

Geocres No:  
31F-137

**DRAFT PRELIMINARY  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
COUNTY ROAD 6 UNDERPASS  
HIGHWAY 17 TWINNING  
ARNPRIOR TO RENFREW, ONTARIO  
G.W.P. 647-92-00, SITE NO. 29-408**

**GEOCRES Number:** 31F-137

**Report to**

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May 28, 2004  
File: 19-3745-0

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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 6 over the widened Highway 17. During a previous preliminary investigation for the existing Highway 17, boreholes were drilled by the Ministry of Transportation (MTO) in the general vicinity of the existing Highway 17 alignment, and the factual data from that investigation has been used as general 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 stratigraphic profile, records of boreholes, laboratory test results and a written description of the subsurface conditions. A model of the subsurface condition 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, Region Municipality of Ottawa, Carleton and Renfrew County", District No. 9 (Ottawa), W.J. 69-F-86 and W.P.'s 5-67 & 190-67, GEOCRES No. 31F-23, dated March 12, 1970 (Reference 1).

**2 SITE DESCRIPTION**

The site is located approximately 3.4 km east of the Bonnechere River, near the existing at grade intersection of Highway 17 and County Road 6, Township of Horton, County of Renfrew

(approximate mainline Station 23+600 on Highway 17). The Borehole Locations and Soil Strata drawing in Appendix E contains further details on the general site location.

The site is flat and there are private dwellings on both sides of County Road 6 south of Highway 17. A small creek traverses the northwest quadrant of the intersection and crosses under County Road 6 via twin CSP culverts. This creek crosses under Highway 17 at about 35 m southeast of the present intersection. Immediately adjacent to the highway, vegetation is light and consists of grass and small shrubs with large trees further beyond.

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 silty clays to clayey silts 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 on October 14, 2003 and consisted of drilling and sampling three boreholes to depths ranging from 4.8 m to 9.1 m. The boreholes were numbered CR6-1, CR6-2 and CR6-3 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 to 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. Auger drilling techniques were used to advance the boreholes and samples were obtained at selected intervals in the overburden using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). The three boreholes were extended about 3 m to 3.8 m into bedrock by NQ size rotary coring techniques.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. A standpipe piezometer was installed in each borehole for monitoring of groundwater level. At this site, 19 mm diameter Schedule 40 PVC pipes with a 1.5 m long slotted screen were installed at the bottom of the open boreholes. The sand screens surrounding the pipes were about 2 m long. A bentonite holeplug seal was placed just above the sand screen in each borehole, and another seal was placed just below ground surface in Borehole CR6-1. 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 the results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures in Appendix B.

The results of point load tests carried out on rock cores retrieved from the boreholes are shown in Table 1.

#### **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 E. 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, fill and a sand deposit overlying bedrock.

##### **5.1 Topsoil**

A 125 mm thick layer of topsoil was encountered at the south abutment location (Borehole CR6-1). Topsoil thicknesses may vary between and beyond the boreholes.

##### **5.2 Sandy Gravel Fill**

Along the existing gravel shoulder of County Road 6, the three boreholes encountered a sandy gravel fill that extends from ground surface to depths ranging from 0.9 m to 1.8 m, or between Elevations 134.8 m and Elev. 136.6 m. The grain size distribution of a sample of this fill is shown on Figure B1.

SPT 'N' values measured in this fill ranged from 29 to 57 blows for 0.3 m penetration indicating a compact to very dense state. The measured moisture content of samples of this fill varies from 2% to 12%. High 'N' values measured at the base of this cohesionless fill in Boreholes CR6-2 and CR6-3 may represent the presence of boulders or rock shatter.

### 5.3 Silty Clay to Clayey Silt

At the south abutment location in Borehole CR6-1, the fill is underlain by a 0.5 m thick layer of grey silty clay to clayey silt, between Elevations 136.6 m and 136.1 m. This layer contains sand seams and occasional organic inclusions.

Results of a grain size analysis performed on a sample from this deposit are shown in Figure B2. This figure indicates that this soil contains 18% clay size particles.

An SPT 'N' value of 4 blows per 0.3 m penetration indicate a soft to firm consistency. The measured moisture content of a sample from this layer was 21%.

### 5.4 Sand and Silt

Below the cohesive layer in Borehole CR6-1, a grey deposit of sand and silt was encountered. This 3.9 m thick deposit was encountered at a depth of 1.4 m (Elevation 136.1 m) extending to a depth of 5.3 m (Elevation 132.2 m). The deposit contains occasional clay seams.

A sample of this deposit was subjected to a grain size distribution analysis and the results are shown on Figure B3.

SPT 'N' measured in this soil ranged from 15 to 5 blows per 0.3 m penetration, indicating a compact to loose state with depth. The measured moisture content of samples from this deposit ranged from 12% to 18%.

### 5.5 Bedrock

The soils described above are underlain by crystalline limestone 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)
CR6-1	5.3	132.2
CR6-2	1.6	136.2
CR6-3	1.8	134.8

The crystalline limestone bedrock is very thinly to thinly bedded and generally in a slightly to moderately weathered state. Its colour varies from grey to brown, with visible grey, white and black horizontal and sub-vertical banding.

The measured core recovery, TCR, in the bedrock was typically 98% to 100%, and the RQD values ranged from 13% (in Borehole CR6-2) to 70%, indicating very poor to fair rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally greater than 10, except in Runs 1 and 3 of Borehole CR6-1 where low values of between 0 and 3 were recorded. Numerous rough sub-vertical to vertical joints and multiple fracture zones were observed in the rock cores, which contributed to the relatively low RQD values. The joints were, however, mostly tight with no infilling or secondary weathering material.

Point load test were conducted on the rock cores at selected intervals. The inferred Unconfined Compressive Strength (UCS) of the rock cores ranges between 55 MPa and 153 MPa indicating that the intact rock is strong to very strong. A summary of the Point Load Test results is presented in Table 1 immediately following the text.

## 5.6 Water Levels

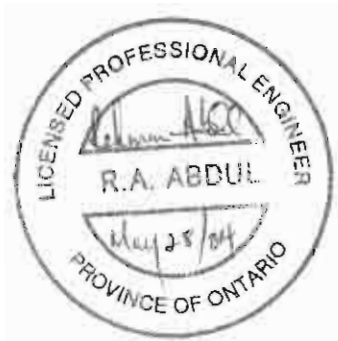
Standpipe piezometers were installed in the three boreholes and their water levels were measured during site visits made after the completion of drilling. The piezometers in Boreholes CR6-1 and CR6-3 were found to have been destroyed during the site visit on December 16, 2003. The readings are presented in the table below.

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
CR6-1	October 16, 2003	1.4	136.1
	October 22, 2003	1.2	136.3
	December 16, 2003	*	*
CR6-2	October 16, 2003	1.7	136.1
	October 22, 2003	1.6	136.2
	December 16, 2003	1.7	136.1
	February 4, 2004	1.7	136.1
	March 11, 2004	0.4	137.4
CR6-3	October 16, 2003	1.7	134.9
	October 22, 2003	1.6	135.0
	December 16, 2003	*	*

\* Piezometer installation destroyed

Based on these observations, local groundwater levels are anticipated to range from Elevations 134.9 m to 137.4 m. All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather 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 geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approach embankments for the proposed structure.

It is understood that Highway 17 will be twinned by constructing two west bound lanes (WBL) about 40 m north of the existing Highway 17 centreline. The existing Highway 17 will become the east bound lanes (EBL) of the twinned Highway 17. The new bridge will carry the realigned County Road 6 over the WBL and EBL, as well as new ramp lanes for the twinned Highway 17.

The preliminary general arrangement drawing indicates a two-span underpass structure. Each span will be approximately 40 m long and the new abutments will be skewed at approximately  $7.5^\circ$  to the highway. The proposed profile grade of County Road 6 is expected to be at about Elevations 145 m and 146 m at the south and north abutments, respectively. This corresponds to approach fill heights of about 8 m to 9 m.

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

**7 STRUCTURE FOUNDATIONS**

The proposed bridge is 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 three foundation elements generally consists of sandy gravel road embankment fill overlying crystalline limestone bedrock at the north abutment and centre pier, and thin surficial fill and native sand and silt overlying bedrock at the south abutment. Water level exists at a level as shallow as 0.4 m depth below existing ground surface.

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
South Abutment	CR6-1	137.5	132.2
Centre Pier	CR6-2	137.8	136.2
North Abutment	CR6-3	136.6	134.8

### 7.1 Foundation Alternatives

This section presents discussions on the feasible 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 if the proposed span lengths can be accommodated. For an integral abutment, all foundation piles will likely have to be socketted into bedrock. The span lengths currently anticipated may not be suitable for a semi-integral abutment design.

The presence of shallow bedrock renders it impractical to use driven piles at this site. Accordingly, the driven pile alternative was eliminated from further consideration.

Consideration may be given to using augered caissons at the two abutments. The caissons are required to be nominally socketted into bedrock to enhance adequate contact between the caisson and the bedrock over the entire base area.

At the centre pier, it is recommended that spread footings founded on bedrock be used for foundation support.

For the north abutment, it is recommended that spread footings founded on bedrock be used, although spread footings founded on an engineered fill pad resting on bedrock is a feasible means of foundation support.

At the south abutment, it may not be practical to use spread footings on bedrock due to the depth of excavation that will be required. Spread footings founded on the native compressible soils are not feasible due to the potentially large magnitude of settlement that may occur under the footing load. It is, however, feasible to found spread footings on engineered fill placed on the sand and silt deposit at about Elevation 136± m after the existing fill and silty clay is sub-excavated, and the new approach fill placed and compacted to induce immediate settlements.

## **7.2 Spread Footings on Bedrock**

### **7.2.1 General**

Based on the subsurface stratigraphy encountered at this site and the vertical alignment of the highway, this option should only be considered at the north abutment and centre pier. It is impractical to use footings on bedrock at the south abutment due to the potential excavation of more than 5 m below the existing ground surface.

The top surface of the bedrock should be stripped of all overburden and be cleaned. Inspection should be carried out to confirm that the bedrock conditions, as exposed at the founding level, are consistent with the design assumptions. All shattered and loosened rock fragments should be removed from the footprint of the footing and replaced with mass concrete fill, where necessary. Where bedrock is lower than anticipated, mass concrete may be placed to raise the subgrade to the design footing level.

Rock excavation is expensive and unnecessary excavation of bedrock should be avoided where practicable.

Where practicable, the underside of the concrete footing should be designed to lie approximately 200 mm above the local top of rock and the difference made up using mass concrete fill. Where necessary, the footing may be stepped down across the width of the structure to accommodate changes in the elevation of the top of bedrock. The recommended design top of rock is as follows:

#### *North Abutment*

The top of rock was encountered at Elevation 134.8 m at the borehole location. Further drilling investigation should be undertaken during the detailed design stage to establish the bedrock profile under the footprint of this foundation element.

#### *Centre Pier*

The top of rock was encountered at Elevation 136.2 m at the borehole location. Further drilling investigation should be undertaken during the detailed design stage to establish the bedrock profile under the footprint of this foundation element.

### 7.2.2 Bearing Resistance

Footings bearing on sound crystalline limestone bedrock encountered at this site may be designed for a factored geotechnical resistance of 3,000 kPa at Ultimate Limit States (ULS) for vertical, concentric loads. Effects of load inclination and eccentricity should be taken into account as illustrated in the CHBDC, 2000 Clause 6.7.3 and Clause 6.7.4. The design of the footing may be governed by other considerations such as sliding resistance or overturning moment.

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

The same value of resistance may be used where mass concrete of suitable strength is poured in neat contact with a clean, sound bedrock surface.

### 7.2.3 Horizontal Resistance of Footings

Resistance to lateral forces / sliding resistance between the concrete footing and the bedrock surface at the foundation elements should be evaluated in accordance with the CHBDC, 2000 assuming an ultimate coefficient of friction of 0.9. *High*

If the frictional component is insufficient to resist lateral forces, the horizontal resistance may be increased by dowelling the footing into the rock mass. Dowels are considered to be comparatively short steel bars that may be assumed to provide shear resistance.

The dowel may be considered as acting as a fully embedded pile in the rock and hence will fail when the ultimate lateral resistance of the rock is exceeded. Using a representative value for the unconfined compressive strength of the rock, an ultimate horizontal resistance of 1.3 MN may be assumed for a 50 mm steel dowel embedded 500 mm into the rock. The depth of embedment is measured below the bearing surface prepared to receive the concrete footing. *Assumed 1.3 MN resistance of dowel*

The shearing resistance of the selected dowel must be checked structurally. It is noted that the above design assumes that a drilled hole in bedrock has a diameter just large enough to accommodate the dowel and, as such, the compressive strength of the rock governs design.

### 7.3 Spread Footings on Engineered Fill

For a perched abutment design, spread footings founded on an engineered fill pad may be considered at the abutments. At the north abutment, the engineered fill pad should be placed directly on bedrock at approximate Elevation 134.8 m. At the south abutment, the engineered fill pad should be placed on the surface of the native compact sand and silt deposit at or below Elevation 136.1 m. All overburden materials including topsoil, fill and surficial silty clay should be removed from both the abutment areas. The engineered fill should consist of OPSS Granular "A" compacted to 100% of its Standard Proctor maximum dry density (SPMDD) at  $\pm 2\%$  of optimum moisture content (OPSS 501, Section

501.08.02 Method A) and conforming to the geometry illustrated in Figure D1 in Appendix D. It is recommended that the thickness of the fill pad be equal to or greater than the footing width, but should not be less than 2 m.

*North Abutment (Engineered Fill Directly on Bedrock)*

Provided a minimum footing width of 2 m is maintained, a footing founded on a compacted Granular A pad may be designed for the following values:

- Factored geotechnical resistance of 900 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 350 kPa at Serviceability Limit States (SLS)

*South Abutment (Engineered Fill on Native Sand and Silt)*

Provided a minimum footing width of 2 m is maintained, a footing founded on a compacted Granular A pad of thickness equal to or greater than the footing width may be designed for the following values:

- Factored geotechnical resistance of 450 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 300 kPa at Serviceability Limit States (SLS)

Should we  
substitute  
to increase  
resistance?

These values are for vertical, concentric loads only. Effects of load inclination and eccentricity should be taken into account as illustrated in the CHBDC, 2000 Clause 6.7.3 and Clause 6.7.4.

The SLS value quoted above corresponds to a settlement of up to 25 mm that is expected to be complete by the end of construction.

Resistance to lateral forces / sliding resistance between the concrete footing and compacted Granular A subgrade should be evaluated in accordance with the CHBDC, 2000 assuming an ultimate coefficient of friction of 0.7.

## 7.4 Augered Caissons

The south abutment 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 for spread footings in Section 7.1.

### 7.4.1 Axial Resistance

For a caisson nominally socketted for 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 3,000 kPa be used for design. This relatively low value reflects the poor to fair rock quality at shallow depths below top of rock, and intact rock strength in the 50 to 60 MPa range.

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

#### 7.4.2 Downdrag

Given the small thickness of the compressible cohesive soils (less than 1 m) present at the south abutment location, the cohesionless nature of the native sand (south abutment) and the existing cohesionless fill across the site, it is estimated that the magnitude of the downdrag force acting on a caisson will be relatively insignificant compared with its end bearing resistance, and will not affect the geotechnical resistance recommended in this report. Based on the existing subsurface information, downdrag is not considered to be a design issue at this site.

#### 7.4.3 Lateral Resistance

At this site, the caissons that may be used at the abutments would be relatively short, particularly at the north abutment, and would be nominally socketted into bedrock only to enhance base contact with sound rock. If fixity is required at the rock contact, the caissons should be drilled to a depth of at least twice its diameter into the rock.

At the north abutment, the soil lateral resistance may be derived from the new embankment fill and the existing sandy gravel fill. At the south abutment, the soil lateral resistance may be derived predominantly from the new embankment fill and the native sand and silt. For these soil conditions, the lateral resistance of the caissons may be calculated using values for the coefficient of horizontal subgrade reaction ( $k_s$ ) and ultimate lateral resistance ( $p_{ult}$ ) calculated from the following:

#### Engineered Granular Fill, Existing Sandy Gravel Fill and Native Sand and Silt

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

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

where  $z$  = depth below abutment base in metres

$D$  = caisson diameter in metres

$n_h$  = constant related to soil density

= 6,000 kPa/m (engineered fill compacted to at least 95% Standard Proctor density) ✓

= 4,500 kPa/m (native sand and silt below groundwater level) ✓

$\gamma$  = average soil unit weight = 20 kN/m<sup>3</sup> ✓

$K_p$  = passive earth pressure coefficient = 3.0 ✓

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

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

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

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

Where a caisson 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:

Caisson Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, $R$
4 D	1.00
1 D	0.50

where  $D$  is the diameter of the caisson, and spacing is measured centre to centre

Where a caisson 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:

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

Intermediate values may be obtained by interpolation.

Where the lateral resistance derived from the soils is insufficient to withstand the design lateral loads, consideration may be given to extending the rock sockets further into bedrock. The ultimate passive force that can be mobilized by the embedded portion of a socket within rock is constant with depth and is given by:

$$P_p = 6 \cdot c \cdot D \cdot L$$



where

c	=	1,500 kPa (equivalent Mohr-Coulomb cohesion based on Hoek and Brown rock mass classification)
L	=	depth of socket in rock, m

#### **7.4.4 Caisson Installation**

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

An engineered fill platform, which may constitute part of the approach embankment, may need to be constructed to accommodate caisson augering equipment prior to installation.

Caisson installation at the abutment locations would generally be carried out through sandy gravel fill, or sand and silt and socketed into bedrock. The water levels at the abutments are 1.2 m to 1.6 m below the existing ground surface. 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 900mm, and as governed by applicable regulations, is required to allow down-the-hole hand cleaning and inspection.

The base of the caisson should be drilled at least 500 mm into the bedrock to remove weathered and more fractured rock, and to mitigate the impact of a sloping rock surface. For a sloping bedrock surface, it is recommended that the base of the caisson should 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. Caissons may have to be drilled deeper into rock if additional lateral resistance is required.

It is anticipated that a liner advanced into the bedrock will provide some seepage cut-off. It is expected that water seepage will be encountered and, therefore, the caisson hole should be pumped dry to allow visual inspection of the base. Caisson augering equipment should be capable of removing and penetrating boulders and rock shatter that may be present above the bedrock surface. The concrete should be placed using good tremie techniques.

#### **7.5 Frost Cover**

The provision of frost cover for footings founded on sound bedrock is not required.

Frost protection should be provided to caisson caps and footings founded on engineered fill. This may take the form of 1.9 m of earth cover, or equivalent insulation, over the underside of the caisson cap or footing base (founding elevation).

It may be possible to eliminate the frost protection 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 at 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 existing sandy gravel fill and the silty clay can be classified as Type 3 soils. Below the water table, the sand and silt is classified as a Type 4 soil.

### **8.2 Foundations**

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

### **8.3 Earth Excavation**

Excavation for footing or caisson cap construction will likely be carried out through the new and existing fill and silty clay, into the sand and silt below the groundwater level. It is anticipated that unsupported open cuts with inclined side slopes are possible, while temporary shoring may be required at locations in close proximity of the existing highway. Recommendations for temporary shoring design should be developed during detailed design as required.

### **8.4 Rock Excavation**

It is anticipated that requirements for rock excavation will be minimal at this site. The strength of intact rock is strong to very strong. Relatively shallow excavation into rock for footing construction may require appropriate excavators equipped with rock teeth and rock splitting equipment. Blasting is not likely required at this site.

Any rock excavation should be carried out in accordance with the Special Provision, Amendment to OPSS 120, 1994.

Should blasting be proposed, the Contractor's blasting and monitoring plan should take into account nearby structures. The contract documents should alert the contractor to these structures. The Contract Administrator should retain a blasting expert for review of the Contractor's blasting procedures prior to approving them.

## **9 GROUNDWATER CONTROL**

At the north abutment and centre pier, perched water in the existing fill could be trapped locally in depressions on the rock surface and may seep into the excavations and cuts. At the south abutment,

water seepage will also occur into excavations extending into the sand and silt below the groundwater level. Surface runoff may also contribute to water accumulation in the excavations.

The design of foundations bearing on bedrock will not be influenced by the groundwater. All concrete placement and engineered fill compaction must be carried out in the dry. The Contractor must, therefore, make provision to control the groundwater seepage and surface water runoff by using sump pumps to remove any accumulated water from the excavation base prior to placing concrete or compacting fill.

*Design  
safe  
abundant!*

Caisson installation will extend below the groundwater level at this site. Detailed recommendations on caisson installation are contained in Section 7.4.4.

## 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

Both approach embankments will be up to 8 m to 9 m in height. At the north approach, the new fill will be placed on the existing dense sandy gravel fill overlying bedrock. At the south approach, assuming that the surficial silty clay is sub-excavated, the foundation soils will consist of the typically compact sand and silt overlying bedrock. 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 caisson installation (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, weak or otherwise unsuitable overburden materials should be removed from the footprint of the embankments. Provided that the core is constructed as an engineered fill, 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.

*global*

Preliminary stability analyses results indicated that Factors of Safety (F.S.) were in the order of 1.4 and 1.5 for earth and rock fill embankments, respectively. Stabilizing berms will not be required to maintain stability. For an earth fill embankment, however, a mid-height berm may be required to address surficial stability as discussed in Section 10.3 Embankment Construction.

## 10.2 Settlement

Some settlement 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 existing sandy gravel fill has been in place for some years and it is expected that settlement of this fill has completed.

The new fill and footing loads will induce elastic settlement within the foundation sands and silts below the south approach embankments. In order to minimize post construction settlement, it is recommended that the approach fills be placed and compacted to induce the elastic settlements prior to construction of the footings or caissons. It is anticipated that the elastic settlement will be completed by the end of engineered fill placement.

## 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, "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 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 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 north approach embankment is considered stable against seismic activities at this site. The south approach embankment is founded on water-bearing sands and silts. The potential for instability due to seismic activities should be assessed during detailed design.

*Magnum!*

*Agreed!*

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. A conventional concrete abutment will be required for the contemplated design but RSS could be used for the wing walls.

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 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 engineered earth or rock fill, existing sandy gravel fill, or the native sand and silt, or directly on limestone bedrock. 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 3,000 kPa at ULS for the RSS block founded on limestone bedrock (SLS is not applicable for foundations on bedrock) at approximate Elevation 134.8 m at the north approach.
- Factored geotechnical resistance of 900 kPa at ULS and geotechnical resistance of 350 kPa at SLS, founded on engineered Granular A pad (thickness equal to or greater than RSS block thickness) at or above approximate Elevations 134.8 m and 136 m at the north and south approaches, respectively.

should use  
subgrade  
clayey silt  
silty clay

high

- Ultimate coefficient of friction between RSS mass and bedrock is 0.9.
- Ultimate coefficient of friction between RSS mass and Granular A pad is 0.7.
- Ultimate coefficient of friction between RSS mass and compact granular fill is 0.55.

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 likely be founded on new engineered fill or existing fill.

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 would be less than 25 mm and would essentially occur as the RSS is constructed.

---

## **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	.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 0.2

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The site area is underlain by compact to dense sandy gravel fill or about 4 m of typically compact native sand and silt. The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

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

The value of  $k_h$  used in calculating the earth pressure coefficients for yielding structures is 67% of the peak value including the effects of site amplification, or 0.124g. The vertical acceleration factor,  $k_v$ , has been taken as 0.6 times  $k_h$ . These design assumptions are based on conventional practice in seismic design. For yielding walls, the recommended  $k_h$  design value is based on allowable wall movements smaller than that outlined in C4.6.4 of the CHBDC. For non-yielding walls, the Woods method adopted is based on elastic theory resulting in seismic earth pressure coefficients comparable to that outlined in the CHBDC.

#### 14.2 Liquefaction Potential

For foundation elements founded on bedrock, there is no potential for liquefaction under the foundations. For foundation elements founded on an engineered fill pad that is itself resting on the bedrock, there is negligible potential for soil liquefaction.

At the south approach, preliminary assessment base on the criteria in Clause C4.6.2 of the CHBDC indicated that the compact to loose sand and silt is potentially liquefiable. The liquefaction potential at the south approach foundation should be further assessed during detailed design.

#### 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 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	

\* After Mononobe and Okabe

\*\* After Woods

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

## 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:

- confirming that immediate settlement has been completed under the weight of the approach fill prior to footing construction at the south abutment.
- variations in the elevation of the bedrock surface, necessitating the use of mass concrete fill to raise the subgrade level and to create uniform founding surface.
- dewatering of temporary excavations for footing or engineered fill construction.



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Review Principal, Designated MTO Contact

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Point Load Test Results  
County Road 6 Underpass

**TABLE B1**  
**County Road 6 Underpass**  
**Point Load Test Results**

Depth			Is50	UCS (MPa)					
feet	Inches	m							
CR6-1									
18	2	5.54	5.09	122.19	}	Total Rock Core			
18	7	5.66	6.50	155.90		Average	Minimum	Maximum	
19	6	5.94	7.46	179.07		141	74	179	MPa
21	3	6.48	5.75	137.99					
22	0	6.71	5.57	133.78		Run #	Average		
23	1	7.04	6.01	144.31		1	152.39		
24	6	7.47	4.70	112.71		2	132.20		
25	6	7.77	5.66	135.88		3	141.41		
26	2	7.98	7.46	179.07					
27	7	8.41	3.07	73.74					
28	10	8.79	7.37	176.97					
Depth			Is50	UCS (MPa)					
feet	Inches	m							
CR6-2									
7	6	2.29	1.76	42.13	}	Total Rock Core			
9	0	2.74	5.71	136.94		Average	Minimum	Maximum	
10	1	3.07	3.47	83.22		117	42	172	MPa
12	0	3.66	5.53	132.72		Run #	Average		
14	9	4.50	5.71	136.94		1	87.43		
15	6	4.72	7.15	171.70		2	147.12		
Depth			Is50	UCS (MPa)					
feet	Inches	m							
CR6-3									
6	9	2.06	2.55	61.10	}	Total Rock Core			
8	8	2.64	0.57	13.69		Average	Minimum	Maximum	
10	6	3.20	4.43	106.39		58	14	106	MPa
12	1	3.68	2.68	64.26		Run #	Average		
12	11	3.94	1.93	46.35		1	60.39		
					2	55.30			

*Note: Insufficient solid rock core to perform point load test at every foot due to multiple vertical and subvertical joints along the rock cores*

## **Appendix A**

### **Record of Borehole Sheets**

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## SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

### 1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

### 3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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


### 4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

### 5. LEGEND FOR RECORDS OF BOREHOLES

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

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

 Water Level


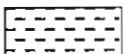



$C_{pen}$  Shear Strength Determination by Pocket Penetrometer

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

# UNIFIED SOILS CLASSIFICATION

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

## EXPLANATION OF ROCK LOGGING TERMS

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

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





# RECORD OF BOREHOLE No CR6-1

2 OF 2

METRIC

G.W.P. 647-92-00 LOCATION N 5 036 561.3 E 295 173.5 (County Road 6) ORIGINATED BY JL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 14.10.03 - 14.10.03 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												
<p>WATER LEVEL READINGS:</p> <table border="1"> <thead> <tr> <th>DATE</th> <th>ELEVATION (m)</th> </tr> </thead> <tbody> <tr> <td>16/10/2003</td> <td>136.1</td> </tr> <tr> <td>22/10/2003</td> <td>136.3</td> </tr> <tr> <td>16/12/2003</td> <td>destroyed</td> </tr> </tbody> </table>																	DATE	ELEVATION (m)	16/10/2003	136.1	22/10/2003	136.3	16/12/2003	destroyed
DATE	ELEVATION (m)																							
16/10/2003	136.1																							
22/10/2003	136.3																							
16/12/2003	destroyed																							

# RECORD OF BOREHOLE No CR6-2

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 036 567.5 E 295 206.5 (County Road 6) ORIGINATED BY JL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NO Coring COMPILED BY SS  
 DATUM Geodetic DATE 14.10.03 - 14.10.03 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			SHEAR STRENGTH kPa						
137.8 0.0	Sandy GRAVEL, occasional cobbles Compact to Very Dense Brown Moist (FILL)		1	SS	29		20	40	60	80	100	PLASTIC LIMIT w <sub>p</sub> NATURAL MOISTURE CONTENT w LIQUID LIMIT w <sub>L</sub>  WATER CONTENT (%)		
			2	SS	57		20	40	60	80	100			
136.2	SAMPLER REFUSAL AT 1.6m.		3	SS	50/075		20	40	60	80	100			
1.6	CRYSTALLINE LIMESTONE (BEDROCK) Slightly to moderately weathered, very thinly to thinly bedded, grey, brown and occasional red with dark grey and white horizontal and subvertical banding, moderately strong to very strong Subvertical joints from 1.98m to 2.08m, 2.24m to 2.26m, 2.31m to 2.72m, 2.79m to 2.92m, 2.97m to 3.02m, 3.25m to 3.3m, 3.4m to 3.45m Vertical joints from 1.65m to 1.78m, 2.41m to 2.57m, 2.92m to 2.97m, 3.12m to 3.2m, 3.75m to 3.64m, 4.22m to 4.32m Multiple fractures from 3.91m to 4.22m, 4.32m to 4.37m		1	RUN	>10 >10 >10 >10 >10		20	40	60	80	100			RUN 1# TCR=100%, SCR=33%, RQD=13%, UCS=87MPa
			2	RUN	>10 >10 >10 >10 >10		20	40	60	80	100			
132.9	END OF BOREHOLE AT 4.88m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS:  DATE ELEVATION (m) 16/10/2003 136.1 22/10/2003 136.2 16/12/2003 136.1 04/02/2004 136.1 11/03/2004 137.4		2	FI			20	40	60	80	100			

# RECORD OF BOREHOLE No CR6-3

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 036 606.5 E 295 247.0 (County Road 6) ORIGINATED BY JL  
HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
DATUM Geodetic DATE 14.10.03 - 14.10.03 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		
136.6 0.0	Sandy GRAVEL, trace silt, frequent cobbles Dense Brown Moist (FILL)		1	SS	43									
			1	GS										
134.9	SAMPLER REFUSAL AT 1.75m		2	SS	101/ Fi 228									
1.8	CRYSTALLINE LIMESTONE (BEDROCK) Slightly to moderately weathered, very thinly to thinly bedded, light grey and light brown with white and black horizontal and subvertical banding, moderately strong to strong Subvertical joints from 2.41m to 2.49m, 3.35m to 3.43m, 3.86m, 4.06m to 4.27m Vertical joints from 2.11m to 2.21m, 2.49m to 2.57m, 2.74m to 2.97m, 3.0m to 3.12m, 3.28m to 3.43m, 3.53m to 3.58m, 4.24m to 4.75m Multiple fracture from 1.75m to 1.98m		1	RUN	>10 >10 >10 >10 >10									
			2	RUN	3 >10 >10 >10 >10									
131.8	END OF BOREHOLE AT 4.8m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.  WATER LEVEL READINGS: DATE ELEVATION (m) 16/10/2003 134.9 22/10/2003 135.0 16/12/2003 destroyed													

## **Appendix B**

### **Laboratory Test Results**

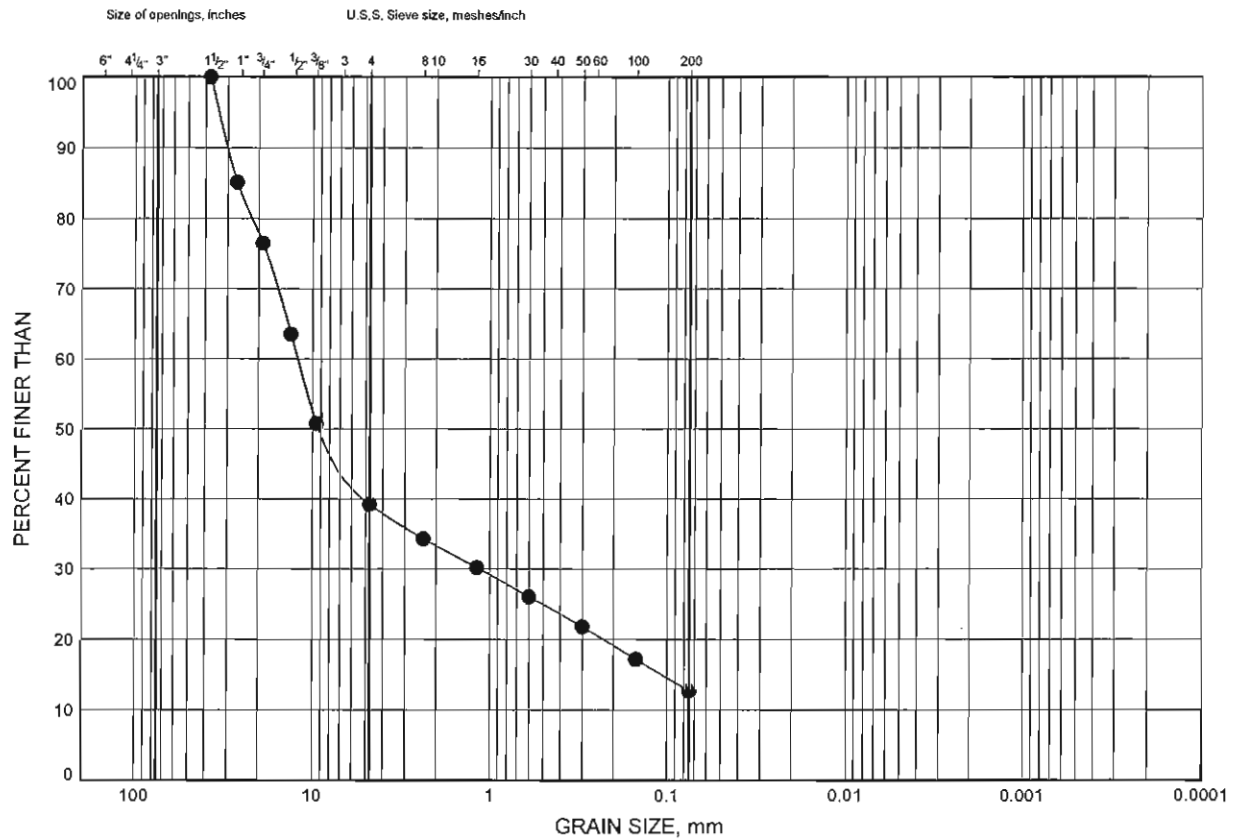
**DRAFT PRELIMINARY**



# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B1

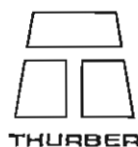
## SANDY GRAVEL (FILL)



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	CR6-3	1.46	135.14

Date May 2004  
Project 647-92-00

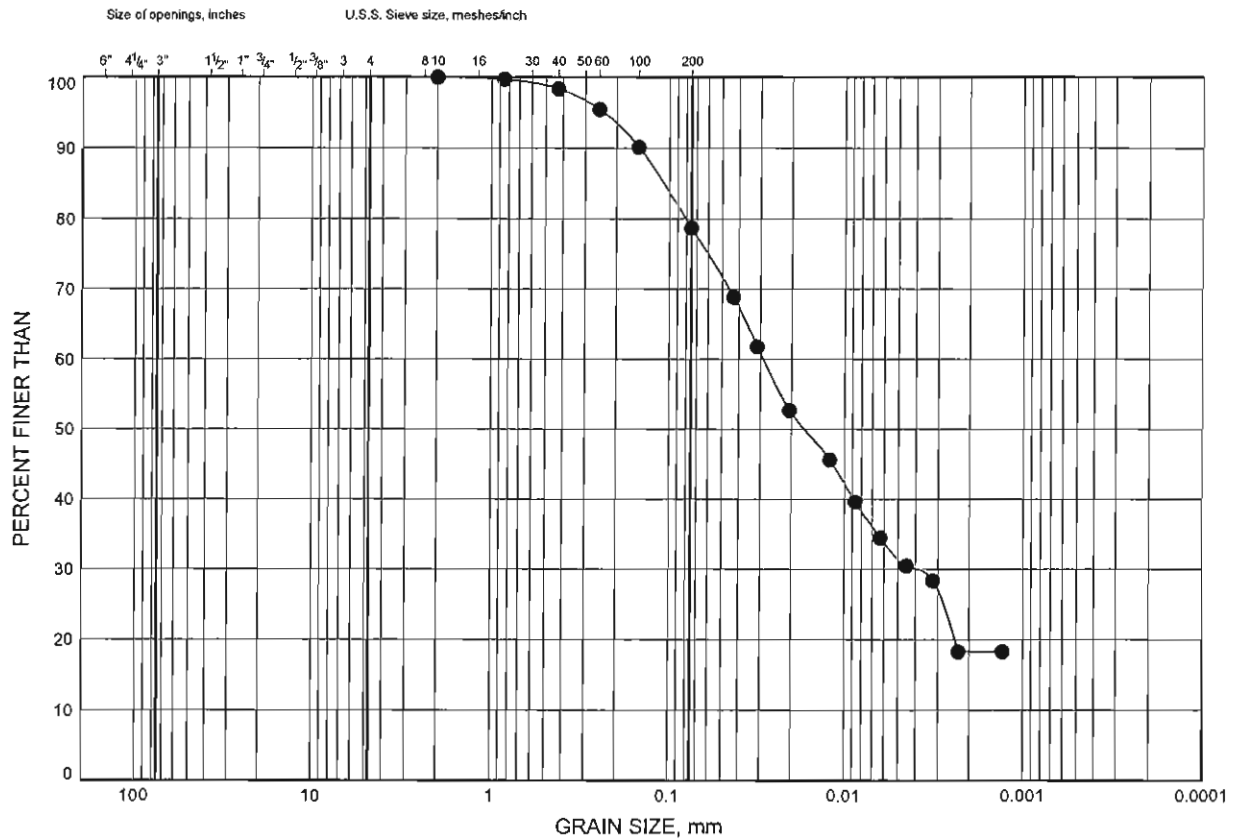


Prep'd SS  
Chkd. SKP

# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B2

## SILTY CLAY TO CLAYEY SILT



## FIGURE B3

The graph displays the grain size distribution of a soil sample. The y-axis represents the percentage of soil finer than a given grain size, ranging from 0 to 100. The x-axis represents the grain size in millimeters on a logarithmic scale, ranging from 100 mm to 0.0001 mm. A curve is plotted through the data points, showing a sharp drop in the percentage finer between 0.075 mm and 0.075 mm sieve size.

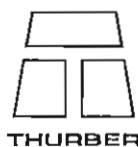
Grain Size (mm)	Percent Finer (%)
100	100
75	100
60	100
47.5	100
37.5	100
30	100
25	100
20	100
15	100
12.5	100
10	100
7.5	100
6	100
4.75	100
3.75	100
3	100
2.5	100
2	100
1.5	100
1.18	100
0.85	100
0.75	100
0.6	100
0.425	100
0.3	100
0.25	100
0.2	100
0.15	100
0.125	100
0.106	100
0.075	100
0.06	100
0.05	100
0.0425	100
0.0375	100
0.03	100
0.025	100
0.02	100
0.015	100
0.0125	100
0.0106	100
0.0085	100
0.0075	100
0.006	100
0.005	100
0.00425	100
0.00375	100
0.003	100
0.0025	100
0.002	100
0.0015	100
0.00125	100
0.00106	100
0.00085	100
0.00075	100
0.0006	100
0.0005	100
0.000425	100
0.000375	100
0.0003	100
0.00025	100
0.0002	100
0.00015	100
0.000125	100
0.000106	100
0.000085	100
0.000075	100
0.00006	100
0.00005	100
0.0000425	100
0.0000375	100
0.00003	100
0.000025	100
0.00002	100
0.000015	100
0.0000125	100
0.0000106	100
0.0000085	100
0.0000075	100
0.000006	100
0.000005	100
0.00000425	100
0.00000375	100
0.000003	100
0.0000025	100
0.000002	100
0.0000015	100
0.00000125	100
0.00000106	100
0.00000085	100
0.00000075	100
0.0000006	100
0.0000005	100
0.000000425	100
0.000000375	100
0.0000003	100
0.00000025	100
0.0000002	100
0.00000015	100
0.000000125	100
0.000000106	100
0.000000085	100
0.000000075	100
0.00000006	100
0.00000005	100
0.0000000425	100
0.0000000375	100
0.00000003	100
0.000000025	100
0.00000002	100
0.000000015	100
0.0000000125	100
0.0000000106	100
0.0000000085	100
0.0000000075	100
0.000000006	100
0.000000005	100
0.00000000425	100
0.00000000375	100
0.000000003	100
0.0000000025	100
0.000000002	100
0.0000000015	100
0.00000000125	100
0.00000000106	100
0.00000000085	100
0.00000000075	100
0.0000000006	100
0.0000000005	100
0.000000000425	100
0.00000000	

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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	CR6-1	3.35	134.15

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

Chkd. ....SKP.....





## **Appendix C**

### **Foundation Comparison**

**DRAFT PRELIMINARY**



### COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	Driven Piles	Footing on Bedrock	Footing on Engineered Fill	Augered Caisson
<b>North Abutment</b>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Required if an integral abutment design is pursued.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Shallow bedrock surface rendering the use of driven piles impractical and unnecessary.</li> <li>ii. Piles are required to be socketted into rock if used at integral abutments.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Shallow bedrock surface below existing ground surface.</li> <li>ii. 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. Due to the proposed height of approach fills (in the order of 9m), the required dimensions of the abutment wall could be relatively large.</li> <li>ii. Mass concrete fill may be required to raise the subgrade level and to create a level founding surface.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Approach fill geometry permits perched abutment design where footings may be placed at a higher elevation, resulting in shorter abutment stem and more economical footing dimensions.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. 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 for foundation construction.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Shallow bedrock surface rendering the use of augered caissons impractical and unnecessary.</li> </ul>
<b>Centre Pier</b>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>None identified.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Shallow bedrock surface rendering the use of driven piles unnecessary and impractical.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Shallow bedrock surface below existing ground surface.</li> <li>ii. 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. Stepped footing may be required.</li> <li>ii. High cost of rock excavation, if any is required.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>None identified.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Shallow bedrock surface making the use of engineered fill unnecessary.</li> <li>ii. Lower geotechnical resistance than bedrock.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>None identified</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Shallow bedrock surface rendering the use of caisson unnecessary and impractical.</li> </ul>

DRAFT PRELIMINARY

County Road 6 Underpass  
Highway 17 Twinning, Amprior to Renfrew

<p><b>South Abutment</b></p>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Required if an integral abutment design is pursued.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Shallow bedrock surface rendering the use of driven piles unnecessary and impractical.</li> <li>ii. Piles are required to be socketted into rock if used at integral abutments.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Shallow bedrock surface below existing ground surface.</li> <li>ii. 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. Required excavation depth in the order of 5 m for footing construction makes this alternative impractical.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Approach fill geometry permits perched abutment design where footings may be placed at a higher elevation, resulting in shorter abutment stem and more economical footing dimensions.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. 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 for foundation construction.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Engineered fill pad is required as a platform to accommodate caisson installation equipment.</li> <li>ii. Nominal rock socketting is required to enhance seating on bedrock.</li> </ul>
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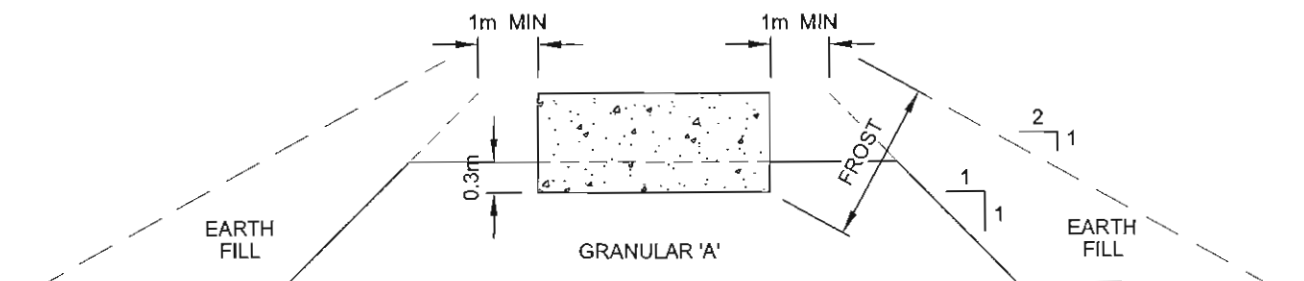
DRAFT PRELIMINARY

## **Appendix D**

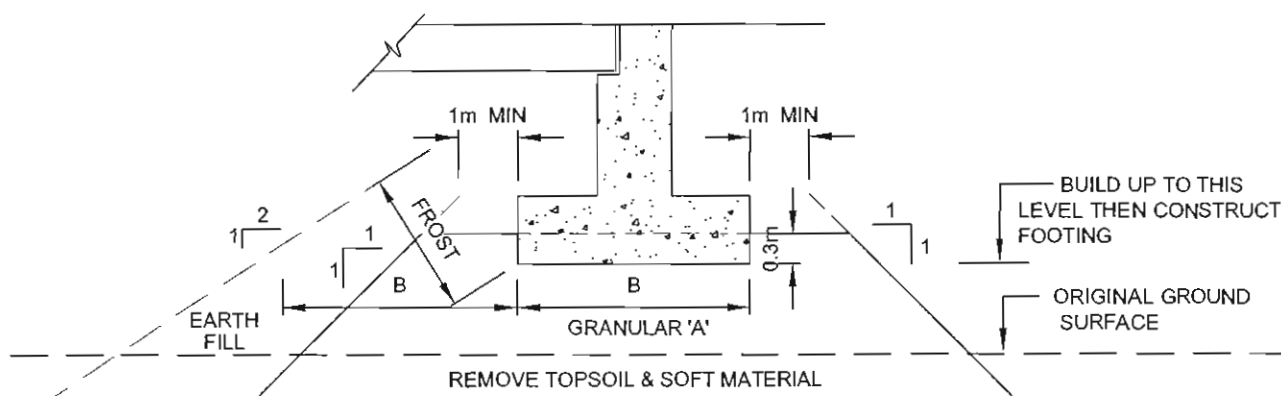
### **Figures**

**DRAFT PRELIMINARY**





## CROSS-SECTION

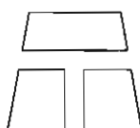


## LONGITUDINAL SECTION

NOT TO SCALE

### NOTES:

1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO O.P.S.S. 501.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C. 1982.

ENGINEER	SP	ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE	 <b>THURBER</b>
DRAWN	SS		
DATE	April , 2004		
APPROVED	PKC		
SCALE	NTS		
			DWG. NO. FIGURE D1

County Road 6 Underpass  
Highway 17 Twinning, Arnprior to Renfrew

## **Appendix E**

### **Drawing**

**DRAFT PRELIMINARY**



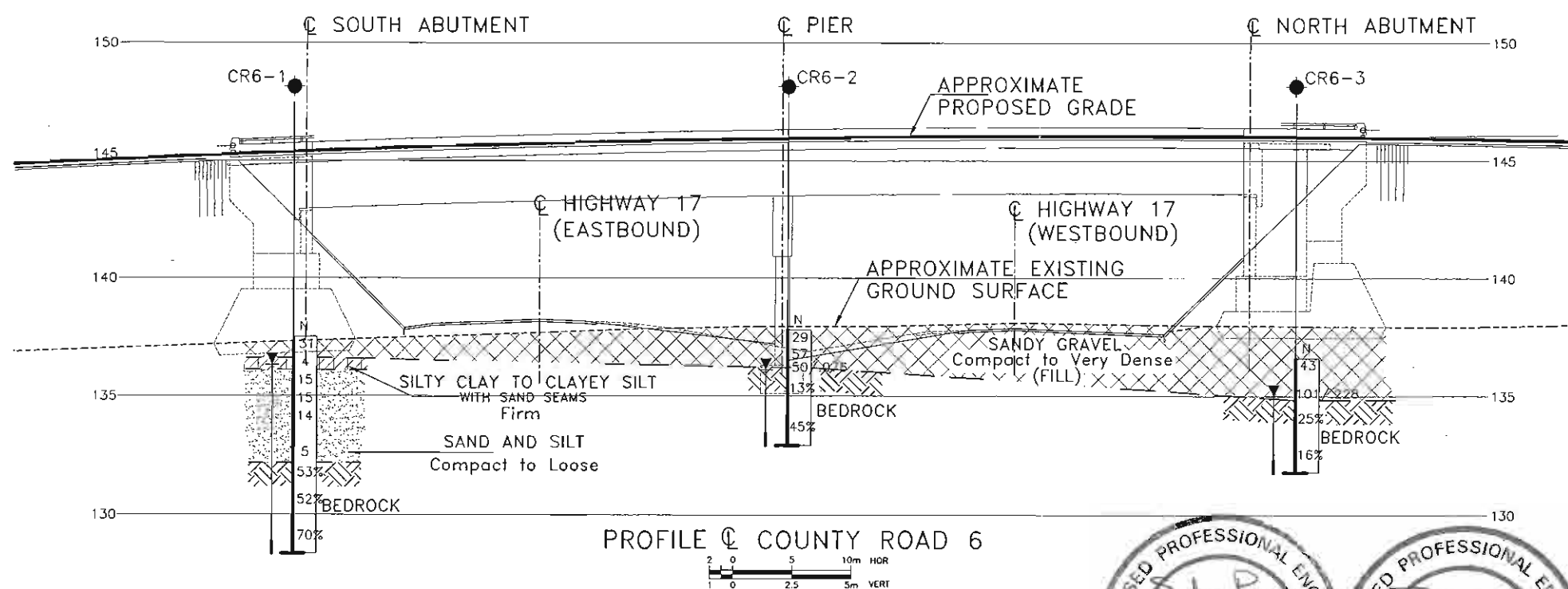
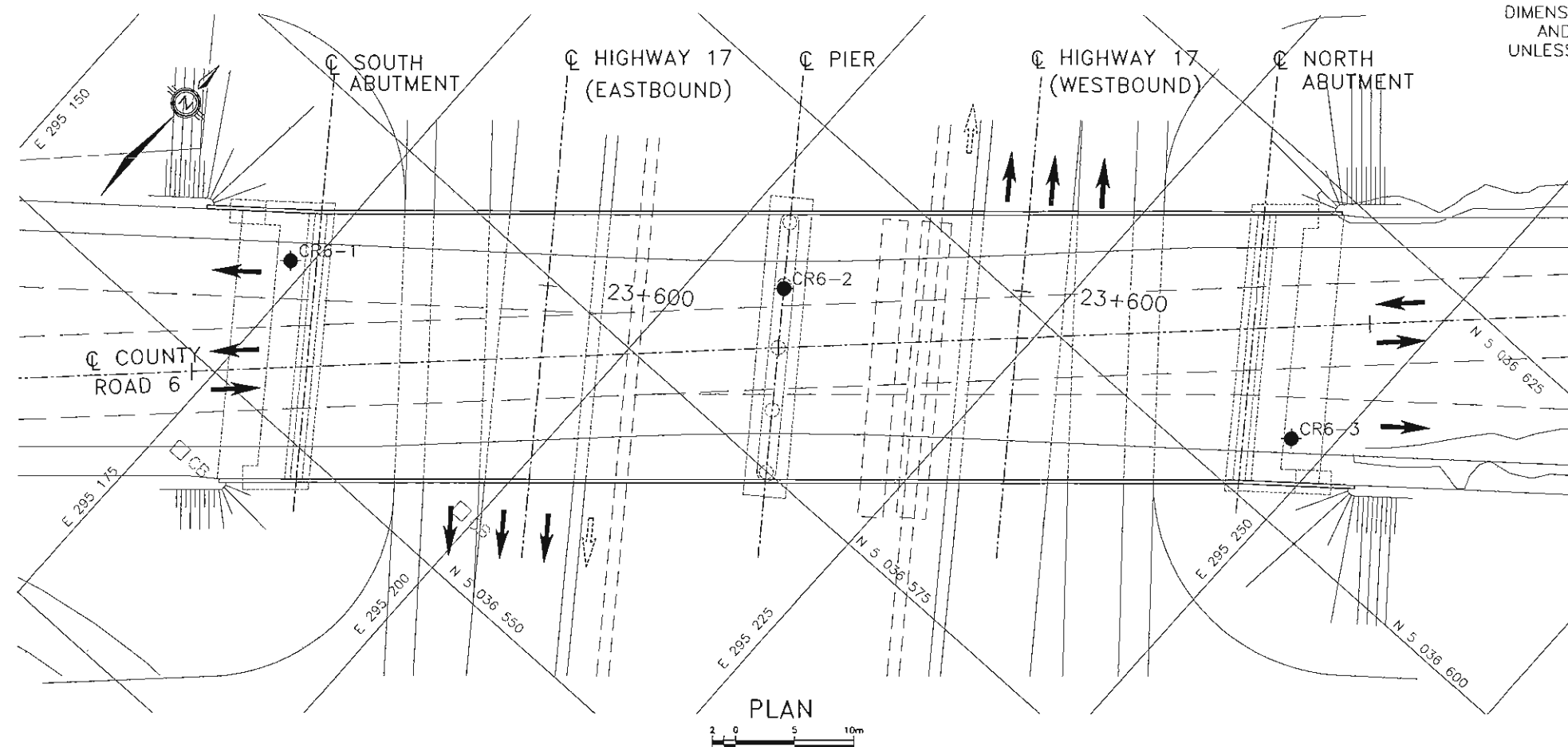
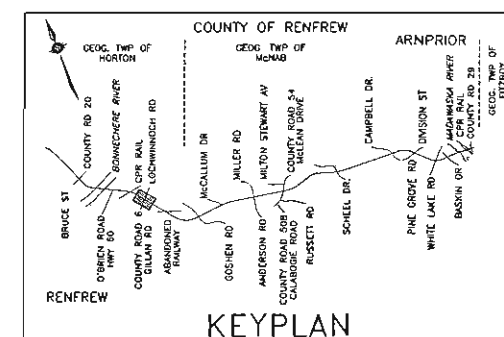
HWY.17  
GWP NO. 647-92-00









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.



REVISIONS							
	MAY. 04	SP	ISSUED AS DRAFT FOR REVIEW				
	DATE	BY	DESCRIPTION				
	DESIGN	SP	CHK	PKC	CHBDC 2000	LOAD	DATE MAY.2003
	DRAWN	SS	CHK	SP	SITE 29-408	STRUCT	DWG.