			13	SS	81		74					
			14	SS	51		73					
			15	SS	50/ .127		72					
			16	SS	27		71					
							70					
							69					
							68					

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Sensitivity

20  
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10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BON-2

3 OF 4

METRIC

G.W.P. 647-92-00 LOCATION N 5 038 548.1, E 292 414.8 ( Bonnechere River ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, HQ Casing, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 14.10.03 - 16.10.03 CHECKED BY SMS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	boulders from 20.6m to 21.34m													start coring from 20.6m to 21.3m
			17	SS	37									2 93 5 (SI+CL)
59.8			18	SS	50									
27.4	SAND, coarse grained, trace gravel, occasional cobbles and boulders				102									continuous coring
	boulders and cobbles from 28.22m to 28.55m													
	400 mm boulder from 29.9m to 30.33m.													

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Sensitivity 20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BON-2

4 OF 4

METRIC

G.W.P. 647-92-00 LOCATION N 5 038 548.1, E 292 414.8 ( Bonnechere River ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, HQ Casing, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 14.10.03 - 16.10.03 CHECKED BY SMS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	
55.9					FI		57					
31.3	<b>MARBLE, BEDROCK</b> Slightly to moderately weathered, very thinly to thinly bedded, grey with dark grey and white subhorizontal banding, very strong Subvertical joints from 31.85m to 31.9m, from 31.93m to 32.13m Multiple rock fragments in gravel and cobble sizes with clay coating from 30.63m to 31.24m Multiple joints from 32.79m to 32.89m, 34.26m to 34.37m		1	RUN	1		56					RUN 1# TCR=100%, SCR=100%, RQD=87%, UCS=139.0MPa RUN 2# TCR=100%, SCR=100%, RQD=83%, UCS=110.4MPa
			2		2		55					
			2	RUN	1		54					
			4		4							
			1		1							
			3	RUN	0							RUN 3# TCR=100%, SCR=100%, RQD=92%, UCS=135.9MPa
			2		2							
			0		0		53					
52.8												
34.4	END OF BOREHOLE AT 34.37m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.83m slotted screen. WATER LEVEL READINGS: DATE DEPTH(m) 22/11/03 3.47 19/12/03 3.42 04/02/04 2.72											

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Sensitivity 20  
15 5  
10 (%) STRAIN AT FAILURE

## METRIC

ORIGINATED BY SL

COMPILED BY SS

CHECKED BY                      SMS

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# RECORD OF BOREHOLE No BON-3

2 OF 3

METRIC

G.W.P. 647-92-00 LOCATION N 5 038 497.2, E 292 509.5 ( Bonnechere River ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NW Casing, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 03.11.03 - 03.11.03 CHECKED BY SMS

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 <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	20 40 60			
							76							
			10	SS	50/ .125		75							
			11	SS	20		74							
							73							
							72							
			12	SS	28		71							
							70							
							69							
							68							
67.7							67							
18.4	Silty SAND, trace gravel, occasional cobbles, and limestone fragments Loose Grey Wet (TILL) (ML- nonplastic)		13	SS	9									10 58 23 8

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Sensitivity 20  
15 5  
10 (%) STRAIN AT FAILURE

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## METRIC

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+ 3, x 3: Numbers refer to Sensitivity



# RECORD OF BOREHOLE No BON-4

2 OF 3

METRIC

G.W.P. 647-92-00 LOCATION N 5 038 463.3, E 292 564.5 ( Bonnechere River ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 30.10.03 - 31.10.03 CHECKED BY SMS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	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	20 40 60 80 100	20 40 60					
10.0	Gravelly SAND with cobbles, occasional boulders Dense to Very Dense Grey Wet Grey		10	SS	50/ .076									
77.4			11	SS	50/ .127									
12.3	COBBLES and BOULDERS, some sand and gravel Very Dense Grey Wet  large boulder from 15.0m to 16.4m												continuous coring	
			1	RUN										
			2	RUN									RUN 2# TCR=83%, SCR=83%, RQD=45%, UCS=127.5MPa	
			3	RUN									RUN 3# TCR=93%, SCR=68%, RQD=63%, UCS=119.6MPa	
			4	RUN									RUN 4# TCR=45%, SCR=23%, RQD=0%	
71.8														
17.9	MARBLE, BEDROCK Moderately weathered, thinly bedded, grey with white subhorizontal banding, very strong		5	RUN									RUN 5# TCR=100%, SCR=100%, RQD=100%, UCS=125.4MPa	
71.2														
18.5	Subvertical joints from 15.72m to 15.9m, 18.06m to 18.16m, 18.21m to 18.29m Vertical joint from 17.91m to 18.16m Multiple joints, broken core and gravel pieces from 14.94m to 15.14m, 16.03m to 16.38m, 16.38m to 17.91m END OF BOREHOLE AT 18.47m. Piezometer installation consists of													

Continued Next Page

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

ONTMT4 7450BON GPJ 11/05/04

# RECORD OF BOREHOLE No BON-4

3 OF 3

METRIC

G.W.P. 647-92-00 LOCATION N 5 038 463.3, E 292 564.5 ( Bonnechere River ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 30.10.03 - 31.10.03 CHECKED BY SMS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH(m) 04/02/04 5.35													

ONTMT4 7450BON GPJ 11/05/04

+ 3 . X 3 : Numbers refer to  
Sensitivity

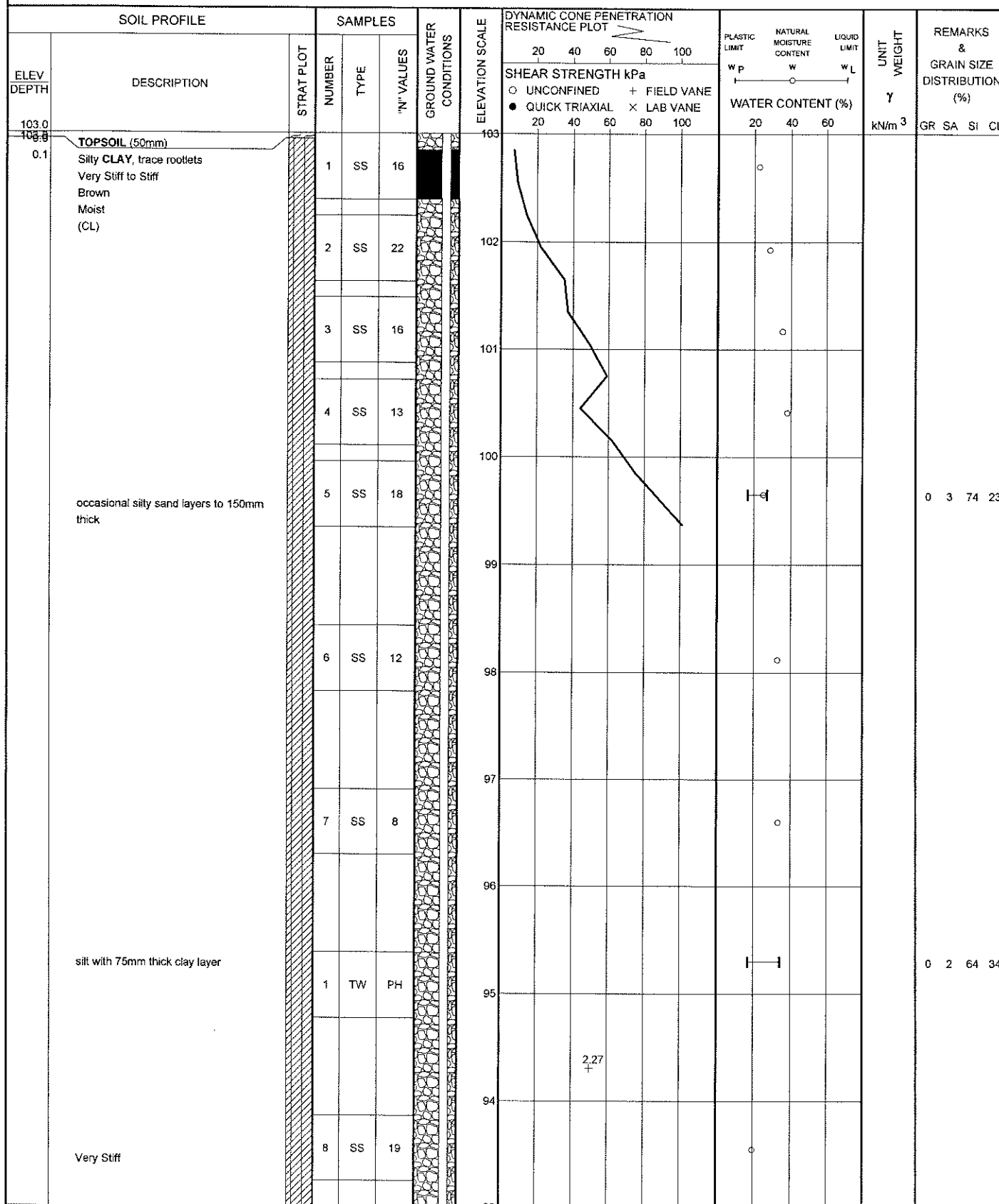
20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BON-6

1 OF 3

METRIC

G.W.P. 647-92-00 LOCATION N 5 038 595.5, E 292 317.1 ( Bonnechere River ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NW Casing, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 20.10.03 - 21.10.03 CHECKED BY SMS



Continued Next Page

+ 3 × 3 : Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

## METRIC

[illegible][illegible]

+ 3, × 3: Numbers refer to Sensitivity

## METRIC

[illegible]

+ 3, × 3: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No BON-7

1 OF 4

METRIC

G.W.P. 647-92-00 LOCATION N 5 038 628.5, E 292 261.1 ( Bonnechere River ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 25.11.03 - 26.11.03 CHECKED BY SMS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	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		
113.0																		
110.9	TOPSOIL (125mm)																	
0.1	Silty CLAY Very Stiff to Stiff Brown Moist (Cl)  occasional sand pockets		1	SS	17													
			2	SS	26													
			3	SS	21													
			4	SS	19										0 2 32 66			
			5	SS	10													
			6	SS	7													
107.2																		
5.8	Silty CLAY, trace shell fragments Firm to Stiff Grey Moist to Wet Grey (Cl)		7	SS	7													
			8	SS	7													
			9	SS	2													
	Soft																	

Continued Next Page

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

# RECORD OF BOREHOLE No BON-7

2 OF 4

METRIC

G.W.P. 647-92-00 LOCATION N 5 038 628.5, E 292 261.1 ( Bonnehcchere River ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 25.11.03 - 26.11.03 CHECKED BY SMS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
	Stiff to Very Stiff (CH)						103	2.4					
			10	SS	5		102						0 0 56 44
			1	TW	PH		101						
							100						
							99						
			11	SS	7		98						
							97						
							96						
			12	SS	10		95						
							94						
							93						

Continued Next Page

+ 3, X 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BON-7

3 OF 4

METRIC

G.W.P. 647-92-00 LOCATION N 5 038 628.5, E 292 261.1 ( Bonnechere River ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 25.11.03 - 26.11.03 CHECKED BY SMS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
92.5	Silty <b>CLAY</b> , with occasional sand layers to 100mm thick Very Stiff Grey (CL)		13	SS	8									
92														
91														
90														
89			14	SS	17									
88														
87														
86														
85			15	SS	8									
84														
83														

Continued Next Page

+ 3, × 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No BON-7

4 OF 4

METRIC

G.W.P. 647-92-00 LOCATION N 5 038 628.5, E 292 261.1 ( Bonnechere River ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 25.11.03 - 26.11.03 CHECKED BY SMS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
81.9			16	SS	11								
31.1	END OF BOREHOLE AT 31.09m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.  WATER LEVEL READINGS: DATE DEPTH(m) 04/02/04 destroyed												

ONTMT4 7450BON.GPJ 11/05/04

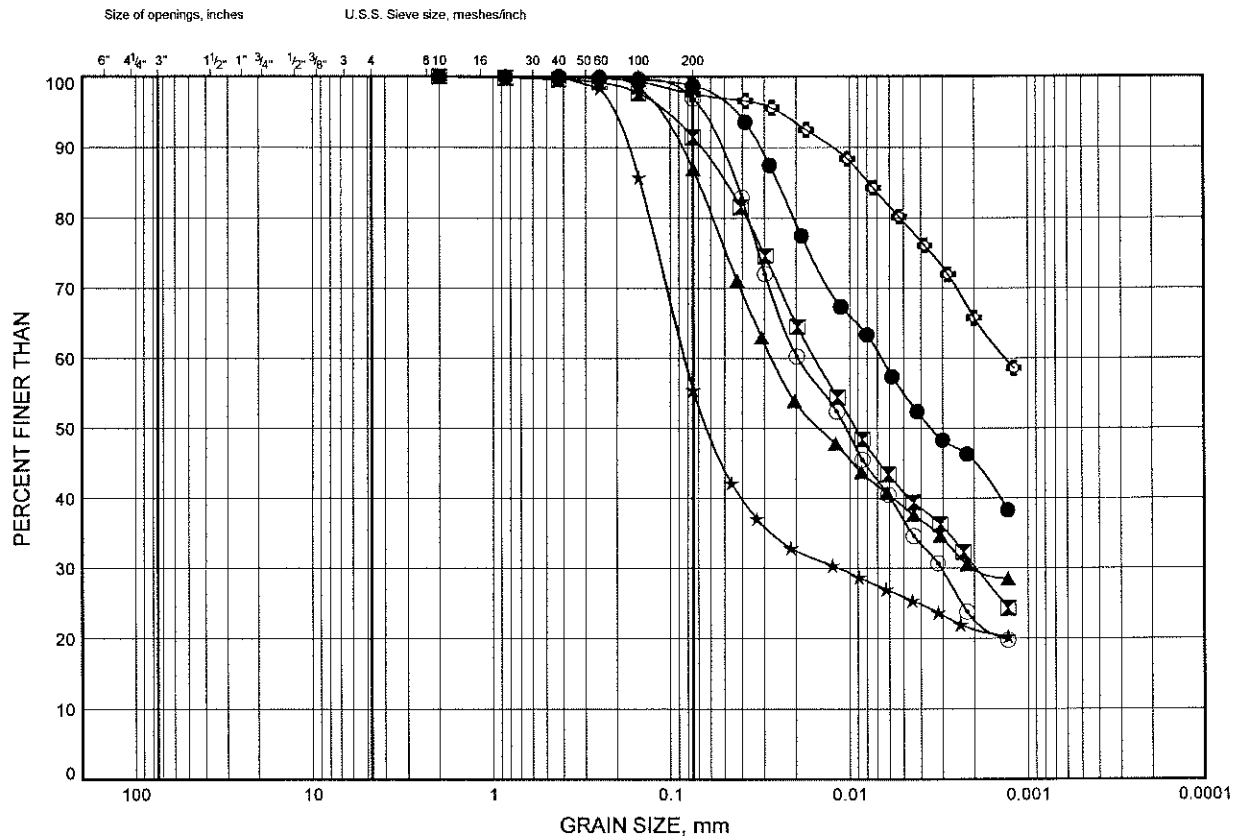
## **Appendix B**

### **Laboratory Test Results**

# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B1

### SILTY CLAY

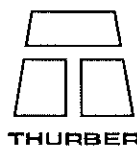


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BON-1	2.59	99.78
⊠	BON-2	4.88	82.36
▲	BON-3	1.83	84.29
★	BON-4	1.83	87.86
⊙	BON-6	3.35	99.65
⊛	BON-7	2.59	110.41

Date April 2004

Project 647-92-00



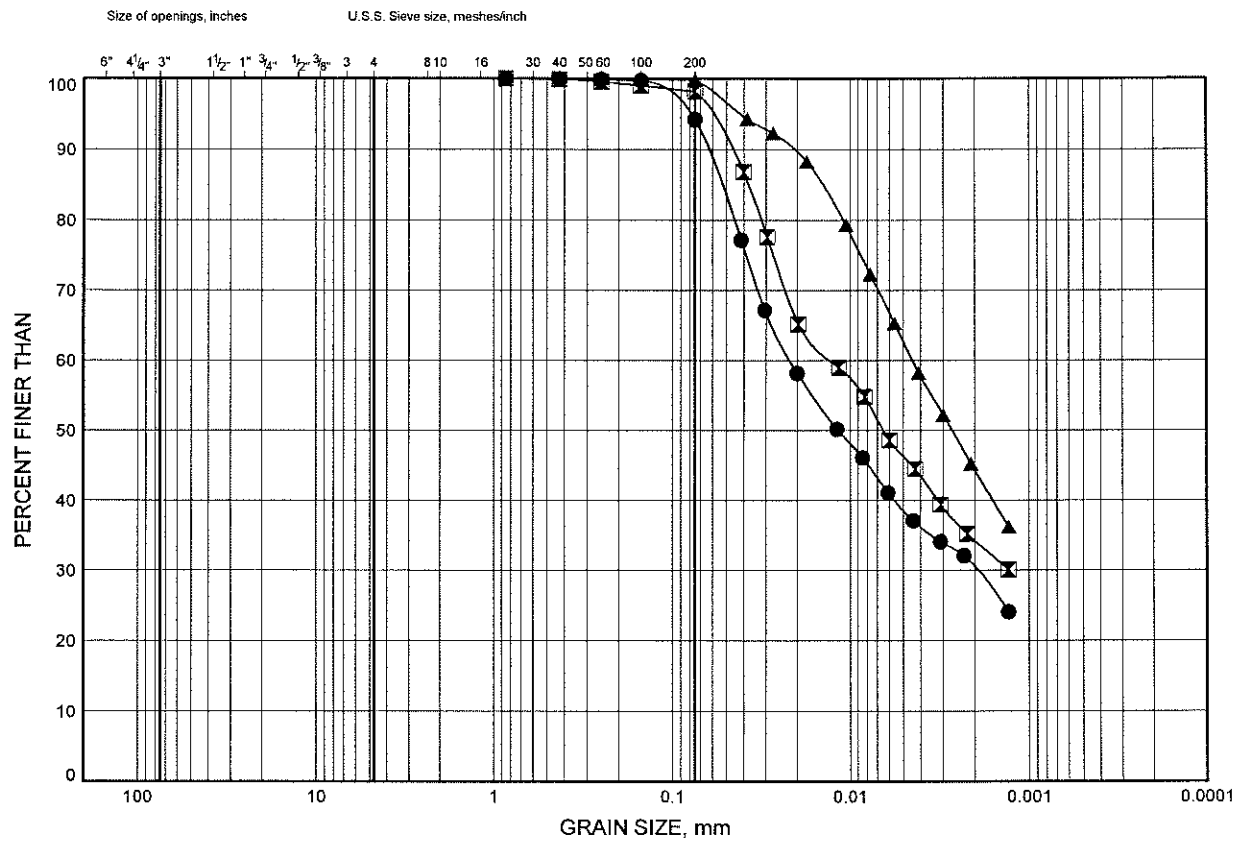
Prep'd SS

Chkd. SMS

# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B2

## SILTY CLAY

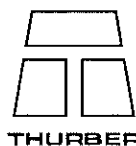


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BON-1	10.97	91.40
⊠	BON-6	7.70	95.30
▲	BON-7	10.97	102.03

Date April 2004

Project 647-92-00



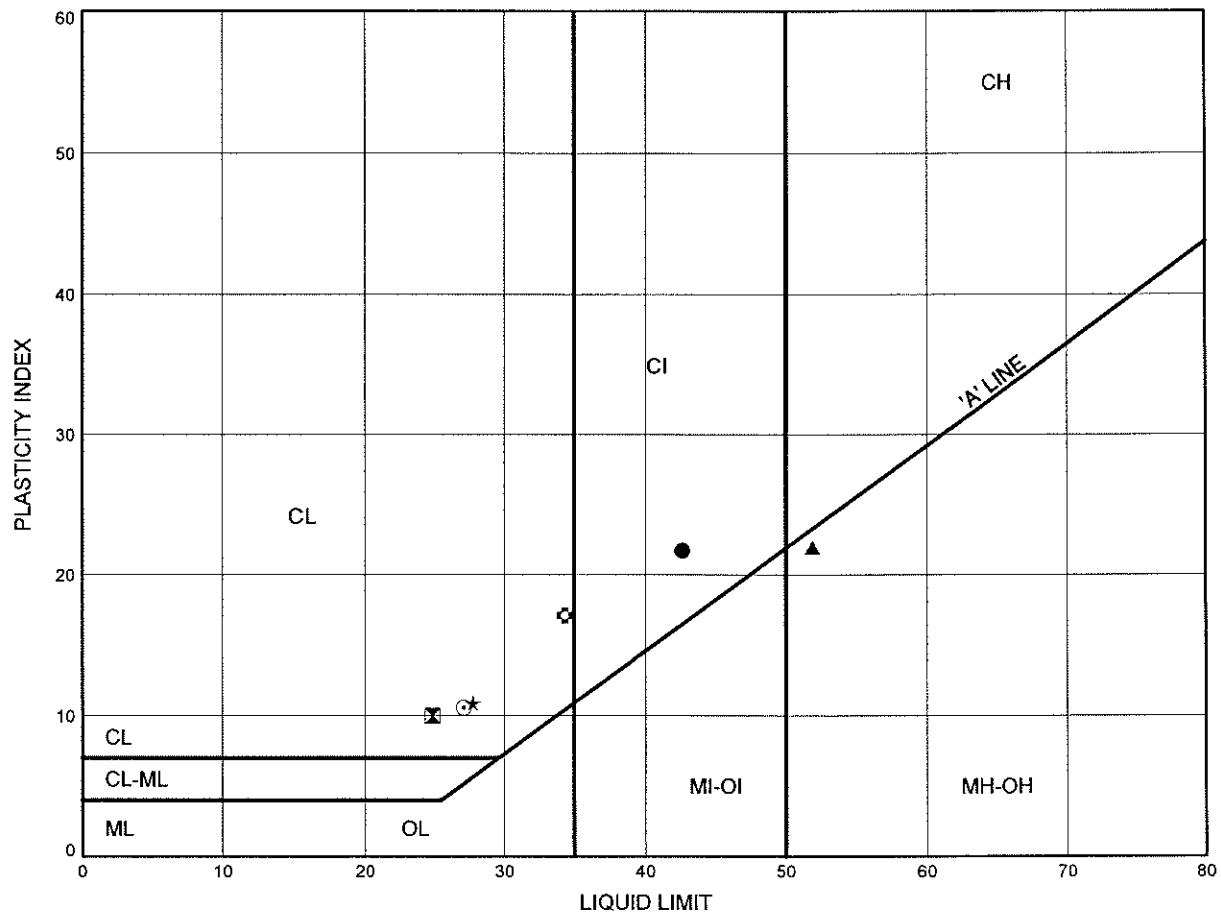
Prep'd SS

Chkd. SMS

# HWY 17 Twinning, Arnprior to Renfrew ATTERBERG LIMITS TEST RESULTS

FIGURE B3

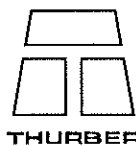
## SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BON-1	2.59	99.78
⊠	BON-1	10.97	91.40
▲	BON-2	4.88	82.36
★	BON-3	1.83	84.29
⊙	BON-6	3.35	99.65
⊛	BON-6	7.70	95.30

Date April 2004

Project 647-92-00



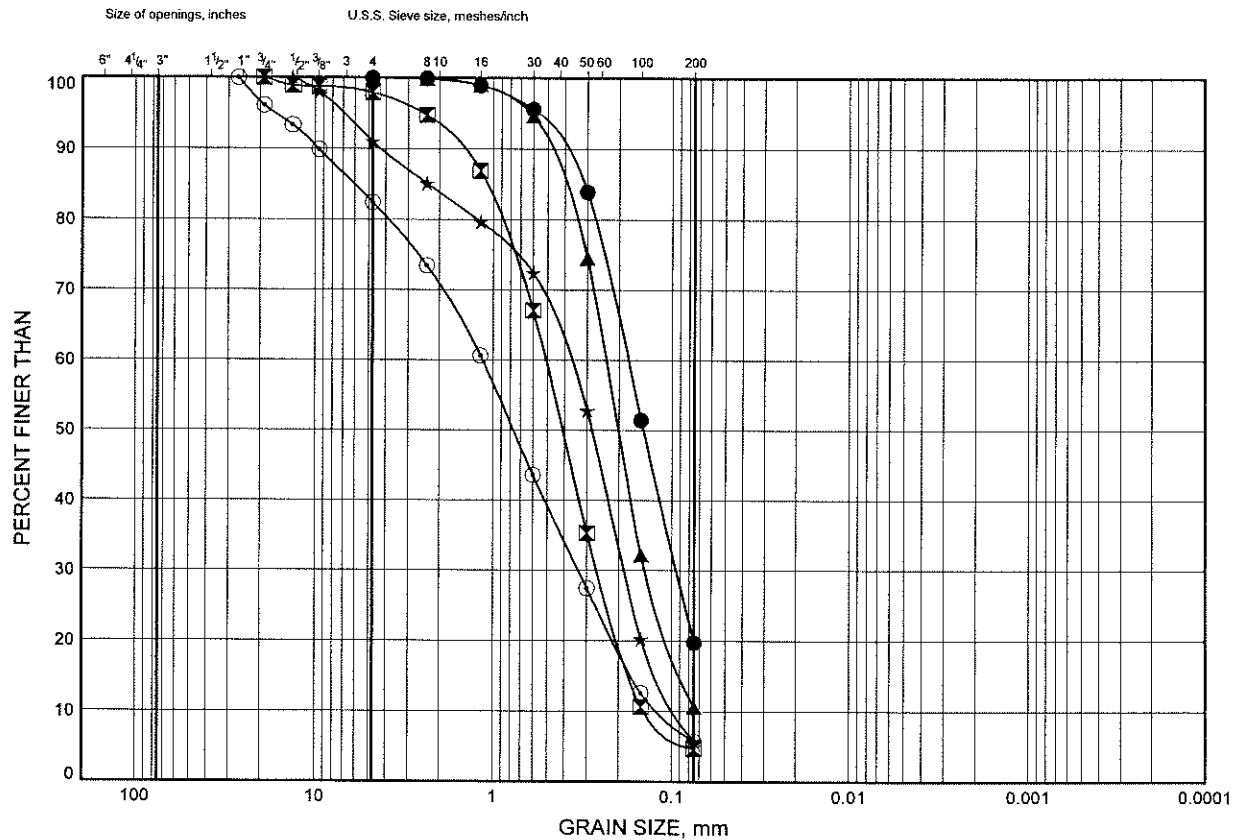
Prep'd SS

Chkd. SMS

# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B4

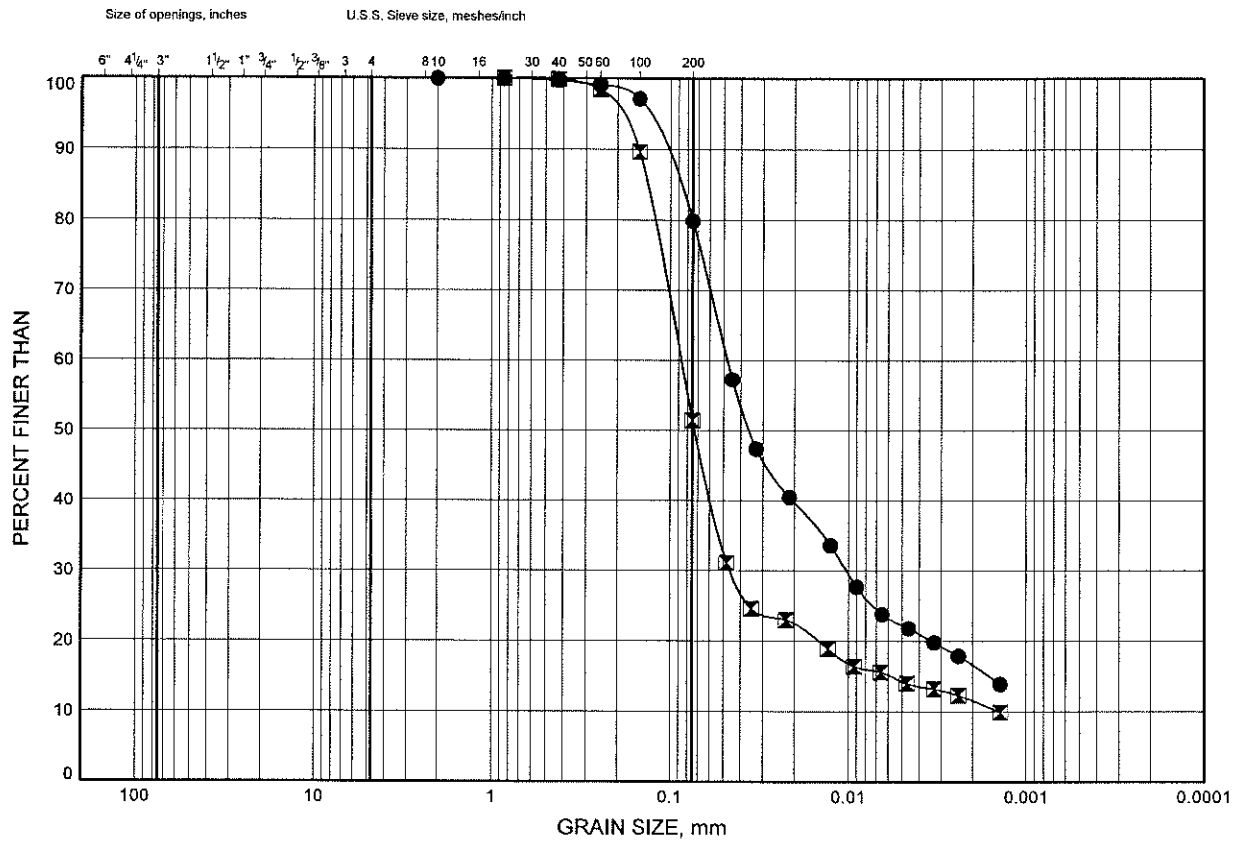
SAND, trace to some silt



# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B5

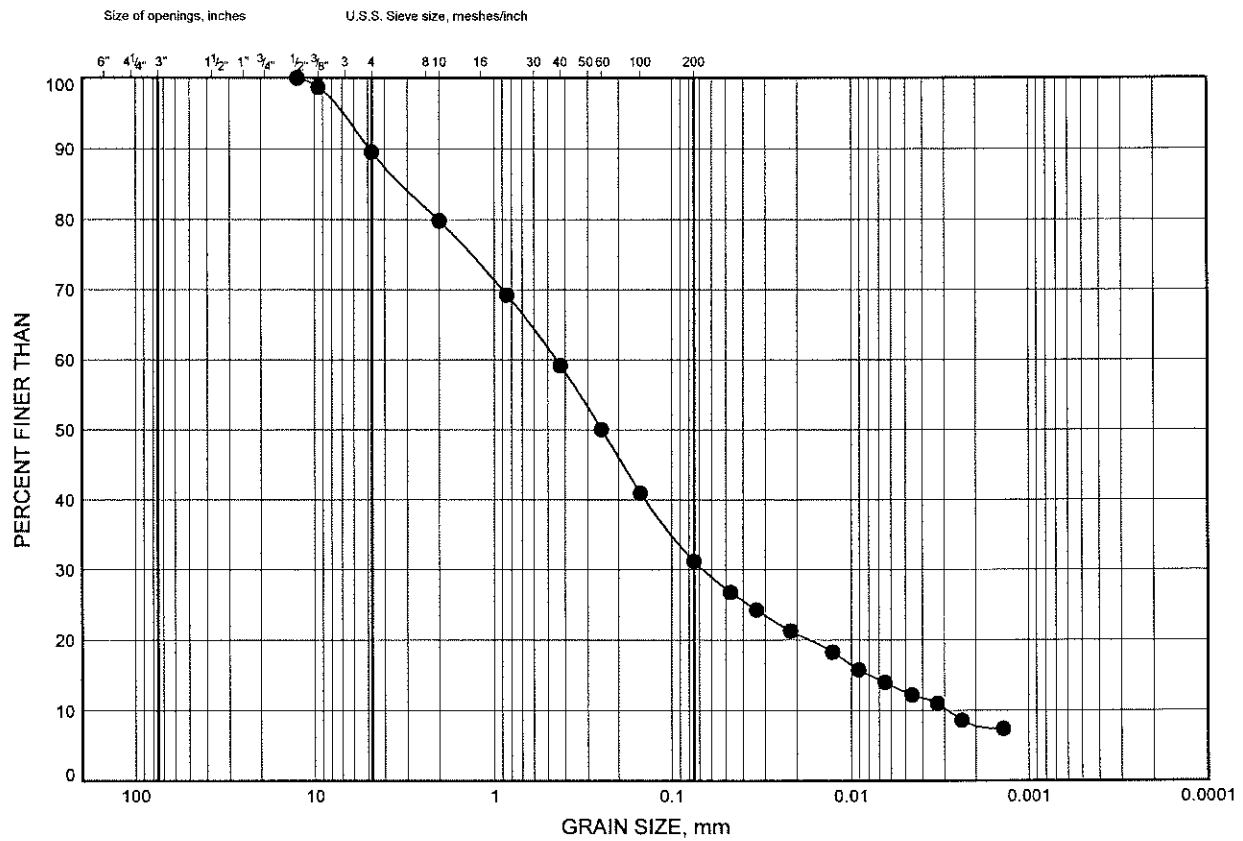
## SANDY SILT



# HWY 17 Twinning, Amprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B6

## SANDY SILT TILL

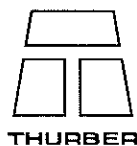


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BON-3	18.67	67.45

Date April 2004

Project 647-92-00



Prep'd SS

Chkd. SMS



## OEDOMETER CONSOLIDATION SUMMARY

## SAMPLE IDENTIFICATION

Project Number	04-1116-011	Sample Number	ST #1
Borehole Number	BON 7	Sample Depth, m	12.2-12.8

## TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	2/3/2004		
Date Completed	2/12/2004		

## SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.92	Unit Weight, kN/m <sup>3</sup>	17.22
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	11.92
Area, cm <sup>2</sup>	31.67	Specific Gravity, measured	2.78
Volume, cm <sup>3</sup>	60.80	Solids Height, cm	0.840
Water Content, %	44.50	Volume of Solids, cm <sup>3</sup>	26.59
Wet Mass, g	106.80	Volume of Voids, cm <sup>3</sup>	34.22
Dry Mass, g	73.91	Degree of Saturation, %	96.1

## TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	cv, cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
0.00	1.920	1.287	1.920				
4.69	1.910	1.275	1.915	124	6.27E-03	1.11E-03	6.82E-07
9.53	1.904	1.268	1.907	158	4.88E-03	6.46E-04	3.09E-07
19.28	1.887	1.248	1.896	321	2.37E-03	9.08E-04	2.11E-07
38.69	1.856	1.211	1.872	540	1.38E-03	8.32E-04	1.12E-07
77.39	1.814	1.161	1.835	356	2.01E-03	5.65E-04	1.11E-07
154.57	1.759	1.095	1.787	304	2.23E-03	3.71E-04	8.10E-08
309.16	1.686	1.008	1.723	171	3.68E-03	2.46E-04	8.87E-08
618.34	1.594	0.899	1.640	211	2.70E-03	1.55E-04	4.10E-08
1237.37	1.501	0.788	1.548	211	2.41E-03	7.82E-05	1.85E-08
2472.12	1.416	0.687	1.459	171	2.64E-03	3.59E-05	9.27E-09
1237.37	1.422	0.694	1.419				
309.16	1.447	0.724	1.435				
77.39	1.479	0.762	1.463				
19.28	1.506	0.794	1.493				
4.69	1.548	0.844	1.527				

Notes:

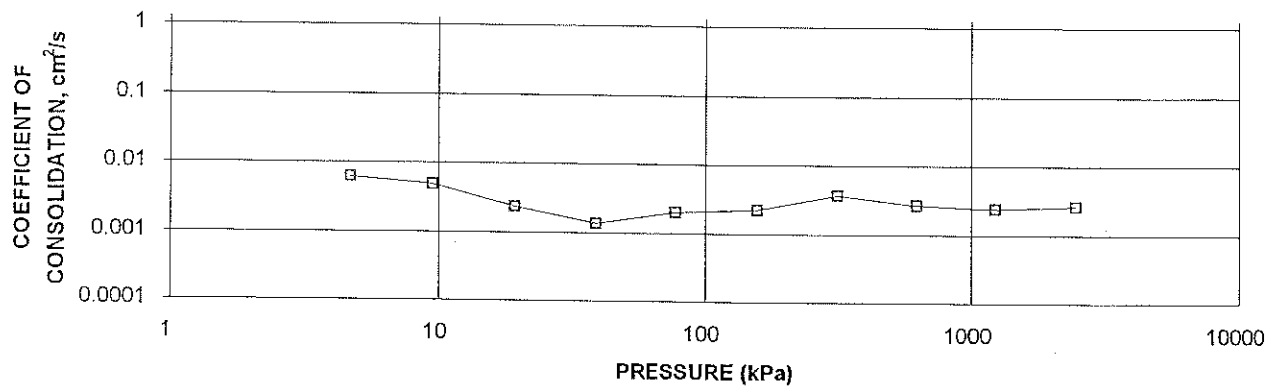
k calculated using cv based on  $\dot{\epsilon}_0$  values.

## SAMPLE DIMENSIONS AND PROPERTIES - FINAL

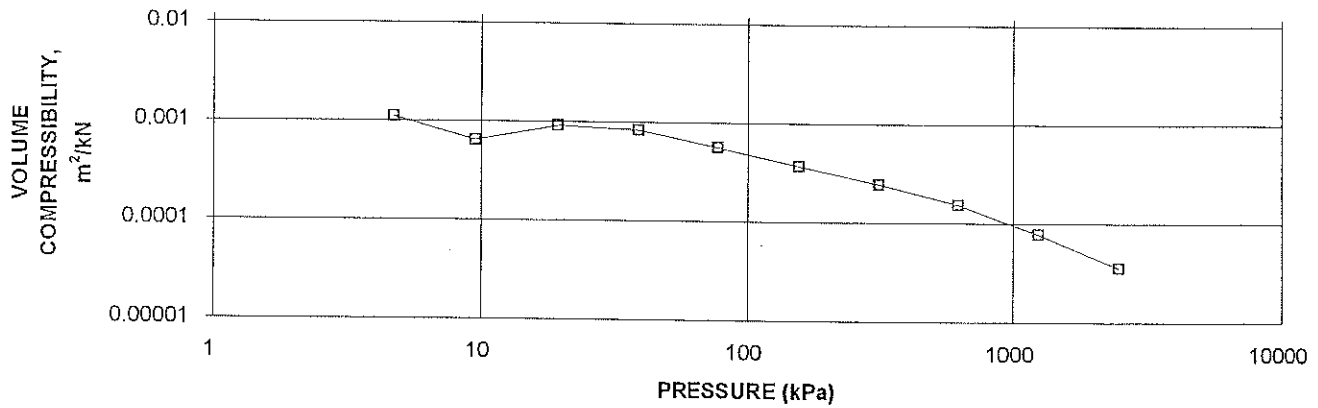
Sample Height, cm	1.55	Unit Weight, kN/m <sup>3</sup>	19.35
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	14.78
Area, cm <sup>2</sup>	31.67	Specific Gravity, measured	2.78
Volume, cm <sup>3</sup>	49.02	Solids Height, cm	0.840
Water Content, %	30.86	Volume of Solids, cm <sup>3</sup>	26.59
Wet Mass, g	96.72	Volume of Voids, cm <sup>3</sup>	22.44
Dry Mass, g	73.91		

# OEDOMETER CONSOLIDATION SUMMARY

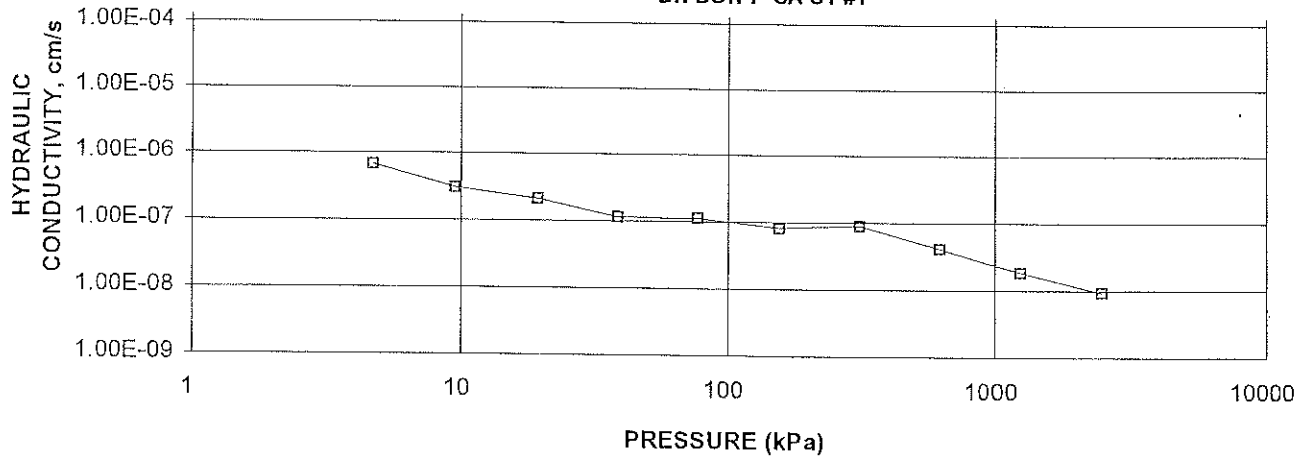
CONSOLIDATION TEST  
CV cm<sup>2</sup>/s VS PRESSURE (kPa)  
BH BON 7 SA ST #1



CONSOLIDATION TEST  
MV m<sup>2</sup>/kN vs PRESSURE (kPa)  
BH BON 7 SA ST #1



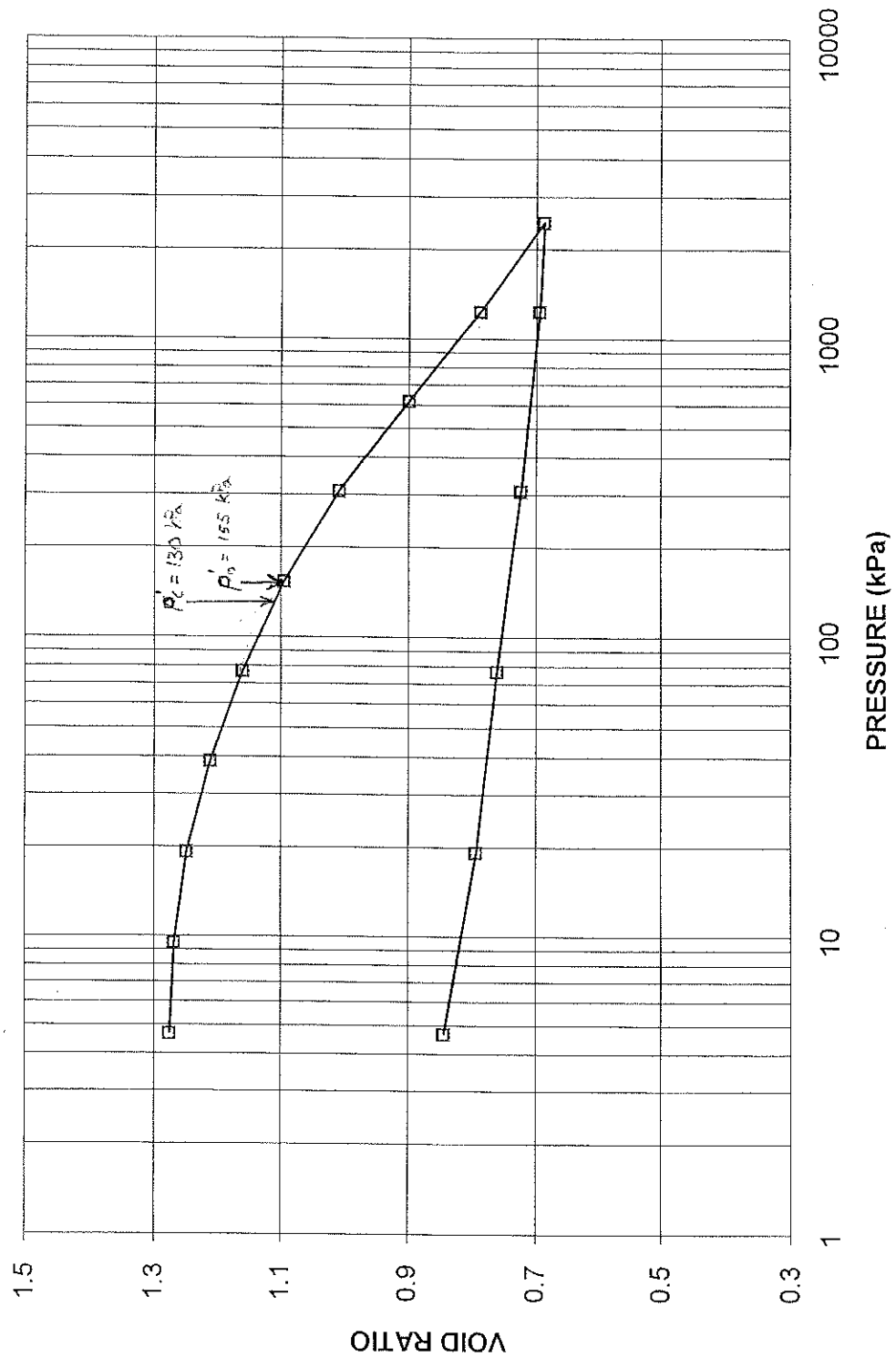
CONSOLIDATION TEST  
HYDRAULIC CONDUCTIVITY vs PRESSURE  
BH BON 7 SA ST #1



CONSOLIDATION TEST  
VOID RATIO VS. LOG PRESSURE

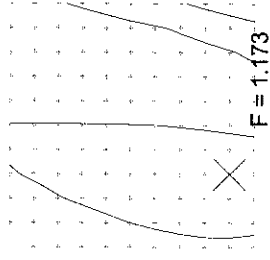
FIGURE B7

CONSOLIDATION TEST  
VOID RATIO vs. PRESSURE  
BH BON 7 SA ST #1

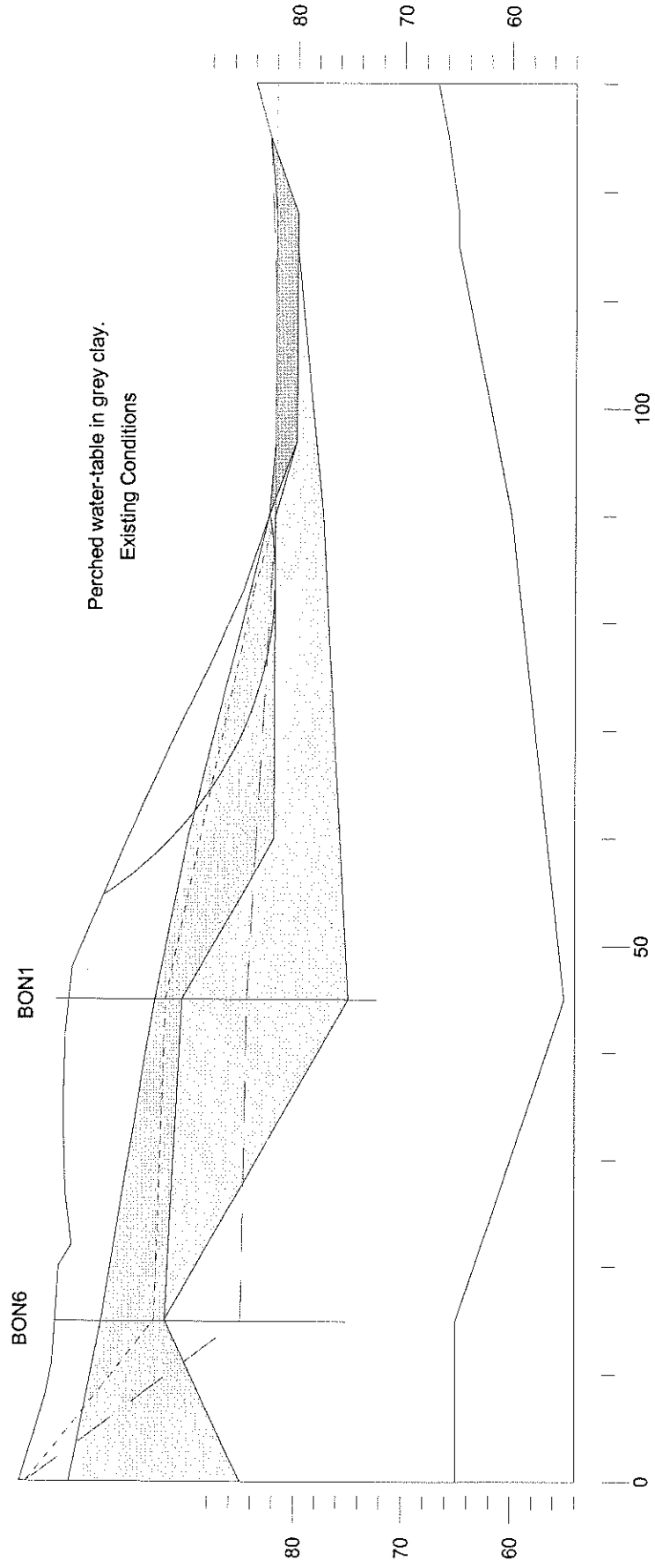


## **Appendix C**

### **Stability Analysis**

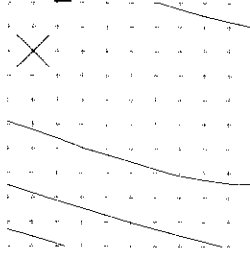


	Gamma C kN/m3	Phi deg	Piezo Surf.
Water	9.81	0	0
Dessicated Clay	19	5	29
Lower Clay	18	0	28
Silty Sand	20	0	30
SAND	20	0	32
Sand & Boulders	21	0	36



Thurber Engineering Ltd. - Toronto  
 19-3745-0  
 Bonnechere River  
 February 3, 2004  
 West Approach Fill  
 Effective Stress Analysis

F = 1.337



	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Water	9.81	0	0
Dessicated Clay	19	5	1
Lower Clay	18	0	1
Silty Sand	20	0	1
SAND	20	0	1
Sand & Boulders	21	0	1

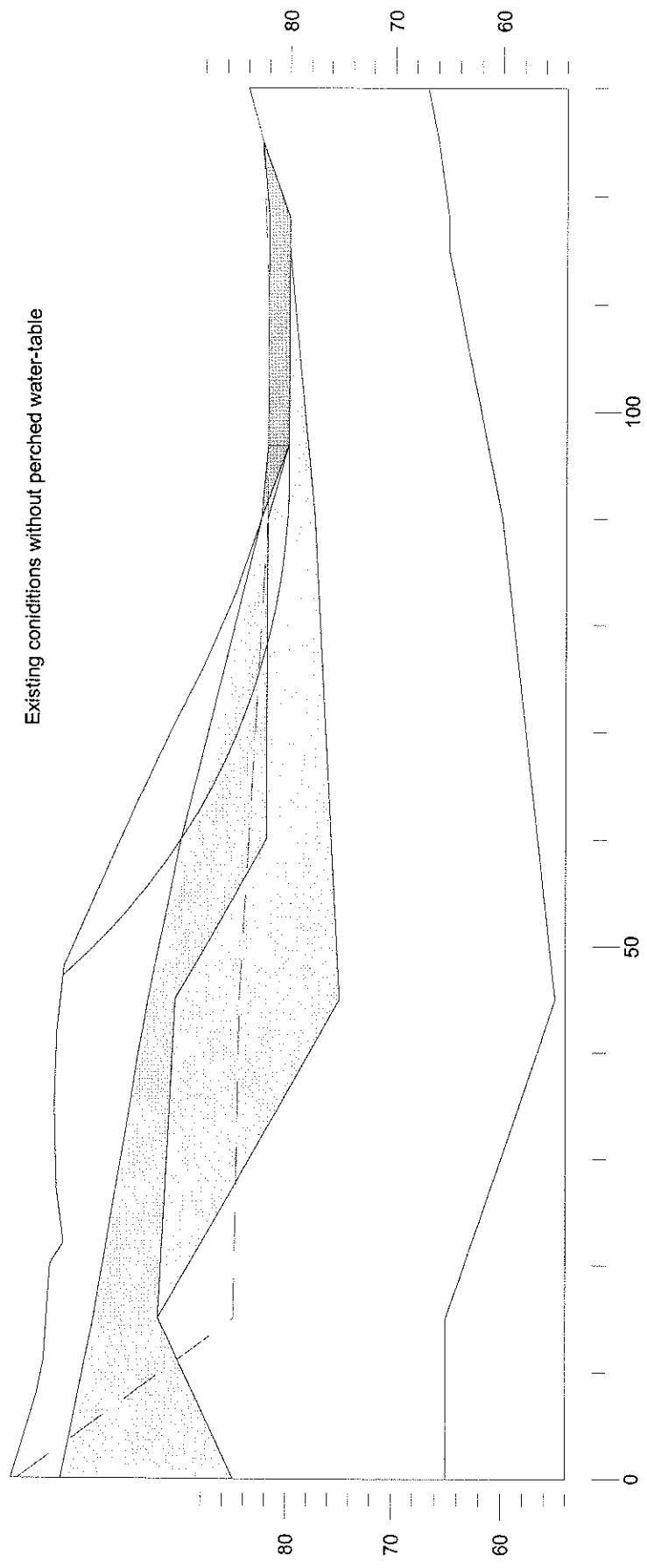
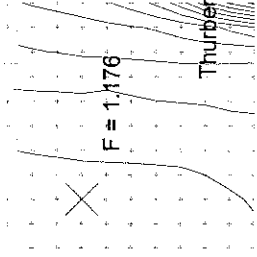
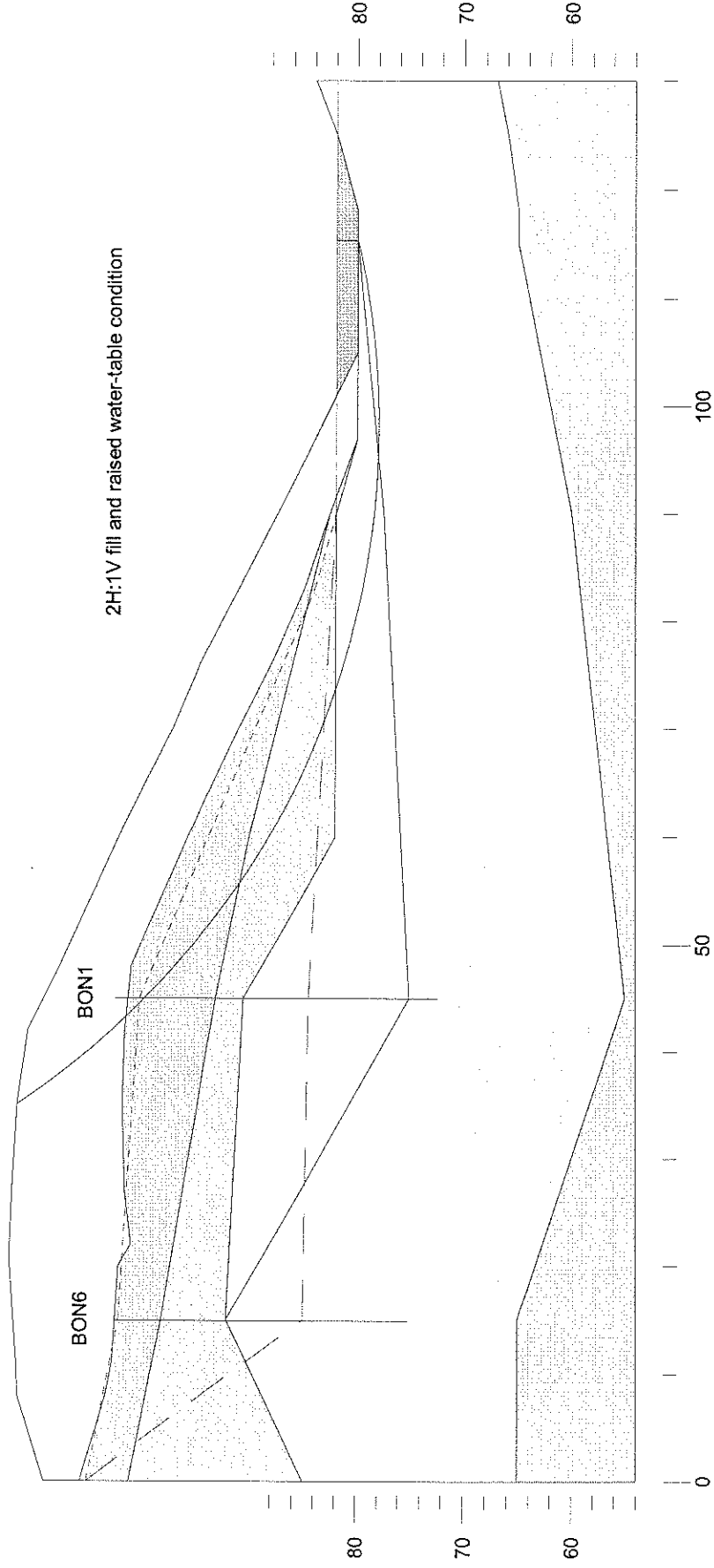


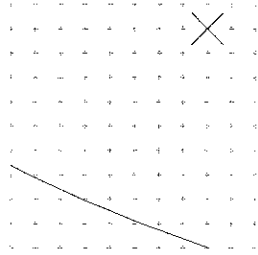
FIGURE-C1A



Thurber Engineering Ltd. - Toronto  
19-3745-0  
Bonnechere River  
February 3, 2004  
West Approach Fill  
Effective Stress Analysis

	Gamma C	Phi	Piezo
	kN/m <sup>3</sup>	deg	Surf.
Water	9.81	0	0
Fill	19.5	4	1
Dessicated Clay	19	5	2
Lower Clay	18	0	2
Silty Sand	20	0	1
SAND	20	0	1
Sand & Boulders	21	0	1





Thurber Engineering Ltd. - Toronto  
 19-3745-0  
 Bonnechere River  
 February 3, 2004  
 West Approach Fill  
 Effective Stress Analysis

F = 1.005

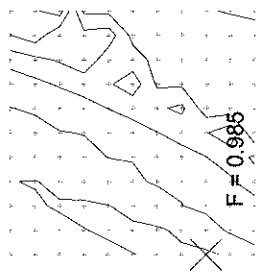
	Gamma C	Phi	Piezo
	kN/m <sup>3</sup>	deg	Surf.
Water	9.81	0	0
Fill	19.5	4	1
Dessicated Clay	19	5	1
Lower Clay	18	0	2
Silty Sand	20	0	1
SAND	20	0	1
Sand & Boulders	21	0	1

Fill at 1.7H:1V with raised water-table condition.





Thurber Engineering Ltd. - Toronto  
 19-3745-0  
 Bonnechere River  
 February 3, 2004  
 West Approach Fill  
 Effective Stress Analysis



	Gamma	C	Phi	Piezo
	kN/m <sup>3</sup>	kPa	deg	Surf.
Water	9.81	0	0	0
Wall	20	100	0	1
Fill	19.5	4	29	1
Dessicated Clay	19	5	29	1
Lower Clay	18	0	28	2
Silty Sand	20	0	30	1
SAND	20	0	32	1
Sand & Boulders	21	0	36	1

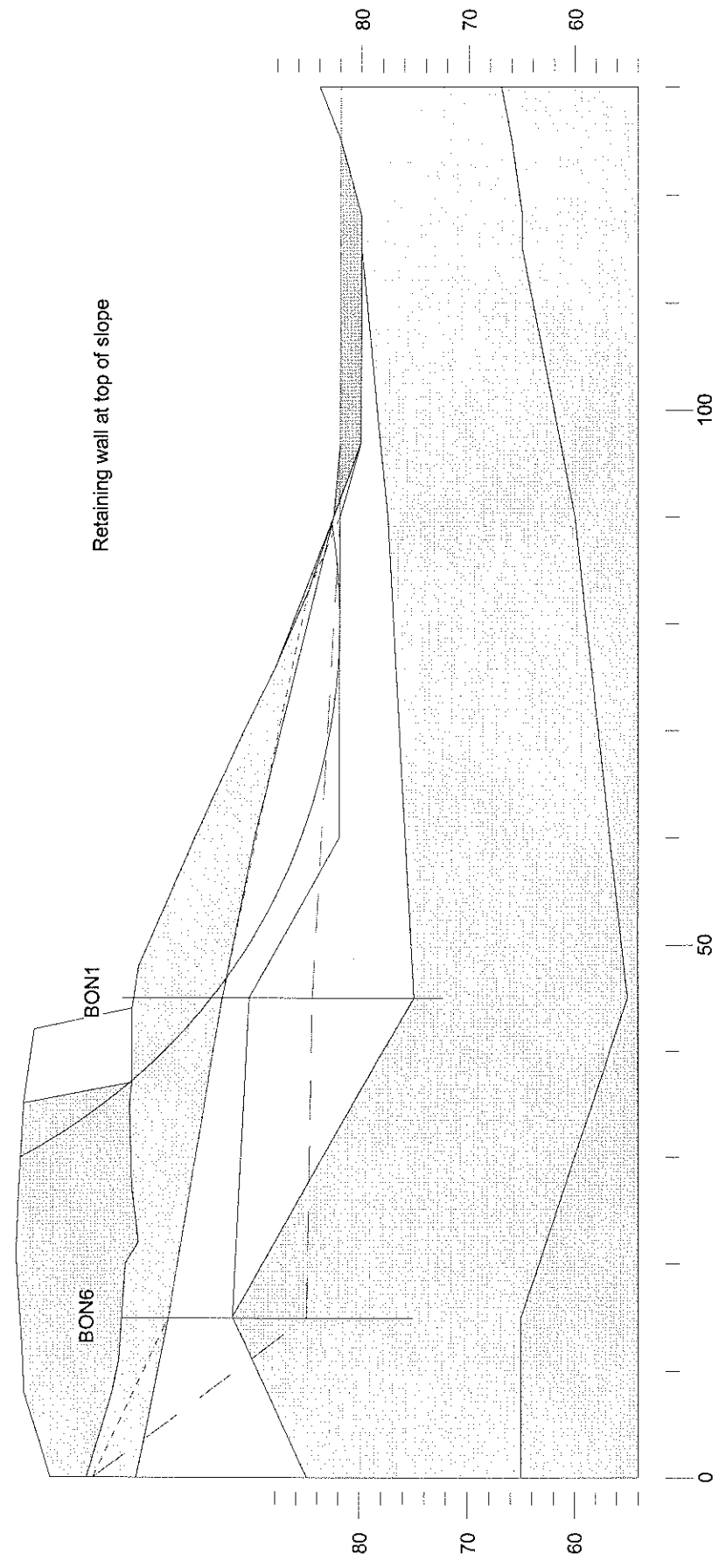


Figure-C4

## **Appendix D**

### **Settlement Analysis**

HWY 17  
BONNECHERE RIVER  
SETTLEMENT ANALYSIS

CONSOLIDATION PARAMETERS

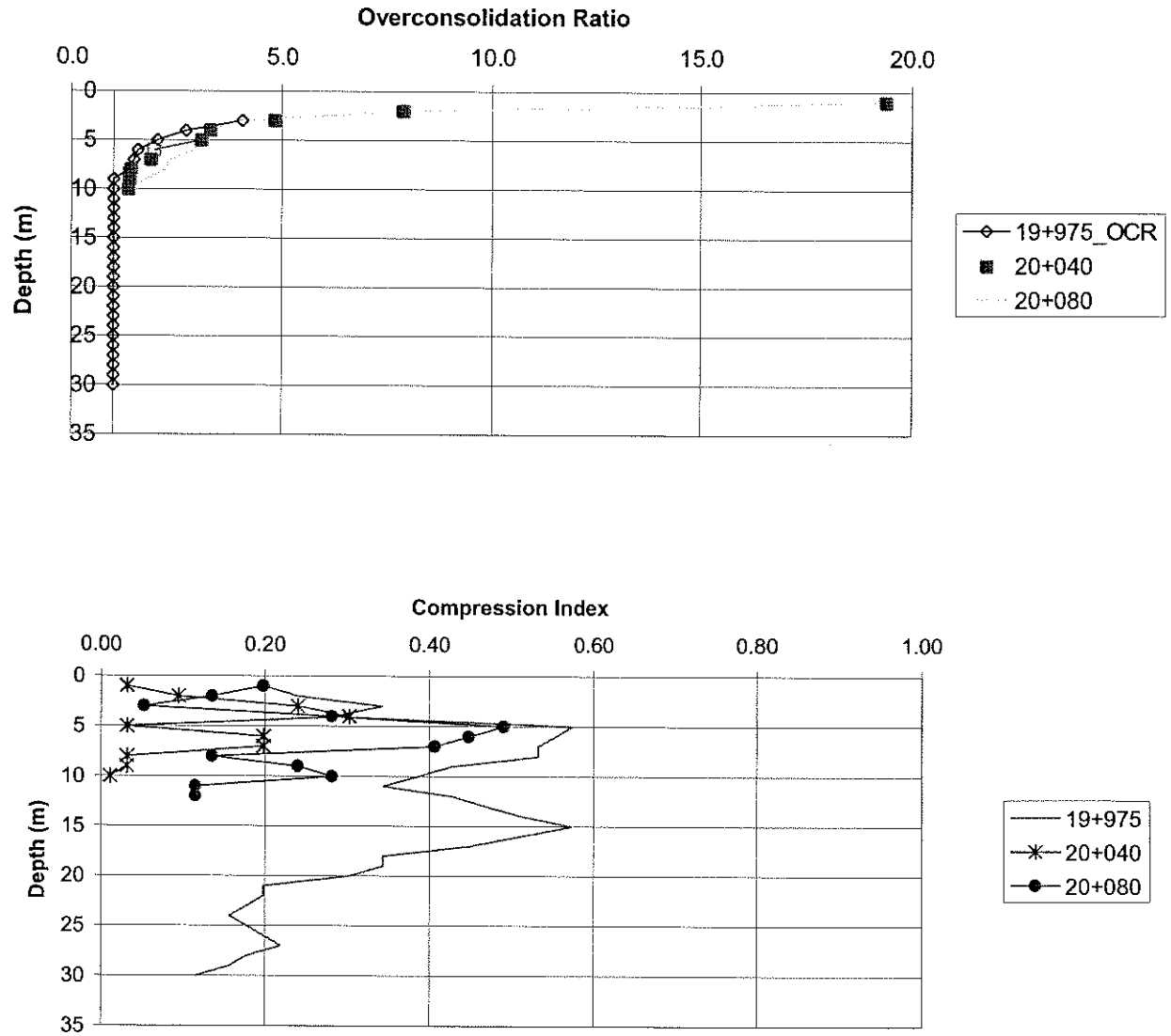
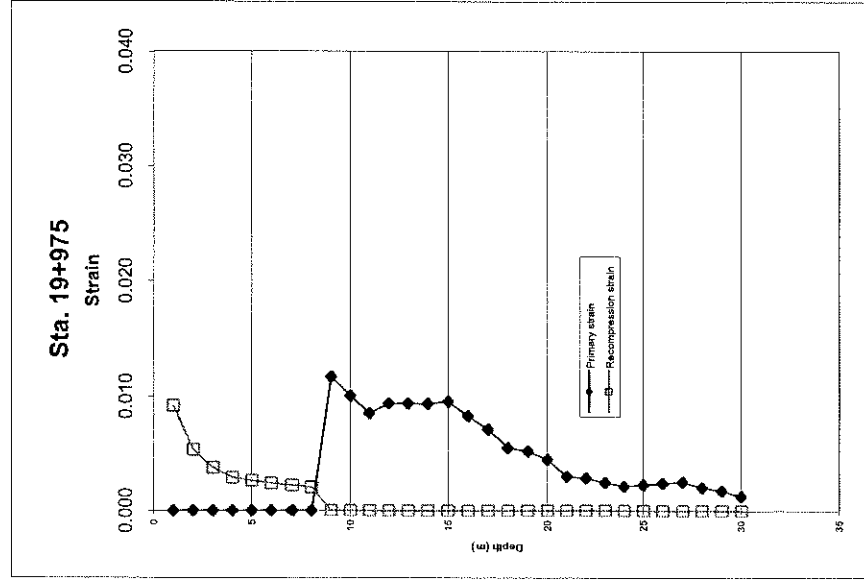
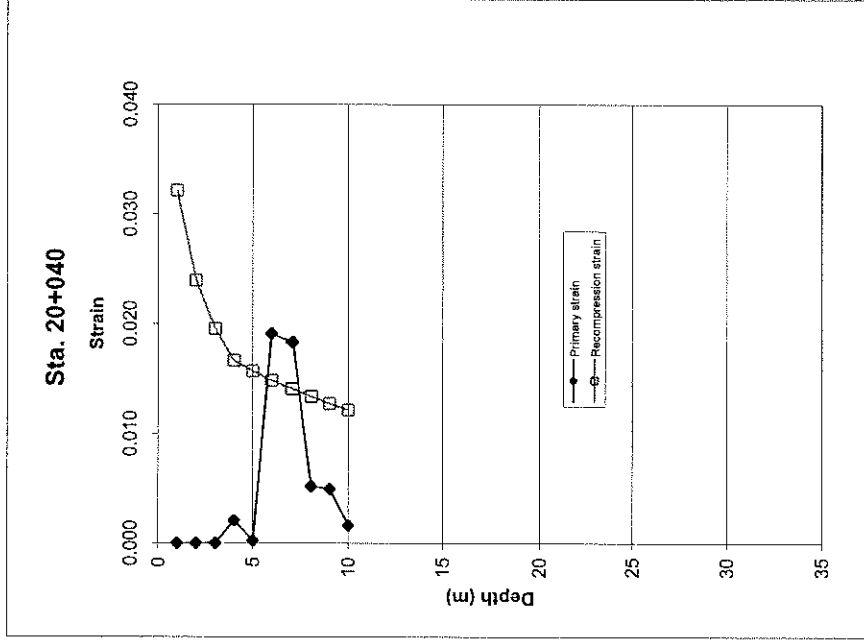


FIGURE D1

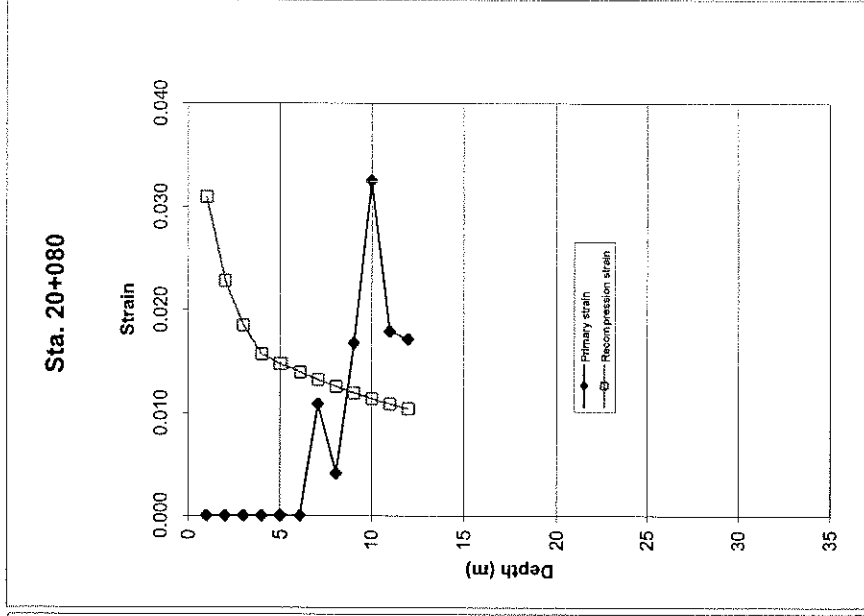
HWY 17  
BONNECHERE RIVER  
SETTLEMENT ANALYSIS



Primary settlement 120 mm  
Recompression settlement 31 mm

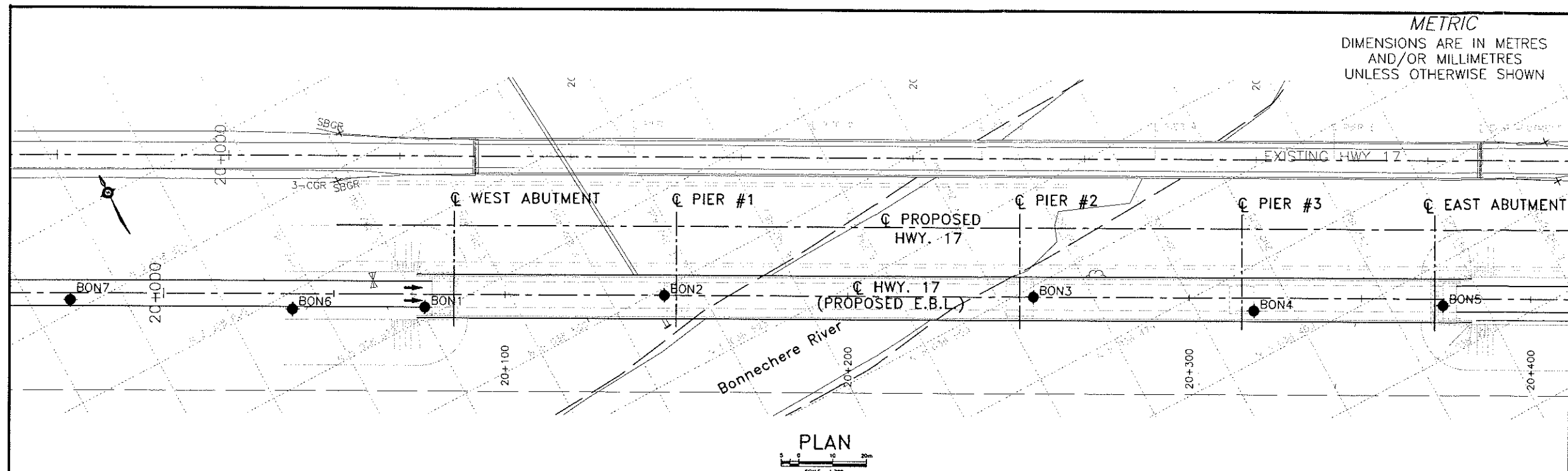


Primary settlement 50 mm  
Recompression settlement 175 mm



Primary settlement 100 mm  
Recompression settlement 190 mm

FIGURE D2



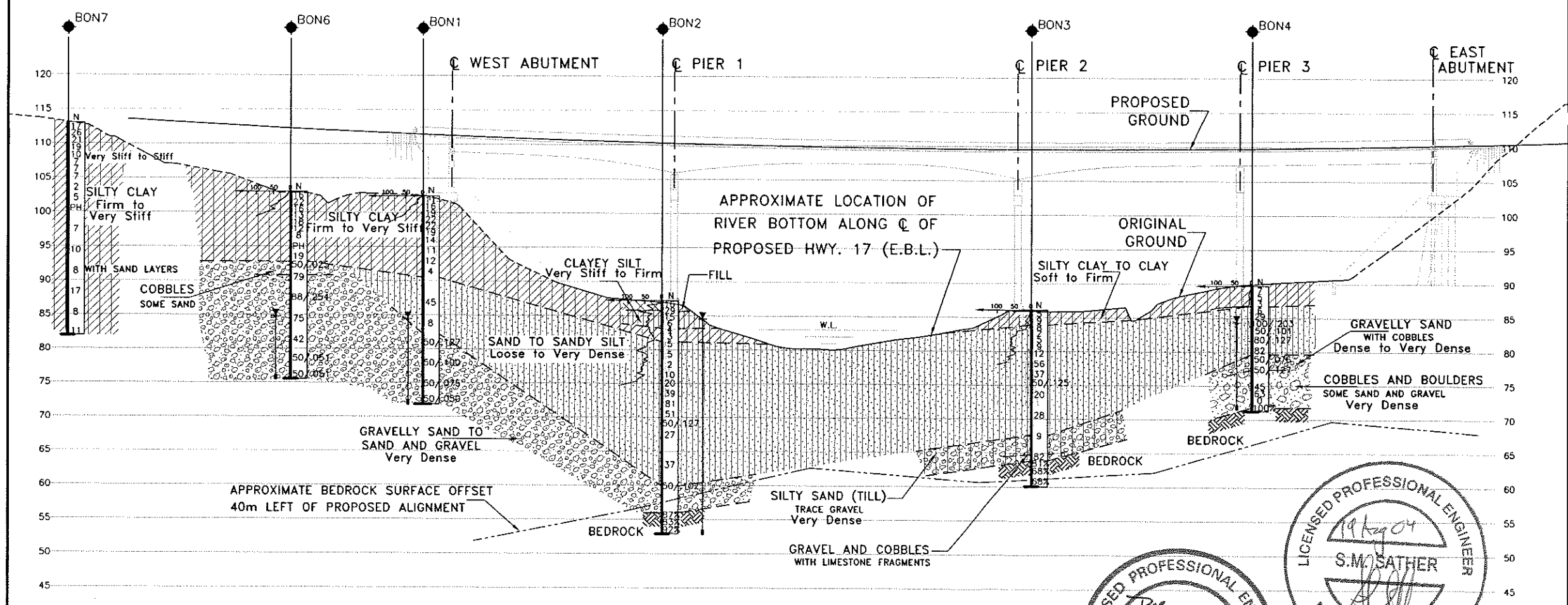
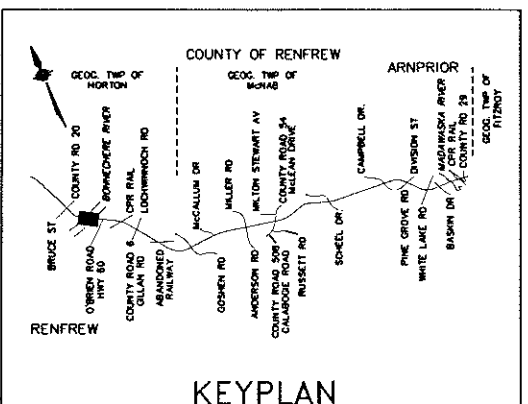
METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

DIST NO. 42, HWY. 17  
WP NO. 647-92-00

HIGHWAY 17 TWINNING  
Bonnechere River Bridge  
BOREHOLE LOCATIONS AND SOIL STRATA

**TH**  
engineers  
architects  
planners

**THURBER**  
THURBER ENGINEERING LTD.



LEGEND			
●	Bore Hole		
⊕	Dynamic Cone Penetration Test (cone)		
⊗	Bore Hole & Cone		
N	Blows/ 0.3m (Std Pen Test, 475 J/blow)		
CONE	Blows/ 0.3m (60° Cone, 475 J/blow)		
PH	Pressure, Hydraulic		
↓	WL at Feb 4/2004		
⊕	Head Artesian Water		
⊕	Piezometer		
90%	Rock Quality Designation (RQD)		
NO	ELEVATION	NORTHING	EASTING
BON1	102.4	5 038 578.0	292 351.3
BON2	87.2	5 038 548.1	292 414.8
BON3	86.1	5 038 497.2	292 509.5
BON4	89.7	5 038 463.3	292 564.5
BON6	103.0	5 038 595.5	292 317.1
BON7	113.0	5 038 628.5	292 261.1

PROFILE  $\phi$  HWY 17 E.B.L.

LICENSED PROFESSIONAL ENGINEER  
19 Aug 04  
S.M. SATHER  
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER  
P. K. CHATTERJI  
Aug 19/04  
PROVINCE OF ONTARIO

— NOTE —  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION
AUG 2004 SMS			FINAL
MAY 2004 SMS			ISSUED AS DRAFT FOR REVIEW
DESIGN	SMS	CHK PKC	CHBDC 2000 LOAD
DRAWN	SS	CHK SWS	SITE 29-192/1 STRUCT DWG.