

**FOUNDATION INVESTIGATION AND DESIGN REPORT
UPPER ACCESS ROAD OVERPASS
HIGHWAY 17 TWINNING
ARNPRIOR TO RENFREW, ONTARIO
G.W.P. 647-92-00, SITE NO. 29-201/1
GEOCRES Number: 31F-124**

Report to

National Capital Engineering

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation carried out at the location of the Upper Access Road Overpass structure that will carry the Highway 17 westbound lanes over the Upper Access Road to the OPG Arnprior Dam. A previous investigation had been carried out by the Ministry of Transportation (MTO) for the culvert that carries the access road under the existing Highway 17, and the factual data from that investigation has been used as reference during the preparation this report.

The purpose of this investigation was to determine the subsurface conditions at the locations of the proposed foundation elements and approach embankments, 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 primarily based on the data obtained from this investigation, and with reference to the data obtained from the previous MTO investigation.

Thurber carried out the current investigation as a sub-consultant to National Capital Engineering, under the MTO Purchase Order Number 4005-A-000157.

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

- MTO Report titled "Foundation Investigation and Design Report, Hwy. 17N, Hydro Dam Upper Access Road Overpass", W.P. 78-76-01, Dist.9, GEOCRES No. 31F-93, dated Nov.23, 1976 (Reference 1)

2 SITE DESCRIPTION

The site is located on the north side of Highway 17, just south of the Town of Arnprior, Township of McNab, County of Renfrew (approximate mainline Station 31+250). The Arnprior Generating Station and associated dam are situated on the south side of the highway at this location. The general site location is shown on the Borehole Locations and Soil Strata drawing in Appendix F.

An existing 8.23 m diameter circular, multi-plate culvert allows the Upper Access Road, which leads to the crest of the dam, to cross under the existing Highway 17.

The site is located in an area characterized by bedrock outcrops and shallow glacial drift. The ground is typically sloping towards the Madawaska River to the west. Vegetation is light around the site area and mainly consists of grass and some small trees. A west facing slope is situated immediately to the east of Upper Access Road. Drainage in the area is largely governed by the nearby Madawaska River.

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

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation program was carried out on August 5 and 6, November 12 and 13, 2003. The borehole investigation program originally consisted of twenty (20) boreholes, numbered UAR-1 to UAR-20. Boreholes UAR-11 to UAR-15, inclusive, at the proposed east abutment location were located on sloping terrain and were, therefore, inaccessible without cutting a bench on the existing slope. Thurber was subsequently advised by MTO and NCE that these holes should be eliminated from the field program. On March 13, 2004, an additional Borehole UAR-16A was advanced in the immediate vicinity UAR-16. Boreholes UAR-7 and UAR-8 were not drilled due to the presence of rock outcrop at the intended borehole locations. Borehole UAR-10 was advanced using hand probing techniques. Depths of termination of the boreholes range from 0.6 m to 16.5 m below existing ground surface. The approximate locations of all relevant boreholes 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 Ltd (Ottawa), and clearance of buried utilities at the borehole locations was obtained by Thurber prior to any drilling being carried out.

The drilling, sampling and in-situ testing operation was carried out using truck or track (Bombardier) mounted CME 55 or 75 drill rigs that were supplied and operated by George Downing Estate Drilling Ltd. Auger drilling techniques were used to advance most of the boreholes and to obtain soil samples using a split spoon sampler in conjunction with the Standard Penetration Testing (SPT) where overburden soils were encountered. Field vane shear tests were carried out within the silty clay deposit in Borehole UAR-16A. Boreholes UAR-1A, UAR-2A, UAR-3A, UAR-4A, UAR-6 and UAR-9 located at the foundation elements (except for the east

abutment), Borehole UAR-16A at the east approach, as well as Boreholes UAR-17, UAR-18, UAR-19 and UAR-20 located along Upper Access Road, were advanced 3 m to 4 m into bedrock by NQ size rotary core drilling techniques.

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, and the recovered soil and rock samples were placed in labelled containers and core boxes, and transported to Thurber's Oakville laboratory for further examination and testing.

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

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers were installed in Boreholes UAR-1A, UAR-3A, UAR-17 and UAR-20 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, 19 mm diameter Schedule 40 PVC pipes with a 1.52 m long slotted screens were installed at the bottom of the open borehole. The sand screens surrounding the pipe were in the order of 2 m long. The remaining space in the borehole was either grouted with a bentonite-based grout, or backfilled with auger cuttings after a bentonite holeplug seal was placed on top of the screen.

Upon completion of drilling and sampling, all boreholes were appropriately backfilled.

The ground surface elevations and plan co-ordinates (northings and eastings) at the locations of all the boreholes have been established in the field and the survey data forwarded to Thurber by J.D. Barnes Ltd.

4 LABORATORY TESTING

All recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A.

Selected soil samples were subjected to grain size distribution analysis and Atterberg Limits tests. A total of 9 samples were selected for these tests and the results are shown on the Record of Borehole sheets in Appendix A and on the plots in Appendix B.

Point load testing was carried out at selected locations on the rock cores and the results are presented in Table 1 attached immediately following the text.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are also presented on the Borehole Locations and Soil Strata drawing in Appendix F. A summary description of the stratigraphy is given in the following paragraphs.

In general, the subsurface conditions encountered in the boreholes consist of topsoil, sand and gravel fill and a relatively thin deposit of native, clayey silt to silty clay, overlying crystalline limestone bedrock.

5.1 Topsoil

Topsoil of a typical black colour was encountered at various locations across the site with thickness ranging from 50 mm to 150 mm as shown in the table below. In Borehole UAR-6, the topsoil was found mixed with clayey silt giving a combined thickness of 400 mm.

Borehole	Topsoil Thickness (mm)
UAR-1	150
UAR-2	150
UAR-3	125
UAR-4	150
UAR-5	50
UAR-6	400 (mixed with clayey silt)
UAR-9	150
UAR-10	150
UAR-16	150
UAR-17	50

5.2 Fill

Along the edges of Upper Access Road, sand and gravel, gravelly to silty sand fill with cobbles was present below the topsoil or the surficial rockfill, which was encountered at ground surface in Boreholes UAR-18, UAR-19 and UAR-20. A thicker layer of silty sand fill was encountered at ground surface near the crest of the existing east slope. The fill was found extending between approximate Elevations 89 m and 92 m. The approximate depths of fill are as shown in the table below.

Borehole	Fill Thickness (mm)
UAR-1	450
UAR-2	750
UAR-3	575
UAR-4	450
UAR-16	1,950
UAR-18	800
UAR-19	300
UAR-20	300

Where measured in the sand and gravel fill, SPT 'N' values range typically between 28 blows per 0.3 m penetration to greater than 50 blows per 0.3 m penetration. These values indicate that the soil matrix of this fill is in a generally compact to dense state, and suggest

the presence of cobbles or rock fragments where high 'N' values were recorded. Figure B1 shows the grain size distribution of two selected samples of the gravelly sand and silty sand fill. Measured moisture contents of this fill were generally less than 5%.

In Borehole UAR-16 at the top of the east slope, measured 'N' values within the silty sand fill range between 3 blows and 10 blows indicating a very loose to loose state. Measured moisture contents of this fill varied between 10% and 20%.

5.3 Clayey Silt to Silty Clay

A deposit of native, cohesive clayey silt to silty clay was encountered below the topsoil and/or fill across the site. At locations to the west and along Upper Access Road, this deposit was relatively thin with thicknesses ranging between 0.2 m and 0.8 m. At locations immediately to the north of the existing culvert on the access road, this deposit had thicknesses varying between 3.9 m (includes a 1.1 m thick sandy silt interlayer) and 2.0 m, respectively. The east slope is underlain by a thicker silty clay deposit which was fully penetrated in Borehole UAR-16A. This deposit was encountered from 2.1 m to a depth of 11.4 m below the existing slope crest. The clay deposit was found extending between approximate Elevations 92 m (slope toe) and 101 m (slope crest).

Cobbles and boulders were encountered within the silty clay in Borehole UAR-4A, and inferred in the clayey silt in Borehole UAR-9. The presence of cobbles and boulders at other locations should also be expected.

The measured SPT 'N' values ranged from 24 blows per 0.3 m penetration, indicating that the silty clay has a very stiff consistency, to greater than 50 blows per 0.3 m penetration, inferring the presence of cobbles. Within the thicker deposits at Boreholes UAR-16, UAR-19 and UAR-20, the 'N' values ranged from 8 blows to 17 blows per 0.3 m penetration indicating stiff to very stiff consistency. Occasional lower 'N' values of 4 to 6 blows were measured in Boreholes UAR-16A and UAR-20. Field vane shear strengths of about 50 kPa and 100 kPa were measured at two elevations in Borehole UAR-16A, indicating firm to stiff consistency.

Figure B2 shows the grain size distributions of four silty clay samples. These tests indicate that the clay content of this soil ranges between 20% and 47%. Figure B3 is a plasticity chart showing that the silty clay samples had measured plasticity indices of between 19% and 26%, indicating a medium plasticity (group symbol of CI). The plasticity is a function of the clay content and is anticipated to be lower as the clay content decreases.

Measured moisture contents of these cohesive deposits typically varied from 20% to greater than 40%.

5.4 Sand to Sandy Silt

A layer of sand and sandy silt was encountered in three of the boreholes (UAR-5, UAR-16A and UAR-19). The thickness of this layer ranged between 0.7 m and 1.8 m. In Boreholes UAR-5 and UAR-19, the measured SPT 'N' values were 16 and 17 blows per 0.3 m penetration indicating a compact state. In Borehole UAR-16A, an 'N' value of 81 blows was measured indicating a very dense state. Measured moisture contents varied from 5% to 25%.

5.5 Bedrock

At locations to the west and along the Upper Access Road (immediately north of the existing culvert), the bedrock surface was exposed or covered by a thin veneer of overburden. The thickness of the silty clay to clayey silt increases toward the east as the ground surface slopes upward. Bedrock was not encountered in Borehole UAR-16 located on the slope crest. Bedrock does not outcrop on the surface of the east slope, where no borehole was drilled due to sloping ground that was inaccessible to drill rigs.

Where encountered at the borehole locations, bedrock surface was either identified by outcrops, proven by coring, or inferred by refusal to auger and split spoon sampler penetration. Bedrock surface depths and elevations at the borehole locations are summarized in the following table.

Borehole	Ground Surface Elevation (m)	Bedrock Surface	
		Depth (m)	Elevation (m)
UAR-1/1A	91.7	0.8	90.9
UAR-2/2A	91.8	1.1	90.7
UAR-3/3A	91.7	1.0	90.7
UAR-4/4A	91.8	2.3	89.4
UAR-5	87.0	0.7*	86.3
UAR-6	89.3	0.4	88.9
UAR-7	89.6	0.0*	89.6
UAR-8	89.2	0.0*	89.2
UAR-9	88.4	1.1	87.3
UAR-10	89.1	1.2*	87.9
UAR-16A	103.0	13.2	89.8
UAR-17	91.7	0.9	90.8
UAR-18	91.4	0.8	90.6
UAR-19	91.9	4.2	87.7
UAR-20	91.8	2.3	89.5

Note : * Bedrock surface inferred from auger or spoon sampler refusal, or rock exposure at the borehole location.

Rock coring was carried out in Boreholes UAR-1A, UAR-2A, UAR-3A, UAR-4, UAR-6, UAR-9, UAR-16A, UAR-17, UAR-18, UAR-19 and UAR-20. Based on the core samples recovered, the bedrock is described as a very thinly to thinly bedded, grey, crystalline limestone with sub-vertical bandings. The cores are generally in a fresh state with slight weathering at the joints, except in Borehole UAR-16A where the cores were slightly to moderately weathered.

The measured Total Core Recovery (TCR) values for the bedrock core runs vary between 92% and 100%. The measured Rock Quality Designation (RQD) values typically range from 77% to 100% (generally increasing with depth), indicating good to excellent rock quality. The RQD ranged from 35% to 60% in Borehole UAR-16A and occasional lower values of 44% and 56% were also measured in other boreholes at this site. These values indicate poor to fair rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally very low with values of 0 and 1, except near the rock surface where values of between 2 to greater 5 were measured. Multiple fractures were, however, noted at shallow depths in Boreholes UAR-16A and UAR-20. Sub-vertical to occasional vertical joints were observed at some locations. The condition of the joints was typically uneven and rough, although some planar and smooth joints were noted. Iron oxide staining and/or calcite infilling was evident in some fractures.

In Borehole UAR-16A, an infilled cavity was encountered below the top of bedrock, between 14.8 m and 15.4 m depths. The infilling material consists of green and grey coloured sand, silt and clay.

Point load tests were carried out on the rock cores at regular 0.3 m intervals. The inferred Unconfined Compressive Strengths (UCS) of the rock cores typically range between 66 MPa and 115 MPa, indicating that the intact rock is strong to very strong. Occasional values of less than 50 MPa were recorded at shallow depths below the rock surface. A summary of the Point Load Test Results is presented in Table 1 attached immediately following the text.

5.6 Water Levels

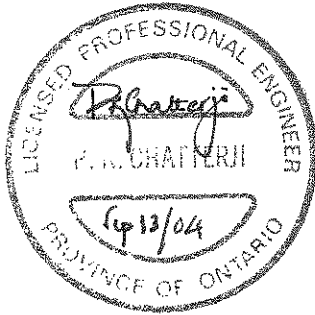
Free groundwater was not observed in any of the boreholes on completion of drilling. A standpipe piezometer was installed within the bedrock in each of Boreholes UAR-1A, UAR-3A, UAR-17 and UAR-20. The piezometers were partially filled with drill water upon completion of rock coring. Based on site observations and information from Reference 1, it is likely that there is downward drainage into bedrock or into the sandy layers immediately above bedrock. Piezometric readings were taken on February 4, 2004 and presented in the following table.

Borehole	Water Levels	
	Depth (m)	Elevation (m)
UAR-1A	3.0	88.7
UAR-3A	3.6	88.1
UAR-17	0.0 (frozen, suspected drill water)	
UAR-20	Covered by ice and snow	

It should be noted that the above groundwater conditions are short term observations and the water levels are subject to seasonal fluctuations and severe climatic events. It is also anticipated that the local groundwater conditions at this site is largely governed by the nearby Madawaska River.



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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 structure to the north of the existing culvert to carry the future westbound lanes of the twinned Highway 17 over the Upper Access Road. The existing culvert will continue to carry Upper Access Road under the existing Highway 17 that will be used as the eastbound lanes of the twinned Highway 17.

The proposed structure will consist of a three-span underpass bridge comprising of CPCI concrete girders. Each approach span will be approximately 10.5 m in length and the centre span will be about 14.5 m in length. The structure alignment will be skewed at about 27° to the Highway 17 alignment.

At the bridge site, the proposed grade of the Highway 17 westbound lanes will be at about approximate Elevation 102.5 m at the east abutment and will decrease to approximate Elevation 101 m at the west abutment. The west approach fills will be up to approximately 14 m above existing ground surface. At the east approach area where there is the existing east slope, it is understood that a wedge of fill of up to 6.5 m in maximum height will be placed to raise the grade. Retaining walls may be required parallel to Upper Access Road to retain the embankment fill.

The discussions 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, with reference to those available from the previous MTO investigation for the existing culvert (Reference 1).

7 STRUCTURE FOUNDATIONS

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



The stratigraphy encountered at the locations of the proposed west abutment and the two piers consists of relatively thin layers of fill and overburden deposits overlying bedrock. Bedrock outcrops at several locations in the areas of the west abutment and approach. At the existing east abutment location, however, a slope consisting of stiff silty clay is present. The top of bedrock was not established at the east abutment during this investigation due to inaccessible sloping ground. All the planned boreholes at the east abutment, UAR-11 to UAR-15, were eliminated upon the direction of MTO and NCE (see previous section). Based on existing boreholes and site observations of the surroundings, it is assumed that bedrock is present at approximately 4 m to 5 m below the slope surface. It is recommended that boreholes be drilled at the location of the east abutment prior to final design.

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

Location	Borehole Number	Elevations (m)	
		Existing Ground Surface	Top of Bedrock Surface
West Abutment			
Northwest Corner	UAR-6	89.3	88.9
Northeast Corner	UAR-7	89.6	89.6*
Interior	UAR-8	89.2	89.2*
Southwest Corner	UAR-9	88.4	87.3
Southeast Corner	UAR-10	89.1	87.9**
West Pier			
North Limit	UAR-3	91.7	90.7
South Limit	UAR-4	91.8	89.4
East Pier			
North Limit	UAR-1	91.7	90.9
South Limit	UAR-2	91.8	90.7
East Abutment			
-	UAR-16A	103.0	89.8
No borehole was drilled at this location due to sloping ground that was inaccessible to drill rigs without significant access preparation (discussed previously in an earlier section of this report). Borehole UAR-16A was drilled instead at the east approach to provide information for interpolation in order to estimate the bedrock surface at the east abutment.			

Note : * rock outcrop.

** inferred from spoon sampler refusal.

7.1 Foundation Alternatives

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

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

- Spread footings
- Driven piles

- Augered caissons (drilled shafts)

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

An integral abutment design was not considered a feasible option for this bridge primarily due to the skew angle of the structure. It is understood that consideration is being given to a semi-integral abutment design.

At the west abutment location where the proposed grade raise is up to 14 m above existing ground, it is recommended that a perched abutment design be considered where spread footings are founded on an engineered fill pad resting on bedrock to provide foundation support. At the two pier locations where bedrock is present at shallow depths, it is recommended that the spread footings be founded directly on bedrock. At the east abutment, it is recommended that spread footings be founded either on bedrock or on an engineered fill pad. A shored excavation will be required to construct the east abutment footing in a cut formed into the existing clay slope.

The presence of shallow bedrock renders it impractical to use driven piles at this site. Consideration may be given to using augered caissons founded on bedrock at the west abutment. However, an engineered fill platform will need to be constructed (as for the footing option) to accommodate caisson augering equipment prior to installation. At the east abutment where bedrock could be at shallow depth below the existing slope face (to be confirmed by future boreholes), the use of augered caissons may also be impractical. If caissons are used, benching into the existing slope and temporary shoring will also be required (as for the footing option) to facilitate access of caisson installation equipment.

7.2 Spread Footings on Bedrock

7.2.1 General

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

Rock excavation is expensive and un-necessary excavation of bedrock should be avoided where practical.

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

structure to accommodate changes in the elevation of the top of bedrock. The recommended design top of rock is as follows.

West Abutment

The top of rock varies from Elevation 89.6 m near the north limit of the proposed footing to Elevation 87.3 m near the south limit.

West Pier

The top of rock varies from Elevation 90.7 m near the north limit of the proposed footing to Elevation 89.4 m near the south limit.

East Pier

The top of rock varies from Elevation 90.9 m near the north limit of the proposed footing to Elevation 90.7 m near the south limit.

East Abutment

The top of rock is estimated to be at approximate Elevation 90 m based on information obtained at the east pier location. Further investigation is required at the east abutment location to confirm bedrock elevation and quality.

7.2.2 Bearing Resistance

Footings bearing on sound crystalline limestone bedrock encountered at this site may be designed for a factored geotechnical resistance of 5,000 kPa at Ultimate Limit States (ULS) for vertical, concentric loads. Effects of load inclination and eccentricity should be taken into account as illustrated in the CHBDC, 2000 Clause 6.7.3 and Clause 6.7.4. The design of the footing may be governed by other considerations such as sliding resistance or overturning moment.

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

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

7.2.3 Horizontal Resistance of Footings

Resistance to lateral forces / sliding resistance between the concrete footing and the bedrock surface at the pier locations should be evaluated in accordance with the CHBDC, 2000 assuming an ultimate coefficient of friction of 0.7.

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

The dowel may be considered as acting as a fully embedded short pile in the rock. Using a lower bound value of 30 MPa for the unconfined compressive strength of the rock (reduced

to allow for fracturing near the surface) or the strength of the grout, an ultimate horizontal resistance of 1.3 MN was calculated for a 50 mm steel dowel embedded 500 mm into the rock. The depth of embedment is measured below the bearing surface prepared to receive the concrete footing.

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

7.3 Spread Footings on Engineered Fill

For a perched abutment design at the west abutment location, spread footings may be founded on an engineered fill pad. Consideration may also be given to founding the footings on engineered fill at the east abutment.

If an engineered fill pad is used at this site, all overburden materials including topsoil, fill and native soils should be removed and the fill placed directly on the bedrock surface. The engineered fill should consist of OPSS Granular "A" compacted to 100% of its Standard Proctor maximum dry density (SPMDD) at $\pm 2\%$ of optimum moisture content (OPSS 501, Section 501.08.02 Method A) and conforming to the geometry illustrated on Figure D1 in Appendix D. Based on the existing sloping topography and the proposed final grade, it is anticipated that the engineered fill pad will range between 6 m and 8 m in thickness at the west abutment and in the order of 4 m to 5 m in thickness at the east abutment.

Provided a minimum footing width of 2 m is maintained, a footing founded on a compacted Granular A pad resting on bedrock at approximate Elevation 90 m or below at the east abutment, and between Elevations 87.3 m and 89.6 m at the west abutment, may be designed for a factored geotechnical resistance of 900 kPa at Ultimate Limit States (ULS) and a geotechnical resistance at Serviceability Limit States (SLS) of 350 kPa. These values are for vertical, concentric loads only. Effects of load inclination and eccentricity should be taken into account as illustrated in the CHBDC, 2000 Clause 6.7.3 and Clause 6.7.4.

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

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

7.4 Frost Cover

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

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

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

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

8 RETAINING WALLS

It is understood that walls may be required along one or both sides of Upper Access Road to retain the new fills to be placed at the approaches. The length of these walls will depend on the configuration of the new embankments and of the new bridge.

8.1 Spread Footings on Bedrock

Given the presence of shallow bedrock below the Upper Access Road grade, it is recommended that the retaining wall footings be founded on bedrock.

On the north side of the proposed bridge, the footings may be founded on relatively shallow bedrock surface varying between approximate Elevations 90.6 m and 90.9 m. On the south side of the proposed bridge, the footings may be stepped down from approximate Elevation 90.7 m southerly to Elevation 87.7 m.

The retaining walls should be designed in accordance with the requirements of CHBDC, 2000. Detailed design recommendations on vertical and horizontal geotechnical resistances, stepped footings, eccentric and inclined loads are similar to those for the piers (see previous sections). Design recommendations on earth pressures are similar to those presented in the subsequent Section 14, Earth Pressure.

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

9 EXCAVATION AND BACKFILL

9.1 General

All excavations 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 and sand at this site are classified as Type 2 soils above the water table. All these soils below the water table and existing fills are classified as Type 3 soils.

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

At the east abutment location, excavation for footing construction will extend through the existing fill and the silty clay on the slope. It is anticipated that excavation for foundation construction should be carried out in conjunction with a temporary shoring system.

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.

A braced soldier pile and lagging wall is considered to be suitable for use as temporary shoring at the east abutment. The soldier piles will need to be socketted into bedrock through pre-augered holes. It is anticipated that the shoring system may be stiffened by struts or cross bracings, where applicable.

An interlocking sheetpiled wall is technically viable, but not considered to be cost effective as trenching into bedrock is required to socket the sheetpiles.

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

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

Below the excavation base, lateral earth pressures are applied over a width of $3B$, where B is the diameter of the socket, to take into account three dimensional effects. The ultimate passive force that can be mobilized by the embedded portion of a socket within rock is constant with depth and is given by :

$$P_p = 6 c B L$$

where

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

9.4 Rock Excavation

It is anticipated that requirements for rock excavation will be minimal at this site. The strength of intact rock is strong to very strong. Relatively shallow excavation into rock for footing construction may require appropriate excavators equipped with rock teeth and rock splitting equipment.

However, where quantities of rock have to be removed and especially where the excavation is extended into largely unfractured bedrock, it is anticipated that the Contractor may elect to use blasting methods. The design of the blast and removal procedures should be the responsibility of the Contractor. However, it is important that his procedures incorporate carefully controlled drill and blast excavation techniques in order to reduce damage to the founding surfaces and nearby structures.

Any damage to the founding surfaces on bedrock must be made good prior to constructing the foundation. Where open vertical to sub-vertical joints and fracture zones are encountered at the design founding elevation, grouting may be required to fill the voids prior to constructing the footing.

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

The Contractor's blasting and monitoring plan must not result in damage to the near-by culvert, the dam, the turbine and generator sets at the dam and associated structures. The contract documents should alert the contractor to these sensitive installations. The Contract Administrator should retain a blasting expert for review of the Contractor's procedures prior to approving them.

10 GROUNDWATER CONTROL

Groundwater in the sand layer and perched water in the fill will be trapped locally in depressions on the rock surface. This trapped water will seep into excavations and cuts through the sand.

The design of foundations bearing on bedrock will not be influenced by the groundwater, but the Contractor must make provision to control the groundwater seepage by using sump pumps to remove any accumulated water from the footing base prior to placing concrete or compacting granular fill.

11 APPROACH EMBANKMENTS

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

Immediate (elastic) settlements due to compression of cohesionless soils have been estimated based on elastic methods. Anticipated settlements due to primary consolidation of the foundation silty clay have been estimated based on the methods described in the CHBDC, 2000 Commentary Section C6.6.3.6.

In order to minimize the span lengths of the bridge, given the site topography and the likely situation where blast rock will be available from sites in this Highway 17 Twinning project (and

perhaps also from other sites in the vicinity of this project), it is recommended that rock fill be used to construct the Upper Access Road approach embankments.

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. 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. Given the space restrictions on site which requires slope inclinations not flatter than 1.5H : 1V, it is preferable to construct the approaches with rockfill. Embankments constructed using granular material and select subgrade material will have stable side slopes at inclinations not steeper than 2H : 1V.

The approach embankments for this structure will be constructed on exposed bedrock or over shallow overburden at the west abutment, and on a cohesive soil slope at the east abutment.

At the west approach, the required embankment will be up to 15 m in height after the overburden materials are removed to expose the underlying bedrock. With the exception of the Granular A core on which the west abutment footings will be founded, the remainder of the embankment may consist of rockfill. The slope of the Granular A core may be formed not steeper than 1 H : 1 V. Provided that the Granular A core is constructed as recommended in this report, and the subgrade is exposed or shallow bedrock, rockfill embankments formed with a slope inclination of 1.25H : 1V will be stable, i.e. a minimum Factor of Safety (F.S.) of 1.3 can be maintained. Some settlement will occur within the rockfill and the compacted granular fill. This settlement should be complete by the end of construction and no post construction settlement is anticipated.

At the east approach, the existing slope consisting of existing fill over predominantly stiff silty clay, at an inclination flatter than 2H : 1V, is stable. Fill placement will be required on the existing slope to raise the grade to a final grade of some 10 m above the Upper Access Road grade. The additional fill may consist of rockfill placed at 1.25H : 1V. The existing slope surfaces should be appropriately benched, as per OPSD 208.010, after stripping of topsoil/organics and prior to placement of new fill. Figures G1 and G2 present selected stability analyses results indicating a minimum F.S. of 1.3 after placement of new fill at the forward slope. Some settlement will occur within the rockfill, but a majority of which should be complete by the end of construction. Post construction settlement within the rock fill, due to particle breakage from contact stresses and particle reorientation, should be negligible. Post construction settlement due to compression of the underlying silty clay is estimated to be less than 25 mm. This magnitude of settlement should be confirmed when boreholes can be drilled at the proposed east abutment location.

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.

Where rock fill embankments are higher than 10 m, berms should be incorporated at a height of 10m below the final road grade. The berms should be 2 m wide and extend for the length through which the embankment height exceeds 10 m. Where earth fill embankments are higher than 8 m,

berms should be incorporated at a height of 8 m below the final road grade. The berms should be 2 m wide and extend for the length through which the embankment height exceeds 8 m, and the berms should maintain a 2% positive drainage grade to shed surface run-off.

In general, the approach embankments will consist of rock fill with a granular core founded on bedrock, except at the east approach where approach fills will be underlain by native silty clay in some areas. The groundwater level is below the base of the embankment. These materials have negligible to no potential for liquefaction. Consequently, the approach embankments will be stable against seismic activities at this site.

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 will be required for the contemplated design but RSS could be used for wing walls and other retaining structures. The risk associated with using RSS walls at this site is low, provided that the foundation subgrade is prepared as recommended in this report.

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

At this site, it is recommended that the levelling pad for the RSS wall be formed either directly on the exposed bedrock, on mass concrete fill, or on a mat of engineered fill. The engineered fill 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.

In addition to the requirements for the levelling pad, the RSS should be founded on approach earth fill compacted in accordance with the Contract requirements, on rock fill, or on native soil (at east abutment). All topsoil, organics, soft or loose soils should be removed from the RSS wall subgrade prior to fill placement.

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

- Factored geotechnical resistance at ULS of 5,000 kPa for walls founded directly on limestone bedrock (SLS is not applicable to foundation on rock).
- Factored geotechnical resistance at ULS of 900 kPa, and geotechnical resistance of 350 kPa at SLS, on an engineered Granular A pad resting on bedrock.

- Factored geotechnical resistance at ULS of 300 kPa, and geotechnical resistance of 200 kPa at SLS, on native silty clay.
- Coefficient of friction of between cast in-situ concrete levelling pad on Granular A is 0.7.
- Coefficient of friction of between RSS mass and Granular A is 0.55.
- Coefficient of friction of between RSS mass and native silty clay is 0.45.

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, if used at this site, are likely to be as wing walls at the abutments. It is envisaged that the RSS will be founded on an engineered fill core that is itself resting on bedrock. The outer shell of the approach embankments is to consist of rock fill, where practicable.

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

- Engineered fill – Granular A compacted to 100% SPMDD at $\pm 2\%$ optimum moisture content, with a slope of 1H : 1V (angle of internal friction, ϕ , of 35° , cohesion of 0, and unit weight, γ , of 22 kN/m^3).
- Rock fill – outer slope of 1.25H : 1V (angle of internal friction, ϕ , of 42° , cohesion of 0, and unit weight, γ , of 20 kN/m^3).
- Groundwater level at 1 m below bedrock surface.
- Vertically sided shoring to retain the existing silty clay slope at the east abutment.
- RSS block with dimensions of 5 m high by 5 m width (length of RSS reinforcement) founded on a 6 m to 8 m thick Granular A core at the west abutment.
- RSS block with dimensions of 7 m high by 7 m width (length of RSS reinforcement) founded on a 4 m thick Granular A core at the east abutment.

Results of the analyses yield Factors of Safety greater than 1.3 for the west and east approaches, respectively. Figures G3 and G4 present selected stability analyses results for the west approach. These values indicate that global stability can be maintained for the assumed RSS / approach fill configurations.

Based on the above, it may be assumed that RSS walls founded on a compacted Granular A core that is itself resting on bedrock will be stable against global failure. 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

For RSS walls founded on bedrock, settlements will be negligible. The settlement of a RSS wall founded on an engineered fill pad will depend on the thickness of the pad, the material used and the quality of construction. At this site, settlements of RSS walls founded on well compacted engineered fill are expected to be less than 25 mm and to occur essentially as the RSS is constructed.

13 BACKFILL TO ABUTMENTS

In the cases where the approach embankment consists of rockfill, the backfill to the abutment wall should consist of OPSS Granular "B" Type II material.

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 not greater than 300 mm and including adequate spalls to fill voids in the rockfill.

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 materials 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 PRESSURE

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

If the support system allows lateral 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 lateral yielding (restrained system), at-rest horizontal earth pressures should be used. The amount of wall movement required for 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 are generally given by the expression:

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

Where P_h = horizontal pressure on the wall (kPa)

K = earth pressure coefficient (see table below)

γ = unit weight of retained soil (typically 21 kN/m³)

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 (modified) $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ, \gamma = 19.0 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*	0.2	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

* For wing walls.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall. 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 should be used for design, based on Table 3.1.7 of the CHBDC, Arnprior/Renfrew area:

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

The Soil Profile Type at this site has been classified as Type I. Thus, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.0 should be used in seismic design.

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

Clause C4.6.4 of the CHBDC suggests that the value of k_h used in calculating the earth pressure coefficients for yielding structures is equivalent to $0.5 \times$ Zonal Acceleration Ratio, A, (including the effects of site amplification), or 0.1g at this site, provided that allowance is made for an outward displacement of the abutment of up to $250A$, or 50 mm. The vertical acceleration factor, k_v , has been taken as 0.6 times k_h . This design assumption is based on conventional practice in seismic design. For non-yielding walls, the recommended k_h design value according to CHBDC is equivalent to $1.5 \times$ 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 the pier foundations are to be founded on bedrock, there is no potential for liquefaction under the foundations. There is negligible potential for liquefaction of the silty clay on the east slope.

The approach rock fill embankments, including the granular core, will be founded on bedrock above the groundwater level and are not considered to be in danger of undergoing liquefaction. Some toe failure may occur in the approach embankments but this is expected to be minor in nature and readily repairable.

15.3 Retaining Wall Dynamic Earth Pressures

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

In calculating the values of (K_{AE}) and (K_{PE}), the following geotechnical parameters were used:

$$\begin{aligned}\phi &= \text{angle of internal friction of backfill} \\ \delta &= \text{angle of friction between the wall and the backfill}\end{aligned}$$

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

Wall Condition	Earth Pressure Coefficient (K) for Earthquake Loading						
	Height of Application From Base as Percentage of Wall Height	Granular A or Granular B Type II $\phi = 35^\circ, \delta = 17.5^\circ$		OPSS Granular B Type I $\phi = 32^\circ, \delta = 16^\circ$		Rock Fill $\phi = 42^\circ, \delta = 21^\circ$	
		Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})	40%	0.33	0.70	0.37	0.90*	0.26	0.40
Passive (K_{PE})	33%	3.5	-	3.0	-	4.8	-
At Rest (K_{OE})**	45%	0.67	-	0.72	-	0.58	-

* Slope may undergo movement for short durations during seismic activities

** After Woods

16 CONSTRUCTION CONCERNS

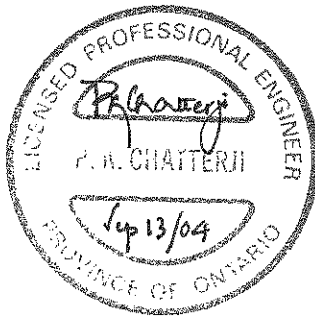
During construction, the Contract Administrator should employ experienced geotechnical staff to observe construction activities related to foundation construction.

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

- disturbance of the bedrock under the foundations due to blasting or other excavation procedures,
- impact of blasting on the dam, turbines and generator sets,
- variations in the elevation of the bedrock surface, necessitating the use of mass concrete fill to build up to the design founding elevation,
- maintaining stability of the existing east slope during construction of the east abutment.



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TABLE 1
Upper Access Road Overpass
Point Load Test Results

Depth			Is50	UCS (MPa)					
feet	Inches	m			Average	Minimum	Maximum	MPa	
UAR-1									
3	5	1.04	4.50	107.97	}	Total Rock Core			
4	6	1.37	3.40	81.64					
5	4	1.63	0.44	10.53					
6	0	1.83	10.75	258.07					
7	6	2.29	2.41	57.94					
8	6	2.59	4.17	100.07					
9	5	2.87	4.06	97.44					
10	3	3.12	4.28	102.70					
11	5	3.48	5.27	126.40					
12	6	3.81	3.95	94.80					
Depth			Is50	UCS (MPa)					
feet	Inches	m			Average	Minimum	Maximum	MPa	
UAR-2									
4	3	1.30	1.32	31.60	}	Total Rock Core			
5	6	1.68	1.76	42.13					
6	6	1.98	3.29	79.00					
7	6	2.29	4.17	100.07					
8	6	2.59	4.50	107.97					
9	6	2.90	2.96	71.10					
10	6	3.20	5.05	121.14					
11	5	3.48	3.51	84.27					
12	4	3.76	3.84	92.17					
13	0	3.96	3.51	84.27					
Depth			Is50	UCS (MPa)					
feet	Inches	m				Average	Minimum	Maximum	MPa
UAR-3									
3	8	1.12	2.08	50.03	}	Total Rock Core			
4	5	1.35	3.95	94.80					
5	4	1.63	4.83	115.87					
6	4	1.93	4.94	118.50					
7	4	2.24	5.27	126.40					
8	4	2.54	4.17	100.07					
9	4	2.84	4.94	118.50					
10	4	3.15	4.28	102.70					
11	4	3.45	4.83	115.87					
12	4	3.76	4.61	110.60					

TABLE 1 (cont'd.)
Upper Access Road
Point Load Test Results

Depth			Is50	UCS (MPa)
feet	Inches	m		
UAR-4				
9	8	2.95	0.44	10.53
10	8	3.25	2.85	68.47
11	5	3.48	2.85	68.47
12	0	3.66	4.39	105.34
13	4	4.06	3.62	86.90
14	6	4.42	3.73	89.54
15	6	4.72	2.52	60.57
16	4	4.98	4.28	102.70
17	0	5.18	4.28	102.70
17	5	5.31	3.29	79.00

Total Rock Core
 Average Minimum Maximum
 77 11 105 MPa

Run # Average
 1 10.53
 2 79.88
 3 94.80

Depth			Is50	UCS (MPa)
feet	Inches	m		
UAR-6				
1	9	0.53	5.05	121.14
2	9	0.84	5.05	121.14
3	9	1.14	4.39	105.34
4	9	1.45	3.73	89.54
5	6	1.68	4.94	118.50
6	0	1.83	4.28	102.70
7	0	2.13	5.71	136.94
8	0	2.44	3.84	92.17
9	0	2.74	5.49	131.67
10	0	3.05	2.08	50.03

Average Minimum Maximum
 107 50 137 MPa

Run # Average
 1 109.73
 2 102.70

Depth			Is50	UCS (MPa)
feet	Inches	m		
UAR-9				
3	9	1.14	4.50	107.97
4	8	1.42	4.83	115.87
5	9	1.75	4.06	97.44
6	10	2.08	3.29	79.00
7	10	2.39	4.50	107.97
8	10	2.69	4.06	97.44
9	10	3.00	2.74	65.84
10	11	3.33	4.28	102.70
12	0	3.66	3.73	89.54
13	0	3.96	4.50	107.97

Total Rock Core
 Average Minimum Maximum
 97 66 116 MPa

Run # Average
 1 107.09
 2 90.59
 3 98.75

TABLE 1 (cont'd.)
Upper Access Road
Point Load Test Results

Depth			Is50	UCS (MPa)
feet	Inches	m		
UAR-17				
3	0	0.91	2.19	52.67
4	0	1.22	5.16	123.77
5	0	1.52	5.71	136.94
5	9	1.75	5.82	139.57
7	0	2.13	4.28	102.70
8	0	2.44	4.17	100.07
9	0	2.74	3.29	79.00
10	0	3.05	4.50	107.97
11	8	3.56	4.50	107.97
12	6	3.81	4.17	100.07

Total Rock Core			
Average	Minimum	Maximum	
105	53	140	MPa
Run # Average			
1	113.24		
2	97.44		
3	104.02		

Depth			Is50	UCS (MPa)
feet	Inches	m		
UAR-18				
3	4	1.02	1.65	39.50
4	0	1.22	3.73	89.54
5	0	1.52	3.07	73.74
5	7	1.70	4.61	110.60
7	1	2.16	3.62	86.90
8	1	2.46	5.05	121.14
8	11	2.72	3.51	84.27
10	0	3.05	3.95	94.80
11	2	3.40	5.05	121.14
12	0	3.66	3.73	89.54

Total Rock Core			
Average	Minimum	Maximum	
91	40	121	MPa
Run # Average			
1	78.34		
2	96.78		
3	105.34		

Depth			Is50	UCS (MPa)
feet	Inches	m		
UAR-19				
14	1	4.29	0.88	21.07
15	1	4.60	3.51	84.27
16	1	4.90	3.84	92.17
17	0	5.18	3.62	86.90
18	0	5.49	3.73	89.54
19	0	5.79	3.51	84.27
20	0	6.10	3.07	73.74
21	0	6.40	2.96	71.10
22	0	6.71	3.18	76.37
23	0	7.01	3.51	84.27

Total Rock Core			
Average	Minimum	Maximum	
76	21	92	MPa
Run # Average			
1	65.84		
2	81.11		
3	80.32		

**TABLE 1 (cont'd.)
Upper Access Road
Point Load Test Results**

Depth			Is50	UCS (MPa)								
feet	Inches	m				Average	Minimum	Maximum				
UAR-20												
8	8	2.64	3.51	84.27	}	95	74	124	MPa			
9	8	2.95	3.95	94.80								
10	8	3.25	3.40	81.64								
11	10	3.61	3.84	92.17								
12	8	3.86	3.95	94.80								
13	8	4.17	5.16	123.77								
14	5	4.39	4.94	118.50								
15	6	4.72	3.07	73.74								
16	8	5.08	4.39	105.34								
17	4	5.28	3.29	79.00								
						Total Rock Core						
						Run #	Average					
						1	86.90					
						2	100.60					
						3	92.17					

Appendix A

Record of Borehole Sheets

SYMBOLS AND TERMS USED ON TEST HOLE LOGS

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ "N" VALUE
Very Soft	Less than 10	Less than 2
Soft	10 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30






NOTE: Hierarchy of Soil Strength Prediction

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


4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR TEST HOLE LOGS

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

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

 Water Level


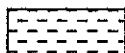


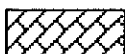
C_{vane}	Shear Strength Determination by Field Insitu Vane
C_{pen}	Shear Strength Determination by Pocket Penetrometer
C_{lab}	Shear Strength Determination using a Laboratory Vane Apparatus
C_U	Undrained Shear Strength determined by Unconfined Compression Test

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

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

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

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

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

RECORD OF BOREHOLE No UAR-1

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 152.8 E 316 863.0 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 05.08.03 - 05.08.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
91.7														
90.9	TOPSOIL (150mm)		1	SS	60/									
0.2	Black				203									
91.0	SAND and GRAVEL, occasional cobbles													
0.6	Very Dense Brown (FILL)													
	END OF BOREHOLE AT 0.64m. AUGER REFUSAL AT 0.64m ON ASSUMED BEDROCK. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION.													

ONTMT4 7450UAR.GPJ 06/02/04

RECORD OF BOREHOLE No UAR-1A

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 152.8 E 316 863.0 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 12.11.03 - 12.11.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40					
91.7														
0.0	Augering to 0.7m Refer to borehole UAR-1 for inferred soil stratigraphy.													
90.9					FI		91							RUN 1# TCR=100%, SCR=77%, RQD=77%, UCS=114.6MPa
0.7	CRYSTALLINE LIMESTONE (BEDROCK) Fresh, slightly weathered at joints, very thinly to thinly bedded, grey with black and white subvertical banding, strong to very strong. Subvertical and vertical joints from 1.68m to 1.78m.		1	RUN	>5 3 1		90							RUN 2# TCR=100%, SCR=100%, RQD=96%, UCS=89.5MPa
			2	RUN	>5 1 0 0		89							
			3	RUN	0 0		88							RUN 3# TCR=100%, SCR=100%, RQD=100%, UCS=110.6MPa
87.8														
3.9	END OF BOREHOLE AT 3.91m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE ELEVATION (m) 04/01/04 88.7													

RECORD OF BOREHOLE No UAR-2

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 142.8 E 316 867.3 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 05.08.03 - 05.08.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L		
91.8														
91.0	TOPSOIL (150mm)													
0.2	Black		1	SS	28									
	Gravelly SAND, some silt, trace cobbles, some limestone pieces													
	Dense													
90.9	Brown		2	SS	50/		91							29 53 18
0.9	(FILL)				.15									(SI+CL)
	END OF BOREHOLE AT 0.94m. AUGER REFUSAL AT 0.94m ON ASSUMED BEDROCK. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION.													

ONTMT4 7450UAR GPJ 06/02/04

RECORD OF BOREHOLE No UAR-2A

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 142.8 E 316 867.3 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 12.11.03 - 12.11.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
91.8														
0.0	Augering to 1.0m Refer to Borehole UAR-2 for inferred soil stratigraphy.													
90.7					FI									
1.2	CRYSTALLINE LIMESTONE (BEDROCK) Fresh, slightly weathered at joints, very thinly to thinly bedded, grey with black and white subvertical banding, strong to very strong. Subvertical joints from 1.32m to 1.68m.		1	RUN	>5 3									RUN 1# TCR=100%, SCR=86%, RQD=56%, UCS=36.9MPa RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=95.9MPa RUN 3# TCR=94%, SCR=94%, RQD=84%, UCS=86.9MPa
			2	RUN	0 0 0 1 0									
			3	RUN	1 1									
87.7														
4.2	END OF BOREHOLE AT 4.17m.													

ONTMT4 7450UAR.GPJ 06/02/04

RECORD OF BOREHOLE No UAR-3

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 153.8 E 316 849.2 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 05.08.03 - 05.08.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
91.7	TOPSOIL (125mm)													
90.6	Black		1	SS	29									14 53 25 7
0.1	Silty SAND, some gravel, trace clay, occasional organics, black staining													
91.0	Compact		2	SS	50/		91							
0.7	Brown													
90.9	(FILL)				.102									
0.9	Silty CLAY, trace sand, occasional topsoil lenses													
	Hard													
	Brown													
	END OF BOREHOLE AT 0.86m. AUGER REFUSAL AT 0.86m ON ASSUMED BEDROCK. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION.													

ONTMT4 7450UAR.GPJ 06/02/04

RECORD OF BOREHOLE No UAR-3A

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 153.8 E 316 849.2 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 13.11.03 - 13.11.03 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 ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
91.7								20 40 60 80 100		W _p W W _L			GR SA SI CL	
0.0	Augering to 1.0m Refer to Borehole UAR-3 for inferred soil stratigraphy.							20 40 60 80 100						
90.7					FI		91							
1.0	CRYSTALLINE LIMESTONE (BEDROCK) Fresh, slightly weathered at joints, very thinly to thinly bedded, grey with black and white subvertical banding, strong to very strong.		1	RUN	2 0 1		90						RUN 1# TCR=100%, SCR=97%, RQD=84%, UCS=86.9MPa	
			2	RUN	2 1 2 1 0 0		89						RUN 2# TCR=100%, SCR=97%, RQD=95%, UCS=113.2MPa	
			3	RUN	0 0		88						RUN 3# TCR=100%, SCR=100%, RQD=100%, UCS=113.2MPa	
87.7														
4.0	END OF BOREHOLE AT 4.01m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.22m slotted screen. WATER LEVEL READINGS DATE ELEVATION (m) 04/01/04 88.1													

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RECORD OF BOREHOLE No UAR-4

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 144.0 E 316 852.9 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 05.08.03 - 05.08.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
91.8	TOPSOIL (150mm)		1	SS	60/									
90.6	Black				102									
0.2	SAND and GRAVEL, some silt													
91.2	Very Dense													
0.6	Brown						91							
	(FILL)		2	SS	24									0 18 61 20
	Silty CLAY, some sand and rootlets													
	Very stiff													
90.4	Brown													
	(CL)													
1.4	END OF BOREHOLE AT 1.37m. AUGER REFUSAL AT 1.37m. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION.													

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No UAR-4A

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 144.0 E 316 852.9 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 06.08.03 - 06.08.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
91.8 0.0	Augering to 1.37m Refer to Borehole UAR-4 for inferred soil stratigraphy.													
90.6 1.2	cobbles and/ or bouldes from 1.2m to 1.5m Inferred soil from 1.5m to 2.3m		1	RUN										RUN 1# TCR=100%, SCR=100%, RQD=69%
89.4 2.3	CRYSTALLINE LIMESTONE (BEDROCK) Fresh, slightly weathered at joints, very thinly to thinly bedded, grey with black and white subvertical banding, strong to very strong. Vertical joint from 2.41m to 2.59m. Multiple fractures from 2.69m to 2.84m.		2	RUN	3 >5 2									RUN 2# TCR=94%, SCR=84%, RQD=44%, UCS=10.5MPa
			3	RUN	1 1 1 2									RUN 3# TCR=100%, SCR=100%, RQD=89%, UCS=79.9MPa
86.4 5.4	END OF BOREHOLE AT 5.38m.		4	RUN	1 3									RUN 4# TCR=100%, SCR=100%, RQD=100%, UCS=94.8MPa

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+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No UAR-5

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 151.3 E 316 820.8 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 13.11.03 - 13.11.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
87.0	TOPSOIL (50mm)						87							
86.9	Clayey SILT		1	SS	16									
0.1	Firm													
86.7	Brown													
0.3	Moist													
86.3	SAND, trace gravel													
0.8	Compact													
	Brown													
	Moist													
	END OF BOREHOLE AT 3.91m. AUGER REFUSAL AT 0.74m ON PROBABLE BEDROCK OR BOULDERS.													

ONTMT4 7450UAR GPJ 06/02/04

RECORD OF BOREHOLE No UAR-6

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 156.4 E 316 836.7 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 13.11.03 - 13.11.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	
89.3													
0.0	Augering to 0.4m				FI		89						RUN 1# TCR=92%, SCR=92%, RQD=82%, UCS=109.7MPa
88.9	Topsoil mixed with clayey silt												
0.4	CRYSTALLINE LIMESTONE (BEDROCK) Fresh, slightly weathered at joints, very thinly to thinly bedded, gray with black and white subvertical banding, strong to very strong.		1	RUN	2								
					0								
					0								
					0								
					0								
			2	RUN	0		87						RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=102.7MPa
					0								
					0								
					0								
					0								
85.9					0		86						
3.4	END OF BOREHOLE AT 3.38m.												

RECORD OF BOREHOLE No UAR-7

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 156.4 E 316 838.1 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE COMPILED BY SS
 DATUM Geodetic DATE 12.11.03 - 12.11.03 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 ³	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	W _p W W _L	20 40 60			
89.6 0.0	ROCK OUTCROP ASSUMED BEDROCK						89							

METRIC

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 ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
89.2 0.0	ROCK OUTCROP ASSUMED BEDROCK						89							

RECORD OF BOREHOLE No UAR-9

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 143.3 E 316 842.4 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 13.11.03 - 13.11.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
88.4								20	40	60	80	100		
88.2	TOPSOIL (150mm)							20	40	60	80	100		
0.2	Black							20	40	60	80	100		
	Clayey SILT, trace gravel, occasional cobbles							20	40	60	80	100		
	Hard							20	40	60	80	100		
87.3	Brown		1	SS	50/			20	40	60	80	100		
	Moist				150			20	40	60	80	100		
1.0	auger refusal at 0.91m inferred cobbles				0			20	40	60	80	100		
	CRYSTALLINE LIMESTONE (BEDROCK)		1	RUN	0			20	40	60	80	100		
	Fresh, slightly weathered at joints, very thinly to thinly bedded, grey with black and white subvertical banding, strong to very strong				1			20	40	60	80	100		
					0			20	40	60	80	100		
			2	RUN	0			20	40	60	80	100		
					0			20	40	60	80	100		
					0			20	40	60	80	100		
					1			20	40	60	80	100		
					0			20	40	60	80	100		
	Subvertical joints from 3.78m to 3.91m.		3	RUN	0			20	40	60	80	100		
84.2								20	40	60	80	100		
4.2	END OF BOREHOLE AT 4.22m.				FI			20	40	60	80	100		

RUN 1#
 TCR=100%,
 SCR=100%,
 RQD=100%,
 UCS=107.1MPa

RUN 2#
 TCR=98%,
 SCR=98%,
 RQD=98%,
 UCS=90.6MPa

RUN 3#
 TCR=100%,
 SCR=100%,
 RQD=100%,
 UCS=98.75MPa

RECORD OF BOREHOLE No UAR-10

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 143.2 E 316 843.8 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hand Probe COMPILED BY SS
 DATUM Geodetic DATE 12.11.03 - 12.11.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
89.1								20 40 60 80 100						
88.9	TOPSOIL (150mm)					89								
0.2	Clayey SILT													
87.9						88								
1.1	END OF BOREHOLE AT 1.12m. BEDROCK INFERRED BY PROBING WITH STEEL ROD.													

ONTMT4 7450UAR.GPJ 06/02/04

+³, ×³: Numbers refer to
Sensitivity

20
15
10
5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No UAR-16

1 OF 2

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 144.3 E 316 896.0 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 13.11.03 - 13.11.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
103.0													
102.9	TOPSOIL (150mm)												
0.2	Silty SAND Very Loose to Loose Brown Moist (FILL)		1	SS	3								
			2	SS	10		102						
			3	SS	6		101						
100.9													
2.1	Silty CLAY, trace sand Stiff Brown Moist to Wet (CL-CI) becoming grey		4	SS	14		100						
			5	SS	8		99						
			6	SS	8		98						
			7	SS	8		97						
			8	SS	14		96						
			9	SS	8		95						
93.2	occasional silt pockets and sand seams						94						
9.8	END OF BOREHOLE AT 9.75m												

Continued Next Page

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

RECORD OF BOREHOLE No UAR-16

2 OF 2

METRIC

G.W.P. 647-92-00 LOCATION N 5.031 144.3 E 316 896.0 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 13.11.03 - 13.11.03 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 ³	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 _p	W	W _L			
	BOREHOLE DRY ON COMPLETION. BOREHOLE OPEN TO 7.77m.															

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

2 OF 2

METRIC

[illegible]

RECORD OF BOREHOLE No UAR-17

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 162.5 E 316 844.8 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 13.11.03 - 13.11.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	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	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
91.7	TOPSOIL (50mm)													
91.6														
0.2	Silty CLAY with topsoil inclusions, trace gravel Very Stiff Dark Brown Moist		1	SS	16		91							RUN 1# TCR=100%, SCR=100%, RQD=97%, UCS=113.2MPa
90.8					FI									
0.8	auger refusal at 0.64m CRYSTALLINE LIMESTONE (BEDROCK) Fresh, slightly weathered at joints, very thinly to thinly bedded, grey with black and white subvertical banding, strong to very strong.		1	RUN	1		90							
					0									RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=97.4MPa
			2	RUN	0		89							
					0									
					0									RUN 3# TCR=100%, SCR=92%, RQD=83%, UCS=104MPa
					0									
			3	RUN	>5		88							
87.7					1									
3.9	END OF BOREHOLE AT 3.94m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.65m slotted screen. WATER LEVEL READINGS: DATE ELEVATION (m) 04/01/04 91.7 (frozen) Note: Suspected frozen drill water in pipe													

RECORD OF BOREHOLE No UAR-18

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 161.7 E 316 859.1 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 12.11.03 - 12.11.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
91.4														
0.0	ROCK FILL (250mm)													
91.1														
0.3	SAND trace to some gravel, topsoil stained						91							
	Very Dense													
90.6	Brown to Dark Brown		1	SS	50/076									
0.8	Moist (FILL)				2									
	AUGER REFUSAL AT 0.84m.		1	RUN	0		90							
	CRYSTALLINE LIMESTONE (BEDROCK)				1									
	Fresh, slightly weathered at joints, very thinly to thinly bedded, grey with black and white subvertical banding, strong to very strong.				0									
					0									
			2	RUN	0		89							
					0									
					0									
					1									
					0									
			3	RUN	0		88							
					0									
87.3					FI									
4.1	END OF BOREHOLE AT 4.11m.													

UNIMI14 / 74500AR.GPJ 06/02/04

ONTMT4 7450UAR.GPJ 06/02/04

RECORD OF BOREHOLE No UAR-19

1 OF 1

METRIC

G.W.P.: 647-92-00 LOCATION N 5 031 132.7 E 316 860.1 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 12.11.03 - 12.11.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
91.9	ROCK FILL													
0.0														
91.6														
0.3	Silty CLAY, with topsoil inclusions Stiff Brown Moist													
90.9			1	SS	17		91							
1.1	Sandy SILT Compact Brown Moist		2	SS	17		90							
89.7														
2.2	Silty CLAY, occasional sand seams Stiff Brown Moist (Cl)		3	SS	15		89							
			4	SS	8		88							
87.7	auger refusal at 4.11m				FI									
4.2	CRYSTALLINE LIMESTONE BEDROCK Fresh, slightly weathered at joints, very thinly to thinly bedded, grey with black and white subvertical banding, strong to very strong.		1	RUN	0		87							RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=65.8MPa
			2	RUN	0		86							RUN 2# TCR=97%, SCR=97%, RQD=97%, UCS=81.1MPa
			3	RUN	0		85							RUN 3# TCR=93%, SCR=93%, RQD=93%, UCS=80.3MPa
84.7					0									
7.3	END OF BOREHOLE AT 7.26m.													

ONTMT4 7450UAR.GPJ 06/02/04

RECORD OF BOREHOLE No UAR-20

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 136.7 E 316 869.3 (Upper Access Road) ORIGINATED BY SL
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 12.11.03 - 12.11.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa								
						WATER CONTENT (%)								
91.8														
0.0	ROCK FILL													
91.5														
0.3	Silty CLAY, some sand Stiff to Firm Brown Moist (Cl)		1	SS	12		91							
			2	SS	5		90							0 21 46 33
	auger refusal at 2.29m				FI									
89.5														
2.3	CRYSTALLINE LIMESTONE (BEDROCK) Fresh, slightly weathered at joints, very thinly to thinly bedded, grey with black and white subvertical banding, strong to very strong. Subvertical and vertical joints from 2.29m to 2.46m and 4.5m to 4.65m.		1	RUN	3 0 0 0		89							RUN 1# TCR=98%, SCR=98%, RQD=90%, UCS=86.9MPa
			2	RUN	0 0 1 1 0		88							RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=100.6MPa
			3	RUN	0		87							RUN 3# TCR=100%, SCR=100%, RQD=100%, UCS=92.2MPa
86.4														
5.4	END OF BOREHOLE AT 5.36m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE ELEVATION (m)													

ONTMT4 7450UAR.GPJ 06/02/04

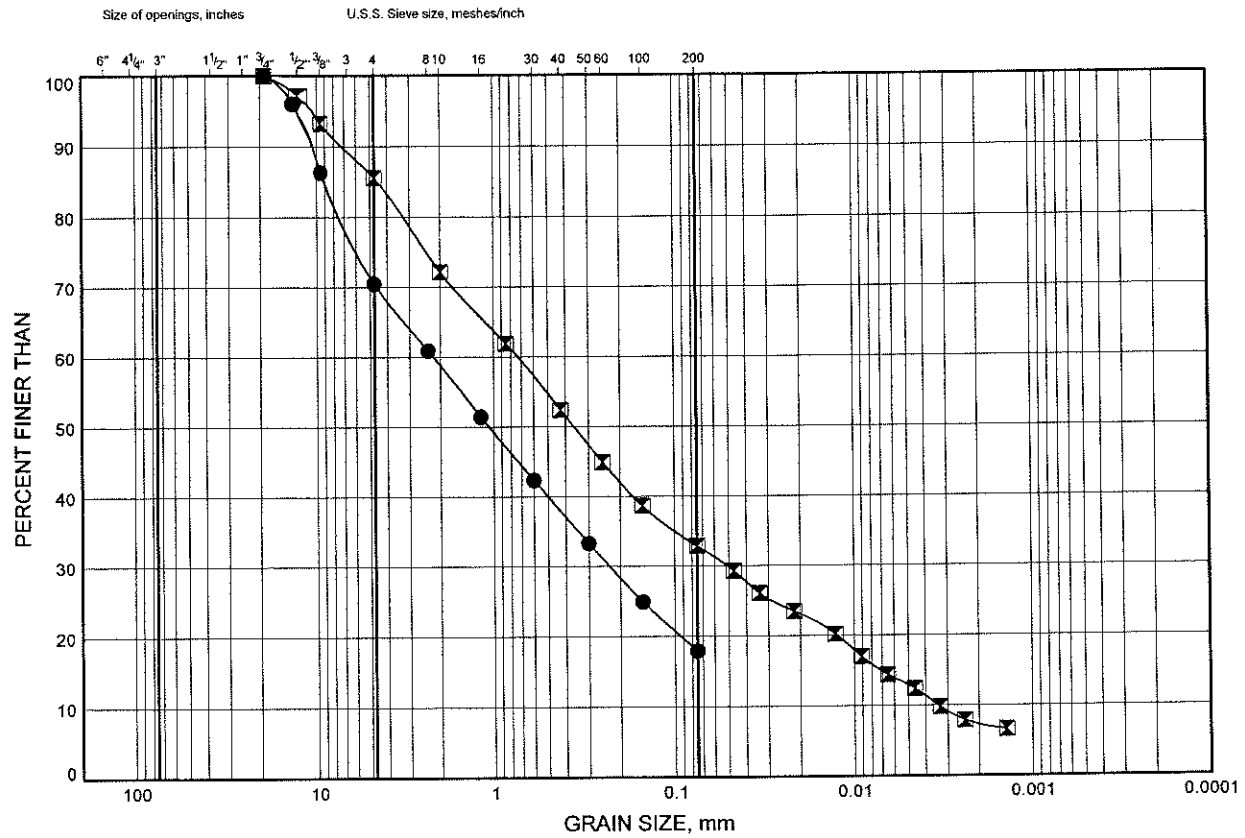
Appendix B

Laboratory Test Results

HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B1

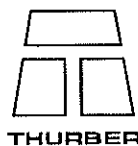
SAND FILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	UAR-2	0.84	91.00
⊠	UAR-3	0.30	91.43

Date January 2004
Project 647-92-00

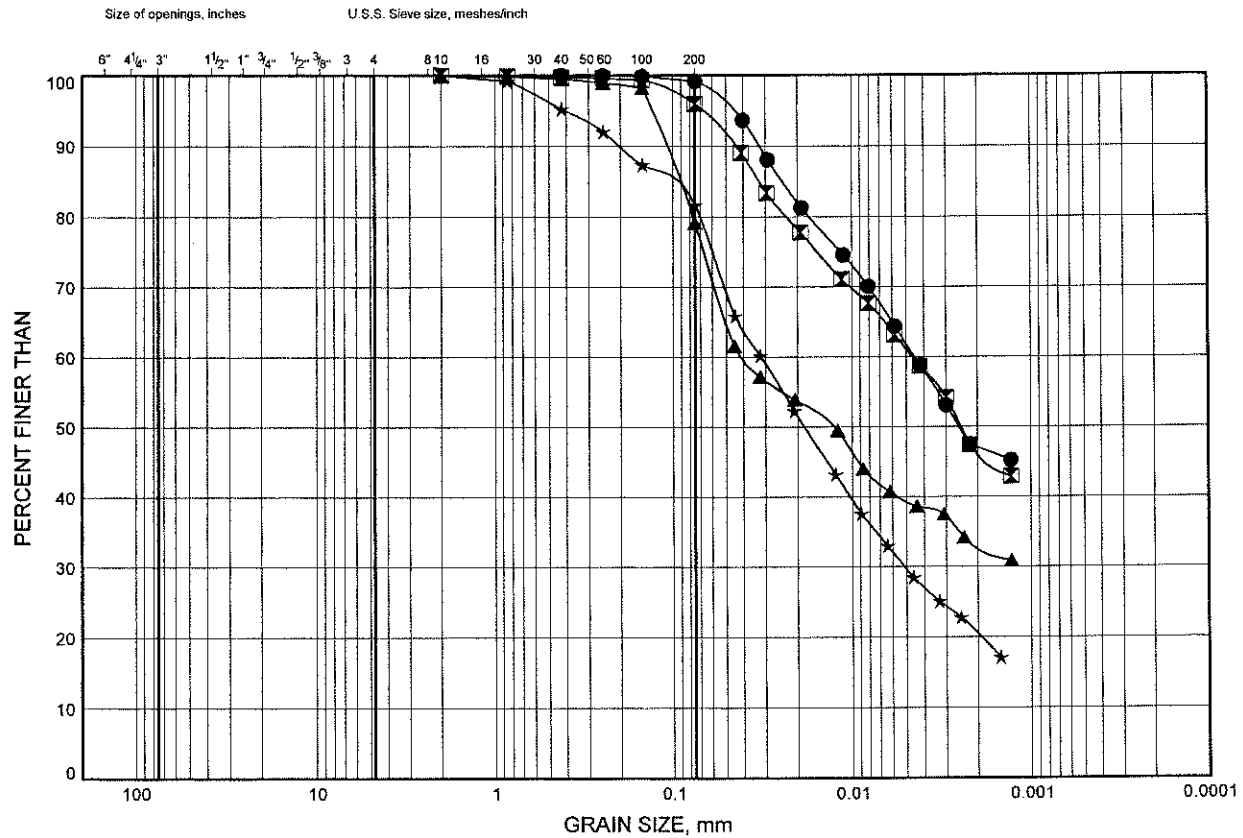


Prep'd SS
Chkd. SKP

HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B2

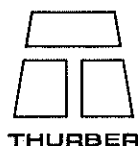
SILTY CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	UAR-16	6.40	96.55
⊠	UAR-19	2.59	89.34
▲	UAR-20	1.83	89.97
★	UAR-4	0.99	90.78

Date January 2004
Project 647-92-00

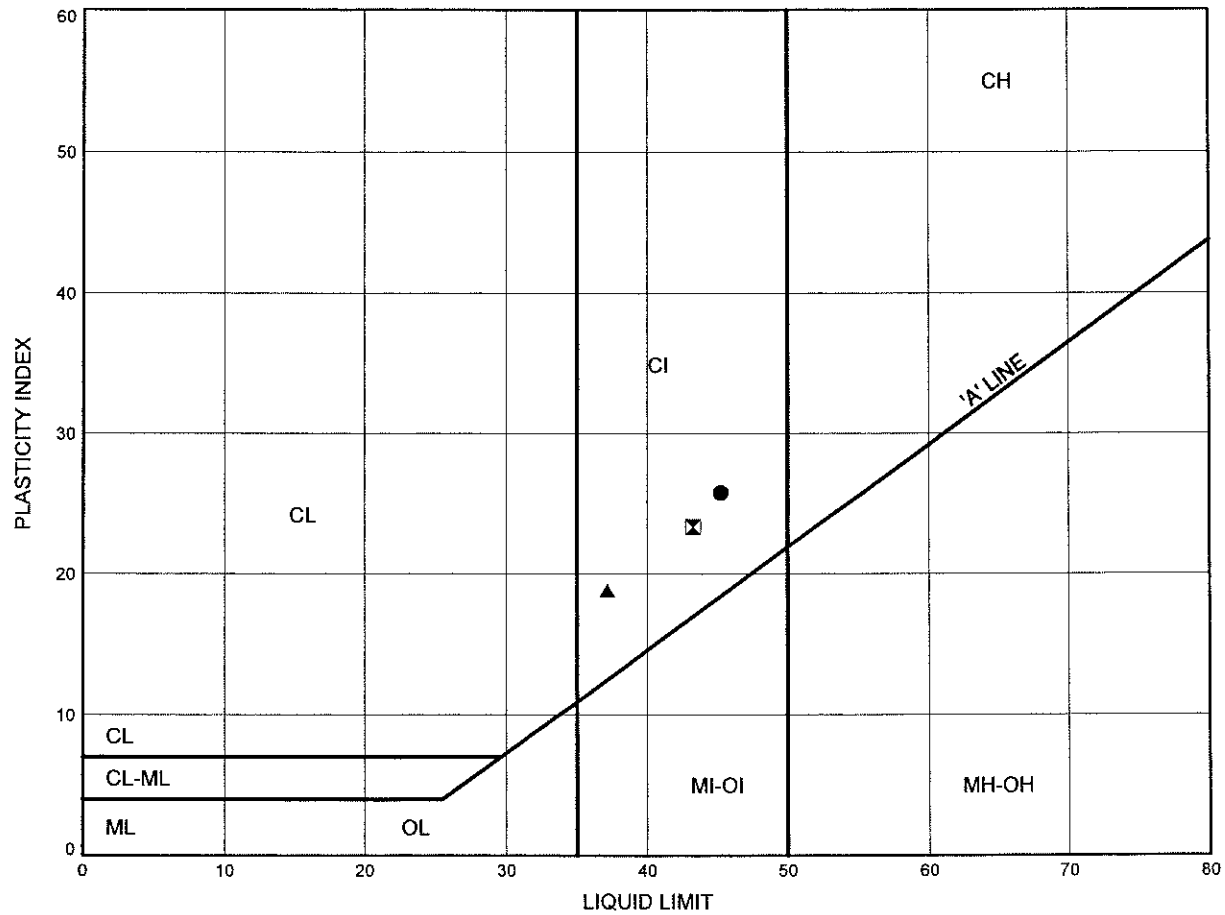


Prep'd SS
Chkd. SKP

HWY 17 Twinning, Arnprior to Renfrew ATTERBERG LIMITS TEST RESULTS

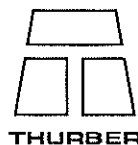
FIGURE B3

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	UAR-16	6.40	96.55
⊠	UAR-19	2.59	89.34
▲	UAR-4	0.99	90.78

Date January 2004
Project 647-92-00



Prep'd SS
Chkd. SKP

Appendix C

Foundation Comparison

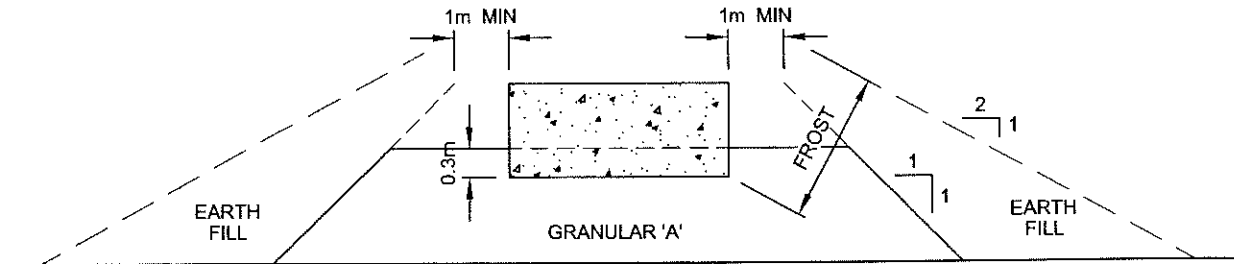
Upper Access Road Overpass
Highway 17 Twinning, Arnprior to Renfrew

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

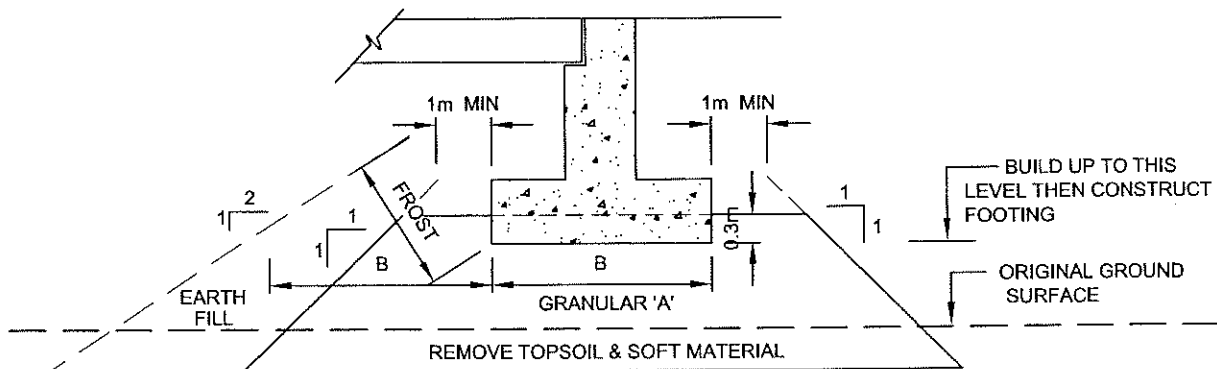
Foundation Element	Driven Piles	Footings on Bedrock	Footings on Engineered Fill	Caisson
West Abutment	<p>Advantages:</p> <ul style="list-style-type: none"> i. Structural capacity of steel piles can be utilized if seated into bedrock. ii. Rock excavation or mass concrete are not required. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Engineered fill pad is required as a platform to accommodate pile installation equipment prior to driving piles. ii. Difficulties when seating piles on potentially sloping bedrock. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Shallow bedrock outcrop at and/or bedrock outcrop at existing ground surface. ii. High values of geotechnical resistance are available on the bedrock. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Due to the proposed height of approach fills (12 m), the required dimensions of the abutment wall would become impractical and uneconomical. ii. High cost of rock excavation, if any is required. iii. Mass concrete fill required to create a level founding surface. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Approach fill geometry permits perched abutment design where footings may be placed at a higher elevation, resulting in shorter abutment stem and more practical footing dimensions <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Lower geotechnical resistance than bedrock ii. Footings has to be located further back resulting in a longer superstructure. 	<p>Advantages:</p> <ul style="list-style-type: none"> High bearing capacity on bedrock. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Engineered fill pad is required as a platform to accommodate caisson installation equipment. ii. Some rock coring may be required to seat into bedrock.
West and East Piers	<p>Advantages:</p> <ul style="list-style-type: none"> None identified. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Shallow bedrock surface and/or bedrock outcrop making the use of driven piles unnecessary and 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Shallow bedrock surface and/or bedrock outcrop at existing ground surface. ii. High values of geotechnical resistance are available on the bedrock. iii. Footings may be placed at 	<p>Advantages:</p> <ul style="list-style-type: none"> None identified. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Shallow bedrock surface and/or bedrock outcrop making the use of engineered fill unnecessary. 	<p>Advantages:</p> <ul style="list-style-type: none"> None identified. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Shallow bedrock surface and/or bedrock outcrop rendering the use of caisson unnecessary and impractical

Upper Access Road Overpass
Highway 17 Twinning, Arnprior to Renfrew

	impractical	the edge of the existing Upper Access Road. Disadvantages: i High cost of rock excavation, if any is required	ii. Footing has to be located further back from the edge of road to accommodate forward slope of fill. iii. Lower geotechnical resistance than bedrock.	
East Abutment (bedrock elevation to be confirmed)	<p>Advantages:</p> <ul style="list-style-type: none"> i Structural capacity of steel piles can be utilized if seated into bedrock. ii Rock excavation or mass concrete are not required. <p>Disadvantages:</p> <ul style="list-style-type: none"> i Benching into existing slope and/or engineered fill pad is required as a platform to accommodate pile installation equipment prior to driving piles. ii Difficulties when seating piles on potentially sloping bedrock. iii Piles may require to be socketted deep into bedrock if bedrock is found to be too shallow, rendering this option impractical. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High values of geotechnical resistance are available on the bedrock. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Comparatively longer abutment stem. ii. Mass concrete fill may be required to create a level founding surface. 	<p>Advantages:</p> <ul style="list-style-type: none"> i Approach fill geometry permits perched abutment design where footings may be placed at a higher elevation, resulting in shorter abutment stem and more practical footing dimensions. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Lower geotechnical resistance than bedrock ii. Footing has to be located further back resulting in a longer superstructure. 	<p>Advantages:</p> <p>High bearing capacity on bedrock.</p> <p>Disadvantages:</p> <ul style="list-style-type: none"> i Benching into existing slope and/or engineered fill pad is required as a platform to accommodate caisson installation equipment. ii Rock coring may be required to seat into bedrock. iii Rock sockets may be required if bedrock is found to be too shallow, rendering this option impractical.



CROSS-SECTION



LONGITUDINAL SECTION

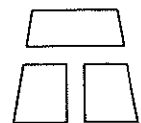
NOT TO SCALE

NOTES:

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

ENGINEER	SP
DRAWN	SS
DATE	JAN , 2004
APPROVED	PKC
SCALE	NTS

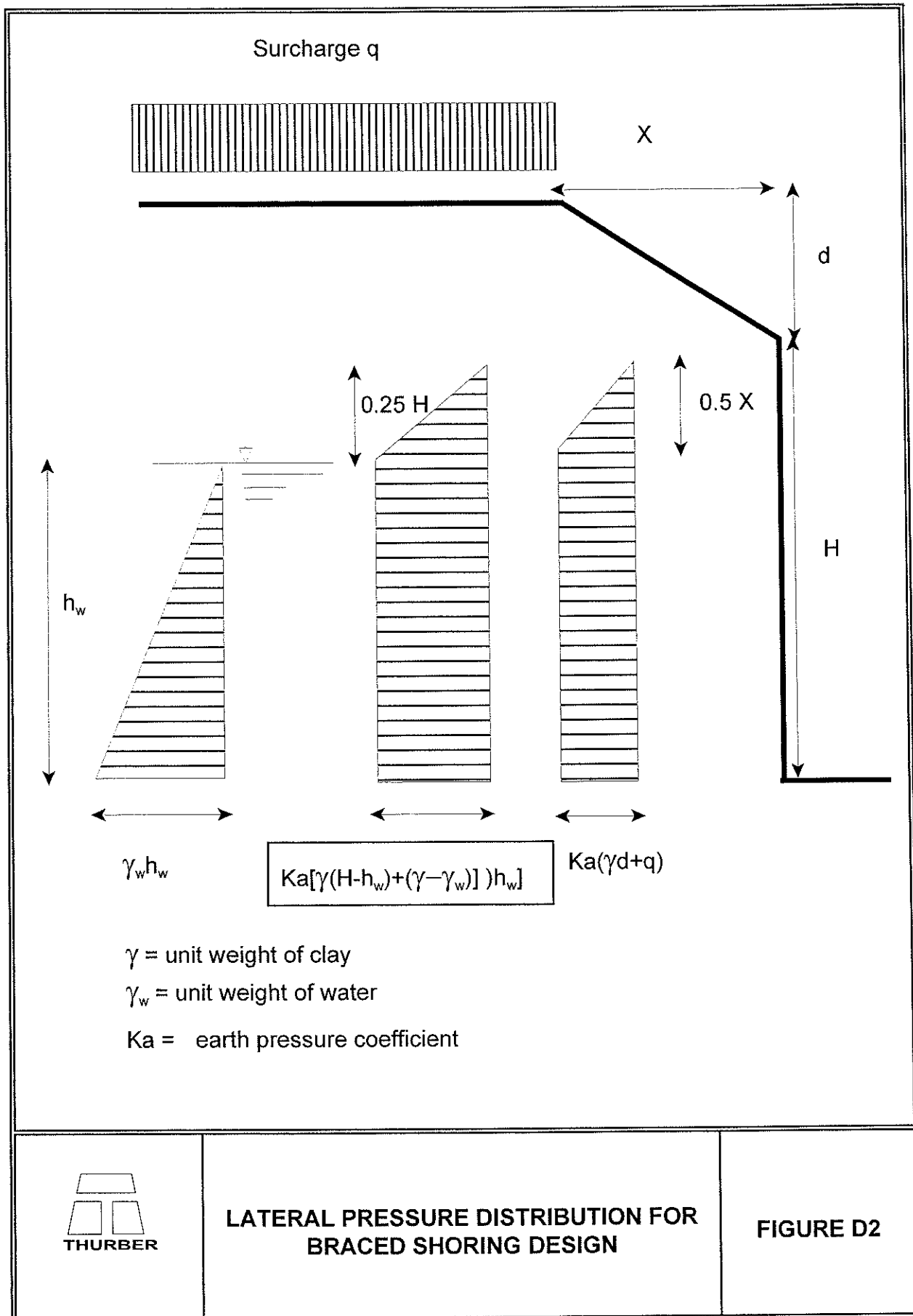
ABUTMENT ON COMPACTED FILL SHOWING
GRANULAR A CORE



THURBER

DWG. NO.

FIGURE D1



Upper Access Road Overpass

Appendix E

Special Provisions

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

Special Provision No. 902S01

September 2003

Excavation and Backfilling-Structures

902.02 REFERENCES

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

OPSS 510

902.03 DEFINITIONS

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

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

902.04 SUBMISSION AND DESIGN REQUIREMENTS

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

902.04.01 Site Survey

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

902.04.02 Working Drawings

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

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

902.04.03 Submission of Certificate of Conformance

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

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

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

902.05.03 Backfill

Subsection 902.05.03 is amended by the addition of the following:

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

902.05.04 Protection System

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

Protection systems shall be according OPSS 539.

902.07.01 Protection Schemes

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

902.07.02 Excavation

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

902.07.02.01 General

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

902.07.02.02 Excavation for Foundation

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

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

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

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

902.07.02.03 Excavation for Backfill and Frost Tapers

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

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

902.07.02.04 Preservation of Channel

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

902.07.02.05 Removals

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

902.07.03 Unwatering Structure Excavation

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

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

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

902.07.04 Backfilling

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

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

Backfilling shall be according to OPSS 501.

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

902.09 Measurement for Payment

902.09.01 Structures

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

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

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

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

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

902.10 Basis of Payment

902.10.01 Excavation and Backfill

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

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

WARRANT: Always with these tender items.

AMENDMENT TO OPSS 120, AUGUST, 1994

Special Provision

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

Construction Special Provision for Rock Excavation Utilizing Blasting

120.01 SCOPE

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

120.02 REFERENCES

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

Canadian Standards Association:

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

Ministry of Transportation Publications:

Ontario Traffic Manual Book 7

Federal Government Publication:

Explosives Act (Canada)

Department of Fisheries and Oceans Publication:

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

120.03 DEFINITIONS

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

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

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

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

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

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

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

120.04 SUBMISSION AND DESIGN REQUIREMENTS

120.04.01 General

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

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

120.04.02 Blasting Design and Monitoring

120.04.02.01 Blasting Consultant

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

120.04.02.02 Blasting Design

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

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

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

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

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

120.04.02.03 Blasting Monitoring

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

The blasting monitoring equipment shall meet the following requirements:

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

120.04.03 Blasting Permits, Approvals and/or Agreements

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

120.04.04 Pre-Blast Survey

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

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

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

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

120.04.05 Trial Blasting

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

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

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

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

120.04.06 Post Blast Survey and Reporting

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

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

120.04.07 Certificate of Conformance

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

120.05 MATERIAL

120.05.01 Explosives

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

120.05.02 Handling and Storage

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

120.06 EQUIPMENT

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

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

120.07 CONSTRUCTION

120.07.01 General

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

120.07.02 Safety Precautions

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

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

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

120.07.03 Notice

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

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

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

120.07.04 Vibration Monitoring

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

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

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

Structures

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

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

Fresh Concrete

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

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

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

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

120.07.05 Utilities

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

120.07.06 Excessive Vibration Readings – Work Stoppage

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

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

AMENDMENT TO OPSS 206, DECEMBER 1993

Special Provision

November 25, 2002

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.

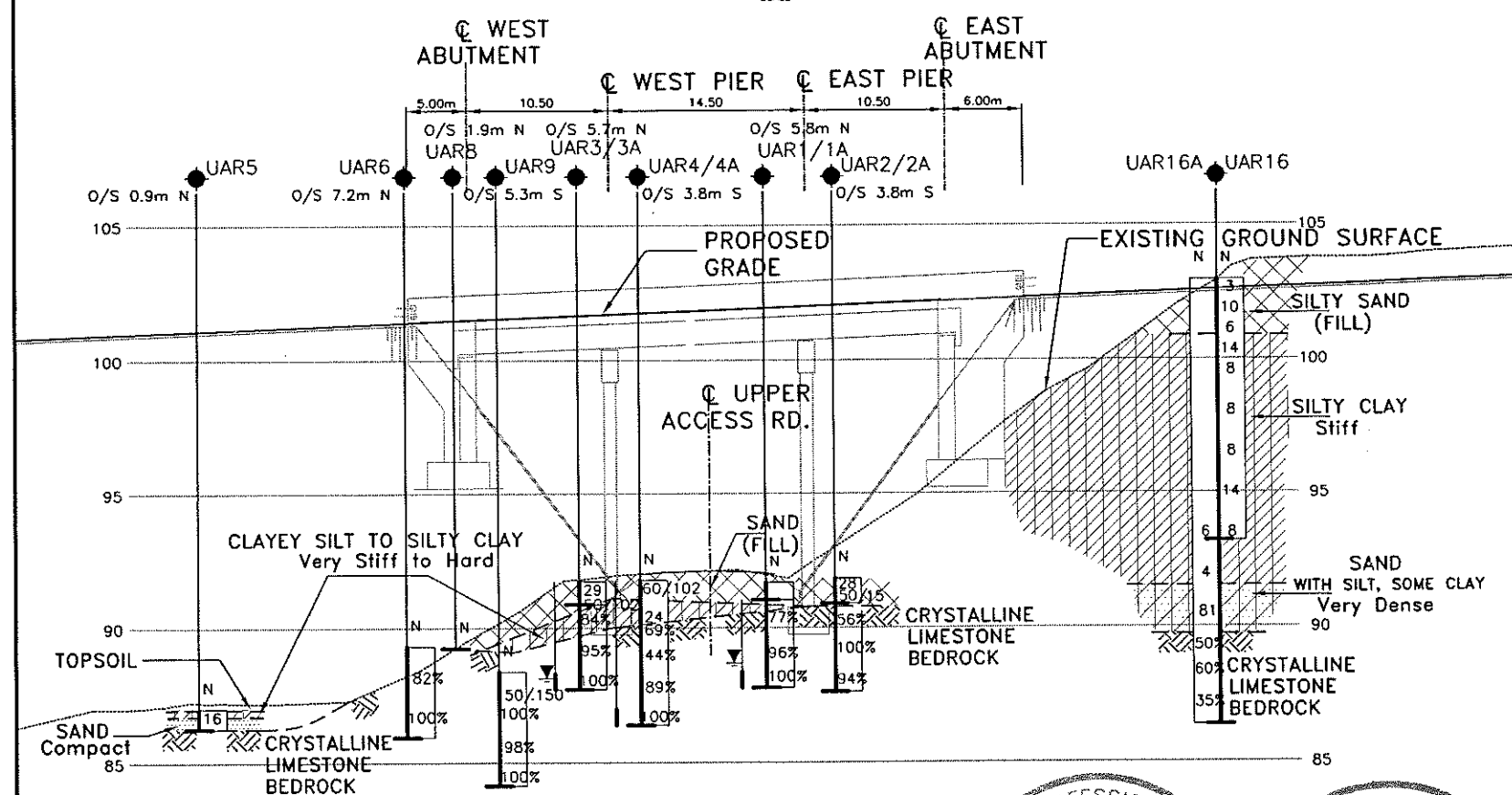
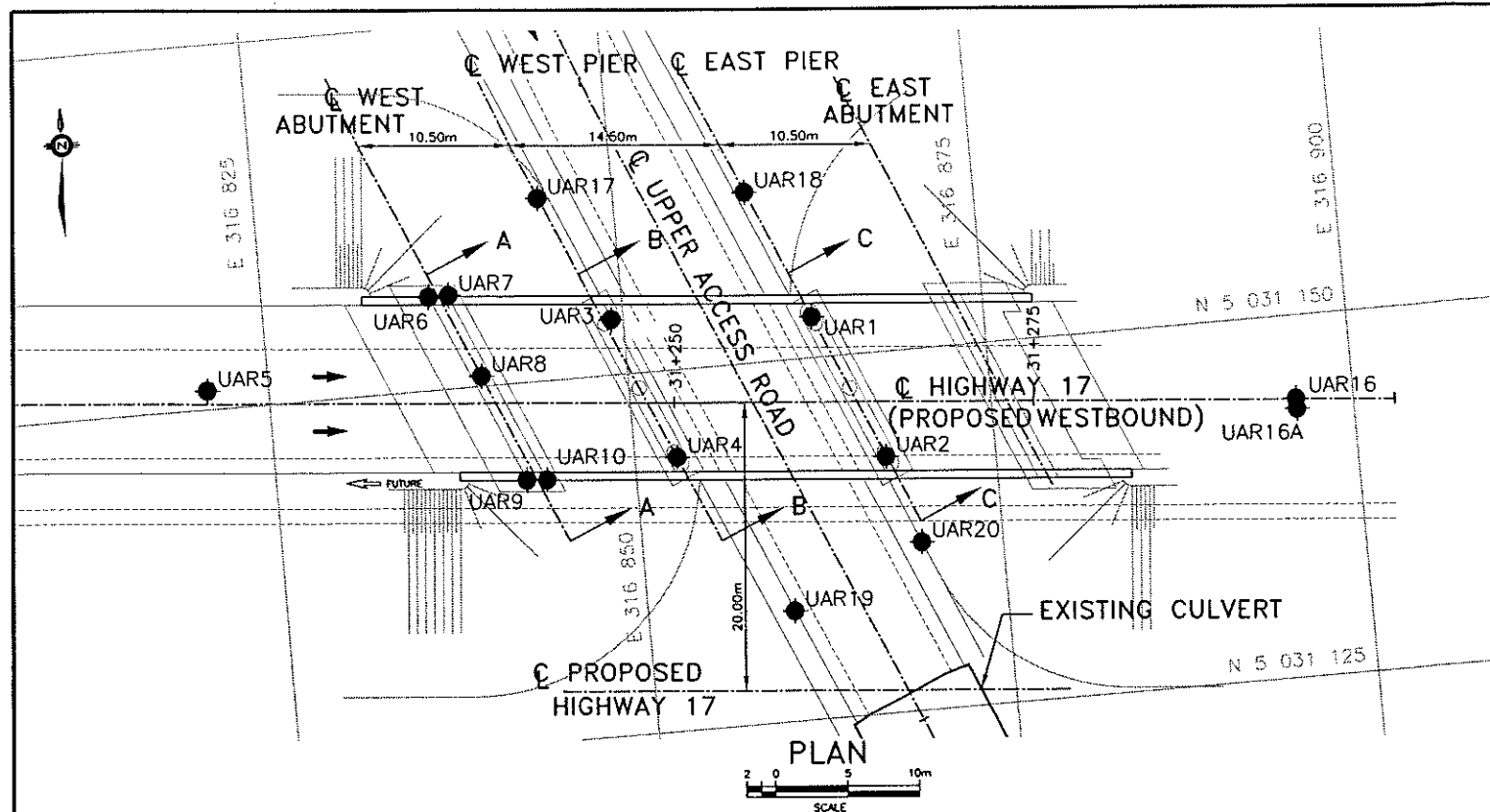
Dumping over the sides of embankments is permitted only if the material is surplus to the contract requirements and after the rock embankments have been completed. Dumping over the sides of embankments shall be restricted to standard offset and right of way limits unless otherwise specified in the Contract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

Upper Access Road Overpass
Highway 17 Twinning, Arnprior to Renfrew

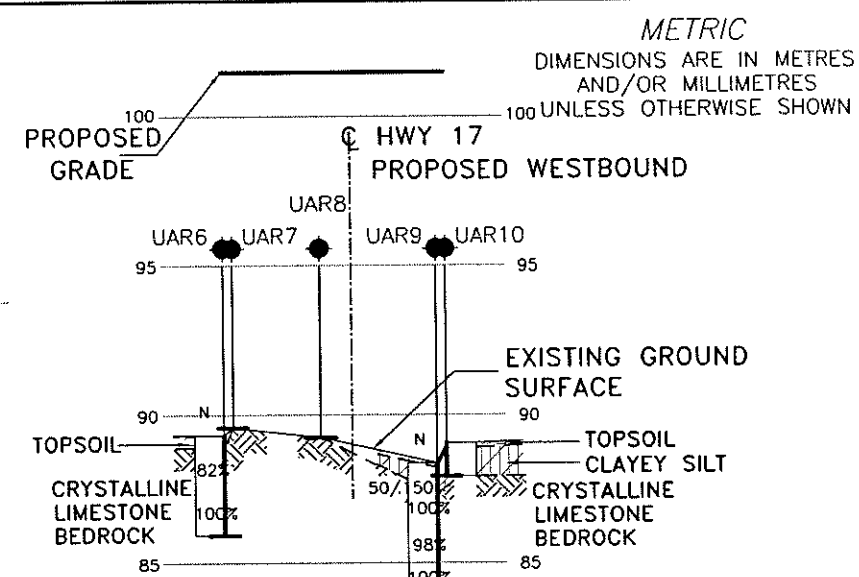
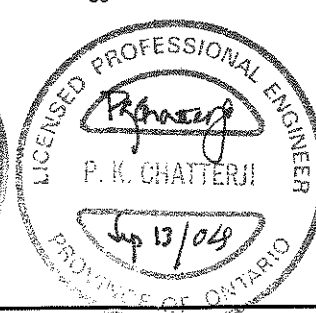
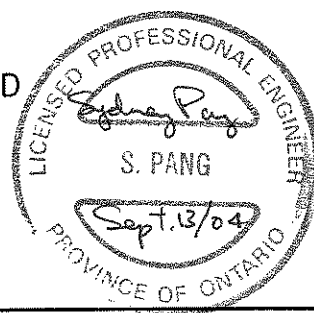
Appendix F

Drawing

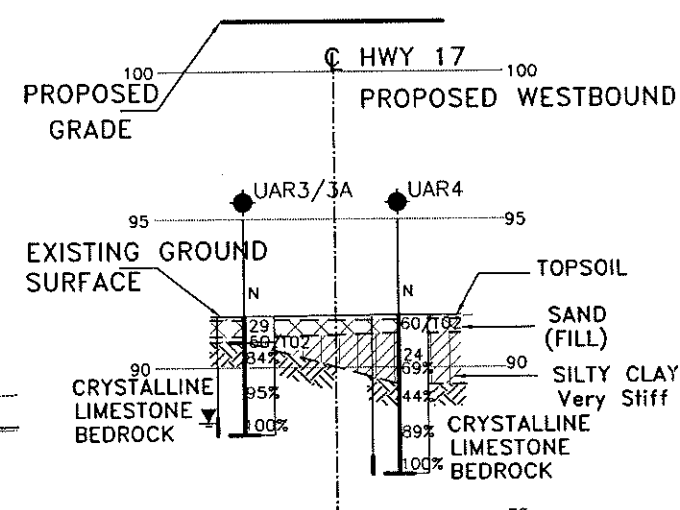




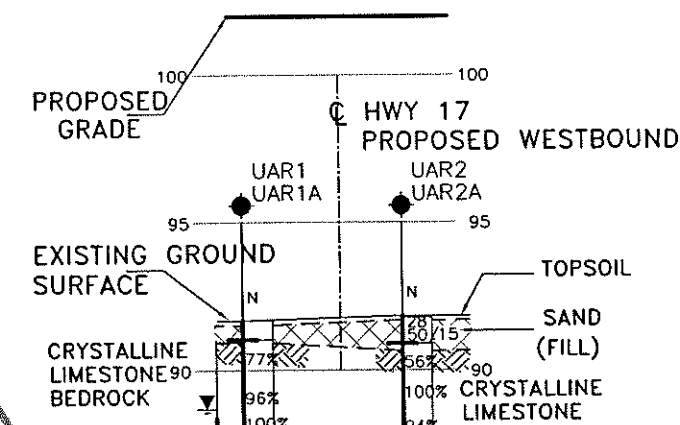
PROFILE @ HWY 17 WESTBOUND



SECTION A-A



SECTION B-B



SECTION C-C

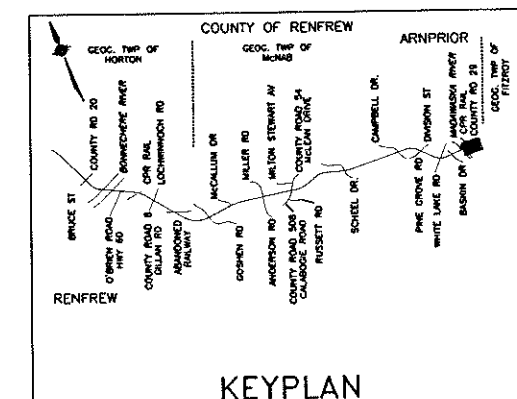
HWY. 17
WP NO. 647-92-00

HIGHWAY 17 TWINNING
UPPER ACCESS ROAD OVERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



THURBER ENGINEERING LTD.



LEGEND			
●	Bore Hole		
⊕	Dynamic Cone Penetration Test (cone)		
⊙	Bore Hole & Cone		
N	Blow/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)		

NO	ELEVATION	NORTHING	EASTING
UAR1	91.7	5 031 152.8	316 863.0
UAR2	91.6	5 031 142.8	316 867.3
UAR3	91.7	5 031 153.8	316 849.2
UAR4	91.8	5 031 144.0	316 852.9
UAR5	87.0	5 031 151.3	316 820.8
UAR6	89.3	5 031 156.4	316 836.7
UAR7	89.6	5 031 156.4	316 838.1
UAR8	89.2	5 031 150.7	316 839.9
UAR9	88.4	5 031 143.3	316 842.4
UAR10	89.1	5 031 143.2	316 843.8
UAR16	103.0	5 031 144.3	316 896.0
UAR17	91.7	5 031 162.5	316 844.8
UAR18	91.4	5 031 161.7	316 859.1
UAR19	91.9	5 031 132.7	316 860.1
UAR20	91.8	5 031 136.7	316 869.3

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

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

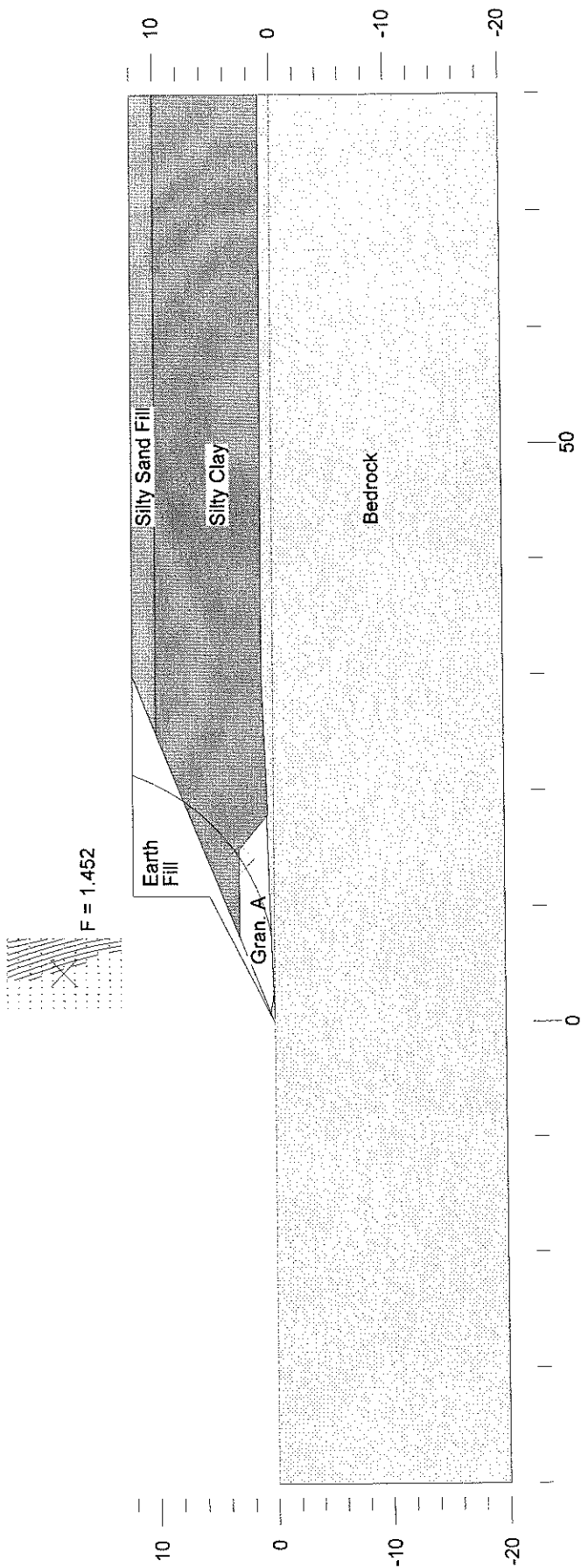
DATE SEP. 2004

Appendix G

Selected Stability Analyses Results

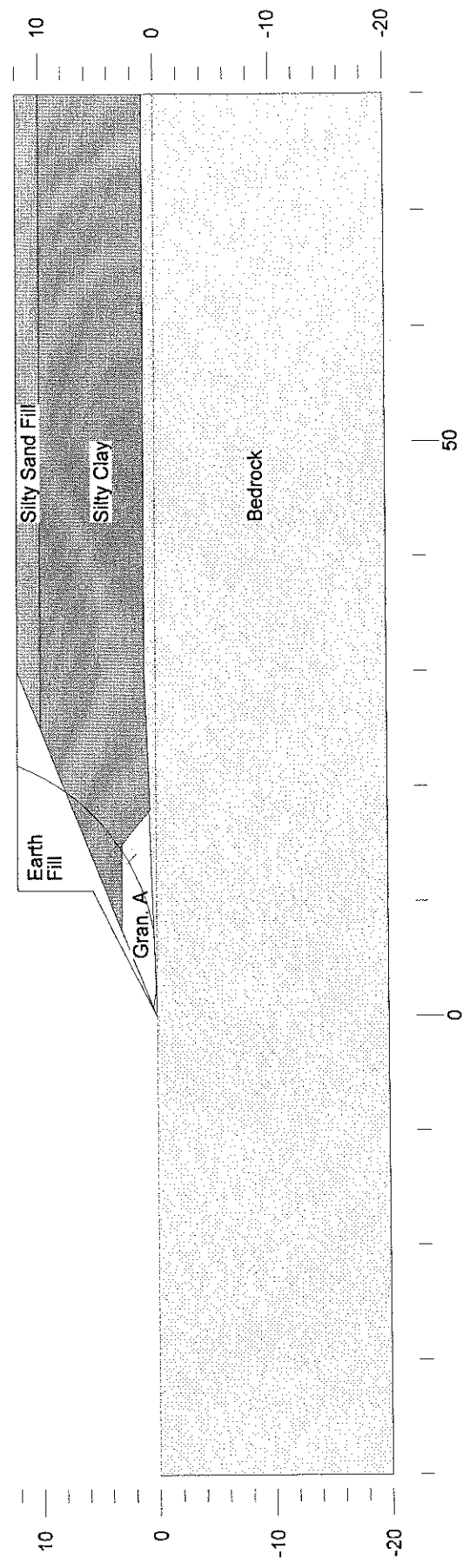
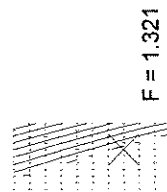
Thurber Engineering Ltd. - Toronto
 17-3745-0
 Highway 17 Twinning - Upper Access Road
 September 10, 2004
 Stability of Forward Slope - East Approach
 Figure G1 Undrained Analysis - Granular Fill

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Earth Fill	20	0	0	1
Silty Sand Fill	20	0	0	1
Silty Clay	20	70	0	1
Granular A	23	0	0	1
Bedrock	(Infinitely Strong)			



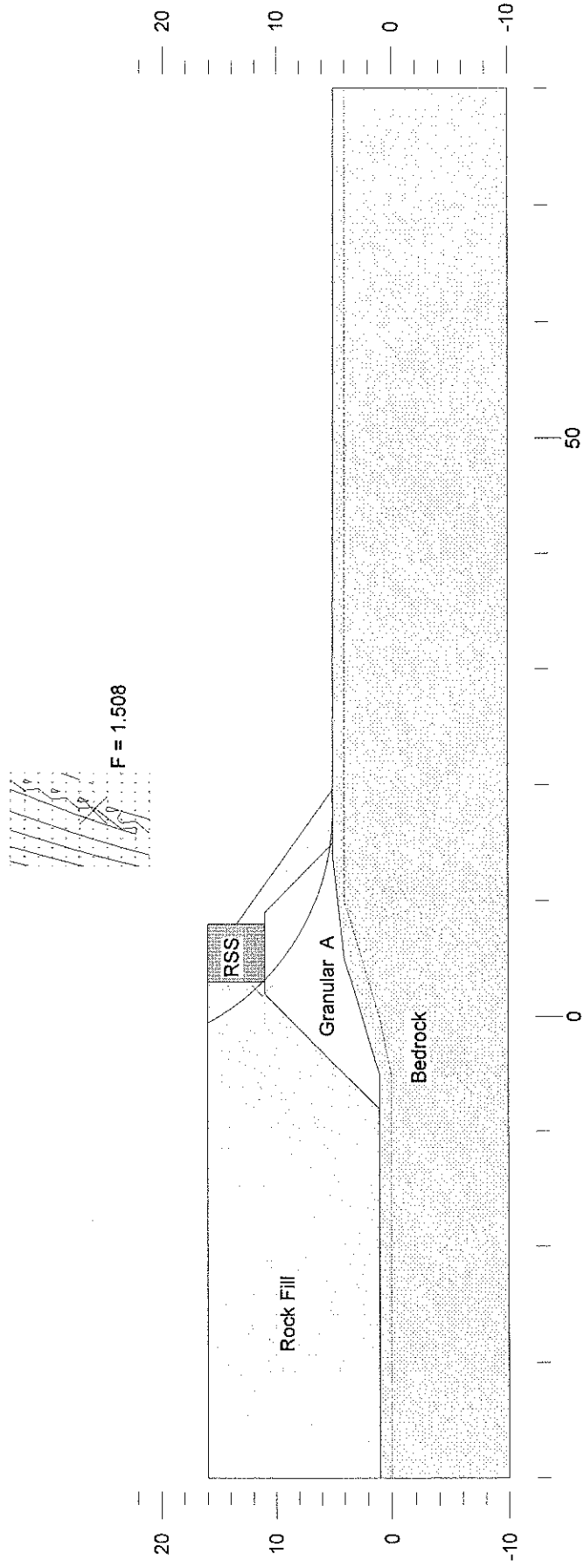
Thurber Engineering Ltd. - Toronto
 17-3745-0
 Highway 17 Twinning - Upper Access Road
 September 10, 2004
 Stability of Forward Slope - East Approach
 Figure G2 Drained Analysis - Granular Fill

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Earth Fill	20	0	0	1
Silty Sand Fill	20	0	0	1
Silty Clay	20	0	0	1
Granular A	23	0	0	1
Bedrock	(Infinitely Strong)			



Thurber Engineering Ltd. - Toronto
 17-3745-0
 Highway 17 Twinning - Upper Access Road
 February 1, 2004
 Stability of RSS Wall and Embankment - West Approach
 Figure G3 Drained Analysis

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Rockfill (front)	19	0	0	0
RSS Wall	21	500	0	0
Rockfill (back)	19	0	0	0
Gran. A Core	23	0	35	1
Bedrock	(Infinitely Strong)			



	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Earth Fill	20	30	0	0
RSS Wall	21	500	0	0
Earth Fill	20	30	0	0
Gran. A Core	23	35	0	1
Bedrock	(Infinitely Strong)			

