FOUNDATION INVESTIGATION AND DESIGN REPORT
BASKIN DRIVE OVERPASS (EASTBOUND LANES)
HIGHWAY 17 TWINNING
ARNPRIOR TO RENFREW, ONTARIO
G.W.P. 647-92-00, SITE NO. 29-423/1
GEOCRES Number: 31F-140

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 Baskin Drive Overpass (Eastbound Lanes) structure that will carry the realigned Baskin Drive under the Highway 17 eastbound lanes near the Town of Arnprior, Ontario. During a previous preliminary investigation for the existing Highway 17, a borehole was drilled by the Ministry of Transportation (MTO) in the general vicinity of the site area, and the factual data from that investigation has been used as reference during the preparation of this report.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan and soil strata drawing with a stratigraphic profile and cross-sections, records of boreholes, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained during the course of the present investigation.

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

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

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

Reference is also made to the Baskin Drive Overpass – Highway 17 WBL report for relevant subsurface information.

2 SITE DESCRIPTION

The site is located near the existing intersection of Highway 17 and Baskin Drive in the Township of McNab. County of Renfrew, Ontario (approximate EBL Mainline Station 29+954). This site is located to the south of the Town of Arnprior.

The site is situated in an area of relatively flat terrain characterized by glacial drift overlying bedrock. Vegetation is light and mainly consists of grass and occasional shrubs. No rock outcrop was observed in the vicinity of the site. The Madawaska River to the east largely governs regional drainage in the area.

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

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project was carried out between November 10 and December 16, 2003. The site investigation consisted of drilling and sampling a total of eight boreholes to depths ranging from 1.2 m to 8.4 m. The boreholes were numbered BAS-14 to BAS-19, BAS-21 and BAS-22. Boreholes BAS-13 and BAS-20 were originally staked in the field at locations in close proximity to a number of buried utilities. Thurber was advised by MTO and NCE that the holes should be eliminated from the field program.

This investigation was carried out in conjunction with the investigation for the westbound lane (WBL) structure. The results for the WBL structure are presented in a separate report.

The borehole locations were marked in the field by surveyors from J. D. Barnes Limited who also provided their coordinates and geodetic elevations. Utility clearances at the borehole locations were obtained by Thurber prior to drilling.

George Downing Estate Drilling Limited of Calumet, Quebec supplied a track mounted CME 55 drill rig and conducted the drilling, sampling and in-situ testing operations. Auger drilling techniques were used to advance the boreholes in the overburden and samples were obtained using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Boreholes BAS-14, BAS-16, BAS-19 and BAS-21 at the foundation elements were advanced 2.9 m to 3.4 m into bedrock using NQ size rotary coring techniques.

A standpipe piezometer was installed in each of Boreholes BAS-16, BAS-19 and BAS-21 to monitor the groundwater level. At this site, 19 mm diameter Schedule 40 PVC pipes with 1.5 m long slotted screens were installed at the bottom of the open boreholes. The sand screens surrounding the pipes were about 1.5 m long. Bentonite holeplug seals were placed just above the sand screen and just below ground surface in each installation. The remaining space in the boreholes was backfilled with drill cuttings.

A member of Thurber's technical staff supervised the drilling and sampling operations on a full time basis. The supervisor logged the boreholes, secured the soil and rock samples in labelled containers and core boxes, respectively, which were than transported to Thurber's Oakville laboratory for further examination and testing.

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

4 LABORATORY TESTING

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.

Seven selected samples were subjected to gradation analysis. Atterberg Limit tests were carried out on four samples of clay to determine the plasticity characteristics. The results of these tests are shown on the Record of Borehole sheets in Appendix A and on the plots in Appendix B.

Point load tests were performed on selected rock cores retrieved from the two boreholes and these results are shown in Table 1 attached immediately following the text.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

5.1 General

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

In general, the site is underlain by 1.2 m to 6.0 m of overburden consisting of topsoil, asphalt, fill and silty clay overlying limestone bedrock of the Ordovician Period.

5.2 Topsoil and Asphalt

Topsoil was encountered in the fields adjacent to the existing highway, in Boreholes BAS-16, BAS-17 and BAS-19, and varied in thickness from 25 mm to 150mm. Topsoil thickness may vary between and beyond the borehole locations.

Asphalt of about 100 mm in thickness was encountered at locations on the highway, in Boreholes BAS-15, BAS-21 and BAS-22.

5.3 Fill

5.3.1 Sand to Sand and Gravel Fill

Cohesionless fill was encountered at ground surface, or beneath the topsoil and asphalt, in all boreholes except BAS-16. This fill generally consists of sand, to sand and gravel. The presence of cobbles or boulders should be anticipated in the granular fill. The thickness of the cohesionless fill varied from about 0.7 m at Boreholes BAS-19 to 2 m in Borehole BAS-14.

The sand was found to be in a typically compact state as indicated by SPT 'N' values of between 13 blows and 30 blows for 0.3 m penetration. The 'N' values were less than 10 blows or greater 30 blows at some locations, indicating the presence of loose or dense zones, respectively. The sand was noted to be wet at some locations. Figure B1 shows the grain size distribution of two samples of the sand fill. The measured moisture content of samples of the cohesionless fill varied from 3% to 55%, but was typically in the range of 6% to 15%.

5.3.2 Clayey Silt to Silty Clay Fill

Clayey silt fill was encountered below the topsoil in Borehole BAS-16. The fill was 0.5 m thick and had an SPT 'N' value of 10 blows for 0.3 m penetration indicating that the clayey silt fill has a stiff consistency. A sample of this fill had a moisture content of 15%.

A silty clay fill was noted between 2 m and 2.3 m depths in Borehole BAS-15.

5.4 Silty Clay

Silty clay was encountered in all boreholes beneath the fill, except in Borehole BAS-17 where auger refusal was met at the bottom of the fill and in Borehole BAS-16 where a silty clay till was present below the fill. The silty clay varied in thickness from about 0.3 m in Borehole BAS-21 (east abutment area) to about 4.7 m in Borehole BAS-22 (east abutment area).

The clay is generally brown to grey in colour and has a typically medium plasticity. The SPT 'N' values generally range from 7 to 16 blows for 0.3 m penetration indicating a stiff to occasionally very stiff consistency. The moisture content of the samples varied from about 10% to 42%. At a few locations in the east abutment area (Boreholes BAS-18, BAS-19 and BAS-22), the lower portion of the clay becomes firm with 'N' values in the range of 5 to 7 blows for 0.3 m penetration.

Figure B2 shows the grain size distribution of four silty clay samples. These tests indicate that the clay content of these samples ranges between 42% and 53%. Figure B3 is a plasticity chart showing that these silty clay samples had measured liquid limits of between 42% and 48%, and corresponding plasticity indices of between 24% and 27%, indicating an intermediate plasticity (group symbol of CI).

The silty clay in Borehole BAS-16 contains a relatively large proportion of sand with trace gravel, and is classified as a till. This till is about 1.1m thick at this location. The till has a very stiff consistency as indicated by a SPT 'N' value of 28 blows for 0.3 m penetration. An 'N' value of 50 blows for less than 0.3 m penetration may be attributed to the possible presence of cobbles or boulders. Figure B4 shows the grain size distribution of a silty clay till sample. The moisture content of the till samples varied from about 10% to 12%.

5.5 Bedrock

The soils described above are underlain by crystalline limestone bedrock of the Ordovician Period. The bedrock was proven by coring in Boreholes BAS-14, BAS-16, BAS-19 and BAS-21. The bedrock surface was inferred from refusal to auger penetration in other boreholes drilled at this site.

Bedrock surface depths and elevations, proven and inferred, at the borehole locations are summarized in the following table.

Borehole	Ground Surface Elevation (m)	Bedrock Surface	
		Depth (m)	Elevation (m)
BAS-14	106.2	4.3*	101.9*
BAS-15	106.2	2.3	103.9
BAS-16	104.8	1.8*	103.0*
BAS-17	105.4	1.2	104.2
BAS-18	105.9	5.1	100.8
BAS-19	105.8	5.3*	100.5*
BAS-21	106.1	2.3*	103.8*
BAS-22	106.0	6.0	100.0

Note: * Proven by coring.

The bedrock surface appears to generally dip to the north and northeast along the Highway 17 EBL alignment, except at the southeasterly corner of the site (Borehole BAS-22) where the bedrock surface drops abruptly.

The rock is generally fresh to slightly weathered, mostly at joints. It is very thinly to thinly bedded, generally whitish grey in colour with black sub-vertical banding.

Rock core recovery, measured in TCR, generally ranged from 93% to 100%, with one lower value of 86% measured in Run #1 of Borehole BAS-14. Measured RQD values typically ranged between 80% and 100% indicating a good to excellent rock quality. An occasional lower value of 73% was measured in Run #1 of Borehole BAS-14.

The Fracture Index (FI) of the rock, expressed as the number of natural fractures per 0.3 m of core, was generally low with values of between 0 and 2, and occasional values up to 3 and 4. The joint orientation is mostly sub-vertical. The joint conditions were typically rough and uneven with evidence of calcite infilling in some fractures.

Point Load Tests were conducted on the rock cores at selected intervals. The inferred Unconfined Compressive Strength (UCS) of the intact rock cores ranged from 95 MPa to 135 MPa indicating that the intact rock is strong to very strong. A summary of the Point Load Test results is presented in Table 1 immediately following the text.

5.6 Water Levels

Standpipe piezometers were installed in Boreholes BAS-16, BAS-19 and BAS-21 and their water levels were measured on separate visits made after the completion of installation. The piezometric readings are presented in the table below.

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
BAS-16	December 18, 2003	0.8	104.0
	February 5, 2003	1.3	103.5
	March 11, 2004	0.6	104.2
BAS-19	December 18, 2003	1.7	104.1
	February 5, 2004	2.3	103.5
	March 11, 2004	Frozen	
BAS-21	Standpipe was apparently broken shortly after installation. No reading was taken.		

Based on these observations, local groundwater levels are anticipated to vary between Elevations 104.2 m and 103.5 m. All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe climatic events.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 GENERAL

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

It is understood that the design plan calls for the construction of a new structure to carry the twinned Highway 17 eastbound lanes (EBL) over the realigned Baskin Drive that will be constructed in a cut. The EBL centreline coincides with the centreline of the existing lanes of Highway 17.

The proposed single span, reinforced concrete rigid frame overpass structure will have an approximate length of 25 m (on the Baskin Drive alignment) with a clear span of approximately 11 m (perpendicular to the Baskin Drive alignment). The abutments will be skewed at about 50° to the Highway 17 EBL.

It is understood that the Highway 17 Twinning EBL mainline will be at the existing intersection grade of about Elevation 106 ± m. Therefore, the alignment for Baskin Drive will be in a cut with a final grade approximately 5 m below the proposed Highway 17 grade which is at approximate Elevation 106 m.

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

7 STRUCTURE FOUNDATIONS

7.1 General

The proposed bridge for this site will consist of a rigid frame structure with an abutment support on each side of Baskin Drive.

The stratigraphy encountered at the site consists of surficial layers of topsoil and asphalt underlain by fill comprising of sand to sand and gravel and clayey silt. These materials are further underlain either by crystalline limestone bedrock or by native deposits of silty clay and silty clay till overlying bedrock.

The elevations at which bedrock was encountered at the foundation elements are summarized in the table below.

Location	Borehole Number	Elevations (m)	
		Existing Ground Surface	Top of Bedrock
West Abutment			
Northeast Corner	BAS-14	106.2	101.9*
Centre	BAS-15	106.2	103.9
Southwest Corner	BAS-16	104.8	103.0*
Southeast Corner	BAS-17	105.4	104.2
East Abutment			
Northwest Corner	BAS-18	105.9	100.8
Northeast Corner	BAS-19	105.8	100.5*
Southwest Corner	BAS-21	106.1	103.8*
Southeast Corner	BAS-22	106.0	100.0

- Proven by coring

The above borehole program satisfies the requirements of the foundation investigation Terms of Reference. However, several boreholes encountered auger refusal but did not prove bedrock. In view of the slight variability of the bedrock surface, it is recommended that two additional boreholes be drilled near the locations of Boreholes BAS-15 and BAS-22 to obtain rock core samples in order to confirm the founding conditions and elevations prior to construction.

7.2 Foundation Alternatives

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

Initial consideration was given to the following foundation types:

- Spread footings
- Driven piles
- Augered caissons (drilled shafts)

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

Based on the geometry of the cut and the presence of bedrock above or slightly below the final Baskin Drive grade, it is considered impractical and unnecessary to use driven piles or augered caissons at this site.

The ground conditions at this site are not considered feasible for an integral abutment design.

Footing on engineered fill is also not considered feasible at this site due to the limited embankment height between the bedrock surface and the proposed Highway 17 grade, and the likelihood that the footings would have to be located further back from the cut resulting in a longer superstructure.

In view of the foregoing and the proposed rigid frame design, it is recommended that the structure be supported by spread footings founded on bedrock.

7.3 Spread Footings on Bedrock

7.3.1 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 found at approximately 200 mm above the local top of rock and the difference made up using mass concrete fill in order to develop a level bearing surface. 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 top of rock elevations for footing design are as follows.

West Abutment

The top of rock varies from Elevation 101.9 m to Elevation 104.2 m across the footprint of the proposed foundation element.

East Abutment

The top of rock varies typically from Elevation 100.8 m to Elevation 100.0 m across the footprint of the proposed foundation element. There is, however, a relatively abrupt rise (rock ridge) in bedrock elevation from Boreholes BAS-22 to BAS 21 at the southerly edge of the foundation.

7.3.2 Bearing Resistance on Bedrock

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

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

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

7.3.3 Horizontal Resistance on Bedrock

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

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

7.3.4 Frost Cover

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

8 CANTILEVERED RETAINING WALLS

8.1 General

An earth cut will be required along the Baskin Drive alignment and it is anticipated that retaining walls may be required along both sides of the road. The required lengths of the retaining walls will depend on the configuration of the new cut, road embankment and the bridge, as well as the length of any permanent open cut sections with inclined side slopes that may be feasible.

8.2 Spread Footings on Bedrock

Consideration may be given to the use of concrete cantilevered walls. Given the presence of shallow bedrock below the proposed Baskin Drive grade, it is recommended that the retaining wall footings be founded on bedrock. The retaining walls should be designed in accordance with the requirements of CHBDC, 2000.

Along the west side of Baskin Drive, from the north limit of the EBL west abutment to the south limit of the WBL west abutment, the footings may be founded on bedrock assumed to be sloping downwards from approximate Elevations 101.9 m to 97.8 m (based on Borehole BAS-5 drilled for the WBL structure). To the south of the EBL west abutment, the footings may be founded at an assumed Elevation 103.0± m (based on Borehole BAS-16).

Along the east side of Baskin Drive, from the north limit of the EBL east abutment to the south limit of the WBL east abutment, the footings may be founded on bedrock at an approximate Elevation 100.3 ± m (based on Borehole BAS-11 drilled for the WBL structure). To the south of the EBL east abutment, the footings may be founded at an assumed Elevation 100.0 m (based on Borehole BAS-22).

There was no borehole drilled between and beyond the two proposed structure locations. Once the locations of the required retaining walls are identified, additional boreholes should be drilled to confirm the founding conditions and elevations.

Detailed design recommendations on vertical and horizontal geotechnical resistances, stepped footings, eccentric and inclined loads are similar to those for the abutment footings (see previous Section 7.3). Design recommendations on earth pressures are similar to those presented in the subsequent Section 15, Earth Pressures.

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

9 RETAINED SOIL SYSTEMS

Retained Soil System (RSS) walls founded directly on sound bedrock may be used for wing walls and other retaining structures at this site. However, due to the anticipated post construction settlement that can be induced on the native silty clay to clayey silt, it is considered that there is a medium level of risk associated with the use of RSS walls founded on the clayey deposits. Should the latter option be pursued, the designers must satisfy themselves that the estimated magnitudes of settlement presented in Section 9.4 below are acceptable for the satisfactory performance of the walls.

RSS walls should be specified to be "High Performance" and "High Appearance". The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

9.1 Foundations

At this site, it is recommended that the levelling pad for an RSS wall be centred on top of a mat of engineered fill that is itself resting on native very stiff to stiff silty clay or bedrock. Where applicable, the native soil under the RSS foundation should be proof-rolled and be compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD) at ±2% of its optimum moisture content. 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 500mm thick and should be at least twice as wide as the levelling pad.

The following parameters may be used for the design of the levelling pad:

- Factored geotechnical resistance at ULS of 900 kPa, and geotechnical resistance of 350 kPa at SLS on an engineered Granular A pad resting directly on bedrock.
- Factored geotechnical resistance at ULS of 250 kPa, and geotechnical resistance of 175 kPa at SLS on an engineered Granular A pad resting on very stiff to firm silty clay.
- Coefficient of friction of between cast in-situ concrete levelling pad on Granular A is 0.7.

In addition to the requirements for the levelling pad, the RSS should be founded on native, stiff silty clay or on bedrock. 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 does not govern design of foundation on rock).
- Factored geotechnical resistance at ULS of 225 kPa, and geotechnical resistance of 150 kPa at SLS, on native, very stiff to firm silty clay.
- Ultimate coefficient of friction between RSS mass and Granular A is 0.55.
- Ultimate coefficient of friction between RSS mass and native, stiff silty clay is 0.45.
- Ultimate coefficient of friction between RSS mass and sound bedrock is 0.85. 0.6

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

9.2 Global Stability

The global stability of the RSS is dependent on the characteristics of the road embankment fill and the foundation soils, the geometry of the cut and road embankment, and location of the RSS within the cut.

RSS may be used as wing walls at the abutments and as retaining walls along both sides of Baskin Drive. It is envisaged that the RSS will be founded on native, stiff to very stiff silty clay, or on prepared bedrock surface. Any soft or otherwise disturbed clay exposed at the subgrade should be sub-excavated and replaced with approved, compacted granular materials.

For the purpose of 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.

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

- RSS block with dimensions of 5 m (assumed retained height) by 3 m (assumed length of RSS reinforcement, 60% of retained height) founded on native silty clay overlying bedrock.
- Groundwater level at 1 m below the base of the cut.

Results of the analyses yield Factors of Safety greater than 1.3, which indicate that global stability can be maintained for the assumed RSS configuration. Figures G1 and G2 show selected stability analyses results. Global stability for an RSS wall founded directly on bedrock is not anticipated to be a design issue at this site.

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

9.3 Internal Stability

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

9.4 Settlement

For RSS walls founded on bedrock, settlements will be negligible. The settlement of a RSS wall founded on engineered fill or native clays will depend on the thickness of the pad, the material used, the conditions of the native subgrade and the quality of construction. At this site, immediate settlements are expected to be less than 25 mm for RSS walls founded on well compacted engineered fill, or native, stiff silty clay subgrade prepared as recommended in this report. These settlements are expected to occur essentially as the RSS is constructed. Post construction settlement is anticipated to be up to 10 to 15 mm at locations where the wall is at its maximum height of about 5 m.

10 PERMANENT CUT

10.1 General

For the purpose of 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.

A permanent cut is required to construct the realigned Baskin Drive at this site. The cut will be formed through up to 5 m of cohesionless fill and typically stiff silty clay overlying bedrock.

10.2 Earth Cuts

Where space permits, open cut excavation with sloping sides may be carried out. Where there is space restriction or in order to minimize excavation, steel soldier pile and timber lagging walls may be used to provide temporary support to the soils during excavation. The base of the cut may be in the order of 3 m below the existing water table. Groundwater control (see later section) will be required when excavating below the groundwater level.

Unsupported earth cuts will be stable provided that the slope inclinations are not steeper than 2H : 1V. Figures G3 and G4 show selected stability analyses results.

Design of a soldier pile and lagging system may be carried out as recommended in Section 11.3. The sockets within bedrock must be formed below the base of the cut.

Based on the existing configuration and in order to accommodate the pavement structure, it is anticipated that the subgrade of the cut may consist of typically stiff silty clay, or bedrock at some locations. It is considered that basal stability can be maintained.

The cut will be formed predominantly in cohesionless fill and the relatively impermeable silty clay. Minimal water seepage from the clay layer is expected. Water perched within the fill could seep into the excavation. Rock may also be exposed in some areas (see later section). However, it is anticipated that a significant amount of water accumulating in the cut would originate from surface precipitation and runoff.

It is recommended that construction traffic be minimized on the silty clay subgrade surface. Any disturbed soil should be sub-excavated and backfilled with approved, compacted granular materials.

It is assumed that a stormwater handling system will be installed to handle and dispose of water in the cut. It is recommended that sub-drains be installed along both sides of the roadway. Consideration may also be given to transverse subdrains orienting perpendicularly across the roadway.

The sub-drains may consist of 150 mm diameter perforated pipes with inverts at least 0.5m below the road subgrade under a minimum of 1.9 m earth cover, or equivalent insulation. The backfill around and above the subdrain should consist of free-draining granular filter materials wrapped in geotextile filter cloth. Flexible perforated pipes wrapped in filter socks may also be used. The subdrains should not be allowed to freeze and must be adequately incorporated into the permanent roadway drainage system. Reference should be made to SP 405S01 Construction Specification for Pipe Subdrains for details.

Roadway drainage, subgrade preparation and associated issues should be considered in conjunction with the roadway and pavement design carried out by others.

Vegetation cover should be established on all exposed earth slopes to protect against surficial erosion. Reference may be made to SP 572S01 (supersedes OPSS 572) for more detailed requirements. If continual seepage and surficial instability are evident, remedial measures including the use of gravel sheeting may be required.

10.3 Rock Cuts

Rock cuts up to 3 m high will be required along Baskin Drive and should conform to OPSD 201.020. The cut face may be formed vertically or at a slope of 1H:4V. Drainage within the cut may be provided by 0.3m of shattered rock below the pavement structure and ditching along the toe of the rock face.

Depending on the location, orientation and height of the rock cut with respect to the pattern of joints or fractures in the rock mass, potentially unstable rock wedges may exist along the final cut slope face at and beyond the bridge abutments.

After excavation of the rock cut in the vicinity of the structure, the Contractor should scale all loosened rock from the face and the Contract Administrator should retain a rock slope stability expert to examine the cut. Where the wall of the rock cut at and beyond the foundation develops potentially unstable wedges or where over-break occurs, the Contractor should place mass concrete fill or install rock bolts as required. The remedial work should be designed by and carried out under the direction of the rock slope stability expert retained by the Contract Administrator. The contract should include an NSSP to this effect.

11 EXCAVATION AND BACKFILL

11.1 General

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the granular fill is classified as a Type 3 soil above the water table and a Type 4 soil below the water table; the stiff native silty clay is classified as a Type 2 soil above the water table and a Type 3 soil below the water table.

11.2 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 C

11.3 Earth Excavation

In addition to SP 902S01, a NSSP should be included in the contract alerting the Contractor to the possible presence of cobbles and boulders in the overburden, particularly in the silty clay till layer above the bedrock.

Where open cutting with inclined slopes (according to OHSA) is not feasible, a braced soldier pile and lagging wall is considered to be suitable for use as temporary shoring at this site. The soldier piles will need to be socketted into bedrock through pre-augered holes. It is anticipated that the shoring system may be stiffened by struts or cross bracings, where applicable.

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

For a temporary braced soldier pile and lagging wall, the lateral pressure diagram as shown on Figure D1 may be used for design in conjunction with the following parameter values.

γ	=	20 kN/m^3
γ_w	=	10 kN/m^3
K_a	=	0.4 (silty clay / silty clay till)
h_w	=	0 (assuming that there is no hydrostatic pressure build-up behind a presumably permeable wall)
H	=	depth to base of excavation (rock surface), m

Below the excavation base, and taking into account three dimensional effects, lateral earth pressures are applied over a width of 3B, where B is the diameter of the socket. 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 c = 2,000 kPa (equivalent Mohr-Coulomb cohesion

based on Hoek and Brown rock mass classification)
L = depth of socket in rock, m

11.4 Rock Excavation

Where quantities of rock have to be removed, 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, rock cut faces 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 footing 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 any nearby structures including temporary structures. The contract documents should alert the contractor to these installations. The Contract Administrator should retain a blasting expert for review of the Contractor's procedures prior to approving them.

12 GROUNDWATER CONTROL

The excavation base for the foundation elements and the base of the permanent cut will be below the groundwater table at this site. The excavations will extend through relatively impermeable silty clay overlain by granular fill near the existing highway. As excavation proceeds, seepage and surface water accumulating within the cut must be controlled until such time that a permanent drainage system (associated with the Baskin Road construction) becomes operational. Footing construction must be carried out in the dry.

The design of the unwatering system should be the responsibility of the Contractor. However, a system of perimeter ditches supplemented by pumping from filtered sumps, may be considered for use at this site to control groundwater seepage, surface runoff and precipitation.

13 ROAD EMBANKMENTS

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 has been estimated based on the methods described in the CHBDC, 2000 Commentary Section C6.6.3.6.

The road fills for this structure will be constructed on compact granular fill or stiff to very stiff silty clay overlying bedrock. These soils will satisfactorily support the low road embankment fills at this site, which are expected to be between 1 m and 2 m in height. Global stability of the approach fills is, therefore, not considered a design issue at this site.

Small magnitudes of settlement, in the order of 10 mm or less, will occur due to compression of the fill if it consists of well compacted granular or SSM materials. The settlement should be complete by the end of construction and post construction settlement is considered to be negligible.

Where applicable, embankment construction should be in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002 and included in Appendix E.

Earth fill embankment slopes must be provided with erosion protection in accordance with SP572S01 (supersedes OPSS 572).

14 BACKFILL TO ABUTMENTS

In the case where the abutment walls are formed at a short distance away from the rock face, it is recommended that backfill to the abutments should consist of OPSS Granular "B" Type II. It is considered impractical to use rock fill in this design configuration.

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, where applicable.

All granular materials should meet the specifications of Special Provision 110F13.

Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.06.

The design of the abutment should incorporate a subdrain as shown in OPSD 3501.000, where applicable.

15 EARTH PRESSURE

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

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

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

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

where
P_h = horizontal pressure on the wall (kPa)
K = earth pressure coefficient (see table below)
γ = unit weight of retained soil (see table below)
h = depth below top of fill where pressure is computed (m)
q = value of any surcharge (kPa)

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or at a depth of 1.7 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical unfactored values are shown in the following table.

Wall Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°, \gamma = 22.8\ kN/m^3$		OPSS Granular B Type I $\phi = 32°, \gamma = 21.2\ kN/m^3$		Rock Fill $\phi = 42°, \gamma = 19.0\ kN/m^3$	
	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	-

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 Canadian Highway Bridge Design Code.

16 SEISMIC CONSIDERATIONS

16.1 Seismic Design Parameters

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

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

The soils at this site consist of compact to dense granular fill and stiff to very stiff silty clay and silty clay till with a total thickness in the order of 5 m. The Soil Profile Type at these locations is classified as Type 1, which, according to Table 4.4.6.1 of the CHBDC is, associated with a Site Coefficient (also referred to as ground motion amplification factor) of 1.0.

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

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

16.2 Liquefaction Potential

The structure foundations will be founded on bedrock, and the cut will be formed through compact to dense sand and gravel fill, stiff clayey silt fill, or stiff silty clay and silty clay till. There is negligible potential for liquefaction of these foundation soils. The rock has no potential for liquefaction.

The embankments themselves will be constructed above the groundwater table and are not considered to be in danger of liquefaction. Some toe failure may occur but it is expected to be of limited nature and readily repairable.

16.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause C4.6.4 of the CHBDC 2000 retaining structures should be designed using active (K_{AE}) and passive earth pressure (K_{PE}) coefficients that include earthquake loading. The rigid frame structure should be designed for at-rest pressure (K_{OE}). The following design parameters were used to calculate the seismic earth pressures:

ϕ = angle of internal friction of backfill

δ = the angle of friction between the wall and the backfill.

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

Wall Condition	Height of Application From Base as Percentage of Wall Height	Earth Pressure Coefficient (K) for Earthquake Loading					
		Granular A or Granular B Type II $\phi = 35°, \delta = 17.5°$		OPSS Granular B Type I $\phi = 32°, \delta = 16°$		Rock Fill $\phi = 42°, \delta = 21°$	
		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

17 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 founding bedrock surface under foundations due to blasting or other excavation procedures.
- Unwatering of the permanent cut and footing excavations to allow construction to be carried out in the dry.
- Stability of the earth cuts during and after construction.
- Inspection of the rock cuts by a rock mechanics specialist is also required to confirm short and long term stability.
- Excavating, dislodging, handling and disposal of boulders.

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Review Principal, Designated MTO Contact

TABLE 1
Baskin Drive Overpass (EBL)
Point Load Test Results

Depth feet	Depth Inches	Depth m	Is50	UCS (MPa)
BAS-14				
14	8	4.47	4.17	100.07
15	7	4.75	3.73	89.54
17	6	5.33	4.28	102.70
18	6	5.64	5.05	121.14
20	8	6.30	4.61	110.60
22	0	6.71	4.17	100.07
23	6	7.16	4.08	97.96
24	10	7.57	3.56	85.32

Average	Minimum	Maximum	
101	85	121	MPa

Depth feet	Depth Inches	Depth m	Is50	UCS (MPa)
BAS-16				
5	10	1.78	4.65	111.66
7	1	2.16	5.14	123.24
8	4	2.54	5.62	134.83
9	10	3.00	4.61	110.60
11	3	3.43	4.43	106.39
14	0	4.27	6.45	154.84
14	9	4.50	6.01	144.31

Average	Minimum	Maximum	
127	106	155	MPa

Depth feet	Depth Inches	Depth m	Is50	UCS (MPa)
BAS-19				
18	10	5.74	3.86	92.70
20	9	6.32	5.00	120.08
22	3	6.78	5.88	141.15
24	3	7.39	4.78	114.82
26	0	7.92	5.05	121.14

Average	Minimum	Maximum	
118	93	141	MPa

Depth feet	Depth Inches	Depth m	Is50	UCS (MPa)
BAS-21				
8	5	2.57	4.83	115.87
10	1	3.07	5.00	120.08
11	7	3.53	3.51	84.27
13	5	4.09	5.79	139.04
14	4	4.37	3.69	88.48
15	8	4.78	4.61	110.60

Average	Minimum	Maximum	
110	84	139	MPa

Appendix A

Record of Borehole Logs

RECORD OF BOREHOLE No BAS-14

METRIC

G.W.P. 647-92-00 | LOCATION N 5 031 152.9 E 315 520.9 (Baskin Drive EBL) | ORIGINATED BY SL

HWY HWY 17 | BOREHOLE TYPE Hollow Stem Augers, NQ Coring | COMPILED BY SS

DATUM Geodetic | DATE 10.11.03 - 10.11.03 | CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
106.2 / 0.0	**SAND and GRAVEL** Compact, Dark Brown, Moist	1	SS	29		
105.5 / 0.7	**SAND**, trace silt, Compact, Brown, Moist (FILL)	2	SS	24		
	trace gravel	3	SS	13		
104.2 / 2.0	Silty **CLAY**, trace sand, Stiff, Grey, Moist (CI)	4	SS	11		0 5 48 47
		5	SS	9		
101.9 / 4.3	AUGER REFUSAL AT 4.27m. **CRYSTALLINE LIMESTONE (BEDROCK** Fresh, slightly weathered at joints, very thinly to thinly bedded, whitish grey with black banding, strong to very strong. Subvertical joint at 4.6m to 4.9m			FI		
		1	RUN	0, 4		RUN 1# TCR=86%, SCR=86%, RQD=73%, UCS=94.8MPa
		2	RUN	0, 0, 1, 0, 2		RUN 2# TCR=100%, SCR=100%, RQD=95%, UCS=111.5MPa
		3	RUN	1, 0, 0, 0		RUN 3# TCR=100%, SCR=100%, RQD=100%, UCS=94.5MPa
98.5 / 7.7	END OF BOREHOLE AT 7.72m.					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (106, 105, 104, 103, 102, 101, 100, 99); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³.

$+^3, \times^3$: Numbers refer to Sensitivity; 15-○-5 (20, 10) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No BAS-15

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 142.9 E 315 526.8 (Baskin Drive EBL) ORIGINATED BY JL

HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 16.12.03 - 16.12.03 CHECKED BY SKP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m3	GR	SA	SI	CL
106.2												
106.1 / 0.1	**ASPHALT** (100mm)											
	SAND, trace to some gravel, trace to some silt, Brown to Grey, Moist to Wet (FILL)	1	GS			106	○					
		1	SS	37		105	○		13	75	12 (SI+CL)	
		2	SS	25			○					
104.2 / 2.0	Silty **CLAY**, some sand, trace gravel, trace rootlets, Very Stiff, Greenish Grey, Moist (FILL)						○					
103.9 / 2.3						104						

END OF BOREHOLE AT 2.29m.
AUGER REFUSAL AT 2.29m ON PROBABLE BEDROCK OR BOULDERS.
BOREHOLE DRY AND OPEN TO 0.41m UPON COMPLETION.
BOREHOLE BACKFILLED WITH DRILL CUTTINGS AND PATCHED WITH ASPHALT AT SURFACE.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No BAS-16

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 131.2 E 315 534.2 (Baskin Drive EBL) ORIGINATED BY SL

HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS

DATUM Geodetic DATE 11.11.03 - 11.11.03 CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
104.8							
104.6 / 0.2	TOPSOIL (150mm)						
104.1 / 0.7	Clayey SILT, some sand, trace gravel Stiff Brown Moist (FILL)	1	SS	10		o	
103.0 / 1.8	Silty CLAY, with sand, trace gravel Very Stiff Brown Moist (TILL)(ML/CL-ML) becoming sandy	2	SS	28	104	o	2 48 38 12
		3	SS	50/.127		o	
		1	RUN	1			RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=120.0MPa
100.1 / 4.7	AUGER REFUSAL AT 1.8m. CRYSTALLINE LIMESTONE (BEDROCK) Fresh to slightly weathered, very thinly to thinly bedded, whitish grey with wavy black banding, very strong	2	RUN	2, 0, 2, 2	103, 102		RUN 2# TCR=97%, SCR=97%, RQD=97%, UCS=135.2MPa
	Subvertical joint at 4.6m to 4.7m	3	RUN	0, 2, 3, 4, 2	101		
				FI			

END OF BOREHOLE AT 4.72m.
BOREHOLE BACKFILLED WITH DRILL CUTTINGS.
Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.

WATER LEVEL READINGS:

DATE	ELEVATION (m)
18/12/03	104.0
05/02/04	103.5

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3

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

RECORD OF BOREHOLE No BAS-17

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 133.8 E 315 538.1 (Baskin Drive EBL) ORIGINATED BY SL

HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 11.11.03 - 11.11.03 CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
105.4											
105.0 / 0.0	TOPSOIL (50mm)										
0.1	SAND, some gravel, some asphalt pieces Loose Brown Moist (FILL)		1	SS	7		105		o		
			2	SS	50/ .05				o		
104.2 / 1.2	END OF BOREHOLE AT 1.17m. AUGER REFUSAL AT 1.17m ON PROBABLE BEDROCK OR BOULDERS.										

Plot legend: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; Dynamic cone scale 20 40 60 80 100; Shear strength scale 20 40 60 80 100; Plastic limit w_P, Natural moisture content w, Liquid limit w_L; Water content scale 20 40 60.

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

Ministry of Transportation
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THURBER

RECORD OF BOREHOLE No BAS-18

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 155.4 E 315 529.3 (Baskin Drive EBL) ORIGINATED BY SL

HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 10.11.03 - 10.11.03 CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
105.9 / 0.0	**SAND and GRAVEL** Compact Brown Moist	1	SS	29		
105.2 / 0.7	**SAND**, trace to some silt Compact to Loose Brown Wet (FILL)	2	SS	16	105	
		3	SS	7		
103.9 / 2.0	Silty **CLAY**, trace sand Stiff Brown Moist (CL)	4	SS	11	104	
	becoming firm (CI)	5	SS	7	103	0 4 54 42
					102	
100.8 / 5.1	some sand seams	6	SS	7	101	
	END OF BOREHOLE AT 5.13m. AUGER REFUSAL AT 5.13m ON PROBABLE BEDROCK OR BOULDERS.					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3

+^{3}, ×^{3}: Numbers refer to Sensitivity   20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No BAS-19

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 157.7 E 315 532.3 (Baskin Drive EBL) ORIGINATED BY SL

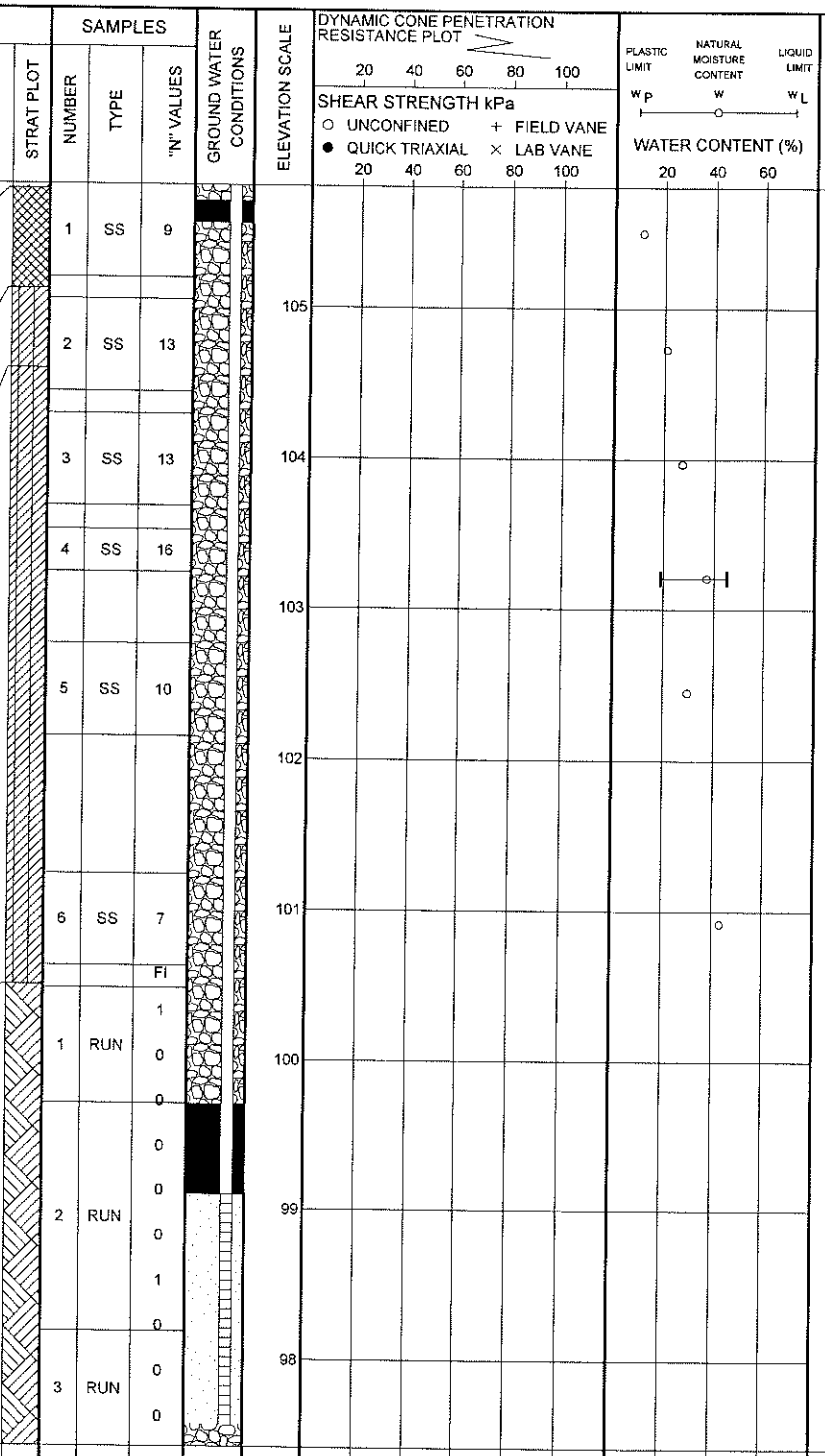
HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS

DATUM Geodetic DATE 10.11.03 - 10.11.03 CHECKED BY SKP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
105.8 / 0.0	**TOPSOIL** (25mm) **SAND and GRAVEL** Loose Brown Moist (FILL)	1	SS	9	
105.1 / 0.7	Silty **CLAY**, with topsoil, trace rootlets Stiff Brown Moist	2	SS	13	
104.6 / 1.2	Silty **CLAY**, trace sand Stiff Grey Moist to Wet (CI)	3	SS	13	
		4	SS	16	
	trace sand seams				0 5 46 50
		5	SS	10	
	becoming firm	6	SS	7	
100.5 / 5.3	AUGER REFUSAL AT 5.30m **CRYSTALLINE LIMESTONE (BEDROCK)** Fresh to slightly weathered, very thinly to thinly bedded, whitish grey with subvertical black banding, strong to very strong			FI	
		1	RUN	1, 0, 0	RUN 1# TCR=100%, SCR=100%, RQD=100%
		2	RUN	0, 0, 0, 1, 0	RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=118MPa
97.4 / 8.4		3	RUN	0, 0	RUN 3# TCR=93%, SCR=93%, RQD=93%, UCS=118MPa

END OF BOREHOLE AT 8.38m.
Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.
WATER LEVEL READINGS:

DATE	ELEVATION (m)
18/12/03	104.1
05/02/04	103.5
11/03/04	frozen



$+^3, \times^3$: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No BAS-21

METRIC

G.W.P. 647-92-00 | LOCATION N 5 031 137.0 E 315 544.6 (Baskin Drive EBL) | ORIGINATED BY JL

HWY HWY 17 | BOREHOLE TYPE Hollow Stem Augers, NQ Coring | COMPILED BY SS

DATUM Geodetic | DATE 16.12.03 - 16.12.03 | CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
106.1 / 0.0	**ASPHALT** (100mm)				106	
0.1	Silty **SAND**, some gravel, Brown (FILL)	1	GS			
105.5 / 0.6	**SAND**, trace to some gravel, trace to some silt, Compact, Brown to Grey, Wet (FILL)	1	SS	25	105	
		2	SS	28		11 79 11 (SI+CL)
104.1 / 2.0	Silty **CLAY**, trace sand, Stiff, Grey, Moist to Wet (CI)	3	SS	50/.05	104	
103.8 / 2.3	SPOON SAMPLER REFUSAL AT 2.34m. **CRYSTALLINE LIMESTONE (BEDROCK)** Fresh, slightly weathered at joints, very thinly to thinly bedded, grey, white with black subvertical banding, very strong	1	RUN	1, 1, 2, 2, 4	103	RUN 1# TCR=100%, SCR=97%, RQD=80%, UCS=106.7MPa
		2	RUN	1, 1, 2, 1, 3	102, 101	RUN 2# TCR=100%, SCR=98%, RQD=84%, UCS=112.7MPa
100.8 / 5.3	END OF BOREHOLE AT 5.28m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS Standpipe was apparently broken shortly after installation. No reading was taken.			FI		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity; 15 20 5 10 (%) STRAIN AT FAILURE

ONTMT4 7450BAS.GPJ 11/06/04

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No BAS-22

METRIC

G.W.P. 647-92-00	LOCATION N 5 031 137.8 E 315 549.3 (Baskin Drive EBL)		ORIGINATED BY JL
HWY HWY 17	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY SS
DATUM Geodetic	DATE 16.12.03 - 16.12.03		CHECKED BY SKP

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
106.0								
105.9 / 0.1	**ASPHALT** (100mm)							
	SAND, some gravel, Compact, Brown, Moist to Wet (FILL)	1	GS					
		1	SS	30				
104.7 / 1.3	Silty **CLAY**, trace sand, some sand seams, Very Stiff to Stiff, Grey, Moist to Wet (CI)	2	SS	15				
		3	SS	11				
	becoming firm	4	SS	7	0	4	44	53
		5	SS	5				
		6	SS	7				
100.0 / 6.0	END OF BOREHOLE AT 6.02m. AUGER REFUSAL AT 6.02m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE DRY AND OPEN TO 5.33m ON COMPLETION. BOREHOLE BACKFILLED WITH DRILL CUTTINGS AND PATCHED WITH ASPHALT AT SURFACE.							

Elevation scale: 106, 105, 104, 103, 102, 101, 100

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Appendix B

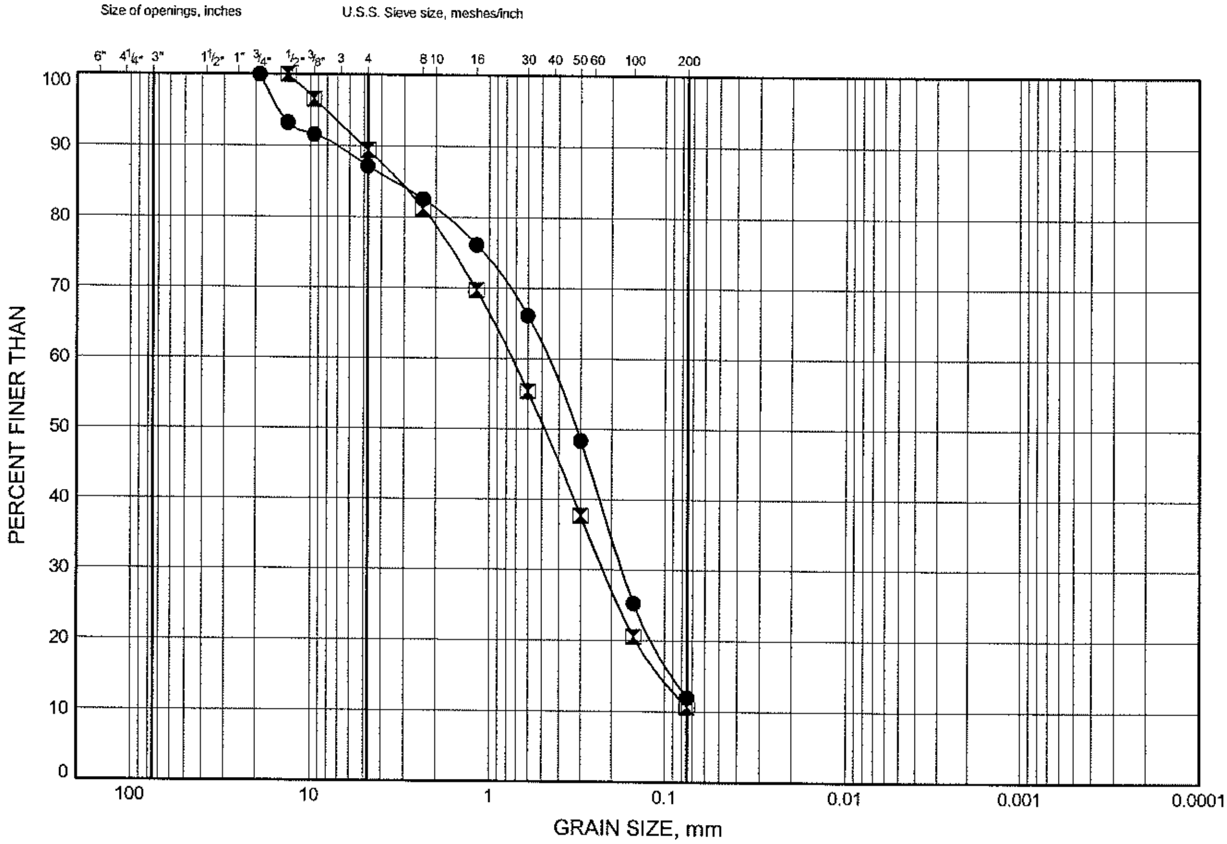
Laboratory Test Results

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)
●	BAS-15	1.07	105.13
⧗	BAS-21	1.74	104.36

Add sand and gravel sample from BAS 05-16 (change title)

THURBGSD 7450BAS.GPJ 07/06/04

Date June 2004

Project 647-92-00

THURBER

Prep'd SS

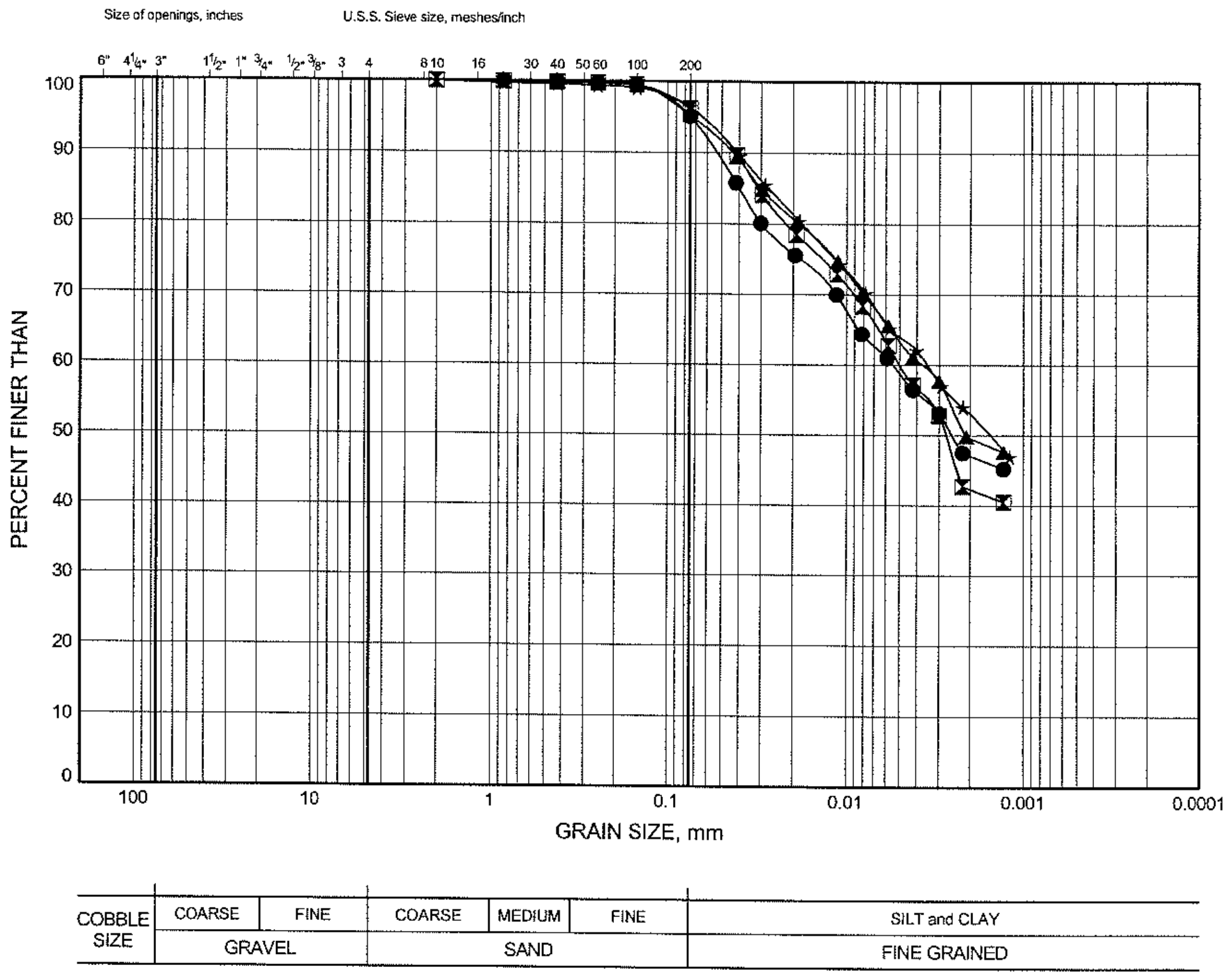
Chkd. SP

HWY 17 Twinning, Arnprior to Renfrew

GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BAS-14	2.59	103.61
⧗	BAS-18	3.35	102.55
▲	BAS-19	2.59	103.21
★	BAS-22	3.35	102.65

THURBGSD 7450BAS.GPJ 07/06/04

Date June 2004

Project 647-92-00

THURBER

Prep'd SS

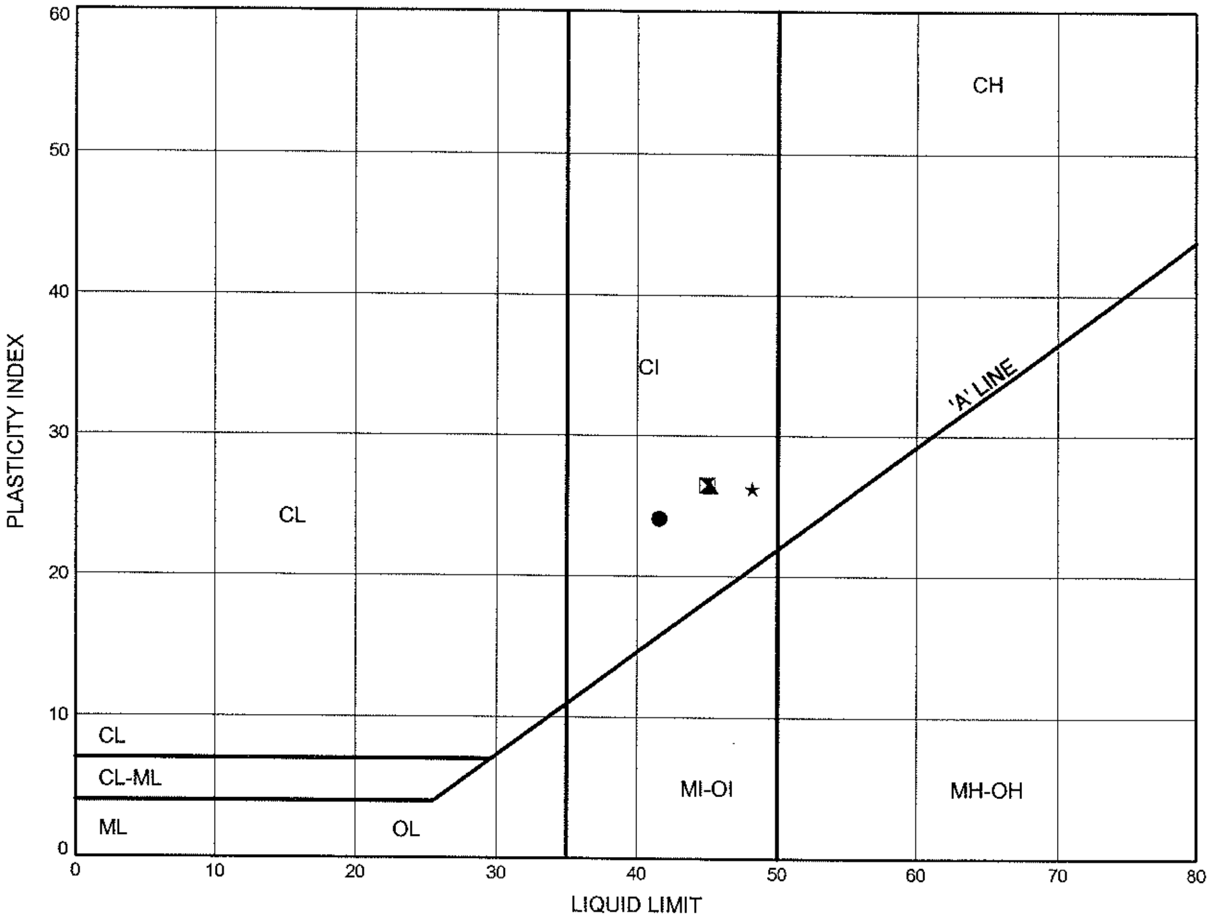
Chkd. SP

HWY 17 Twinning, Arnprior to Renfrew

ATTERBERG LIMITS TEST RESULTS

FIGURE B3

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BAS-14	2.59	103.61
⧗	BAS-18	3.35	102.55
▲	BAS-19	2.59	103.21
★	BAS-22	3.35	102.65

THURBALT 7450BAS.GPJ 07/06/04

Date June 2004

Project 647-92-00

THURBER

Prep'd SS

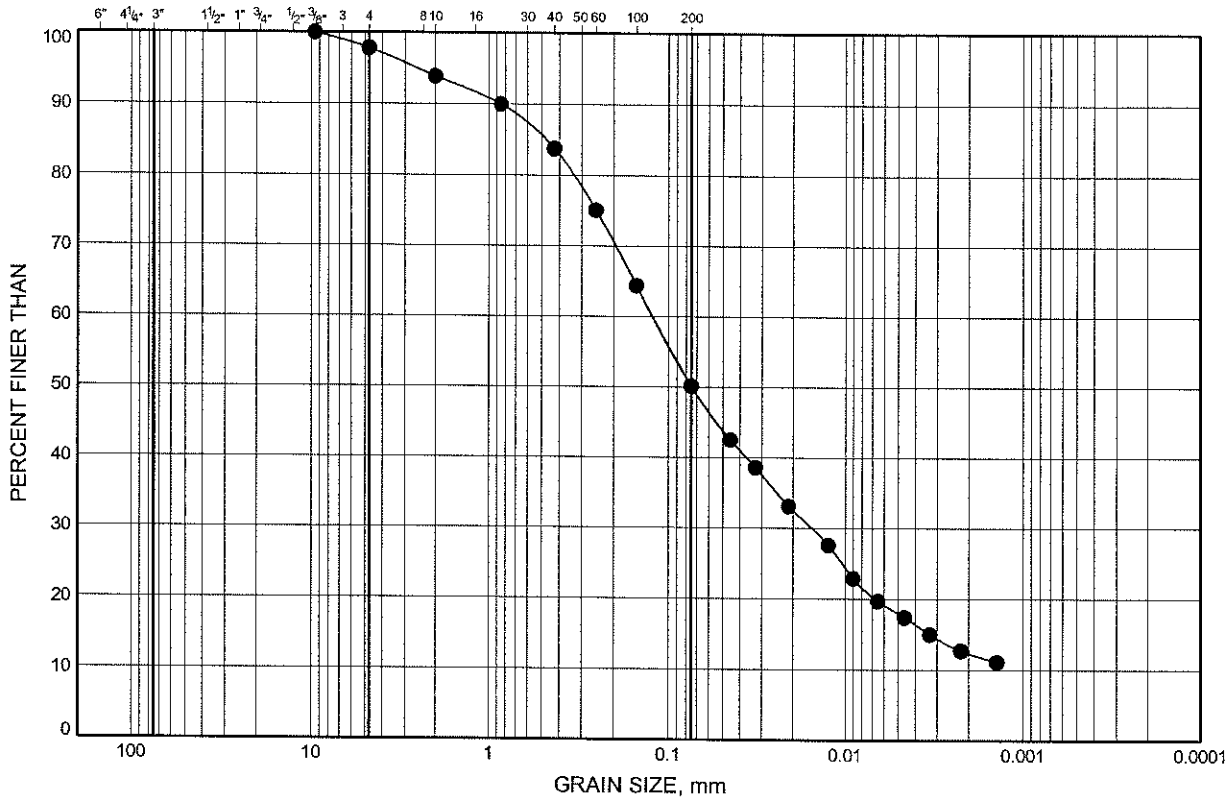
Chkd. SP

HWY 17 Twinning, Arnprior to Renfrew

GRAIN SIZE DISTRIBUTION

FIGURE B4

SILTY CLAY TILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BAS-16	1.07	103.73

change to B6

Date June 2004

Project 647-92-00

THURBER

Prep'd SS

Chkd. SP

THURBGSD 7450BAS.GPJ 07/06/04

Appendix C

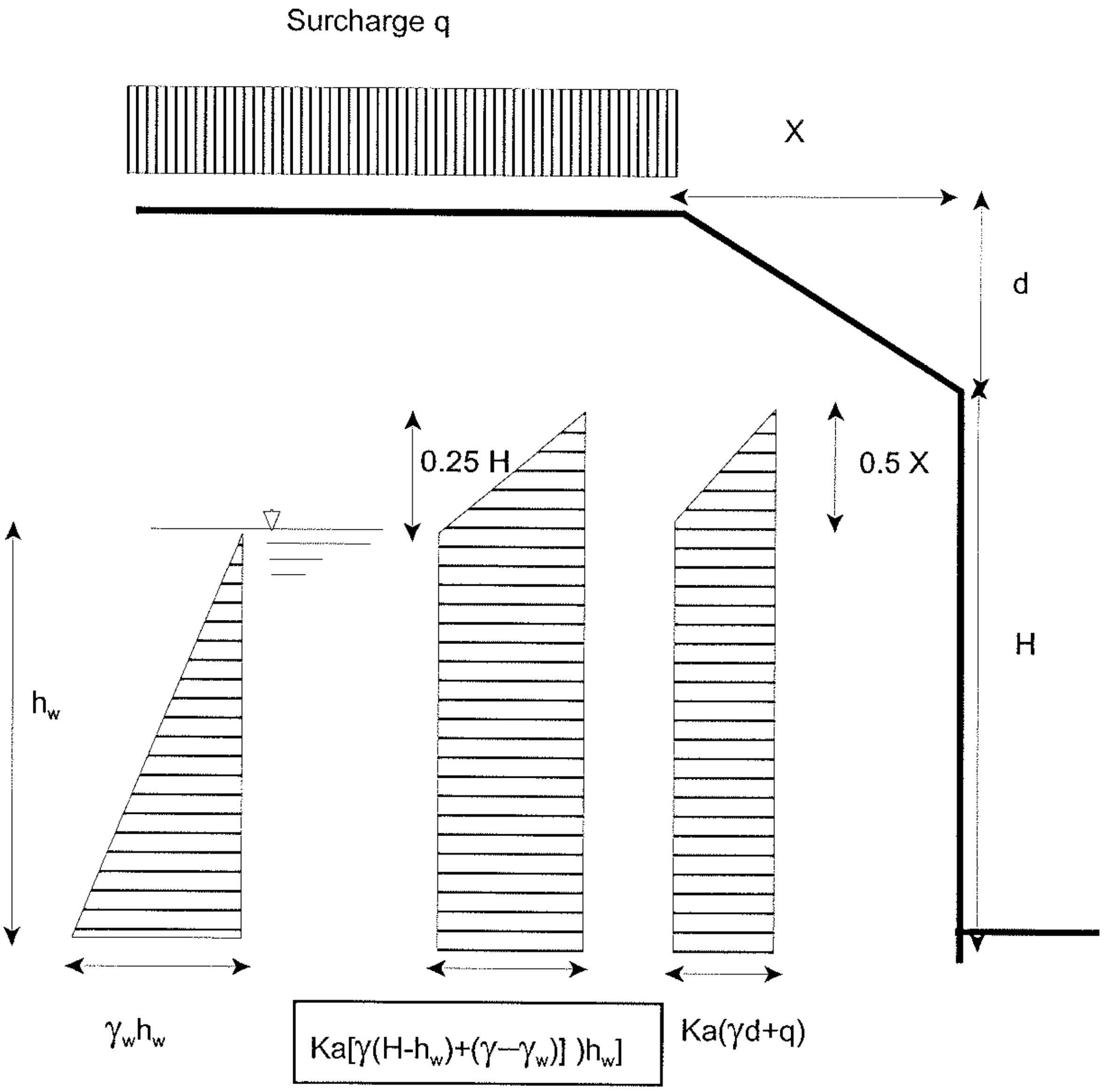
Foundation Alternatives

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	Driven Piles	Footing on Bedrock	Footing on Engineered Fill	Caisson
East and West Abutments	***Advantages:*** None identified ***Disadvantages:*** i. Proximity of bedrock surface below the grade of the Baskin Drive cut rendering the use of driven piles unnecessary and impractical	***Advantages:*** i. Shallow bedrock surface at or below the grade of the Baskin Drive cut ii. High values of geotechnical resistance are available on the bedrock iii. Allows footing to be placed close to the edge of Baskin Drive ***Disadvantages:*** i. High cost of excavation, if any is required ii. Mass concrete fill required to create a level founding surface	***Advantages:*** None identified ***Disadvantages:*** i. Insufficient height between top of bedrock and proposed highway grade. ii. Lower geotechnical resistance than bedrock iii. Footing has to be located further back from the edge of the cut to accommodate forward slope of engineered fill	***Advantages:*** None identified ***Disadvantages:*** i. Proximity of bedrock surface below the grade of the Baskin Drive cut rendering the use of caissons unnecessary and impractical

Appendix D

Figures



γ = unit weight of clay

γ_w = unit weight of water

Ka = earth pressure coefficient

THURBER

LATERAL PRESSURE DISTRIBUTION FOR BRACED SHORING DESIGN

FIGURE D1

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 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 of rock fill compaction. The equipment t5 sed shall be described in the submission. Where applicable, thèn t 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 65Rock 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, excnpt 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 maxiumum 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 l chink the top surface of the embankment using rock material not exceeding 100 mm in size by conventional blading techniques.

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

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

AMENDMENT TO OPSS 120, AUGUST, 1994

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 The 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.

Appendix F

Drawing

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

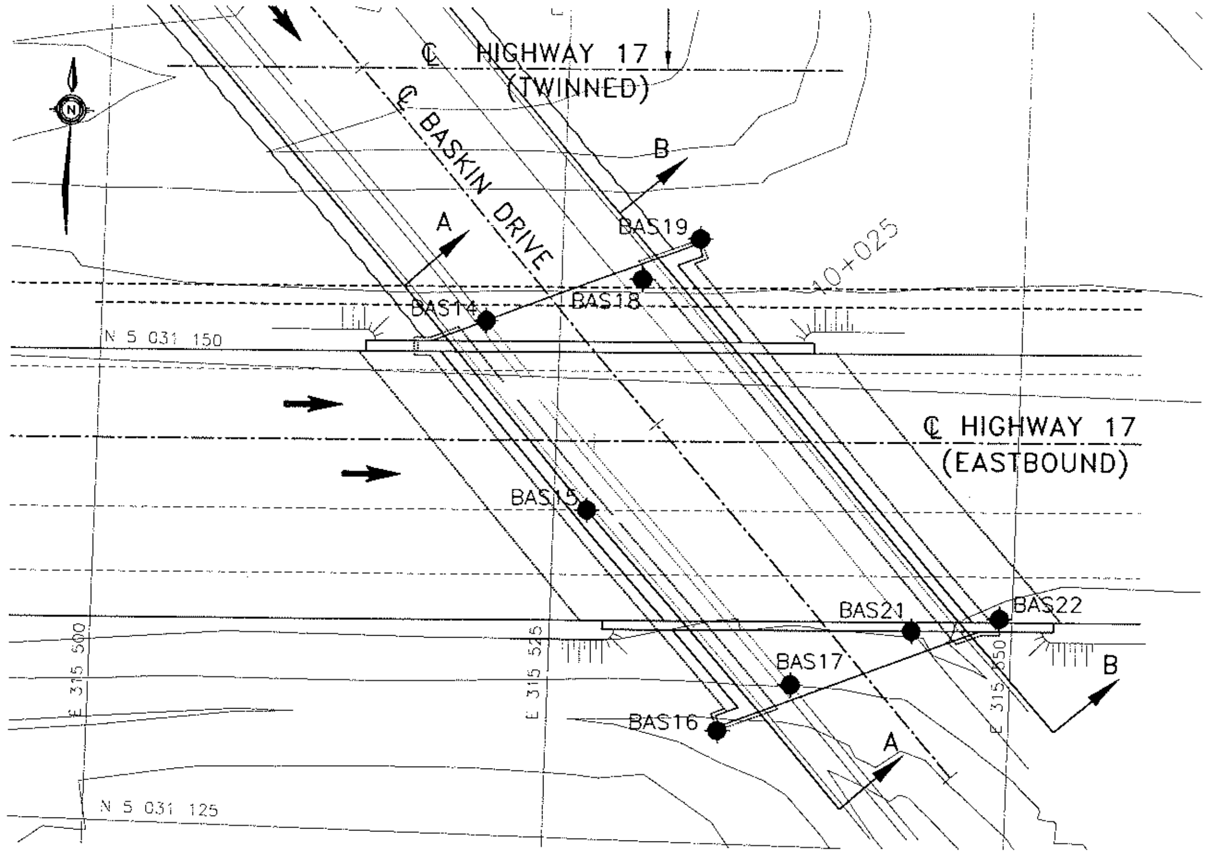
HWY.17
GWP NO. 647-92-00

HIGHWAY 17 TWINNING
BASKIN DRIVE OVERPASS
(EASTBOUND LANES)
BOREHOLE LOCATIONS AND SOIL STRATA

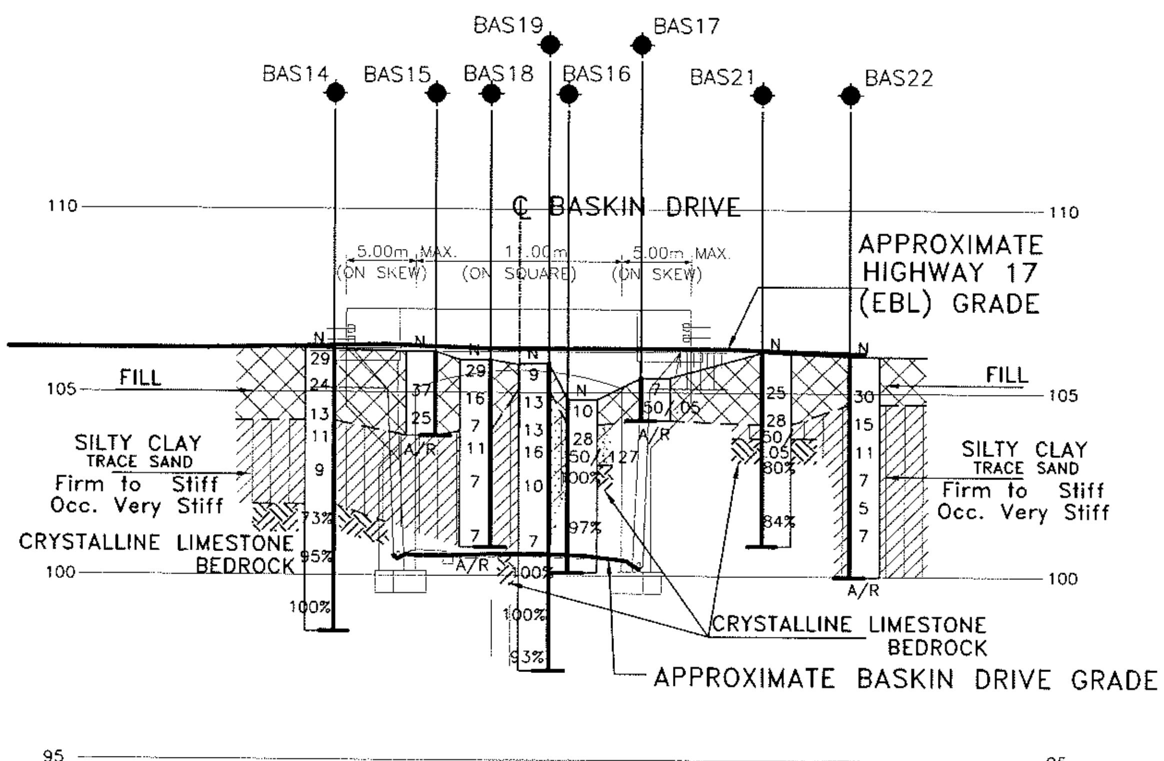
SHEET

TSH engineers architects planners

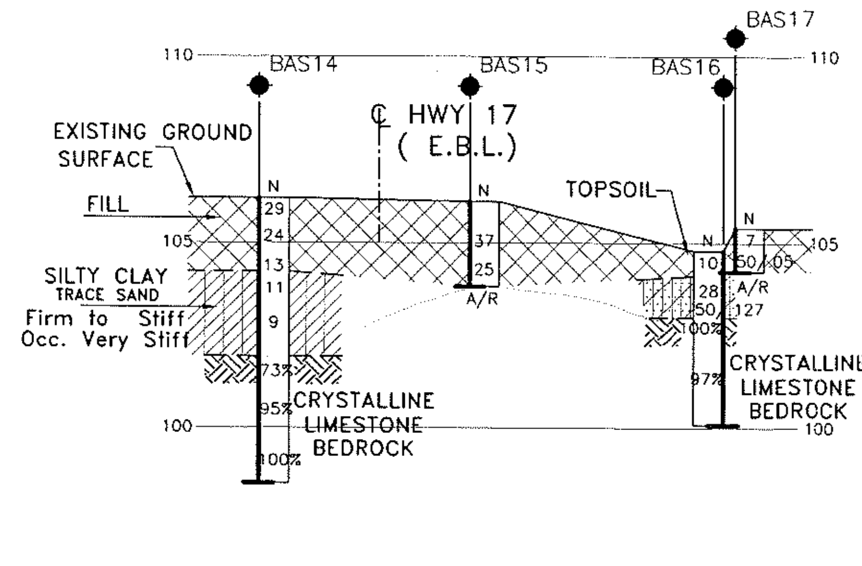
THURBER ENGINEERING LTD.



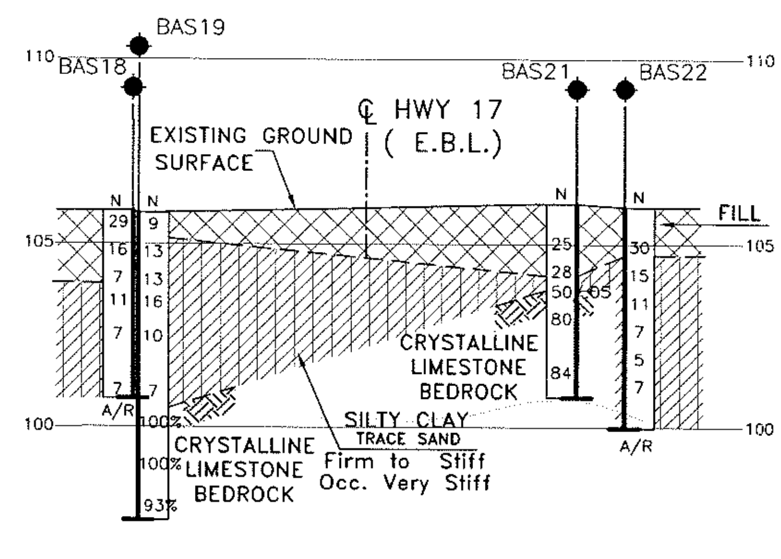
PLAN



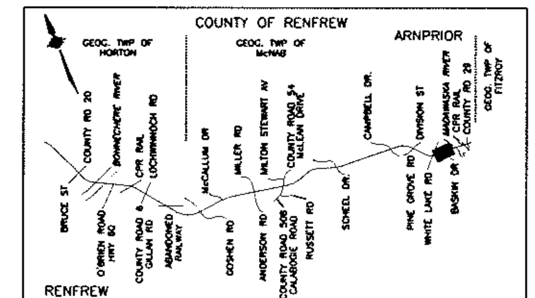
PROFILE ℄ HWY 17



SECTION A-A



SECTION B-B



KEYPLAN

LEGEND

●	Bore Hole
⊕	Dynamic Cone Penetration Test (cone)
◆	Bore Hole & Cone
N	Blows/ 0.3m (Std Pen Test, 475 J/blow)
CONE	Blows/ 0.3m (60° Cone, 475 J/blow)
PH	Pressure, Hydraulic
▼	WL at Time of Investigation
⊼	Head Artesian Water
⊥	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
BAS14	106.2	5031152.9	315520.9
BAS15	106.2	5031142.9	315526.8
BAS16	104.8	5031131.2	315534.2
BAS17	105.4	5031133.8	315538.1
BAS18	105.9	5031155.4	315529.3
BAS19	105.8	5031157.7	315532.3
BAS21	106.1	5031137.0	315544.6
BAS22	106.0	5031137.8	315549.3

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

LICENSED PROFESSIONAL ENGINEER
S. PANG
Aug. 20/04
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
Aug 20/04
PROVINCE OF ONTARIO

REVISIONS

DATE	BY	DESCRIPTION
AUG. 04	SP	FINAL
JUNE 04	SP	ISSUED AS DRAFT FOR REVIEW

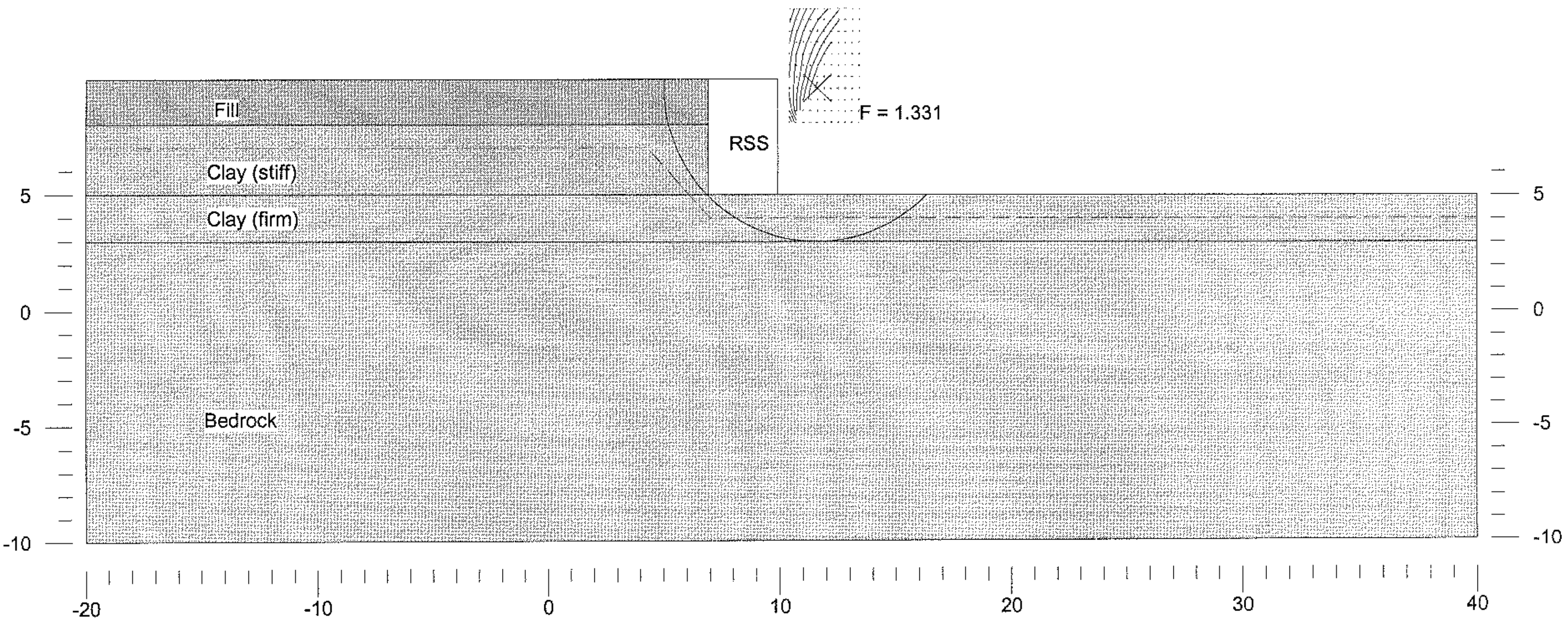
DESIGN SP	CHK PKC	CHBDC 2000	LOAD	DATE AUG.2004
DRAWN SS	CHK SP	SITE 29-423/1	STRUCT	DWG.

Appendix G

Selected Stability Analyses Results

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
RSS Wall	21	500	0	0	0
Fill	20	0	30	0	0
Clay (stiff)	20	0	29	0	1
Clay (firm)	19	0	27	0	1
Bedrock	(Infinitely Strong)				

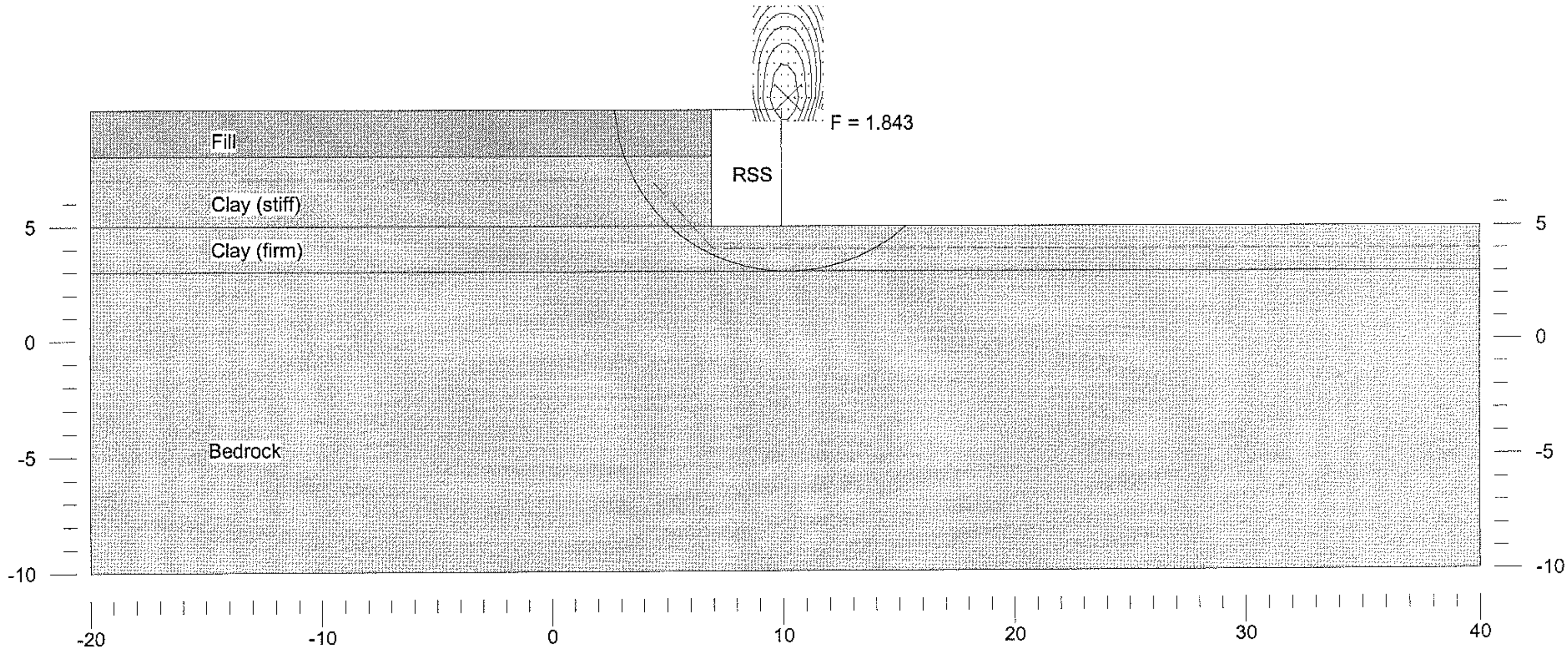
Thurber Engineering Ltd. - Toronto
19-3745-0
Highway 17 Twinning
August 17, 2004
Global Stability of RSS Wall (Retained height of 5 m) Baskin Drive EBL
Figure G1 Drained Analysis



18/08/2004 9:41:02 AM C:\SPANG\THURBE~8\19-374~1\BASKIN~3\BASFIG1.GSL Thurber Engineering Ltd. - Toronto F = 1.331

Thurber Engineering Ltd. - Toronto
19-3745-0
Highway 17 Twinning
August 17, 2004
Global Stability of RSS Wall (Retained height of 5 m) Baskin Drive EBL
Figure G2 Undrained Analysis

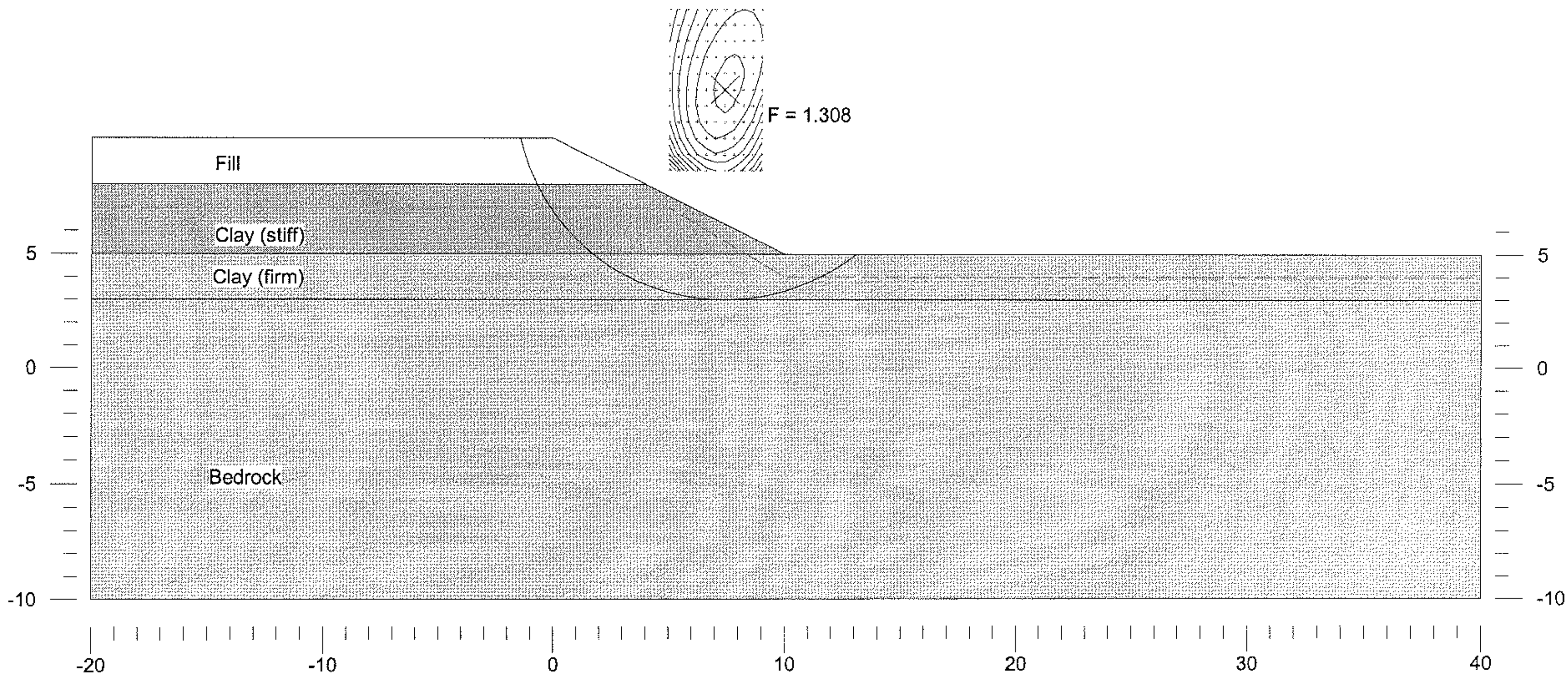
	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
RSS Wall	21	500	0	0	0
Fill	20	0	30	0	0
Clay (stiff)	20	60	0	0	1
Clay (firm)	19	30	0	0	1
Bedrock	(Infinitely Strong)				



18/08/2004 9:41:24 AM C:\SPANG\THURBE~8\19-374~1\BASKIN~3\BASFIG2.GSL Thurber Engineering Ltd. - Toronto F = 1.843

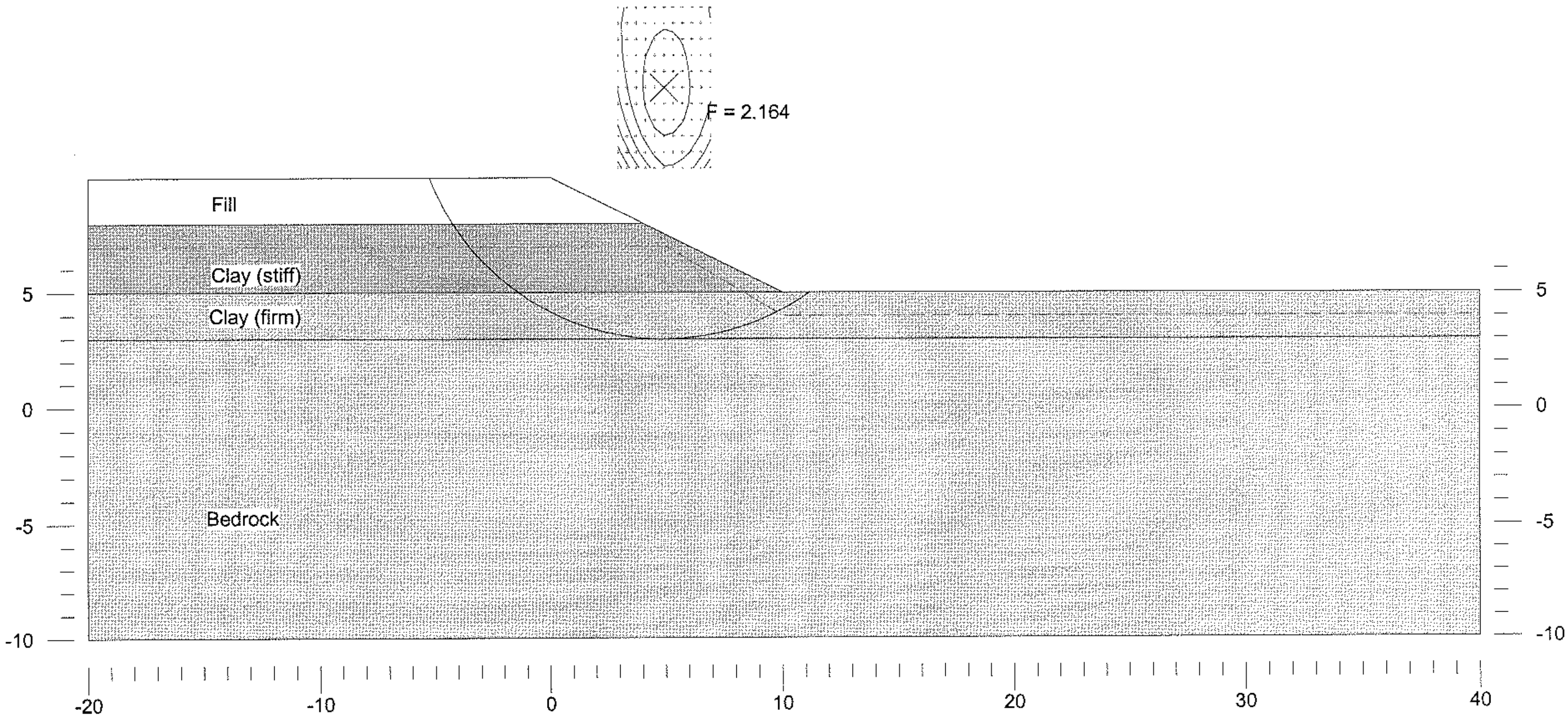
	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Fill	20	0	30	0	0
Clay (stiff)	20	0	29	0	0
Clay (firm)	19	0	27	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-3745-0
Highway 17 Twinning
August 17, 2004
Stability of Permanent Cut (2H : 1V) Baskin Drive EBL
Figure G3 Drained Analysis



	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Fill	20	0	30	0	0
Clay (stiff)	20	60	0	0	0
Clay (firm)	19	30	0	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-3745-0
Highway 17 Twinning
August 17, 2004
Stability of Permanent Cut (2H : 1V) Baskin Drive EBL
Figure G4 Undrained Analysis



18/08/2004 9:41:52 AM C:\SPANG\THURBE~8\19-374~1\BASKIN~3\BASFIG3.GSL Thurber Engineering Ltd. - Toronto F = 2.164