

**FOUNDATION INVESTIGATION AND DESIGN REPORT**

**C.P.R. OVERHEAD (ARNPRIOR)**

**HIGHWAY 17/417 TWINNING**

**ARNPRIOR, ONTARIO**

**G.W.P. 647-92-00, SITE NO. 29-200**

**GEOCRES Number: 31F-131**

**Report to**

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### **PART 1: FACTUAL INFORMATION**

#### **1 INTRODUCTION**

This report presents the factual findings obtained from a foundation investigation conducted at the location where the new Highway 17 W.B.L will pass over the C.P.R. (also known as the Ottawa Valley Railway, or OVR) track near Arnprior. 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 of 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 initial investigation as a sub-consultant to National Capital Engineering (NCE), under the MTO Agreement Number 4005-A-000157. A final report (Reference 2 below) had been issued in August, 2004. In 2005, Thurber was retained by McCormick Rankin Corporation (MRC) to carry out a supplementary investigation at this site primarily to better define the top of rock at the wingwall locations. The present report incorporates all available subsurface information to date and supersedes the previous report.

The following documents are referenced in the preparation of this report:

- MTO report titled "Preliminary Foundation Report for Structure Crossings of Revised Hwy #17 From Antrim Westerly to Locheil Creek" Region Municipality of Ottawa, Carleton and Renfrew County, District No. 9 (Ottawa), W.J. 69-F-86, W.P.'s 5-67 & 190-67, GEOCRES No. 31F23 dated March 12, 1970 (Reference 1).
- Thurber report titled "Foundation Investigation and Design Report, C.P.R. Overhead (Arnprior), Highway 17 Twinning, Arnprior to Renfrew, Ontario", G.W.P. 647-92-00, Site No. 29-200, GEOCRES No. 31F-131, dated August 20, 2004 (Reference 2).

## **2 SITE DESCRIPTION**

The site is located about 165 m east of the existing at grade intersection of Highway 17 and Upper Access Road (approximate mainline Station 31+400 on existing Highway 17), Township of McNab, County of Renfrew, Ontario. This location is south of the Town of Arnprior and east of the Madawaska River. At this site, the existing Highway 17 crosses over the present C.P.R. track, which is oriented northwest to southeast relative to the highway. The site is located immediately to the north of the existing bridge.

The terrain at the site is fairly rugged with bedrock outcrops and rock faces. The exposed bedrock was cut to provide the required grade separation between the railway below and Highway 17 above. Vegetation is light and consists mainly of grass and shrubs growing in the shallow pockets of overburden across the site. Further away clumps of larger trees are evident.

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

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

The initial site investigation and field testing for this project were conducted during the period of August 12 to 13, 2003. The field investigation consisted of drilling and sampling a total of nine boreholes to depths ranging from 1.5 to 4.9 m. The bedrock surface was also visually inspected at four borehole locations, numbered OVR-1, OVR-6, OVR-7 and OVR-12, where the bedrock was exposed at ground surface. Borehole OVR-3 was inaccessible and was not drilled. The supplementary investigation was carried out on December 14, 15 and 19, 2005 when three additional boreholes, numbered OVR 05-2 to 05-4, were drilled and sampled to depths of 4.8 m to 8.5 m. Borehole OVR 05-1 was not drilled as it was located on a rock outcrop. A visual inspection was also carried out during the supplementary investigation to delineate the areal extents of the rock outcrops.

The approximate locations of the boreholes, numbered OVR-2, OVR-4, OVR-5, OVR-8 to OVR-11, OVR-13, OVR-14, and OVR 05-2 to OVR 05-4, are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix F.

The borehole locations were marked in the field by surveyors from J. D. Barnes Limited (Ottawa) who also provided us with the geodetic elevations and coordinates of the as-drilled boreholes. Thurber obtained utility clearances prior to drilling. Borehole OVR 05-3 was relocated to the crest

of the east slope as it was impossible to drill at the original location which was staked/marked on a slope.

George Downing Estate Drilling Limited of Calumet, Quebec (Downing) supplied a track mounted CME 75 drill rig and conducted the drilling, sampling and in-situ testing operations for the initial investigation which includes Boreholes OVR-2, OVR-4, OVR-5, OVR-8 to OVR-11, OVR-13 and OVR-14. Downing also supplied a track-mounted CME 55 drill rig to advance Boreholes OVR 05-2 to 05-4. Auger drilling techniques were used to advance the boreholes in the overburden and samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Field vane shear tests were conducted within cohesive soils, where appropriate, using an MTO 'N' size vane. Five of the boreholes at or near the foundation elements were advanced 2.9 m to 3.3 m into bedrock by NQ size rotary coring techniques.

Groundwater conditions in the open boreholes were observed throughout the drilling operations and a standpipe piezometer was installed in Borehole OVR-4 to permit longer term groundwater level monitoring. The installation details are presented on the Records of Boreholes in Appendix A and summarized as follows. At this site, a 19 mm diameter Schedule 40 PVC pipe with a 1.52 m long slotted screen was installed at the bottom of the open borehole. The sand screen surrounding the pipe was about 2 m long. The remaining space in the borehole was grouted with a bentonite-based grout.

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

#### **4 LABORATORY TESTING**

All recovered soil samples were subjected to visual identification and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A. Selected samples were subjected to gradation analysis and Atterberg Limit Tests were performed on samples retrieved from the cohesive silty clay deposit. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures contained in Appendix B.

Point load testing was carried out on selected rock cores retrieved from Boreholes OVR-2, OVR-4, OVR-11, OVR-13, OVR 05-2, 05-3 and 05-4. These results are shown in Tables 1 and 2 attached immediately following the text.

## 5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in this appendix and on the "Borehole Locations and Soil Strata" 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, bedrock either outcrops or exists at relatively shallow depths at the site. The overburden ranges in thickness from 0.3 m to 5.3 m and consists of topsoil overlying silty clay.

### 5.1 Topsoil

Topsoil was encountered across the site in eleven boreholes to depths of 75 mm to 325 mm as shown below.

Borehole	Topsoil Thickness (mm)
OVR-4	100
OVR-5	125
OVR-8	75
OVR-9	150
OVR-10	150
OVR-11	325
OVR-13	125
OVR-14	125
OVR 05-2	200
OVR 05-3	200
OVR 05-4	150

Topsoil thickness may vary between and beyond the boreholes.

### 5.2 Silty Clay

In Boreholes OVR-4, OVR-5, OVR-8, OVR-9, OVR-10, OVR-13, OVR-14, OVR 05-2, OVR 05-3 and OVR 05-4, the topsoil is underlain by a silty clay layer encountered at depths ranging from 0.1 to 0.2 m below ground surface. This deposit extends to depths ranging from 1.1 m to 5.3 m, or from Elevations 94.0 m to 97.2 m.

Grain size analyses conducted on samples retrieved from this unit are presented in Figures B1 and B3. These results show that the clay content of this soil ranges between 20% and 40%. The plasticity charts in Figures B2 and B4 show that the silty clay samples had measured plasticity indices of between 15% and 24% indicating a low to medium plasticity (group symbol CL-CI). The plasticity is a function of the clay content and is anticipated to be lower as the clay content decreases.

Standard Penetration Tests conducted within this deposit gave 'N' values ranging from 2 blows to 64 blows for less than 0.3 m penetration but generally, most 'N' values ranged from 3 to 11 blows per 0.3 m penetration. A field vane shear strength of about 48 kPa was measured in Borehole OVR 05-3. Based on these results the deposit is considered to be generally stiff to soft. It should be pointed out that the higher blow counts (where the full 0.3 m of penetration was not achieved) were recorded in the lower portion of the deposit where boulders or rock pieces may be present.

The measured moisture contents of samples recovered from this unit ranged from 9% to 62%, with most values lying within the range of 20% to 40%.

### 5.3 Bedrock

Across the site crystalline limestone bedrock is either exposed or overlain by the overburden soils described above. Bedrock was proven by coring in Boreholes OVR-2, OVR-4, OVR-11, OVR-13, OVR 05-2, OVR 05-3 and OVR 05-4. In some of the boreholes drilled across the site, the bedrock surface was inferred from refusal to auger penetration and/or sampler refusal. The following table summarizes the depth to bedrock and indicates that the elevation of the bedrock surface across the site varies from 92.7 m to 97.1m.

Borehole Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
OVR-1	Outcrop	96.4
OVR-2*	Outcrop	96.2
OVR-4*	1.8	95.9
OVR-5**	2.0	97.1
OVR-6	Outcrop	96.7
OVR-7	Outcrop	95.5
OVR-8**	2.2	95.1
OVR-9**	2.3	95.7
OVR-11*	0.3	94.8
OVR-12	Outcrop	94.1
OVR-13*	1.1	96.4
OVR-14**	1.5	96.8
OVR 05-1	Outcrop	95.2
OVR 05-2*	2.9	94.0
OVR 05-3*	5.3	94.6
OVR 05-4*	1.7	96.4

\* Proven by coring

\*\* Inferred from auger and/or sampler refusal.



The crystalline limestone bedrock is very thinly to thinly bedded and generally in a slightly weathered state with slightly to moderately weathered joints. Its colour is grey to whitish grey, with dark grey and white horizontal and sub-vertical banding visible in most cores.

The measured Total Core Recovery (TCR) for the core runs was between 96% and 100%, and the RQD values ranged from 61% to 100%. The RQD values typically ranged between 92% and 100% indicating an excellent rock quality. It should be noted that the recorded RQD of 61% in Run #3 of Borehole OVR-13 is due to the presence of multiple vertical and sub-vertical joints. A RQD of 80% was measured for Run #1 in Borehole OVR 05-2.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was low and generally ranged from 0 to 3, except at Boreholes OVR-4 and OVR-13 where FI values of 6 and 10 were recorded, respectively.

The joint orientation was typically sub-vertical to vertical. The joint surfaces were generally rough and were mostly tight with no infilling or secondary weathering material, except in Run#2 of Borehole OVR-11 where gypsum crystals were present.

The inferred Unconfined Compressive Strength (UCS) of intact rock cores (expressed as average value per run) typically range from 50 MPa to 103 MPa indicating a strong to occasionally very strong rock. These estimated rock strength values are based on point load tests that were conducted on rock cores recovered from the boreholes. A summary of the Point Load Test Results is presented in Tables 1 and 2 attached immediately following the text.

#### 5.4 Water Levels

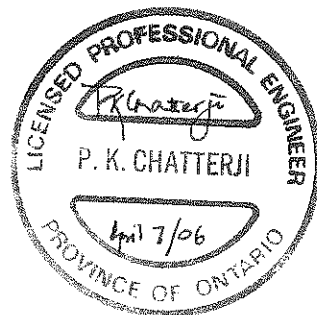
The groundwater conditions were observed in the open boreholes on completion of drilling. A standpipe piezometer was installed in Borehole OVR-4 and the water level was measured on two separate visits made after the completion of drilling. These readings are presented in the table below.

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
OVR-4	October 22, 2003	4.1	93.6
	February 4, 2004	4.3	93.4

Based on these observations, the local groundwater level is anticipated to be at approximate Elevation 93.4 m. It is also anticipated that the local groundwater level at this site is influenced by the nearby Madawaska River. All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe climatic events.



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**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**6 GENERAL**

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

It is understood that the preliminary design plan calls for the construction of a new bridge to carry the proposed westbound lanes (WBL) of the twinned Highway 17 over the existing C.P.R. track. The new bridge will be located to the north of the existing C.P.R. Overhead bridge. The existing bridge will carry the two existing lanes that will become the eastbound lanes of the twinned Highway 17.

The proposed overpass structure will be a single span bridge comprising of CPCI precast, prestressed concrete girders, with a span length of approximately 34 m between the abutments. The new bridge will have a skew of about 40° to the abutment bearings, similar to that of the existing bridge.

At the abutments, the proposed WBL highway grade will be at approximate Elevation 104 m. Consequently, new fill heights of about 6 m and 7 m will be required at the east and west approaches, 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 this investigation.

**7 STRUCTURE FOUNDATIONS**

The proposed structure at this site is a single span overpass bridge with two foundation elements at the abutments.

The stratigraphy encountered at the locations of the proposed east abutment consists of 0.3 m to 5.3m of topsoil and silty clay overlying bedrock. Bedrock is either exposed at ground surface or covered by a veneer of topsoil in the boreholes drilled at the west abutment. The groundwater level exists at elevations below the bedrock surface.

The elevations at which bedrock was encountered or inferred at the two abutment locations are as follows:

Location	Borehole Number	Elevations (m)	
		Existing Ground Surface	Top of Bedrock
East Abutment			
Northeast Corner	OVR-4	97.7	95.9**
West Centre	OVR-8	97.3	95.0±
East Centre	OVR-9	98.0	95.7±
Southwest Corner	OVR-13	97.5	96.4**
Southeast Corner	OVR-14	98.3	96.8±
West Abutment			
Northwest Corner	OVR-1	96.4	96.4*
Northeast Corner	OVR-2	96.2	96.2**
West Centre	OVR-6	96.7	96.7*
East Centre	OVR-7	95.5	95.5*
Southwest Corner	OVR-11	95.1	94.8**
Southeast Corner	OVR-12	94.1	94.1*

\* Outcrop

\*\* Proven by coring.

The elevations at which bedrock was encountered or inferred at the abutment wingwall locations are as follows:

Location	Borehole Number	Elevations (m)	
		Existing Ground Surface	Top of Bedrock
Northwest Wingwall	OVR 05-1	95.2	95.2*
Northeast Wingwall	OVR 05-2	97.0	94.0**
Southwest Wingwall	OVR 05-3	99.9	94.6**
Southeast Wingwall	OVR 05-4	98.1	96.4**

\* Outcrop

\*\* 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 not considered feasible for this bridge primarily due to the skew angle of the structure. Consideration may be given to adopting a semi-integral design using footings founded on bedrock at the abutments.

The limited height of proposed fill above bedrock renders it impractical and unnecessary to use driven piles or augered caissons at this site. Such deep foundation elements were therefore eliminated from further consideration.

It is not considered feasible to use spread footings on engineered fill due to the presence of bedrock at shallow depth, the limited embankment height between the bedrock surface and the proposed Highway 17 grade, and the likelihood that the footings would have to be located further back from the cut resulting in a longer superstructure.

In view of the above, it is recommended that spread footings founded directly on bedrock be used to provide foundation support to the abutments at this site.

## **7.2 Spread Footings on Bedrock**

Based on the subsurface stratigraphy and the proposed vertical alignment of the highway, this appears to be the most feasible foundation option for the two bridge abutments at this site.

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 sound and 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. Should open vertical to sub-vertical joints, fracture zones or solution cavities be identified at the foundation subgrade areas, grouting or void filling with cement based materials may be required to strengthen the rock mass.

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

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

### *East Abutment*

The top of rock varies between approximate Elevations 95.0 m and 96.8 m across this foundation.

### *West Abutment*

The top of rock varies from approximate Elevations 96.2 m to 96.7 m in the northern and central portions to approximate Elevation 94.1 m in the southern portion of the foundation area.

In view of the close proximity of the proposed abutments to the exposed rock faces adjacent to the existing cut and the potentially adverse joint orientation associated with the bedrock, it is recommended that the footing be positioned such that a 45° degree line drawn from the outermost edge of the underside of the footing base should not intersect with the exposed face of the rock cut. In addition, a minimum horizontal distance of 1.5 m should be maintained between the outermost edge of the footing base and the crest of the rock cut.

Depending on the location, orientation and height of the exposed rock cut with respect to the pattern of joints or fractures within the rock mass, potentially unstable rock wedges may exist below the proposed footing foundation. After the rock subgrade is exposed, the Contractor should remove any loosen rock and the Contract Administrator (CA) should retain a rock slope stability/rock mechanics specialist to examine the rock subgrade and rock cut adjacent to the footings. Should any potentially unstable wedges or over-break zones be identified that could adversely impact footing performance, rock dowelling, rock anchoring and/or mass concreting may be necessary to reinforce the rock mass prior to footing construction. Any remedial work should be designed by and carried out under the supervision of the rock slope stability/ rock mechanics specialist retained by the CA.

The contract should include an NSSP to this effect.

## **7.3 Bearing Resistance**

Footings bearing on crystalline limestone bedrock encountered at this site may be designed for the following resistances:

- Factored geotechnical resistance of 5,000 kPa at Ultimate Limit States (ULS).

It is noted that the values recommended above are 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.

#### **7.4 Horizontal Resistance of Footings**

Resistance to lateral forces / sliding resistance between the concrete footing and the bedrock surface at the abutment locations 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 direct 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 structural capacity of the selected dowel should not be exceeded. It is noted that the above design assumes that a drilled hole in bedrock has a diameter just large enough to accommodate the dowel and, as such, the compressive strength of the rock governs design.

#### **7.5 Frost Cover**

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

### **8 CANTILEVERED RETAINING WALLS**

#### **8.1 General**

Given the existing rock cut and the proposed highway grade at this site, it is anticipated that retaining walls will be required to support the approach fills. It is also understood that modifications to the existing retaining walls will be required to accommodate the new approach configurations. The required lengths of the new retaining walls will depend on the configuration of the new bridge abutment and approaches, as well as the length of any permanent unsupported open cut sections with inclined side slopes that might be feasible.

#### **8.2 Spread Footings on Bedrock**

Consideration may be given to the use of concrete cantilevered walls. Given the presence of bedrock as outcrop or at very shallow depth below existing ground surface, it is recommended that the retaining wall footings be founded on bedrock. The retaining walls should be designed in accordance with the requirements of CHBDC, 2000.

Along the east side of the existing rock cut, the retaining wall footings may be founded on bedrock estimated to exist at between approximate Elevations 95.0 m and 96.8 m. Along

the west side of the cut, the footings may be founded on bedrock estimated to be sloping in a north to south direction from approximate Elevations 96.7 m to 94.1 m.

Detailed design recommendations on vertical and horizontal geotechnical resistances, stepped footings, eccentric and inclined loads are similar to those for the abutment footings (see previous Sections 7.3 and 7.4). Design recommendations on earth pressures are similar to those presented in the subsequent Section 14, Earth Pressures.

Design of retaining walls must take into account stability against overturning and sliding. Global stability of the retaining wall, cut and approach fill configurations can be maintained provided the footings are designed and constructed on prepared bedrock surfaces as recommended in this report.

## **9 EXCAVATION AND BACKFILL**

### **9.1 General**

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native clayey silt to silty clay deposit can be classified as Type 2 soil above the water table and a Type 3 soil below the water table.

### **9.2 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.

### **9.3 Earth Excavation**

In addition to SP 902S01, a NSSP should be included in the contract alerting the Contractor to the possible presence of cobbles, boulders or rock pieces in the overburden, immediately above the bedrock.

It is anticipated that earth excavation to expose bedrock will be relatively shallow at the abutment locations. However, 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. Soldier piles will likely have to be socketted into bedrock.

An item titled "Road Protection" as per SP 539S01 will have to 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 in Figure D1 may be used for design in conjunction with the following parameter values.

$$\begin{aligned}\gamma &= 20 \text{ kN/m}^3 \\ \gamma_w &= 10 \text{ kN/m}^3\end{aligned}$$



$$\begin{aligned}K_a &= 0.4 \text{ (silty clay)} \\h_w &= 0 \text{ (assuming no hydrostatic pressure build-up behind a} \\&\text{presumably permeable wall)} \\H &= \text{depth to base of excavation (rock surface) (m)}\end{aligned}$$

Below the base of the excavation in intact bedrock, lateral pressures in the rock 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:

$$\begin{aligned}P_p &= 6 c B L \\ \text{where } c &= 2000 \text{ kPa (equivalent Mohr-Coulomb cohesion based on} \\&\text{the Hoek and Brown strength criterion)} \\L &= \text{depth of socket in rock, (m)}\end{aligned}$$

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.

#### 9.4 Rock Excavation

It is anticipated that requirements for rock excavation will be minimal at this site. The strength of intact rock is typically strong to occasionally 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 such as the hydro dam. The contract documents should alert the contractor to these structures and that no damage will be induced on any adjacent structure (such as the hydro dam and its facilities) due to any blasting scheme. The Contract Administrator should retain a blasting expert for review of the Contractor's blasting procedures prior to approving them.

### 10 GROUNDWATER CONTROL

The relatively impervious deposit of clayey silt to silty clay is not expected to yield a significant quantity of seepage water in the short term. Water seepage will occur with time into the excavations for footing construction and where water-bearing seams are exposed. Water may also be trapped locally in depressions on the rock surface.

The design of foundations bearing on bedrock will not be influenced by the groundwater, but 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.

## **11 APPROACH EMBANKMENTS**

For the purpose of embankment stability analyses in this report, the commercially available slope stability program GSLOPE developed by Mitre Software Inc. was used. The Bishop's simplified method for stability analysis was employed for short term (total stress, undrained) and long term (effective stress, drained) conditions where applicable.

Immediate (elastic) settlements due to recompression of over-consolidated cohesive soils have been estimated based on elastic and other methods described in the CHBDC, 2000 Commentary Section C6.6.3.6.

### **11.1 Stability**

The global, internal and surficial stability of the approach embankment fill will depend on the slope geometry but also to a large degree on the material used to construct the embankment. In general, if the embankment is constructed of blast rockfill, it may be assumed that the side slopes will be stable at inclinations not steeper than 1.25H : 1V; embankments constructed using granular material and select subgrade material will have stable side slopes at inclinations not steeper than 2H : 1V.

The east approach embankment for this structure will be constructed on a relatively thin deposit of clayey silt to silty clay overlying bedrock. The embankments can be expected to be up to 6 m high decreasing in height with increasing distance from the bridge structure. The new fill may consist of rock fill or earth fill. Stability analyses were carried out for the above slope configurations and yielded Factors of Safety (F.S.) in the order of 1.4 for earth fill slopes, and in the order of 1.5 for rock fill slopes. Figures G1 to G4 present selected stability analyses results for the east approach.

The west approach embankment will be constructed on exposed bedrock after the topsoil or a veneer of surficial native soil is stripped. The new fill may consist of rock fill or earth fill. Stability analyses were carried out for the above slope configurations and yielded Factors of Safety (F.S.) in the orders of magnitude similar to those at the east approach.

### **11.2 Settlement**

At both approaches, some settlement will occur within the new fill. This settlement should be complete by the end of construction. Immediate settlement due to recompression of the over-consolidated clayey silt to silty clay is expected to be less than 25 mm and should be complete by the end of construction. Post construction settlement may be considered negligible at this site.

### **11.3 Embankment Construction**

Embankment construction should be carried out in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002 and included in Appendix E. Earth fill should consist of granular materials or Select Subgrade Material (SSM) in compliance with Special Provision 110F13, "Amendment to OPSS 1010, March 1993". Clean, inorganic earth fill (in accordance with OPSS 212) may consist of clayey materials that could itself settle in the order of 30 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.

Prior to placing new fill, the existing slope surfaces should be appropriately benched, as per OPSD 208.010, after stripping of topsoil, organics or otherwise unsuitable overburden materials.

Berms are not required at this site to maintain stability, nor to address surficial stability and post construction maintenance.

In general, the approach embankments will consist of rock fill or inorganic earth fill founded on bedrock, or on a thin deposit of native clayey silt to silty clay overlying shallow bedrock. The groundwater level is at or 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.

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

## **12 RETAINED SOIL SYSTEMS**

Retained soil system (RSS) walls may be used at this site. A conventional concrete abutment was used in the preliminary design, but RSS could be used for the retaining structures. It is understood that consideration is currently being given to using RSS at the abutments (false abutments) and the wingwalls. Given the immediate nature of the small anticipated settlement (negligible post construction settlement), the risk of using RSS walls at this site is low.

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.

### **12.1 Foundation**

It is recommended that the levelling pad for an RSS wall be centred on top of a mat of engineered fill that is itself resting on native firm to stiff clays or bedrock. Where applicable, the native soil under the RSS foundation 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 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 300 kPa, and geotechnical resistance of 200 kPa at SLS on an engineered Granular A pad placed on bedrock at the following approximate elevations.

Northwest Wingwall – Elevations 96.4 m to 95.2 m (south to north – rock outcrop)

Northeast Wingwall – Elevations 95.9 m to 94.0 m (south to north)

Southwest Wingwall – Elevations 94.8 m to 94.6 m (north to south)

Southeast Wingwall – Elevations 96.8 m to 96.4 m (north to south)

- 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 native stiff silty clay or on bedrock. All topsoil, organics and soft soils should be removed from the subgrade of the RSS wall prior to placement of the fill.

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

- Factored geotechnical resistance at ULS of 5,000 kPa for walls founded directly on limestone bedrock at the elevations quoted above. The SLS design criterion is not applicable to foundation on rock.
- Factored geotechnical resistance at ULS of 225 kPa, and geotechnical resistance of 150 kPa at SLS, on native silty clay above bedrock.
- Ultimate coefficient of friction of between RSS mass and native clays is 0.45.
- Ultimate coefficient of friction of between RSS mass and sound bedrock is 0.6.

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

## **12.2 Global Stability**

The global stability of the RSS is dependent on the characteristics of the embankment fill and the foundation soils, the geometry of the embankment and location of the RSS within the embankment. RSS walls are likely to be used at the abutment and wing wall locations, and for retaining walls to support the new approaches.

Stability analyses on selected configurations, where the approach fills are founded on clays overlying bedrock at the east abutment, were carried out considering the following variables:

*East Abutment*

- Earth fill – granular materials or 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 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 94 m (below bedrock surface).
- RSS block with full retained height and block width (length of RSS reinforcement) equal to 50% of the height, founded on silty clay overlying shallow bedrock.

Results of the analyses yielded Factors of Safety in the order of 1.4 to 1.5 which indicated that global stability can be maintained for the assumed RSS configuration.

Global stability for an RSS wall founded directly on bedrock is not anticipated to be a design issue at this site.

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

### **12.3 Internal Stability**

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

### **12.4 Settlement**

Settlement will be negligible for RSS walls founded directly on bedrock at the west abutment. At the east abutment, settlement of RSS walls founded on native, stiff over-consolidated clayey silt to silty clay overlying shallow bedrock are expected to be less than 25 mm and to occur essentially as the RSS is constructed.

## **13 BACKFILL TO ABUTMENTS**

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

In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. 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.

#### 14 EARTH PRESSURES

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

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), at-rest horizontal earth pressures should be used. The amount of wall movement required for the development of active, passive and at-rest earth pressures may be interpreted using Figure C6.9.1(a) in the Commentary to the CHBDC.

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

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

where

$P_h$	=	horizontal pressure on the wall (kPa)
$K$	=	earth pressure coefficient (see below)
$\gamma$	=	unit weight of retained soil (see 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 Conditions	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ; \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ; \gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ; \gamma = 19 \text{ kN/m}^3$	
	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	.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

\* For wing walls.

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

The factors in the table above are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC, 2000.

## 15 SEISMIC CONSIDERATIONS

### 15.1 Seismic Design Parameters

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

- Velocity Related Seismic Zone 2
- Zonal Velocity Ratio 0.10
- Acceleration Related Seismic Zone 4
- Zonal Acceleration Ratio 0.2

The subsurface at the two abutments consist of topsoil and/or thin veneer of stiff clayey silt to silty clay overlying very shallow bedrock. The Soil Profile Type at these locations is classified as Type 1, which, according to Table 4.4.6.1 of the CHBDC is, associated with a Site Coefficient (also referred to as ground motion amplification factor) of 1.0.

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

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

## **15.2 Liquefaction Potential**

Since both abutments are to be founded directly on bedrock by means of spread footings, there is no potential for liquefaction under the foundation.

The approach embankments will be founded directly on bedrock or on stiff clayey silt to silty clay overlying shallow bedrock, 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.

## **15.3 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

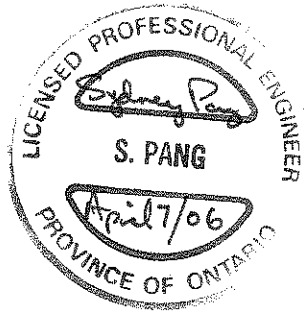
\*\* After Woods

## 16 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:

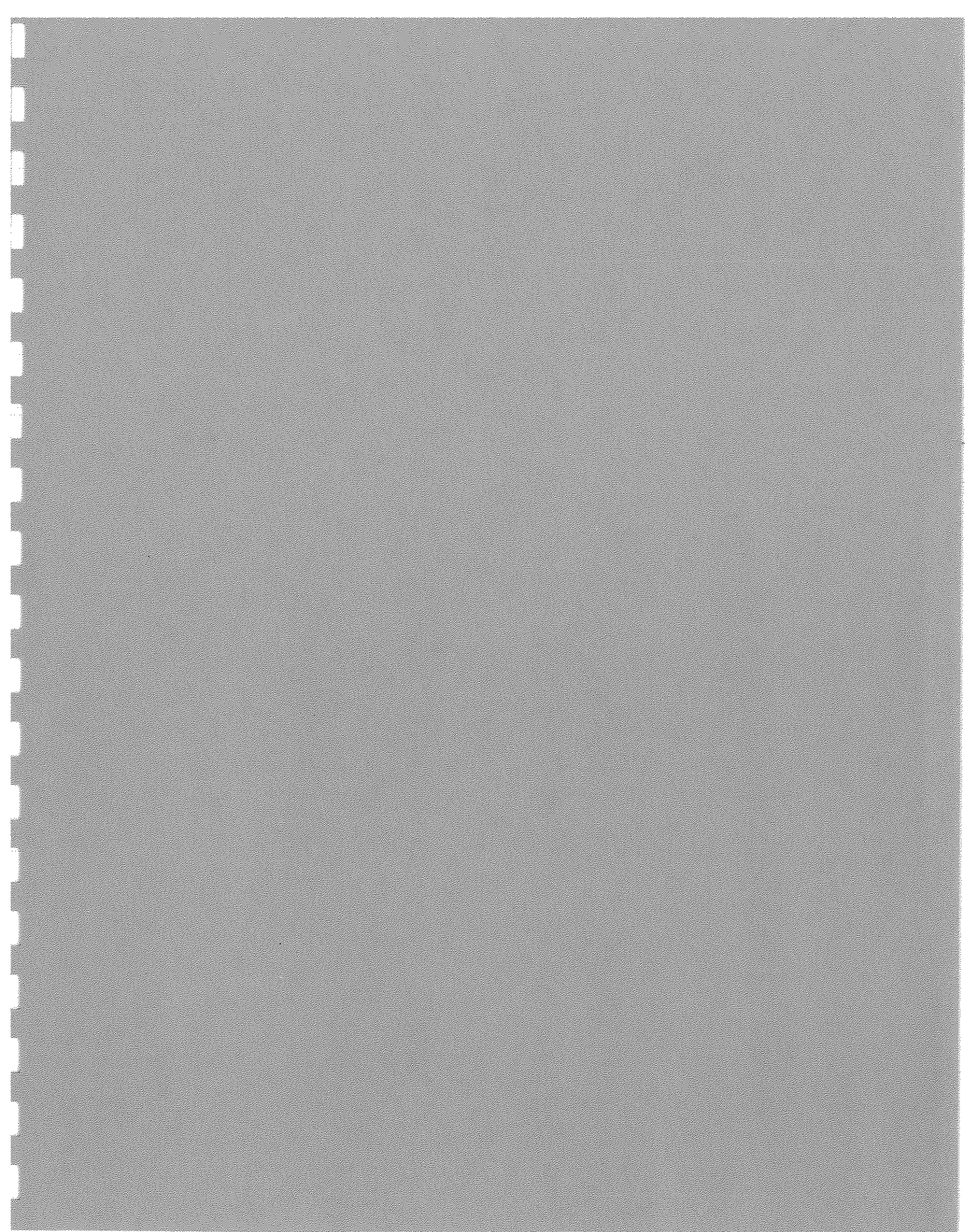
- presence of potentially unstable rock wedges and adverse joint systems at the abutment locations adjacent to the existing rock cut; rock slope stability/rock mechanics specialist(s) must be retained by the Contract Administrator to assess exposed rock faces and rock subgrade in the vicinities of the abutment and other critical locations; the rock specialist should design and implement rock stabilization measures as required.
- maintaining stability at all stages of construction at the approach embankments.
- 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.
- boulders and cobbles may be encountered during construction.



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# Point Load Test Results

**TABLE 1**  
**CPR (Arnprior) (OVR)**  
**Point Load Test Results**

Depth			Is50	UCS (MPa)
feet	Inches	m		
OVR-2				
1	0	0.30	0.66	15.80
2	2	0.66	3.73	89.54
3	0	0.91	0.55	13.17
3	9	1.14	3.29	79.00
4	5	1.35	3.95	94.80
6	0	1.83	3.40	81.64
7	0	2.13	1.98	47.40
7	9	2.36	1.65	39.50
8	9	2.67	3.07	73.74
9	4	2.84	2.85	68.47

Total Rock Core			
Average	Minimum	Maximum	MPa
60	13	95	
Run #      Average			
1	58.46		
2	62.15		

Depth			Is50	UCS (MPa)
feet	Inches	m		
OVR-4				
6	3	1.91	4.61	110.60
7	6	2.29	5.00	120.08
8	0	2.44	4.43	106.39
8	8	2.64	2.50	60.04
9	5	2.87	4.17	100.07
10	6	3.20	6.19	148.52
11	5	3.48	3.42	82.16
12	4	3.76	3.91	93.75
13	7	4.14	3.29	79.00
14	4	4.37	3.29	79.00
15	4	4.67	3.20	76.90
15	11	4.85	3.64	87.43

Total Rock Core			
Average	Minimum	Maximum	MPa
95	60	149	
Run #      Average			
1	99.44		
2	96.49		
3	82.16		

Point Load Test Results

**TABLE 1 (continued)**  
**CPR (Arnprior) (OVR)**  
**Point Load Test Results**

Depth			Is50	UCS (MPa)				
feet	Inches	m						
OVR-11								
1	4	0.41	0.61	14.75	}	Total Rock Core		
2	2	0.66	3.07	73.74		Average	Minimum	Maximum
4	5	1.35	2.24	53.72		67	8	258 MPa
5	9	1.75	2.19	52.67		Run # Average		
6	1	1.85	10.75	258.07				
6	6	1.98	0.35	8.43		1	47.40	
7	3	2.21	4.17	100.07		2	102.18	
8	3	2.51	1.76	42.13		3	49.77	
9	1	2.77	2.63	63.20				
9	10	3.00	4.21	101.12				
10	7	3.23	0.83	20.01				
11	4	3.45	0.61	14.75				

Note: Point load test at 1.85 m was performed in Fresh Sandstone  
Point load test at 1.98 m was performed in Highly Weathered Sandstone

Depth			Is50	UCS (MPa)				
feet	Inches	m						
OVR-13								
3	6	1.07	1.58	37.92	}	Total Rock Core		
4	1	1.24	1.05	25.28		Average	Minimum	Maximum
4	11	1.50	3.73	89.54		77	25	103 MPa
6	2	1.88	3.34	80.06		Run # Average		
6	8	2.03	3.77	90.59				
7	7	2.31	2.28	54.78		1	58.20	
8	5	2.57	4.13	99.02		2	89.36	
9	1	2.77	4.17	100.07		3	83.22	
10	2	3.10	4.30	103.23				
11	1	3.38	3.69	88.48				
12	5	3.78	3.47	83.22				

# Point Load Test Results

**TABLE 2**  
**Highway 17 CPR (OVR) Overpass (Westbound Lanes)**  
**Point Load Test Results**

feet	Depth		Is50	UCS (MPa)
	Inches	m		
OVR 05-2				
10	1	3.07	4.320	103.68
11	8	3.56	2.160	51.84
13	1	3.99	4.320	103.68
14	6	4.42	4.320	103.68
16	5	5.00	3.888	93.31
17	9	5.41	3.024	72.57
19	2	5.84	5.184	124.41

Average	Total Rock Core		MPa
	Minimum	Maximum	
93	52	124	

Run #	Average
1	103.68
2	86.40
3	96.77

feet	Depth		Is50	UCS (MPa)
	Inches	m		
OVR 05-3				
17	10	5.44	3.672	88.13
19	5	5.92	3.456	82.94
20	10	6.35	3.456	82.94
21	10	6.65	2.592	62.21
23	11	7.29	3.888	93.31
25	5	7.75	3.672	88.13
26	4	8.03	1.296	31.10
27	10	8.48	3.024	72.57

Average	Total Rock Core		MPa
	Minimum	Maximum	
75	31	93	

Run #	Average
1	84.67
2	81.21
3	51.84

Depth			Is50	UCS (MPa)
feet	Inches	m		
OVR 05-4				
6	2	1.88	3.456	82.94
8	2	2.49	2.592	62.21
10	0	3.05	3.456	82.94
11	5	3.48	3.456	82.94
13	0	3.96	3.456	82.94
15	0	4.57	3.888	93.31

Average	Total Rock Core		MPa
	Minimum	Maximum	
81	62	93	

Run #	Average
1	76.03
2	86.40

## **Appendix A**

### **Record of Borehole Sheets**



# 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


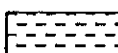



- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration 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
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

## EXPLANATION OF ROCK LOGGING TERMS

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

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
		Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
		Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

TERMS	
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

# RECORD OF BOREHOLE No OVR-1

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 144.3 E 316 991.7(CPR Overhead - Amprior) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Visual Inspection COMPILED BY SS  
 DATUM Geodetic DATE 12.08.03 - 12.08.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
96.4 0.0	BEDROCK on surface						96							


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# RECORD OF BOREHOLE No OVR-2

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 144.1 E 316 994.7 (CPR Overhead - Amprior) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Solid Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 12.08.03 - 12.08.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
96.2 0.0	CRYSTALLINE LIMESTONE (BEDROCK) Fresh to slightly weathered, slightly to moderately weathered at joints, very thinly to thinly bedded, grey with dark grey, white and occasional pink and brown subvertical banding, strong          Subvertical joint at 2.4m		1	RUN	1 0 0 1 1 1										
93.2			2	RUN	0 1 1 1 1 2										
3.1	END OF BOREHOLE AT 3.05m.														

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

# RECORD OF BOREHOLE No OVR-4

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 141.3 E 317 028.6 (CPR Overhead - Amprior) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Solid 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 C
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60					
97.7 96.6	TOPSOIL (100mm)														
0.1	Silty CLAY, some sand seams Firm to Stiff Brown (CL)		1	SS	8										
			2	SS	7										0 5 63 32
			3	SS	11										
95.9					FI										
1.8	END OF SOIL SAMPLING AT 1.8m. CRYSTALLINE LIMESTONE (BEDROCK) Slightly weathered, moderately weathered at joints, very thinly to thinly bedded, grey with dark grey and white subvertical banding, strong Subvertical joints at 1.9m, 2.1m, 2.2m, 3.3m, 3.4m, 3.9m, and 4.1m		1	RUN	2 6 1 1										
					2										
			2	RUN	0 0 3 1										
					1										
92.8			3	RUN	2										
4.9	END OF BOREHOLE AT 4.9m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS DATE ELEVATION (m) 22/10/03 93.6 04/02/04 93.4														

# RECORD OF BOREHOLE No OVR-5

1 OF 1

METRIC


W.P. 647-92-00 LOCATION N 5 031 140.1 E 317 043.5 (CPR Overhead - Amprior) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Solid Stem Augers 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	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
99.1																	
99.0	TOPSOIL (125mm)																
0.1	Silty CLAY with sand seams Firm to Stiff Brown (CL-CI)		1	SS	6												
			2	SS	11											0 12 58 30	
97.2	Hard		3	SS	64/ 280												
2.0	END OF BOREHOLE AT 1.96m. SPOON SAMPLER REFUSAL ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION.																

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

## METRIC

ELEV DEPTH	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 (%)
	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			
96.7								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%)		
96.2								20 40 60 80 100	20 40 60		GR SA SI C

BEDROCK on surface	
96	

(%) STRAIN AT FAILURE

## METRIC

[illegible]



# RECORD OF BOREHOLE No OVR-8

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 134.2 E 317 030.8 (CPR Overhead - Amprior) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Solid Stem Augers 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			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	
97.3 96.8 0.1	TOPSOIL (75mm) Silty CLAY, some sand seams Firm Brown (Cl)		1	GS			97					
			1	SS	6		96					
			2	SS	5							
95.0			3	SS	50/							0 12 63 25
2.2	END OF BOREHOLE AT 2.24m. SPOON SAMPLER REFUSAL ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION.				.102							

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

20  
15 10 5  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No OVR-9

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 134.0 E 317 033.8 (CPR Overhead - Amprior) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Solid Stem Augers 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	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
98.0														
99.0	TOPSOIL (150mm)													
0.2	Silty CLAY, some sand seams Soft to Stiff Brown (Cl)		1	GS										
			1	SS	2									
			2	SS	10									
	some sand pockets		3	SS	50/									
95.7														
2.3	END OF BOREHOLE AT 2.26m. SPOON SAMPLER REFUSAL ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION.				.127									

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# RECORD OF BOREHOLE No OVR-10

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 131.1 E 316 986.2 (CPR Overhead - Amprior) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Solid Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 12.08.03 - 12.08.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT  Y  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								WATER CONTENT (%)				
98.9												
98.8	TOPSOIL (150mm)											
0.2	Silty CLAY, some topsoil to 0.6m, some sand seams Firm to Stiff Brown (CL)		1	SS	5							
			2	SS	11		98					
97.4												
1.5	Soft		3	SS	3		97					
96.8												
2.1	becoming grey		4	SS	5							
			5	SS	10		96					
95.6												
3.4	END OF BOREHOLE AT 3.35m. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION.											

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No OVR-11

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 129.8 E 317 001.2 (CPR Overhead - Amprior) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Solid Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 12.08.03 - 12.08.03 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 (%)  GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)
								20 40 60 80 100					
95.1													
0.0	TOPSOIL (325mm)				FI		95						
94.7													
0.3	CRYSTALLINE LIMESTONE (BEDROCK) Slightly weathered, thinly bedded, grey with dark grey and white subvertical banding, moderately strong to very strong Multiple vertical and subvertical joints from 0.8m to 1.2m. Subvertical joints from 0.8m to 1.2m, 1.7m, 1.9m, 2.0m, 2.4m, 2.9m, 3.1m, and 3.4m.		1	RUN	1 3 0 1		94						
			2	RUN	2 2 0 3 2		93						
91.6			3	RUN	3		92						
3.5	END OF BOREHOLE AT 3.51m.												

# RECORD OF BOREHOLE No OVR-12

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 129.5 E 317 005.1 (CPR Overhead - Amprior) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Visual Inspection COMPILED BY SS  
 DATUM Geodetic DATE 12.08.03 - 12.08.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
94.1 0.0	BEDROCK on surface						94						

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# RECORD OF BOREHOLE No OVR-13

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 127.0 E 317035.0 (CPR Overhead - Amprior) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Solid Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 12.08.03 - 12.08.03 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 C				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			
97.5								20	40	60	80	100								
96.9	TOPSOIL (125mm)																			
0.1	Silty CLAY, some sand seams Soft Brown (CL)		1	SS	3		97													
	Sampler refusal at 0.8m.		2	SS	50/15												1 28 53 18			
96.4					FI															
1.1	CRYSTALLINE LIMESTONE (BEDROCK) Fresh to slightly weathered, very thinly bedded, grey with white and dark grey subvertical banding, strong Subvertical joints at 0.9m, 1.6m, 2.9m, and 3.0m.		1	RUN	1		96													
					2															
					3															
					1															
					0		95													
			2	RUN	1															
					3															
					0															
	Multiple vertical and subvertical joints from 3.4m to 3.7m.				0		94													
93.5			3	RUN	0															
					>10															
4.0	END OF BOREHOLE AT 3.96m.																			

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No OVR-14

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 126.8 E 317 039.01 (CPR Overhead - Amprior) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Solid Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 12.08.03 - 12.08.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20 40 60 80 100					W <sub>P</sub>	W	W <sub>L</sub>		
98.3																	
98.0	TOPSOIL (125mm)																
0.1	Silty CLAY, some sand seams Stiff Brown (Cl)						98										
			1	SS	8												
96.8							97									0 3 75 21	
1.5	END OF BOREHOLE AT 1.47m. AUGER REFUSAL ON PROBABLE BEDROCK OR BOULDERS.																

ONTMT4S 7450OVR.GPJ 07/04/06

## METRIC

ELEV DEPTH	SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT  NATURAL MOISTURE CONTENT  LIQUID LIMIT	UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			
95.2								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	20 40 60 20 40 60		

[illegible]

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



# RECORD OF BOREHOLE No OVR 05-2

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 156.8 E 317 016.5 (CPR Overhead - Amprior) ORIGINATED BY SLL  
 HWY 17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM  
 DATUM Geodetic DATE 15.12.05 - 15.12.05 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
97.0								20 40 60 80 100				
0.0	TOPSOIL (200mm)							20 40 60 80 100				
0.2	Silty CLAY, occasional silty sand seams Firm Brown		1	SS	7							
			2	SS	7							
94.0												
2.9	AUGER REFUSAL AT 2.95 m. CORING STARTED AT 2.95 m. CRYSTALLINE LIMESTONE (BEDROCK), Slightly weathered to fresh, very thin to thin bedded, whitish grey with dark grey and white subvertical banding, strong to very strong		1	RUN								
			2	RUN								
			3	RUN								
90.7												
6.2	END OF BOREHOLE AT 6.25 m. BOREHOLE GROUTED WITH BENTONITE TO GROUND SURFACE.											

ONTMT4S 5182-PHASE II.GPJ 30/03/06

RECORD OF BOREHOLE No OVR 05-3

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 119.5 E 316 993.8 (CPR Overhead - Amprior) ORIGINATED BY SLL  
HWY 17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM  
DATUM Geodetic DATE 14.12.05 - 14.12.05 CHECKED BY SP

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	
99.9							20	40	60	80	100	20	40	60				
0.0	TOPSOIL (200mm)																	
0.2	Silty CLAY, occasional topsoil inclusions, within 1m depth Very Stiff Brown		1	SS	21													
	Becoming Firm		2	SS	5													
	occasional sand seams		3	SS	3													
			4	SS	3													
94.6																		
5.3	AUGER REFUSAL AT 5.31 m. CORING STARTED AT 5.31 m. CRYSTALLINE LIMESTONE BEDROCK, Slightly weathered to fresh, very thinly to thinly bedded, whitish grey with dark grey and white subvertical banding, strong		1	RUN														
	occasional quartzite infilling		2	RUN														
			3	RUN														
91.4																		
8.5	END OF BOREHOLE AT 8.51 m. BOREHOLE GROUTED WITH BENTONITE TO GROUND SURFACE.																	




ONTMT4S 5182-PHASE II.GPJ 05/04/06

RECORD OF BOREHOLE No OVR 05-4

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 112.0 E 317 047.2 (CPR Overhead - Amprior) ORIGINATED BY SLL  
HWY 17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM  
DATUM Geodetic DATE 19.12.05 - 19.12.05 CHECKED BY SP

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 (%)					
								20	40	60						80	100	20	40	60	
								○ UNCONFINED	+	FIELD VANE						×	LAB VANE				
98.1																					
0.0	TOPSOIL (150mm)																				
0.2	Silty CLAY, trace rootlets Firm to Stiff Brown		1	SS	8																
96.4			2	SS	50/																
1.7	END OF SOIL SAMPLING AT 1.70 m. CORING STARTED AT 1.70 m. CRYSTALLINE LIMESTONE (BEDROCK). Slightly weathered to fresh, very thinly to thinly bedded, whitish grey with dark grey and white subvertical banding, strong				.025																
			1	RUN																	
			2	RUN																	
93.3																					
4.8	END OF BOREHOLE AT 4.78 m. BOREHOLE GROUTED WITH BENTONITE TO GROUND SURFACE.																				

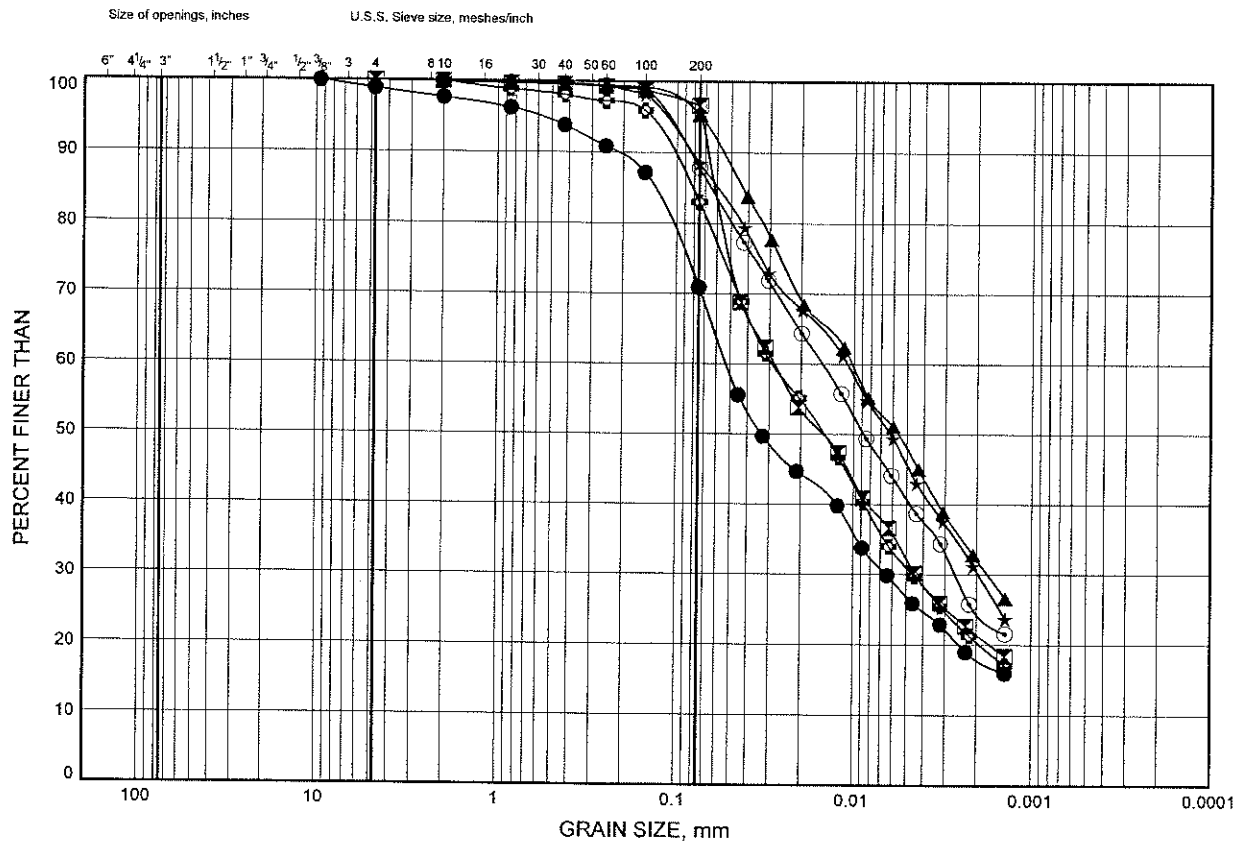
## **Appendix B**

### **Laboratory Test Results**

# HWY 17 Twinning, Amprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B1

## SILTY CLAY

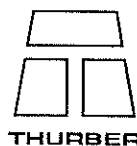


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	OVR-13	0.76	96.74
⊠	OVR-14	1.22	97.08
▲	OVR-4	0.91	96.75
★	OVR-5	0.91	98.20
⊙	OVR-8	1.83	95.45
⊛	OVR-9	1.83	96.13

Date April 2004

Project 647-92-00



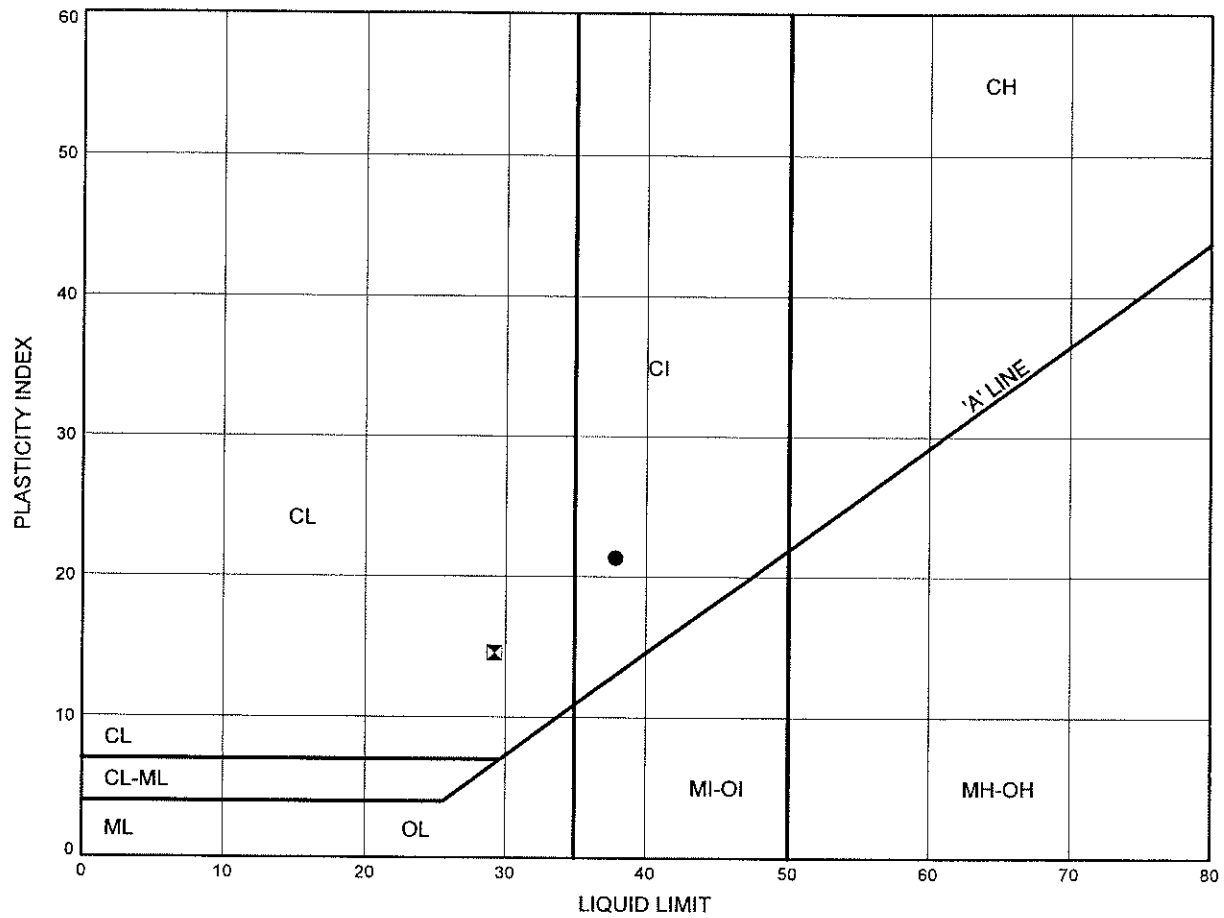
Prep'd SS

Chkd. SKP

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

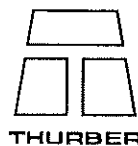
FIGURE B2

### SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	OVR-4	0.91	96.75
⊠	OVR-9	1.83	96.13

Date April 2004  
Project 647-92-00

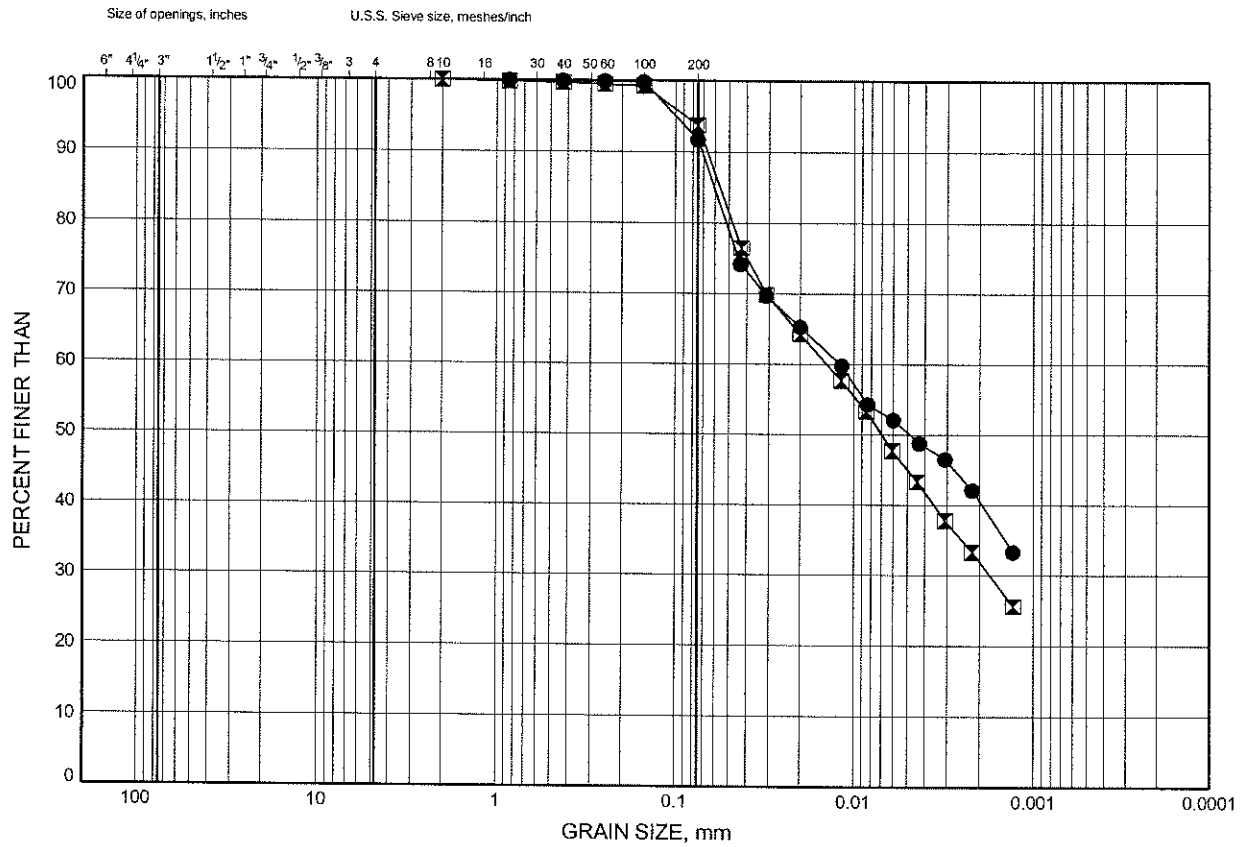


Prep'd SS  
Chkd. SKP

# HWY 17-417 GRAIN SIZE DISTRIBUTION

FIGURE B3

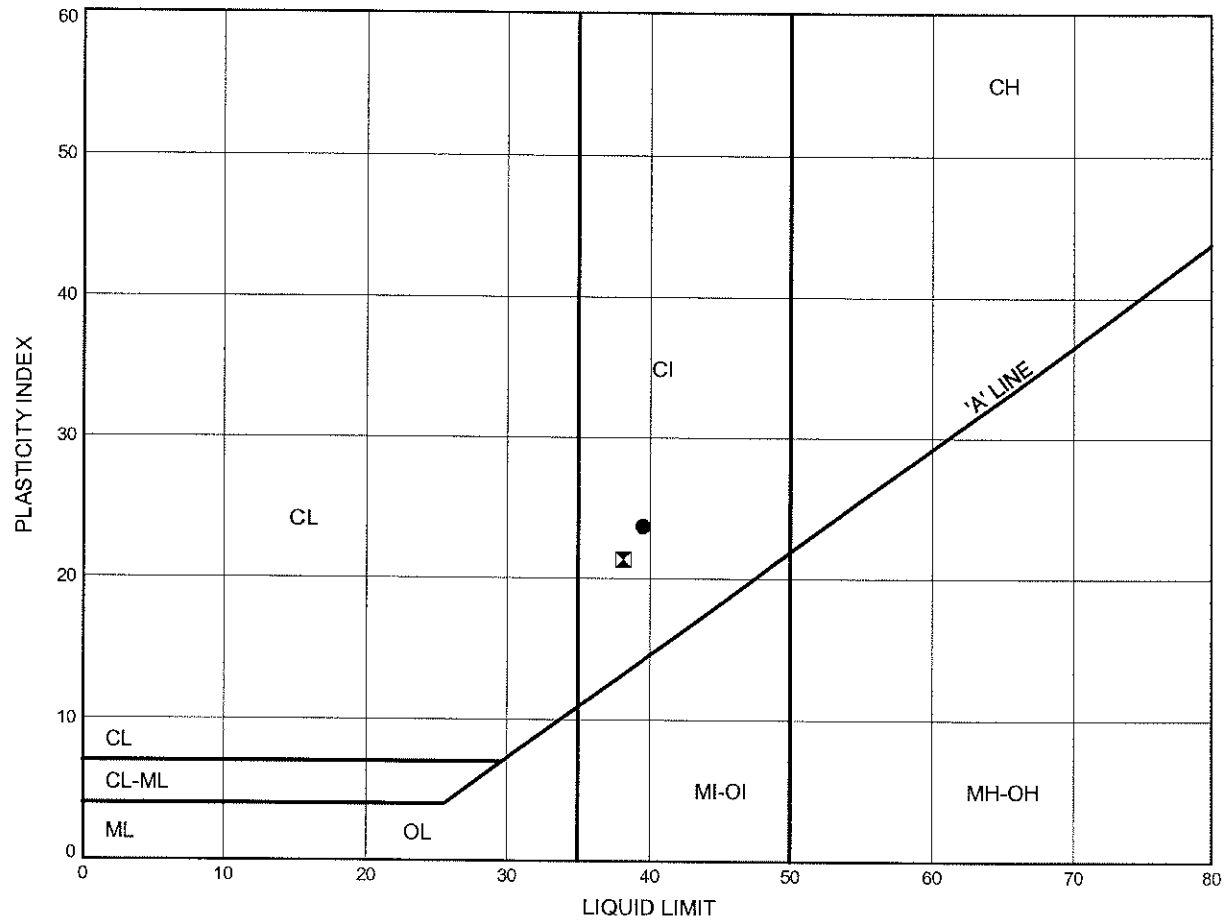
## SILTY CLAY



# HWY 17-417 ATTERBERG LIMITS TEST RESULTS

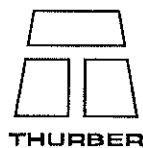
FIGURE B4

## SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	OVR 05-2	1.83	95.12
⊠	OVR 05-3	3.35	96.55

Date March 2006  
 Project 647-92-00



Prep'd JHL  
 Chkd. SKP



## **Appendix C**

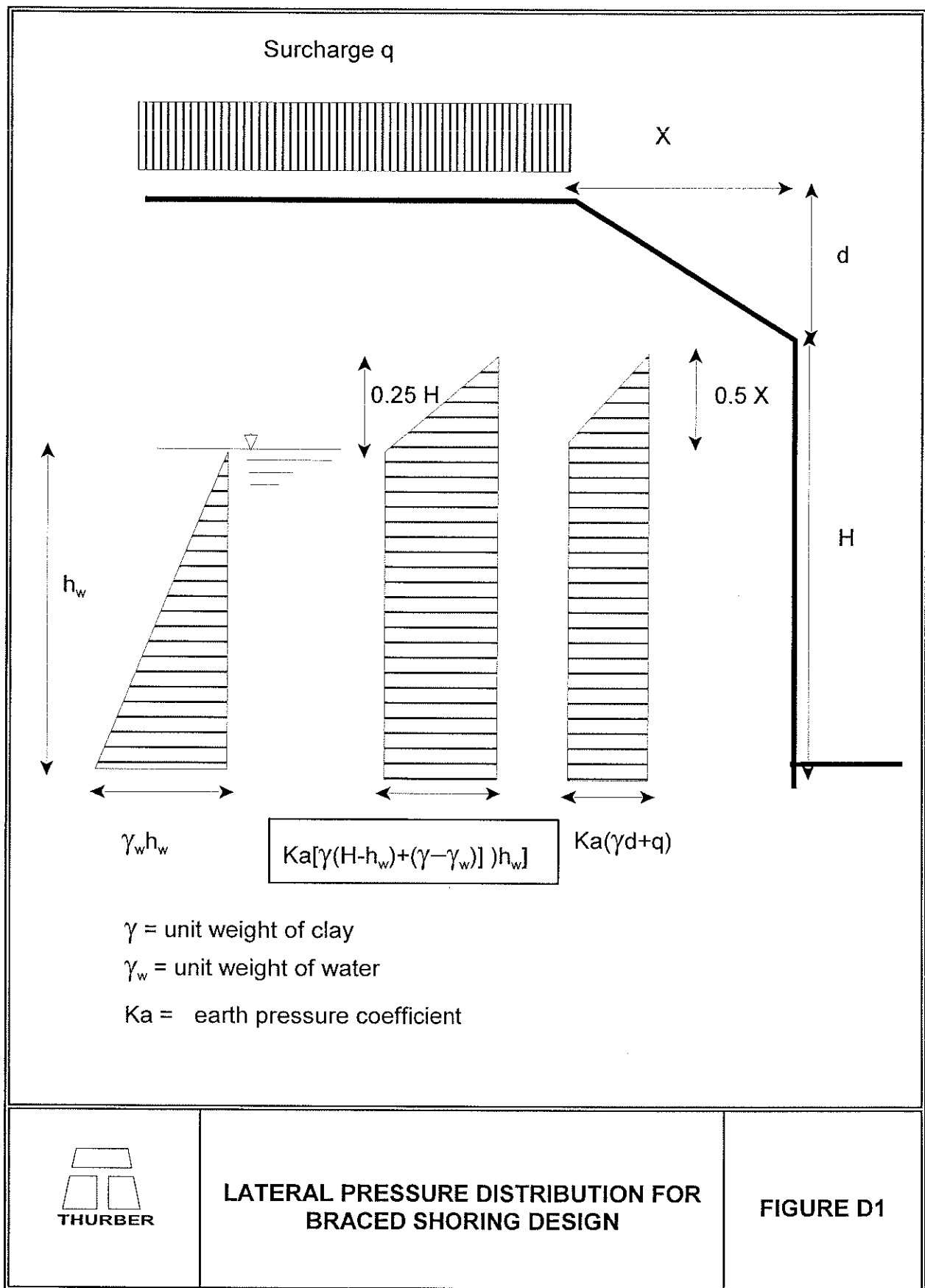
### **Foundation Comparison**

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

Foundation Element	Driven Piles	Footings on Bedrock	Footings on Engineered Fill	Augered Caisson
West and East Abutments	<p><b>Advantages:</b> None identified</p> <p><b>Disadvantages:</b> i. Shallow bedrock surface rendering the use of driven piles unnecessary and impractical.</p>	<p><b>Advantages:</b> i. Shallow bedrock surface and/or bedrock outcrop at existing ground surface. ii. High values of geotechnical resistance are available on the bedrock.</p> <p><b>Disadvantages:</b> i. High cost of rock excavation, if any, is required. ii. Mass concrete fill required to create a level founding surface.</p>	<p><b>Advantages:</b> None identified.</p> <p><b>Disadvantages:</b> i. Lower geotechnical resistance than bedrock ii. Insufficient height between top of bedrock and proposed highway grade. iii. Footings may have to be located further back resulting in a longer superstructure.</p>	<p><b>Advantages:</b> None identified.</p> <p><b>Disadvantages:</b> i. Shallow bedrock surface rendering the use of augered caissons unnecessary and impractical.</p>

## **Appendix D**

### **Figures**



## **Appendix E**

### **Special Provisions**

## **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.



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

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### **Special Provision**

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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.

**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**

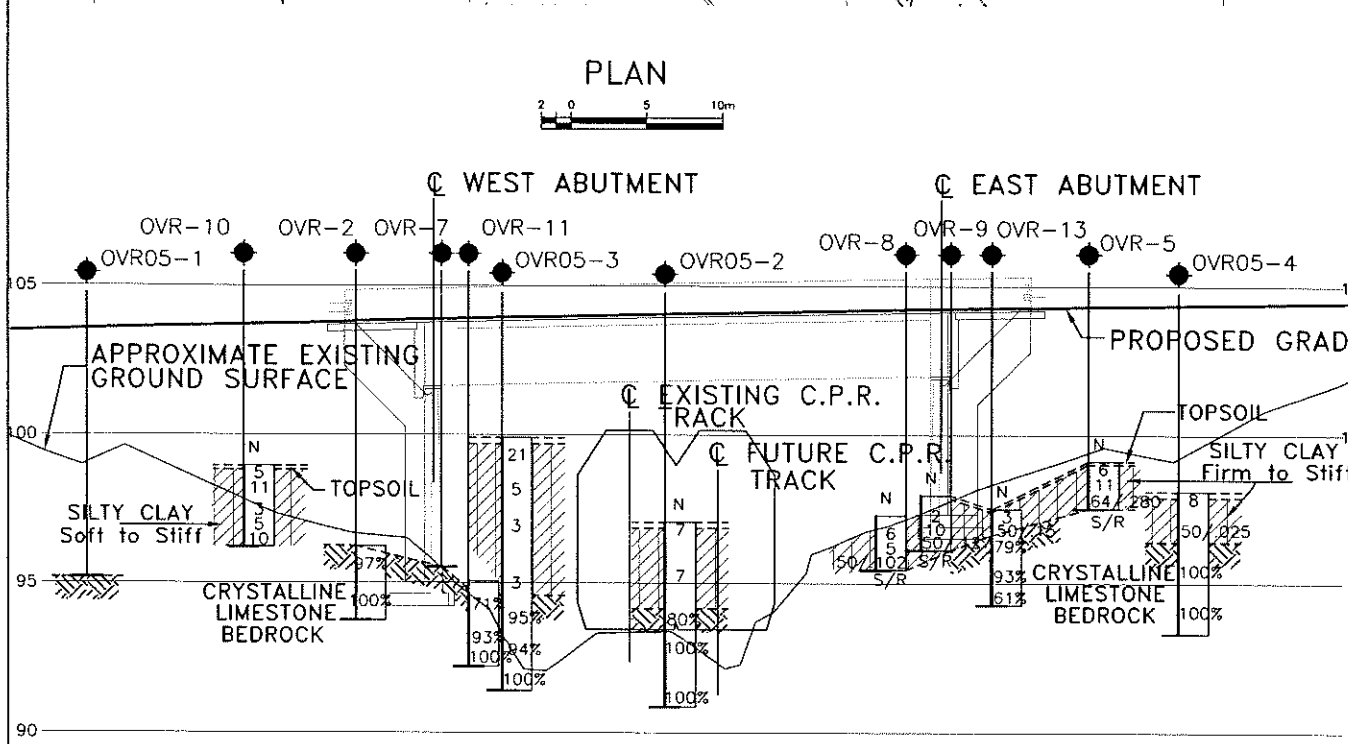
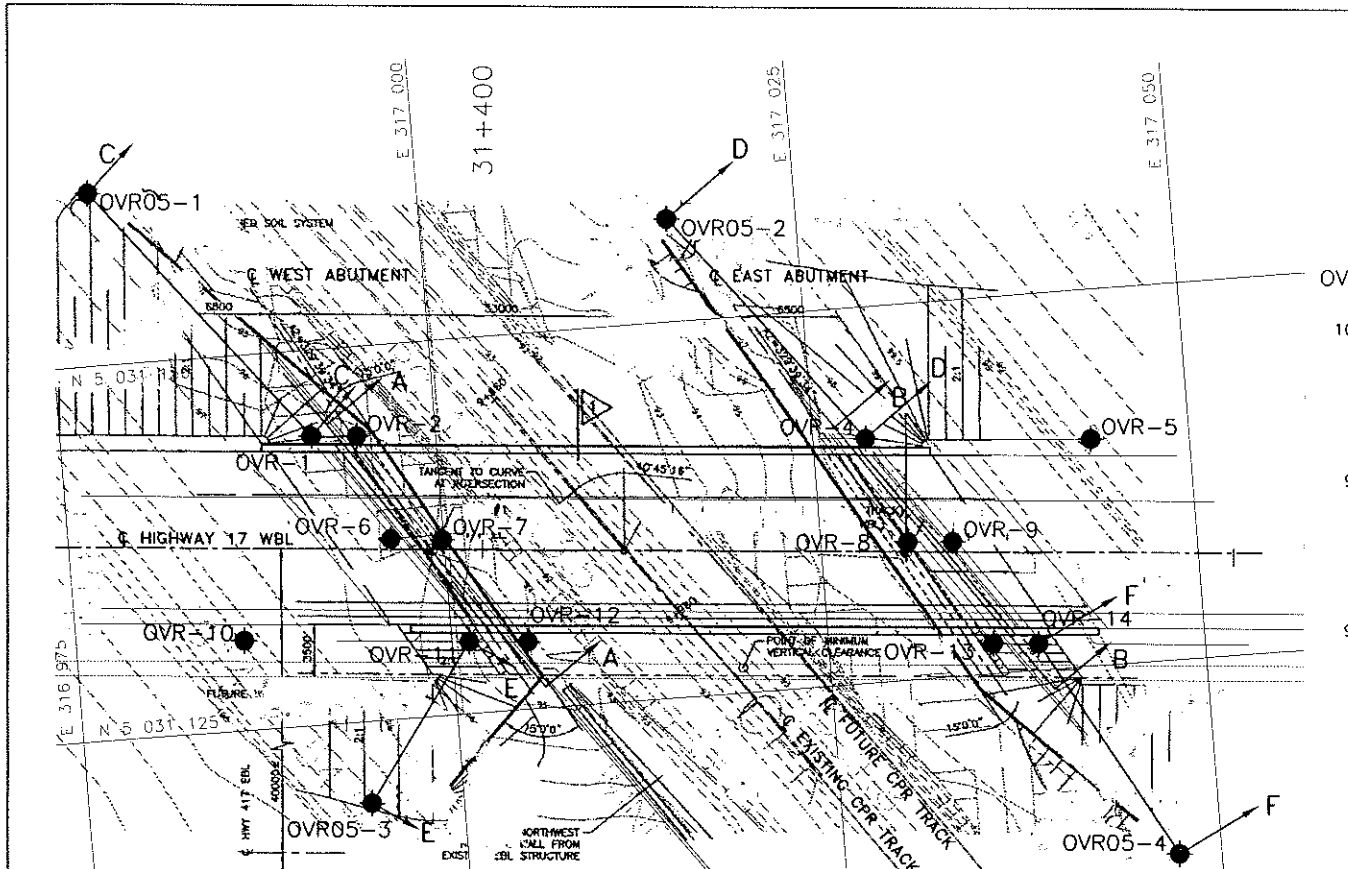
Subsection 902.10.01 of OPSS 902 is amended by deleting the first paragraph and replacing it with the following:

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.

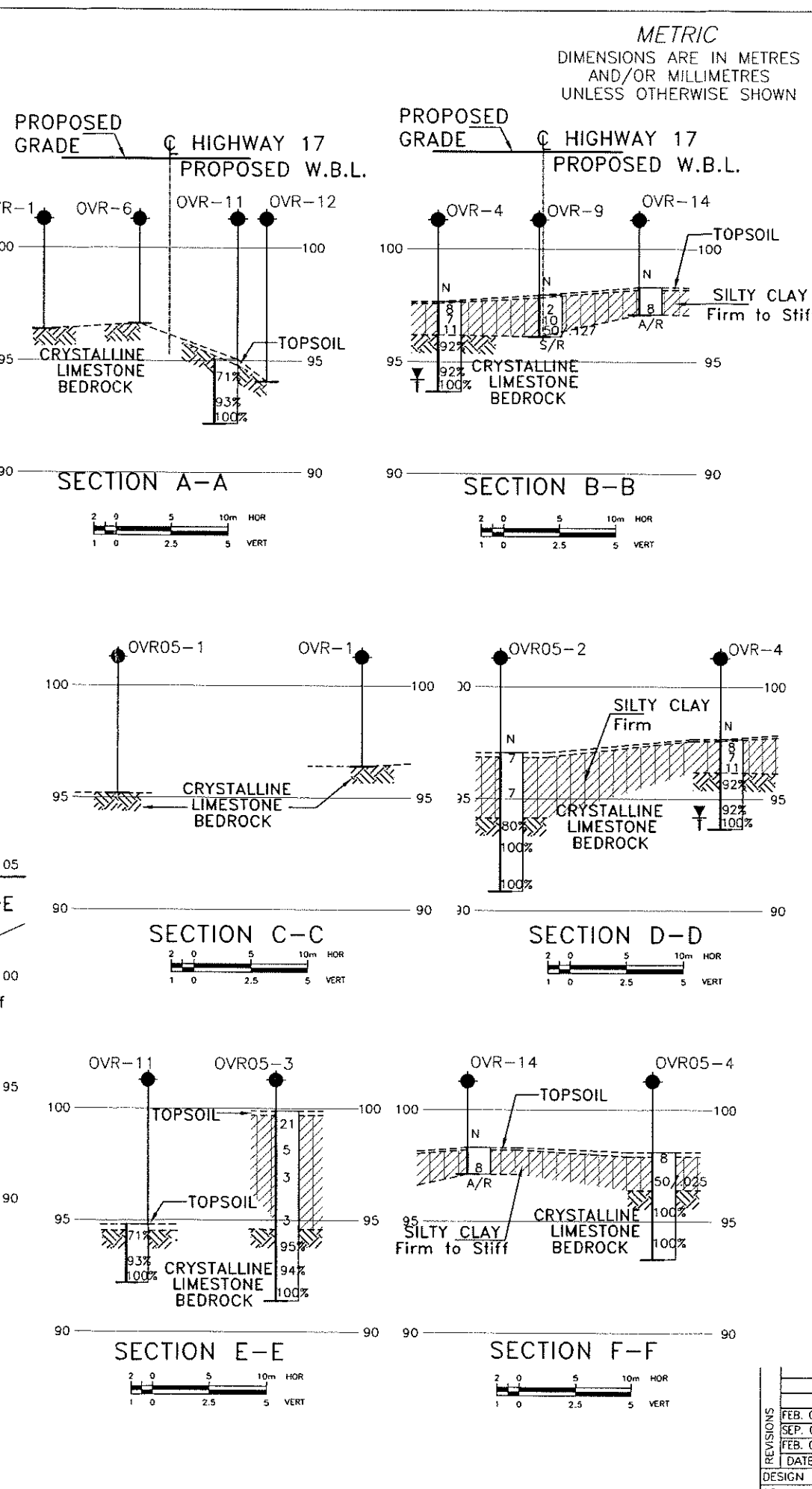
## **Appendix F**

### **Drawings**



LICENSED PROFESSIONAL ENGINEER  
S. PANG  
April 7/06  
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER  
P. K. CHATTERJI  
April 7/06  
PROVINCE OF ONTARIO



HWY.17/417  
WP NO. 647-92-00

HIGHWAY 17 TWINNING  
C.P.R. OVERHEAD ARNPRIOR  
HIGHWAY 17 WBL  
BOREHOLE LOCATIONS AND STRATA DRAWING

MCCORMICK RANKIN CORPORATION

THURBER ENGINEERING LTD.

SHEET

**KEYPLAN**

**LEGEND**

- Bore Hole
- Bore Hole with piezometer
- 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
OVR05-1	95.2	5 031 161.4	316 978.2
OVR05-2	97.0	5 031 156.8	317 016.5
OVR05-3	99.9	5 031 119.5	316 993.8
OVR05-4	98.1	5 031 112.0	317 047.2
OVR-1	96.4	5 031 144.3	316 991.7
OVR-2	96.2	5 031 144.1	316 994.7
OVR-4	97.7	5 031 141.3	317 028.6
OVR-5	99.1	5 031 140.1	317 043.5
OVR-6	96.7	5 031 137.1	316 996.4
OVR-7	95.5	5 031 136.8	316 999.8
OVR-8	97.3	5 031 134.2	317 030.8
OVR-9	98.0	5 031 134.0	317 033.8
OVR-10	98.9	5 031 131.1	316 986.2
OVR-11	95.1	5 031 129.8	317 001.2
OVR-12	94.1	5 031 129.5	317 005.1
OVR-13	97.5	5 031 127.0	317 036.0
OVR-14	98.3	5 031 126.8	317 039.0

**NOTES**

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

**REVISIONS**

NO	DATE	BY	DESCRIPTION
1	FEB. 06	SP	FINAL (REVISED)
2	SEP. 04	SP	FINAL
3	FEB. 04	SP	ISSUED AS DRAFT FOR REVIEW

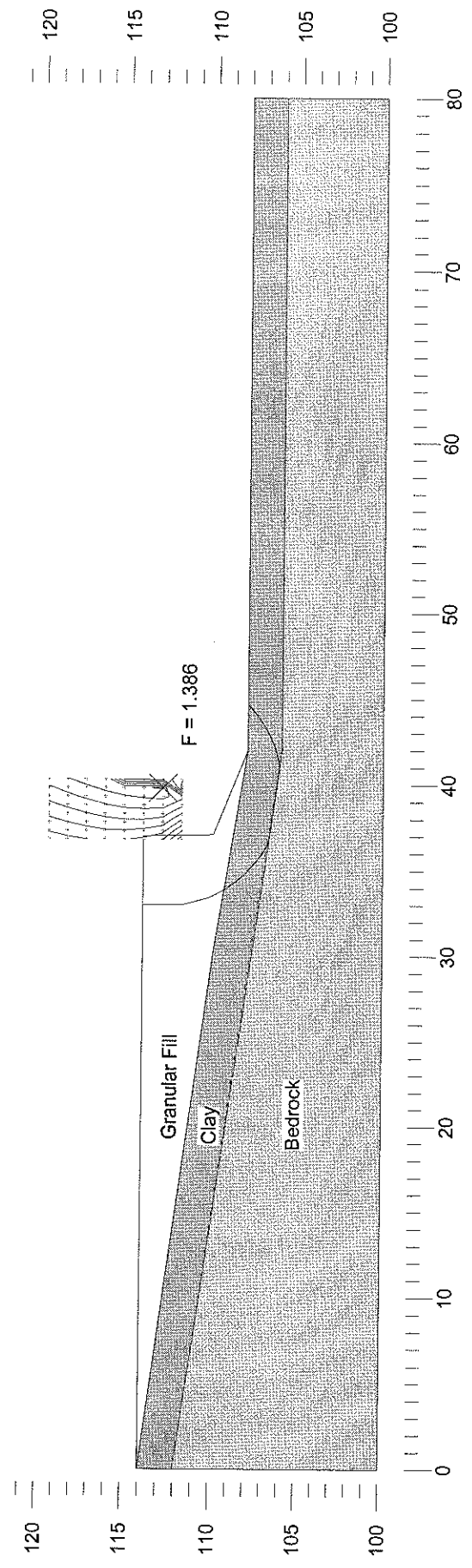
**DESIGN** SP CHK PKC CHBDC 2000 LOAD DATE FEB 2006  
**DRAWN** SS CHK SP SITE29-423/2 STRUCT DWG. 19-1351-82-CP1

## **Appendix G**

### **Selected Stability Analyses Results**

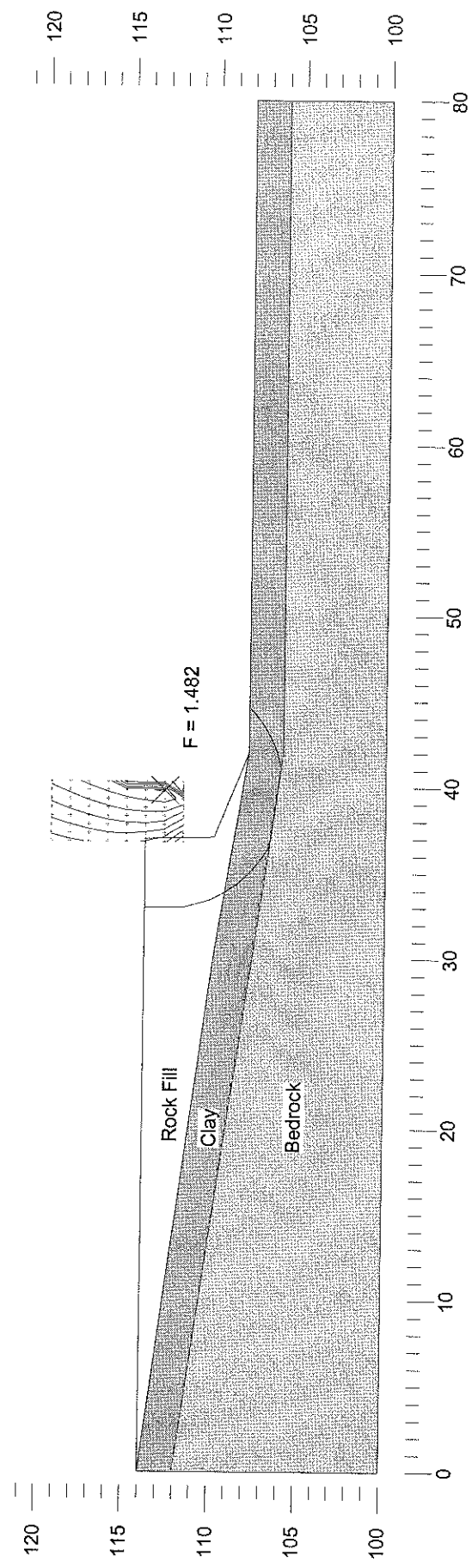
	Gamma	C	Phi	Min	Piezo
	kN/m3	kPa	deg	c/p	Surf.
Granular Fill	21	0	30	0	0
Firm Clay	20	0	26	0	0
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto  
 19-3745-0  
 Highway 17 Twinning  
 August 16, 2004  
 Stability of Forward Slope - CPR Overhead (Amprior) - East Approach  
 Figure G1 Drained Analysis - Granular Fill



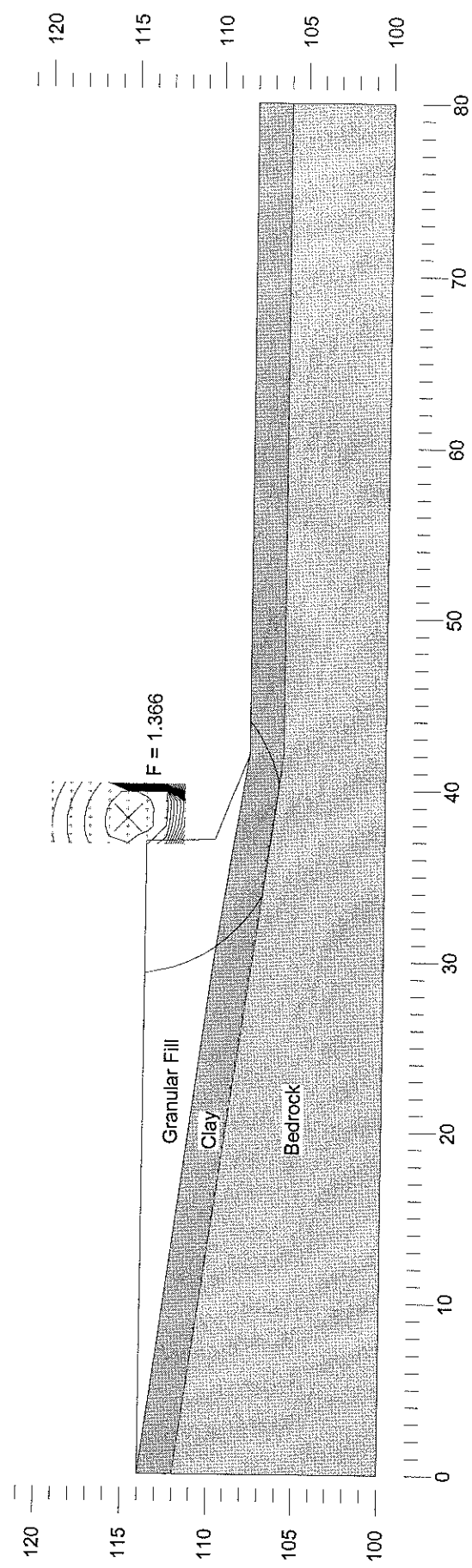
	Gamma	C	Phi	Min	Piezo
	kN/m3	kPa	deg	c/p	Surf.
Rock Fill	19	0	42	0	0
Firm Clay	20	0	26	0	0
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto  
 19-3745-0  
 Highway 17 Twinning  
 August 16, 2004  
 Stability of Forward Slope - CPR Overhead (Arnprior) - East Approach  
 Figure G2 Drained Analysis - Rock Fill



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

Thurber Engineering Ltd. - Toronto  
 19-3745-0  
 Highway 17 Twinning  
 August 16, 2004  
 Stability of Forward Slope - CPR Overhead (Amprior) - East Approach  
 Figure G3 Undrained Analysis - Granular Fill





	Gamma	C	Phi	Min	Piezo
	kN/m3	kPa	deg	c/p	Surf.
Rock Fill	19	0	42	0	0
Firm Clay	20	30	0	0	0
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto  
 19-3745-0  
 Highway 17 Twinning  
 August 16, 2004  
 Stability of Forward Slope - CPR Overhead (Arnprior) - East Approach  
 Figure G4 Undrained Analysis - Rock Fill

