

**PRELIMINARY  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
COUNTY ROAD 508 UNDERPASS  
HIGHWAY 17 TWINNING  
ARNPRIOR TO RENFREW, ONTARIO  
G.W.P. 647-92-00, SITE NO. 29-412  
GEOCRES Number: 31F-133**

**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 County Road 508 over the widened Highway 17. During a previous preliminary investigation for the existing Highway 17, a borehole was drilled by the Ministry of Transportation (MTO) in the general vicinity of the site area, and the factual data from that investigation has been used as reference during the preparation this report.

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. A model of the subsurface conditions was developed based on the data obtained from the present investigation.

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

The following document is referenced in the preparation of this report:

- MTO Report titled "Preliminary Foundation Report for Structure Crossings of Revised Hwy. #17, from Antrim Westerly to Locheil Creek, Regional Municipality of Ottawa, Carleton and Renfrew County", District No. 9 (Ottawa), W.J. 69-F-86, W.P.'s 5-67 & 190-67, GEOCRES No. 31F-23, dated March 12, 1970 (Reference 1).

**2 SITE DESCRIPTION**

The site is located about 100 m east of the existing at grade intersection of Highway 17 and County Road 508, Township of McNab, County of Renfrew, Ontario (approximate mainline Station 18+040 on Highway 17). The Borehole Locations and Soil Strata drawing in Appendix D contains further details on the general site location.

The site is flat and large open areas exist on both sides of Highway 17. Vegetation is generally light adjacent to the existing Highway and consists mainly of grass and small shrubs with large

trees further beyond. An outcrop of bedrock, identified as belonging to the Canadian Shield, exists approximately 500 m west of the present at grade intersection.

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 consist 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 crystalline limestone of the Ordovician Period.

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

The site investigation and field testing for this project were carried out between September 18 and 23, 2003 and consisted of drilling and sampling a total of three boreholes to depths ranging from 19.1 m to 22.7 m. The boreholes were numbered 508-1, 508-2 and 508-3. The approximate locations of the boreholes are shown on the Borehole Locations and Soil Strata Drawing in Appendix D.

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 to drilling.

George Downing Estate Drilling Limited of Calumet, Quebec supplied a track mounted CME 75 drill rig and conducted the drilling, sampling and in-situ testing operations. Auger drilling techniques were used to advance the boreholes and samples were obtained at selected intervals 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 soil was measured in-situ by field vane tests using an MTO ‘N’ size vane. All three of the boreholes were extended about 3 m into bedrock by NQ size rotary coring techniques.

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 2 m to 3 m long slotted screen was installed at the bottom of the open boreholes. The sand screens surrounding the pipes were about 3.5 m long. Bentonite holeplug seals were placed just above the sand screen and just below ground surface in each installation. The remaining space in the boreholes was appropriately 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 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 clay retrieved from Borehole 508-1. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures in Appendix B.

Point load testing was carried out on selected rock cores retrieved from the three boreholes and these results are shown 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 these appendices and on the "Borehole Locations and Soil Strata" drawing 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 topsoil and a surficial sand layer followed by a major deposit of silty clay to clay. These deposits are further underlain by deposits of sand, gravel and silty sand till overlying crystalline limestone bedrock.

##### **5.1 Topsoil**

A surficial layer of topsoil varying in thickness from 150 mm to 250 mm was encountered at the borehole locations. Topsoil thicknesses may vary between borehole locations and in other areas of the site.

##### **5.2 Upper Sand**

North of existing Highway 17 (at Boreholes 508-2 and 508-3) the topsoil is underlain by a reddish brown deposit of sand encountered at depths ranging from 0.2 m to 0.3 m and extending to a depth of 0.7 m or to elevations ranging from Elev. 129.2 m to Elev. 129.4 m. Standard Penetration Tests conducted within this sand deposit gave 'N' values of 3 blows to 11 blows for 0.3 m penetration indicating a loose to compact relative density. The measured moisture content of samples from this stratum range from 7% to 9%.

### 5.3 Silty Clay to Clay

The topsoil and the upper sand layer are underlain by a silty clay to clay deposit across the site at depths ranging from 0.3 m to 0.7 m below ground surface. This deposit extends to depths ranging from 13.3 m to 18.5 m or from elevations varying from 111.6 m to 116.8 m.

Based on measured SPT 'N' values, the deposit can be divided into two zones. The upper zone or "crust" is about 5 m to 7 m thick, brown to grey-brown in colour, and extends to elevations of between 123 m and 125 m. The lower zone is weaker, grey in colour, and its thickness ranges from about 7 m to 12 m. Frequent silt seams and partings were observed within this lower zone.

Standard Penetration Tests conducted within the upper zone of the deposit gave 'N' values ranging typically from 22 blows to 8 blows per 0.3 m penetration indicating a very stiff to stiff consistency. Similar trends of consistency were inferred from pocket penetrometer values.

In the lower zone, Standard Penetration Tests gave 'N' values ranging from 1 blow to 6 blows per 0.3 m penetration indicating a firm to soft consistency. Field vane shear strengths in this lower zone were typically between 28 kPa and 60 kPa, with occasional values up to 80 kPa. The relatively higher measured in-situ shear strengths are attributed to the presence of silt seams and partings in the soil structure in the lower region of this clay deposit. Cobbles were encountered in this clay at a depth of 17.8 m (Elevation 112.3 m) in Borehole 508-3.

Grain size analyses conducted on samples retrieved from this unit are presented in Figures B1 and B2. These results show that the clay content of this soil ranges between 35% and 69%. Atterberg Limit Tests conducted on selected samples from this clay are presented in the plasticity chart shown as Figure B3. The results show that the silty clay to clay is typically of medium to high plasticity (group symbol CI to CH). The moisture contents of these tested samples are generally close to the Liquid Limit values. The measured moisture content of samples recovered from this deposit generally range from 32% to 58%.

A consolidation test was performed on an undisturbed sample of this clay obtained at about Elevation 121 m from Borehole 508-1. These results are presented in Figure B4. This test indicates a probable preconsolidation pressure ( $P'_c$ ) of about 220 kPa, which is about 130 kPa in excess of the existing effective overburden pressure ( $P'_o$ ), indicating that the clay at that location has an over-consolidation ratio (OCR) of 1.7. The initial void ratio  $e_0$  is approximately 1.4. The test results also show a compression index,  $C_c$ , value of about 0.74 and a recompression index,  $C_r$ , of about 0.066. The measured specific gravity of the sample was 2.78 and the unit weight was 16.9 kN/m<sup>3</sup>.

The coefficient of consolidation,  $C_v$ , value ranges from about 0.02 to 0.005 cm<sup>2</sup>/s within the range of stresses anticipated to be acting on the foundation soils.

The parameters obtained from this test is considered representative of the lower, lightly over-consolidated portion of the clay deposit.

#### **5.4 Lower Sand**

The silty clay deposit is underlain by a layer of grey sand in two boreholes (Boreholes 508-1 and 508-2). This deposit was encountered at depths of 13.3 m to 18 m below ground surface and it extends to depths of 14 m to 18.4 m or from elevations ranging from Elev. 111.5 to 116.1 m.

A selected sample from this layer was subjected to a grain size analysis and the grain size distribution curve is illustrated in Figure B5. The results show that the deposit is basically sand with trace clay and trace gravel. The measured moisture content of a sample obtained from this deposit was 23%.

#### **5.5 Gravel with Cobbles**

South of the existing Highway 17 (Borehole 508-1) the lower sand deposit is underlain by a deposit of gravel with some sand and occasional cobbles. Frequent cobbles were encountered in this stratum below Elev. 115.2 m. This grey deposit was encountered at a depth of 14 m (Elev. 116.1 m) below ground surface and it extends to a depth of 16.1 m (Elev. 114 m). This deposit is considered to have a dense relative density based on a recorded 'N' value of 38 blows for 0.3 m penetration. The measured moisture content of a sample from this layer is 5%.

#### **5.6 Silty Sand Till**

In one borehole (Borehole 508-2) the lower sand stratum is underlain by a 1.2 m thick deposit of silty sand till encountered at a depth of 18.4 m (Elev. 111.5 m) below ground surface. This stratum extends to a depth of 19.6 m (Elev. 110.3 m). A Standard Penetration Test in this material yielded an 'N' value of 29 blows for 0.3 m penetration indicating a compact relative density. The moisture content of a sample of this material was 8%. Glacial tills inherently contain boulders and cobbles.

#### **5.7 Bedrock**

The soils described above are underlain by crystalline limestone bedrock. Bedrock was proven by coring about 3 m of rock in all three boreholes. The table below summarizes the depth to bedrock and the elevations of the bedrock surface.

<b>Borehole Number</b>	<b>Depth to Bedrock (m)</b>	<b>Top of Bedrock Elevation (m)</b>
508-1	16.1	114.0
508-2	19.6	110.3
508-3	18.5	111.6



The crystalline limestone bedrock is very thinly to thinly bedded and generally in a fresh to slightly weathered state. Its colour is grey with dark grey and white sub-vertical banding.

The measured Total Core Recovery (TCR) in the bedrock was generally 97% to 100%, and the RQD values ranged from 40% to 88%. An RQD value of 40% was recorded in Run#1 of Borehole 508-1. Most RQD values ranged between 70% and 88% indicating a fair to good rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was low and generally ranged from 0 to 4 with occasionally higher values of 5 and 6 being recorded in Borehole 508-1. The joints were generally sub-vertical to vertical. The joint conditions were rough with occasional iron oxide staining, and were mostly tight with no infilling or secondary weathering material.

Point load tests were conducted on the rock cores at selected intervals. The inferred Unconfined Compressive Strength (UCS) of the rock cores ranges between 98 MPa and 157 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.

## 5.8 Water Levels

Standpipe piezometers were installed in Boreholes 508-1, 508-2 and 508-3 and their water levels were measured on three separate visits. These readings are presented in the table below.

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
508-1	October 22, 2003	2.2	127.9
	December 18, 2003	1.2	128.9
	February 4, 2004	1.9	128.2
	March 11, 2004	1.6	128.5
508-2	October 22, 2003	2.1	127.8
	December 18, 2003	1.1	128.8
	February 4, 2004	1.7	128.2
	March 11, 2004	1.5	128.4
508-3	October 22, 2003	2.4	127.7
	December 18, 2003	1.0	129.1
	February 4, 2004	1.1	129.0
	March 11, 2004	1.0	129.1

Based on these observations, local groundwater levels are anticipated to range between Elevations 128.4 m and 129.1 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 geotechnical 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 County Road 508 over the twinned Highway 17. The existing Highway 17 will become the eastbound lanes of the twinned Highway 17, and a new roadway will be constructed on the north side to form the westbound lanes.

The proposed underpass structure will be approximately 88 m long between abutment bearings and will have two spans. Each span will be approximately 44 m long and the new structure will be skewed at 20° to the abutment bearings. The front face of the abutment walls will be located as close as 4 m from the road shoulder.

It is understood that the proposed Highway 17 will remain at the same grade as the existing at-grade intersection. The proposed grade of the realigned County Road 508 at the north and south abutments will be at approximate Elevation 139 m. This corresponds to approach fill heights of about 9 m at both abutments.

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 of silty clay to clay and sand to sandy gravel with cobbles overlying crystalline limestone bedrock.

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

Location	Borehole Number	Elevations (m)	
		Existing Ground Surface	Top of Bedrock Surface
South Abutment	508-1	130.1	114.0*
Centre Pier	508-2	129.9	110.3*
North Abutment	508-3	130.1	111.6*

\* Proven by coring

### 7.1 Foundation Alternatives

This section presents discussions on available foundation alternatives, and provides 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 if the relatively long span lengths can be accommodated. The span lengths currently anticipated may not be suitable for a semi-integral abutment design.

Spread footings founded on the 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 load and post construction settlement at the approaches. 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.

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 9 m, a perched abutment design may be considered in conjunction with the piles. Alternatively, augered caissons socketted into bedrock may be used provided that the downdrag forces (relatively higher due to the larger contact surface) resulting from consolidation of the clay deposit can be accommodated in the design.

The foundation design recommendations given below are for preliminary design and planning purposes. Additional field investigation and analyses are required during detailed design to confirm the preliminary recommendations.

## 7.2 Driven Piles

Steel piles driven to bedrock may be used to provide foundation support at both abutments and the centre pier. Based on the borehole information, bedrock at this site is present at between 16 m and 19 m depth below existing ground surface. The following pile tip elevations are recommended for design purposes.

Foundation Element	Reference Boreholes	Estimated Pile Tip Elevation (m)
South Abutment	508-1	114 ±
Centre Pier	508-2	110 ±
North Abutment	508-3	111.5 ±

### 7.2.1 Axial Resistance

For designing HP 310 x 110 piles driven to bedrock, the following recommended pile capacities may be used:

- Factored geotechnical resistance at ULS of 2,000 kN per pile.

The SLS condition does not apply for piles founded on bedrock.

### 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 9 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 high end bearing resistance provided by the bedrock, the neutral plane may be assumed to be located at the bedrock surface. Settlement of the pile toe will be 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 zone of the silty clay to clay deposit (approximate Elevations 123 m to 112 m) where time-dependent consolidation settlement is anticipated. The following table presents estimates of the

factored and unfactored downdrag forces that can be experienced by HP 310 x 110 piles driven to bedrock at the abutments.

Location	Elevations of Time Dependent Compressible Zone* (m)	Estimated Downdrag Force (kN)	
		Unfactored	Factored**
North Abutment	123 to 112	500	625
South Abutment	123 to 117	270	340

\* It is assumed that the upper, over-consolidated, 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 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)

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

$$p_{ult} = 3 \cdot \gamma \cdot z \cdot K_p \quad (\text{kPa}) \text{ (from underside of pile cap to Elevation 130 m)}$$

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)

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

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

For both new approach fill and native silty clay to clay deposits, the following parameters are recommended:

where  $z$  = depth below underside of pile cap, m  
 $B$  = pile width, m

$n_h$	=	6,000 kPa/m (new approach fill compacted to at least 95% Standard Proctor density)
$\gamma$	=	20 kN/m <sup>3</sup>
$K_p$	=	3.0 (passive earth pressure coefficient)
$S_u$	=	undrained shear strength of native silty clay to clay
	=	100 kPa (above Elevation 126 m)
	=	60 kPa (between Elevations 126 m and 123 m)
	=	30 kPa (below Elevation 123 m)

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. The Contractor should be alerted of the potential presence of boulders and cobbles above bedrock. Based on existing borehole information, however, it is considered that there is a low risk of piles encountering refusal on boulders and cobbles.

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

Moderately sloping bedrock surface should be anticipated at this site. In order to enhance adequate seating of the pile tips into bedrock, it is recommended that the pile tips be reinforced with rock points such as the Titus "H" Bearing Pile Point, Rock Injector design, or equivalent.

The appropriate pile driving note to be shown on the contract drawing is "Piles to be fitted with rock points and driven into bedrock in accordance with 903S01 (Note 6 in Clause 3.3.3 of Section 3 Piles, the Ministry of Transportation, Ontario "Structural Manual").

#### 7.3 Augered Caissons

The abutments and the centre pier may be supported by augered caissons (drilled shafts) founded on bedrock. In order to found the caissons below the surficial, typically more fractured zone of the bedrock and to enhance caisson base contact with sound bedrock, it is recommended that the caissons be designed to be nominally socketted at least 500 mm into bedrock. The sockets should be formed below the low side of a sloping bedrock surface. The recommended design top of bedrock is the same as those presented in Section 7.



### 7.3.1 Axial Resistance

For a caisson nominally socketted 500 mm into bedrock, the axial capacity is assumed to be derived from end bearing only. It is recommended that a factored geotechnical resistance at ULS of 10,000 kPa be used for design.

The SLS condition will not govern for caissons founded on bedrock.

### 7.3.2 Downdrag

Downdrag forces could be induced on the caissons as a result of consolidation of the cohesive foundation soils under the loading of the 9 m high approach fills. The magnitude of the downdrag force depends on the contact area and the negative skin friction between the caisson surface and the surrounding soil. Reference should be made to the CHBDC (2000) Clauses 6.8.4 and C6.8.4 for downdrag calculations. Downdrag forces could be minimized provided pre-loading amongst other means of ground improvement is carried out at the approach fills prior to caisson installation.

Given the high end bearing resistance provided by the bedrock, the neutral plane may be assumed to be located at the bedrock surface. Settlement of the caisson toe will be 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 caisson, between the top and bottom surfaces of the silty clay to clay deposit. The unfactored downdrag load per caisson,  $Q$  (kN), can be calculated as follows:

$$Q = q_s \cdot C_s \cdot L_s$$

where  $q_s$  = ultimate unit negative skin friction (kPa)  
 $C_s$  = unit shaft surface area ( $m^2/m$ )  
 $L_s$  = length of caisson embedded in settling soil (m)

For the soil conditions encountered at the abutments, the values in the following table should be used for the ultimate unit skin friction. These values are calculated based on the alpha method, and are largely consistent with values obtained from the beta method except for the upper over-consolidated zone.

A load factor of 1.25 should be applied to obtain a factored downdrag force for pile design.

Elevation (m)	Soil Type	Ultimate Unit Negative Skin Friction (kPa)	
		Permanent Steel Casing	Concrete
Above 123	SILTY CLAY TO CLAY Very stiff to stiff	35	55
Below 123	SILTY CLAY TO CLAY Firm to soft	30	45

Further assessment of the skin friction values will be required during detailed design.

### **7.3.3 Lateral Resistance**

For lateral soil-pile interaction analysis, the recommendations, expressions and parameter values for coefficient of horizontal subgrade reaction ( $k_s$ ), ultimate lateral resistance ( $p_{ult}$ ) and group action reduction factors in Section 7.2.3 are applicable. The pile flange width,  $B$ , is to be replaced by the caisson diameter,  $D$ .

### **7.3.4 Caisson Installation**

Caisson installation should be in accordance with Special Provision No. 903S01.

Caisson installation at the abutment and pier locations would be carried out through silty clay to clay, and sand to sandy gravel with possible cobbles and boulders and socketted into bedrock. It is recommended that the caissons be constructed by using temporary steel liners to support the sidewalls and to allow hand cleaning and inspection of the rock bearing surface. A minimum caisson diameter of 900 mm, and as governed by applicable regulations, is required to allow down-the-hole hand cleaning and inspection.

It is recommended that an NSSP be included in the Contract Documents alerting the Contractor of the potential presence of boulders and cobbles.

The base of the caisson should be drilled at least 500 mm into the bedrock to remove weathered and highly fractured rock, and to mitigate the impact of a sloping rock surface. For moderately sloping bedrock surface anticipated at this site, it is recommended that the base of the caisson be drilled to 500 mm below the low side of the rock surface. Stepping of the caisson base is allowed in SP 903S01, but is likely not required for this site.

The caisson installation equipment should be capable of dislodging, handling and removing cobbles and boulders.

It is anticipated that a liner advanced into the bedrock will provide some seepage cut-off. Should water seepage be encountered, the caisson hole should be pumped dry prior to allowing personnel into the hole. The concrete should be placed using good tremie techniques.

## **7.4 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 or caisson 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, silty clay to clay deposit can be classified as a Type 2 soil above the groundwater table and a Type 3 soil below the groundwater table.

### **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 will be provided during detailed design as required.

#### **8.2.2 Rock Excavation**

Rock excavation will not be required at this site. Should the caisson option be adopted for foundation support, recommendations for socketting the caissons will be provided for detailed design as required.

## **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 and where water-bearing 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 programme 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**

The 9 m high approach embankments for this structure will be constructed over about 18 m of silty clay to clay overlying bedrock at the north approach. At the south approach, the foundation soils consist of about 13 m of silty clay to clay which is underlain by a 3 m thick layer of sand to sandy gravel overlying bedrock. At the north approach, the foundation soils consist of about 18 m of silty clay to clay. Earth fill or rock fill may be used to construct the embankments. For the rock fill option, an engineered fill core is 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 (granular core should extend at least 1.5 m beyond the footing perimeter).

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 or select subgrade material will have stable side slopes at inclinations not steeper than 2H : 1V.

Provided that preloading and surcharging is carried out at the approach fills as described in Section 10.2, stabilizing berms will not be required to maintain stability. 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 present selected stability analyses results. For an earth fill embankment, however, a mid-height berm will be required to address surficial stability as discussed in Section 10.3.

### **10.2 Settlement**

Settlement in the order to 40 to 50 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 (below approximate Elevation 123 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 (heavily over-consolidated zone above Elevation 123 m, and lightly over-consolidated zone below Elevation 123 m) under 9 m of fill could be in the order of 200mm to 225mm, while the settlement due to primary consolidation could be in the order of 500mm. Additional settlement due to secondary consolidation in the order of 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 most appropriate alternative for this site 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 and included in Appendix E. Earth fill should consist of granular materials or Select Subgrade Material (SSM) in compliance with Special Provision 110F13, "Amendment to OPSS 1010, March 1993". Clean, inorganic earth fill (in accordance with OPSS 212) may consist of clayey materials that could itself settle in the order of 50 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. SSM should be used within the 20 m zone immediately behind the abutment wall.

Where earth fill embankments are higher than 8 m, 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 is carried out as recommended in this report. RSS walls are not recommended without measures such as preloading to minimize post construction settlement. A conventional concrete abutment will be required for the contemplated design, but RSS could be used for wing walls and other retaining

structures that might be required. 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.

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**

It is recommended that the levelling pad for an RSS wall 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 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 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 native stiff silty clay to clay, compacted earth fill or rock fill. All topsoil, organics and soft soils should be removed from the subgrade of the RSS wall prior to placement of the fill.

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 at or above Elevation 130m.
- 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 130 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 the 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 existing compact fill or newly compacted embankment fill will depend on the thickness of the pad, the material used, the conditions of the subgrade and the quality of construction. Preliminary calculations indicated that settlements of RSS walls founded on the compact embankment fill and native stiff clay would be less than 25 mm provided that the approach fill subgrade is preloaded/surcharged.

## **12 BACKFILL TO ABUTMENTS**

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 "Amendment to OPSS 1010, March 1993".

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 PRESSURES

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*	0.2	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 or semi-integral abutments, material with a lower passive pressure coefficient (e.g. Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass. However, the use of Granular "B" Type I may be restricted if the approach embankment consist of rock fill.

The factors in the table above are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the 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 15 m and 20 m, with some sandy layers overlying

bedrock. The Soil Profile Type at these locations is classified as Type 1, 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 1.0.

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 x 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 x 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 above the groundwater level, 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 2000, retaining structures should be designed using active ( $K_{AE}$ ) and passive ( $K_{PE}$ ) earth pressure coefficients that include the effects of earthquake loading. The following geotechnical parameters were used to calculate the seismic earth pressures :

$\phi$  = angle of internal friction of backfill

$\delta$  = angle of internal friction between the wall and the backfill

The seismic earth pressure coefficients to be used in design at this site are shown in table below.

Wall Condition	Earth Pressure Coefficient (K) for Earthquake Loading						
	Height of Application From Base as Percentage of Wall Height	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 ( $K_{AE}$ )	40%	0.33	0.70	0.37	0.90*	0.26	0.40
Passive ( $K_{PE}$ )	33%	3.5	-	3.0	-	4.8	-
At Rest ( $K_{OE}$ )**	45%	0.67	-	0.72	-	0.58	

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

\*\* After Woods

#### 14.4 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:

- maintaining stability of the preloading and surcharge fill at all times
- confirming that settlement has stabilized at the approaches before removing the surcharge fill and commencement of deep foundation installation.
- potential for encountering boulders and cobbles during deep foundation installation



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# Point Load Test Results

**TABLE 1**  
**County Road 508**  
**Point Load Test Results**

Depth			Is50	UCS (MPa)	
feet	Inches	m			
508-1					
53	4	16.26	5.35	128.51	} <div>Total Rock Core</div> <div>Average Minimum Maximum MPa</div> <div>134 110 157</div> <div>Run # Average</div> <div>1 127.46</div> <div>2 146.42</div> <div>3 121.84</div>
54	7	16.64	5.27	126.40	
55	2	16.81	6.14	147.47	
56	8	17.27	6.19	148.52	
57	8	17.58	6.54	156.95	
59	3	18.06	5.53	132.72	
60	2	18.34	4.56	109.55	
61	1	18.62	5.62	134.83	
61	10	18.85	5.05	121.14	
508-2					
64	8	19.71	7.07	169.59	} <div>Total Rock Core</div> <div>Average Minimum Maximum MPa</div> <div>133 54 176</div> <div>Run # Average</div> <div>1 116.50</div> <div>2 149.37</div>
65	6	19.96	2.24	53.72	
66	10	20.37	3.69	88.48	
67	11	20.70	7.11	170.65	
68	8	20.93	4.17	100.07	
69	10	21.29	6.58	158.00	
70	11	21.62	5.35	128.51	
72	1	21.97	7.33	175.91	
72	10	22.20	5.71	136.94	
73	9	22.48	6.14	147.47	
508-3					
61	0	18.59	6.36	152.74	} <div>Total Rock Core</div> <div>Average Minimum Maximum MPa</div> <div>126 17 170</div> <div>Run # Average</div> <div>1 157.58</div> <div>2 98.84</div>
62	10	19.15	6.28	150.63	
63	11	19.48	6.54	156.95	
64	10	19.76	7.07	169.59	
65	7	19.99	6.58	158.00	
66	4	20.22	0.70	16.85	
67	0	20.42	2.72	65.31	
67	10	20.68	5.27	126.40	
68	9	20.96	3.91	93.75	
69	8	21.23	6.85	164.33	
70	5	21.46	5.27	126.40	

Note: Point load test at 20.22 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

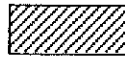

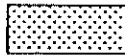


- (1) SPT Standard Penetration Test – refers to the number of blows from a 63.5kg hammer falling through 0.76m to advance a 60 degree truncated cone 0.3m.

# 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		Pt	Peat and other 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		Field Estimation of Hardness*
			(MPa)	(psi)	
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.
		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
		Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
		Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

TERMS	
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
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 508-1

1 OF 3

METRIC

G.W.P. 647-92-00 LOCATION N 5 033 313.1 E 304 380.2 ( County Road 508 ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 22.09.03 - 23.09.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W P		W		W L			
130.1								20 40 60 80 100									
0.0								20 40 60 80 100									
129.9	TOPSOIL (250mm)																
0.3	Silty <b>CLAY</b> to <b>CLAY</b> , trace sand seams, trace rootlets to 1.4m Very Stiff to Stiff Brown to Grey Moist to Wet (CI-CH)		1	SS	5												
			2	SS	20												
			3	SS	17												
			4	SS	18												
			5	SS	10												
			6	SS	7												
	becoming stiff to firm grey/ brown		7	SS	5												
	grey		8	SS	2												
	becoming firm to soft																
			1	TW	PH												

Continued Next Page

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

## METRIC

G.W.P.	647-92-00	LOCATION	N 5 033 313.1 E 304 380.2 ( County Road 508 )	ORIGINATED BY	SL
HWY	HWY 17	BOREHOLE TYPE	Hollow Stem Augers, NQ Coring	COMPILED BY	SS
DATUM	Geodetic	DATE	22.09.03 - 23.09.03	CHECKED BY	SKP

[illegible]

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

ONTMT4 7450-508.GPJ 30/04/04

# RECORD OF BOREHOLE No 508-1

3 OF 3

METRIC

G.W.P. 647-92-00 LOCATION N 5 033 313.1 E 304 380.2 ( County Road 508 ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 22.09.03 - 23.09.03 CHECKED BY SKP

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					W <sub>p</sub> W W <sub>L</sub> 20 40 60					
110.1																
20.0	WATER LEVEL READINGS: DATE ELEVATION (m) 22/10/2003 127.9 18/12/2003 128.9 04/02/2004 128.2 11/03/2004 128.5															

ONTMT4 7450-508.GPJ 30/04/04

# RECORD OF BOREHOLE No 508-2

1 OF 3

METRIC

G.W.P. 647-92-00 LOCATION N 5 033 342.1 E 304 413.2 ( County Road 508 ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 19.09.03 - 22.09.03 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			SHEAR STRENGTH kPa				
								20 40 60 80 100				
								20 40 60 80 100				
129.9												
0.0	TOPSOIL (250mm)											
129.7												
0.3	SAND		1	SS	11							
129.2	Compact Reddish Brown											
0.7	Moist Silty CLAY to CLAY Very Stiff to Stiff Brown Moist (CI-CH)		2	SS	22		129					
			3	SS	15		128					
	grey/ brown		4	SS	14		127					
	grey		5	SS	8		126					0 1 63 37
	becoming firm to stiff											
			6	SS	8		125					
	with silt seams/ partings											
			7	SS	4		124					
							123					
	occasional silt pockets											
			8	SS	3		122					
							121					
			9	SS	2		120					
119.9												

Continued Next Page

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

ONTMT4 7450-508 GPJ 30/04/04

# RECORD OF BOREHOLE No 508-2

2 OF 3

METRIC

G.W.P. 647-92-00 LOCATION N 5 033 342.1 E 304 413.2 ( County Road 508 ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 19.09.03 - 22.09.03 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							

Continued Next Page

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

20  
15  
10  
5  
0  
(%) STRAIN AT FAILURE

ONTMT4 7450-508.GPJ 30/04/04

## METRIC

[illegible]

# RECORD OF BOREHOLE No 508-3

1 OF 3

METRIC

G.W.P. 647-82-00 LOCATION N 5 033 370.4 E 304 450.6 ( County Road 508 ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 18.09.03 - 18.09.03 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			20	40	60	80	100					
130.1																	
130.0	TOPSOIL (150mm)																
0.2	SAND, some silt Loose Reddish Brown Moist		1	SS	3		130										
129.4																	
0.7	Silty CLAY to CLAY, trace sand Very Stiff to Stiff Brown to Grey Moist to Wet (Cl-CH)		2	SS	19		129										
			3	SS	14		128										
			4	SS	10		127										0 1 31 69
			5	SS	9		126										
							125										
	becoming stiff to firm with silt seams/ partings		6	SS	6		124										0 0 43 57
							123										
			7	SS	4		122										
							121										
			8	SS	3												
			9	SS	2												
120.1																	

Continued Next Page

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

20  
15  
10

(%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No 508-3

2 OF 3

METRIC

G.W.P. 647-92-00 LOCATION N 5 033 370.4 E 304 450.6 ( County Road 508 ) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 18.09.03 - 18.09.03 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			20 40 60 80 100	20 40 60 80 100					
10.0	Silty CLAY to CLAY, with silt seams/ partings Firm to Soft Grey Wet (CI-CH)		10	SS	3		120	5.56						
							119							
							118	3.71						
			11	SS	1		117							0 0 45 55
							116	4.33						
			12	SS	2		115							
							114	2.8						
	soft to firm		13	SS	4		113							
							112	3						
			14	SS	2		111							
	frequant cobbles below 17.8m auger refusal at 18.28m						110	2.57						
111.6					FI		109							
18.5	CRYSTALLINE LIMESTONE, (BEDROCK) Slightly weathered, very thinly bedded, grey and light brown with dark grey and white subvertical banding, occasional iron oxide staining at joints, very strong to strong Subvertical joints from 18.49m to 18.54, 18.64m to 18.75m, 18.8m to		1	RUN	4 2 2 2		108							RUN 1# TCR=100%, SCR=100%, RQD=77%, UCS=157MPa

Continued Next Page

+ 3, x 3 : Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

ONTMT4 7450-508.GPJ 30/04/04

## METRIC

[illegible]

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

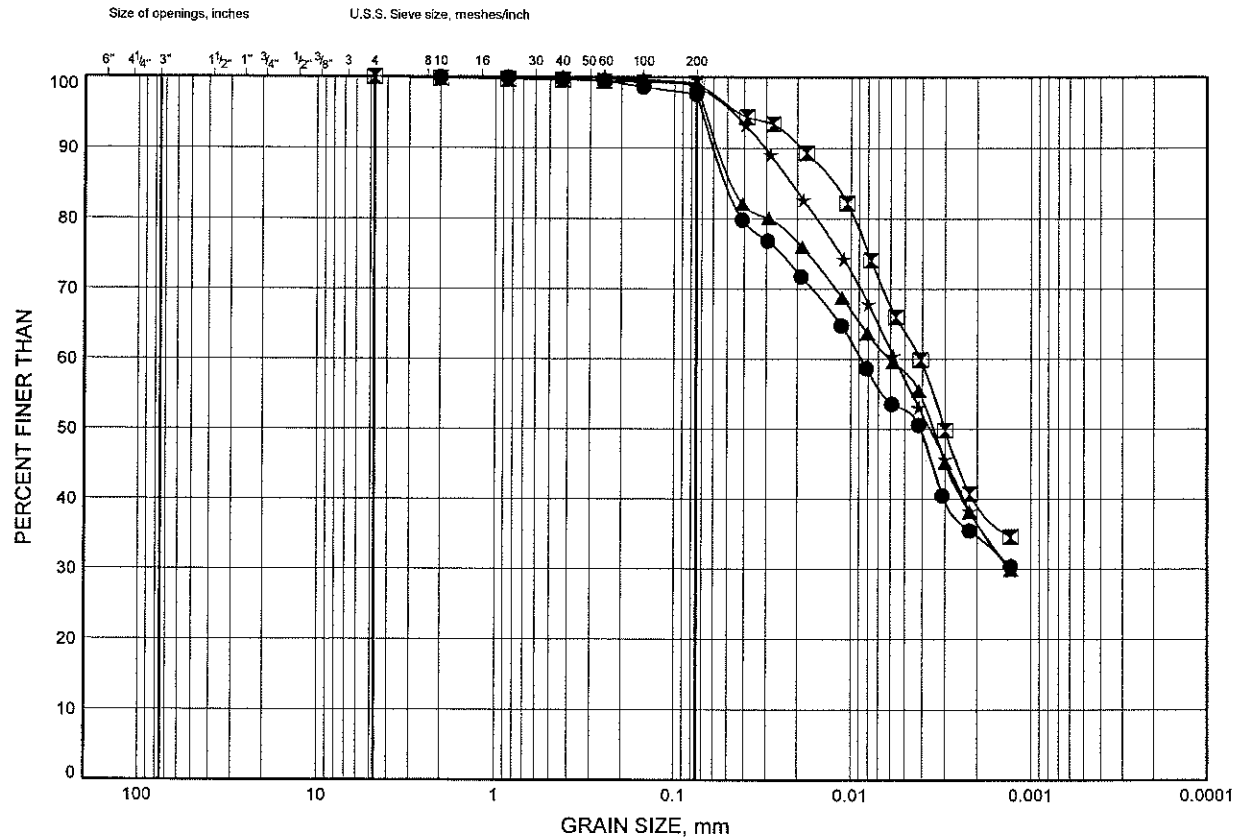
## **Appendix B**

### **Laboratory Test Results**

# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B1

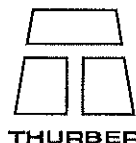
## SILTY CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	508-1	2.59	127.51
⊠	508-1	10.97	119.13
▲	508-2	3.35	126.55
★	508-2	12.50	117.40

Date March 2004  
Project 647-92-00



Prep'd SS  
Chkd. RAA

## FIGURE B2

Size of openings, inches

U.S.S. Sieve size, meshes/inch

PERCENT FINER THAN

GRAIN SIZE, mm

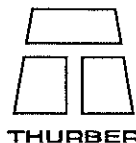
Grain Size (mm)	Percent Finer Than (Solid Circles)	Percent Finer Than (Solid Triangles)	Percent Finer Than (Open Squares)
100	100	100	100
40	100	100	100
20	100	100	100
15	100	98	98
10	98	95	95
7.5	95	92	92
5	92	88	88
3.75	88	82	82
2.5	82	72	72
1.5	72	58	58
0.85	68	55	55
0.425	59	46	46

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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	508-3	2.59	127.51
☒	508-3	4.88	125.22
▲	508-3	12.50	117.60

Date March 2004

Project 647-92-00.....



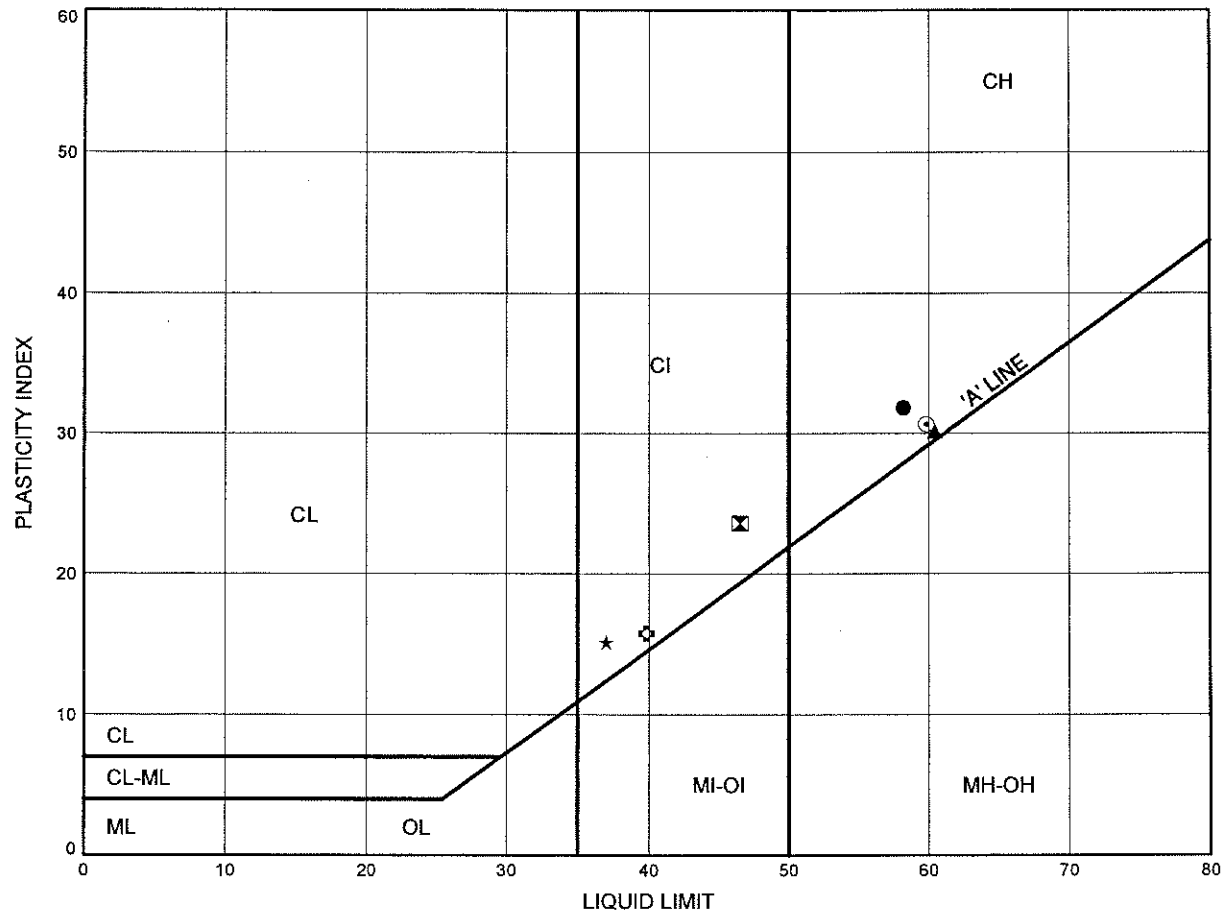
Prep'd .....SS.....

Chkd. .... RAA .....

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

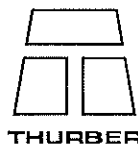
FIGURE B3

**SILTY CLAY**



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	508-1	2.59	127.51
⊠	508-1	10.97	119.13
▲	508-2	3.35	126.55
★	508-2	12.50	117.40
⊙	508-3	2.59	127.51
⊠	508-3	12.50	117.60

Date March 2004  
 Project 647-92-00

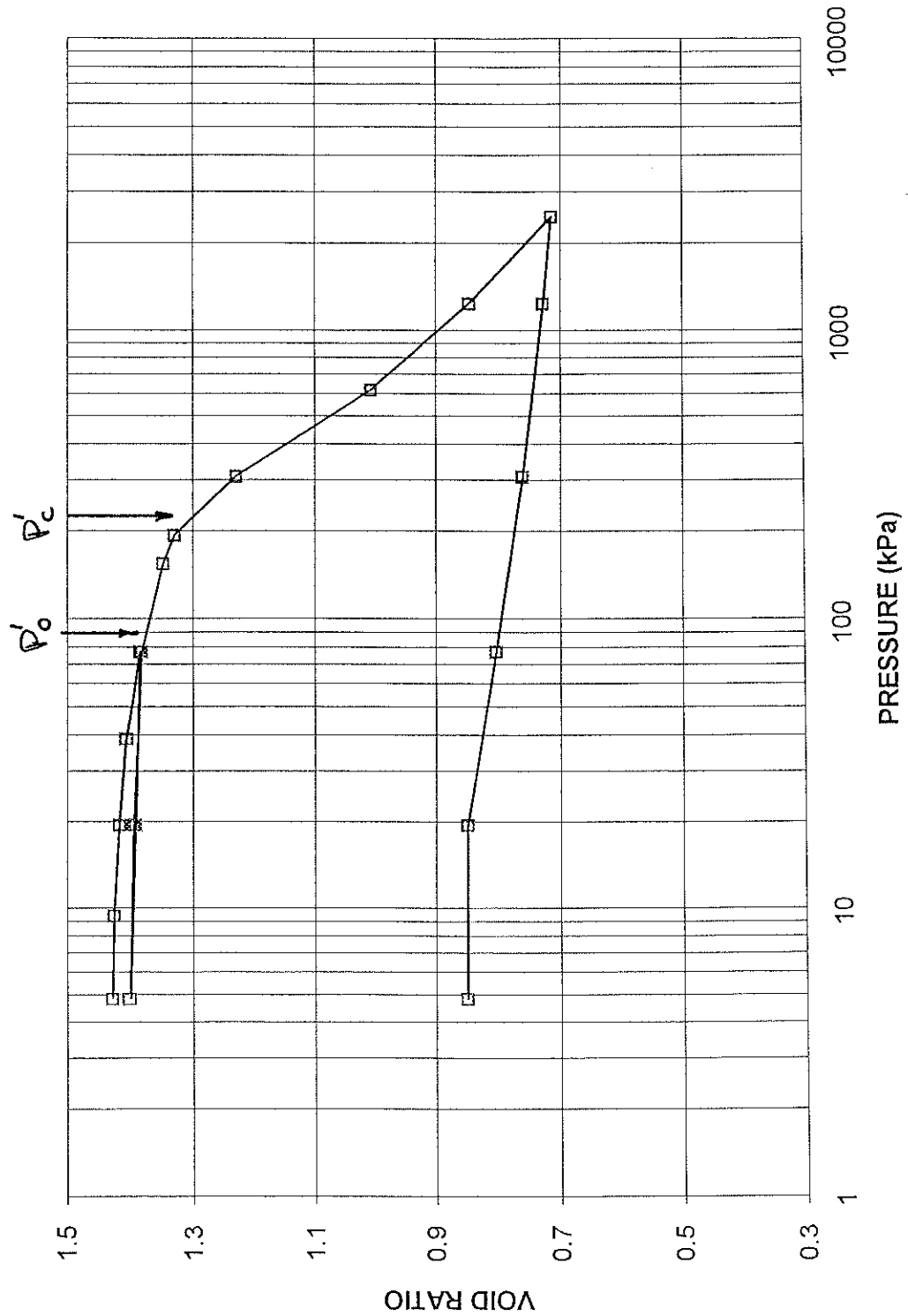


Prep'd SS  
 Chkd. RAA

CONSOLIDATION TEST  
VOID RATIO VS. LOG PRESSURE

FIGURE B4

CONSOLIDATION TEST  
VOID RATIO vs. PRESSURE  
BH 508-1 SA TW1



## OEDOMETER CONSOLIDATION SUMMARY

### SAMPLE IDENTIFICATION

Project Number	04-1116-011	Sample Number	TW1
Borehole Number	508-1	Sample Depth, m	11.6-12.2

### TEST CONDITIONS

Test Type	Standard	Load Duration, hr	(0.8-24)
Oedometer Number	7		
Date Started	1/30/2004		
Date Completed	2/11/2004		

### SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m <sup>3</sup>	16.93
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	11.23
Area, cm <sup>2</sup>	31.65	Specific Gravity, measured	2.78
Volume, cm <sup>3</sup>	60.13	Solids Height, cm	0.782
Water Content, %	50.76	Volume of Solids, cm <sup>3</sup>	24.76
Wet Mass, g	103.79	Volume of Voids, cm <sup>3</sup>	35.37
Dry Mass, g	68.84	Degree of Saturation, %	98.8

### 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	1.428	1.900				
4.83	1.900	1.428	1.900	13	5.89E-02	5.45E-05	3.14E-07
9.46	1.898	1.426	1.899	94	8.13E-03	1.93E-04	1.54E-07
19.51	1.892	1.418	1.895	85	8.95E-03	3.30E-04	2.89E-07
38.91	1.883	1.407	1.887	60	1.26E-02	2.31E-04	2.84E-07
77.57	1.867	1.386	1.875	60	1.24E-02	2.18E-04	2.65E-07
19.51	1.872	1.393	1.870				
4.83	1.879	1.402	1.876				
19.51	1.875	1.396	1.877	15	4.98E-02	1.43E-04	7.00E-07
77.57	1.864	1.382	1.870	31	2.39E-02	9.97E-05	2.34E-07
155.05	1.837	1.348	1.851	94	7.72E-03	1.83E-04	1.39E-07
193.95	1.823	1.330	1.830	586	1.21E-03	1.89E-04	2.25E-08
309.36	1.744	1.229	1.784	94	7.17E-03	3.60E-04	2.53E-07
618.55	1.570	1.007	1.657	304	1.91E-03	2.96E-04	5.56E-08
1237.39	1.444	0.846	1.507	211	2.28E-03	1.07E-04	2.40E-08
2475.52	1.341	0.714	1.393	146	2.82E-03	4.38E-05	1.21E-08
1237.39	1.351	0.727	1.346				
309.36	1.377	0.760	1.364				
77.57	1.410	0.802	1.394				
19.51	1.446	0.848	1.428				
4.83	1.447	0.849	1.447				

Notes:

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

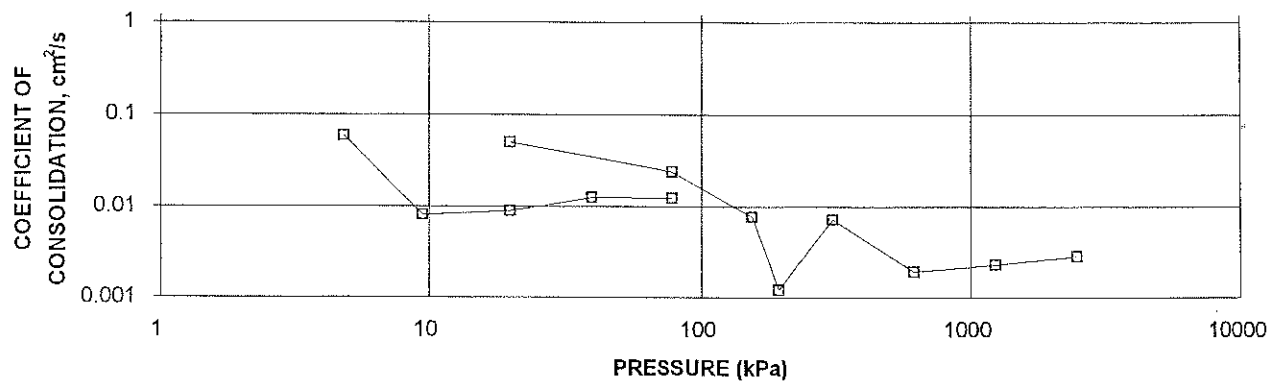
### SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.45	Unit Weight, kN/m <sup>3</sup>	19.52
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	14.74
Area, cm <sup>2</sup>	31.65	Specific Gravity, measured	2.78
Volume, cm <sup>3</sup>	45.80	Solids Height, cm	0.782
Water Content, %	32.44	Volume of Solids, cm <sup>3</sup>	24.76
Wet Mass, g	91.18	Volume of Voids, cm <sup>3</sup>	21.03
Dry Mass, g	68.84		

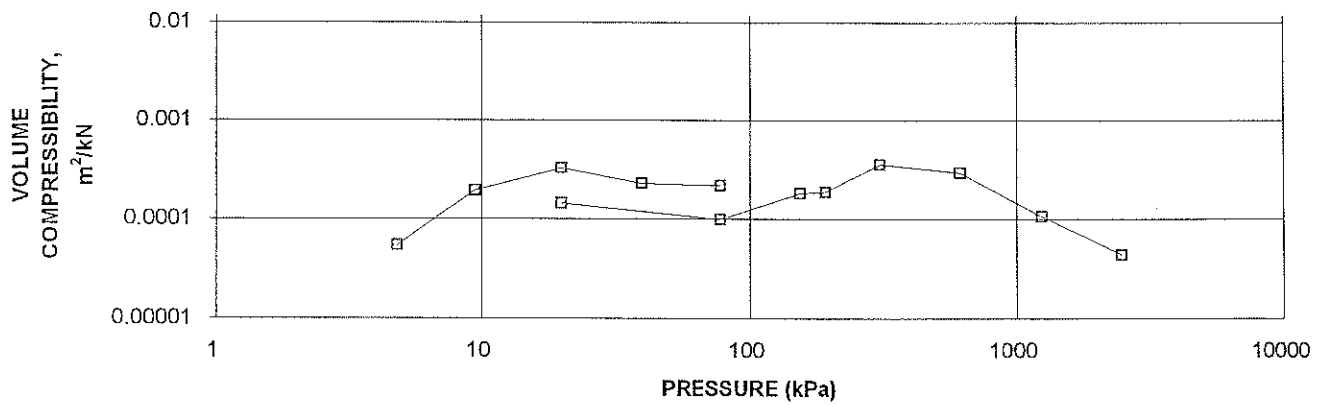


# OEDOMETER CONSOLIDATION SUMMARY

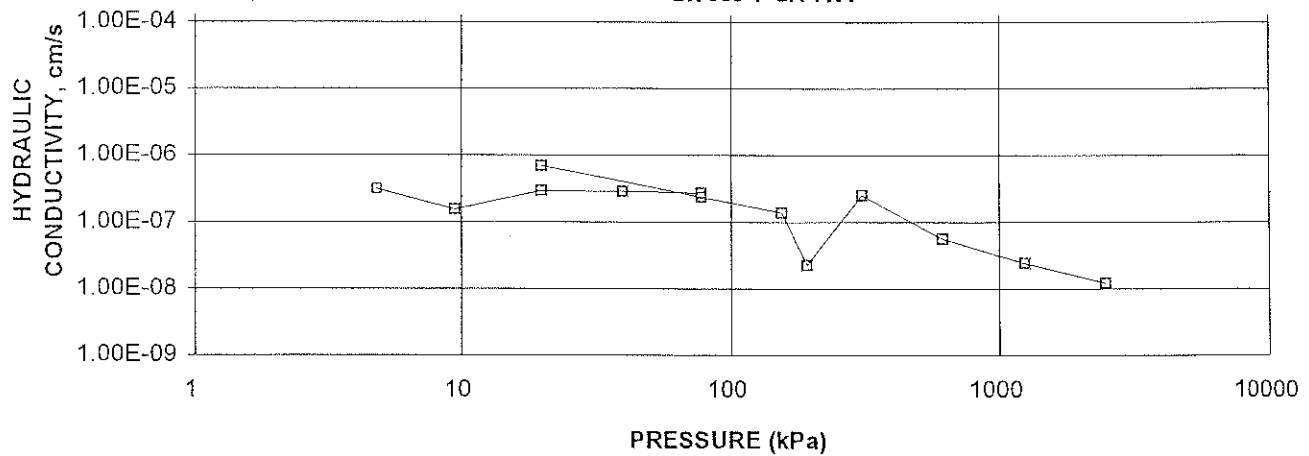
CONSOLIDATION TEST  
CV cm<sup>2</sup>/s VS PRESSURE (kPa)  
BH 508-1 SA TW1



CONSOLIDATION TEST  
MV m<sup>2</sup>/kN vs PRESSURE (kPa)  
BH 508-1 SA TW1



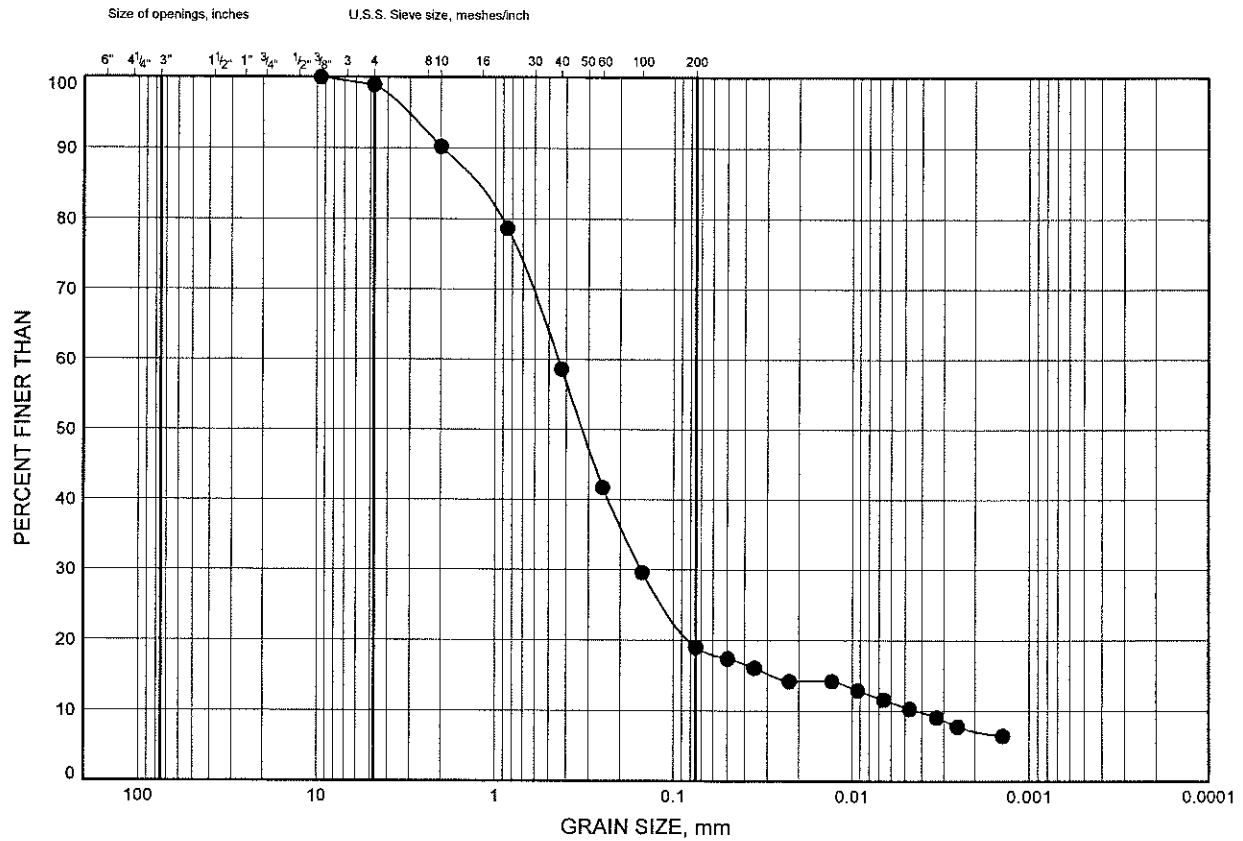
CONSOLIDATION TEST  
HYDRAULIC CONDUCTIVITY vs PRESSURE  
BH 508-1 SA TW1



# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B5

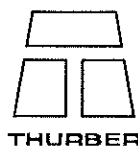
## SAND



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	508-1	13.87	116.23

Date April 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
North Abutment Centre Pier South 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. Nominal rock socketting is required to enhance seating on bedrock.</li> </ul>

## **Appendix D**

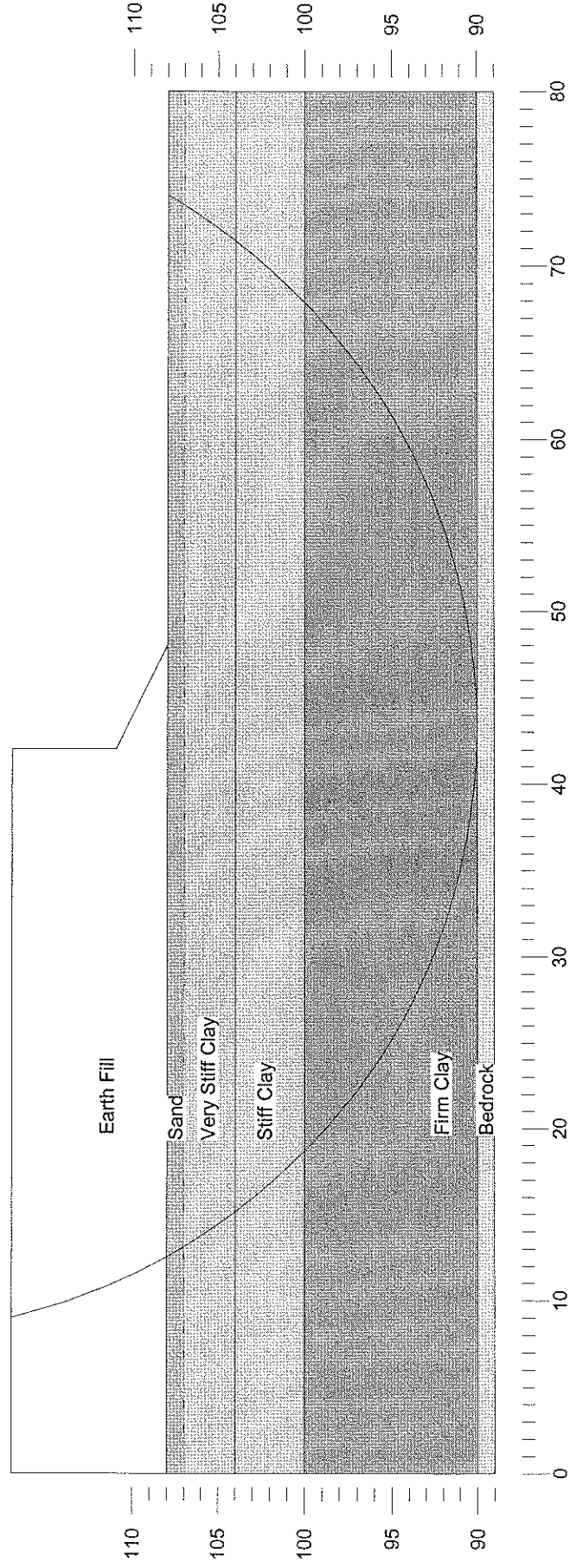
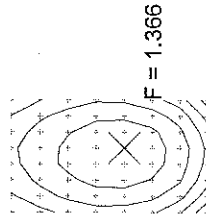
### **Figures**



## **Appendix E**

### **Selected Stability Analyses Results**

	Gamma	C	Phi	Min	Piezo
	kN/m <sup>3</sup>	kPa	deg	c/p	Surf.
Earth Fill	21	0	30	0	0
Sand	20	0	30	0	0
Very Stiff Clay	20	100	0	0	1
Stiff Clay	20	60	0	0	1
Firm Clay	20	40	0	0	1
Bedrock	(Infinitely Strong)				





	Gamma	C	Phi	Min	Piezo
	kN/m <sup>3</sup>	kPa	deg	c/p	Surf.
Earth Fill	21	0	30	0	0
Sand	20	0	30	0	0
Very Stiff Clay	20	0	30	0	1
Stiff Clay	20	0	28	0	1
Firm Clay	20	0	26	0	1
Bedrock	(Infinitely Strong)				

