

**PRELIMINARY  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
BRUCE STREET UNDERPASS  
HIGHWAY 17 TWINNING  
ARNPRIOR TO RENFREW, ONTARIO  
G.W.P. 647-92-00, SITE No. 29-406  
GEOCRES Number: 31F-139**

**Report to**

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**PART 1: FACTUAL INFORMATION**

**1 INTRODUCTION**

This report presents the factual findings obtained from a preliminary foundation investigation conducted at the location where a new underpass structure will carry Bruce Street over the twinned Highway 17.

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 and soil strata drawing with a stratigraphic profile, records of boreholes, laboratory test results and a written description of the subsurface conditions.

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 site is located about 50 m west of the existing at grade intersection of Highway 17 and Bruce Street, Township of Horton, County of Renfrew (approximate mainline Station 18+517 on the present Highway 17). The Borehole Locations and Soil Strata drawing in Appendix D contain further details on the general site location.

The site terrain is relatively flat with open areas on both sides of Highway 17. Vegetation is light and consists mostly of grass and small shrubs with occasional patches of larger trees.

The project area is located between the Laurentian upland to the north and west, and the Ottawa lowland to the south and east. This area is situated within a physiographic region known as the Ottawa Valley Clay Plains. In this region, native soil deposits typically consists of glacio-lacustrine clayey silts to silty clays that were deposited when the Champlain Sea inundated the Ottawa – St. Lawrence lowland. This clay deposit varies in thickness over the region. In Renfrew County, there are prominent east-west trending scarps (fault zones), including a major depression geologically known as the “Ottawa-Bonnechère” graben. Bedrock in the site area consists of marble of the Precambrian Age.

### 3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out during the period of October 7 to 29, 2003 and consisted of drilling and sampling a total of three boreholes to depths ranging from 50.2 m to 53.2 m. The boreholes were numbered BRU-1, BRU-2 and BRU-3 and their approximate locations are shown on the Borehole Locations and Soil Strata Drawing in Appendix E.

The borehole locations were marked in the field by surveyors from J. D. Barnes Limited who also provided us with the coordinates and geodetic elevations of the boreholes. Utility clearances at the borehole locations were obtained by Thurber prior drilling.

George Downing Estate Drilling Limited of Calumet, Quebec supplied a track mounted CME 55 drill rig and conducted the drilling, sampling and in-situ testing operations. Drilling commenced by using continuous flight hollow stem augers and was later modified to using NW casing in conjunction with wash boring techniques. In all of the boreholes, frequent cobbles and boulders were encountered in the silty sand till above the bedrock that required a combination of diamond coring and wash boring techniques for advancing the boreholes to reach bedrock.

Soil samples were obtained using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). In the cohesive deposits, undisturbed samples were taken at selected depths with thin-walled (Shelby) tubes, and the undrained shear strength of the clay was measured in-situ by field vane tests using an MTO 'N' size vane. Where diamond coring and wash boring techniques were used in combination, both wash samples and core samples were collected. All three boreholes were advanced for about 1.4 m to 2.9 m into bedrock by NQ size rotary coring techniques.

Groundwater conditions were observed in the open boreholes throughout the drilling operations. A piezometer was installed in each borehole for monitoring of groundwater level. The installation details are presented on the Records of Boreholes in Appendix A and summarized as follows. At this site, 19 mm diameter Schedule 40 PVC pipes with a 1.5 m long slotted screen were installed near the bottom of the open boreholes. The sand screens surrounding the pipes were about 2 m long. Bentonite holeplug seals were placed just above the sand screen and just below the ground surface in each installation. The remaining space in the boreholes was backfilled with drill cuttings.

A member of Thurber's technical staff supervised the drilling and sampling operations on a full time basis. The supervisor logged the boreholes, secured the soil and rock samples in labelled containers and core boxes, respectively, which were then transported to Thurber's Oakville laboratory for further examination and testing.

All rock cores were logged, and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

#### **4 LABORATORY TESTING**

The recovered soil samples 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. Selected samples were subjected to gradation analysis and Atterberg Limit Tests. A consolidation test was also performed on an undisturbed sample of the silty clay to clay retrieved from Borehole BRU-1. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures contained in Appendix B.

Point load tests were carried out on selected rock cores retrieved from the three boreholes. These results are presented in Table 1 attached immediately following the text.

#### **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 this appendix and on the "Borehole Locations and Soil Strata" in Appendix D of the report. A description of the stratigraphy is given in the following paragraphs. The factual information at the borehole locations governs any interpretation of site conditions.

In general, the site is underlain by thin veneers of topsoil or fill overlying an extensive deposit of silty clay to clay, which is underlain by silty sand till with cobbles and boulders. The overburden is underlain by marble bedrock.

##### **5.1 Topsoil**

A surficial layer of topsoil varying in thickness from 125 mm to 150 mm was encountered on both sides of the existing highway (at Boreholes BRU-1 and BRU-3).

##### **5.2 Fill (Sand and Gravel)**

Fill was encountered on the west side of the existing Highway 17 in Boreholes BRU-1 and BRU-2. Borehole BRU-1 was likely drilled through the remnants of an old gravel road, and Borehole BRU-2 was extended through the shoulder of an existing Highway 17 ramp. The thickness of this fill ranges from 0.6 m to 0.7 m.

The fill generally is comprised of sand and gravel to sandy silt, some gravel. Recorded SPT 'N' values of 9 blows and 16 blows for 0.3 m penetration indicated that this fill was in a loose to compact state. Moisture contents of samples from this deposit were measured at 7% and 20%.

##### **5.3 Silty Clay to Clay**

The topsoil or cohesionless fill is underlain by a major deposit of silty clay to clay. This deposit was encountered at depths ranging from 0.2 m to 0.7 m below ground surface, or

from Elevations 150.9 m to 149.1 m. The clay extends to depths of 42.5 m to 44.1 m, or from Elevations 105.2 m to 109.1 m.

Based on strength correlations, texture and appearance, the deposit can be divided into three zones. The upper zone or “dessicated crust” is about 5 m to 6 m thick, brown to grey-brown in colour, and extends to between Elevations 146 m and 143.5 m. The middle zone has a firm to soft consistency, grey in colour, and its thickness ranges from about 26 m to 32 m, or extending to approximate Elevation 115 m. From about Elevations 115 m to 107 m, frequent sand and silt layers and seams, and inferred cobbles, boulders were present within this lower zone.

Standard Penetration Tests conducted within the upper zone of the deposit gave ‘N’ values ranging typically from 7 to 16 blows per 0.3 m penetration indicating a firm to very stiff consistency. Similar trends of consistency were inferred from pocket penetrometer values.

In the middle zone, SPT ‘N’ values typically ranged from 3 blows to 8 blows per 0.3 m penetration indicating a soft to firm consistency, although zones of very soft clay (‘N’ values of 1 to 2 blows) were encountered. Some ‘N’ values of greater than 10 blows were measured mostly in Borehole BRU-3. Field vane shear strengths in this lower zone, however, ranged generally between 48 kPa and 58 kPa, with occasional values up to 80 kPa. The relatively higher in-situ vane strengths are attributed to the presence of sand and silt layers and seams.

The silty clay to clay deposit interlayers with a compact to very dense silty sand to sandy silt layer encountered between approximate Elevations 116 m and 111 m.

Below this cohesionless layer, the lower zone of the silty clay to clay deposit contains inferred cobbles and boulders (particularly in Borehole BRU-3) and frequent sand and silt layers and seams. SPT ‘N’ values ranged from 9 blows per 0.3 m penetration to greater than 50 blows for less than 0.3 m penetration indicating a stiff consistency and possible presence of cobbles and boulders.

Grain size analyses conducted on samples from this unit are presented in Figures B1 and B2. These results show that the clay content of this soil ranges between 19% and 63%. The plasticity chart shown on Figure B3 shows that the silty clay to clay samples had plasticity indices of between 13% and 33%, indicating a medium to highly plastic soil (group symbol of CI to CH). It is noted that there is a general trend of the clay content and the plasticity decreasing with depth. The moisture contents of samples of this deposit range from 24% to 49%. The moisture contents of samples subjected to Atterberg Limit Tests are generally close to or higher than their corresponding Liquid Limit values.

A consolidation test was performed on an undisturbed sample of this deposit retrieved from Borehole BRU-1 and the results are presented in Figure B4. This test indicates a probable preconsolidation pressure ( $P'_c$ ) of about 380 kPa, which is about 220 kPa in excess of the existing effective overburden pressure ( $P'_o$ ). The initial void ratio  $e_0$  corresponding to the

existing overburden pressure is approximately 1.05. The test results also show an estimated compression index,  $C_c$ , value of about 0.45 and a recompression index,  $C_r$ , of about 0.05. The measured specific gravity and unit weight of the sample was 2.79 and 18.0 kN/m<sup>3</sup>, respectively.

Two selected samples from the sandy silt to silty sand layer were subjected to grain size distribution tests and the results are presented in Figure B5.

#### 5.4 Silty Sand with cobbles and boulders (Till)

Below the silty clay to clay, a silty sand till deposit was encountered between 42.5 m and 50.3 m depths, or approximate Elevations 109 m to 100 m. Frequent cobbles and boulders were encountered in this deposit and a combination of coring and wash boring techniques was adopted in order to advance the boreholes through this stratum.

A SPT 'N' value of 110 blow for 0.3 m penetration was measured within the upper portion of this till in Borehole BRU-1, indicating a very dense state. Only wash or grab samples were obtained elsewhere within this deposit. The measured moisture contents of samples of this till range from 5% to 35%.

#### 5.5 Bedrock

The soils described above are underlain by marble bedrock. Bedrock was proven by coring in all three boreholes. The table below summarizes the depth to bedrock and the elevations of the bedrock surface.

Borehole Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
BRU-1	48.8	102.8
BRU-2	50.3	100.0
BRU-3	48.6	100.7

The marble bedrock is generally in a slightly to moderately weathered state. Its colour is grey, brown and white with sub-horizontal black banding.

Rock core recovery, measured in TCR values, was generally between 98% to 100%. Most RQD values ranged between 63% and 79% indicating a fair to good rock quality. An RQD value of 52% was, however, recorded in the upper run (Run #1) of Borehole BRU-2.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was relatively low and generally ranged from 0 to 4, with occasional higher values of 5. The joint orientation was generally sub-vertical to vertical. The joint conditions were rough but were mostly tight with no infilling or secondary weathering material.



Point load tests were conducted on rock cores at selected intervals. The inferred Unconfined Compressive Strength (UCS) of the intact rock cores ranges between 79 MPa and 133 MPa, indicating that the rock is strong to very strong. A summary of the Point Load Test results is presented in Table 1 attached immediately following the text.

## 5.6 Water Levels

Standpipe piezometers were installed in the three boreholes and their water levels were measured on three separate site visits. These readings are summarized below.

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
BRU-1	October 22, 2003	6.4	145.2
	December 18, 2003	5.4	146.2
	February 4, 2004	5.8	145.8
	March 11, 2004	5.6	146.0
BRU-2	October 22, 2003	1.8	148.5
	December 18, 2003	*	*
BRU-3	December 18, 2003	3.5	145.8
	February 4, 2004	4.1	145.2
	March 11, 2004	3.6	145.7

\* Piezometer destroyed.

Based on these observations, local groundwater levels are anticipated to range between Elevations 145.2 m and 148.5 m.

All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe climatic events.



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**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**6 GENERAL**

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

It is understood that the preliminary design plan calls for the construction of a new structure to carry the realigned Bruce Street over the twinned Highway 17. The existing Highway 17 will become the westbound lanes of the twinned Highway 17, and a new roadway will be constructed on the south side to form the eastbound lanes. Two new ramps will also be constructed at the northwest and southeast quadrants to replace the existing at-grade ramps.

The proposed structure will consist of a two-span underpass bridge. Each span will be approximately 40 m long and the structure will be skewed at about 11° to the abutment bearings.

It is understood that the proposed Highway 17 will remain at approximately the same grade as the existing at-grade intersection. The proposed grade of the realigned Bruce Street at the abutments will be at approximate Elevation 158 m. This corresponds to approach fill heights of about 8m above existing grade.

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

**7 STRUCTURE FOUNDATIONS**

The proposed bridge for this site will consist of a two-span underpass structure with a total of three foundation elements: two abutments and one pier.

The stratigraphy encountered at the locations of the proposed abutments and pier consists predominantly of silty clay to clay overlying a bouldery silty sand till underlain by marble bedrock.

The elevations at which bedrock was encountered at the three foundation elements are as follows:



Location	Borehole Number	Elevations (m)	
		Existing Ground Surface	Top of Bedrock Surface
West Abutment	BRU-1	151.6	102.8*
Centre Pier	BRU-2	150.3	100.0*
East Abutment	BRU-3	149.3	100.7*

\* Proven by coring

### 7.1 Foundation Alternatives

This section presents discussions on available foundation alternatives, and provides preliminary recommendations and foundation design parameters for feasible and/or preferred foundation option(s) for this site.

During initial assessment, consideration was given to the following foundation types:

- Spread footings
- Driven piles
- Augered caissons (drilled shafts)

A comparison of the foundation alternatives based on advantages and disadvantages of each is included in Appendix C.

An integral abutment design is possible at this site. However, the span lengths currently anticipated may not be suitable for a semi-integral abutment design.

Spread footings founded on the native, compressible silty clay to clay, or on an engineered fill pad resting on the silty clay to clay, are not feasible due to the potentially large magnitude of settlement that may occur under the footing and embankments loads. Even if post construction settlement due to embankment loading can be reduced to within acceptable limits by means of preloading and surcharging amongst other means, the design bearing resistance will be limited due to the compressibility of the clay.

Given the relatively large depth to bedrock, it is not considered feasible to use augered caissons socketted into bedrock, due to potential difficulties during installation and the downdrag forces (relatively higher due to the larger contact surface) resulting from consolidation of the clay deposit under the weight of the approach fills.

In view of the above, it is considered that piled foundation consisting of steel piles driven to bedrock is the most feasible means of providing foundation support for both abutments and the centre pier. Given a proposed grade raise in the order of 8 m, a perched abutment design may be considered in conjunction with the piles.

### 7.2 Driven Piles

Based on the borehole information, bedrock at this site is present at between 49 m and 50m depth below existing ground surface. Steel driven piles may be used to provide foundation

support at both abutments and the centre pier. It is possible that some of the piles could meet refusal within the very dense silty sand till with cobbles and boulders immediately overlying bedrock. For preliminary design, it has been assumed that all piles will encounter refusal in the bouldery till above bedrock.

The following pile tip elevations are recommended for preliminary design purposes.

Foundation Element	Reference Boreholes	Estimated Pile Tip Elevation (m)
West Abutment	BRU-1	105±
Centre Pier	BRU-2	102±
East Abutment	BRU-3	102±

The estimated pile tip elevations shown above are based on the assumption that all driven piles will encounter refusal within the bouldery till above bedrock.

#### 7.2.1 Axial Resistance

For preliminary design, HP 310 x 110 piles driven to practical refusal within the bouldery silty sand till, the following recommended pile capacity may be used:

- Factored geotechnical resistance at ULS of 1,800 kN per pile.
- Geotechnical resistance at SLS of 1,400 kN per pile for up to 25 mm settlement.

For preliminary design, HP 310 x 132 piles driven to practical refusal within the bouldery silty sand till, the following recommended pile capacity may be used:

- Factored geotechnical resistance at ULS of 2,000 kN per pile.
- Geotechnical resistance at SLS of 1,600 kN per pile for up to 25 mm settlement.

The design pile capacities should be reviewed during detailed design.

#### 7.2.2 Downdrag on Abutment Piles

Downdrag forces could be induced on the piles at both abutments as a result of consolidation of the cohesive foundation soils under the loading of the 8 m high approach fills. The magnitude of the downdrag force depends on the contact area and the mobilized negative skin friction between the pile surface and the surrounding soil. Reference should be made to the CHBDC (2000) Clauses 6.8.4 and C6.8.4 for downdrag calculations.

It is not considered feasible to eliminate the downdrag force by means of preloading and surcharging, amongst other methods, at this site since it is known that negative skin friction can be mobilized by only several millimetres of soil settlement. As such, potential downdrag forces must be considered in the design of the piles.

Given the length of the pile and the frictional resistance provided by the lower portion of the silty clay to clay and the silty sand till with cobbles and boulders, the neutral plane is anticipated to be located within the middle portion of the clay deposit. Settlement of the pile toe should be assumed negligible and the downdrag load will act as an additional vertical load.

Downdrag forces may be calculated assuming that the negative skin friction will be mobilized on the outside perimeter of the pile, within the firm to soft middle zone of the silty clay to clay deposit where time-dependent consolidation settlement is anticipated. The following table presents preliminary estimates of the factored and unfactored downdrag forces that can be experienced by HP 310 x 110 and HP 310 x 132 piles driven to bedrock or silty sand till with cobbles and boulders at the abutments.

Location	Pile Type	Estimated Downdrag Force (kN)	
		Unfactored	Factored**
West Abutment	HP 310 x 110	675	845
	HP 310 x 132	720	900
East Abutment	HP 310 x 110	525	660
	HP 310 x 132	565	705

\* It is assumed that the upper, over-consolidated, typically stiff to very stiff clay "crust" will only undergo elastic recompression.

\*\* A load factor of 1.25 is applied as per the CHBDC.

### 7.2.3 Lateral Resistance

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

For lateral soil-pile interaction, including integral abutment design, at this site, the lateral resistance of the piles may be calculated using a value for the coefficient of horizontal subgrade reaction ( $k_s$ ) and ultimate lateral resistance ( $p_{ult}$ ) as follows:

#### New Approach Fill (compact)

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

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

Note: Due to the proximity of the embankment slope to the pile cap, it is recommended that a reduction factor be applied to the  $k_s$  values calculated above. The reduction factor

can be assumed to decrease from 1.0 to 0, corresponding to a horizontal distance of  $8B$  to  $B$ , respectively, between the outermost pile and the slope surface.

Silty Clay to Clay (very stiff to stiff, becoming firm to soft)

$$k_s = 125 \cdot S_u / B \quad (\text{kPa/m})$$

$$p_{ult} = 9 \cdot S_u \quad (\text{kPa})$$

where  $z$  = depth below underside of pile cap (conventional design), m

= depth below underside of CSP (integral design), m

$B$  = pile width, m

$n_h$  = 6,000 kPa/m (engineered fill above Elevation 150m compacted to at least 95% Standard Proctor density)

= 11,000 kPa/m (very dense silty sand till between Elevations 108m and 102m)

$\gamma$  = 20 kN/m<sup>3</sup>

$K_p$  = 3.0 (passive earth pressure coefficient)

$S_u$  = undrained shear strength of silty clay to clay

= 70 kPa (between Elevations 150m and 140m)

= 40 kPa (below Elevation 140m)

The above equations and recommended parameters may be used for numerical analysis of the interaction between a pile and the surrounding soil. The lateral pressures obtained from the numerical analysis should not exceed the ultimate lateral resistance.

The spring constant,  $K$ , for analysis may be obtained by the expression,  $K = k_s \times L \times B$  (MN/m), where  $k_s$  is the coefficient of horizontal subgrade reaction (MPa/m),  $B$  is the pile width (m),  $L$  is the length (m) of the pile segment or element used in the analysis.

Since the piles are end bearing on rock, the vertical resistance will not be significantly affected by the pile spacing. Pile interaction should be considered with reference to CHBDC Clause 6.8.9.2.

For lateral soil / pile group interaction analysis, the equation for  $k_s$  quoted above may be used in conjunction with appropriate reduction factors.

Where a pile group is oriented *perpendicular* to the direction of loading, group action may be considered by reducing values for  $k_s$  by a reduction factor  $R$  as follows :

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 B	1.00
1 B	0.50

where B is the width of the pile, and spacing is measured centre to centre

Where a pile group is oriented *parallel* to the direction of loading, group action may be considered by reducing values for  $k_h$  by a reduction factor R as follows :

Pile Spacing Parallel To Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 B	1.00
6 B	0.70
4 B	0.40
3 B	0.25

Intermediate values may be obtained by interpolation.

#### 7.2.4 Pile Installation

All piles shall be installed in accordance with Special Provision SP No. 903S01.

Prior to pile installation, an engineered fill core consisting of approved granular materials compacted to the specifications of OPSS 501 may be required. This granular core may have side slopes not steeper than 1.5H : 1V, should be free of boulders, cobbles and particles of nominal diameter not exceeding 75 mm, and should extend a minimum distance of 1.5 m beyond the perimeter of the pile cap.

In order to minimize post construction settlement and the associated downdrag, it is recommended that preloading/surcharging amongst other means of ground improvement (to be further discussed in the approach embankment section) be completed prior to driving the piles. Further assessment of this issue should be carried out during detailed design.

In order to protect the pile tips from damage while driving into the bouldery till, it is recommended that the pile tips be reinforced with driving shoes such as the standard Titus "H" Bearing Pile Point or the conventional flange plates (Standard SS 103-12).

For preliminary design, it is considered appropriate to show a pile driving note on the contract drawing indicating "Piles to be driven in accordance with Standard SS 103-11 Pile Driving Control, using an ultimate geotechnical resistance of 3,600 kN per pile (for HP 310 x 110) and 4,000 kN per pile (for HP 310 x 132), but must be driven below Elevation 107 m", in accordance with SP 903S01 (Note 2 in Clause 3.3.3 of Section 3 Piles, the Ministry of Transportation, Ontario "Structural Manual").



The piles should be driven to Elevation 107 m below which driving should be controlled by the Hiley Formula.

Further assessment on pile installation should be made during detailed design.

As per SP 903S01, retapping and/or redriving of piles should be carried out as required.

### **7.3 Frost Cover**

Frost protection should be provided to the pile caps at this site. This may take the form of 1.9 m of earth cover, or equivalent insulation, over the underside of the pile cap.

It may be possible to eliminate the depth of frost cover if:

- The foundation is underlain by at least 2.5 m of free-draining, non-frost susceptible granular fill or by rock fill, and
- The water table is maintained more than 2.5 m below the underside of the foundation.

## **8 EXCAVATION AND BACKFILL**

### **8.1 General**

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the upper stiff to very stiff zone of the silty clay to clay deposit can be classified as a Type 2 soil, and the middle soft to firm zone of this deposit (below the groundwater level) can be classified as a Type 3 soil.

### **8.2 Foundations**

The excavation and backfilling for foundations must be carried out in accordance with SP 902S01.

#### **8.2.1 Earth Excavation**

Excavation for pile cap construction will likely be carried out within the upper, stiff to very stiff, portion of the silty clay to clay. It is anticipated that such excavations may be carried out as unsupported open cuts with inclined side slopes (according to OHSA). Where open cutting is not feasible due to space restrictions and other reasons, temporary shoring will be required. Recommendations for temporary shoring design should be developed during detailed design as required.

#### **8.2.2 Rock Excavation**

Based on the existing borehole information, rock excavation will not be required at this site.

## **9 GROUNDWATER CONTROL**

The relatively impervious deposit of silty clay to clay is not expected to yield a significant quantity of water in the short term. Water seepage will occur with time into the excavations for pile cap construction that extend below the groundwater level and where water-bearing layers and seams are exposed. Surface runoff may also contribute to water accumulation in the excavations. The Contractor must control the groundwater seepage into the excavation prior to placing concrete or compacting granular fill. One possible means is to pump from filtered sumps to remove any accumulated water from the excavation base.

## **10 APPROACH EMBANKMENTS**

For the purpose of preliminary embankment stability analyses, the commercially available slope stability program GSLOPE developed by Mitre Software Inc. was used. The Bishop's simplified method for stability analysis was employed for short and long term conditions.

Preliminary estimates of foundation settlements have been made based on the methods described in the CHBDC, 2000 Commentary Section C6.6.3.6.

The global, internal and surficial stability of the approach embankment fill will depend on the slope geometry, foundation conditions and also to a large degree on the material used to construct the embankment.

### **10.1 Stability**

At the west approach, the 8 m high approach embankments for this structure will be constructed over about 42 m of stiff becoming soft silty clay to clay overlying silty sand till underlain by bedrock. At the east approach, the foundation soils consist of about 45 m of silty clay to clay overlying silty sand till underlain by bedrock. Earth fill or rock fill may be used to construct the embankments. For the rock fill option, an engineered fill core may be required to facilitate pile driving (as discussed earlier). The slope of the core may be formed not steeper than 1H : 1V for Granular A material and 1.5H : 1V for other types of cohesionless fill (the granular core should extend at least 1.5 m beyond the footing perimeter).

It is recommended that all organic, soft or otherwise unsuitable overburden materials should be removed from the footprint of the embankments. Provided that the core is constructed as recommended in this report, blast rockfill embankments formed with a slope inclination not steeper than 1.25H : 1V will be stable. Earth embankments constructed using granular, select subgrade material or clean inorganic earth fill will have stable side slopes at inclinations not steeper than 2H : 1V.

Preliminary stability analyses results indicated that a minimum Factor of Safety (F.S.) of 1.3 can be achieved for both short and long term conditions. Figures E1 and E2 in Appendix E present selected stability analyses results. Further analyses should be carried

out during detailed design, as more information becomes available, to confirm stability. For an earth fill embankment, however, a mid-height berm will be required to address surficial stability as discussed in Section 10.3 Embankment Construction. Berms are not required to maintain surficial stability of rock fill embankments at this site.

## **10.2 Settlement**

Settlement in the order to 30 to 40 mm will occur within the rock fill or well compacted non-cohesive earth fill. This settlement should be complete by the end of construction and negligible post construction settlement is anticipated in the fill.

The new approach fills will induce significant settlement within the foundation soils. The majority of the settlement is associated with the lower portion (approximate Elevations 130m to 115 m) of the clay deposit where it is lightly over-consolidated to normally consolidated. Results of preliminary calculations indicate that the settlement due to elastic recompression under 8 m of fill could be in the order of 150 mm to 175 mm, while the settlement due to primary consolidation of the clay deposit could be in the order of 550mm to 750 mm. Additional settlement due to secondary consolidation in the order of 120 to 150 mm could also be expected in 10 years after completion of construction.

In order to reduce post construction settlement, several alternatives including preloading/surcharging the foundation soils, the use of wick drains and/or lightweight fill (such as EPS), or reduction of the embankment height are available. A combination of some of the above methods may be required to limit the induced final stress on the foundation soils to less than the preconsolidation pressure. Design of the most cost-effective means of reducing post construction settlement is beyond the scope of this preliminary investigation. Additional field investigation such as CPT and laboratory testing will be required to better characterize the foundation clays (such as more accurate determination of the preconsolidation pressure with depth) during detailed design. Further investigation will also be required during detailed design for the road embankment beyond the 20 m zone adjacent to the abutments, where settlement will also take place. Detailed analyses will also be required to determine the scope of such measures and any associated instrumentation program.

## **10.3 Embankment Construction**

Embankment construction should be carried out in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002. Earth fill should consist of granular materials or Select Subgrade Material (SSM) in compliance with Special Provision 110F13. Clean, inorganic earth fill (in accordance with OPSS 212) may consist of clayey materials that could itself settle in the order of 40 mm. The use of cohesive earth fill is, therefore, not recommended for use within a 20 m zone immediately behind the abutments, but may be considered for use beyond the 20 m zone.

Granular materials or SSM should be used within the 20 m zone immediately behind the abutment wall.

Where earth fill embankments reach a height of 8 m or greater, mid-height berms should be incorporated. The berms should be 2 m wide and extend for the length through which the embankment height exceeds 8 m, and the berms should maintain a 2% positive drainage grade to shed surface run-off. This requirement of a 2 m wide berm for an 8 m high earth embankment is in place to address surficial stability and to provide access for post construction maintenance.

The approach embankment is considered stable against seismic activities at this site.

Earth fill embankment slopes must be provided with erosion protection in accordance with OPSS 572 and related special provision(s).

## 11 RETAINED SOIL SYSTEMS

Retained soil system (RSS) walls may be used at this site provided preloading / surcharging and/or other means of ground improvement is carried out as outlined in this preliminary report. RSS walls are not recommended without such measures to minimize post construction settlement. A conventional concrete abutment will be required for the contemplated design, but RSS could be used for wing walls. Given the presence of compressible foundation clays, it is considered that there is medium to high risk associated with using RSS walls at this site, unless lightweight fill (such as EPS) amongst other methods is used to limit the induced final stress on the foundation soils to less than the preconsolidation pressure, such that post construction settlement becomes negligible.

RSS walls should be specified to be “High Performance” and “High Appearance”. The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

### 11.1 Foundation

The levelling pad for an RSS wall should be centred on top of a mat of engineered fill that is itself resting on the existing compact fill or well compacted embankment fill. Where applicable, the engineered fill subgrade should be proof-rolled and be compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD) at  $\pm 2\%$  of its optimum moisture content. The engineered fill mat for the levelling pad should consist of OPSS Granular A compacted in accordance with the OPSS 501, Method A. The engineered fill mat should be at least 500 mm thick and should be at least twice as wide as the levelling pad.

The following parameters may be used for the design of the levelling pad:

- Factored geotechnical resistance at ULS of 320 kPa, and geotechnical resistance of 250 kPa at SLS on an engineered Granular A pad.

- Ultimate coefficient of friction between cast in-situ concrete levelling pad on Granular A is 0.7.

In addition to the requirements for the levelling pad, the main body of the RSS may be founded on the engineered earth or rock fill, or on the native, stiff silty clay to clay. All topsoil, organics and soft soils should be removed from the subgrade of the RSS wall prior to placement of the fill. The RSS subgrade should be proof-rolled and be compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD) at  $\pm 2\%$  of its optimum moisture content.

The following parameters may be used for the foundation design of an RSS wall:

- Factored geotechnical resistance of 225 kPa at ULS and geotechnical resistance of 150 kPa at SLS, founded on compact embankment fill above Elevation 150 m.
- Factored geotechnical resistance of 225 kPa at ULS and geotechnical resistance of 150 kPa at SLS, founded on native stiff silty clay to clay at or below approximate Elevation 150 m.
- Ultimate coefficient of friction between RSS mass and compact fill is 0.55.
- Ultimate coefficient of friction between RSS mass and native stiff clay is 0.45.

The entire block of reinforced earth must also be designed against various modes of failure including sliding and overturning.

## **11.2 Global Stability**

The global stability of the RSS is dependent on the characteristics of the embankment fill and the foundation soils, the geometry of the embankment and location of the RSS within the embankment.

RSS walls, if used, are likely to be as wing walls at the abutments. It is envisaged that the RSS will be founded on compact embankment fill, or native stiff clay.

Results of preliminary stability analyses yielded F.S. values not less than 1.3 indicating that global stability can be maintained for an assumed RSS configuration.

The actual design configuration must be checked for global stability during detailed design.

## **11.3 Internal Stability**

The internal stability of the RSS should be analyzed by the supplier/designer of the proprietary product selected for this site.

## **11.4 Settlement**

The settlement of a RSS wall founded on compacted embankment fill will depend on the thickness of the pad, the material used, the conditions of the subgrade and the quality of

construction. If RSS walls are to be used at this site, they should be built after preloading / surcharging amongst other means such that post construction settlement does not exceed 25 mm. Further assessment should be carried out during detailed design to better establish the likely range of post construction settlements.

## **12 BACKFILL TO ABUTMENTS**

In the case of 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. A NSSP is required to specify grading limits for the rock fill. The rock fill used as backfill to the abutment should 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 granular backfill to the abutment wall is used, 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.

All granular material should meet the specifications of Special Provision 110F13.

Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.06.

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

## **13 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. The amount of wall movement required for the development of active, passive and at-rest earth pressures 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:

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

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

$K$  = earth pressure coefficient (see table below)

$\gamma$  = unit weight of retained soil (see table below)

$h$  = 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 unfactored values are shown in the following table.

Wall 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 (modified) $\phi = 32^\circ; \gamma = 21.2 \text{ kN/m}^3$		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	0.48*	0.20	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	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 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 CHBDC, 2000.

## 14 SEISMIC CONSIDERATIONS

### 14.1 Seismic Design Parameters

The following seismic parameters are provided in Table A3.1.7 of the CHBDC for the Arnprior/Renfrew area:

- Velocity Related Seismic Zone: 2
- Zonal Velocity Ratio: 0.10
- Acceleration Related Seismic Zone: 4
- Zonal Acceleration Ratio: 0.20

The entire site area is underlain by very stiff to stiff clays, becoming firm to soft with depth, for a total thickness of between 40 m and 45 m, overlying a very dense, bouldery silty sand till underlain by bedrock. The Soil Profile Type at these locations is classified as Type III, which, according to Table 4.4.6.1 of the CHBDC, is associated with a Site Coefficient (also referred to as ground motion amplification factor) of 1.5.

Site specific seismic hazard calculations for this area were obtained from Earthquakes Canada. For the design level earthquake (1 in 475 year event), the Peak Horizontal Ground Acceleration (PHA) is 0.184g, and the Peak Horizontal Ground Velocity (PHV) is 0.091 m/sec. These values should be used for the seismic design of the bridge at this site.

Clause C4.6.4 of the CHBDC suggests that the value of  $k_h$  used in calculating the earth pressure coefficients for yielding structures is equivalent to  $0.5 \times$  Zonal Acceleration Ratio,  $A$ , (including the effects of site amplification), or 0.1g at this site, provided that allowance is made for an outward displacement of the abutment of up to  $250A$ , or 50 mm. The vertical acceleration factor,  $k_v$ , has been taken as 0.6 times  $k_h$ . This design assumption is based on conventional practice in seismic design. For non-yielding walls, the recommended  $k_h$  design value according to CHBDC is equivalent to  $1.5 \times A$ , or 0.3g at this site. The Woods method adopted in this report is based on elastic theory resulting in seismic earth pressure coefficients comparable to those outlined in the CHBDC.

### 14.2 Liquefaction Potential

Since the abutments are to be founded on bedrock using piles, there is no potential for soil liquefaction under the foundations.

The approach embankments will be founded on the native silty clay to clay, and are not considered to be in danger of undergoing liquefaction. Some toe failure may occur in the approach embankments but this is expected to be minor in nature and readily repairable.



### 14.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active ( $K_{AE}$ ) and passive ( $K_{PE}$ ) earth pressure coefficients that includes the effects of earthquake loading.

In calculating the active, passive and at rest earth pressure coefficients, the angle of friction,  $\delta$ , between the wall and backfill material is assumed to be  $0.5 \phi$ , the angle of internal friction of the backfill.

For the design of retaining walls, the seismic earth pressure coefficients shown in the following table may be used:

Wall Condition	Earth Pressure Coefficient (K) for Earthquake Loading						
	Height of Application From Base as % of Wall Height for Horizontal Surface	Granular A or Granular B Type II $\phi = 35^\circ, \delta = 17.5^\circ$		OPSS Granular B Type I $\phi = 32^\circ, \delta = 16^\circ$		Rock Fill $\phi = 42^\circ, \delta = 21^\circ$	
		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 ( $K_{AE}$ )	48%	0.39	1.0*	0.43	1.0*	0.30	0.55
Passive ( $K_{PE}$ )	25%	3.3	-	2.8	-	4.5	-
At Rest ( $K_{OE}$ )**	47%	0.79	-	0.84	-	0.70	-

\* Slope may undergo movement for short durations during seismic activities

\*\* After Woods

## 15 CONSTRUCTION CONCERNS

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

Potential construction concerns include, but are not necessarily limited to:

- additional investigation including CPT testing and laboratory testing should be carried out during detailed design, in order to arrive at a suitable scheme to accelerate and stabilize settlement under the approach fills prior to foundation installation and paving.
- confirming that the anticipated settlement has stabilized at the approaches before removing the surcharge fill and commencement of deep foundation installation.
- potential for encountering boulders and cobbles during pile driving.
- maintaining stability of the preloading and surcharge fill at all stages.



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# Point Load Test Results

**TABLE 1**  
**Bruce Street Underpass**  
**Point Load Test Results**

Depth		Is50	UCS (MPa)		
feet	Inches m				
BRU-1					
144	9	44.12	4.30	103.23	} <div>Total Rock Core Average 79 Minimum 66 Maximum 106 MPa  Run # 1 Average 79.18</div>
151	6	46.18	5.27	126.40	
152	2	46.38	4.48	107.44	
160	6	48.92	3.38	81.11	
161	1	49.10	2.81	67.42	
161	6	49.23	2.94	70.58	
162	4	49.48	4.43	106.39	
162	10	49.63	3.47	83.22	
164	4	50.09	2.77	66.36	

Note: Point load test at 44.12, 46.18 and 46.38 m were performed on boulders

Depth			Is50	UCS (MPa)		Total Rock Core		
feet	Inches	m						
BRU-2								
160	10	49.02	0.61	14.75		Average	Minimum	Maximum
161	1	49.10	6.10	146.42				
165	6	50.44	5.00	120.08		131	85	147
166	6	50.75	6.10	146.42				
167	7	51.08	6.06	145.36		1	128.51	
168	6	51.36	4.26	102.18				
169	7	51.69	6.10	146.42		2	133.15	
170	5	51.94	5.88	141.15				
171	2	52.17	6.14	147.47				
172	2	52.48	3.56	85.32				
173	6	52.88	6.06	145.36				

Note: Point load test at 49.02 and 49.10 m were performed on boulders

Depth			Is50	UCS (MPa)					
feet	Inches	m							
BRU-3									
148	5	45.24	14.18	340.24					
149	0	45.42	13.30	319.17					
159	10	48.72	3.03	72.68					
160	8	48.97	4.26	102.18					
161	8	49.28	4.43	106.39					
162	8	49.58	3.38	81.11					
163	9	49.91	6.19	148.52					
164	8	50.19	0.53	12.64					
165	5	50.42	3.03	72.68					
166	3	50.67	6.85	164.33					
167	9	51.13	4.04	96.91					
					Total Rock Core				
					Average	Minimum	Maximum		
					95	13	164	MPa	
					1	90.59			
					2	99.02			

Note: Point load test at 45.24 and 45.42 m were performed on granitic boulders  
Point load test at 50.19 m was performed at hidden joint

**Appendix A**

**Record of Borehole Sheets**

## SYMBOLS AND TERMS USED ON TEST HOLE LOGS

### 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 <sup>(1)</sup> "N" VALUE
Very Soft	Less than 10	Less than 2
Soft	10 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 TEST HOLE LOGS

SYMBOLS FOR	 Shelby Tube	A - Casing
SAMPLE TYPE	 SPT	 Grab/Auger sample
	 No Recovery	 Core

- MC – Moisture Content (% by Weight) as determined by sample

 Water Level


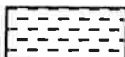
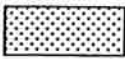


$C_{vane}$	Shear Strength Determination by Field Insitu Vane
$C_{pen}$	Shear Strength Determination by Pocket Penetrometer
$C_{lab}$	Shear Strength Determination using a Laboratory Vane Apparatus
$C_U$	Undrained Shear Strength determined by Unconfined Compression Test

- (1) SPT Standard Penetration Test – refers to the number of blows from a 63.5kg hammer falling through 0.76m to advance a 60 degree truncated cone 0.3m.

# 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	
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

## EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION		SYMBOLS	
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.		Bedrock (general)

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength (MPa) (psi)	Field Estimation of Hardness*	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.

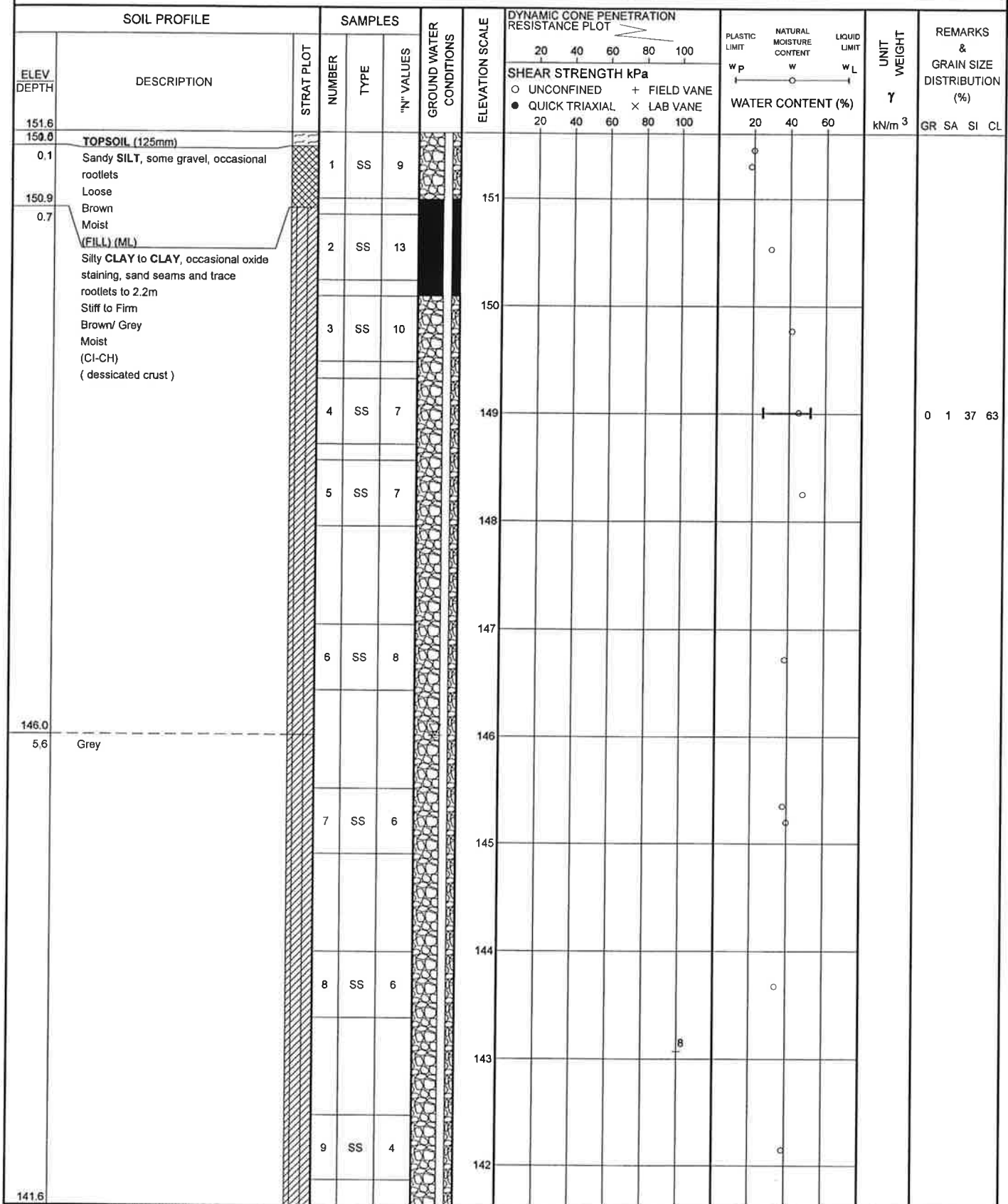
TERMS		Weak	Very Weak	Extremely Weak (Rock)
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	5.0 to 25.0	1.0 to 5.0	0.25 to 1.0
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	750 to 3,500	150 to 750	35 to 150
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.			
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen			
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.			

# RECORD OF BOREHOLE No BRU-1

1 OF 6

METRIC

G.W.P. 647-92-00 LOCATION N 5 039 748.0 E 291 406.0 (Bruce Street) ORIGINATED BY JL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Casing and Washboring, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 07.10.03 - 10.10.03 CHECKED BY SKP



Continued Next Page

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



# RECORD OF BOREHOLE No BRU-1

2 OF 6

METRIC

G.W.P. 647-92-00 LOCATION N 5 039 748.0 E 291 406.0 (Bruce Street) ORIGINATED BY JL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Casing and Washboring, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 07.10.03 - 10.10.03 CHECKED BY SKP

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 (%)
								○ UNCONFINED	+ FIELD VANE	×						
								● QUICK TRIAXIAL	×	LAB VANE						
10.0	Silty CLAY, to CLAY, sand seams Firm Grey (CI-CH)							20 40 60 80 100	20 40 60							
			10	SS	4		141							0 1 63 37		
			1	TW	PH		140							Consolidation Test		
			11	SS	3		139									
							138									
	inferred sand and silt layers and seams		12	SS	3		137							Blow back in augers		
							136									
			13	SS	3		135									
							134									
			14	SS	4		133							Commence casing and washboring		
							132									
131.6			15	SS	5											

Continued Next Page



+<sup>3</sup> ×<sup>3</sup>; Numbers refer to  
Sensitivity 20  
15 10 5  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BRU-1

3 OF 6

METRIC

G.W.P. 647-92-00 LOCATION N 5 039 748.0 E 291 406.0 (Bruce Street) ORIGINATED BY JL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Casing and Washboring, NO Coring COMPILED BY SS  
 DATUM Geodetic DATE 07.10.03 - 10.10.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			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				PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
								● QUICK TRIAXIAL	×	LAB VANE							+																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
20.0	Silty CLAY, to CLAY Firm to Soft Grey (CI-CH)		2	TW	PH		131																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BRU-1

4 OF 6

METRIC

G.W.P. 647-92-00 LOCATION N 5 039 748.0 E 291 406.0 (Bruce Street) ORIGINATED BY JL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Casing and Washboring, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 07.10.03 - 10.10.03 CHECKED BY SKP

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	
30.0	Silty <b>CLAY to CLAY</b> , with sand and silt layers and seams Firm Grey (CI-CH)		21	SS	5		121					
							120					
							119					
	move frequent sand layers		22	SS	23		118					
							117					
116.0							116					
35.6	Silty <b>SAND to Sandy SILT</b> , trace clay Dense Grey Wet		23	SS	32		115					
							114					
113.2							113					
38.4	Silty <b>CLAY to CLAY</b> , with sand and silt layers and seams Stiff Grey Moist to Wet		24	SS	15		112					

Continued Next Page

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

# RECORD OF BOREHOLE No BRU-1

5 OF 6

METRIC

G.W.P. 647-92-00 LOCATION N 5 039 748.0 E 291 406.0 (Bruce Street) ORIGINATED BY JL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Casing and Washboring, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 07.10.03 - 10.10.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			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								
							20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	WATER CONTENT (%)			
							20 40 60 80 100						20 40 60	
109.1	Silty <b>CLAY</b> to <b>CLAY</b> , with sand and silt layers and seams Stiff Grey							111						
								110						
42.5	Silty <b>SAND</b> , some gravel, frequent cobbles and boulders Very Dense Grey (TILL)		25	SS	110			109						
			1	RUN				108						
	no recovery from 44.42m to 45.62m		2	RUN				107						
			3	RUN				106						
	boulder from 46.13m to 46.76m							105						
	no recovery from 46.69m to 47.14m		4	RUN				104						
								103						
102.8					FI			102						
48.8	<b>MARBLE (BEDROCK)</b> Slightly to moderately weathered, grey, brown and white with subhorizontal black banding, strong. Subvertical joint at 49.8m. Multiple vertical and subvertical joints at 49.9m		5	RUN	1 0 1 >5									

RUN 5#  
TCR=98%,  
SCR=67%,  
RQD=66%,  
UCS=79MPa

RUN 5#  
TCR=98%,  
SCR=67%,  
RQD=66%,  
UCS=79MPa

Continued Next Page

+ 3 x 3

Numbers refer to Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

ONTMT4 7450BRU.GPJ 04/06/04

# RECORD OF BOREHOLE No BRU-1

6 OF 6

METRIC

G.W.P. 647-92-00 LOCATION N 5 039 748.0 E 291 406.0 (Bruce Street) ORIGINATED BY JL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Casing and Washboring, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 07.10.03 - 10.10.03 CHECKED BY SKP

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			20 40 60 80 100	SHEAR STRENGTH kPa							WATER CONTENT (%)			
									○ UNCONFINED	+ FIELD VANE						● QUICK TRIAXIAL	× LAB VANE		
101.4																			
50.2	END OF BOREHOLE AT 50.19m. BOREHOLE OPEN TO 45.72m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.						101												
	WATER LEVEL READINGS: DATE ELEVATION(m) 22/10/03 145.2 18/12/03 146.2 04/02/04 145.8 11/03/04 146.0																		

# RECORD OF BOREHOLE No BRU-2

1 OF 6

METRIC

G.W.P. 647-92-00 LOCATION N 5 039 748.7 E 291 445.8 (Bruce Street) ORIGINATED BY JL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Casing and Washboring, NO Coring COMPILED BY SS  
 DATUM Geodetic DATE 17.10.03 - 21.10.03 CHECKED BY SKP

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 ○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE						
150.3 0.0	SAND and GRAVEL Compact Brown Moist (FILL)  Silty CLAY to CLAY, sand seams Firm to Stiff Grey/ Brown (CI-CH) ( desiccated crust )		1	SS	16		150							0 1 46 53
149.6 0.7			2	SS	5		149							
			3	SS	10		148							
			4	SS	14		147							
			5	SS	10		146							
			6	SS	7		145							
144.5 5.8	grey		7	SS	6	144								
						143								
			8	SS	8	142								
						141								

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

METRIC

G.W.P. 647-92-00	LOCATION N 5 039 748.7 E 291 445.8 (Bruce Street)	ORIGINATED BY JL
HWY HWY 17	BOREHOLE TYPE Hollow Stem Augers, Casing and Washboring, NQ Coring	COMPILED BY SS
DATUM Geodetic	DATE 17.10.03 - 21.10.03	CHECKED BY SKP

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 (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE						
	Silty CLAY to CLAY, sand seams Firm Grey (CI-CH)		10	SS	7									0 2 61 37	
			11	SS	6										
			12	SS	5										
			13	SS	7										
			1	TW	PH										
	Very Soft to Soft		14	SS	1									Commence casing and washboring	

Continued Next Page

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

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BRU-2

3 OF 6

METRIC

G.W.P. 647-92-00 LOCATION N 5 039 748.7 E 291 445.8 (Bruce Street) ORIGINATED BY JL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Casing and Washboring, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 17.10.03 - 21.10.03 CHECKED BY SKP

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 (%)					
	Silty CLAY to CLAY with sand and silt layers and seams Very Soft to Soft Grey (CI-CH)													
			15	SS	2									
			16	SS	3									0 0 62 37
			2	TW	PH									

Continued Next Page

+<sup>3</sup> × 3 : Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE



## METRIC

[illegible]

(%) STRAIN AT FAILURE

ONTMT4 7450BRU.GPJ 04/06/04

# RECORD OF BOREHOLE No BRU-2

5 OF 6

METRIC

G.W.P. 647-92-00 LOCATION N 5 039 748.7 E 291 445.8 (Bruce Street) ORIGINATED BY JL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Casing and Washboring, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 17.10.03 - 21.10.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			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						
							WATER CONTENT (%)							
							20 40 60 80 100			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>		
							20 40 60 80 100			20 40 60				
107.4	Silty CLAY to CLAY						110							
							109							
							108							
42.9	Silty SAND, some gravel, frequent cobbles and boulders, occasional clayey silt seams/ partings Grey Very Dense Wet (TILL)		21	SS	26		107							
			1	WS			106							
			2	WS			105							
			3	WS			104							
			4	WS			103							
			5	WS			102							
							101							

Advancing NW casing and cleaning with NQ core barrel

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

METRIC

G.W.P.	647-92-00	LOCATION	N 5 039 748.7 E 291 445.8 (Bruce Street)	ORIGINATED BY	JL
HWY	HWY 17	BOREHOLE TYPE	Hollow Stem Augers, Casing and Washboring, NQ Coring	COMPILED BY	SS
DATUM	Geodetic	DATE	17.10.03 - 21.10.03	CHECKED BY	SKP

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No BRU-3

1 OF 6

METRIC

G.W.P. 647-92-00 LOCATION N 5 039 768.7 E 291 479.6 (Bruce Street) ORIGINATED BY JL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Casing and Washboring, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 27.10.03 - 29.10.03 CHECKED BY SKP

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			20 40 60 80 100	20 40 60 80 100					
149.3														
149.2	TOPSOIL (150mm)													
0.2	Black		1	SS	8		149							
	Silty CLAY to CLAY, trace rootlets, trace gravel to 1.4m Stiff to Very Stiff Brown Moist to Wet (CI-CH) (dissicated crust)		2	SS	15		148							
			3	SS	16		147							
			4	SS	10		146							0 0 40 60
			5	SS	6		145							
			6	SS	10		144							
			7	SS	10		143							
142.2	becoming grey		8	SS	8		142							
7.1	frequent sand and silt layers and seams		9	SS	11		141							Vane could not be turned
139.3							140							

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity 20  
15 Φ 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BRU-3

2 OF 6

METRIC

G.W.P. 647-92-00 LOCATION N 5 039 768.7 E 291 479.6 (Bruce Street) ORIGINATED BY JL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Casing and Washboring, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 27.10.03 - 29.10.03 CHECKED BY SKP

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												
							20	40	60	80	100	20	40	60	kN/m <sup>3</sup>	GR	SA	SI	CL	
10.0	Silty CLAY to CLAY Stiff Grey (CI-CH) ( dessicated crust )						139													
			10	SS	12		138													
							137													
			11	SS	12		136													
							135													
	with sand and silt layers and seams below this level		12	SS	12		134													
							133													
			13	SS	9		132													
							131													
	Firm		14	SS	7		130													
			15	SS	5															
129.3																				

Continued Next Page

+ 3 × 3  
Sensitivity

Numbers refer to

20  
15  
10

(%) STRAIN AT FAILURE

## METRIC

DATUM Geodetic DATE 27.10.03 - 29.10.03 CHECKED BY SKP

[illegible]

Continued Next Page

+ <sup>3</sup>, × <sup>3</sup>: Numbers refer to Sensitivity

ONTMT4 7450BRU.GPJ 04/06/04

# RECORD OF BOREHOLE No BRU-3

4 OF 6

METRIC

G.W.P. 647-92-00 LOCATION N 5 039 768.7 E 291 479.6 (Bruce Street) ORIGINATED BY JL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Casing and Washboring, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 27.10.03 - 29.10.03 CHECKED BY SKP

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>	
30.0	Silty CLAY to CLAY, with sand and silt layers and seams Stiff Grey (CI-CH)		18	SS	14		119					Vane could not be turned
115.6							118					
							117					
							116					
33.7	Silty SAND to Sandy SILT Very Dense Grey Wet		19	SS	97/ 279		115					0 73 27 (SI+CL)
							114					
113.2							113					
36.1	Silty CLAY to CLAY, with sand and silt layers and seams Hard Grey Wet		20	SS	50/ 127		112					
							111					
							110					
	frequent inferred cobbles		21	SS	50/ 076							
109.3												

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BRU-3

5 OF 6

METRIC

G.W.P. 647-92-00 LOCATION N 5 039 768.7 E 291 479.6 (Bruce Street) ORIGINATED BY JL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Casing and Washboring, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 27.10.03 - 29.10.03 CHECKED BY SKP

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 (%) GR SA SI CL
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>	
40.0	Silty <b>CLAY to CLAY</b> , with sand and silt layers and seams Grey						109					
	some gravel, occasional cobbles and boulders		1	WS			108					
105.2							107					
44.1	Silty <b>SAND</b> , some gravel, occasional cobbles and boulders Grey Wet (TILL)						106					
	boulder from 45.11m to 45.49m		2	WS			105					
			3	WS			104					
							103					
							102					
100.7							101					
48.6	<b>MARBLE (BEDROCK)</b> Slightly to moderately weathered, grey, brown and white with subhorizontal black banding, strong Multiple subvertical joints at 50.2m Subvertical joints at 49.2m and 50.0m Vertical joint at 50.9m		1	RUN			100					

RUN 1#  
TCR=100%,  
SCR=100%,  
RQD=79%,  
UCS=90MPa

Continued Next Page

+ 3 x 3



Numbers refer to  
Sensitivity

20  
15 5  
10  
(%) STRAIN AT FAILURE

ONTMT4 7450BRU.GPJ 04/06/04



## METRIC

SOIL PROFILE				SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			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			W <sub>P</sub>
98.1	MARBLE		2	RUN	2 3 0 5		99								
51.2	END OF BOREHOLE AT 51.18m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.  WATER LEVEL READINGS: DATE      ELEVATION(m) 18/12/2003    145.8 04/02/2004    145.2 11/03/2004    145.7														

## **Appendix B**

### **Laboratory Test Results**

## FIGURE B1

The graph displays the relationship between grain size and the percentage of material finer than that size. The top x-axis shows sieve sizes in inches (6", 4 1/4", 3", 1 1/2", 1", 3/4", 1/2", 3/8", 3/16", 4, 8, 10, 16, 30, 40, 50, 60, 100, 200). The bottom x-axis shows grain size in millimeters (100, 10, 1, 0.1, 0.01, 0.001, 0.0001). The y-axis represents the percentage of material finer than the specified grain size, ranging from 0 to 100.

Key data series include:

- Top Curve (Solid Circles):** Represents the finest gradation shown, with 100% finer than 200 mm and approximately 55% finer than 0.075 mm.
- Second Curve (Open Circles):** Represents a gradation with 100% finer than 100 mm and approximately 45% finer than 0.075 mm.
- Third Curve (Open Squares):** Represents a gradation with 100% finer than 60 mm and approximately 30% finer than 0.075 mm.
- Fourth Curve (Solid Triangles):** Represents a gradation with 100% finer than 40 mm and approximately 25% finer than 0.075 mm.
- Fifth Curve (Solid Crosses):** Represents a gradation with 100% finer than 30 mm and approximately 25% finer than 0.075 mm.

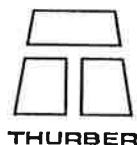
The graph illustrates how different sieve sizes correspond to specific grain size distributions, showing that as the sieve size decreases, the percentage of material finer than that size also decreases.

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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BRU-1	2.59	149.01
☒	BRU-1	10.97	140.63
▲	BRU-1	20.73	130.87
★	BRU-1	27.13	124.47
⊙	BRU-2	2.59	147.71

Date May 2004  
Project 647-92-00

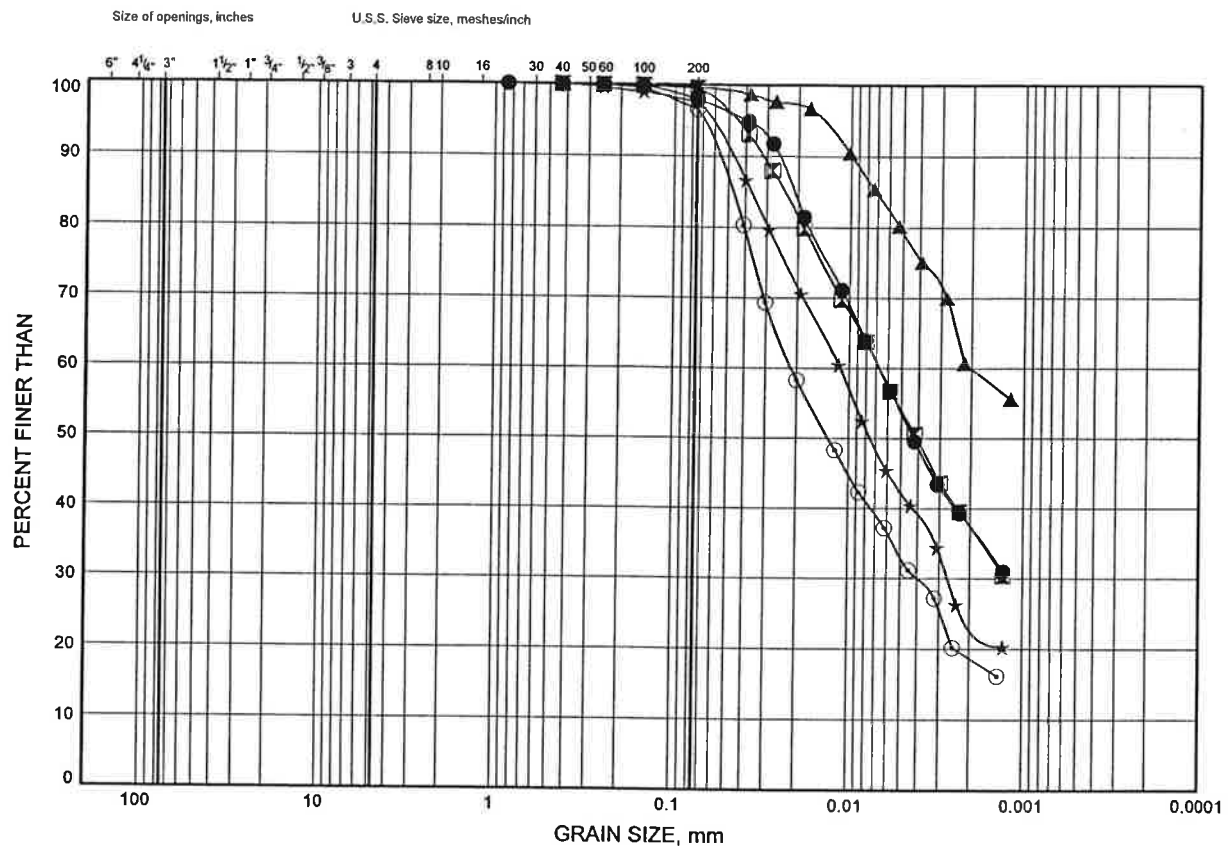
Prep'd SS  
Chkd. SP



# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B2

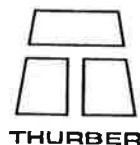
## SILTY CLAY TO CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BRU-2	10.97	139.33
⊠	BRU-2	24.69	125.61
▲	BRU-3	2.59	146.71
★	BRU-3	15.54	133.76
⊙	BRU-3	28.65	120.65

Date May 2004  
Project 647-92-00

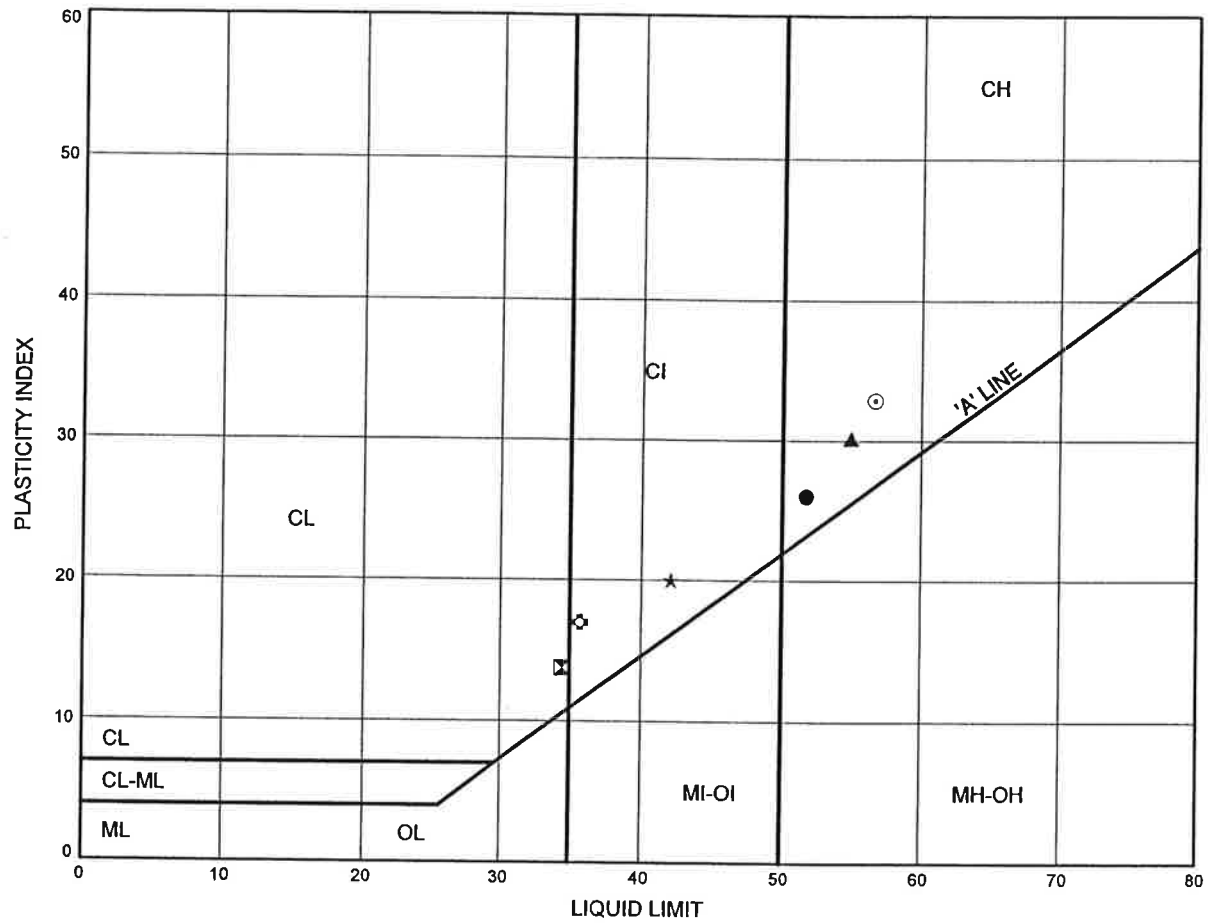


Prep'd SS  
Chkd. SP

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

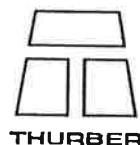
FIGURE B3

### SILTY CLAY TO CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BRU-1	2.59	149.01
⊠	BRU-1	10.97	140.63
▲	BRU-2	2.59	147.71
★	BRU-2	10.97	139.33
⊙	BRU-3	2.59	146.71
⊕	BRU-3	15.54	133.76

Date May 2004  
Project 647-92-00

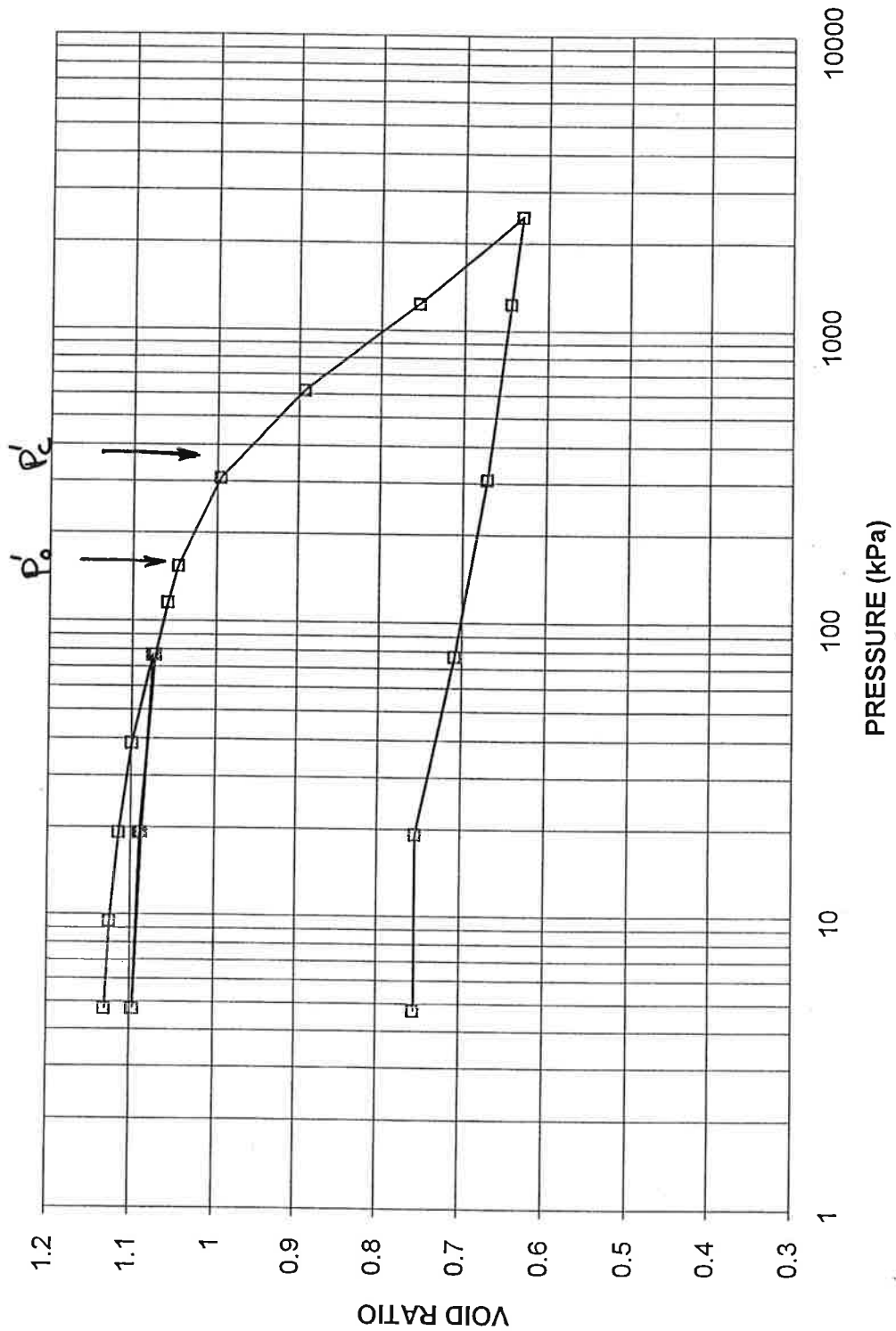


Prep'd SS  
Chkd. SP

CONSOLIDATION TEST  
VOID RATIO VS. LOG PRESSURE

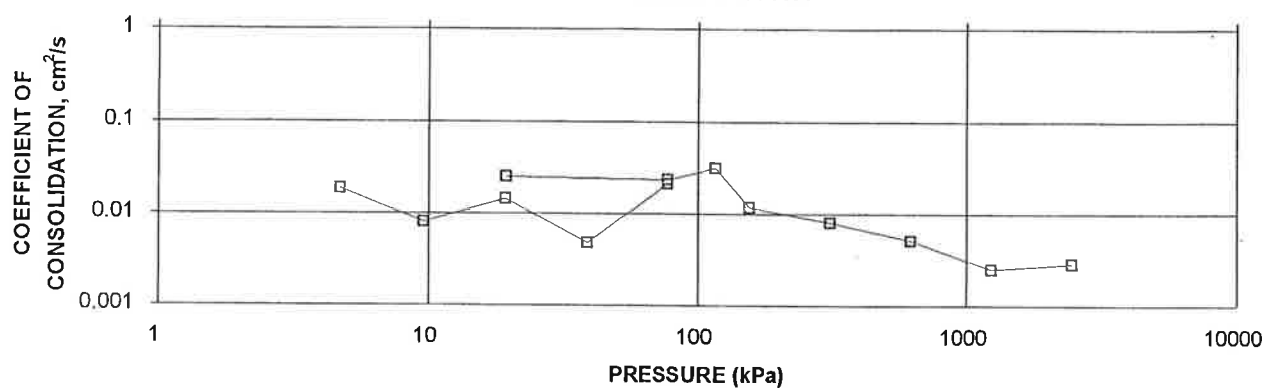
FIGURE B4

CONSOLIDATION TEST  
VOID RATIO vs PRESSURE  
BH BRU-1 SA TW1

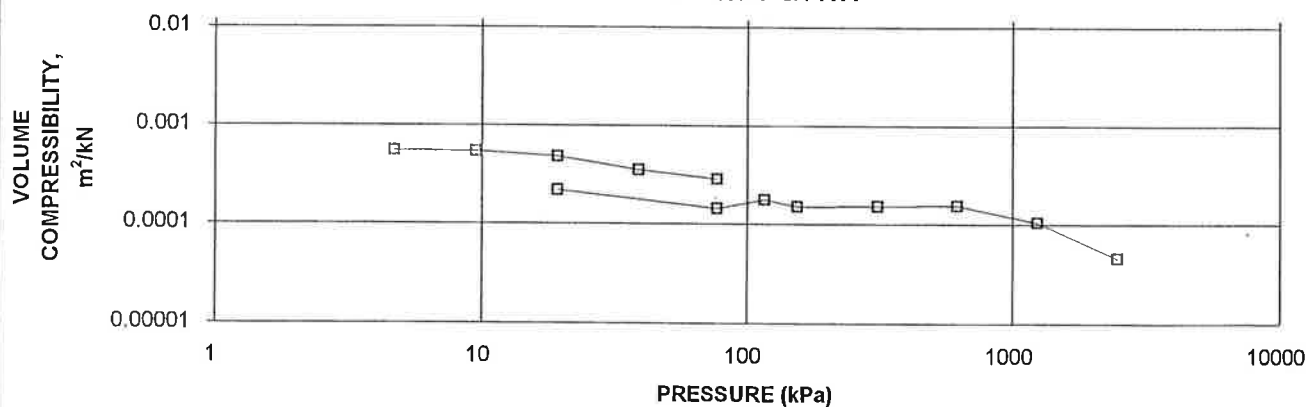


## OEDOMETER CONSOLIDATION SUMMARY

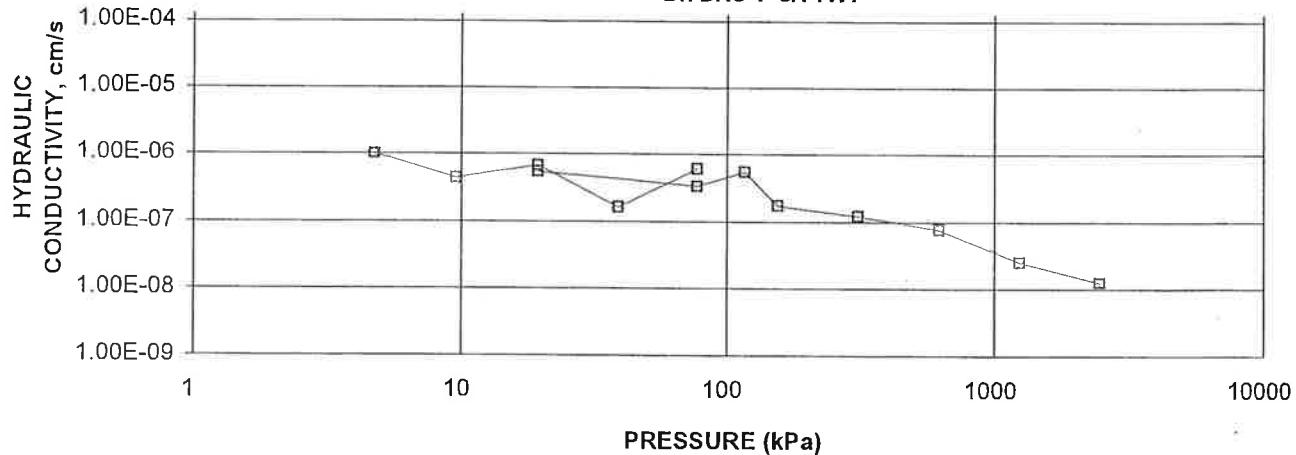
CONSOLIDATION TEST  
CV  $\text{cm}^2/\text{s}$  VS PRESSURE (kPa)  
BH BRU-1 SA TW1



CONSOLIDATION TEST  
MV  $\text{m}^2/\text{kN}$  vs PRESSURE (kPa)  
BH BRU-1 SA TW1



CONSOLIDATION TEST  
HYDRAULIC CONDUCTIVITY vs PRESSURE  
BH BRU-1 SA TW1



# OEDOMETER CONSOLIDATION SUMMARY

## SAMPLE IDENTIFICATION

Project Number	04-1116-011	Sample Number	TW1
Borehole Number	BRU-1	Sample Depth, m	11.6-12.2

## TEST CONDITIONS

Test Type	Standard	Load Duration, hr	(0.7-24)
Oedometer Number	6		
Date Started	1/30/2004		
Date Completed	2/12/2004		

## SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m <sup>3</sup>	18.03
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	12.81
Area, cm <sup>2</sup>	31.67	Specific Gravity, measured	2.79
Volume, cm <sup>3</sup>	60.17	Solids Height, cm	0.890
Water Content, %	40.70	Volume of Solids, cm <sup>3</sup>	28.18
Wet Mass, g	110.62	Volume of Voids, cm <sup>3</sup>	31.99
Dry Mass, g	78.62	Degree of Saturation, %	100.0

## 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	1.900	1.135	1.900				
4.75	1.895	1.130	1.898	41	1.86E-02	5.54E-04	1.01E-06
9.54	1.890	1.124	1.893	94	8.08E-03	5.49E-04	4.35E-07
19.25	1.881	1.114	1.886	53	1.42E-02	4.88E-04	6.80E-07
38.68	1.868	1.099	1.875	158	4.71E-03	3.52E-04	1.63E-07
77.38	1.847	1.076	1.858	34	2.15E-02	2.86E-04	6.02E-07
19.25	1.857	1.087	1.852				
4.75	1.866	1.097	1.862				
19.25	1.860	1.090	1.863	29	2.54E-02	2.18E-04	5.42E-07
77.38	1.844	1.072	1.852	31	2.35E-02	1.45E-04	3.33E-07
116.07	1.831	1.058	1.838	23	3.11E-02	1.77E-04	5.39E-07
154.68	1.820	1.045	1.826	60	1.18E-02	1.50E-04	1.73E-07
309.16	1.775	0.995	1.798	85	8.06E-03	1.53E-04	1.21E-07
618.45	1.684	0.893	1.730	124	5.11E-03	1.55E-04	7.76E-08
1237.35	1.560	0.753	1.622	225	2.48E-03	1.05E-04	2.56E-08
2472.95	1.451	0.631	1.506	171	2.81E-03	4.64E-05	1.28E-08
1237.35	1.463	0.644	1.457				
309.16	1.487	0.671	1.475				
77.38	1.520	0.708	1.504				
19.25	1.560	0.753	1.540				
4.75	1.561	0.754	1.561				

Notes:

k calculated using cv based on t<sub>90</sub> values.

## SAMPLE DIMENSIONS AND PROPERTIES - FINAL

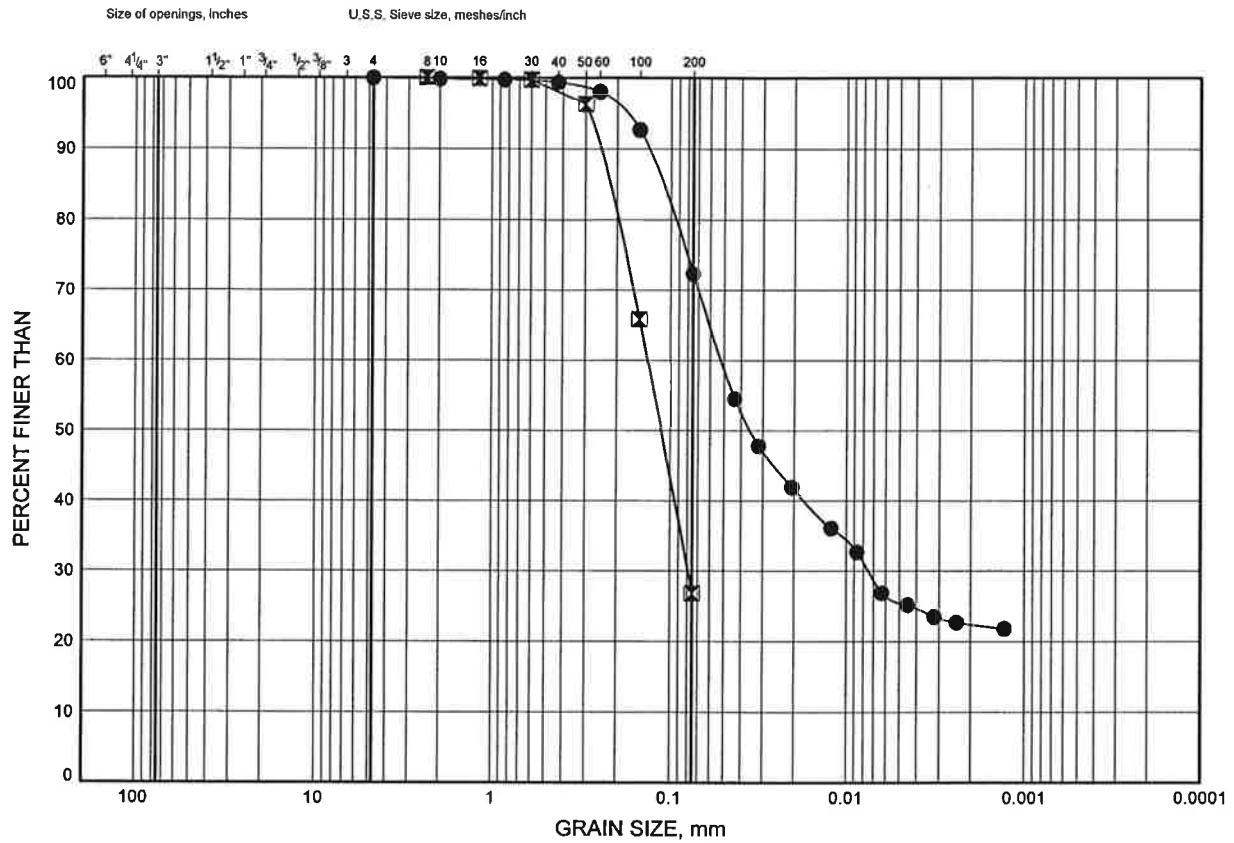
Sample Height, cm	1.56	Unit Weight, kN/m <sup>3</sup>	20.41
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	15.60
Area, cm <sup>2</sup>	31.67	Specific Gravity, measured	2.79
Volume, cm <sup>3</sup>	49.44	Solids Height, cm	0.890
Water Content, %	30.87	Volume of Solids, cm <sup>3</sup>	28.18
Wet Mass, g	102.89	Volume of Voids, cm <sup>3</sup>	21.26
Dry Mass, g	78.62		



# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B5

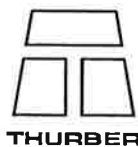
## SILTY SAND TO SANDY SILT



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BRU-2	36.88	113.42
⊠	BRU-3	33.83	115.47

Date June 2004  
Project 647-92-00



Prep'd SS  
Chkd. SP

## **Appendix C**

### **Foundation Comparison**

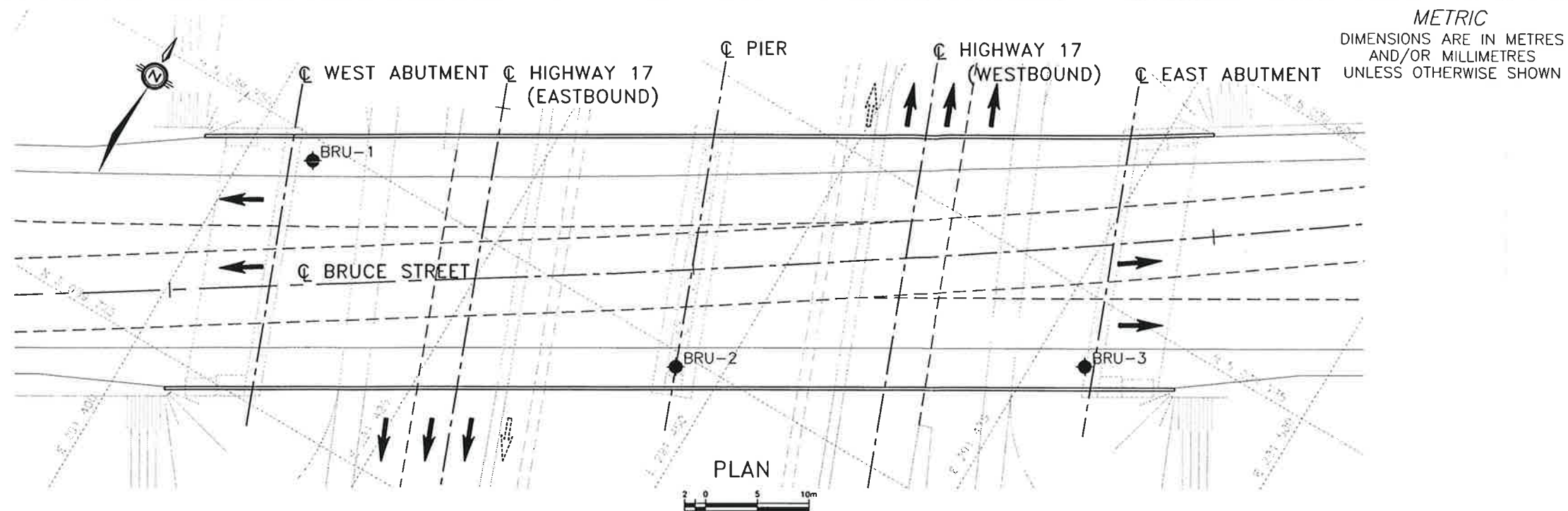
**COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT**

Foundation Element	Driven Piles	Footing on Bedrock	Footing on Engineered Fill	Augered Caisson
East Abutment Centre Pier West Abutment	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Relatively high pile capacity is available for end bearing on bedrock.</li> <li>ii. Minimal excavation required, if any, for foundation construction.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Downdrag on piles due to consolidation of the relatively deep clay deposit.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High values of geotechnical resistance are available on the bedrock.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Deep bedrock surface rendering the use of footing on bedrock impractical.</li> <li>ii. Footing on clay not desirable due to potentially large long term settlements resulting from footing load and new approach fill (at abutments).</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. None identified.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Deep bedrock surface rendering the use of footing on granular pad resting on bedrock impractical.</li> <li>ii. Footing on granular pad resting on clay not desirable due to long term settlements resulting from footing load and new approach fill (at abutments).</li> <li>iii. Lower geotechnical resistance than bedrock.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High values of geotechnical resistance are available on the bedrock.</li> <li>ii. Minimal excavation required, if any, for foundation construction.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Larger downdrag on caissons than on piles (large contact surface area) due to consolidation of the relatively deep clay deposit.</li> <li>ii. Not suitable for integral abutments.</li> <li>iii. Nominal rock socketting is required to enhance seating on bedrock.</li> </ul>

## **Appendix D**

### **Drawings**





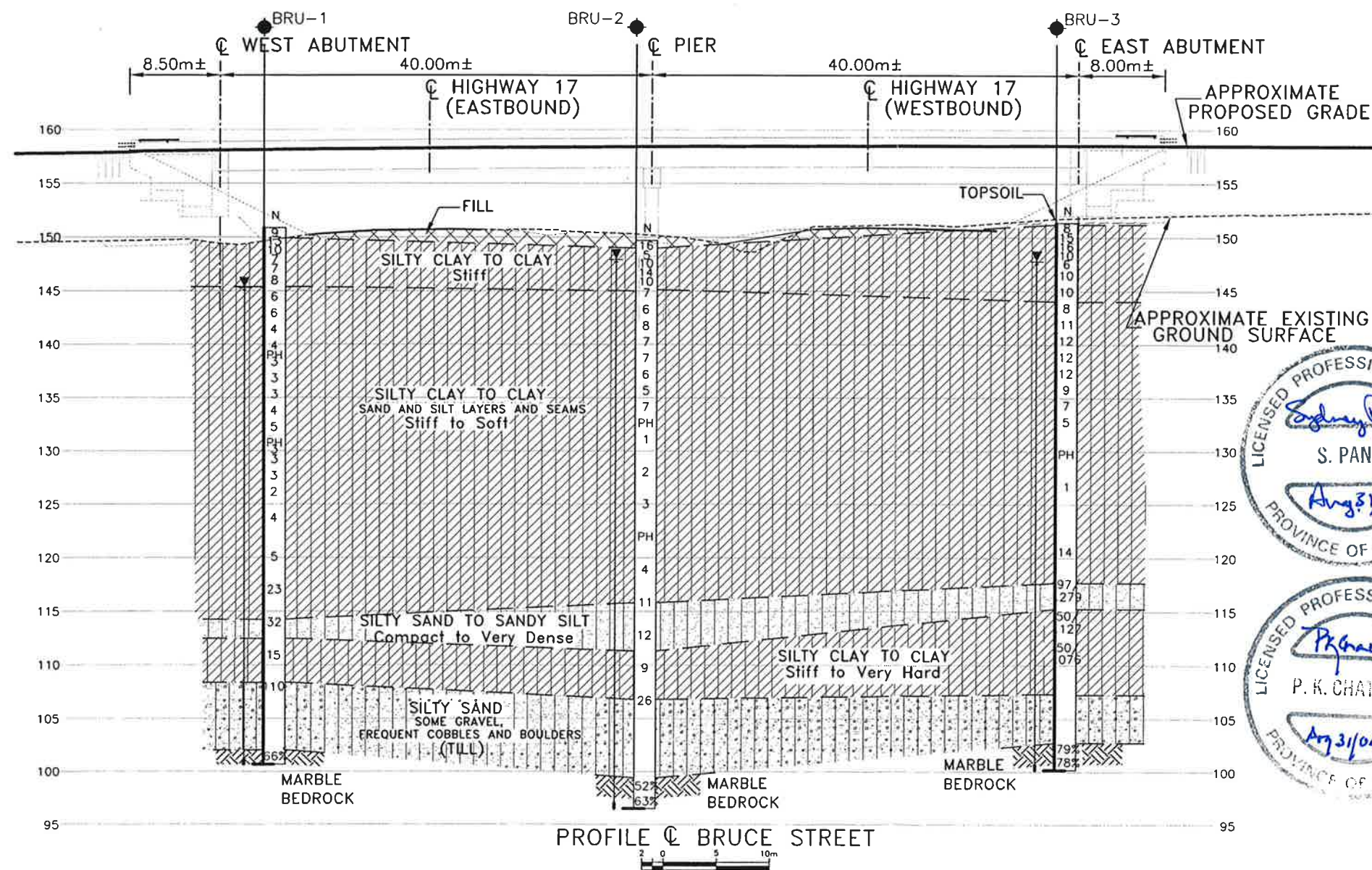
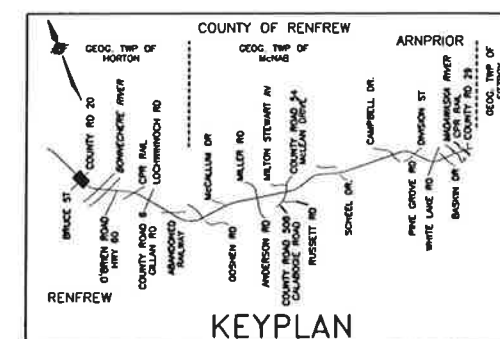
HWY.17  
GWP NO. 647-92-00

HIGHWAY 17 TWINNING  
BRUCE STREET UNDERPASS  
BOREHOLE LOCATIONS AND SOIL STRATA






SHEET



THURBER ENGINEERING LTD.



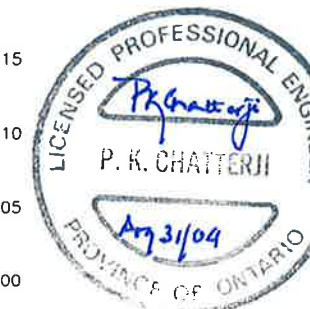
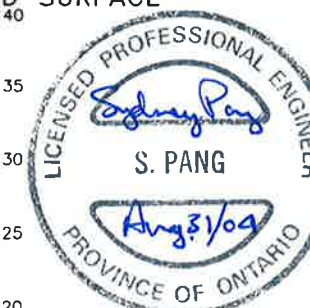
### LEGEND

- |   |   |
|---|---|
|  | 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 Time of Investigation             |
|  | Head Artesian Water                     |
|   | Piezometer                              |
| 90%   | Rock Quality Designation (RQD)          |
| A/R   | Auger Refusal                           |

[illegible]

— NOTE —

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

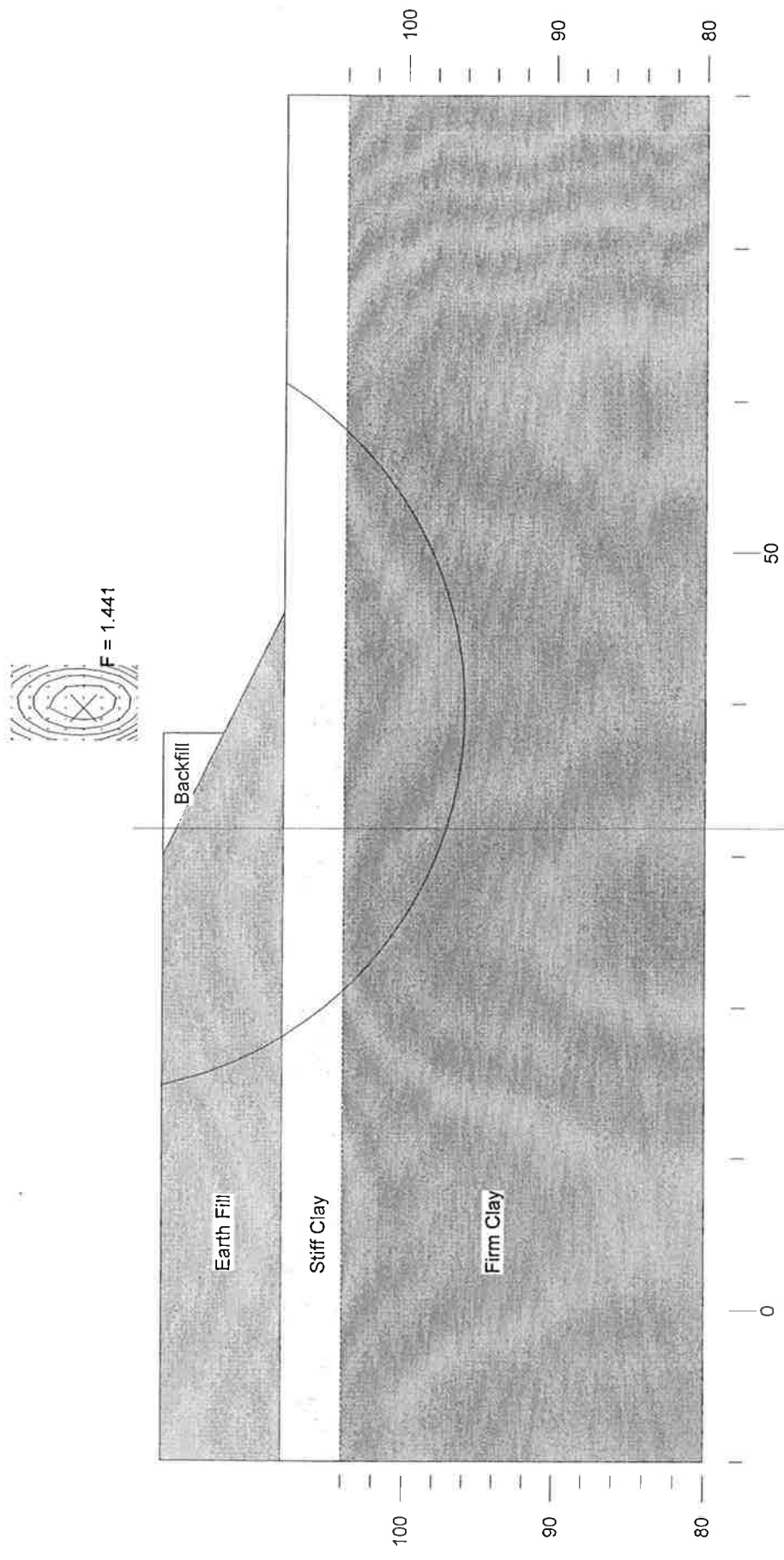
[illegible]

## **Appendix E**

### **Selected Stability Analyses Results**



	Gamma	C	Phi	Min	Piezo
	kN/m <sup>3</sup>	kPa	deg	c/p	Surf.
Abut. Backfill	21	0	30	0	0
Earth Fill	21	0	30	0	0
Stiff Clay	20	70	0	0	0
Firm Clay	20	40	0	0	1



	Gamma	C	Phi	Min	Piezo
	kN/m <sup>3</sup>	kPa	deg	c/p	Surf.
Abut. Backfill	21	0	30	0	0
Earth Fill	21	0	30	0	0
Stiff Clay	20	0	27	0	0
Firm Clay	20	0	25	0	1

