

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
WHITE LAKE ROAD UNDERPASS  
HIGHWAY 17 TWINNING  
ARNPRIOR TO RENFREW, ONTARIO  
G.W.P. 647-92-00, SITE NO. 29-421  
GEOCRES Number: 31F-132**

**Report to**

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September 30, 2004

File: 19-3745-0

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**PART 1: FACTUAL INFORMATION**

**1 INTRODUCTION**

This report presents the factual findings obtained from a foundation investigation carried out at the location of the White Lake Road Underpass structure that will carry White Lake Road over the twinned Highway 17 westbound and eastbound lanes, Ontario. 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 and cross-sections, 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 during 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 near the at-grade intersection of Highway 17 Twinning and White Lake Road in the Township of McNab, County of Renfrew, Ontario (approximate Mainline Station 25+150). This site is located to the south of the Town of Arnprior.

The site is situated in an area of relatively flat terrain characterized by shallow bedrock underlying glacial drifts. Vegetation is light and mainly consists of grass and occasional small trees and shrubs. Regional drainage in the area is largely governed by the Madawaska River to the east.

The project area is located within a physiographic region known as the Ottawa Valley Clay Plains. This area is located between the Laurentian upland to the north and west, and the Ottawa lowland to the south and east. 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. In Renfrew County, there are prominent east-west trending scarps (fault zones), including a major depression geologically known as the “Ottawa-Bonnechere” graben. Bedrock in the site area consists of crystalline limestone of the Ordovician Period that had been subjected to faulting, weathering and erosion.

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

The site investigation and field testing for this project were carried out between August 6 and August 13, 2003. The site investigation consisted of drilling and sampling a total of nine boreholes to depths ranging from 4.3 m to 14.7 m. The boreholes were numbered WLR-1 to WLR-9. An additional Borehole WLR-5A was later drilled on March 13, 2004 to recover Shelby tube samples.

Surveyors from J. D. Barnes Ltd. (Ottawa) marked the borehole locations in the field. Thurber obtained utility clearances prior to any drilling being carried out.

George Downing Estate Drilling Limited of Calumet, Quebec supplied a CME 55 track-mounted drill rig and conducted the drilling, sampling and in-situ testing operation. Hollow stem augering techniques were used to advance the boreholes. Overburden samples were obtained using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Undisturbed samples of the cohesive soils were recovered using 2-7/8 in. (70 mm) inside diameter thin-walled Shelby tubes. Field vane shear tests using an MTO ‘N’ size vane were carried out at selected depths within the cohesive deposits. Two boreholes within each of the abutment foundation elements, and one borehole at the central pier foundation element were advanced 3 m into bedrock by NQ size diamond coring.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. A standpipe piezometer was installed at each of the abutment foundation elements in Boreholes WLR-2 and WLR-4. The installation details are presented on the Records of Boreholes in Appendix A and summarized as follows. At this site, 50 mm diameter Schedule 40 PVC pipes with a 1.52 m long slotted screen was installed at the bottom of the open boreholes. The sand screen surrounding the pipe was about 3.5 m long. Bentonite holeplug seals were placed just above the sand screen and just below ground surface. The remaining space in the boreholes was grouted with a bentonite-based grout.

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.

On completion of drilling and sampling, the boreholes without piezometer installations were grouted to the surface using a bentonite grout. An asphalt patch was placed at the surface above the grout in Boreholes WLR-5, WLR-6 and WLR-7.

#### **4 LABORATORY TESTING**

All 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.

Sixteen selected samples were subjected to gradation analysis. One sample was selected near the bottom of the sand fill in WLR-1, while the remaining samples were taken at selected depths in the clay deposit. Atterberg Limit tests were carried out on eleven samples of clay to determine the plasticity characteristics. A laboratory oedometer (consolidation) test is carried out on a silty clay sample obtained from Borehole 5A. The results of these tests are shown on the Record of Borehole sheets in Appendix A and on the plots in Appendix B.

Point load testing was carried out on selected rock cores retrieved from Boreholes WLR-1 to WLR-4, and WLR-6. These results are shown in Table 1 attached immediately following the text.

#### **5 DESCRIPTION OF SUBSURFACE CONDITIONS**

Reference should be made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are also presented on both the "Borehole Locations and Soil Strata" drawing and the "Soil Strata" drawing in Appendix F. A description of the stratigraphy is given in the following paragraphs. The factual information at the borehole locations govern any interpretation of site conditions.

In general terms, the site was found to be underlain by topsoil or asphalt and sand fill overlying silty clay. The overburden is underlain by crystalline limestone bedrock.

##### **5.1 Topsoil and Asphalt**

Topsoil of 175 mm in thickness was encountered in Borehole WLR-2.

Asphalt was encountered at locations in the vicinity of the existing intersection between White Lake Road and Highway 17 (Boreholes WLR-4, WLR-5 and WLR-7) ranging in thickness from 25 mm to 50 mm.

##### **5.2 Sand or Sand and Gravel Fill**

Compact to very dense cohesionless fill was found at the surface or beneath the asphalt across the site except at the location of Borehole WLR-2 where fill was absent.

The cohesionless fill generally consists of sand or sand and gravel. The fill is described as very fine to coarse grained, with variable amounts of silt, gravel, cobbles and boulders. The presence of boulders where encountered are shown on the Records of Boreholes. A thin layer of crushed limestone from 0.1m to 0.2m thick was found at the surface of the fill at locations to the north of the intersection (Boreholes WLR03-3, WLR03-8 and WLR03-9).

The thickness of the fill varied from 0.7 m near the northerly limit of the site (Borehole WLR-9) to 3.0 m near its southerly limit (south of the intersection in Borehole WLR-4). The sand was found to be typically in a compact to very dense state. The SPT values were typically greater than 30 blows per 0.3 m penetration and occasionally in excess of 50 blows for less than 0.3 m penetration. At some locations, the SPT values were in the range of 14 blows to 20 blows for 0.3 m penetration. The lower 0.8 m of sand fill in Borehole WLR-4 has a SPT value of 5 which indicates that the sand is loose. This portion of the fill is wet. The measured moisture contents of samples of the cohesionless fill ranged between 2% and 18% with most values lying below 10%.

Figure B1 shows the grain size distribution of a selected sample of sand fill.

### 5.3 Silty Clay to Clay

A deposit of silty clay to clay was encountered below the sand fill across the site, starting at approximate Elevations 105 m to 106.5 m. The thickness of this deposit ranges from about 2 m near the northerly limit of the site to about 10 m near its southerly limit.

The upper "crust" of the clay is generally brown to greyish-brown in colour and extends to about 4 m to 6 m depth below existing ground surface, at approximate Elevation 103 m. The SPT 'N' values generally range from 8 blows to 24 blows for 0.3 m penetration, indicating a typically very stiff to stiff consistency. Atterberg Limit tests conducted on selected samples of this crust are presented in Figure B2. The results indicated that the liquid limits vary between 56% and 44%, and corresponding plasticity indices between 32% and 25%. These values indicate that the crust is typically of medium to high plasticity (CI-CH). Figures B3 and B4 show the grain size distributions of selected samples of this crust. The clay content of these samples varied between 18% and 30%. The measured moisture content varied from about 19% to 42%.

Below a depth of about 4 m to 6 m, the clay changes in colour to grey and becomes firm to soft as indicated by the SPT 'N' values ranging from 2 blows to 4 blows per 0.3 m penetration, and field vane shear tests done at selected depths gave typical values ranging from about 38 kPa to 48 kPa and a few values as high as 60 kPa to 68 kPa. It is noted that some of these higher values may be attributed to sand and silt seams present in this deposit. The sensitivity ranges from 2 to 3.

Atterberg Limit tests conducted on selected samples of this lower portion of the silty clay to clay are presented in Figure B5. The results indicated that the liquid limits vary typically between 46% and 32%, and corresponding plasticity indices between 27% and 15%. These values indicate typically intermediate plasticity (CI) with occasional low plasticity zones (CL). Figure B6 shows the grain size distributions of selected samples of this crust. The clay content of these samples varied between 22% and 37%. The moisture content varied from about 40% to 60%.

One laboratory consolidation (oedometer) test was carried out on an undisturbed specimen prepared from a Shelby tube sample obtained in Borehole WLR-5A. Inferred parameters from the test are summarized in the following table.

Borehole and Sample Number	Existing Overburden Pressure, $p'_0$ (kPa)	Pre-consolidation Pressure, $p'_c$ (kPa)	Compression Index, $C_c$	Re-compression Index, $C_r$	Initial Void Ratio	Over-consolidation Ratio OCR
WLR-5A TW 2	120	180	0.75	0.045	1.45	1.5

The coefficient of consolidation,  $C_v$ , value is estimated to be in the order of  $0.033 \text{ cm}^2/\text{s}$ , or about  $100 \text{ m}^2/\text{yr}$ , 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.

A specific gravity value of 2.80 was measured for the tested specimen. This value corresponds to a unit weight of approximately  $17 \text{ kN/m}^3$ .

Detailed results of this oedometer test are included in Appendix B.

#### 5.4 Bedrock

The soils described above were found to be underlain by crystalline limestone bedrock of the Ordovician Period. The bedrock was proven by coring in Boreholes WLR-1 to WLR-4, and WLR-6. The bedrock surface was inferred from refusal to auger penetration in other boreholes, or refusal to sampler penetration in Borehole WLR-9, drilled at this site. Bedrock surface depths and elevations at the borehole locations are summarized in the following table.

Borehole Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
WLR-1	4.6*	103.1*
WLR-2	2.7*	104.0*
WLR-3	4.6*	102.7*
WLR-4	9.8*	98.8*
WLR-5	10.7	97.9
WLR-6	12.2*	96.3*
WLR-7	7.9	99.9
WLR-8	4.3	103.5
WLR-9	4.7	102.6

Notes : \* Proven by coring

The bedrock is present at relatively shallow depths of 2.7 m to 4.6 m between the north abutment and the centre pier (approximate Elevations 102.6 m to 104.0 m). The bedrock then dips in a northeast to southwest direction along the bridge alignment to approximately 12.2m depth (approximate Elevation 96.3 m) at the west end of the south abutment. In addition, the bedrock also appears to dip from east to west (approximate Elevations 102.7m to 99.9 m) at the centre pier location.

The crystalline limestone is very thinly to thinly bedded and generally in a fresh to slightly weathered state, except in Borehole WLR-2 where the upper 0.8 m of cored bedrock was noted to be moderately to highly weathered. The rock is typically grey in colour with dark grey, black and white horizontal and sub-vertical banding.

The measured Total Core Recovery (TCR) for the core runs was 100% at all locations. The Rock Quality Designation (RQD) values ranged from 55% (near the surface at Borehole WLR-3) to 100%, but typically between 86% and 100% indicating a good to excellent rock quality.

The Fracture Index (FI) of the rock, expressed as the frequency of natural fractures per 0.3m of core, was generally very low ( $<2$ ), except in the upper 1.2 m of Borehole WLR-3 at the centre pier where the FI values of 3 to 4 were measured. Thin, multiple fracturing (broken core) zones were noted in Boreholes WLR-2 and WLR-6.

The joints were typically sub-vertical to vertical. The condition of the joints ranged from planar to uneven and were generally rough though some smooth, planar joints were noted. Occasional calcite infilling and brown (iron oxide) staining was evident in some fractures.

The inferred Unconfined Compressive Strength (UCS) of intact rock cores (expressed as average value per run) range between 77 MPa to greater than 146 MPa with most values greater than 100 MPa, indicating that the intact rock is typically strong to very strong. These estimated rock strength values are based on point load tests that were conducted at

selected locations on rock cores recovered from the boreholes. A summary of the Point Load Test results is presented in Table 1 attached immediately following the text.

### 5.5 Water Levels

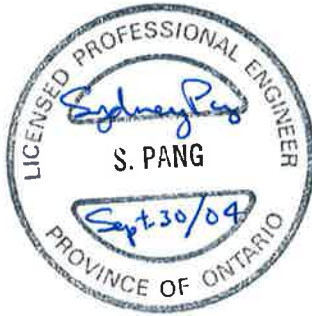
The groundwater levels observed in the open boreholes on completion of drilling ranged from about 4 m depth, or Elevation 103.3m in Borehole WLR-3 to 9.1 m depth, or Elevation 99.6 m, in Boreholes WLR-4 and WLR-5.

The groundwater levels observed at the standpipe piezometers installed in Boreholes WLR-2 and WLR-4 are summarized in the table below and also shown on the “Borehole Locations and Soil Strata” and “Soil Strata” drawings.

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
WLR-2	October 22, 2003	0.7	106.0
	December 18, 2003	0.7	106.0
	February 5, 2004	1.3	105.4
	March 11, 2004	0.7	106.0
WLR-4	October 22, 2003	2.7	105.9
	December 18, 2003	3.2	105.4
	February 05, 2004	2.6	106.0

The piezometric level was consistent at approximate Elevations 105.4 m to 106 m, 0.7 m to 3 m below existing ground surface.

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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**WHITE LAKE ROAD UNDERPASS**

**HIGHWAY 17 TWINNING**

**ARNPRIOR TO RENFREW, ONTARIO**

**G.W.P. 647-92-00, SITE NO. 29-421**

**GEOCRES Number: 31F-132**

**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**6 INTRODUCTION**

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 White Lake Road 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 80 m long between abutment bearings and will have two spans. Each span will be approximately 40 m long and the structure will be skewed at 24°. The front face of the vertical abutment walls will be located about 5 m from the road shoulder at the South Abutment and about 3m from the road shoulder at the North Abutment.

At the site, the proposed grade of Highway 17 will remain at the grade of the existing at-grade intersection of about Elevations 107.5 m to 108 m. The proposed grade of White Lake Road at the north and south abutments will be at approximate Elevation 116.5 m. This corresponds to approach fill heights of about 9 m and 8.5 m at the north and south abutments, respectively.

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 overlying 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			
East Side	WLR-4	108.6	98.8*
West Side	WLR-6	108.5	96.3*
Centre Pier			
East Side	WLR-3	107.3	102.7*
West Side	WLR-7	107.8	99.9±
North Abutment			
East Side	WLR-2	106.7	104.0*
Centre	WLR-9	107.3	102.6±
West Side	WLR-1	107.7	103.2±

\* Proven by coring

## 7.1 Foundation Alternatives

This section presents discussions on available foundation alternatives, 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 if the proposed grade permits. Due to the presence of bedrock at shallow depths below ground surface at the north abutment, integral abutment piles will likely have to be socketted into bedrock at the north abutment in order to develop the required pile flexibility. The span lengths currently anticipated may be too long 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 anticipated large magnitude of post construction settlement. Footings placed directly on bedrock or on an engineered fill pad resting on bedrock may be considered at the north abutment, although excavation up to 5 m below existing ground surface will be required for construction.

In view of the above, augered caissons socketted into bedrock, or piles driven to or socketted into bedrock, are feasible foundation options at both abutments and the centre pier. At the abutment locations where the proposed grade raise is in the order of 8 m to 9m, a perched abutment design may be considered in conjunction with caissons or piles.

Spread footings on bedrock or footings on engineered Granular A pad resting on bedrock are feasible alternatives at the north abutment.

## **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. It is impractical to use footings at the centre pier due to potential excavation of 4.6 m to 7.9 m to expose bedrock.

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. This approach will reduce the risk of having to excavate bedrock under a footing. 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 varies between approximate Elevations 102.6 m and 104.0 m across this foundation.

### **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 5,000 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 design 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 north abutment location should be evaluated in accordance with the CHBDC, 2000 assuming an ultimate coefficient of friction of 0.85.

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 short pile in the rock. Using a lower bound value of 30 MPa for the unconfined compressive strength of the rock (reduced to allow for fracturing near the surface) or the strength of the grout, an ultimate horizontal resistance of 1.3 MN was calculated 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.

The shearing resistance of the selected dowel must be checked structurally.

### **7.3 Spread Footings on Engineered Fill**

For perched abutment design, spread footings founded on an engineered fill pad may also be considered at the north abutment.

If an engineered fill pad is used, all overburden materials including topsoil, fill and native soils should be removed, and the new fill placed directly on the bedrock surface. 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.

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)

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 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 for spread footings in Section 7 and Subsection 7.1.

##### **7.4.1 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.

##### **7.4.2 Downdrag**

Downdrag forces could be induced on the caissons at the south abutment as a result of consolidation of the cohesive foundation soils under the loading of the 8 m to 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 (to be discussed later) is carried out at the approach fills prior to caisson installation.

At the north abutment, compression of the foundation soils involves immediate settlement due to recompression of over-consolidated clays. In order to minimize the potential of downdrag forces acting on the caissons, it is recommended that the approach fills be placed as early as possible prior to installing the caissons.

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<sup>2</sup>/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 103.5	SILTY CLAY TO CLAY Very stiff to stiff	40	60
103.5 to 100.0	SILTY CLAY TO CLAY Stiff to firm	30	45
Below 100.0	SILTY CLAY TO CLAY Firm	25	35

### 7.4.3 Lateral Resistance

At this site, the caissons that may be used at the north abutment and centre pier would be relatively short and 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.

For the subsurface conditions at this site, the lateral resistance of the caissons may be calculated using a value for the coefficient of horizontal subgrade reaction ( $k_s$ ) and ultimate lateral resistance ( $p_{ult}$ ) as follows:

#### Existing Sand and Gravel Fill (compact)

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

$$p_{ult} = 3 \cdot \gamma \cdot z \cdot K_p \quad (\text{kPa}) \quad (\text{from Elevations 108m to 106m})$$

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

$$k_s = 250 \cdot S_u / D \quad (\text{kPa/m})$$

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

where	$z$	=	depth below abutment base in metres or
		=	depth below ground surface in metres at centre pier
	$D$	=	caisson diameter in metres
	$n_h$	=	4,000 kPa/m (existing compact fill)
	$\gamma$	=	20 kN/m <sup>3</sup>
	$K_p$	=	3.0 (passive earth pressure coefficient)
	$S_u$	=	undrained shear strength of silty clay to clay
		=	100 kPa (above Elevation 103.5 m)
		=	50 kPa (between Elevations 103.5 m and 100 m)
		=	35 kPa (below Elevation 100.0 m)

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 numerical analysis should not exceed the ultimate lateral resistance.

The spring constant,  $K$ , for analysis may be obtained by the expression,  $K = k_s \times L \times D$  (MN/m), where  $k_s$  is the coefficient of horizontal subgrade reaction (MPa/m),  $D$  is the caisson width (m),  $L$  is the length (m) of the pile 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 / pile group interaction analysis, the equation for  $k_s$  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

$$\begin{aligned} c &= 2,000 \text{ kPa (equivalent Mohr-Coulomb cohesion based on} \\ &\text{Hoek and Brown rock mass classification)} \\ L &= \text{depth of socket in rock, m} \end{aligned}$$

#### 7.4.4 Caisson Installation

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

Caisson installation at the abutment locations would be carried out through silty clay to clay 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.

It should be expected that caisson installation would encounter cobbles and boulders likely near the base of the clay deposit. 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.

## 7.5 Driven Piles

Steel piles driven to or socketted into bedrock may be considered for use to provide foundation support at the abutments and the pier. Based on the borehole information, bedrock is present at between 2.7 m and 4.7 m depths at the north abutment, between 4.6 m and 7.9 m at the centre pier and 9.8 m and 12.2 m at the south abutment. The following pile tip elevations are recommended for design purposes.

Foundation Element	Reference Boreholes	Estimated Pile Tip Elevation (m)
North Abutment	WLR-1, WLR-2, WLR-9	102.6± to 104±
Centre Pier	WLR-3, WLR-7	99.9± to 102.7± (west to east)
South Abutment	WLR-4, WLR-6	96.3± to 98.8± (west to east)

For integral abutment design, the piles at the north abutment may have to be socketted into bedrock in order to provide base fixity such that adequate pile flexibility can be developed within the upper 3 m of the pile.

### 7.5.1 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 to piles founded on bedrock.

The structural resistance of the pile should be reviewed by the structural designer to confirm that the value given above is not exceeded.

### 7.5.2 Downdrag on Abutment Piles

Downdrag forces could be induced on the piles at the south abutment as a result of consolidation of the cohesive foundation soils under the loading of the new approach fills. The method of calculating downdrag force of steel piles is similar to those for caissons outlined in the previous Section 7.4.2. Downdrag forces could be minimized provided preloading (to be discussed later) is carried out for the approach fill prior to pile installation.

At the north abutment, compression of the foundation soils involves immediate settlement due to recompression of over-consolidated clays. In order to minimize the potential of downdrag forces acting on the piles, it is recommended that the approach fills be placed as early as possible prior to driving the piles.

For the Equation in Section 7.4.2, the unit pile surface area to be used is  $1.81 \text{ m}^2/\text{m}$ .

### **7.5.3 Lateral Resistance**

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

For lateral soil-pile interaction analysis, the recommendations, expressions and parameter values for coefficient of horizontal subgrade reaction ( $k_g$ ), ultimate lateral resistance ( $p_{ult}$ ) and group action reduction factors in Section 7.4.3 are applicable. The caisson diameter,  $D$ , is to be replaced by the pile flange width,  $B$ .

For integral abutments, the flexibility of the pile can be increased by providing a double or single corrugated steel pipe (CSP) system. Considering that the flexible zone is situated within compacted fill, the double concentric pipe configuration is likely more suitable.

The sand for filling the CSPs should be uniformly graded and meet the gradation requirements presented in Table D1 in Appendix D.

### **7.5.4 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 at this site. Sloping bedrock surface is also anticipated. In order to be able to penetrate boulders, cobbles and harder/denser zones in the foundation soils and to enhance adequate seating 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.

In view of the assessed low to medium risk of piles encountering boulders and cobbles above bedrock, it is recommended that if any driven pile achieves the required set at an elevation higher than 2 m above the estimated pile tip elevation tabulated in the table in Section 7.5, the Contractor should immediately advise the Contract Administrator.

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").

## **7.6 Frost Cover**

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

Frost protection should be provided to caisson and pile 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 or pile cap, or footing base (founding elevation).

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 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 silty clay to clay at this site is classified as a Type 2 soil above the groundwater table and a Type 3 soil below the groundwater table. Fill is classified as a Type 3 soil.

### **8.2 Foundations**

The excavation and backfilling for foundations must be carried out in accordance with SP 902S01. A copy of this Special Provision is included in Appendix E.

#### **8.2.1 Earth Excavation**

In addition to SP 902S01, a NSSP should be included in the contract alerting the Contractor to the possible presence of cobbles and boulders in the overburden.

Should the north abutment be founded on spread footings, excavation for footing construction will extend through the existing fill, silty clay to clay and bedrock. Excavation for caisson or pile cap construction at the centre pier will extend through the existing fill into the upper, very stiff to stiff silty clay to clay. Where open cutting with inclined slopes (according to OHSA) is not feasible, a braced soldier pile and lagging wall is considered to be suitable for use as temporary shoring at this site. The soldier piles will need to be socketted into bedrock through pre-augered holes. It is anticipated that the shoring system may be stiffened by struts or cross bracings, where applicable.

An item titled "Roadway Protection" as per SP 539S01 should be included in the contract documents. It is recommended that Performance Level 2 as per Clause 539.04.02.01 be specified for this site.

For a temporary braced soldier pile and lagging wall, the lateral pressure diagram as shown on Figure D2 may be used for design in conjunction with the following parameter values.

$$\begin{aligned}\gamma &= 20 \text{ kN/m}^3 \\ \gamma_w &= 10 \text{ kN/m}^3 \\ K_a &= 0.4 \text{ (silty clay to clay)} \\ h_w &= 0 \\ &\text{(assuming that there is no hydrostatic pressure build-up} \\ &\text{behind a presumably permeable wall)} \\ H &= \text{depth to base of excavation (rock surface), m}\end{aligned}$$

Below the excavation base, lateral earth pressures are applied over a width of  $3B$ , where  $B$  is the diameter of the socket, to take into account three dimensional effects. 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 :

$$P_p = 6 c B L$$

$$\begin{aligned}\text{where } c &= 2,000 \text{ kPa (equivalent Mohr-Coulomb cohesion} \\ &\text{based on Hoek and Brown rock mass classification)} \\ L &= \text{depth of socket in rock, m}\end{aligned}$$

It should be pointed out that the actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the shoring wall. These factors should also be considered when designing the shoring system.

### 8.3 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 included in Appendix E.

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

The relatively impervious deposits of silty clay to clay should not yield a significant quantity of seepage water in the short term. Water seepage will occur with time into the excavations (caisson or pile cap, or spread footing construction) and where water-bearing seams are exposed. 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.

Caisson installation will extend below the groundwater level at this site. The construction of the caissons must be carried out in the dry and the base of the caisson must be cleaned prior to placing concrete. Therefore, unwatering of the caissons may be required if the steel liners, in conjunction with continual pumping, do not adequately control groundwater seepage. Detailed recommendations on caisson installation are contained in Section 7.4.4.

## 10 APPROACH EMBANKMENTS

For the purpose of embankment stability analyses in this report, a 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 term (total stress, undrained) and long term (effective stress, drained) conditions, where applicable.

Immediate (elastic) settlements due to recompression of heavily over-consolidated silty clay to clay have been estimated based on elastic methods. Anticipated settlements due to primary consolidation of the foundation clays, as well as secondary compression where applicable, have been estimated 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 approach embankments for this structure will be constructed on 2 m to 3 m of existing compact cohesionless fill, overlying up to 4 m and 10 m of native silty clay to clay overlying bedrock at the north and south abutments, respectively. With the exception of the compacted Granular A or other engineered fill core, the remainder of the embankment may consist of rockfill. The slope of the core may be formed not steeper than 1H : 1V for Granular A and 1.5H : 1V for other types of cohesionless fill (the 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. Embankments constructed using granular material and select subgrade material will have stable side slopes at inclinations not steeper than 2H : 1V. Berms will not be required to satisfy global stability requirements.

#### *North Approach*

The north approach embankments for this structure will be constructed on dense to very dense sand fill which is underlain by very stiff to firm silty clay to clay overlying bedrock.

Results of stability analyses carried out for a 9 m high Select Subgrade Material (SSM) fill embankment constructed on these soils yielded F.S. for rotational type failure at the forward slope in the order of 1.3 for short term (undrained) and long term (drained) conditions. The calculated F.S. values for a rock fill forward slope were in the order of 1.4 or greater. Figures G1 to G4 present selected stability analyses results for the forward slopes.

The calculated F.S. values for side slopes of both earth (2H : 1V) and rock (1.25H : 1V) fill were in the order of 1.4 or greater. Figures G5 and G6 present selected graphical results of the stability analyses for SSM embankments.

The north approach embankment may be constructed using rock fill with a granular core, or using inorganic earth (SSM) fill founded on native silty clay to clay overlying bedrock.

#### *South Approach*

The south approach embankments for this structure will be constructed on compact to dense sand fill which is underlain by very stiff to stiff becoming firm to soft silty clay to clay overlying bedrock. Bedrock is deeper and the clay deposit is thicker at this location. The lower portion of the clay is firm to soft in consistency and consolidation settlement will occur under the fill weight. The approach fills at this location will be up to 8 m in height. Results of stability analyses carried out for a 8 m high SSM fill embankment constructed on these soils yielded F.S. for rotational type failure at the forward slope in the order of 1.3 for short and long term conditions. The calculated F.S. values for rock fill forward slopes were in the order of 1.6. Figures G7 to G10 present selected stability analyses results for the forward slopes.

The calculated F.S. values for side slopes were in the order of 1.5. Figures G11 and G12 present selected graphical results of the stability analyses for SSM embankments.

However, as discussed in details in the following Section 10.2 Settlement, it is recommended that preloading / surcharging be carried out prior to bridge construction in order to mitigate post construction settlements.

The silty clay below the crust will develop excess pore pressures as the embankment is raised. For a SSM embankment, stability analyses indicated that a F.S. of 1.24 would result upon completion of initial fill placement to the top of surcharge (2 m above the proposed final road grade). Figure G13 illustrates the critical situation immediately after the SSM fill is placed to the top of surcharge. Staged construction would be required to construct such a SSM fill embankment in order to maintain a minimum F.S. of 1.3.

In order to achieve higher F.S. and to avoid staged construction, and in view of the likely situation where blast rock will be available from sites in this Highway 17 Twinning project (and perhaps also from other sites in the vicinity of this project), it is recommended that rock fill be used to construct the south approach embankment.

Figure G14 illustrates that a F.S. of 1.3 can be achieved for the critical situation immediately after completion of placement of 7.5 m of rock fill (to the assumed pavement subgrade level) and 2.5 m of granular fill (assumed top of surcharge at 2 m above proposed final grade). Figure G15 illustrates that granular fill is placed at the forward slope area for preloading purposes.

### *Seismic Stability*

In general, the approach embankments may consist of rock fill with a granular core or SSM fill founded on native silty clay to clay overlying bedrock. The groundwater level is below the base of the embankment. These materials have negligible to no potential for liquefaction. Consequently, the approach embankments will be stable against seismic activities at this site.

## **10.2 Settlement**

Settlement in the order of 40 to 50 mm will occur within the rock fill or well compacted SSM fill. This settlement should be complete by the end of construction and negligible post construction settlement is anticipated within the fill.

The foundation silty clay to clay deposit is heavily over-consolidated within the upper portion, i.e. above Elevation 103 m, changing with depth to lightly over-consolidated above bedrock. As such, a significant proportion of the total settlement (see below) is attributed to the elastic recompression of the clay. It is noted that the oedometer test sample with an estimated OCR of 1.5 (see previous Section 5.3) was retrieved from the lightly over-consolidated zone above bedrock.

### *North Approach*

The new fill will induce settlement within the foundation soils. At the north approach, bedrock is present at Elevation 103 m. As such, foundation settlement will be in the form of immediate settlement of the existing fill and recompression of the heavily over-consolidated foundation silty clay to clay that are expected to be complete by the end of construction. Results of settlement analyses, assuming an embankment height of 9 m, indicate that the magnitude of such immediate foundation soil settlement would be in the order of 25 mm. It is anticipated that there will be negligible post construction settlement due to primary and secondary consolidation.

### *South Approach*

The new fill will induce significant foundation settlement due to primary consolidation as the final pressure exerted on the clay deposit is expected to exceed the preconsolidation pressure. The settlement analysis results are presented as follows.

Without preloading/surcharging and assuming that the 8 m high fill is placed, the analysis results indicate that immediate foundation soil settlement could be in the order of 70 to 80

mm and is anticipated to be complete by the end of fill placement. Subsequent settlement due to primary consolidation of the lower, lightly over-consolidated, portion (below about Elevation 103m) of the clay deposit is estimated to be in the order of 300 to 325 mm. Primary consolidation is estimated to be complete within 9 to 12 months after completion of fill placement.

For preloading/surcharging, assuming that rock fill is placed to the assumed pavement subgrade level (7.5 m in height) and granular fill is placed to the top of surcharge (2 m above proposed final grade), the analysis results show that removal of the surcharge (regrading to the proposed final grade) in 4 to 5 months would result in over-consolidation of the clay and negligible post construction settlement due to primary consolidation. Under such loading conditions, it is estimated that immediate settlement of the foundation soils could be in the order of 70 to 80 mm and is anticipated to be complete by the end of fill placement. Settlement due to primary consolidation is estimated to be in the order of 400 to 425 mm.

Provided that the preloading/surcharging is carried out as recommended in this report, settlement due to secondary compression is anticipated to be less than 25 mm in 10 years after construction.

### **10.3 Embankment Design**

Based on the stability analyses results in Section 10.1 Stability, both the north and south approaches may be constructed by placing rock or SSM fill to the final grade without the need of any ground improvement or the use of lightweight fill. At the south approach, however, in order to limit post construction settlements, several embankment design alternatives are considered and discussed in the following.

#### *Preloading/Surcharging*

The settlement analysis results in Section 10.2 Settlement indicate that conventional preloading/surcharging will be feasible in reducing post construction settlement to within tolerable limits, provided that sufficient time of up to 6 months is available prior to removal of surcharge.

#### *Wick Drains*

Wick drains increase the rate of foundation pore water pressure dissipation and reduce the time required to complete primary consolidation. Its effectiveness can only be verified by installing and monitoring instrumentation. In view of the scope of work anticipated at this site (south approach only) and the feasibility of preloading/surcharging, the use of wick drains is not considered necessary unless there is insufficient time available (e.g. less than 6 months) for preloading the foundation soils. In such case, it is recommended that wick drains be used within the 20 m zone immediately behind the south abutment, in order not to jeopardize the bridge construction schedule. The detailed design of wick drains and

associated schedule of preloading/surcharging and embankment construction is beyond the scope of work of the current investigation.

#### *Lightweight Fill*

Lightweight fill commonly used on MTO projects include blast furnace slag and expanded polystyrene (EPS).

At this site, the use of the Type II ultra lightweight slag (bulk unit weight of about 12 kN/m<sup>3</sup>) is not considered feasible due to the following reasons; (1) some preloading/surcharging may also be required to limit post construction settlement and (2) the Type II slag is only available from a plant in Hamilton, which makes this option unlikely to be cost effective due to the large hauling distance.

Given the limited scope of work and the relatively high unit cost, it is considered that the use of EPS is not a cost effective alternative at this site.

Based on the above, it is considered that conventional preloading/surcharging is the most cost effective means of constructing the south approach embankment.

The recommended procedures for carrying out the preloading/surcharging scheme is outlined as follows:

- Place rock fill up to the design pavement subgrade level. Place Granular B Type II fill to top of surcharge level. The toe of the rock slope (1.25H : 1V) should coincide with the heel of the proposed abutment footing (see Figure G15).
- Place Granular B, Type II fill (2H : 1V) in front of the rock fill slope. The toe of the granular fill should be at least 20 m in front of the heel of the proposed footing (see Figure G15).
- Leave all fill in place for six months (waiting period) after the completion of fill placement. This period is estimated to allow dissipation of a substantial proportion of excess pore pressures generated within the foundation clay under the loading of the new fill.
- Construct the abutment.
- Geotechnical instrumentation in the form of settlement plates and piezometers should be installed at selected locations across the embankment footprint to monitor the magnitude and rate of foundation settlement. Detailed design for an instrumentation and monitoring program is beyond the scope of work of this investigation.

Provided that pre-loading/surcharging is carried out as recommended above, it is estimated that post construction settlement at the south approach would be less than 25 mm.

#### **10.4 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 rock fill embankments are higher than 10 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 10 m. 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. It is noted that the requirements for a 2 m wide berm for a 10 m high rock fill, and for a 8 m high earth fill, are in place to address surficial stability and to provide access for post construction maintenance.

Vegetation cover should be established on all exposed earth slopes to protect against surficial erosion. Reference may be made to SP 572S01 (supersedes OPSS 572) for more detailed requirements.

### **11 RETAINED SOIL SYSTEMS**

Retained soil system (RSS) walls may be considered for use at this site. Assuming that in the south approach area, preloading/surcharging is carried out as recommended in this report and that construction starts only after the foundation settlement has stabilized, the risk of using RSS wall at this site is considered low. 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.

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 Foundations**

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 existing compact fill, 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 300 kPa at ULS and geotechnical resistance of 200 kPa at SLS, founded on existing compact fill or newly compacted earth fill at or above approximate Elevation 107 m at the north approach, and at or above approximate Elevation 108 m at the south approach.
- Ultimate coefficient of friction between RSS mass and compact 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 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 at this site, are likely to be for wing walls at the abutments. It is envisaged that the RSS will be founded on compacted SSM fill overlying native stiff silty clay to clay. The outer shell of the approach embankments may consist of rock fill, where practicable, or approved, compacted granular materials.

Stability analyses on selected configurations were carried out considering the following variables:

- Compact embankment fill – approved SSM compacted to 100% SPMDD at  $\pm 2\%$  optimum moisture content, with a slope of 2H : 1V (angle of internal friction,  $\phi$ , of  $30^\circ$ , cohesion of 0, and unit weight,  $\gamma$ , of  $20 \text{ kN/m}^3$ ).
- Rock fill outer shell – outer slope of 1.25H : 1V (angle of internal friction,  $\phi$ , of  $42^\circ$ , cohesion of 0, and unit weight,  $\gamma$ , of  $19 \text{ kN/m}^3$ ).
- Groundwater level at Elevation 106 m, or about 2 m depth below existing ground surface.
- RSS block with full retained height and block width (length of RSS reinforcement) equal to 50% of the height, founded on compacted earth fill.

Results of the analyses yield Factors of Safety in the same order of magnitude as those presented in the previous Section 10.1 which indicate that global stability can be maintained for the assumed RSS configuration.

The actual design configuration must be checked for global stability prior to finalization.

### 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. At this site, settlements of RSS walls founded on the existing compact fill and well compacted engineered fill prepared as recommended in this report, are expected to be less than 25 mm provided that the subgrade is preloaded as recommended in this report.

## 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 "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 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:

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

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 abutments, a material with a lower passive pressure coefficient (e.g. Granular B Type I) might be preferred as it results in smaller 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 north abutment and the east portion of the centre pier areas are underlain by compact to dense sandy fill and generally stiff clays with a total thickness of typically less than 6 m. 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.

The west portion of the centre pier and the south abutment areas are underlain by compact to dense sandy fill and typically stiff to firm clays with a total thickness generally between 8 m and 12 m. The thickness of the firm clay deposit ranges between about 2 m and 7 m. The soil conditions at these locations do not entirely match the Soil Profile Types given in Table 4.4.6.1 of the CHBDC. However, in view of the preloading, surcharging and the anticipated strength gain, the foundation soils at these locations are also classified as Type 1 with a Site Coefficient 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 0.091m/sec.

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, there is no potential for soil liquefaction under the foundations.

The approach embankments will be founded on existing compact sandy fill overlying stiff silty clay to clay above the groundwater level 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	Height of Application From Base as Percentage of Wall Height	Earth Pressure Coefficient (K) for Earthquake Loading					
		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

## 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:

- maintaining stability of the preloading and surcharge fill at all times with the assistance of geotechnical monitoring data from settlement plates and piezometers
- confirming that settlement has stabilized at the south approach before removing the surcharge fill and commencement of caisson or pile installation.
- potential for encountering boulders and cobbles during piling or caisson installation procedures
- disturbance of the bedrock under the foundations due to excavation and other procedures
- variations in the elevation of the bedrock surface, necessitating the use of mass concrete fill to prepare the design founding elevation.



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# Point Load Test Results

**TABLE 1**  
**White Lake Road**  
**Point Load Test Results**

Depth			Is50	UCS (MPa)					
feet	Inches	m							
WLR-1					}	Average	Minimum	Maximum	MPa
15	9	4.80	3.42	82.16					
18	0	5.49	5.57	133.78					
19	2	5.84	5.71	136.94					
20	8	6.30	3.12	74.79					
22	6	6.86	3.99	95.86					
						105	75	137	
						Run #	Average		
						1	117.63		
						2	85.32		
Depth			Is50	UCS (MPa)					
feet	Inches	m							
WLR-2					}	Average	Minimum	Maximum	MPa
9	4	2.84	4.74	113.76					
11	7.5	3.54	2.59	62.15					
14	6	4.42	3.82	91.64					
15	8	4.78	4.78	114.82					
17	0	5.18	5.44	130.62					
18	6	5.64	4.61	110.60					
						104	62	131	
						Run #	Average		
						1	113.76		
						2	76.90		
						2	118.68		
Depth			Is50	UCS (MPa)					
feet	Inches	m							
WLR-3					}	Average	Minimum	Maximum	MPa
15	3	4.65	5.62	134.83					
19	0	5.79	6.58	158.00					
20	6	6.25	3.29	79.00					
22	7	6.88	5.40	129.56					
24	0	7.32	5.71	136.94					
						128	79	158	
						Run #	Average		
						1	146.42		
						2	115.17		
Depth			Is50	UCS (MPa)					
feet	Inches	m							
WLR-4					}	Average	Minimum	Maximum	MPa
32	4	9.86	5.53	132.72					
33	6	10.21	3.95	94.80					
34	8	10.57	6.10	146.42					
36	0	10.97	3.51	84.27					
40	0	12.19	4.74	113.76					
41	9	12.73	6.89	165.38					
						123	84	165	
						Run #	Average		
						1	114.55		
						2	139.57		
Depth			Is50	UCS (MPa)					
feet	Inches	m							
WLR-6					}	Average	Minimum	Maximum	MPa
41	0	12.50	4.48	107.44					
42	1	12.83	3.82	91.64					
43	8	13.31	4.61	110.60					
46	0	14.02	4.56	109.55					
48	0	14.63	5.35	128.51					
						110	92	129	
						Run #	Average		
						1	99.54		
						2	116.22		

## **Appendix A**

### **Record of Borehole Logs**

## 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  
 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

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

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength (MPa) (psi)	Field Estimation of Hardness*	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
		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 WLR-1

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION White Lake Road ( N 5 031 259.6 E 314 774.5 ) ORIGINATED BY GA  
HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
DATUM Geodetic DATE 06.08.03 - 06.08.03 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		
107.7 0.0	SAND, trace gravel, trace silt Very Dense to Dense Brown Dry to Moist (FILL)  inferred cobbles and/or boulders from 0.9m to 2.1m		1	SS	62		107							2 91 7 (SI+CL)
			2	SS	50/ .15									
			3	SS	34		106							
105.2 2.6	Wet  Silty CLAY, trace sand, laminated Stiff Brown Wet  inferred cobbles and/or boulders from 3.7m to 4.6m  No sample recovery Spoon sampler refusal at 4.57m.		4	SS	10		105							
			5	SS	8									0 7 74 19
							104							
103.2 4.6	CRYSTALLINE LIMESTONE (BEDROCK) Fresh, slightly to moderately weathered at joints, very thin to thinly bedded, grey with black, white, occasional pink and brown subvertical banding, strong to very strong				FI		103							
			1	RUN	1		102							
														RUN 2# TCR=100%, SCR=98%, RQD=91%, UCS=85.3MPa
			2	RUN	1		101							
100.2 7.5	Subvertical and vertical joint from 7.16m to 7.52m  END OF BOREHOLE AT 7.52m. BOREHOLE OPEN TO 7.52m. BOREHOLE GROUTED TO SURFACE WITH BENTONITE GROUT.				>5									

ONTMT4 7450WLR.GPJ 23/04/04

# RECORD OF BOREHOLE No WLR-2

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION White Lake Road (N 5 031 248.0 E 314 798.3) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 13.08.03 - 13.08.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE						
								● QUICK TRIAXIAL	× LAB VANE						
106.7							20 40 60 80 100		20 40 60						
108.8	TOPSOIL (175mm)														
0.2	Silty CLAY, some thin sand seams, trace gravel to 1.4m Very Stiff Brown (CI)		1	SS	15										
			2	SS	19										
			3	SS	13								0 4 72 24		
	Stiff		4	SS	10 FI										
104.0	END OF SAMPLING AT 2.74m. CRYSTALLINE LIMESTONE (BEDROCK) Moderately to highly weathered becoming fresh at 3.5m, very thinly to thinly bedded, grey with dark grey and white subvertical banding, strong to very strong Subvertical joint from 3.81m to 3.91m. Broken core zones: 75mm (2.9m) 50mm (3.2m) 75mm (3.4m) 25mm (3.5m)		1	RUN	2								RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=113.8MPa RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=76.9MPa RUN 3# TCR=100%, SCR=100%, RQD=100%, UCS=118.7MPa		
			2	RUN	3										
					1										
					1										
			3	RUN	0										
					1										
					1										
100.9	END OF BOREHOLE AT 5.79m. Piezometer installation consists of 50mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.  WATER LEVEL READINGS: DATE ELEVATION (m) 22/10/03 106.0 18/12/03 106.0 05/02/04 105.4 11/03/04 106.0				0										
5.8															

# RECORD OF BOREHOLE No WLR-3

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION White Lake Road ( N 5 031 220.9 E 314 768.8 ) ORIGINATED BY GA  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 07.08.03 - 07.08.03 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		
107.3														
106.8														
0.1	CRUSHED LIMESTONE		1	SS	25		107							
	Grey													
	(FILL)													
	SAND, trace gravel													
	Compact													
	Brown to Grey													
	Moist													
106.1	(FILL)		2	SS	30		106							
1.2	Silty CLAY, trace sand, laminated													
	Very Stiff													
	Mottled Grey and Brown													
	(Cl)		3	SS	16		105							
	Brown													
			4	SS	17		104							0 4 77 18
	Stiff													
			5	SS	10		103							
102.7					FI		102							
4.6	AUGER REFUSAL AT 4.57m.													
	CRYSTALLINE LIMESTONE													
	(BEDROCK)													
	Fresh to slightly weathered, very thinly													
	to thinly bedded, grey with black and													
	white subvertical and horizontal													
	banding, strong to very strong													
	Subvertical joint from 4.7m to 4.78m													
	Vertical joint from 4.8m to 5.23m, and													
	5.61m to 5.69m													
			1	RUN	4		101							
			2	RUN	2		100							
99.7					1									
7.6	END OF BOREHOLE AT 7.57m.													
	BOREHOLE OPEN TO 7.57m.													
	WATER LEVEL AT 4.0m DEPTH													
	UPON COMPLETION.													
	BOREHOLE GROUTED TO													
	SURFACE.													

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

20  
15  
10

(%) STRAIN AT FAILURE

## METRIC

[illegible]

+ 3, × 3: Numbers refer to Sensitivity

ONTMT4 7450WLR.GPJ 23/04/04

# RECORD OF BOREHOLE No WLR-4

2 OF 2

METRIC

G.W.P. 647-92-00 LOCATION White Lake Road (N 5 031 196.5 E 314 739.7) ORIGINATED BY GA  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 08.08.03 - 08.08.03 CHECKED BY SKP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								WATER CONTENT (%)
						20	40	60	80	100	20	40	60			
98.5 10.0	CRYSTALLINE LIMESTONE (BEDROCK) Fresh, slightly weathered at joints, very thinly to thinly bedded, grey with black, white and occasional pink, subvertical wavy banding, strong to very strong  Vertical joint from 11.28m to 12.09m		1	RUN	0 0 1 1	98									GR SA SI CL SCR=100%, RQD=95%, UCS=114.6MPa  RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=139.6MPa	
			2	RUN	1 0 1 0	97										
95.8 12.8	END OF BOREHOLE AT 12.8m. BOREHOLE OPEN TO 12.8m. WATER LEVEL AT 9.1m DEPTH UPON COMPLETION. Piezometer installation consists of 50mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.  WATER LEVEL READINGS: DATE ELEVATION (m) 22/10/03 105.9 05/02/04 105.4 11/03/04 106.0															

# RECORD OF BOREHOLE No WLR-5

1 OF 2

METRIC

G.W.P. 647-92-00 LOCATION White Lake Road (N 5 031 186.8 E 314 728.2) ORIGINATED BY GA  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 07.08.03 - 07.08.03 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					
108.6	ASPHALT (25mm)												
108.5	SAND and GRAVEL		1	SS	32								
0.0	Dense Brown Dry to Moist (FILL)		2	SS	34								
107.2	SAND, trace gravel		3	SS	14								
1.4	Compact Brown Dry (FILL)												
106.3	Silty CLAY, trace sand		4	SS	14								
2.2	Stiff Greyish Brown (CH)		5	SS	13								
	occasional Iron oxide staining		6	SS	8								
	Soft to Firm		7	SS	2								
	Grey		8	SS	3								
			9	SS	2								

ONTMT4 7450WLR.GPJ 23/04/04

98.6

Continued Next Page

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

## 2 OF 2

METRIC

ORIGINATED BY GA

COMPILED BY SS

CHECKED BY            SKP

[illegible]

+ 3, x 3: Numbers refer to Sensitivity

## 1 OF 2

METRIC

ORIGINATED BY JL

COMPILED BY SS

CHECKED BY        SKP

[illegible]

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity

20  
15-5  
10

(%) STRAIN AT FAILURE

ONTM4 7450WLR.GPJ 23/09/04



Ontario



**RECORD OF BOREHOLE No WLR-5A**

2 OF 2

METRIC

G.W.P. 647-92-00

LOCATION

White Lake Road (N 5 031 186.8 E 314 729.2)

BOREHOLE TYPE

### Hollow Stem Augers

HWY HWY 17

DATE \_\_\_\_\_

13.03.04 - 13.03.04

DATUM Geodetic

ORIGINATED BY JL

COMPILED BY SS

CHECKED BY SKP

[illegible]

# RECORD OF BOREHOLE No WLR-6

1 OF 2

METRIC

G.W.P. 647-92-00

LOCATION

White Lake Road (N 5 031 208.1 E 314 715.9)

ORIGINATED BY GA

HWY HWY 17

BOREHOLE TYPE

Hollow Stem Augers, NQ Coring

COMPILED BY SS

DATUM Geodetic

DATE

08.08.03 - 08.08.03

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					
108.5 0.0	SAND, trace gravel Compact to Dense Brown Dry (FILL)		1	SS	29								
			2	SS	28								
	Greyish Brown, Moist		3	SS	32								
106.3 2.2	Silty CLAY, trace sand Very Stiff to Stiff Mottled Greenish Grey and Brown (CH)		4	SS	24								0 6 71 23
	Brown		5	SS	14								
	becoming grey becoming firm to soft		6	SS	4								0 2 67 31
	occasional sand seams (CI)		7	SS	2								
			8	SS	2								
			9	SS	2								0 18 46 36
98.5	occasional sand pockets/ seams (CI)												

ONTMT4 7450WLR.GPJ 23/04/04

Continued Next Page

+ 3 x 3 : Numbers refer to Sensitivity

20 15 10 5 0 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No WLR-6

2 OF 2

METRIC

G.W.P. 647-92-00 LOCATION White Lake Road (N 5 031 208.1 E 314 715.9)  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring  
 DATUM Geodetic DATE 08.08.03 - 08.08.03  
 ORIGINATED BY GA  
 COMPILED BY SS  
 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 ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE					
							20 40 60 80 100	20 40 60 80 100					
10.0	Silty CLAY, trace sand Soft to Firm Grey		10	SS	2								
96.3					FI								
12.2	AUGER REFUSAL AT 12.19m. CRYSTALLINE LIMESTONE (BEDROCK) Fresh, slightly weathered at joints, very thinly to thinly bedded, grey with dark grey and white subvertical banding, strong to very strong Subvertical joint from 12.32m to 12.37m, and 12.6m to 12.67m Broken core zones: 25mm (12.9m)		1	RUN	5								
					1								
					1								
					1								
			2	RUN	5								
93.8													
14.7	END OF BOREHOLE AT 14.73m. BOREHOLE GROUTED AND PATCHED WITH ASPHALT AT SURFACE.												

3

3

Numbers refer to

20

15

5

(%) STRAIN AT FAILURE

RUN 1#  
TCR=100%,  
SCR=86%,  
RQD=82%,  
UCS=99.5MPa

RUN 2#  
TCR=100%,  
SCR=100%,  
RQD=88%,  
UCS=116.2MPa

## 1 OF 1

METRIC

CHECKED BY SK

+ 3, X 3; Numbers refer to Sensitivity

# RECORD OF BOREHOLE No WLR-8

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION White Lake Road ( N 5 031 269.3 E 314 786.0 ) ORIGINATED BY GA  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 06.08.03 - 06.08.03 CHECKED BY SKP

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
107.8 0.0 107.6 0.2	CRUSHED LIMESTONE Grey (FILL) SAND and GRAVEL, occasional cobbles and/ or boulders Dense to Very Dense Brown Moist (FILL)		1	SS	33	107									0 11 70 19	
			2	SS	50/											
106.4							102									
1.4	Silty CLAY, occasional sand seams Stiff Brown Moist (Cl)		3	SS	10		106									
			4	SS	14		105									
			5	SS	14	104										
103.6 4.3	END OF BOREHOLE AT 4.27m. AUGER REFUSAL AT 4.27m ON PROBABLE BEDROCK. BOREHOLE GROUTED TO SURFACE.															

ONTMT4 7450WLR.GPJ 23/04/04

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

20  
15 10 5  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No WLR-9

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION White Lake Road (N 5 031 256.7 E 314 789.6) ORIGINATED BY GA  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 07.08.03 - 07.08.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE			
107.3							20 40 60 80 100	20 40 60				
0.0 107.0	CRUSHED LIMESTONE											
0.2	(FILL) SAND and GRAVEL		1	SS	32							
106.6	Dense Brown Dry											
0.7	(FILL) Silty CLAY, trace sand Very Stiff to Stiff Brown Moist (Cl)		2	SS	16							
			3	SS	14						0 5 68 27	
			4	SS	13							
	some sand Firm		5	SS	7						0 11 70 18	
102.6			6	SS	50/							
4.7	END OF BOREHOLE AT 4.67m. SAMPLER REFUSAL AT 4.67m ON PROBABLE BEDROCK. BOREHOLE OPEN TO 4.67m. WATER LEVEL AT 4.27m DEPTH UPON COMPLETION. BOREHOLE GROUTED TO SURFACE.				102							

ONTMT4 7450WLR.GPJ 23/04/04

ONTVMT4 7450WLR GPJ 23/04/04

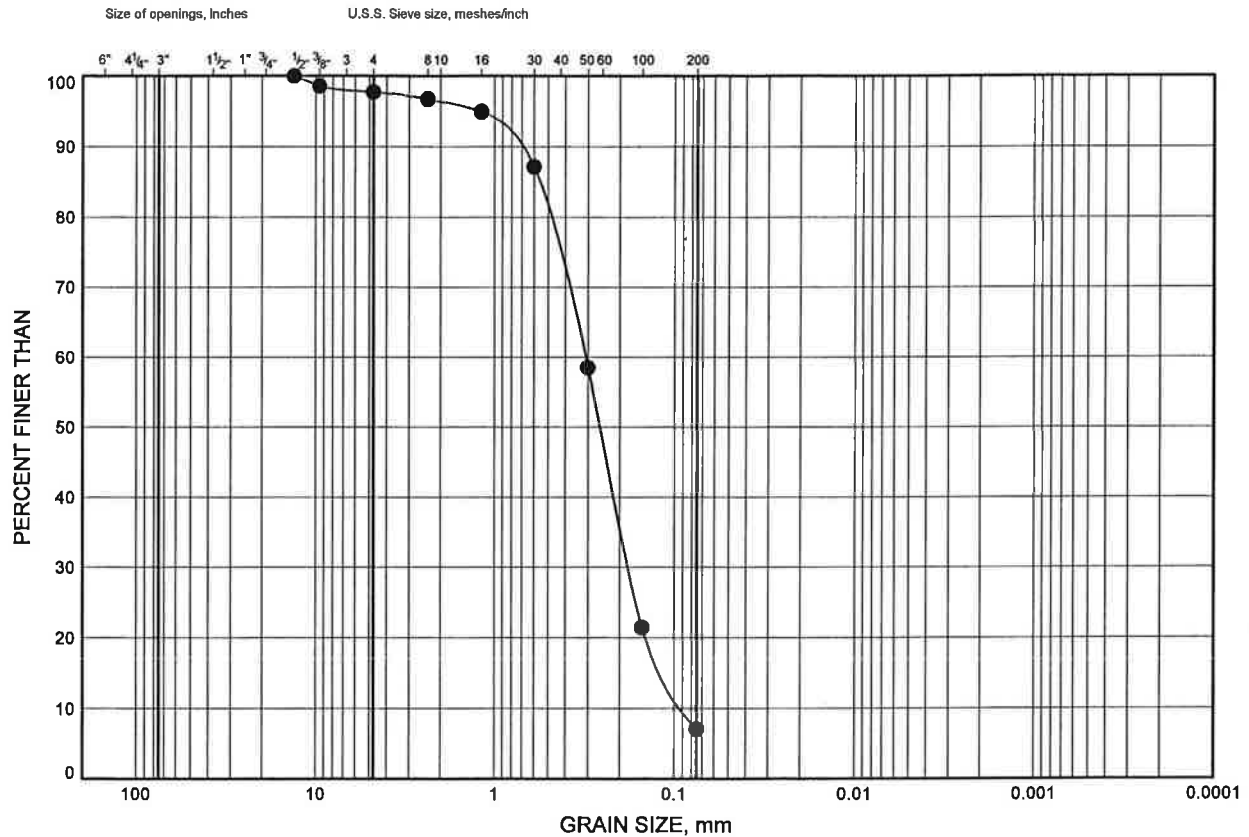
## **Appendix B**

### **Laboratory Test Results**

# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

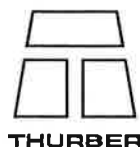
FIGURE B1

## SAND



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	WLR-1	2.32	105.42



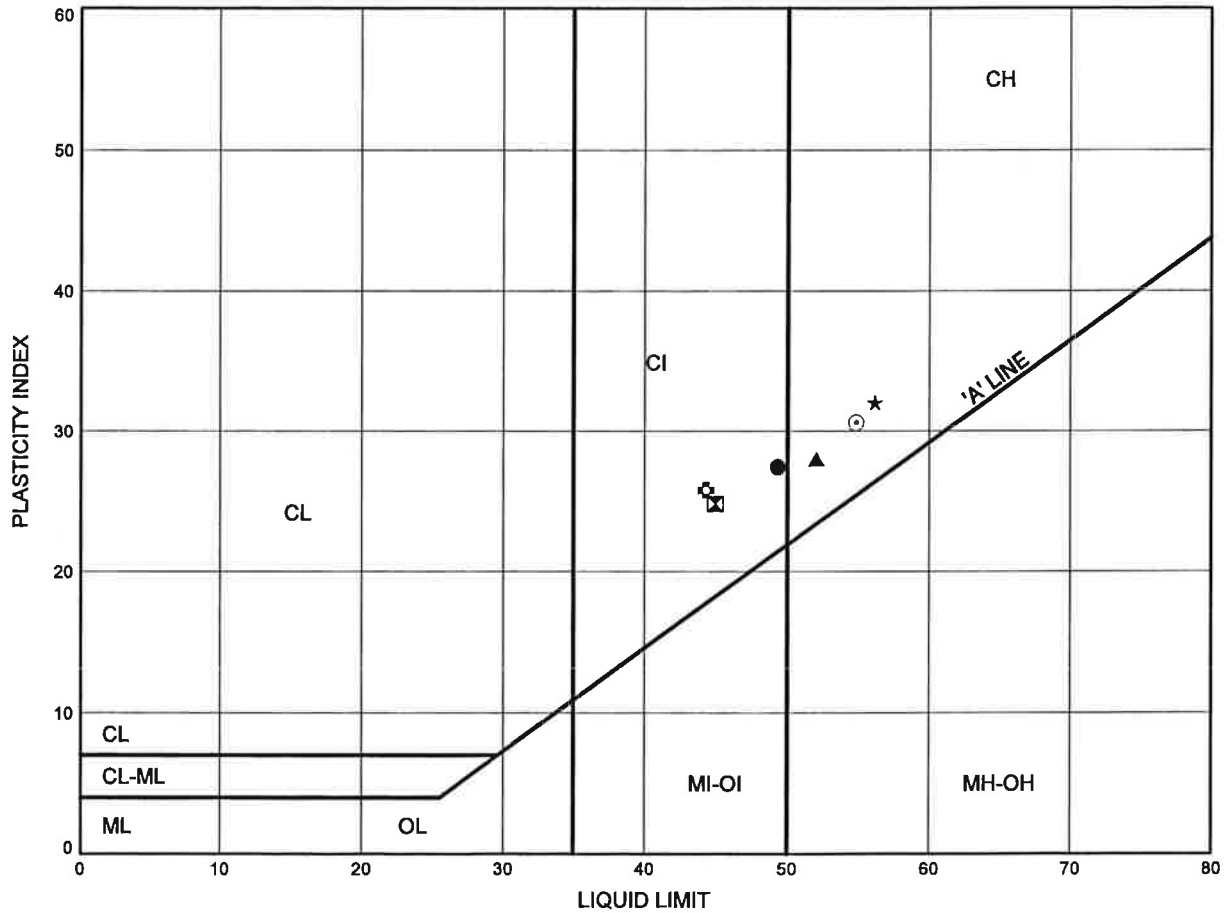
Date April 2004  
Project 647-92-00

Prep'd SS  
Chkd. SP

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

FIGURE B2

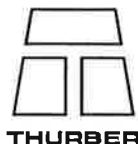
**SILTY CLAY TO CLAY (CRUST)**



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	WLR-2	1.75	104.98
⊠	WLR-3	2.59	104.66
▲	WLR-4	3.35	105.27
★	WLR-5	3.35	105.20
⊙	WLR-6	2.59	105.90
⊕	WLR-9	3.35	103.92

THURBALT 7450WLR.GPJ 14/04/04

Date April 2004  
 Project 647-92-00

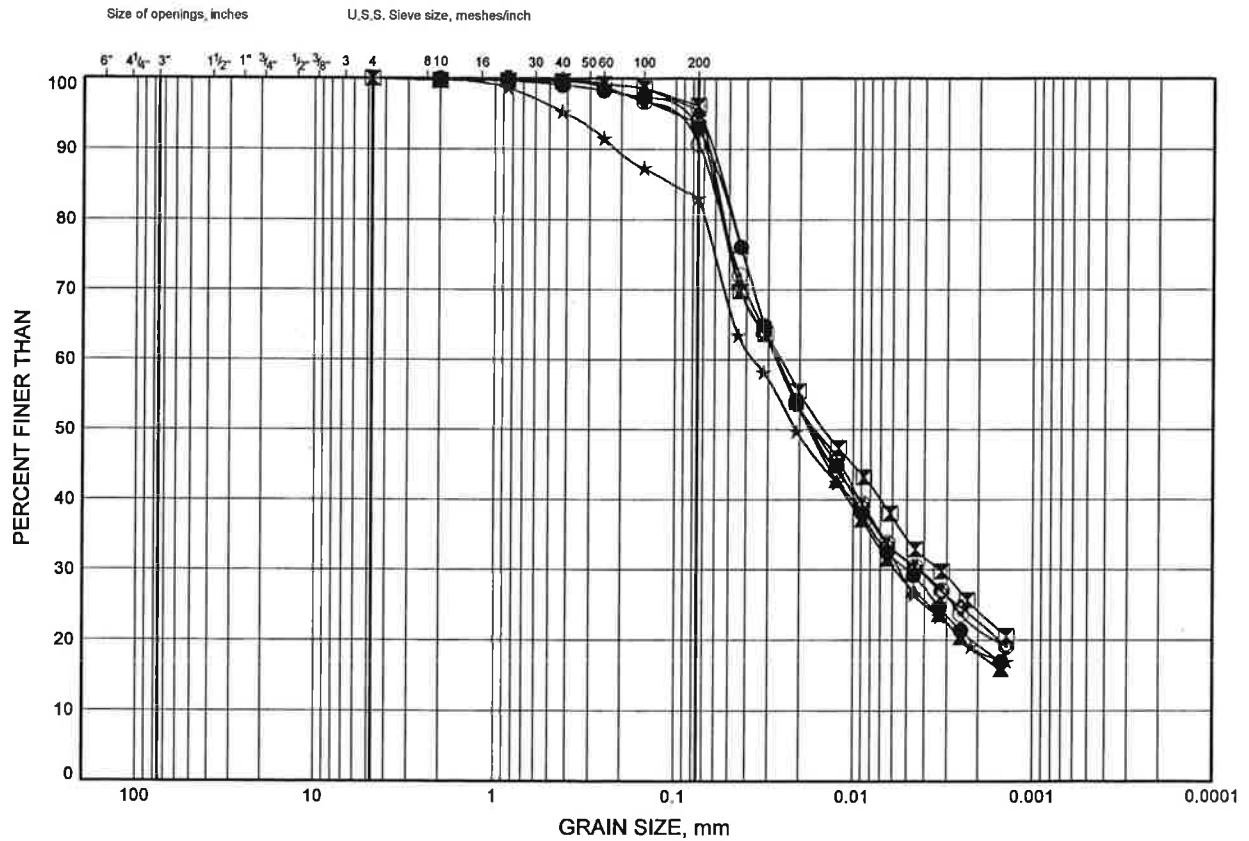


Prep'd SS  
 Chkd. SP

# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B3

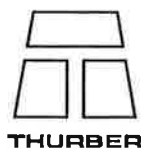
## SILTY CLAY TO CLAY (CRUST)



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	WLR-1	3.35	104.39
⊠	WLR-2	1.75	104.98
▲	WLR-3	2.59	104.66
★	WLR-4	3.35	105.27
⊙	WLR-5	3.35	105.20
⊕	WLR-6	2.59	105.90

Date April 2004  
Project 647-92-00

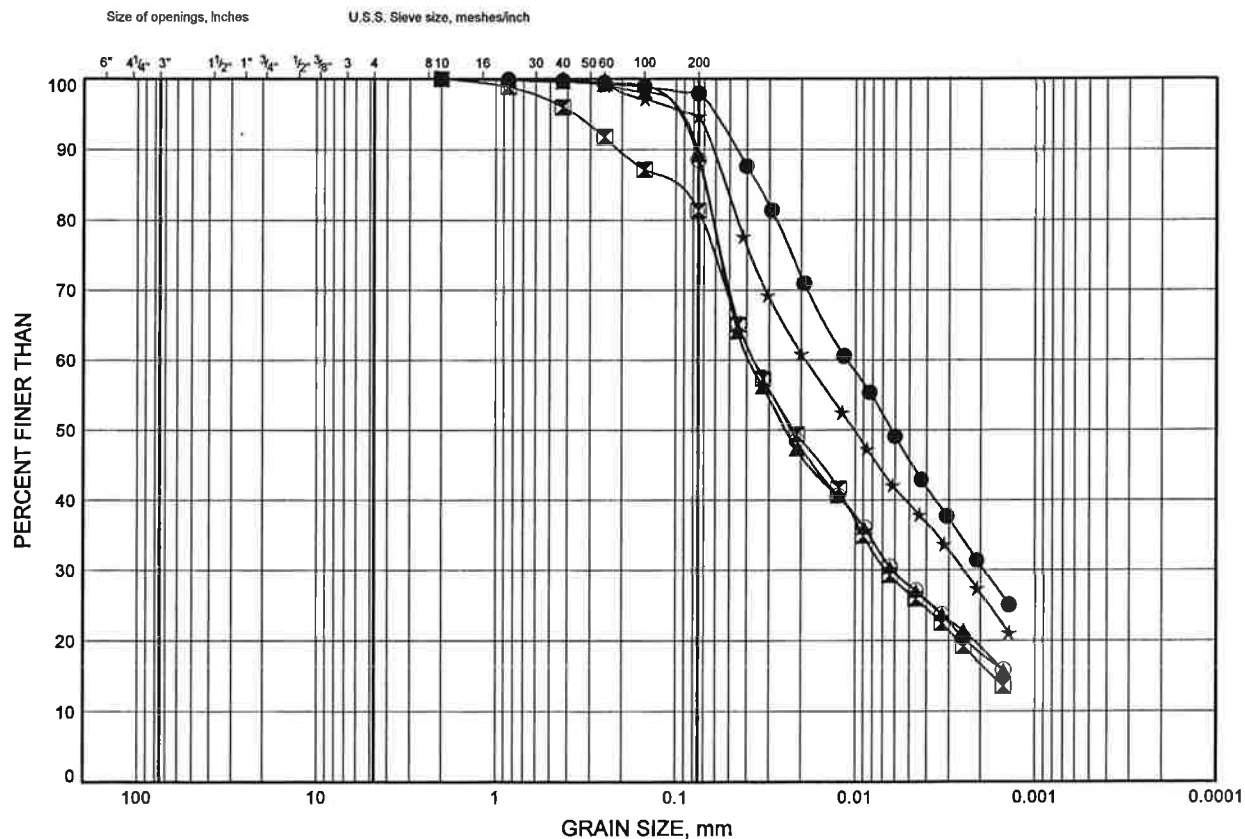


Prep'd SS  
Chkd. SP

# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B4

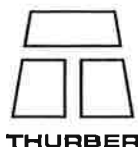
## SILTY CLAY TO CLAY (CRUST)



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	WLR-6	4.88	103.61
⊠	WLR-7	2.59	105.25
▲	WLR-8	2.59	105.23
★	WLR-9	1.83	105.44
⊙	WLR-9	3.35	103.92

Date April 2004  
Project 647-92-00

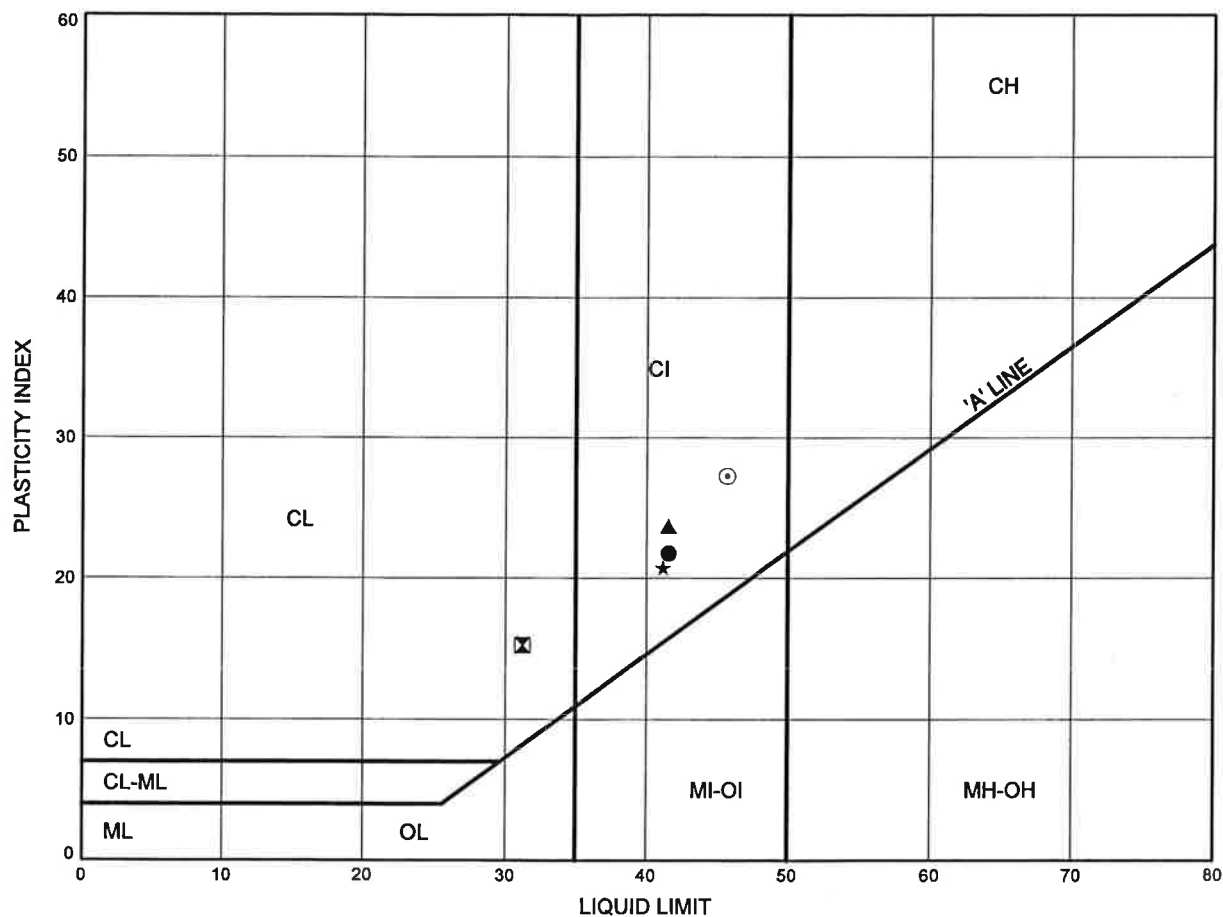


Prep'd SS  
Chkd. SP

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

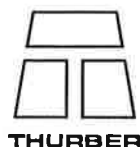
FIGURE B5

## **SILTY CLAY TO CLAY**



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	WLR-4	7.92	100.70
⊠	WLR-5	7.92	100.63
▲	WLR-6	4.88	103.61
★	WLR-6	9.45	99.04
⊙	WLR-7	4.88	102.96

Date April 2004  
 Project 647-92-00

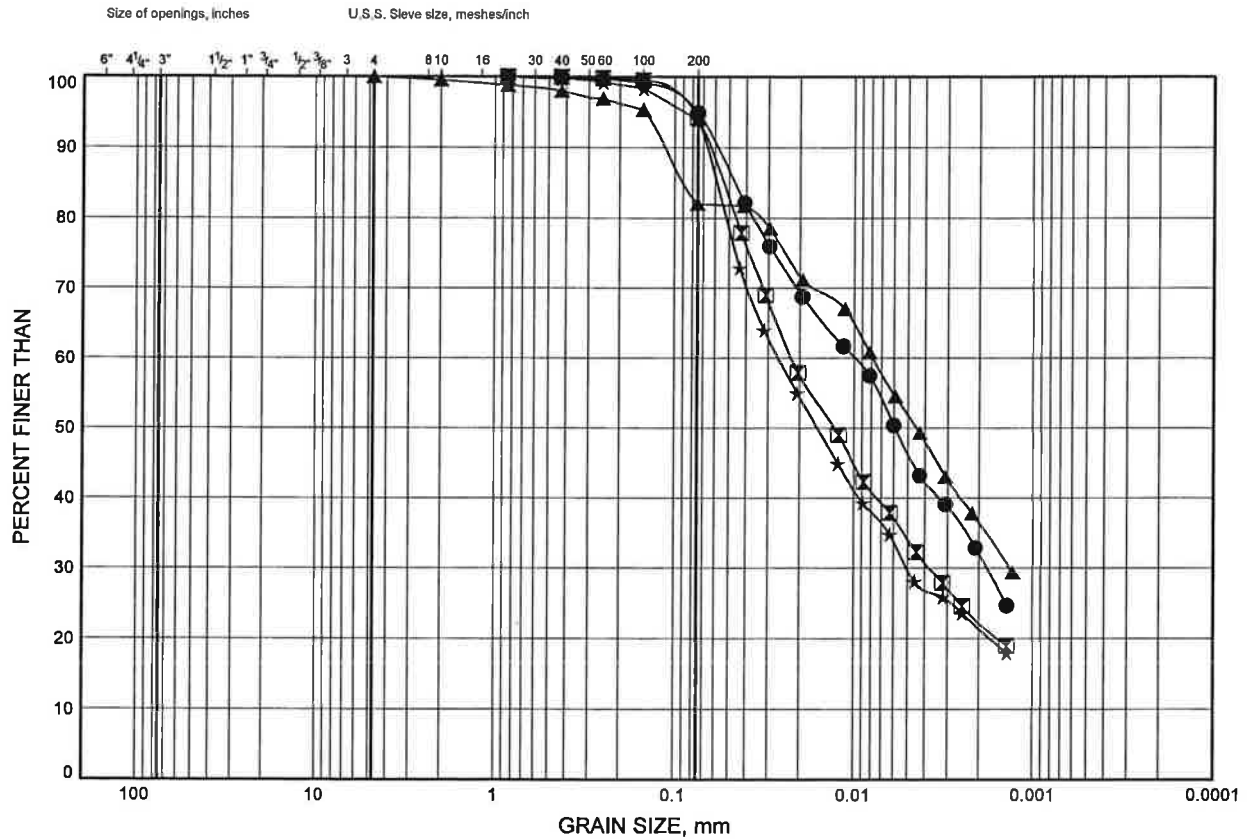


Prep'd SS  
 Chkd. SP

# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B6

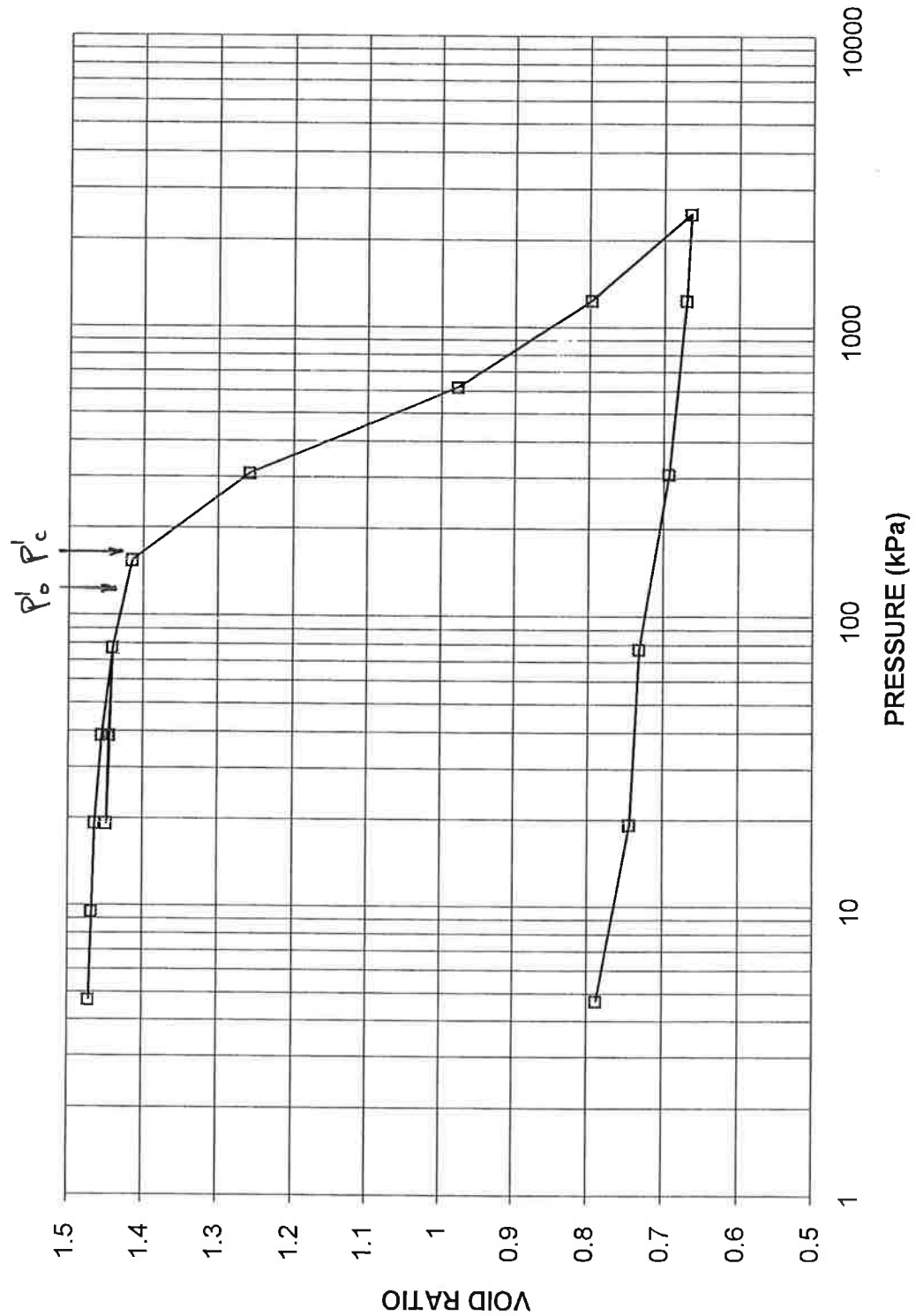
## SILTY CLAY TO CLAY



CONSOLIDATION TEST  
VOID RATIO VS. LOG PRESSURE

FIGURE B7

CONSOLIDATION TEST  
VOID RATIO vs. PRESSURE  
BH WLR03-5A ST-2



## OEDOMETER CONSOLIDATION SUMMARY

### SAMPLE IDENTIFICATION

Project Number	04-1116-026	Sample Number	ST-2
Borehole Number	WLR03-5A	Sample Depth, m	8.5-9.1

### TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	03/18/2004		
Date Completed	03/29/2004		

### SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m <sup>3</sup>	16.84
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	11.09
Area, cm <sup>2</sup>	31.65	Specific Gravity, measured	2.80
Volume, cm <sup>3</sup>	60.45	Solids Height, cm	0.772
Water Content, %	51.76	Volume of Solids, cm <sup>3</sup>	24.43
Wet Mass, g	103.79	Volume of Voids, cm <sup>3</sup>	36.03
Dry Mass, g	68.39	Degree of Saturation, %	98.3

### TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	c <sub>v</sub> cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
0.00	1.910	1.475	1.910				
4.70	1.907	1.471	1.909	15	5.15E-02	3.34E-04	1.69E-06
9.54	1.904	1.467	1.906	15	5.13E-02	3.25E-04	1.63E-06
19.29	1.901	1.463	1.903	5	1.53E-01	1.61E-04	2.42E-06
38.71	1.894	1.454	1.898	8	9.54E-02	1.89E-04	1.76E-06
77.44	1.883	1.440	1.889	15	5.04E-02	1.49E-04	7.35E-07
38.71	1.886	1.444	1.885				
19.29	1.889	1.448	1.888				
38.71	1.887	1.445	1.894	15	5.07E-02	5.39E-05	2.68E-07
77.44	1.883	1.440	1.889	10	7.56E-02	1.49E-04	4.01E-07
154.67	1.863	1.414	1.873	15	4.96E-02	1.36E-04	6.59E-07
309.92	1.742	1.257	1.803	21	3.28E-02	4.08E-04	1.31E-06
619.04	1.526	0.977	1.634	375	1.51E-03	3.66E-04	5.41E-08
1236.79	1.387	0.797	1.457	94	4.78E-03	1.18E-04	5.52E-08
2474.94	1.284	0.664	1.336	171	2.21E-03	4.36E-05	9.44E-09
1236.79	1.289	0.670	1.287				
309.92	1.306	0.692	1.298				
77.44	1.336	0.731	1.321				
19.29	1.345	0.743	1.341				
4.70	1.379	0.787	1.362				

Notes:

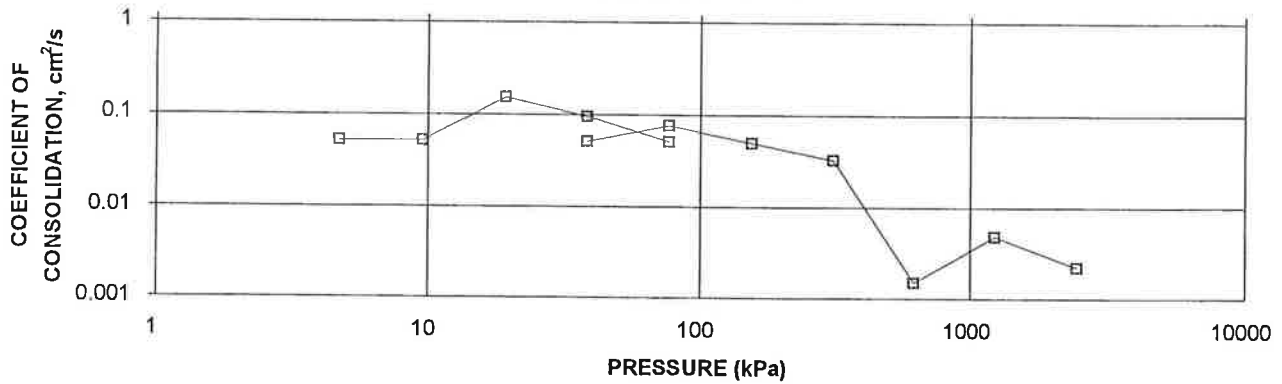
k calculated using c<sub>v</sub> based on t<sub>90</sub> values.

### SAMPLE DIMENSIONS AND PROPERTIES - FINAL

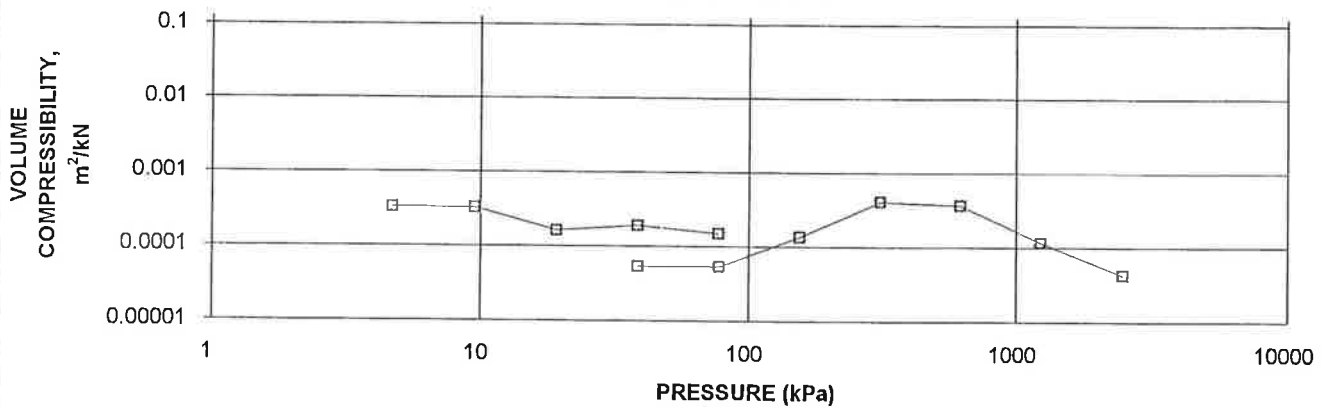
Sample Height, cm	1.38	Unit Weight, kN/m <sup>3</sup>	19.86
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	15.37
Area, cm <sup>2</sup>	31.65	Specific Gravity, measured	2.80
Volume, cm <sup>3</sup>	43.64	Solids Height, cm	0.772
Water Content, %	29.26	Volume of Solids, cm <sup>3</sup>	24.43
Wet Mass, g	88.40	Volume of Voids, cm <sup>3</sup>	19.22
Dry Mass, g	68.39		

# OEDOMETER CONSOLIDATION SUMMARY

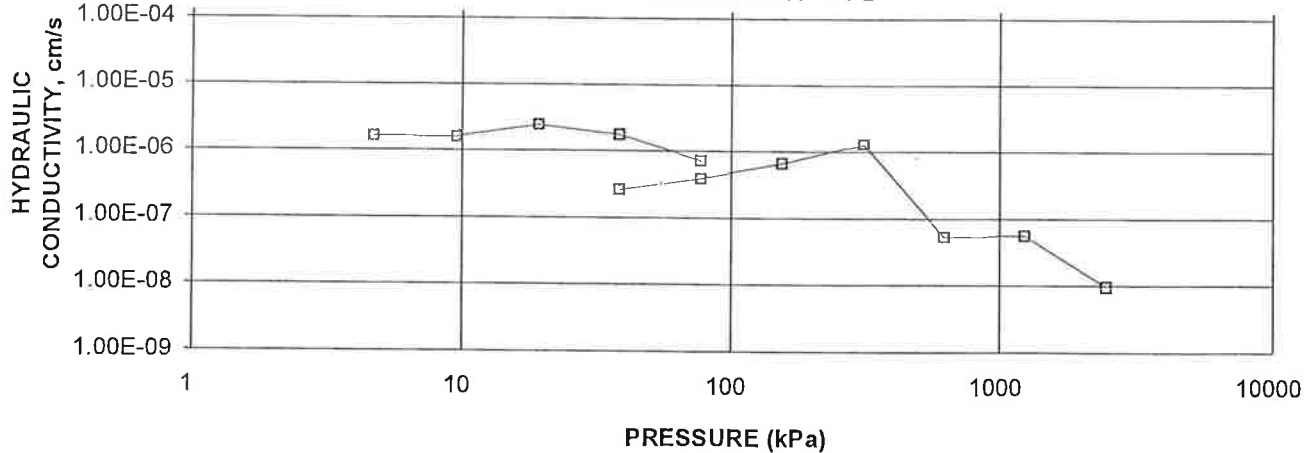
CONSOLIDATION TEST  
CV cm<sup>2</sup>/s VS PRESSURE (kPa)  
BH WLR03-5A ST-2



CONSOLIDATION TEST  
MV m<sup>2</sup>/kN vs PRESSURE (kPa)  
BH WLR03-5A ST-2



CONSOLIDATION TEST  
HYDRAULIC CONDUCTIVITY vs PRESSURE  
BH WLR03-5A ST-2



## **Appendix C**

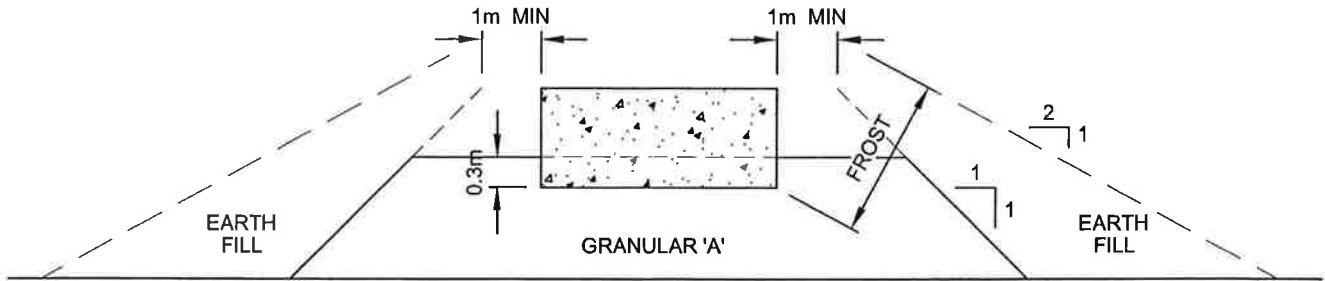
### **Foundation Alternatives**

# COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

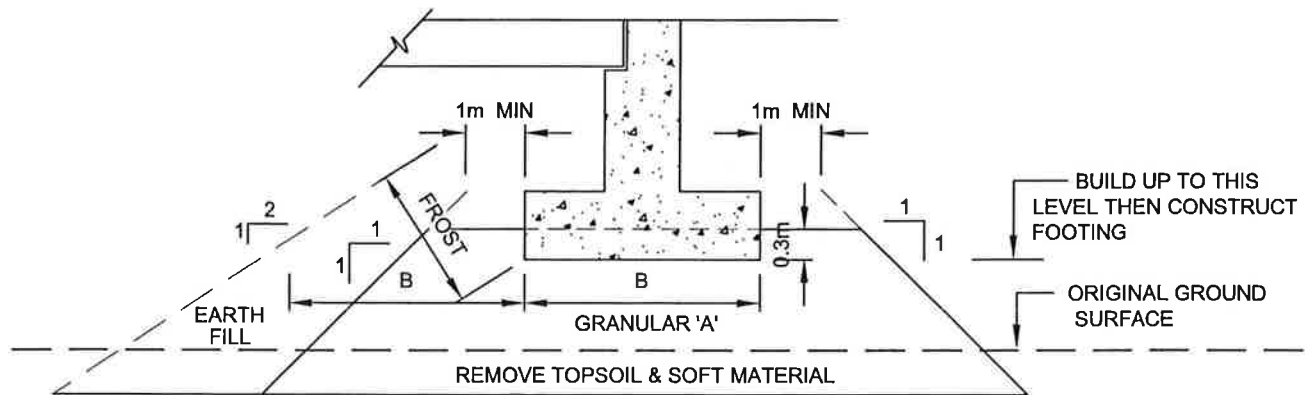
Foundation Element	Driven Piles	Spread Footing	Augered Caisson
North Abutment	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. None identified</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Shallow bedrock precludes the use of driven piles</li> <li>ii. If integral abutment is selected for this structure, pre-drilling into the bedrock is required</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Shallow and relatively thin, uniform thickness of very stiff clay across the footing</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. High cost of excavation if footings founded on bedrock</li> <li>ii. Mass concrete fill required to create a level founding surface</li> <li>iii. Footing on clay not desirable due to consolidation settlement.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High value of geotechnical resistance on bedrock</li> <li>ii. Avoids localized deep excavation to bedrock</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. None identified</li> </ul>
Centre Pier	<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. The pier foundation will be from 2.5m to 6m above the top of bedrock, effectively precluding the use of driven piles</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. Footing on clay not desirable due to consolidation settlement, especially differential settlement</li> <li>ii. Deep excavation required over a portion of the footing if it is founded on bedrock</li> <li>iii. Mass concrete fill required to create a level founding surface</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High value of geotechnical resistance on bedrock</li> <li>ii. Avoids deep excavation to bedrock</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. None identified</li> </ul>
South Abutment	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Pile lengths of 8m to 10m to bedrock allows high capacity to be achieved</li> <li>ii. Overcomes long term settlement issues due to clay consolidation</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>Downdrag on piles due to long term consolidation in the clay</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. Footing on clay not desirable due to consolidation settlement</li> <li>ii. Differential settlement between the central pier and abutment</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High value of geotechnical resistance on bedrock</li> <li>ii. Overcomes long term settlement issues due to clay consolidation</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>Higher downdrag load than piles due to larger diameter</li> </ul>

## **Appendix D**

### **Figures**



CROSS-SECTION

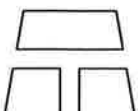


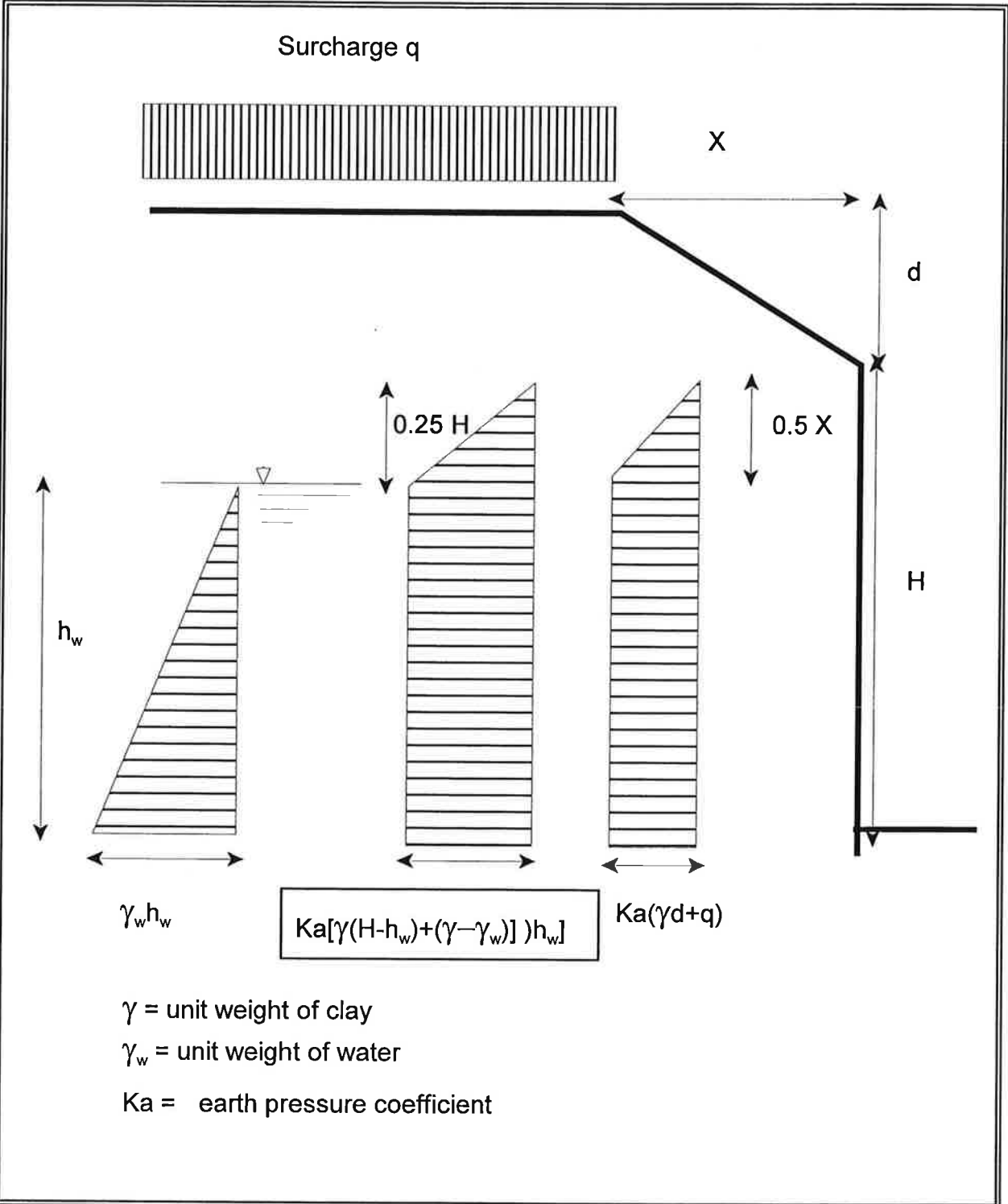
LONGITUDINAL SECTION

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	<div> <b>THURBER</b></div>
DRAWN	SS		
DATE	April , 2004		
APPROVED	PKC		
SCALE	NTS		
			DWG. NO.
			FIGURE D1



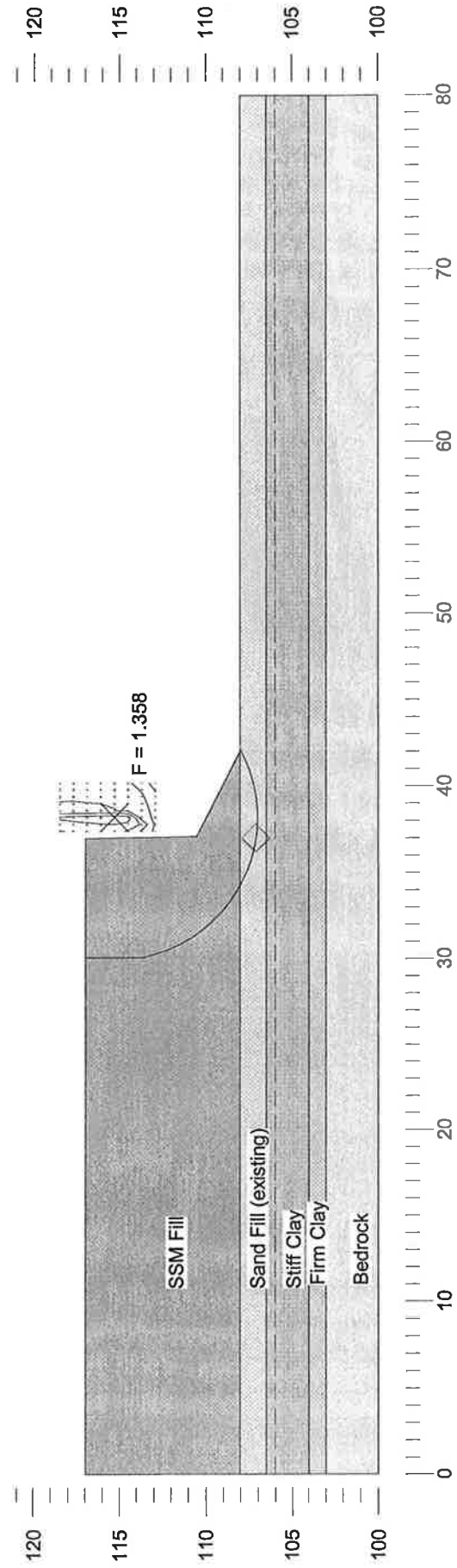
LATERAL PRESSURE DISTRIBUTION FOR  
 BRACED SHORING DESIGN

FIGURE D2

## **Appendix G**

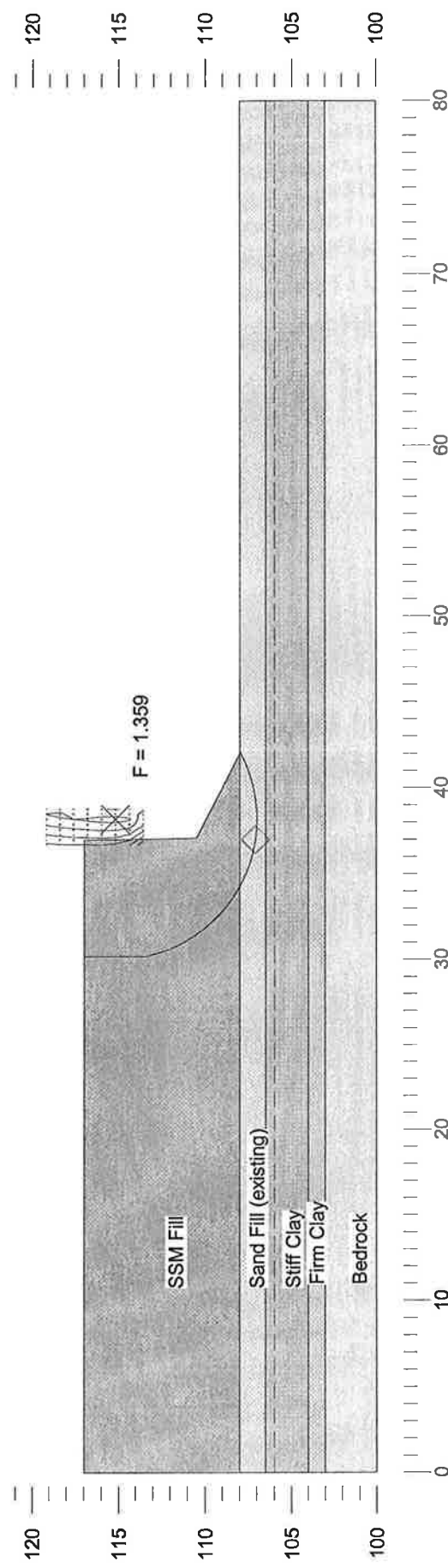
### **Selected Stability Analyses Results**

	Gamma C	Phi	Min	Piezo
	kN/m <sup>3</sup>	deg	c/p	Surf.
SSM Fill	21	0	30	0
Sand Fill	20	0	30	0
Stiff Clay	20	90	0	0
Firm Clay	20	45	0	1
Bedrock	(Infinitely Strong)			

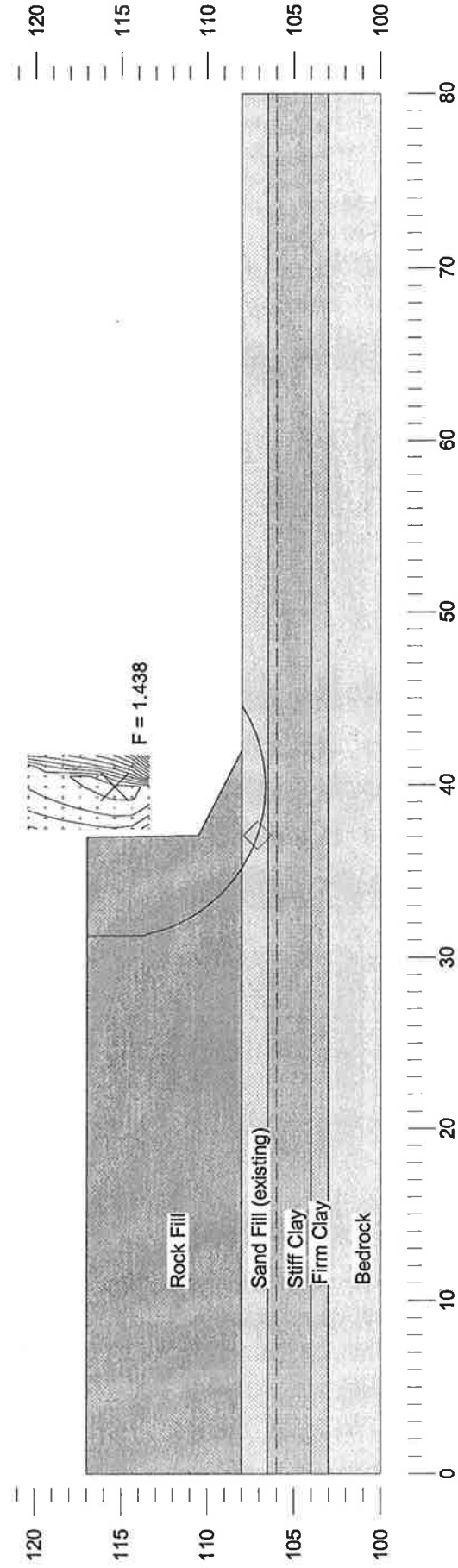


Thurber Engineering Ltd. - Toronto  
 19-3745-0  
 Highway 17 Twinning - White Lake Road  
 August 20, 2004  
 Stability of Forward Slope - North Approach  
 Figure G2 Drained Analysis - SSM Fill

	Gamma	C	Phi	Min	Plezo
	kN/m3	kPa	deg	c/p	Surf.
SSM Fill	21	0	30	0	0
Sand Fill	20	0	30	0	0
Stiff Clay	20	0	28	0	0
Firm Clay	20	0	27	0	1
Bedrock	(Infinitely Strong)				

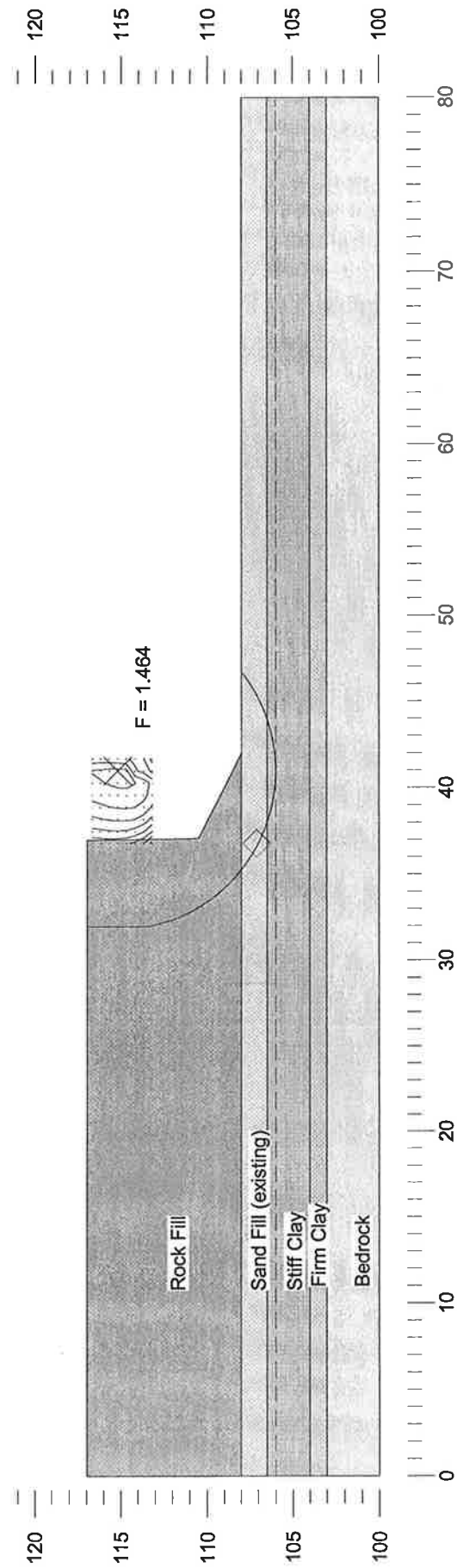


	Gamma	C	Phi	Min	Piezo
	kN/m <sup>3</sup>	kPa	deg	c/p	Surf.
Rock Fill	19	0	42	0	0
Sand Fill	20	0	30	0	0
Stiff Clay	20	90	0	0	0
Firm Clay	20	45	0	0	1
Bedrock	(Infinitely Strong)				

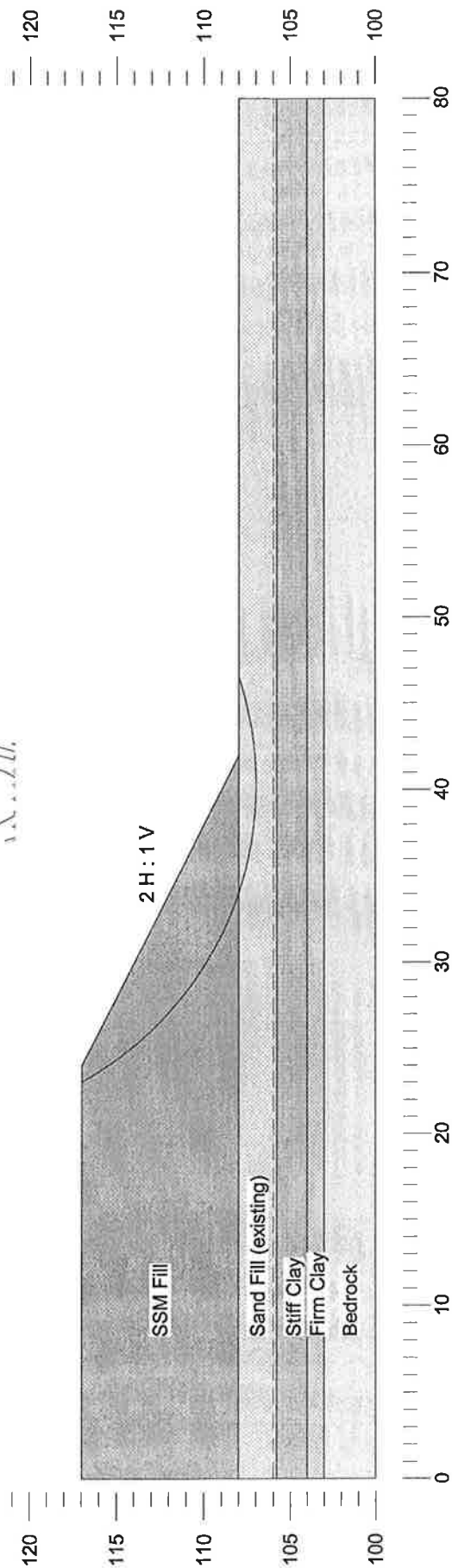
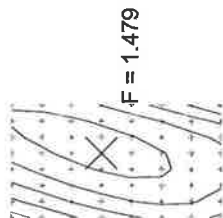


Thurber Engineering Ltd. - Toronto  
 19-3745-0  
 Highway 17 Twinning - White Lake Road  
 August 20, 2004  
 Stability of Forward Slope - North Approach  
 Figure G4 Drained Analysis - Rock Fill

	Gamma	C	Phi	Min	Piezo
	kN/m <sup>3</sup>	kPa	deg	c/p	Surf.
Rock Fill	19	0	42	0	0
Sand Fill	20	0	30	0	0
Stiff Clay	20	0	28	0	0
Firm Clay	20	0	27	0	1
Bedrock	(Infinitely Strong)				

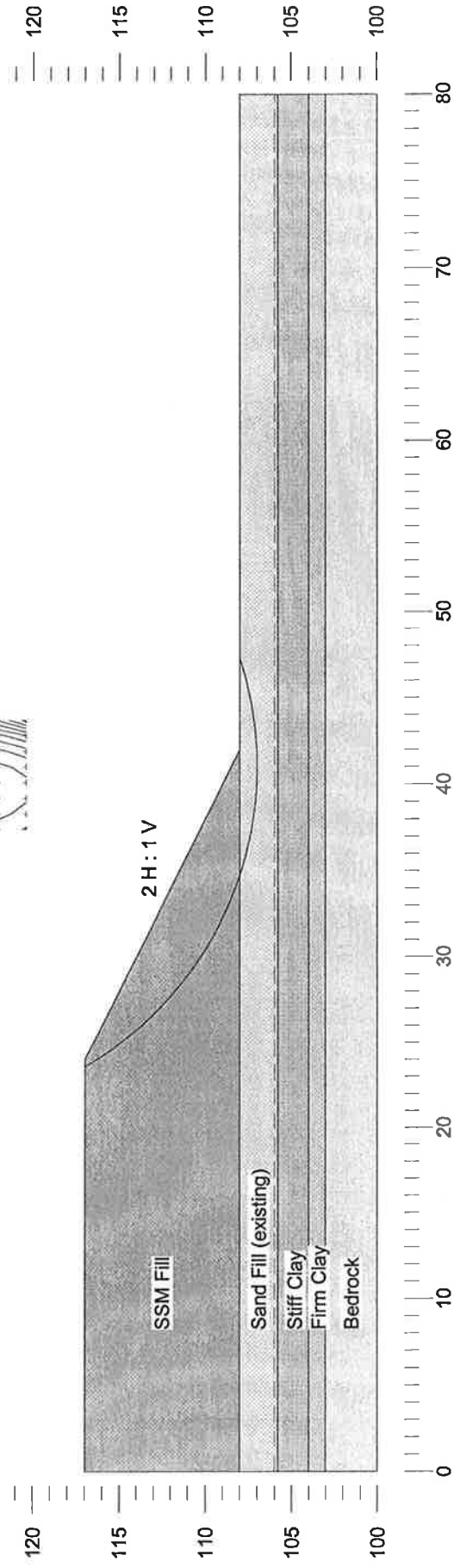
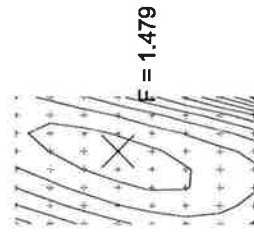


	Gamma	C	Phi	Min	Piezo
	kN/m <sup>3</sup>	kPa	deg	c/p	Surf.
SSM Fill	21	0	30	0	0
Sand Fill	20	0	30	0	0
Stiff Clay	20	90	0	0	0
Firm Clay	20	45	0	0	1
Bedrock	(Infinitely Strong)				

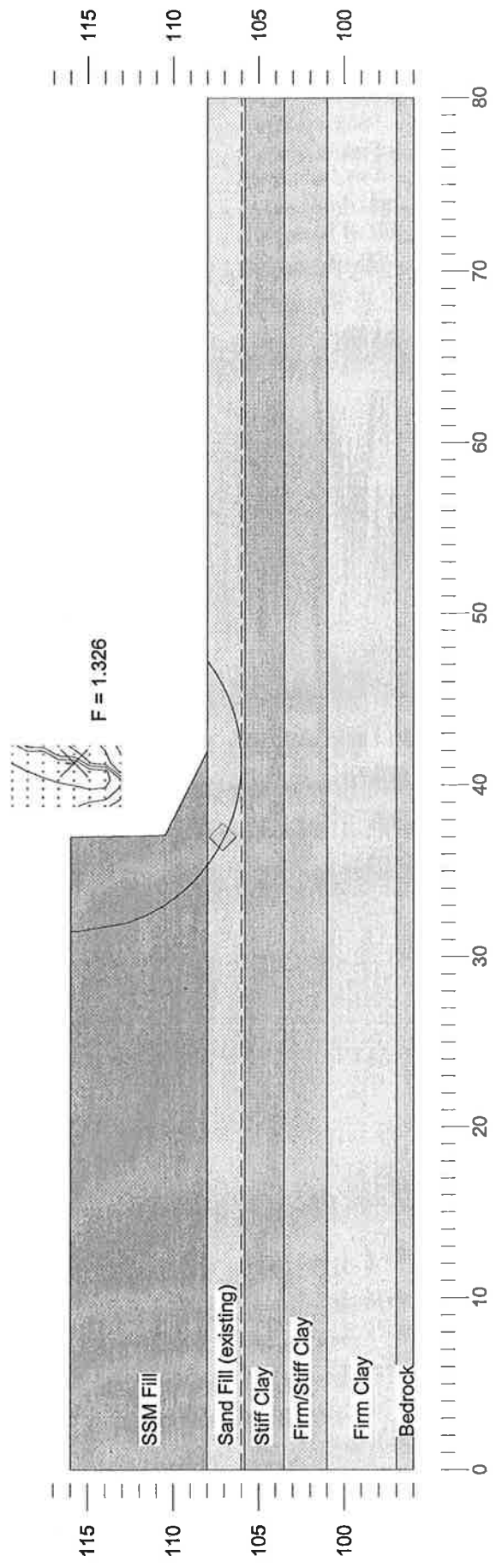


Thurber Engineering Ltd. - Toronto  
 19-3745-0  
 Highway 17 Twinning - White Lake Road  
 August 20, 2004  
 Stability of Side Slope - North Approach  
 Figure G6 Drained Analysis - SSM Fill

	Gamma	C	Phi	Min	Piezo
	kN/m <sup>3</sup>	kPa	deg	c/p	Surf.
SSM Fill	21	0	30	0	0
Sand Fill	20	0	30	0	0
Stiff Clay	20	0	28	0	1
Firm Clay	20	0	27	0	1
Bedrock	(Infinitely Strong)				

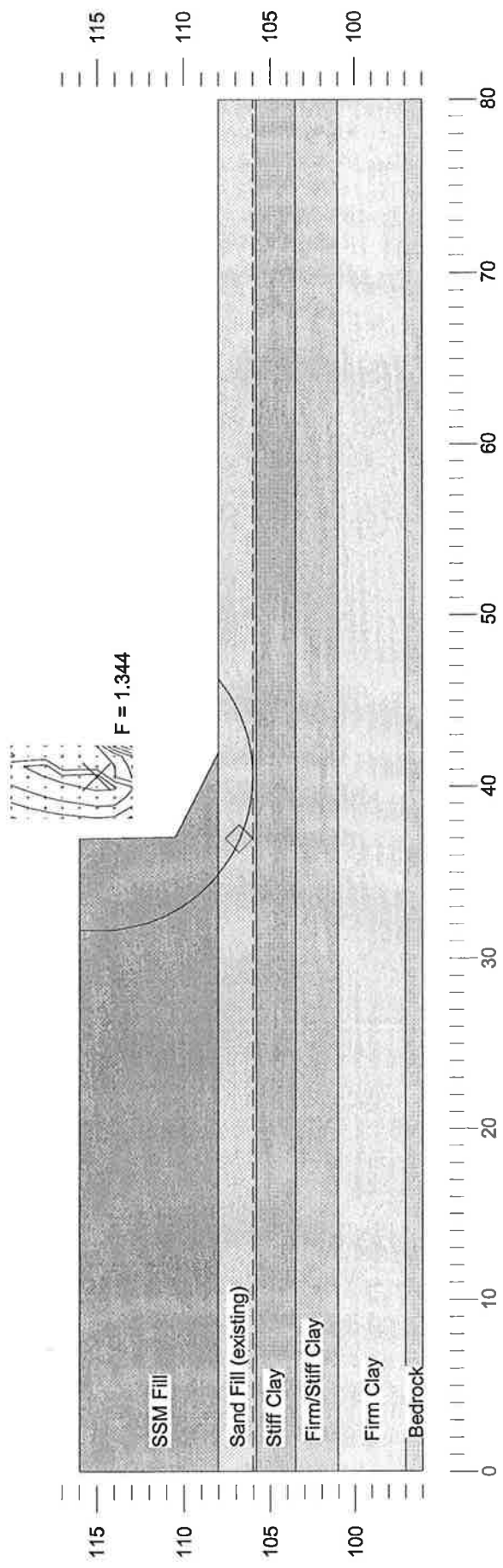


	Gamma C	Phi	Min	Piezo
	kN/m <sup>3</sup>	deg	c/p	Surf.
SSM Fill	21	0	30	0
Sand Fill	20	0	30	0
Stiff Clay	20	90	0	0
Firm/Stiff Clay	20	45	0	1
Firm Clay	20	35	0	1
Bedrock	(Infinitely Strong)			

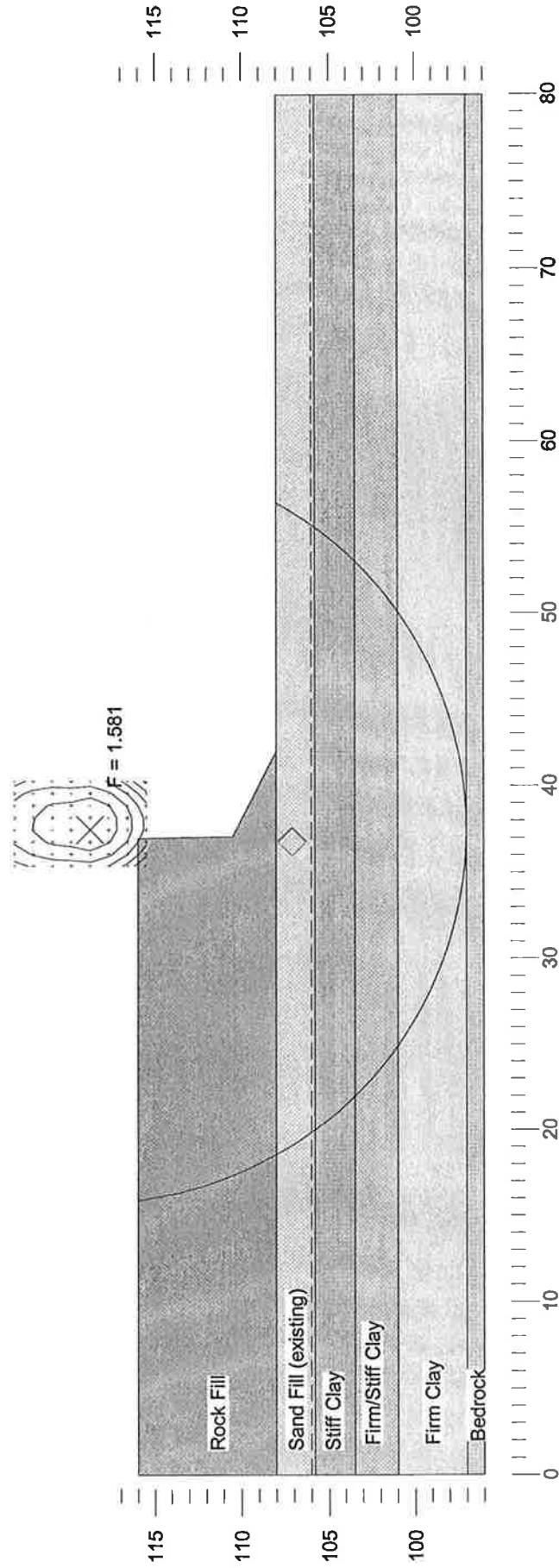


Thurber Engineering Ltd. - Toronto  
 19-3745-0  
 Highway 17 Twinning - White Lake Road  
 August 20, 2004  
 Stability of Forward Slope - South Approach  
 Figure G8 Drained Analysis - SSM Fill

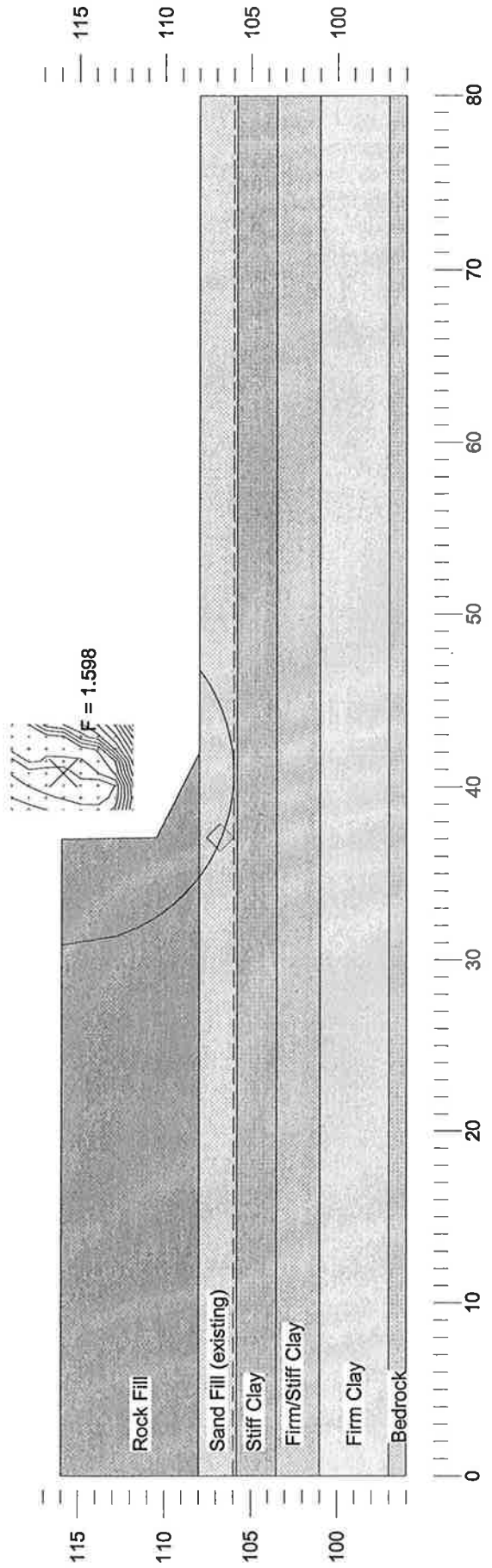
	Gamma	C	Phi	Min	Piezo
	kN/m3	kPa	deg	c/p	Surf.
SSM Fill	21	0	30	0	0
Sand Fill	20	0	30	0	0
Stiff Clay	20	0	28	0	0
Firm/Stiff Clay	20	0	27	0	1
Firm Clay	20	0	27	0	1
Bedrock	(Infinitely Strong)				



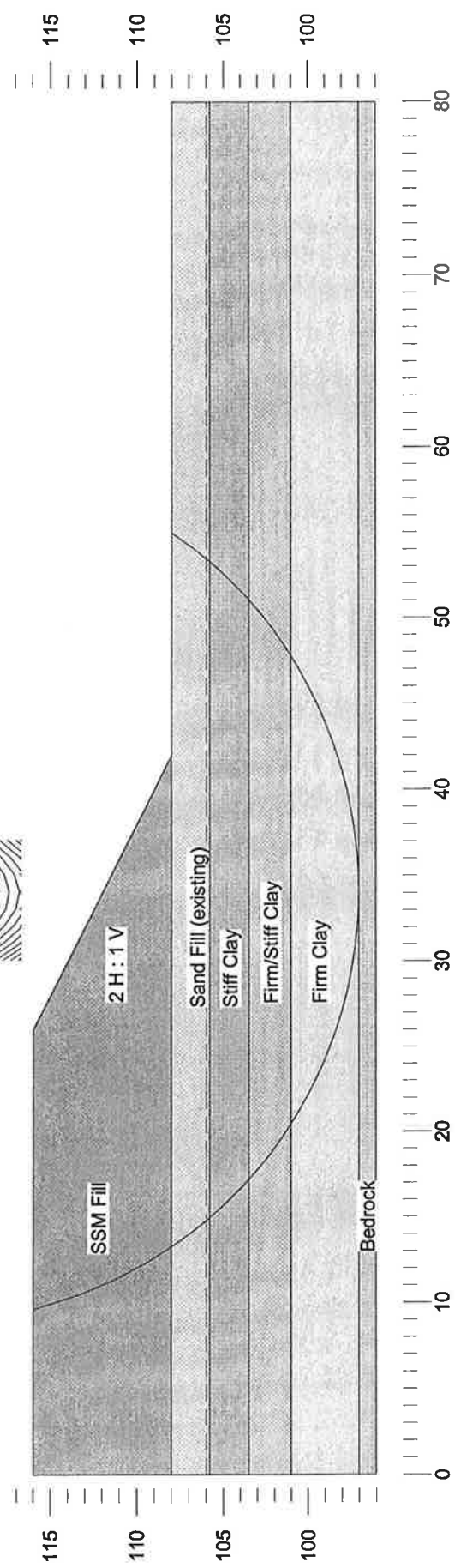
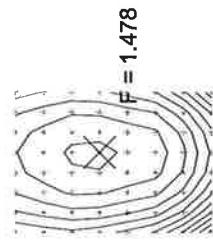
	Gamma C	Phi	Min	Piezo
	kN/m <sup>3</sup>	deg	c/p	Surf.
Rock Fill	19	0	42	0
Sand Fill	20	0	30	0
Stiff Clay	20	90	0	0
Firm/Stiff Clay	20	45	0	1
Firm Clay	20	35	0	1
Bedrock	(Infinitely Strong)			



	Gamma	C	Phi	Min	Piezo
	kN/m <sup>3</sup>	kPa	deg	c/p	Surf.
Rock Fill	19	0	42	0	0
Sand Fill	20	0	30	0	0
Stiff Clay	20	0	28	0	0
Firm/Stiff Clay	20	0	27	0	1
Firm Clay	20	0	27	0	1
Bedrock	(Infinitely Strong)				

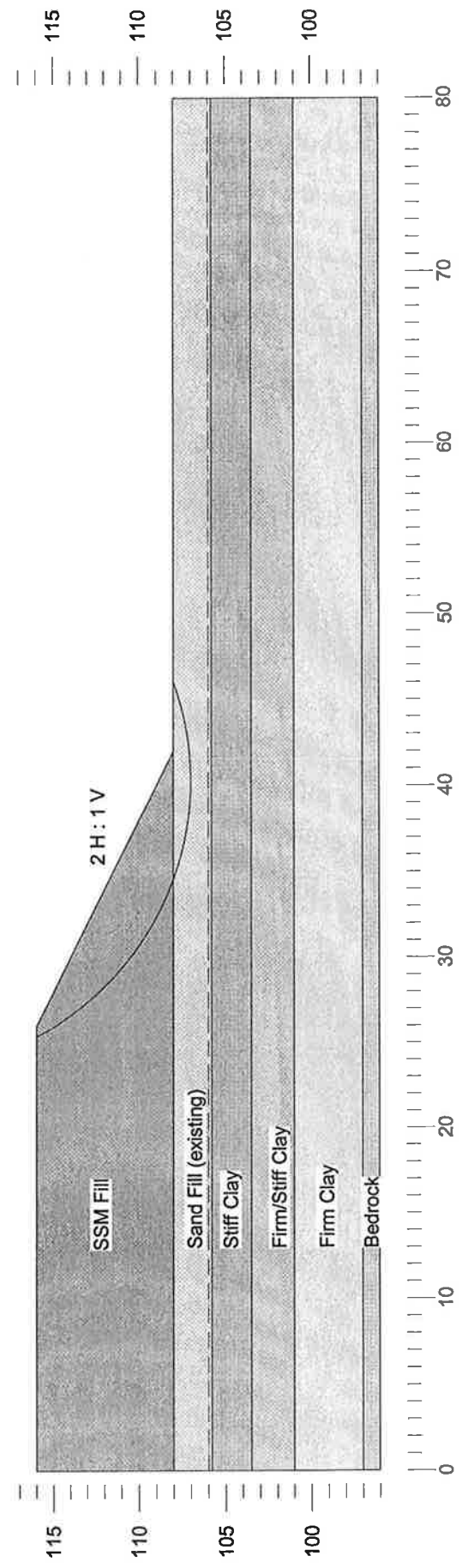
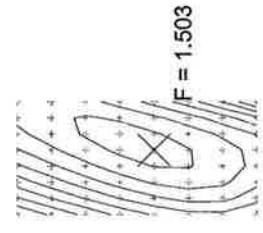


	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
SSM Fill	21	0	0	0
Sand Fill	20	0	0	0
Stiff Clay	20	90	0	1
Firm/Stiff Clay	20	45	0	1
Firm Clay	20	35	0	1
Bedrock	(Infinitely Strong)			



Thurber Engineering Ltd. - Toronto  
 19-3745-0  
 Highway 17 Twinning - White Lake Road  
 August 20, 2004  
 Stability of Side Slopes - South Approach  
 Figure G12 Drained Analysis - SSM Fill

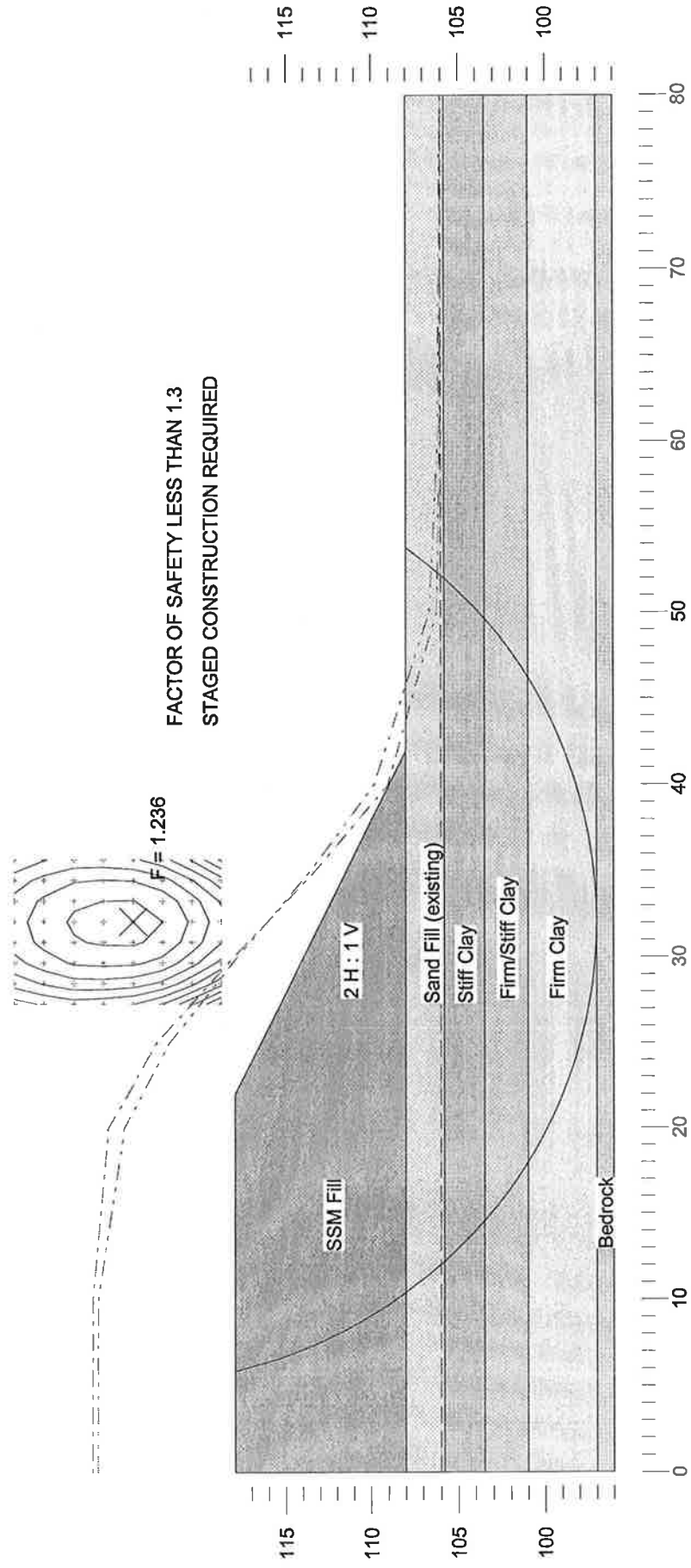
	Gamma C	Phi	Min	Piezo
	kN/m <sup>3</sup>	deg	c/p	Surf.
SSM Fill	21	0	0	0
Sand Fill	20	0	0	0
Stiff Clay	20	0	0	1
Firm/Stiff Clay	20	0	0	1
Firm Clay	20	0	0	1
Bedrock	20	0	0	1
	(Infinitely Strong)			



Stability of Side Slopes - South Approach (8 m + 2 m surcharge)  
Figure G13 Undrained Analysis - SSM Fill (excess pore water pressure development)

	Gamma C	Phi	Min	Piezo
	kN/m <sup>3</sup>	deg	c/p	Surf.
SSM Fill	21	0	0	0
Sand Fill	20	0	0	0
Stiff Clay	20	90	0	1
Firm/Stiff Clay	20	45	0	3
Firm Clay	20	35	0	4
Bedrock	(Infinitely Strong)			

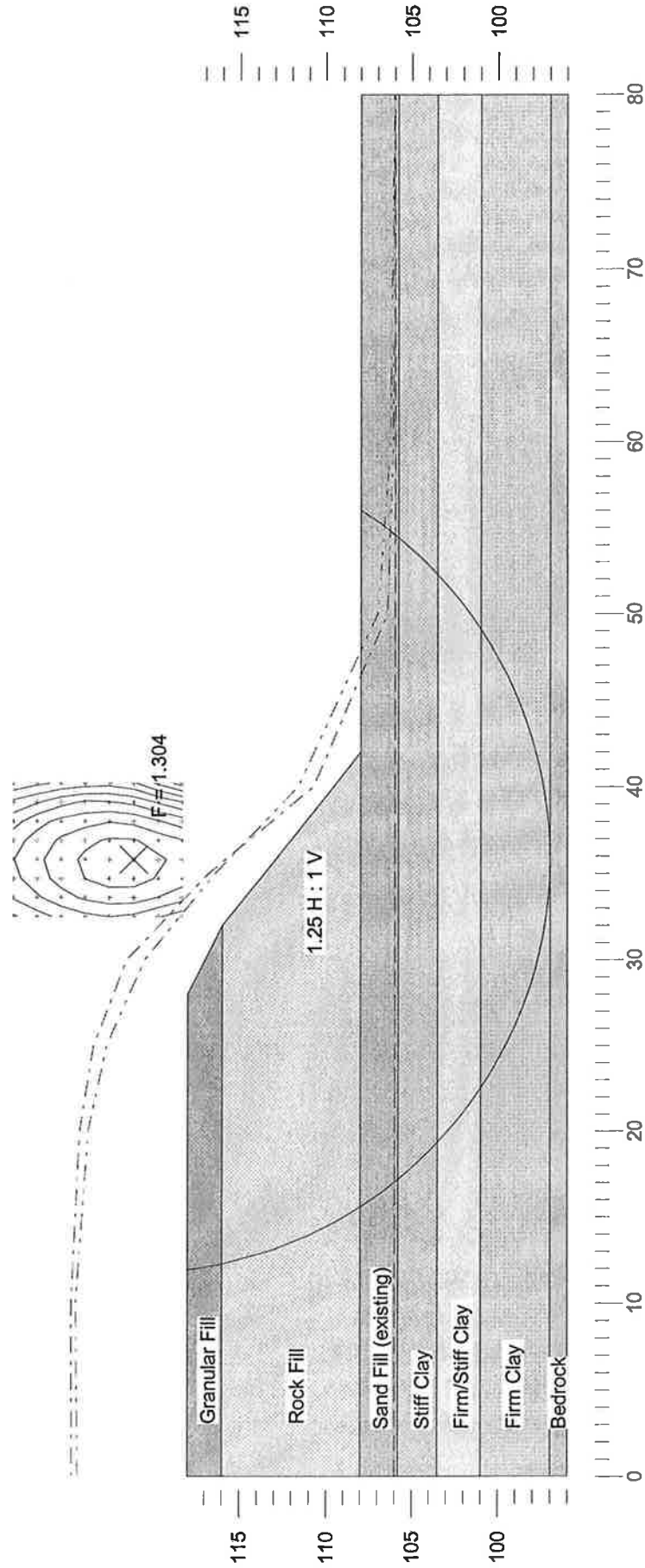
Pore pressure parameter,  $B = 0.95$  (initial fill placement)



Stability of Side Slopes - South Approach (8 m + 2 m surcharge)  
 Figure G14 Undrained Analysis - Rock Fill (excess pore water pressure development)

	Gamma C	Phi	Min	Piezo
	kN/m <sup>3</sup>	deg	c/p	Surf.
Granular Fill	22	0	0	0
Rock Fill	19	0	0	0
Sand Fill	20	0	0	0
Stiff Clay	20	90	0	1
Firm/Stiff Clay	20	45	0	3
Firm Clay	20	35	0	4
Bedrock	(Infinitely Strong)			

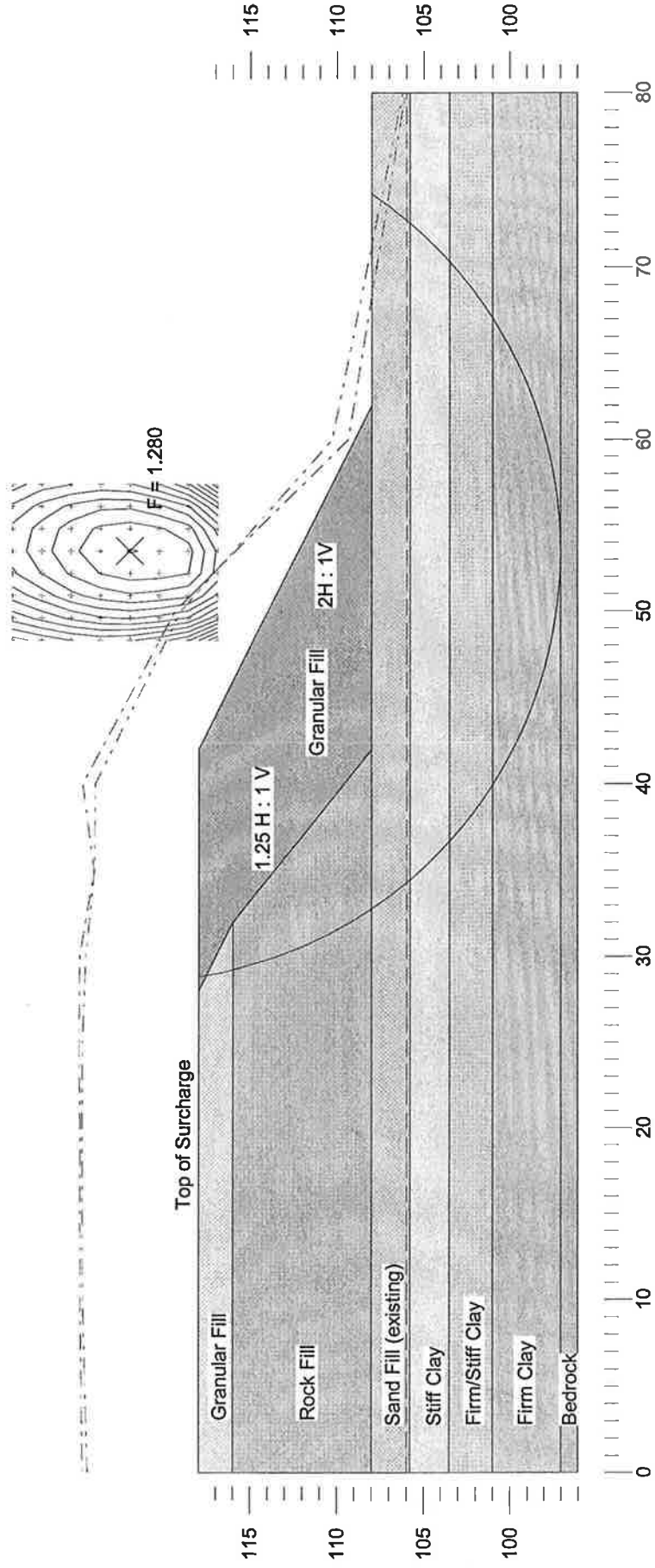
Pore pressure parameter,  $B = 0.95$  (initial fill placement)



Stability of Forward Slopes - South Approach (8 m + 2 m surcharge)  
Figure G15 Undrained Analysis - Rock Fill (excess pore water pressure development)

	Gamma	C	Phi	Min	Piezo
	kN/m <sup>3</sup>	kPa	deg	c/p	Surf.
Granular Fill	22	0	32	0	0
Granular Fill	22	0	32	0	0
Rock Fill	19	0	42	0	0
Sand Fill	20	0	30	0	0
Stiff Clay	20	90	0	0	1
Firm/Stiff Clay	20	45	0	.22	3
Firm Clay	20	35	0	.22	4
Bedrock	(Infinitely Strong)				

Pore pressure parameter,  $B = 0.95$  (initial fill placement)



## **Appendix E**

### **Special Provisions**

**EARTH EXCAVATION FOR STRUCTURE - Item No.**  
**ROCK EXCAVATION FOR STRUCTURE - Item No.**  
**UNWATERING STRUCTURE EXCAVATION - Item No.**  
**CLAY SEAL - Item No.**

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Special Provision No. 902S01

September 2003

Excavation and Backfilling-Structures

**902.02 REFERENCES**

Section 902.02 of OPSS 902, December, 1983, is amended by the addition of the following:

OPSS 510

**902.03 DEFINITIONS**

Section 902.03 of OPSS 902, December, 1983, is amended by the addition of the following:

**Quality Verification Engineer:** means an Engineer with a minimum of five (5) years experience related to excavation and backfilling of structures, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and issue of certificate(s) of conformance.

**902.04 SUBMISSION AND DESIGN REQUIREMENTS**

Section 902.04 of OPSS 902, December, 1983, is deleted and replaced with the following:

**902.04.01 Site Survey**

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

**902.04.02 Working Drawings**

Working drawings for protection systems shall be according to OPSS 539.

Where unwatering is required, the Contractor shall be responsible for the design of the unwatering scheme for the intended purpose. The design of temporary structures or protection system for unwatering shall be according to OPSS 539.

**902.04.03 Submission of Certificate of Conformance**

The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer upon completion of each of the following operations and prior to commencement of each subsequent operation:

- Excavation for Foundation
- Excavation for Backfill and Frost Tapers
- Use of Excavated Material
- Unwatering of Excavation for Structure
- Backfilling

The Certificate of Conformance shall state that the work has been carried out in general conformance with the contract documents, specifications and/or stamped working drawings.

#### **902.05.03                      Backfill**

Subsection 902.05.03 is amended by the addition of the following:

The Contractor shall be responsible for ensuring the quality of the material used for backfill. The quality of the material shall be verified by test results from a qualified and recognized testing laboratory. The frequency of sampling and testing shall be according to ASTM D75-87 and D3665.

#### **902.05.04                      Protection System**

Section 902.05 of OPSS 902, December, 1983, is amended by the addition of the following:

Protection systems shall be according OPSS 539.

#### **902.07.01                      Protection Schemes**

Subsection 902.07.01 of OPSS 902, December, 1983, is amended by replacing the word "Engineer" in the last paragraph with the words "Contract Administrator".

#### **902.07.02                      Excavation**

Subsection 902.07.02 of OPSS 902, December, 1983, is deleted and replaced with the following:

##### **902.07.02.01                      General**

For excavation, the Contractor shall be responsible for preventing any deterioration of the foundation soil or rock, surface water from entering and eroding the face of the excavation, and build up of hydrostatic pressures which may have harmful effects upon the temporary or permanent structures.

##### **902.07.02.02                      Excavation for Foundation**

The excavation for foundation shall be inspected and approved by the Quality Verification Engineer prior to construction of the footing. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator.

The Contractor shall be responsible for maintaining the stability of the excavation if any excavation below stream or channel bed is carried out.

The Contractor shall be responsible for restoring the over excavated area to its original conditions. For over excavation in earth, the backfill material shall be granular material such as Granular A or B compacted according to OPSS 501. For over excavation in rock, concrete shall be placed to achieve the original excavation limits. The concrete shall be of the same class concrete as the element it supports.

The Contractor shall be responsible for all additional costs due to excavation beyond the required tolerance limits, including but not limited to additional structure design, granular materials, concrete, reinforcing steel and retention of the services of a blasting consultant.

#### **902.07.02.03                      Excavation for Backfill and Frost Tapers**

Excavation for backfill and frost tapers shall be carried out according to the specifications and details shown on the contract drawings. The Contractor shall be responsible for restoring the over excavated portion with backfill and shall be compacted according to OPSS 501.

The excavation for backfill and frost tapers shall be inspected and approved by the Quality Verification Engineer prior to placement of fill material. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator.

#### **902.07.02.04                      Preservation of Channel**

The Contractor shall be responsible for restoring the channel back to its original conditions unless otherwise specified in the contract.

#### **902.07.02.05                      Removals**

Removal of pavement, curb and gutter, and sidewalks shall be according to OPSS 510.

#### **902.07.03                          Unwatering Structure Excavation**

Subsection 902.07.03 of OPSS 902, December, 1983, is amended by replacing the first paragraph with the follows:

The Contractor shall carry out all work necessary to prevent disturbance to the founding material. Concrete shall be placed in the dry, unless otherwise specified in the contract.

After the unwatering, the excavation shall be inspected and approved by the Quality Verification Engineer prior to construction of the footing. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator.

#### **902.07.04                          Backfilling**

Subsection 902.07.04 of OPSS 902, December, 1983, is deleted and replaced with the following:

The Contractor shall ensure that the concrete has reached at least 70 percent of its design strength before placing the backfill against an abutment, wingwall, retaining wall or concrete culvert.

Backfilling shall be according to OPSS 501.

The backfilling operation shall be inspected and approved by the Quality Verification Engineer. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator.

## **902.09 Measurement for Payment**

### **902.09.01 Structures**

Subsection 902.09.01 of OPSS 902, is amended by deleting the first five paragraphs and replacing them with the following:

"Earth Excavation for Structure" and "Rock Excavation for Structure" applies to the specific structure(s) designated, i.e., Bridge, Retaining Wall or Concrete Culvert, and is measured by Plan Quantity, as may be revised by Adjusted Plan Quantity, of the volume in cubic metres below the designated payment surface.

The above measurement also includes, where applicable, the excavation quantities, below the designated payment surface, for placing granular backfill and for placing the granular frost tapers.

For open footing culverts, the above measurement also includes the excavation quantities below the designated payment surface but between the plan areas of the footings and above the stream bed or the top of the footings, whichever is higher.

Where the structure excavation overlaps excavation required for other work, deductions will not be made to the structure excavation measurement.

## **902.10 Basis of Payment**

### **902.10.01 Excavation and Backfill**

Subsection 902.10.01 of OPSS 902 is amended by deleting the first paragraph and replacing it with the following:

Payment at the contract price(s) for the tender item(s) "Earth Excavation for Structure" and "Rock Excavation for Structure" shall be full compensation for all labour, equipment and material for all excavation required, for removal of pavement, curb and gutter and sidewalk except where there is a separate item for removal of pavement, curb and gutter and sidewalk which overlaps pavement, curb and gutter and sidewalk removal required for structure excavation, protection of adjacent works, unwatering, backfilling and compacting around the footing according to subsection 902.07.04, placing and compacting of suitable material in fill in accordance with OPSS 206 and management of any surplus or unsuitable excavated material, including the cost of disposal areas, all according to the requirements of this specification.

**WARRANT:** Always with these tender items.

## **AMENDMENT TO OPSS 120, AUGUST, 1994**

---

### **Special Provision**

---

OPSS 120, August 1994, General Specification For The Use of Explosives is deleted in its entirety and replaced with the following:

#### **Construction Special Provision for Rock Excavation Utilizing Blasting**

##### **120.01 SCOPE**

This special provision describes the conditions under which explosives are to be used on the Contract.

##### **120.02 REFERENCES**

This special provision refers to the following standards, special provisions or publications:

###### **Canadian Standards Association:**

CAN3-Z107.54-M85(R 1999) – Procedure for Measurement of Sound and Vibration Due to Blasting Operations

###### **Ministry of Transportation Publications:**

Ontario Traffic Manual Book 7

###### **Federal Government Publication:**

Explosives Act (Canada)

###### **Department of Fisheries and Oceans Publication:**

Guidelines for the Use of Explosives in or near Canadian Fisheries Waters, 1998

##### **120.03 DEFINITIONS**

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

**Blaster:** means a competent person knowledgeable and experienced in the handling, use and storage of explosives and their effect on adjacent property and persons

**Blasting Consultant:** means an Engineer, with a minimum of five (5) years experience related to blasting design, blasting operations and monitoring of rock excavation blasting. The Blasting Consultant shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and issue of certificate(s) of conformance.

**Fugitive Flyrock:** means broken rock that is dislodged from the parent bedrock formation during the blasting operation and escapes the blasting rock control measures

**Peak Particle Velocity(PPV):** means the maximum speed that ground particles move as a result of energy released from explosive detonations. PPV is measured in millimeters per second.

**Pre-Blast Survey:** means a detailed record, in written form accompanied by film or video, of the condition of private or public property, prior to the commencement of blasting operations.

**Rock Excavation:** means the removal of material from solid masses of igneous, sedimentary or metamorphic rock which prior to removal was integral with the parent mass and the removal of boulders and rock fragments larger than 1.0 cubic metre in volume.

## **120.04 SUBMISSION AND DESIGN REQUIREMENTS**

### **120.04.01 General**

The Contractor shall submit the following to the Contract Administrator a minimum of five (5) working days prior to the use of explosives:

- a) The name and qualifications of the blasting contractor.
- b) The names of the blasting supervisor and blasters in charge of the blasting including a record of their experience and safety training.
- c) A detailed plan of the blasting operation and schedule of excavation.
- d) Blasting Permits, Approvals and/or Agreements
- e) Name and qualifications of the Blasting Consultant.
- f) A blasting design and blasting vibration and noise monitoring program.
- g) A pre-blast survey.
- h) A trial blasting program.

### **120.04.02 Blasting Design and Monitoring**

#### **120.04.02.01 Blasting Consultant**

The Contractor shall retain the services of a Blasting Consultant to produce a blasting design and blasting vibration and noise monitoring program.

#### **120.04.02.02 Blasting Design**

A blasting design shall be submitted to the Contract Administrator and shall include but not be limited to the following:

- a) Design peak particle velocity (PPV) and design peak sound pressure level.
- b) Number, pattern, orientation, size and depth of drill holes

- c) Collar and toe load; number and time of delays, distribution of explosives per hole, per delay and per blast and the sequence and pattern of the delays.
- d) Setback distances to affected waterbodies, substrate type and the explosive products to be used.

The following shall be submitted to the Contract Administrator 48 hrs prior to the use of explosives:

- a) A letter signed by the Blast Consultant indicating the areas for which a blast design has been completed.
- b) A letter signed by the Blaster indicating receipt of the blast design and agreement that the blasting shall be according to the design.

#### **120.04.02.03                      Blasting Monitoring**

A detailed blasting monitoring program to monitor ground vibrations, water pressure and noise for each blast shall be produced and submitted to the Contract Administrator. The blasting monitoring shall be carried out by the Blasting Consultant

The blasting monitoring equipment shall meet the following requirements:

- a) Be capable of measuring and recording ground vibrations up to 200 mm/s of peak particle velocity (PPV) in each of three (3) mutually perpendicular directions.
- b) Be capable of measuring and recording frequency in all three (3) mutually perpendicular directions.
- c) Be capable of monitoring impact noise levels from the blasting up to a minimum 142 dbf.
- d) Instrumentation shall have been calibrated within six (6) months prior to commencement of blasting operations. Proof of calibration shall be submitted to the Contract Administrator five(5) working days prior to the commencement of blasting operation.

#### **120.04.03                      Blasting Permits, Approvals and/or Agreements**

Any blasting permits, approvals and/or agreements shall be obtained by the Contractor and submitted to the Contract Administrator at no additional cost to the Owner.

#### **120.04.04                      Pre-Blast Survey**

Prior to any blasting operation, the Contractor shall conduct a pre-construction inspection of all buildings, structures, utilities, residences and facilities within a minimum 100 m radius of the location where explosives are to be used and submit a pre-blast survey certificate. The pre-blast survey certificate shall be signed by the Blasting Consultant and shall be submitted to the Contract Administrator 48 hours prior to the use of explosives.

A formal request for permission to carry out an inspection with an explanatory letter to the owner shall be produced. In the event that a property owner refuses to permit inspection, the property owner shall be asked to sign a refusal to inspect release form and the Contract Administrator shall be informed prior to the use of explosives.

The pre-blast survey report shall include as a minimum the following information:

- a) Type of structure or utility, including type of construction and the date, if possible, when built.
- b) Any differential settlements: visible cracks in walls, floors and ceiling shall be identified and described, including a diagram, if applicable.
- c) Any other apparent structural or cosmetic damage or defect.
- d) As a minimum, clear quality photographs, as deemed necessary for proper recording of significant concerns.

#### **120.04.05                      Trial Blasting**

The Contractor shall undertake trial blasting based on the proposed blasting design as produced by the Blasting Consultant to demonstrate that the blast design satisfies the vibration and noise level limits specified in the contract.

The Contractor shall carry out a series of a minimum of three (3) trial blasts based on the Contractor's blasting design. These trial blasts shall be carried out at a location selected by the Contractor and approved by the Contract Administrator.

The trial blasting shall be carried out with the appropriate monitoring of blast vibrations and noise levels in the presence of the Contract Administrator. A report summarizing the trial blast monitoring results shall be submitted to the Contract Administrator signed and sealed by the Blasting Consultant.

The Contractor shall review the trial blast results and revise the blasting design as required to ensure satisfactory levels of blast vibrations, noise, shatter depth and to prevent flyrock.

#### **120.04.06                      Post Blast Survey and Reporting**

Following the blasting operation, the Contractor shall conduct a post-construction inspection of all buildings, structures, utilities, residences and facilities within a minimum 100 m radius of the location where explosives were used to determine if any damage has resulted from the blast. A post-construction survey certificate signed by the Blasting Consultant shall be submitted to the Contract Administrator ten(10) working days following the blasting operation.

A blast report summarizing the results of the vibration and air blast levels signed and sealed by the Blasting Consultant shall be submitted to the Contract Administrator at the end of each work day in which blasting is carried out.

#### **120.04.07                      Certificate of Conformance**

Upon completion of the blasting work, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Blasting Consultant. The certificate shall state that the work has been carried out in general conformance with the requirements of the contract .

### **120.05                              MATERIAL**

#### **120.05.01                      Explosives**

Only explosive products that are approved for use in Canada shall be used.

**120.05.02                      Handling and Storage**

Explosives and all detonating apparatus shall be handled and stored according to the Explosives Act (Canada) and any applicable Municipal By-laws.

**120.06                              EQUIPMENT**

Only firing devices approved for use by the manufacturer of the selected detonator shall be used. All apparatus shall be thoroughly inspected before and after each blasting operation.

All wiring connected to electrical firing devices shall be properly insulated.

**120.07                              CONSTRUCTION**

**120.07.01                      General**

Blasting shall be carried out only during daylight hours and at any time when atmospheric conditions provide clear observation of the blasting site from a distance of 1,000m. Blasting shall not be conducted on Sundays, statutory holidays or during electrical storms

**120.07.02                      Safety Precautions**

All safety precautions shall be observed prior to blasting, including the sounding of audible warning device before and after blasting as required. The Contractor shall be responsible for all claims arising from the transportation, handling, storage and use of explosives.

The Contractor shall comply with the requirements and limitations for protecting fish and fish habitat according to the Department of Fisheries and Oceans (DFO) Guidelines and as detailed elsewhere in the Contract.

Prior to blasting, investigations shall be done to determine if radio frequency hazards exists. Where they exist, necessary precautions shall be taken.

**120.07.03                      Notice**

The Contractor Administrator shall be provided in writing with a blasting schedule a minimum of 48 hr prior to the start of blasting.

All departments or agencies of government, persons, partnerships and corporations affected shall be notified a minimum of 48 hrs prior to blasting

The Contractor shall provide notice to property owners within a 200 metre radius from the actual blasting site, a minimum of four (4) hours prior to the commencement of blasting.

**120.07.04                      Vibration Monitoring**

Vibration monitoring shall be carried out, unless otherwise specified in the Contract Documents. Ground vibration levels shall be monitored 200 m from the blast site or at the closest structure within the radius during

each blast. Water pressure shall be monitored adjacent to the shore, closest to the blast site.

The Contractor shall limit PPV ground vibrations to the following maximum levels:

#### Structures

50 mm/s measured in one of three planes where the predominating frequency exceeds 40 Hz.

20 mm/s measured in one of three planes where the predominating frequency is below 40 Hz.

#### Fresh Concrete

For concrete or grout in place less than 60 hours, the maximum PPV due to blasting operations shall not exceed 10 mm/sec.

No blasting shall be carried within 30 m of freshly placed concrete for the following periods:

For concrete curing in ambient temperatures less than 20°C, 72 hours from completion of placement.

For concrete curing in ambient temperatures greater than 20°C, 24 hours from completion of placement.

#### **120.07.05                      Utilities**

The Contractor shall provide written notification to all utility authorities within vicinity of the blasting operation a minimum of five (5) working days prior to the use of explosives.

#### **120.07.06                      Excessive Vibration Readings – Work Stoppage**

Should any two (2) consecutive readings exceed the maximum PPV values specified in sub-section 120.07.04, all blasting operations shall cease. A new blast design shall be submitted to the Contract Administrator to reduce PPV values within the allowable limits prior to resuming the blasting operation.

**WARRANT:** Always when the use of explosives is permitted in the contract.

## **AMENDMENT TO OPSS 206, DECEMBER 1993**

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Special Provision

November 25, 2002

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OPSS 206, Construction Specification for Grading (December 1993) is amended as follows:

### **206.01 SCOPE**

Section 206.01 of OPSS 206, December, 1993, is amended by the addition of the following:

Included in this specification are the requirements for the construction and compaction of rock fill embankments to minimize and control settlements within the rock fill.

### **206.04 SUBMISSION AND DESIGN REQUIREMENTS**

OPSS 206, December, 1993, is amended by the addition of the following:

The Contractor shall submit to the Contract Administrator for record purposes a detailed construction procedure outlining the method and sequence of placement of rock fill and method of rock fill compaction. The equipment to be used shall be described in the submission. Where applicable, the submission shall describe modifications to be used during winter construction, to avoid the incorporation of frozen materials into the embankments.

### **206.06 EQUIPMENT**

OPSS 206, December, 1993, is amended by the addition of the following

Equipment for compacting rock fill shall be of the vibratory, smooth, steel drum roller type with a minimum static drum weight of 8000 kg and minimum operating dynamic force of 150 kN.

### **206.07 CONSTRUCTION**

Section 206.07 of OPSS 206, December, 1993, is amended as follows:

#### **206.07.01.03 Compaction**

The contents of subsection 206.07.01.03 are deleted and replaced with the following:

##### **206.07.01.03.01 Compaction of Earth Embankments**

Compaction of earth materials shall conform to OPSS 501.

##### **206.07.01.03.02 Compaction of Rock Embankments**

Compaction of rock materials shall conform to the method and equipment requirements of this specification. Each rock fill layer shall be compacted with the equipment specified.. The minimum number of passes shall be four. Each roller pass shall overlap the edge of preceding passes by minimum 0.3 m. One hundred per cent roller pass coverages on the surface of each layer shall be provided

#### **206.07.05      Rock Excavation, Grading**

##### **206.07.05.01    General**

The first paragraph in subsection 206.07.05.01 is deleted and replaced with the following:

The work to be done under the item Rock Excavation, Grading shall include drilling and blasting to obtain the required rock excavation and shatter, mucking, hauling and placing, handling, compacting and bringing to grade any excavation taken below grade. Whenever a rock face item is not included in the Contract, rock scaling and the removing of all overbreak and scaled materials and their incorporation into embankments shall be included in the rock excavation item.

#### **206.07.08      Rock Embankments**

Subsection 206.07.08 Rock Embankments is deleted and replaced with the following:

Construction of embankments using shale shall be carried out conforming to shale embankment requirements as specified in OPSS 206.07.08.01.

Embankments to be constructed of excavated rock other than shale shall be constructed by placing embankment materials full width in successive, uniform layers. Layers shall not exceed 1.5 m thickness prior to compaction. Material in each layer shall be fully compacted before the succeeding layer is placed.

Materials shall be placed in final position by blading. End dumping or depositing of rock over the end of any layer by hauling equipment is not permitted, except as otherwise noted below. Each layer shall be levelled in place and compacted to minimize voids and bridging of large rock fragments within the embankment.

Rocks exceeding 1 metre in size shall be well distributed throughout the embankment. Rock fragments up to a maximum size of 3 metres in size may be incorporated into the embankment provided that the rock fragments are less than two-thirds the remaining embankment height and are sufficiently spaced to allow free access of the specified equipment to compact the intervening fill. The remaining height shall be defined as the distance between the bottom of the oversized rock fragment at point of placement to the top of the rock fill embankment.

Placement in layers and compaction is not required for rock to be placed under water. Rock placed underwater may be placed by end dumping. End dumping shall only be used to an elevation of 1.0 m above the water level after which rock embankments shall be constructed using the equipment and method specified in this special provision. The rock used for end dumping shall be deposited on the

surface of the embankment and pushed forward by blading or dozing over the edge of the embankment. The materials shall be well distributed to form a solid embankment constructed to full width as the work progresses, or as stage construction allows.

Where rock fill is placed in a wet area (such as swamps with full, partial or no excavation), the direction of the rock fill placement shall be such that mud waves generated by the rock fill placement would move away from the embankment. Mud waves shall be displaced or removed to prevent its entrapment below or within the embankment.

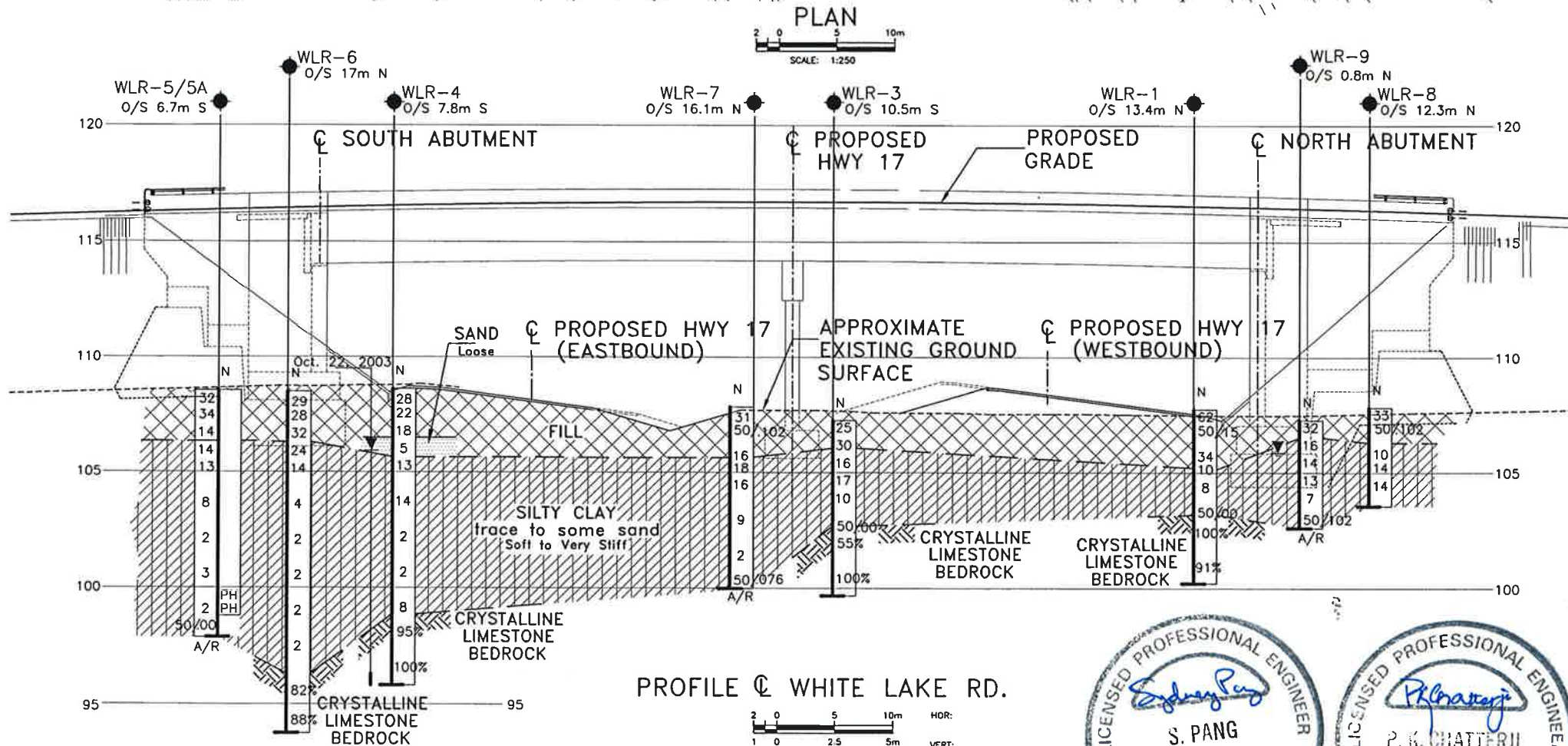
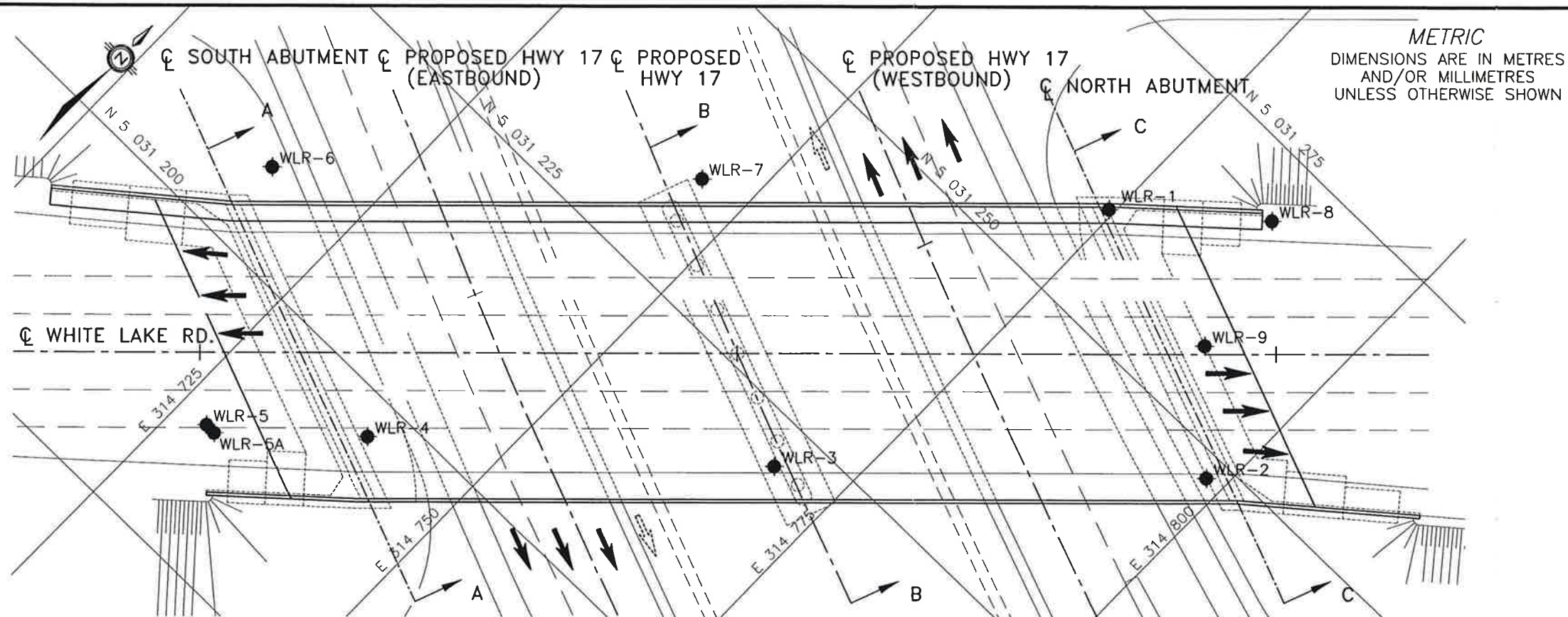
Voids on the top surface of the embankment shall be minimized to prevent migration of the roadway subbase and base into the rock fill embankment by chinking the top surface with rock fragments and spalls to form the subgrade prior to the placement of the roadway subbase. The Contractor shall chink the top surface of the embankment using rock material not exceeding 100 mm in size by conventional blading techniques.

Care shall be taken to avoid large boulders and rock fragments protruding above the average embankment surface within a distance of 3 m beyond the edge of the shoulder for future roadside safety.

Dumping over the sides of embankments is permitted only if the material is surplus to the contract requirements and after the rock embankments have been completed. Dumping over the sides of embankments shall be restricted to standard offset and right of way limits unless otherwise specified in the Contract Documents. The Contractor shall receive written approval from the Contract Administrator before commencing the above operations.

## **Appendix F**

### **Drawings**

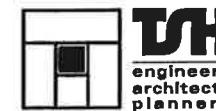


HWY.17  
GWP NO. 647-92-00

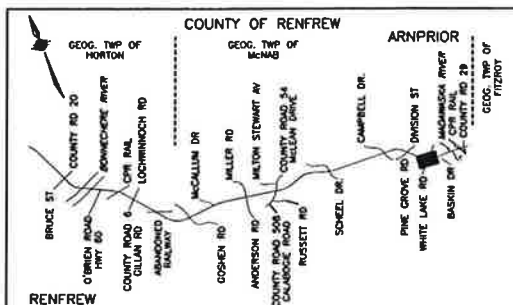


HIGHWAY 17 TWINNING  
WHITE LAKE ROAD UNDERPASS  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



THURBER ENGINEERING LTD.



KEYPLAN

### 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

NO	ELEVATION	NORTHING	EASTING
WLR-1	107.7	5 031 259.6	314 774.5
WLR-2	106.7	5 031 248.0	314 798.3
WLR-3	107.3	5 031 220.9	314 768.8
WLR-4	108.6	5 031 196.5	314 739.7
WLR-5	108.6	5 031 186.8	314 728.2
WLR-5A	108.6	5 031 186.8	314 729.2
WLR-6	108.5	5 031 208.1	314 715.9
WLR-7	107.8	5 031 235.2	314 745.4
WLR-8	107.8	5 031 269.3	314 786.0
WLR-9	107.3	5 031 256.7	314 789.6

### NOTE

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



REVISIONS	DATE	BY	DESCRIPTION
SEP. 04	SP	FINAL	
APR. 04	SP	ISSUED AS DRAFT FOR REVIEW	
DESIGN	SP	CHK PKC	CHBDC 2000
DRAWN	SS	CHK SP	SITE
			STRUCT
			DWG.

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

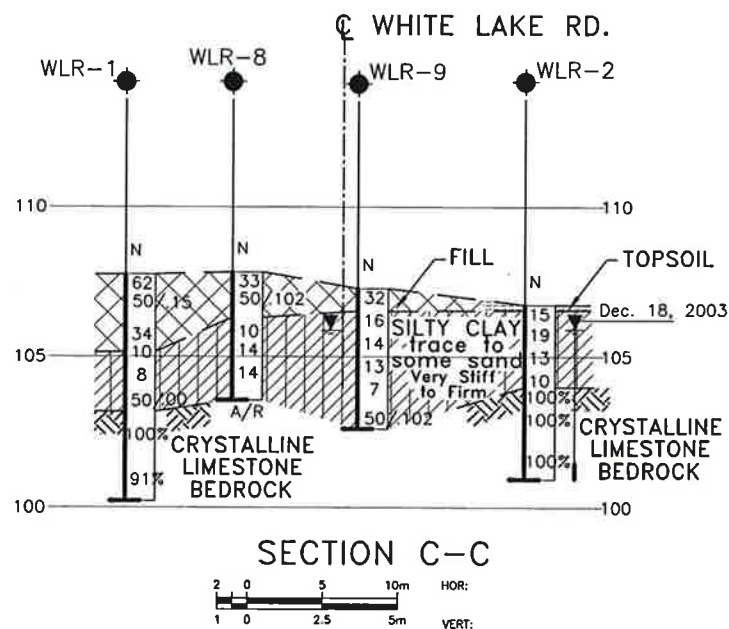
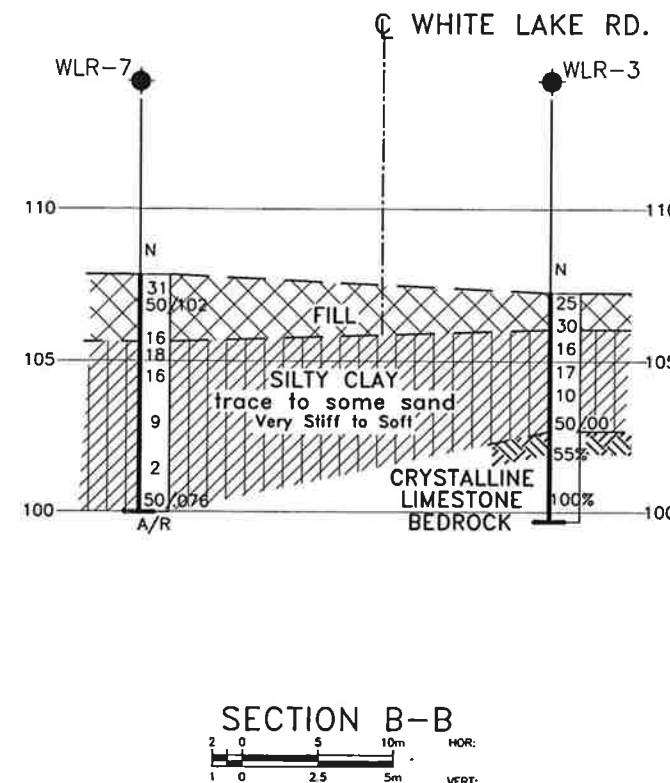
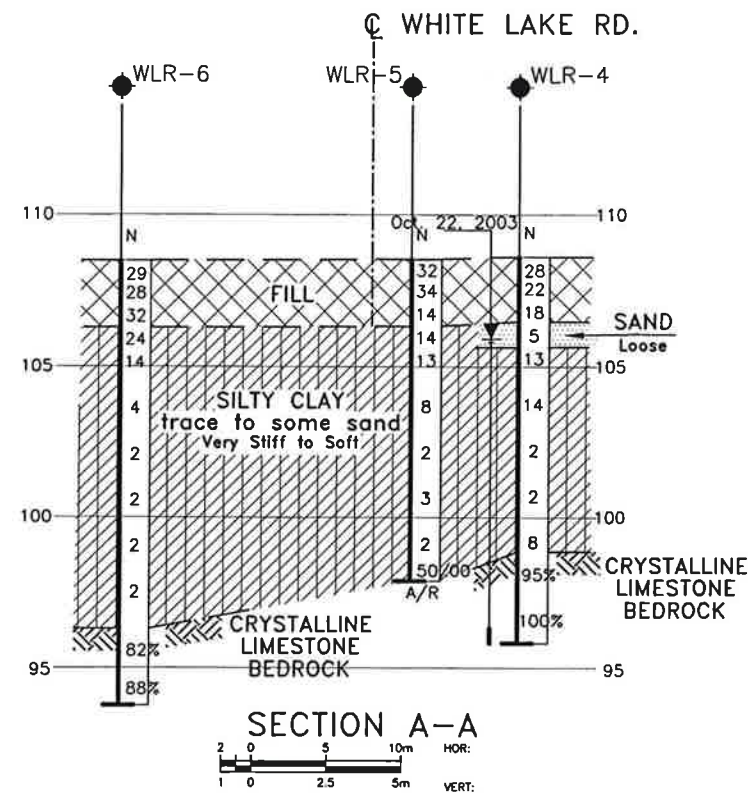
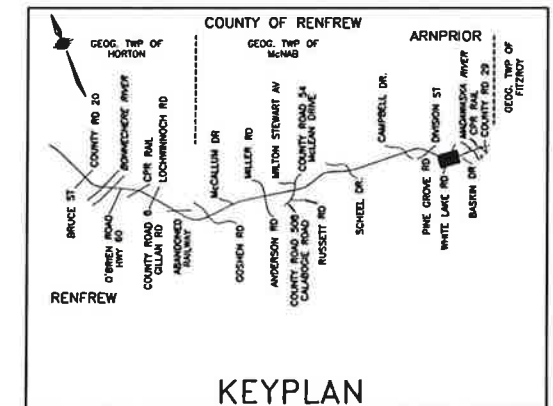
HWY.17  
GWP NO. 647-92-00

HIGHWAY 17 TWINNING  
WHITE LAKE ROAD UNDERPASS  
SOIL STRATA

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WLR-5A	108.6	5 031 186.8	314 729.2
WLR-6	108.5	5 031 208.1	314 715.9
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REVISIONS	DATE	BY	DESCRIPTION	DATE
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DESIGN	SP	CHK PKC	CHBDC 2000	LOAD
DRAWN	SS	CHK SP	SITE	STRUCT
				DWG.