

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
BASKIN DRIVE OVERPASS (WESTBOUND LANES)  
HIGHWAY 17/417 TWINNING  
ARNPRIOR, ONTARIO  
G.W.P. 647-92-00, SITE NO. 29-423/2  
GEOCRES Number: 31F-140 125**

**Report to**

**McCormick Rankin Corporation**

Thurber Engineering Ltd.  
2010 Winston Park Drive, Suite 103  
Oakville, Ontario  
L6H 5R7  
Phone: (905) 829 8666  
Fax: (905) 829 1166

March 9, 2006  
File: 19-1351-82

## TABLE OF CONTENTS

SECTION	PAGE
<b>PART 1      FACTUAL INFORMATION</b>	
1    INTRODUCTION .....	1
2    SITE DESCRIPTION .....	2
3    SITE INVESTIGATION AND FIELD TESTING .....	2
4    LABORATORY TESTING.....	3
5    DESCRIPTION OF SUBSURFACE CONDITIONS .....	3
5.1    General .....	3
5.2    Topsoil.....	4
5.3    Fill .....	4
5.4    Silty Clay.....	4
5.5    Sand, Silty Sand and Sandy Silt .....	5
5.6    Boulders and Cobbles.....	5
5.7    Bedrock .....	5
5.8    Water Levels .....	7
<b>PART 2      ENGINEERING DISCUSSION AND RECOMMENDATIONS</b>	
6    INTRODUCTION .....	9
7    STRUCTURE FOUNDATIONS.....	9
7.1    General .....	9
7.2    Foundation Alternatives .....	10
7.3    Spread Footings on Bedrock .....	11
7.3.1    General.....	11
7.3.2    Bearing Resistance on Bedrock .....	12
7.3.3    Horizontal Resistance on Bedrock.....	12
7.3.4    Frost Cover .....	12
8    CANTILEVERED RETAINING WALLS.....	13
8.1    General .....	13
8.2    Spread Footings on Bedrock .....	13
9    RETAINED SOIL SYSTEMS.....	13
9.1    Foundations .....	14
9.2    Global Stability .....	15

9.3	Internal Stability .....	16
9.4	Settlement .....	16
10	PERMANENT CUT .....	16
10.1	General .....	16
10.2	Earth Cuts .....	16
10.3	Rock Cuts .....	17
11	EXCAVATION AND BACKFILL .....	18
11.1	General .....	18
11.2	Foundations .....	18
11.3	Earth Excavation .....	18
11.4	Rock Excavation .....	19
12	GROUNDWATER CONTROL .....	19
13	ROAD EMBANKMENTS .....	20
14	BACKFILL TO ABUTMENTS .....	20
15	EARTH PRESSURE .....	21
16	SEISMIC CONSIDERATIONS .....	22
16.1	Seismic Design Parameters .....	22
16.2	Liquefaction Potential .....	23
16.3	Retaining Wall Dynamic Earth Pressures .....	23
17	CONSTRUCTION CONCERNS .....	24

## Tables

Tables 1 and 2                      Point Load Test Results

## Appendices

Appendix A	Record of Borehole Logs
Appendix B	Laboratory Test Results
Appendix C	Foundation Alternatives
Appendix D	Figures
Appendix E	Special Provisions
Appendix F	Drawings titled "Borehole Locations and Soil Strata"

**FOUNDATION INVESTIGATION AND DESIGN REPORT**  
**BASKIN DRIVE OVERPASS (WESTBOUND LANES)**  
**HIGHWAY 17/417 TWINNING**  
**ARNPRIOR, ONTARIO**  
**G.W.P. 647-92-00, SITE NO. 29-423/2**  
**GEOCRES Number: 31F-140**

**PART 1: FACTUAL INFORMATION**

**1 INTRODUCTION**

This report presents the factual findings obtained from a foundation investigation at the Baskin Drive Overpass (Westbound) structure that will carry the realigned Baskin Drive under the Highway 17 westbound 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 the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan and soil strata drawing with a stratigraphic profile and cross-sections, records of boreholes and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the present investigation.

Thurber carried out the initial investigation as a sub-consultant to National Capital Engineering (NCE), under the Ministry of Transportation Ontario (MTO) Agreement Number 4005-A-000157. A final report (Reference 2 below) has been issued in September 2004. In 2005, Thurber was retained by McCormick Rankin Corporation (MRC) to carry out a supplementary investigation at this site primarily to better define the top of rock along the abutment footing alignments and at the RSS wingwall locations. The present report incorporates all available subsurface information to date and supersedes the previous report.

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).
- Thurber report titled "Foundation Investigation and Design Report, Baskin Drive Overpass (Westbound Lanes), Highway 17 Twinning, Arnprior to Renfrew, Ontario", G.W.P. 647-92-00, Site No. 29-423/2, GEOCRES No. 31F-140, dated September 20, 2004 (Reference 2).

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

## **2 SITE DESCRIPTION**

The site is located at the intersection of Highway 17 and Baskin Drive in the Township of McNab, County of Renfrew, Ontario (approximate Mainline Station 29+890). 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. Regional drainage in the area is largely governed by the Madawaska River to the east.

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

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

The initial site investigation and field testing for this project was carried out between November 6 and November 11, 2003. The field investigation consisted of drilling and sampling a total of eight boreholes to depths ranging from 4.5 m to 11.6 m. The boreholes were numbered BAS-1, BAS-2, BAS-4, BAS-5, BAS-8, BAS-9, BAS-11 and BAS-12. Boreholes BAS-3, BAS-6, BAS-7 and BAS-10 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 Baskin Drive Overpass – Highway 17 EBL structure. The results for the EBL structure are presented in a separate report. The supplementary investigation was carried out on December 6 to 9, 2005 when eight additional boreholes, numbered BAS 05-1 to 05-8, were drilled and sampled to depths of 6.8m to 11.8 m.

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

Downing Estate Drilling Limited of Calumet, Quebec (Downing) supplied a CME 55 track-mounted drill rig and conducted the initial drilling, sampling and in-situ testing operation. Downing also supplied a track-mounted CME-55 drill rig to advance the boreholes for the supplementary investigation. Hollow stem auger, casing and tricone drilling techniques were used



to advance the boreholes. Overburden samples were obtained using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Field vane shear tests were conducted within the cohesive soils, where appropriate, using an MTO 'N' size vane. Boreholes BAS-2, BAS-5, BAS-8, BAS-11, BAS 05-1 to 05-8, inclusive were generally advanced 3 m into bedrock by diamond coring.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers were installed in Boreholes BAS-5 and BAS-8 to permit longer term groundwater level monitoring. The installation details are presented on the Records of Boreholes in Appendix A and summarized as follows. At this site, 19 mm diameter Schedule 40 PVC pipes with 1.2 m to 1.5 m long slotted screens were installed at the bottom of the open boreholes. The sand screens surrounding the pipe were in the order of 2 m long. The remaining space in the borehole was backfilled with auger cuttings after a bentonite holeplug seal was placed on top of the screen.

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

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

On completion of drilling and sampling, the boreholes without piezometer installations were grouted to the surface with bentonite grout.

#### **4 LABORATORY TESTING**

All recovered soil samples were subjected to visual identification and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A. Selected samples were subjected to gradation analysis and Atterberg Limits tests were carried out on samples of the silty clay. The results of these tests are shown on the Record of Borehole sheets in Appendix A and on the plots in Appendix B.

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

#### **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 also presented on the "Borehole Locations and Soil Strata" drawings in Appendix F. A description of the stratigraphy is given in the

following paragraphs. The factual information at the borehole locations governs any interpretation of site conditions.

In general, the site was found to be underlain by 3.6 m to 8.8 m of topsoil, silty clay, sand, cobbles and boulders, overlying Ordovician Period limestone bedrock.

## **5.2 Topsoil**

Topsoil was encountered in all boreholes and varied in thickness from 100mm to 225mm. Topsoil thickness may vary between and beyond the borehole locations.

## **5.3 Fill**

Approximately 1.5 to 1.6 m of silty clay fill was encountered immediately below the topsoil at Boreholes BAS 05-3 and 05-7. This fill contains rootlets, occasional topsoil inclusions and some asphalt fragments. SPT 'N' values ranging between 2 and 9 blows per 0.3 m penetration indicate that the fill has a variable soft to stiff consistency. Measured moisture contents ranged between 25% and 35%.

## **5.4 Silty Clay**

Silty clay was encountered in all boreholes beneath the topsoil or fill. Its thickness varied between 3.1 m and 8.6 m. The level of the base of this silty clay deposit ranged from Elevations 97.6 m to 101.2 m.

The clay is generally brown to greyish-brown, is low to intermediate plastic, and very stiff to firm in consistency. The clay generally changes to grey in colour with depth. The SPT 'N' values generally range from 4 to 28 blows per 0.3 m penetration, and the moisture content typically varies from about 20% to 40%. Zones of soft to firm clay with 'N' values ranging from 2 to 4 blows per 0.3 m penetration were encountered in Boreholes BAS-1, BAS-5, BAS-12, BAS 05-3, 05-5, 05-6 and 05-8. A field vane shear test was done at a depth of 7.2 m in BAS-1, and at a depth of 6.5 m in BAS 05-6. The measured vane shear strength was about 38 kPa and 80 kPa in Boreholes BAS-1 and BAS 05-6, respectively, with a sensitivity of about 3 at both locations.

Figures B1, B2, B7 and B8 show the grain size distributions of several silty clay samples. These tests indicate that the clay content of these samples ranges between 38% and 49%. Figures B4, B10 and B11 present plasticity charts to show that the silty clay samples had measured liquid limits of between 38% and 52%, and plasticity indices of between 22% and 25%, indicating an intermediate to high plasticity (group symbol of CI to CH). Figures B3 shows the grain size distributions of another two silty clay samples with clay content of 33% and 34%. Figure B5 is a plasticity chart showing that these two silty clay samples had measured liquid limits of less than 35% and corresponding plasticity indices of less than 20%, indicating a low plasticity (group symbol of CL). These test results confirm that the

plasticity is a function of the clay content and is anticipated to be lower as the clay content decreases.

A 2.5 m thick clayey silt layer was encountered below the topsoil in Borehole BAS-1. Another clayey silt layer, about 0.4 m thick, was encountered immediately above bedrock in Borehole BAS 05-2.

A thin sand layer was encountered in the clay deposit at the West Abutment in Boreholes BAS-1 and BAS-4. This layer was from 0.4m to 0.5m thick. The level at which the sand layer was encountered ranged from Elevations 103.6m to 104.7m.

### **5.5 Sand, Silty Sand and Sandy Silt**

At the West Abutment, sand, silty sand or sandy silt was encountered beneath the clay and above the bedrock surface in Boreholes BAS-2, BAS-4 and BAS-5. This deposit was encountered in Borehole BAS-12 at the east approach. Sand and sandy silt layers were also encountered in Boreholes BAS 05-4 and 05-8, respectively. These soil layers varied in thickness from 0.3 m to 1.5 m, and were generally wet. The elevations of the base of these layers ranged from 97.6 m to 101.1 m. The sands and silts are generally compact to dense with SPT 'N' values ranging from 19 blows per 0.3 m penetration to more than 50 blows for less than 0.3 m penetration, and the moisture content varies from about 10% to 30%.

Although not directly encountered in the boreholes, these sand and silt layers may contain cobbles and possibly boulders.

Figure B6 shows the grain size distribution of a sample of the silty sand.

### **5.6 Boulders and Cobbles**

Auger refusal was met at a depth of 3.9m in Borehole BAS-8 near the northeast of the east abutment, and diamond coring was carried out. Limestone boulders were encountered in the initial 2m of core runs. In the next 1.7m of core run, no recovery was obtained. This indicates that the soil matrix was washed out during the coring process and observations indicated that the material is probably silty clay.

A cobble 100 mm in size was encountered immediately below the silty clay in Borehole BAS 05-2. The presence of cobbles and possibly boulders in glacially derived soils is to be expected.

### **5.7 Bedrock**

The soils described above were found to be underlain by crystalline limestone bedrock of the Ordovician Period. The bedrock was proven by coring in Boreholes BAS-2, BAS-5, BAS-8, BAS-11, and BAS 05-1 to 05-8. The bedrock surface was inferred from refusal to auger penetration in other boreholes drilled at this site.



Borehole	Ground Surface Elevation (m)	Bedrock Surface	
		Depth (m)	Elevation (m)
BAS-1	106.2	8.4	97.8
BAS-2	106.2	6.3*	99.9*
BAS-4	106.3	6.4	100.0
BAS-5	106.2	8.4*	97.8*
BAS-8	105.1	7.6*	97.5*
BAS-9	105.5	4.5	101.0
BAS-11	105.2	4.9*	100.3*
BAS-12	104.6	6.4	98.2
BAS 05-1	106.1	6.1*	100.0*
BAS 05-2	106.2	5.6*	100.6*
BAS 05-3	105.7	4.9*	100.8*
BAS 05-4	104.7	3.6*	101.1*
BAS 05-5	106.5	8.4*	98.1*
BAS 05-6	106.3	8.7*	97.6*
BAS 05-7	105.8	5.8*	100.0*
BAS 05-8	104.7	6.8*	97.9*

Note: \* Proven by coring.

The bedrock surface was inferred from refusal to auger penetration in other boreholes drilled at this site. However, as noted above, refusal on boulders occurred at Borehole BAS-8 and it is possible that boulders may be present at other locations where auger refusal was met.

Total Core Recovery (TCR) in the bedrock generally varied from 95% to 100%, except in Borehole BAS 05-6 where TCR values of 71% to 89% were recorded. The RQD values were typically between 50% and 100% indicating fair to excellent rock quality, except at the following locations:

- RQD of 43% (poor quality) for Run #1 in Borehole BAS 05-4.
- RQD of 42% (poor quality) for Run #1 in Borehole BAS 05-5.
- RQD of 36% (poor quality) for Run #1 in Borehole BAS 05-6.
- RQD of 25% (poor quality) for Run #2 in Borehole BAS 05-6.

The rock is generally fresh to slightly weathered except in Boreholes BAS 05-2 and 05-6 where it is moderately weathered. It is very thinly to thinly bedded with sub-vertical jointing. Sub-vertical banding was also noted in most of the rock core.

In most borehole locations, the Fracture Index (FI) of the rock, expressed as the frequency of natural fractures per 0.3 m of core, was generally low (<3), with occasional values greater than 5. In Borehole BAS 05-6, the measured FI values ranged from 6 to greater than 10 in some zones.

The condition of the joints ranged from planar to uneven and were generally rough though some smooth, planar joints were noted. Calcite infilling and occasional brown (iron oxide) staining was evident in many fractures.

Strength values of the intact rock estimated from Point Load Tests in selected rock cores ranged from 66 MPa to 179 MPa, with most values in the range of 80 MPa to 110 MPa. The rock is generally described as strong to very strong. A summary of the Point Load Test results is presented in Table 1.

## 5.8 Water Levels

The groundwater levels observed in the open boreholes at the completion of drilling ranged from about 1.9 m in BAS-9 to 4.6 m in BAS-3.

The groundwater level observed at the standpipe piezometers is summarized in the table below and also shown on Drawing "Borehole Locations and Soil Strata".

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
BAS-5	December 18, 2003	2.5	103.7
	February 5, 2004	3.0	103.2
BAS-8	December 18, 2003	1.3	103.8
	February 5, 2004	1.8	103.3

The groundwater level is at approximately 2 to 3 m depth below the existing ground surface at Boreholes BAS-5 and BAS-8. Based on site observations and information from Reference 1, it is likely that there is downward drainage into bedrock or into the sandy layers immediately above bedrock.

All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe climatic events. It is also anticipated that the local groundwater conditions at this site is largely governed by the nearby Madawaska River.



Engineering Analysis and Report Preparation by:  
Sydney Pang, P.Eng.  
Associate, Senior Project Engineer



Report Reviewed by:  
P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact

**FOUNDATION INVESTIGATION AND DESIGN REPORT**  
**BASKIN DRIVE OVERPASS (WESTBOUND LANES)**  
**HIGHWAY 17/417 TWINNING**  
**ARNPRIOR, ONTARIO**  
**G.W.P. 647-92-00, SITE NO. 29-423/2**  
**GEOCRES Number: 31F-140**

**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**6 INTRODUCTION**

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

It is understood that the preliminary design plan calls for the construction of a structure to carry the future Highway 17 Westbound Lanes (WBL) over the vertically realigned Baskin Drive that will be constructed in a cut.

The proposed single span, reinforced concrete rigid framed overpass structure will have an approximate length of 22m (along the Baskin Drive alignment) and an approximate clear span width of 11 m (perpendicular to the Baskin Drive alignment). The abutments will be skewed at about 50° to the Highway 17 WBL centreline.

It is understood that the proposed Highway 17 Twinning WBL mainline will be at the existing intersection grade of approximate Elevation 106 m. The alignment for Baskin Drive will be in a cut with a proposed final grade at approximate Elevation 101 m, or 5 m depth below the final highway grade.

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 on each side of Baskin Drive.

The stratigraphy encountered at the site of the proposed bridge typically consists of about 3.6 m to 8.8 m of topsoil, silty clay to occasional clayey silt, overlying sand and silt layers



with cobbles and boulders at some locations. The overburden is underlain by crystalline limestone bedrock.

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

Approximate Location	Borehole Number	Elevations (m)	
		Existing Ground Surface	Top of Bedrock
East Abutment (north to south)			
Northeast Corner	BAS-8	105.1	97.5**
Northeast Centre	BAS 05-3	105.7	100.8**
Centre	BAS-9	105.5	101.0*
Southeast Centre	BAS 05-7	105.8	100.0**
Southeast Corner	BAS-11	105.2	100.3**
West Abutment (north to south)			
Northwest Corner	BAS-2	106.2	99.9**
Northwest Centre	BAS 05-2	106.2	100.6**
Centre	BAS-4	106.3	100.0*
Southwest Centre	BAS 05-6	106.3	97.6**
Southwest Corner	BAS-5	106.2	97.8**

\* Auger Refusal

\*\* Proven by coring.

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

Location	Borehole Number	Elevations (m)	
		Existing Ground Surface	Top of Bedrock
Northwest Wingwall	BAS-2 (east)	106.2	99.9**
	BAS 05-1 (west)	106.1	100.0**
Northeast Wingwall	BAS 05-3 (west)	105.7	100.8**
	BAS 05-4 (east)	104.7	101.1**
Southwest Wingwall	BAS 05-6 (east)	106.3	97.6**
	BAS 05-5 (west)	106.5	98.1**
Southeast Wingwall	BAS-11 (west)	105.2	100.3**
	BAS 05-8 (east)	104.7	97.9**

\* Auger Refusal

\*\* Proven by coring.

The variation of the bedrock surface elevations along the foundation elements at the abutments and wingwalls are illustrated on the Borehole Locations and Strata Drawings.

## 7.2 Foundation Alternatives

This section presents discussions on feasible foundation alternatives, recommendations and foundation design parameters for feasible and/or preferred foundation options for this site.

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

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

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

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

An integral abutment design was not considered a feasible option at this site due to the skewness of the structure and the impracticality of using driven piles.

Footings on engineered fill is also not considered feasible at this site due to the presence of shallow bedrock below the proposed Baskin Drive grade, and the likelihood that the footings would have to be located further back from the cut resulting in a longer superstructure.

In view of the above 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 unnecessary 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 typically varies between Elevations 99.9 m and 100.6 m between the northwest corner and the centre, beyond which it dips down to Elevations 97.6 m and 97.8m.

#### *East Abutment*

The top of rock varies between Elevations 100.0 m and 101.0 m, except near the north limit where it dips to approximate Elevation 97.5 m.

### **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 poured 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 pier locations should be evaluated in accordance with the CHBDC, 2000 assuming an ultimate coefficient of friction of 0.85.

If the frictional component is insufficient to resist lateral forces, the horizontal resistance may be increased by dowelling 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**

It is understood that the proposed Baskin Drive will be constructed in open cut with side slopes at an inclination of 2H : 1V. Alternatively, retaining walls may be used along some sections on either side of the road. The required lengths of the retaining walls will depend on the configuration of the cut, road embankment and the bridge, as well as the length of the permanent open cut sections.

### **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 south limit of the WBL west abutment to the north limit of the EBL west abutment, the footings may be founded on bedrock assumed to be sloping up from approximate Elevations 97.5 m to 101 m (north to south).

Along the east side of Baskin Drive, from the south limit of the WBL east abutment to the north limit of the EBL east abutment, the footings may be founded on bedrock at an approximate Elevation 100.3 m.

There was no borehole drilled between and beyond the two proposed structure locations. If the retaining wall option is to be pursued, additional boreholes should be drilled along the retaining wall alignments 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 in 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 firm to stiff clays or bedrock. Where applicable, the native soil under the RSS foundation should be proof-rolled and be compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD) at  $\pm 2\%$  of its optimum moisture content. The engineered fill 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 at the following elevations:
  - Northwest Wingwall – Elevations 99.9 m to 100.0 m (east to west)
  - Northeast Wingwall – Elevations 100.8 m to 101.1 m (west to east)
  - Southwest Wingwall – Elevations 97.6 m to 98.1 m (east to west)
  - Southeast Wingwall – Elevations 100.3 m to 97.9 m (west to east)
- Factored geotechnical resistance at ULS of 250 kPa, and geotechnical resistance of 175 kPa at SLS on an engineered Granular A pad resting on the firm to stiff silty clay to occasional clayey silt.
- 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 main body of the RSS may be founded on native silty clay to clayey silt or on bedrock. All topsoil, organics, soft or otherwise disturbed soils should be removed from the subgrade of the RSS wall prior to placement of the fill.

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

- Factored geotechnical resistance at ULS of 5,000 kPa for walls founded directly on limestone bedrock (SLS is not applicable to foundation on rock).

- Factored geotechnical resistance at ULS of 225 kPa, and geotechnical resistance of 150 kPa at SLS, for walls founded on native silty clay to clayey silt.
- Ultimate coefficient of friction of between RSS mass and Granular A is 0.55.
- Ultimate coefficient of friction of between RSS mass and native clays is 0.45.
- Ultimate coefficient of friction of between RSS mass and sound bedrock is 0.6.

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

## **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, firm to stiff silty clay to clayey silt, 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 6 m (retained height) by 4.5 m (assumed length of RSS reinforcement, 75% of retained height) founded on native silty clay and sands overlying bedrock.
- Groundwater level at 1 m below the base of the cut.
- Vertically sided shoring to retain the existing native clay slopes.

Results of the analyses yield Factors of Safety greater than 1.5, which indicate that global stability can be maintained for the assumed RSS configuration. Global stability for an RSS wall founded directly on bedrock is not anticipated to be a design issue at this site.

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

### **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, settlements of RSS walls founded on well compacted engineered fill, or native, firm to stiff silty clay subgrade prepared as recommended in this report, are expected to be less than 25 mm and to occur essentially as the RSS is constructed.

## **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 about 5 m of typically stiff silty clay and about 1 m of sands and silts into 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 2 m to 3 m below the existing water table. Groundwater control (see later section) will be required prior to excavating below the groundwater level.

Unsupported earth cuts will be stable provided that the slope inclinations are not steeper than 2H : 1V.

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 firm to stiff silty clay, with occasional soft zones, overlying compact sand to sandy silt, cobbles and boulders, or

even bedrock at some locations. It is considered that basal stability can be maintained provided that the groundwater level is kept below the base of the cut.

The cut will be formed predominantly in the relatively impermeable silty clay. Some water seepage should be expected from water-bearing interlayers within the cohesive soils and exposed sands and silts, or cobbles and boulders near the base of the cut. 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.

Water-bearing sands and silts, where exposed, are prone to disturbance by construction traffic and the like. It is recommended that construction traffic be minimized on such 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.

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 OPSS 572 and related special provisions for more detailed requirements. Where continual seepage and surficial instability are evident, particularly in the sands and silts above bedrock, remedial measures including the use of gravel sheeting may be required.

### **10.3 Rock Cuts**

A rock cut in the order of 1 to 2 m high will be required along some sections of the north side of Baskin Drive. Rock cuts 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 stiff native silty clay is classified as a Type 2 soil. The sand layer between the silty clay and bedrock is classified as a Type 3 soil provided the groundwater level is maintained below the cut.

### **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 sand 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 D4 may be used for design in conjunction with the following parameter values.

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

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

$$\begin{aligned}
 P_p &= 6 c B L \\
 \text{Where } c &= 2,000 \text{ kPa (equivalent Mohr-Coulomb cohesion} \\
 &\text{based on Hoek and Brown rock mass classification)} \\
 L &= \text{depth of socket in rock, m}
 \end{aligned}$$

#### 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. Prior to excavating below the groundwater table, it is recommended that the groundwater level be temporarily lowered to at least 1 m below the base of the cut until such time that a permanent drainage system (associated with the Baskin Road construction) becomes operational. Moreover, the groundwater at the footing

excavations must be further lowered to the top of bedrock in order to allow footing construction to be carried out in the dry.

The design of the unwatering system should be the responsibility of the Contractor. However, suitable systems that might be employed at this site include vacuum well points or eductors for lowering of the groundwater level. These systems should be supplemented by pumping from filtered sumps, which may be used to control 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 have been estimated based on the methods described in the CHBDC, 2000 Commentary Section C6.6.3.6.

The approach fills for this structure will be constructed on stiff to very stiff clay overlying bedrock. These soils will satisfactorily support the low road embankment fills at this site, which are expected to be in order of 1 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 OPSS 572 and related special provision(s).

### **14 BACKFILL TO ABUTMENTS**

In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. A NSSP is required to specify grading limits for the rock fill. The rock fill used as backfill to the abutment should be limited to fragments with a maximum dimension of not greater than 300 mm and including adequate spalls to fill voids in the rock fill.

In all cases where the approach embankment consists of rock fill and granular backfill to the abutment wall is used, the granular backfill should consist of OPSS Granular "B" Type II.

The backfill to the abutment walls should be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill should be placed to the extents shown in OPSD 3501.000, and rock backfill should be placed to the extents shown in OPSD 3505.000.

All granular materials should meet the specifications of Special Provision 110F13 "Amendment to OPSS 1010, March 1993".

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

The design of the abutment should incorporate a subdrain as shown in OPSD 3501.000 or OPSD 3505.000, as applicable.

## 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:

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

where  $P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient (see table below)

$\gamma$  = unit weight of retained soil (typically 21 kN/m<sup>3</sup>)

$h$  = 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 values are shown in the following table.



Wall Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ, \gamma = 19.0 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*	0.2	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

\* For wing walls.

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

The factors in the table above are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the 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 very stiff to stiff clays overlying compact to dense sand with a total thickness generally less than 9m. The Soil Profile Type at these locations is classified as Type 1, which, according to Table 4.4.6.1 of the CHBDC is, associated with a Site Coefficient (also referred to as ground motion amplification factor) of 1.0.

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

Clause C4.6.4 of the CHBDC suggests that the value of  $k_h$  used in calculating the earth pressure coefficients for yielding structures is equivalent to 0.5 x Zonal Acceleration Ratio,  $A$ , (including the effects of site amplification), or 0.1g at this site, provided that allowance is made for an outward displacement of the abutment of up to 250A, or 50 mm. The vertical acceleration factor,  $k_v$ , has been taken as 0.6 times  $k_h$ . This design assumption is based on conventional practice in seismic design. For non-yielding walls, the recommended  $k_h$  design value according to CHBDC is equivalent to 1.5 x Zonal Acceleration Ratio, or 0.3g at this site. The Woods method adopted in this report is based on elastic theory resulting in seismic earth pressure coefficients comparable to those outlined in the CHBDC.

## 16.2 Liquefaction Potential

The structure foundations will bear on bedrock, and the cut will be formed through stiff to firm clay and dense to very dense sand. 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 on the stiff clay 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:

$\phi'$  = angle of internal friction of backfill

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

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

\*\* After Woods

## 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:

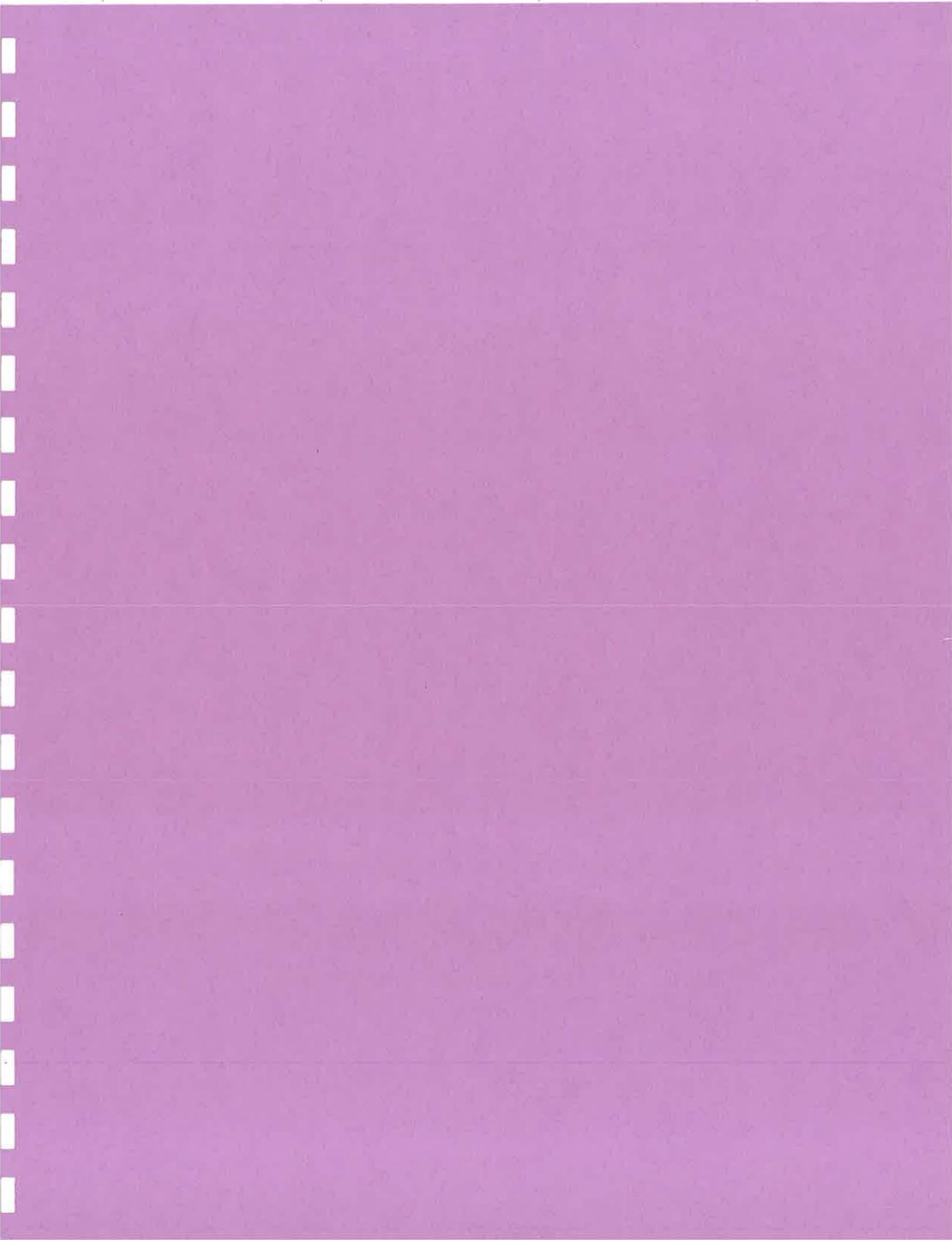
- Disturbance of the founding bedrock surface under foundations due to blasting or other excavation procedures.
- Excavating, dislodging, handling and disposing of boulders.
- Unwatering of the permanent cut and footing excavations to allow footing and other construction to be carried out in the dry.
- 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.



Engineering Analysis and Report Preparation by:  
Sydney Pang, P.Eng.  
Associate, Senior Project Engineer



Report Reviewed by:  
P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact



Point Load Test Results

**TABLE 1**  
Baskin Drive Overpass (WBL)  
Point Load Test Results

Depth			Is50	UCS (MPa)
feet	Inches	m		
BAS-2				
22	7	6.88	3.99	95.86
23	7	7.19	4.04	96.91
24	9	7.54	4.39	105.34
26	6	8.08	2.77	66.36
29	6	8.99	3.69	88.48
30	4	9.25	3.51	84.27

Run #	Average	
1	99.4	MPa
2	79.7	MPa

Depth			Is50	UCS (MPa)
feet	Inches	m		
BAS-5				
29	2	8.89	5.49	131.67
29	9	9.07	7.46	179.07
33	2	10.11	3.77	90.59
35	3.5	10.76	5.35	128.51
37	0	11.28	4.83	115.87

Run #	Average	
1	131.7	MPa
2	179.1	MPa
3	109.6	MPa
4	115.9	MPa

Depth			Is50	UCS (MPa)
feet	Inches	m		
BAS-8				
26	3	8.00	4.83	115.87
27	9	8.46	4.26	102.18
29	6	8.99	5.05	121.14
31	3	9.53	3.86	92.70
32	4	9.86	3.20	76.90
34	0	10.36	4.48	107.44
35	0	10.67	4.74	113.76

Run #	Average	
4	113.1	MPa
5	97.7	MPa

Runs 1 to 3 consist of boulders, cobbles  
or loose rock fragments

Depth			Is50	UCS (MPa)
feet	Inches	m		
BAS-11				
14	6	4.42	4.13	99.02
16	0	4.88	3.38	81.11
17	0	5.18	3.42	82.16
18	9	5.72	5.00	120.08
21	0	6.40	6.32	151.68

Run #	Average	
1	95.6	MPa
2	151.7	MPa

# Point Load Test Results

**TABLE 2**  
**Hwy 17 Baskin Drive (Westbound)**  
**Point Load Test Results**

feet	Depth		Is50	UCS (MPa)
	Inches	m		
BAS 05-1				
20	3	6.17	3.46	82.94
21	7	6.58	3.02	72.57
23	2	7.06	4.32	103.68
24	5	7.44	3.02	72.57
26	0	7.92	4.97	119.23
27	7	8.41	4.75	114.05
29	3	8.92	3.89	93.31

Total Rock Core			
Average	Minimum	Maximum	MPa
94	73	119	

Run #	Average
1	86.40
2	101.95
3	93.31

Depth			Is50	UCS (MPa)
feet	Inches	m		
BAS 05-2				
18	9	5.72	5.18	124.41
20	2	6.15	3.89	93.31
21	9	6.63	3.46	82.94
24	4	7.42	1.94	46.66
25	9	7.85	0.00	0.00
27	4	8.33	0.00	0.00

Total Rock Core			
Average	Minimum	Maximum	MPa
58	0	124	

Run #	Average
1	100.22
2	15.55

Depth			Is50	UCS (MPa)
feet	Inches	m		
BAS 05-3				
16	11	5.16	2.59	62.21
17	11	5.46	5.62	134.78
20	1	6.12	3.89	93.31
21	5	6.53	3.89	93.31
23	0	7.01	6.26	150.33
24	4	7.42	5.18	124.41
25	8	7.82	4.32	103.68
26	8	8.13	4.75	114.05

Total Rock Core			
Average	Minimum	Maximum	MPa
110	62	150	

Run #	Average
1	98.49
2	112.32
3	114.05

Point Load Test Results

**TABLE 2 (continued)**  
**Hwy 17 Baskin Drive (Westbound)**  
**Point Load Test Results**

Depth			Is50	UCS (MPa)
feet	Inches	m		
BAS 05-4				
12	7	3.84	7.99	191.80
14	6	4.42	4.32	103.68
16	2	4.93	1.30	31.10
17	3	5.26	2.81	67.39
19	5	5.92	4.75	114.05
20	9	6.32	3.67	88.13
21	10	6.65	2.16	51.84

Total Rock Core			
Average	Minimum	Maximum	
93	31	192	MPa

Run #	Average
1	191.80
2	67.39
3	84.67

Depth			Is50	UCS (MPa)
feet	Inches	m		
BAS 05-5				
27	9	8.46	3.46	82.94
29	10	9.09	4.97	119.23
31	4	9.55	7.34	176.25
32	7	9.93	0.43	10.37
25	1	7.65	3.89	93.31
26	5	8.05	7.34	176.25

Total Rock Core			
Average	Minimum	Maximum	
110	10	176	MPa

Run #	Average
1	82.94
2	101.95
3	134.78

Depth			Is50	UCS (MPa)
feet	Inches	m		
BAS 05-6				
30	1	9.17	5.62	134.78
34	2	10.41	3.89	93.31
34	9	10.59	2.59	62.21

Total Rock Core			
Average	Minimum	Maximum	
97	62	135	MPa

Run #	Average
1	134.78
2	77.76

feet	Depth		Is50	UCS (MPa)
	Inches	m		
BAS 05-7				
21	5	6.53	6.26	150.33
22	0	6.71	4.10	98.49
23	1	7.04	3.89	93.31
23	8	7.21	0.43	10.37
25	3	7.70	5.18	124.41
26	5	8.05	5.40	129.60

Total Rock Core			
Average	Minimum	Maximum	
101	10	150	MPa

Run #	Average
1	150.33
2	95.90
3	88.13



# Point Load Test Results

**TABLE 2 (continued)**  
**Hwy 17 Baskin Drive**  
**Point Load Test Results**

Depth			Is50	UCS (MPa)
feet	Inches	m		
BAS 05-8				
23	0	7.01	6.26	150.33
25	1	7.65	4.32	103.68
26	5	8.05	4.75	114.05
27	10	8.48	4.75	114.05
28	11	8.81	3.89	93.31
30	4	9.25	5.18	124.41
31	7	9.63	4.32	103.68

Total Rock Core			
Average	Minimum	Maximum	MPa
115	93	150	
Run #	Average		
1	150.33		
2	110.59		
3	107.13		

## **Appendix A**

### **Record of Borehole Logs**

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

### 1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

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

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

NOTE: Hierarchy of Soil Strength Prediction

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



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

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

### 5. LEGEND FOR RECORDS OF BOREHOLES

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

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

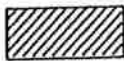

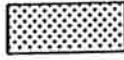


 Water Level  
 Shear Strength Determination by Pocket Penetrometer

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

# UNIFIED SOILS CLASSIFICATION

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

## EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION		SYMBOLS			
Fresh (FR)	No visible signs of weathering.				
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE		
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE		
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE		
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL		
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)		
DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength (MPa) (psi)	Field Estimation of Hardness*	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
TERMS		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.				
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

## METRIC

[illegible]

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

## METRIC

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

# RECORD OF BOREHOLE No BAS-4

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 184.5 E 315 489.1 (Baskin Drive) 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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	
106.3												
106.2	TOPSOIL (150mm)											
0.2	Silty CLAY, trace sand, trace rootlets Very Stiff to Stiff Brown Moist (CL)		1	SS	16		106					
			2	SS	7		105					
104.7												
1.6	SAND, trace gravel Compact Brown		3	SS	24		104					
104.2	Moist											
2.1	Silty CLAY, trace sand Stiff Brown to Grey Moist (CL)		4	SS	9		103					
			5	SS	10		102					
	some sand seams											
101.4			6	SS	10		101					
4.9	Silty SAND, some clay, trace gravel Compact to Very Dense Grey Wet (TILL-LIKE) (ML-nonplastic)											
			7	SS	50/ .102		100					
100.0												
6.4	END OF BOREHOLE AT 6.35m. AUGER REFUSAL AT 6.36m ON PROBABLE BEDROCK OR BOULDERS. BOREHOLE OPEN TO 4.7m. WATER LEVEL IN OPEN BOREHOLE AT 4.6m DEPTH UPON COMPLETION.											

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No BAS-5

1 OF 2

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 174.8 E 315 494.9 (Baskin Drive) 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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
106.2								20 40 60 80 100						
106.0	TOPSOIL (175mm)							20 40 60 80 100						
0.2	Silty CLAY, trace gravel, occasional rootlets Stiff Brown Moist (CL)		1	SS	10						○			
			2	SS	14						○			
104.5														
1.7	Silty CLAY, trace sand Stiff Brown Moist to Wet (CL)		3	SS	7						○			
			4	SS	15						○			
			5	SS	9						○			
	becoming firm													
			6	SS	7						○			
	becoming grey, wet, (CI)													
			7	SS	4									0 2 50 48
99.3														
6.9	Sandy SILT, trace gravel Compact Grey Wet (ML- nonplastic)		8	SS	21						○			
97.8					FI									
8.4	AUGER REFUSAL AT 8.38m. CRYSTALLINE LIMESTONE (BEDROCK) Slightly to moderately weathered, very thin to thin bedded, whitish grey with black banding, strong to very strong Subvertical joints (occasional calcite infilling) at 8.4m to 8.5m, 8.5m to 8.8m, 9.8m to 10.0m, 10.8m to 11.1m.		1	RUN	3									RUN 1# TCR=100%, SCR=100%, RQD=55%, UCS=131.7MPa
					2									
			2	RUN	1									RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=179.1MPa
					2									

RUN 1#  
TCR=100%,  
SCR=100%,  
RQD=55%,  
UCS=131.7MPa  
RUN 2#  
TCR=100%,  
SCR=100%,  
RQD=100%,  
UCS=179.1MPa

Continued Next Page

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

**METRIC**

CONCRETE	COMPUTER			DYNAMIC CONE PENETRATION			
----------	----------	--	--	--------------------------	--	--	--

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

## METRIC

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20  
15-⊙-5  
10

(%) STRAIN AT FAILURE

CONTMT4 7450BAS.GPJ 20/09/04

## METRIC

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

[illegible]

+ 3, x 3: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No BAS-9

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 186.9 E 315 503.3 (Baskin Drive) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 06.11.03 - 06.11.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
105.5								20 40 60 80 100						
0.0 105.3	TOPSOIL (225mm)													
0.2	Silty CLAY, trace sand Stiff to Very Stiff Brown Moist (CL)   some sand seams  becoming stiff (CI)		1	SS	11									
			2	SS	16									
			3	SS	15									
			4	SS	9									
			5	SS	12									
101.0														
4.5	END OF BOREHOLE AT 4.5m. AUGER REFUSAL AT 4.5m ON PROBABLE BEDROCK OR BOULDERS BOREHOLE OPEN TO 4.32m. WATER LEVEL IN OPEN BOREHOLE AT 1.86m DEPTH UPON COMPLETION.													

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

20  
15  
10  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BAS-11

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 181.6 E 315 510.2 (Baskin Drive) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SS  
 DATUM Geodetic DATE 07.11.03 - 07.11.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
105.2												
105.0	TOPSOIL (150mm)											
0.2	Silty CLAY, trace sand, trace rootlets Stiff to Very Stiff Brown Moist (CL-Cl)  some sand seams		1	SS	9		105					
			2	SS	22		104					
			3	SS	14		103					
			4	SS	13		102					
			5	SS	11		101					
100.3	occasional rock pieces and sand pockets		6	SS	53/ 254		100					
4.9	AUGER REFUSAL AT 4.88m. CRYSTALLINE LIMESTONE (BEDROCK) Fresh to slightly weathered, very thinly to thinly bedded, whitish grey with black banding, strong to very strong Subvertical joints at 5.1m, 7.0m, 7.3m and 7.5m.		1	RUN	0		99					RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=95.6MPa
			2	RUN	1		98					RUN 2# TCR=97%, SCR=97%, RQD=97%, UCS=151.7MPa
97.4												
7.6	END OF BOREHOLE AT 7.6m.				FI							




ONTMT4 7450BAS.GPJ 20/09/04

# RECORD OF BOREHOLE No BAS-12

1 OF 1

METRIC

G.W.P. 647-92-00 LOCATION N 5 031 186.5 E 315 521.7 (Baskin Drive) ORIGINATED BY SL  
 HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 06.11.03 - 06.11.03 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT  γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI C			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
								○ UNCONFINED + FIELD VANE							
								● QUICK TRIAXIAL × LAB VANE							
							20 40 60 80 100			PLASTIC LIMIT w <sub>p</sub> NATURAL MOISTURE CONTENT w LIQUID LIMIT w <sub>L</sub>					
							20 40 60 80 100			WATER CONTENT (%)					
104.6															
104.4	TOPSOIL (200mm)														
0.2	Silty CLAY, trace sand, trace rootlets Firm to Stiff Brown Moist (CI)		1	SS	7		104								
			2	SS	3										
			3	SS	10		103								
			4	SS	9		102								
	trace sand seams		5	SS	9		101								
100.6															
4.0	becoming soft		6	SS	3		100							0 9 58 3	
99.1															
5.5	SAND, some clay, occasional inferred cobbles and boulders Very dense Grey Wet (SC)		7	SS	50/ .127		99								
98.2															
6.4	END OF BOREHOLE AT 6.4m. AUGER REFUSAL AT 6.4m ON PROBABLE BEDROCK OR BOULDERS BOREHOLE OPEN TO 4.5m. WATER LEVEL IN OPEN BOREHOLE AT 2.62m DEPTH UPON COMPLETION.														

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

20  
15-5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BAS 05-1

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 193.98 E 315 468.43 (Baskin Drive WBL) ORIGINATED BY SLL  
 HWY 17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM  
 DATUM Geodetic DATE 08.12.05 - 08.12.05 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	
106.1												
0.0	TOPSOIL (100 mm)											
0.1	Silty CLAY, trace sand, some rootlets Stiff Brown		1	SS	13		106					
			2	SS	13		105					0 2 52 46
	some silt pockets, becoming grey						104					
			3	SS	10		103					
							102					0 8 57 35
			4	SS	9		101					
							100					
100.0			5	SS	14		99					
6.1	AUGER REFUSAL AT 6.12 m. CRYSTALLINE LIMESTONE (BEDROCK). Fresh to slightly weathered, very thinly to thinly bedded, whitish grey with black banding, strong to very strong Subvertical joints at 6.27, 6.55, 6.93 m		1	RUN	1		98					RUN 1# TCR=98%, SCR=98%, RQD=95%, UCS=86.4MPa
					2							
					0							RUN 2# TCR=100%, SCR=100%, RQD=98%, UCS=102.0MPa
					0							
	Subvertical joints at 7.80, 7.82, 7.85, 8.36 m		2	RIN	3							
					1							
					0							
97.0			3	RUN	0							RUN 3# TCR=100%, SCR=100%, RQD=100%, UCS=93.3MPa
9.1	END OF BOREHOLE AT 9.09 m. BOREHOLE GROUTED WITH BENTONITE TO GROUND SURFACE.											

ONTMT4S 5182-PHASE II.GPJ 07/03/06



# RECORD OF BOREHOLE No BAS 05-2

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 189.92 E 315 483.84 (Baskin Drive WBL) ORIGINATED BY SLL  
HWY 17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM  
DATUM Geodetic DATE 08.12.05 - 08.12.05 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
106.2 0.0	TOPSOIL: (100 mm)													
0.1	Silty CLAY, some rootlets, trace sand seams Stiff to Very Stiff Dark Brown to Brown		1	SS	12		106							
105.3 0.9	Firm		2	SS	5		105							
104.4 1.8	Becoming Grey						104							
			3	SS	16		103							
							102							
	trace gravel		4	SS	13		101							
101.1 5.2	COBBLE: (100 mm)						100							
100.6 5.6	Clayey SILT, trace sand, trace gravel Grey (till-like)				FI		99							
	AUGER REFUSAL AT 5.56 m. CORING STARTED AT 5.56 m. CRYSTALLINE LIMESTONE (BEDROCK). Slightly weathered to moderately weathered, very thinly to thinly bedded, whitish grey with black banding, medium strong to strong Subvertical joints at 5.97, 6.22, 6.27, 6.53, 6.76, 6.88 m with some calcite infilling		1	RUN	2		98							
			2	RUN	3									
97.6 8.6	END OF BOREHOLE AT 8.61 m. BOREHOLE GROUTED WITH BENTONITE TO GROUND SURFACE.													

RUN 1#  
TCR=98%,  
SCR=93%,  
RQD=73%,  
UCS=100.2MPa

RUN 2#  
TCR=100%,  
SCR=100%,  
RQD=75%,  
UCS=46.7MPa

# RECORD OF BOREHOLE No BAS 05-3

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 194.92 E 315 496.20 (Baskin Drive WBL) ORIGINATED BY SLL  
 HWY 17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM  
 DATUM Geodetic DATE 07.12.05 - 07.12.05 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
105.7														
0.0	TOPSOIL (175mm)													
0.2	Silty CLAY, some sand seams, some rootlets Soft Dark Brown to Brown (FILL)		1	SS	9		105							
			2	SS	2		104							
103.9														
1.8	Silty CLAY, trace sand seams Very Stiff to Stiff Grey						103							
	Inferred cobble at 2.5m depth		3	SS	24		102							
							101							
100.8														
4.9	AUGER REFUSAL AT 4.88 m. CORING STARTED AT 4.88 m. CRYSTALLINE LIMESTONE (BEDROCK), Fresh to slightly weathered, very thinly to thinly bedded, whitish grey with black banding, strong to very strong		1	RUN	1		100							
	Subvertical joints at 5.89, 5.96, 6.00, 6.35, 6.43, 6.61, 6.65, 6.83, 6.93, 7.03 m		2	RUN	3		99							
	Subvertical joints at 7.26, 7.57, 7.82, 7.75, 8.08 m		3	RUN	2		98							
97.5														
8.2	END OF BOREHOLE AT 8.23 m. BOREHOLE GROUTED WITH BENTONITE TO GROUND SURFACE.													

ONTMT4S 5182-PHASE II.GPJ 09/03/06

# RECORD OF BOREHOLE No BAS 05-4

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 193.93 E 315 504.78 (Baskin Drive WBL) ORIGINATED BY SLL  
 HWY 17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM  
 DATUM Geodetic DATE 06.12.05 - 06.12.05 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
104.7															
0.0	TOPSOIL (150mm)														
0.2	Silty CLAY, trace sand Very Stiff Brown		1	SS	17										
	some sand seams														
			2	SS	21										0 5 60 35
101.5															
3.3	Silty SAND, trace clay, trace gravel Compact		3	SS	19										
101.1	Grey (fill-like)				FI										
3.6	AUGER REFUSAL AT 3.63 m. CORING STARTED AT 3.63 m. CRYSTALLINE LIMESTONE (BEDROCK), Fresh to slightly weathered, very thinly to thinly bedded, whitish grey with black banding Subvertical joints at 3.73, 3.86, 3.91 m with occasional quartzite inclusions		1	RUN	4										RUN 1# TCR=95%, SCR=95%, RQD=43%, UCS=191.8MPa
					2										RUN 2# TCR=100%, SCR=100%, RQD=95%, UCS=67.4MPa
			2	RUN	2										
					1										
					2										
					1										
			3	RUN	0										RUN 3# TCR=100%, SCR=100%, RQD=98%, UCS=84.7MPa
					1										
97.9															
6.8	END OF BOREHOLE AT 6.78 m. BOREHOLE GROUTED WITH BENTONITE TO GROUND SURFACE.														

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

20  
15  
10

(%) STRAIN AT FAILURE

## METRIC


(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BAS 05-5

2 OF 2

METRIC

W.P. 647-92-00 LOCATION N 5 031 180.07 E 315 484.76 (Baskin Drive WBL) ORIGINATED BY SLL  
 HWY 17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM  
 DATUM Geodetic DATE 07.12.05 - 08.12.05 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
95.0			3	RUN	3											RUN 3# TCR=100%, SCR=100%, RQD=97%, UCS=134.8MPa	
	1																
	1																
	1																
	4																
11.4	END OF BOREHOLE AT 11.43 m. BOREHOLE GROUTED WITH BENTONITE TO GROUND SURFACE.																

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

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BAS 05-6

1 OF 2

METRIC

W.P. 647-92-00 LOCATION N 5 031 180.82 E 315 491.95 (Baskin Drive WBL) ORIGINATED BY SLL  
 HWY 17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM  
 DATUM Geodetic DATE 09.12.05 - 09.12.05 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N° VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
106.3												
0.0	TOPSOIL (150mm)		1	SS	50/							
0.2	Silty CLAY, trace sand seams Soft Brown				.050		106					
			2	SS	3		105					
104.5												
1.8	Very Stiff Grey						104					
			3	SS	20							0 2 52 46
102.7							103					
3.7	Becoming Stiff to Firm						102					
			4	SS	8		101					
							100					0 4 57 39
			5	SS	9		99					
							98					
			6	SS	6		97					
97.6	some gravel		7	SS	50/							
8.8	AUGER REFUSAL AT 8.76 m. CORING STARTED AT 8.76 m. CRYSTALLINE LIMESTONE (BEDROCK). Moderately weathered, very thinly to thinly bedded, whitish grey with black banding				.075 >10							
			1	RUN	6							RUN 1# TCR=71%, SCR=62%, RQD=36%, UCS=134.8MPa

Continued Next Page

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

20  
15  
10  
(%) STRAIN AT FAILURE

## METRIC

[illegible]

# RECORD OF BOREHOLE No BAS 05-7

1 OF 1

METRIC

W.P. 647-92-00 LOCATION N 5 031 184.10 E 315 503.92 (Baskin Drive WBL) ORIGINATED BY SLL  
 HWY 17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM  
 DATUM Geodetic DATE 06.12.05 - 07.12.05 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)					
								UNCONFINED					FIELD VANE				
								QUICK TRIAXIAL	LAB VANE								
105.8						20	40	60	80	100	20	40	60	GR	SA	SI	CL
0.0	TOPSOIL: (200mm)																
0.2	Silty CLAY, some rootlets, occasional topsoil staining, some asphalt fragments Stiff to Firm Brown (FILL)		1	SS	8												
			2	SS	7												
104.1																	
1.7	Silty CLAY, trace sand seams Very Stiff to Stiff Grey																
			3	SS	18												
			4	SS	8												
			5	SS	50/ FL25												
100.0																	
5.8	AUGER REFUSAL AT 5.84 m. CORING STARTED AT 5.84 m. CRYSTALLINE LIMESTONE (BEDROCK), Fresh to slightly weathered, very thinly to thinly bedded, whitish grey with black banding Subvertical jointing at 6.1 m		1	RUN	3 2												
			2	RUN	3 1												
					1 1												
			2	RUN	1 1 0												
97.2																	
8.6	END OF BOREHOLE AT 8.61 m. BOREHOLE GROUTED WITH BENTONITE TO GROUND SURFACE.																

0 4 58 38

RUN 1#  
TCR=100%,  
SCR=100%,  
RQD=74%,  
UCS=150.3MPa  
RUN 2#  
TCR=100%,  
SCR=100%,  
RQD=100%,  
UCS=95.9MPa  
RUN 2#  
TCR=100%,  
SCR=100%,  
RQD=100%,  
UCS=88.1MPa

ONTMT4S 5182-PHASE II.GPJ 07/03/06



# RECORD OF BOREHOLE No BAS 05-8

1 OF 2

METRIC

W.P. 647-92-00 LOCATION N 5 031 181.20 E 315 519.51 (Baskin Drive WBL) ORIGINATED BY SLL  
HWY 17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM  
DATUM Geodetic DATE 06.12.05 - 06.12.05 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	
104.7												
0.0	TOPSOIL (175mm)											
0.2	Silty CLAY, trace sand Stiff to Very Stiff Brown		1	SS	10							
			2	SS	16							0 2 58 39
102.5												
2.1	Becoming Firm		3	SS	7							
101.0												
3.7	Frequent sand seams Becoming Soft to Very Soft		4	SS	4							0 3 65 32
98.6			5	SS	1							
6.1	Sandy SILT Grey											
97.9					FI							
6.8	AUGER REFUSAL AT 6.81 m. CORING STARTED AT 6.81 m. CRYSTALLINE LIMESTONE (BEDROCK), Fresh to slightly weathered, very thinly to thinly bedded, whitish grey with black banding Subvertical joints at 7.19 and 7.49 m		1	RUN	1							RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=150.3MPa
			2	RUN	0							RUN 2# TCR=97%, SCR=97%, RQD=92%, UCS=110.6MPa
			3	RUN	0							RUN 3# TCR=98%, SCR=98%, RQD=98%, UCS=107.1MPa
94.8					0							
9.9	END OF BOREHOLE AT 9.86 m											

Continued Next Page

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BAS 05-8

2 OF 2

METRIC

W.P. 647-92-00 LOCATION N 5 031 181.20 E 315 519.51 (Baskin Drive WBL) ORIGINATED BY SLL  
 HWY 17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM  
 DATUM Geodetic DATE 06.12.05 - 06.12.05 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)						
						20	40	60	80	100	20	40	60				
	BOREHOLE GROUTED WITH BENTONITE TO GROUND SURFACE.				0												

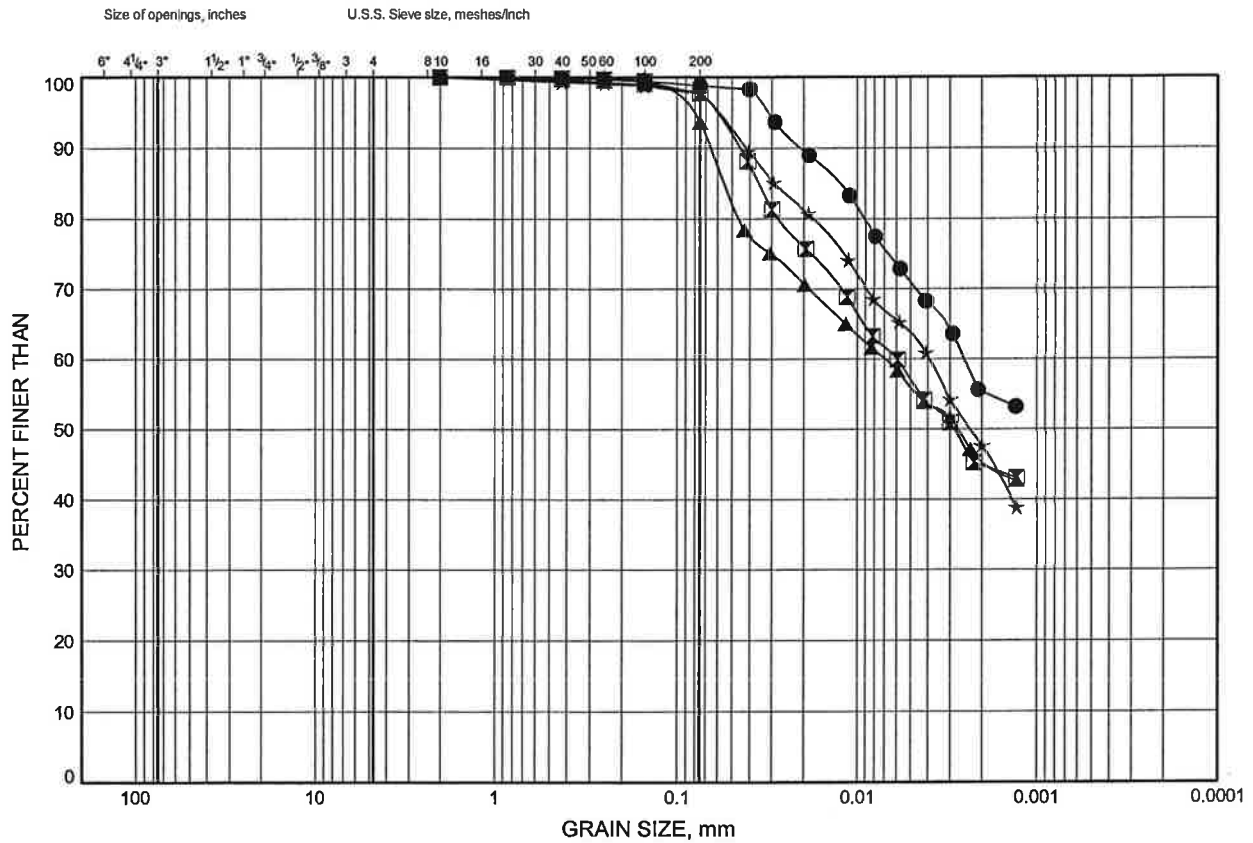
## **Appendix B**

### **Laboratory Test Results**

# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

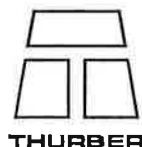
FIGURE B1

## SILTY CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BAS-1	3.35	102.85
◻	BAS-2	1.83	104.37
▲	BAS-4	2.59	103.71
★	BAS-5	6.40	99.80



THURBER

Date September 2004...

Project 647-92-00...

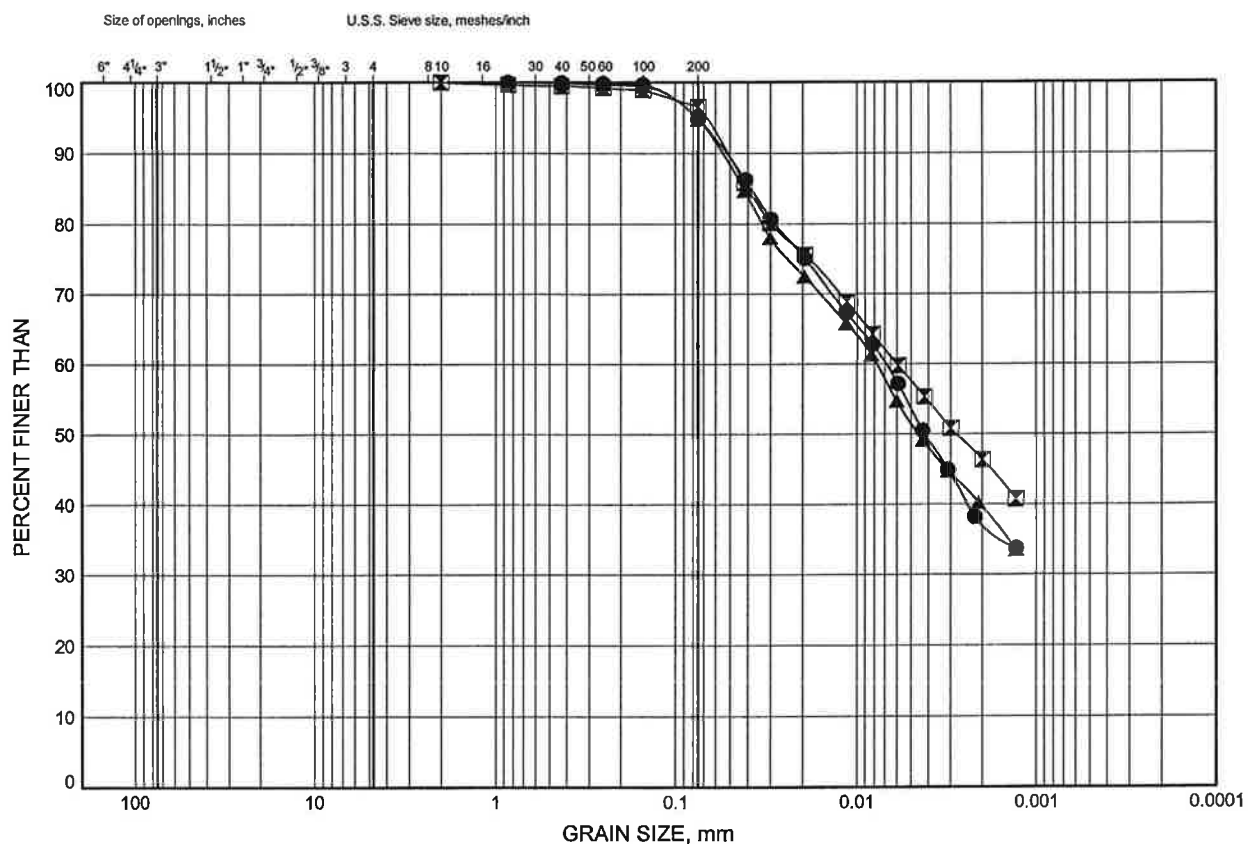
Prep'd HS

Chkd. SKP

# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B2

## SILTY CLAY



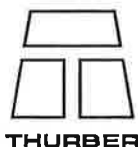
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BAS-11	3.35	101.85
■	BAS-8	1.83	103.27
▲	BAS-9	2.59	102.91

THURBGSD 7450BAS.GPJ 20/09/04

Date September 2004

Project 647-92-00



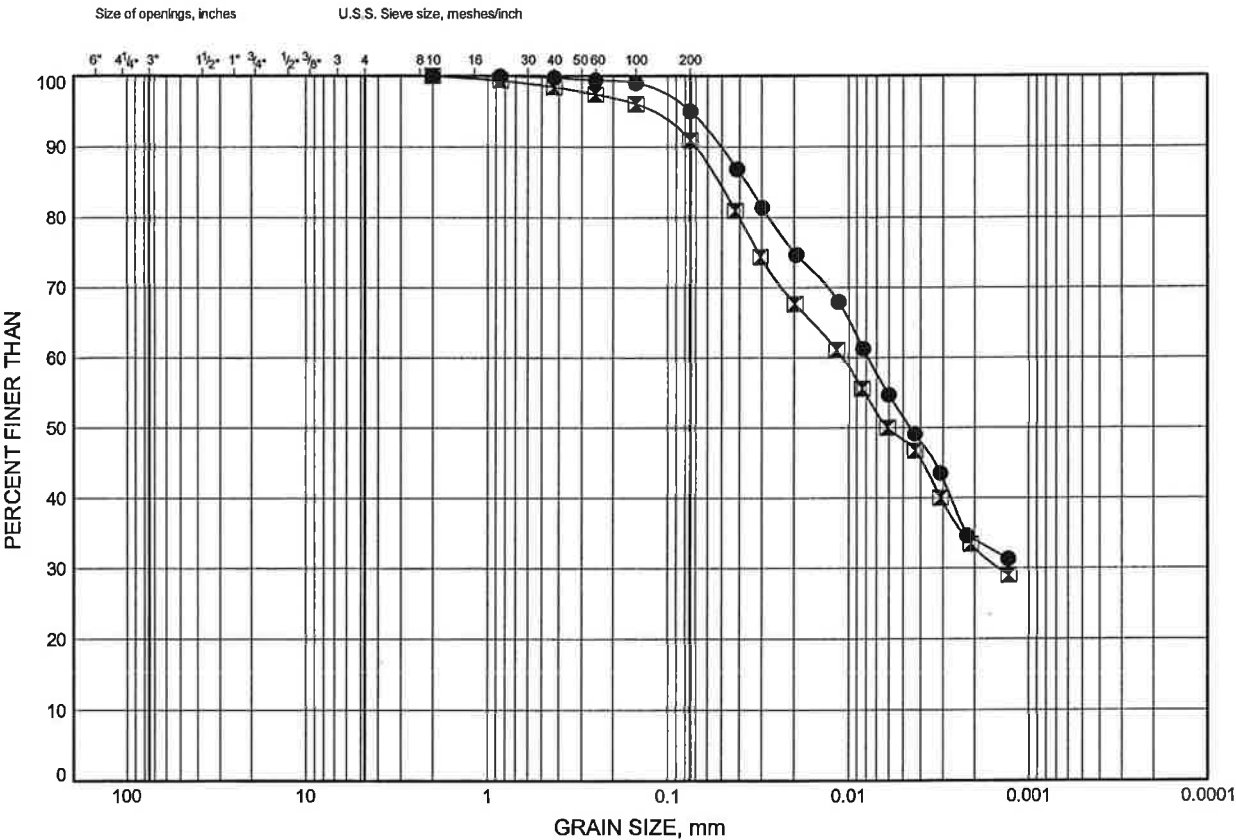
Prep'd HS

Chkd. SKP

# HWY 17 Twinning, Arnprior to Renfrew GRAIN SIZE DISTRIBUTION

FIGURE B3

### SILTY CLAY

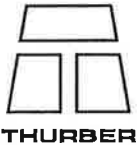


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BAS-1	4.88	101.32
◻	BAS-12	4.88	99.72

THURBGSD 7450BAS.GPJ 20/09/04

Date September 2004  
 Project 647-92-00

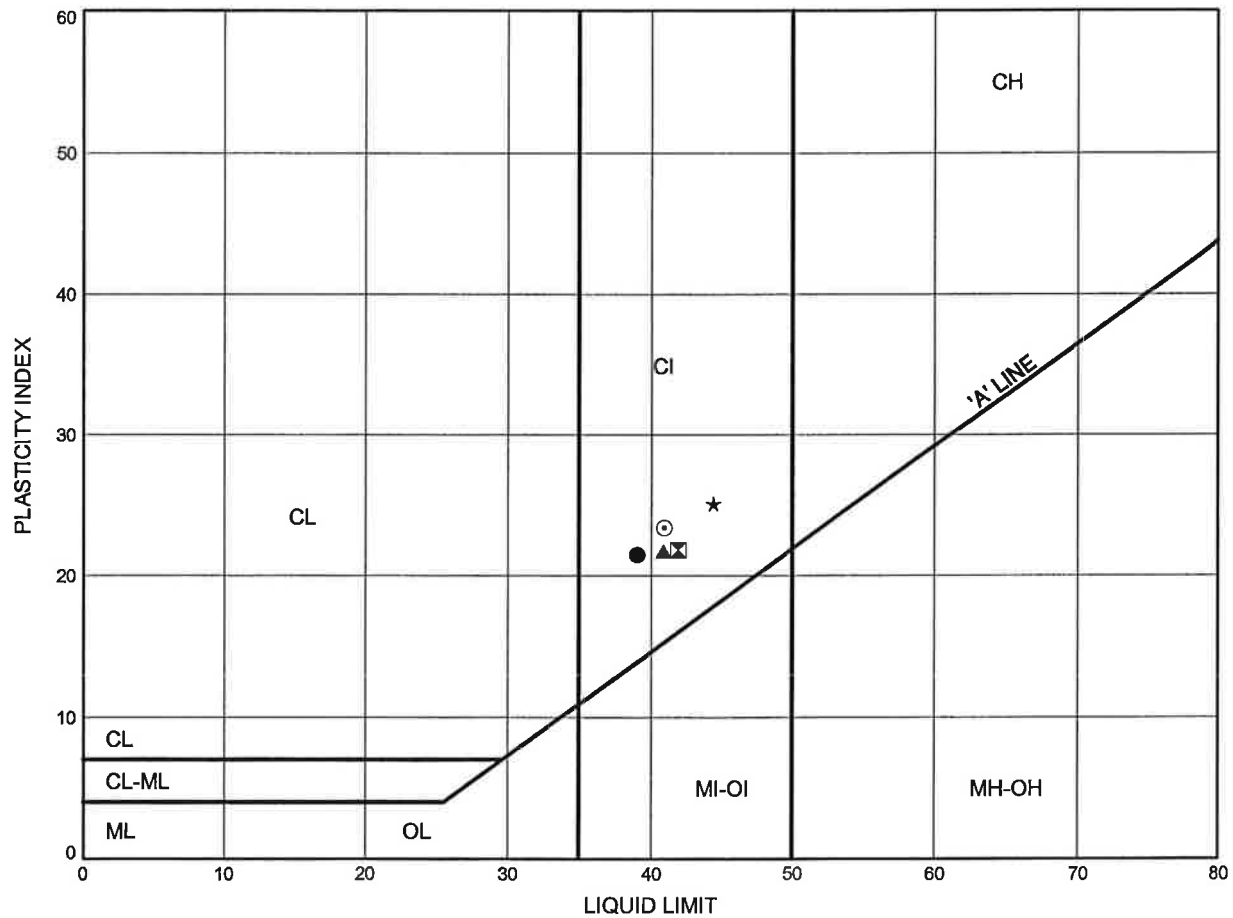


Prep'd HS  
 Chkd. SKP

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

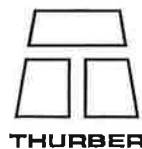
FIGURE B4

## SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BAS-11	3.35	101.85
⊠	BAS-2	1.83	104.37
▲	BAS-5	6.40	99.80
★	BAS-8	1.83	103.27
⊙	BAS-9	2.59	102.91

Date September 2004  
 Project 647-92-00

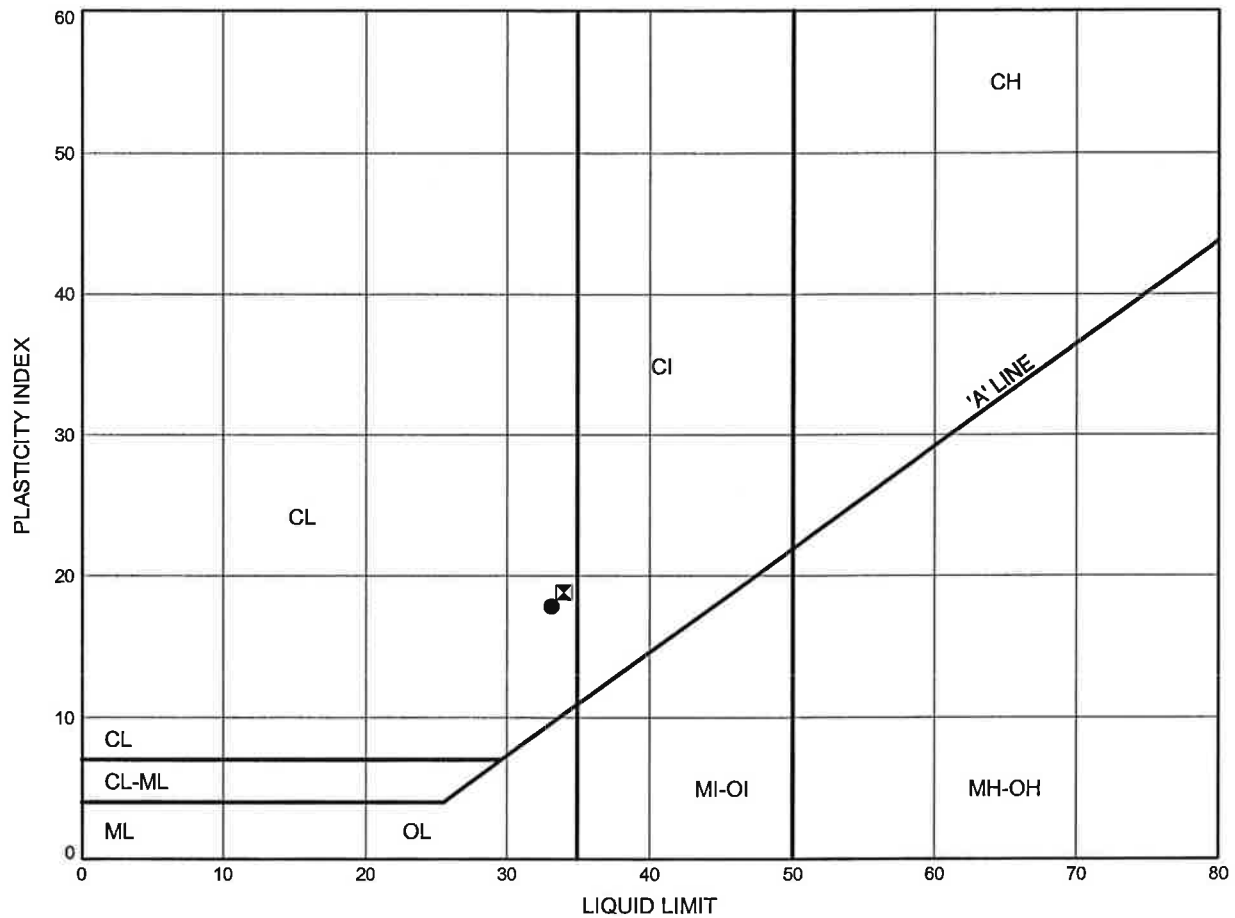


Prep'd HS  
 Chkd. SKP

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

FIGURE B5

**SILTY CLAY**



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BAS-1	4.88	101.32
⊠	BAS-12	4.88	99.72



## FIGURE B6

Size of openings, inches

U.S.S. Sieve size, meshes/inch

PERCENT FINER THAN

GRAIN SIZE, mm

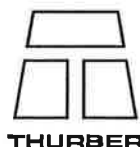
Grain Size (mm)	Percent Finer (%)
100	100
75	100
60	100
48	100
40	100
30	100
25	100
20	100
16	100
12	100
10	100
8	94
6	86
5	80
4	70
3	63
2.5	56
2	47
1.5	41
1.25	37
1.0	33
0.85	28
0.75	25
0.60	23
0.50	20
0.425	18
0.35	15
0.25	13

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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BAS-4	5.03	101.27

THURBGSD 7450BAS.GPJ 20/09/04

Project 647-92-00.....

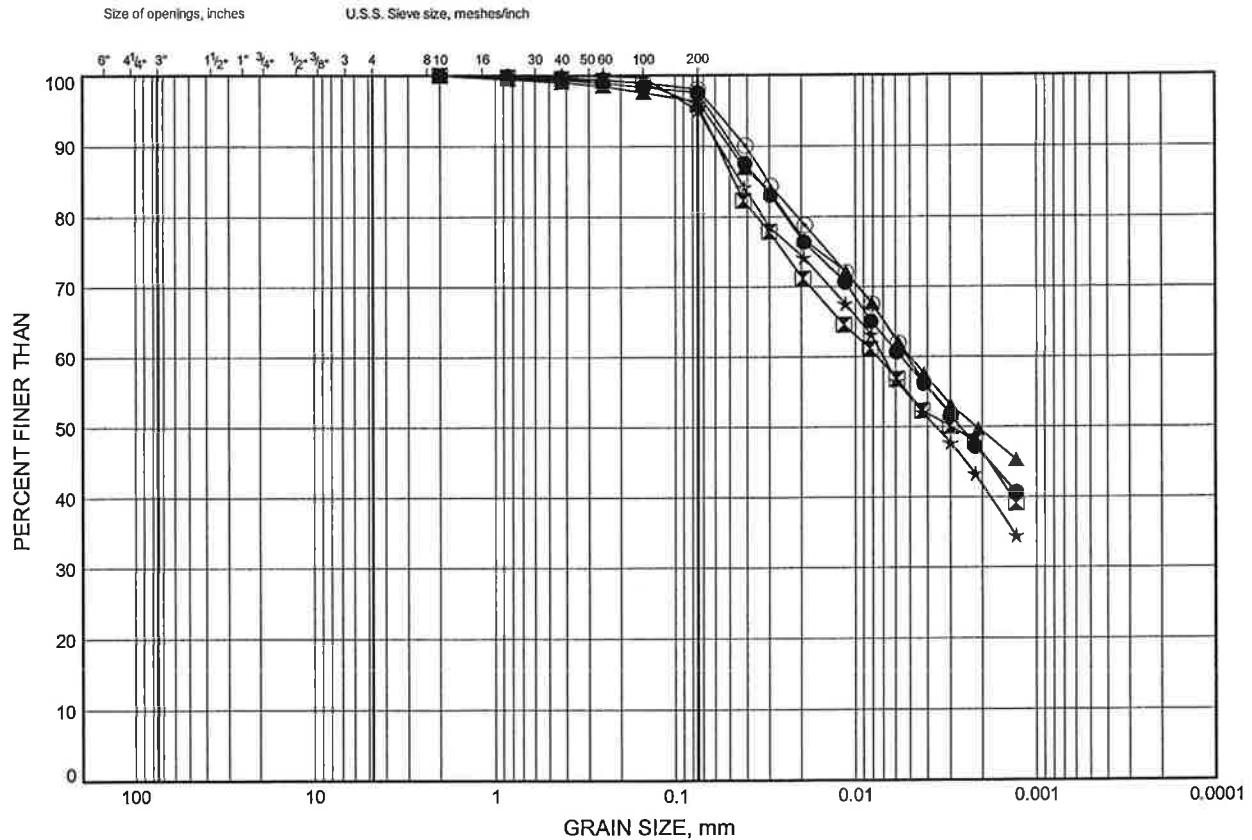


Chkd. SKP

# HWY 17-417 WBL GRAIN SIZE DISTRIBUTION

FIGURE B7

## SILTY CLAY



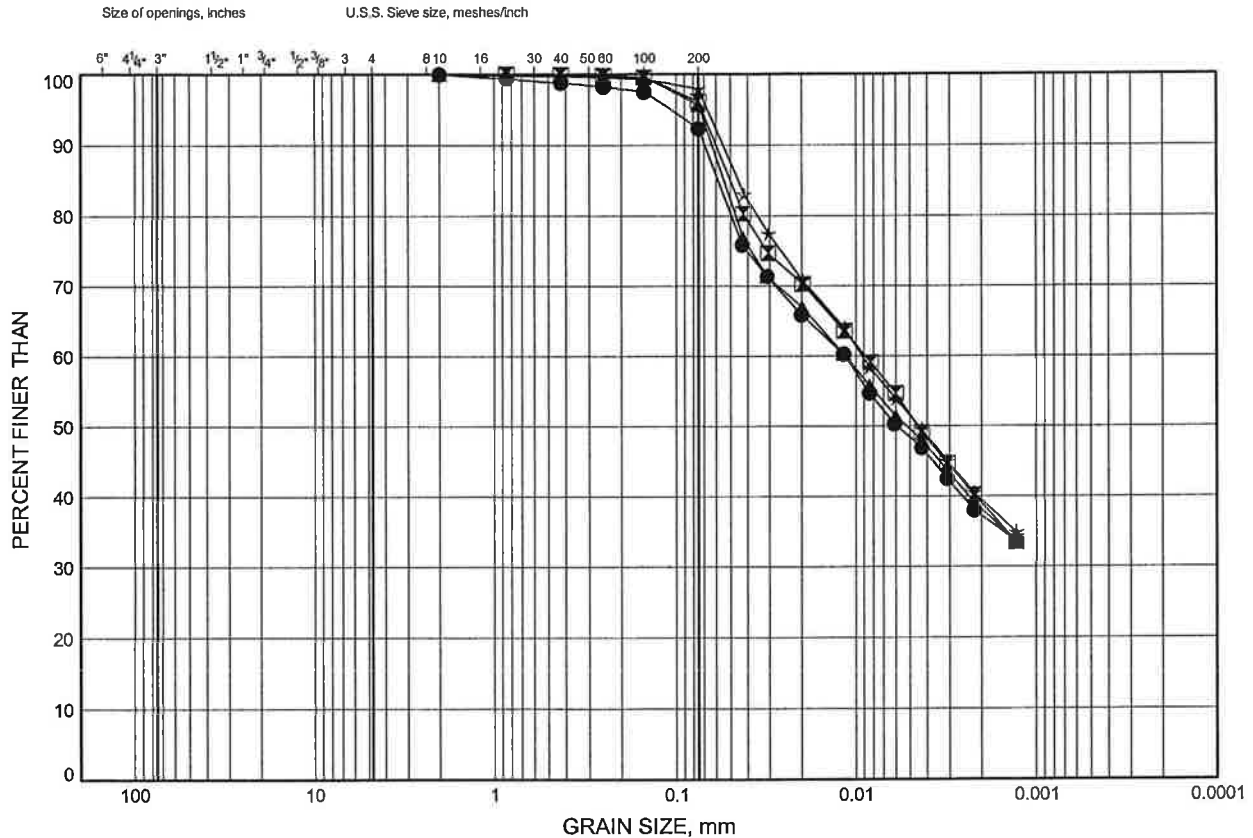
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BAS 05-1	1.22	104.88
⊠	BAS 05-2	2.74	103.47
▲	BAS 05-5	1.22	105.24
★	BAS 05-5	5.79	100.66
⊙	BAS 05-6	2.74	103.58

# HWY 17-417 WBL GRAIN SIZE DISTRIBUTION

FIGURE B8

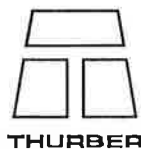
## SILTY CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BAS 05-3	4.34	101.38
⊠	BAS 05-6	5.79	100.53
▲	BAS 05-7	4.09	101.71
★	BAS 05-8	1.28	103.40

Date February 2006  
Project 647-92-00

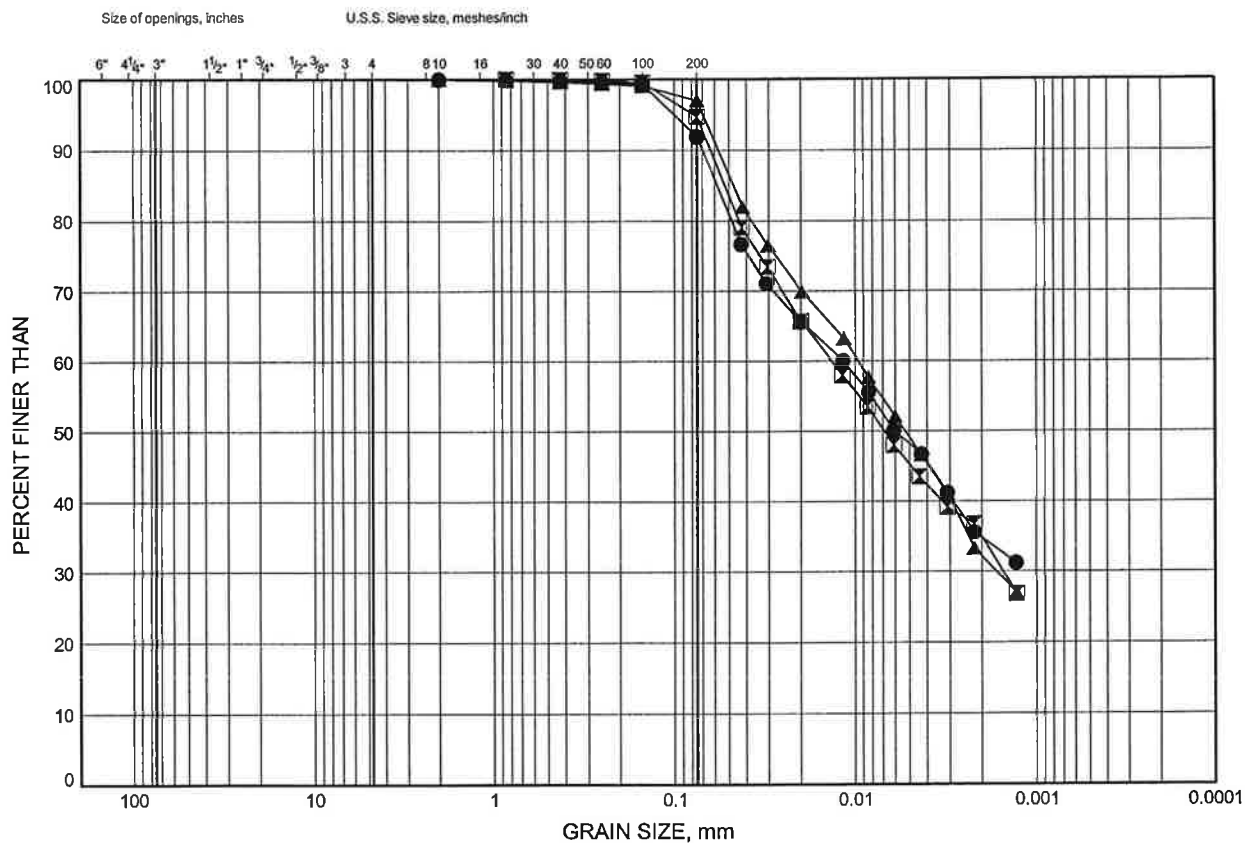


Prep'd JHL  
Chkd. SKP

# HWY 17-417 WBL GRAIN SIZE DISTRIBUTION

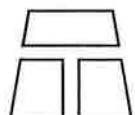
FIGURE B9

## SILTY CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BAS 05-1	4.27	101.83
◻	BAS 05-4	1.83	102.89
▲	BAS 05-8	4.29	100.39



THURBER

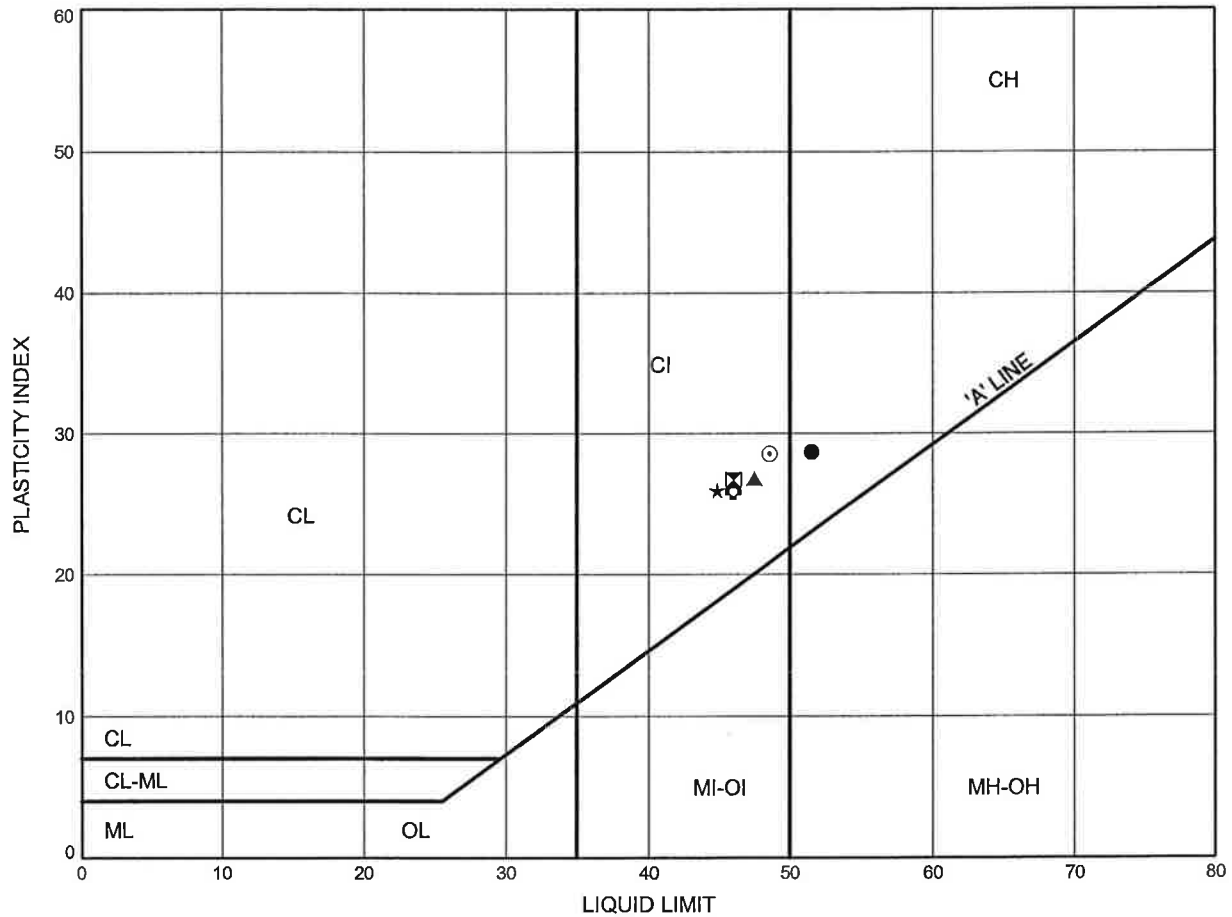
Date February 2006  
Project 647-92-00

Prep'd JHL  
Chkd. SKP

HWY 17-417 WBL  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE B10

**SILTY CLAY**

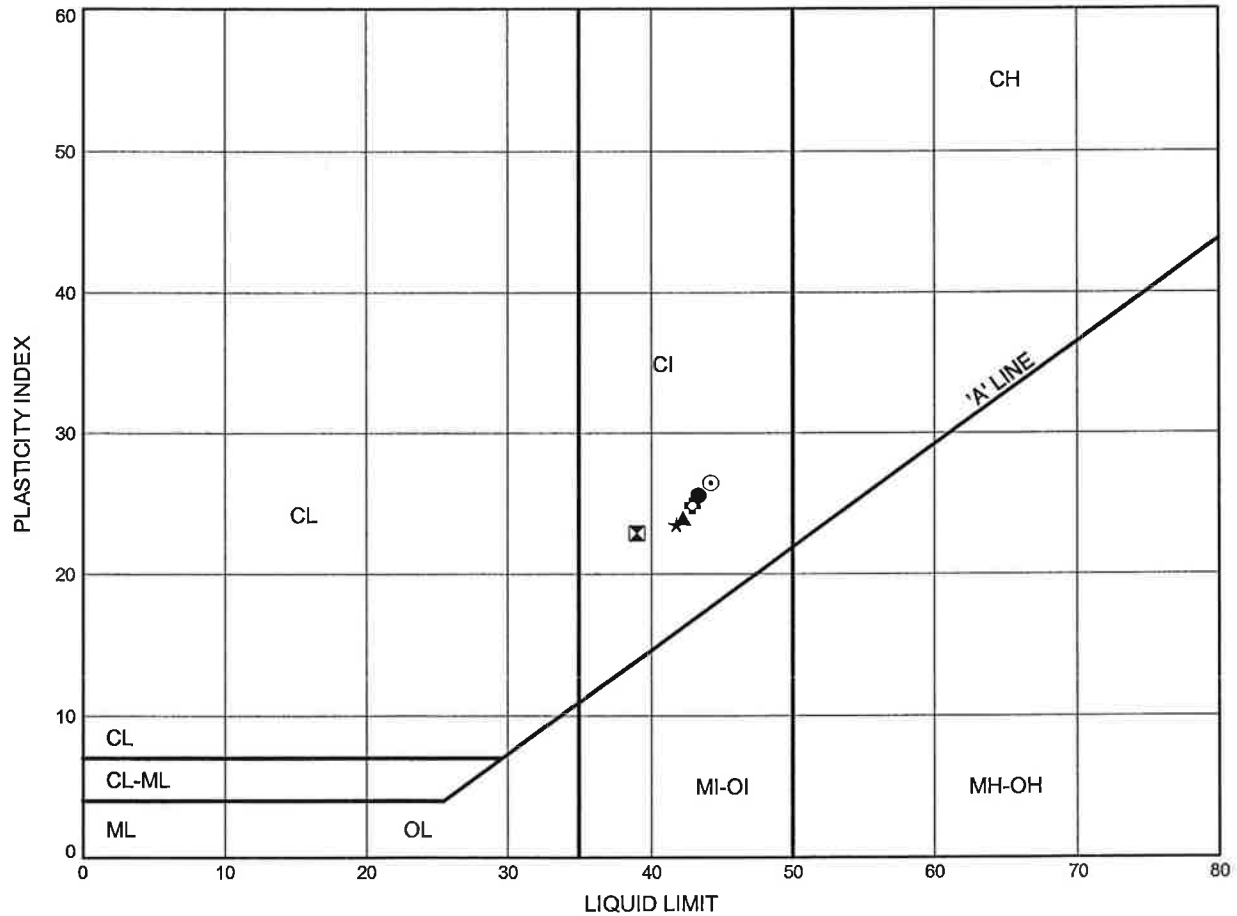


SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BAS 05-1	1.22	104.88
⊠	BAS 05-2	2.74	103.47
▲	BAS 05-5	1.22	105.24
★	BAS 05-5	5.79	100.66
⊙	BAS 05-6	2.74	103.58
⊛	BAS 05-8	1.28	103.40

# HWY 17-417 WBL **ATTERBERG LIMITS TEST RESULTS**

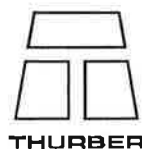
FIGURE B11

## **SILTY CLAY**



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	BAS 05-1	4.27	101.83
⊠	BAS 05-3	4.34	101.38
▲	BAS 05-4	1.83	102.89
★	BAS 05-6	5.79	100.53
⊙	BAS 05-7	4.09	101.71
⊕	BAS 05-8	4.29	100.39

Date February 2006  
 Project 647-92-00



Prep'd JHL  
 Chkd. SKP

## **Appendix C**

### **Foundation Alternatives**

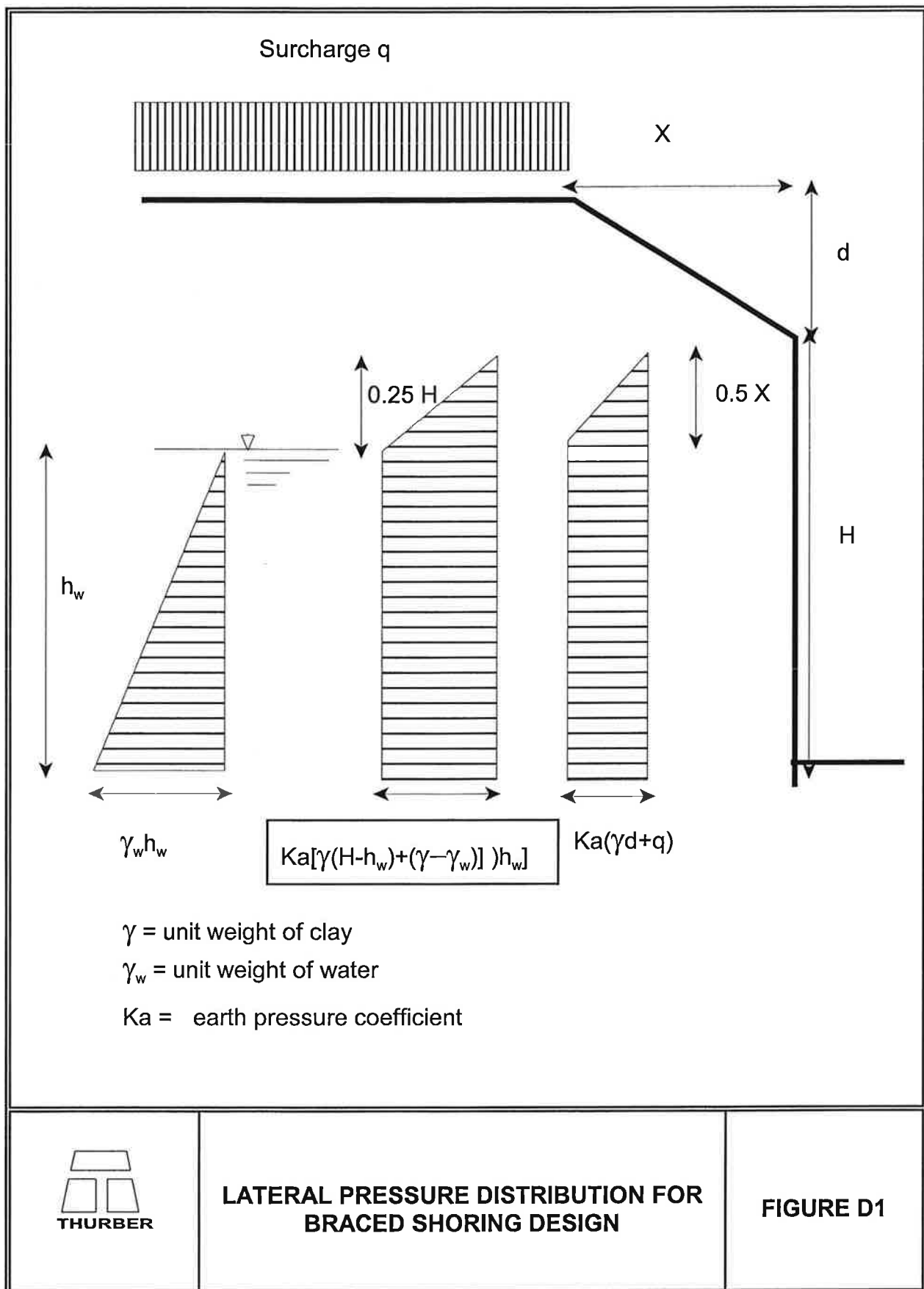
**COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT**

Foundation Element	Driven Piles	Footing on Bedrock	Footing on Engineered Fill	Caisson
West and East Abutments	<p><b>Advantages:</b> None identified</p> <p><b>Disadvantages:</b> i. Proximity of bedrock surface below the grade of the Baskin Drive cut rendering the use of driven piles unnecessary and impractical</p>	<p><b>Advantages:</b> 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</p> <p><b>Disadvantages:</b> i. High cost of excavation, if any is required ii. Mass concrete fill required to create a level founding surface</p>	<p><b>Advantages:</b> None identified</p> <p><b>Disadvantages:</b> i. Shallow bedrock surface at or below the grade of the Baskin Drive cut rendering the use of engineered fill unnecessary 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</p>	<p><b>Advantages:</b> None identified</p> <p><b>Disadvantages:</b> i. Proximity of bedrock surface below the grade of the Baskin Drive cut rendering the use of caissons unnecessary and impractical</p>



## Appendix D

### Figures



## **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 rock fill compaction. The equipment to be used shall be described in the submission. Where applicable, the submission shall describe modifications to be used during winter construction, to avoid the incorporation of frozen materials into the embankments.

### **206.06 EQUIPMENT**

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

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

### **206.07 CONSTRUCTION**

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

#### **206.07.01.03 Compaction**

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

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

Compaction of earth materials shall conform to OPSS 501.

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



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

## **206.07.05      Rock Excavation, Grading**

### **206.07.05.01    General**

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

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

## **206.07.08      Rock Embankments**

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

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

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

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

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

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

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

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

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

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

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

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

---

### **Special Provision**

---

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

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

##### **120.01 SCOPE**

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

##### **120.02 REFERENCES**

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

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

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

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

Ontario Traffic Manual Book 7

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

Explosives Act (Canada)

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

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

##### **120.03 DEFINITIONS**

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

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

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

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

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

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

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

## **120.04 SUBMISSION AND DESIGN REQUIREMENTS**

### **120.04.01 General**

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

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

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

#### **120.04.02.01 Blasting Consultant**

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

#### **120.04.02.02 Blasting Design**

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

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

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

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

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

#### **120.04.02.03                      Blasting Monitoring**

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

The blasting monitoring equipment shall meet the following requirements:

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

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

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

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

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

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

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

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

#### **120.04.05 Trial Blasting**

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

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

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

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

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

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

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

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

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

### **120.05 MATERIAL**

#### **120.05.01 Explosives**

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

#### **120.05.02 Handling and Storage**

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

#### **120.06 EQUIPMENT**

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

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

#### **120.07 CONSTRUCTION**

##### **120.07.01 General**

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

##### **120.07.02 Safety Precautions**

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

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

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

##### **120.07.03 Notice**

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

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

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

##### **120.07.04 Vibration Monitoring**

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

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

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

Structures

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

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

Fresh Concrete

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

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

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

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

**120.07.05                      Utilities**

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

**120.07.06                      Excessive Vibration Readings – Work Stoppage**

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

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



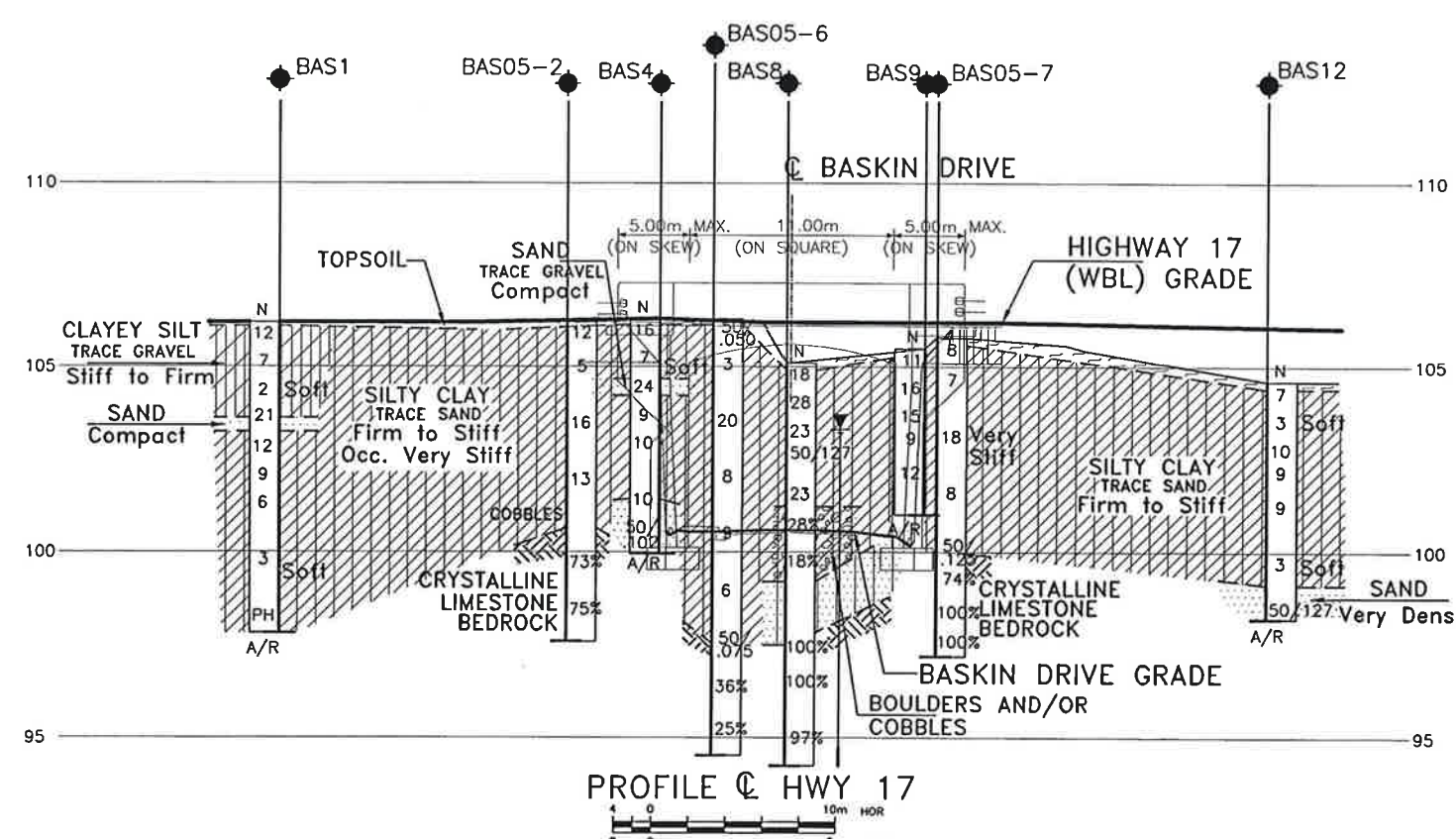
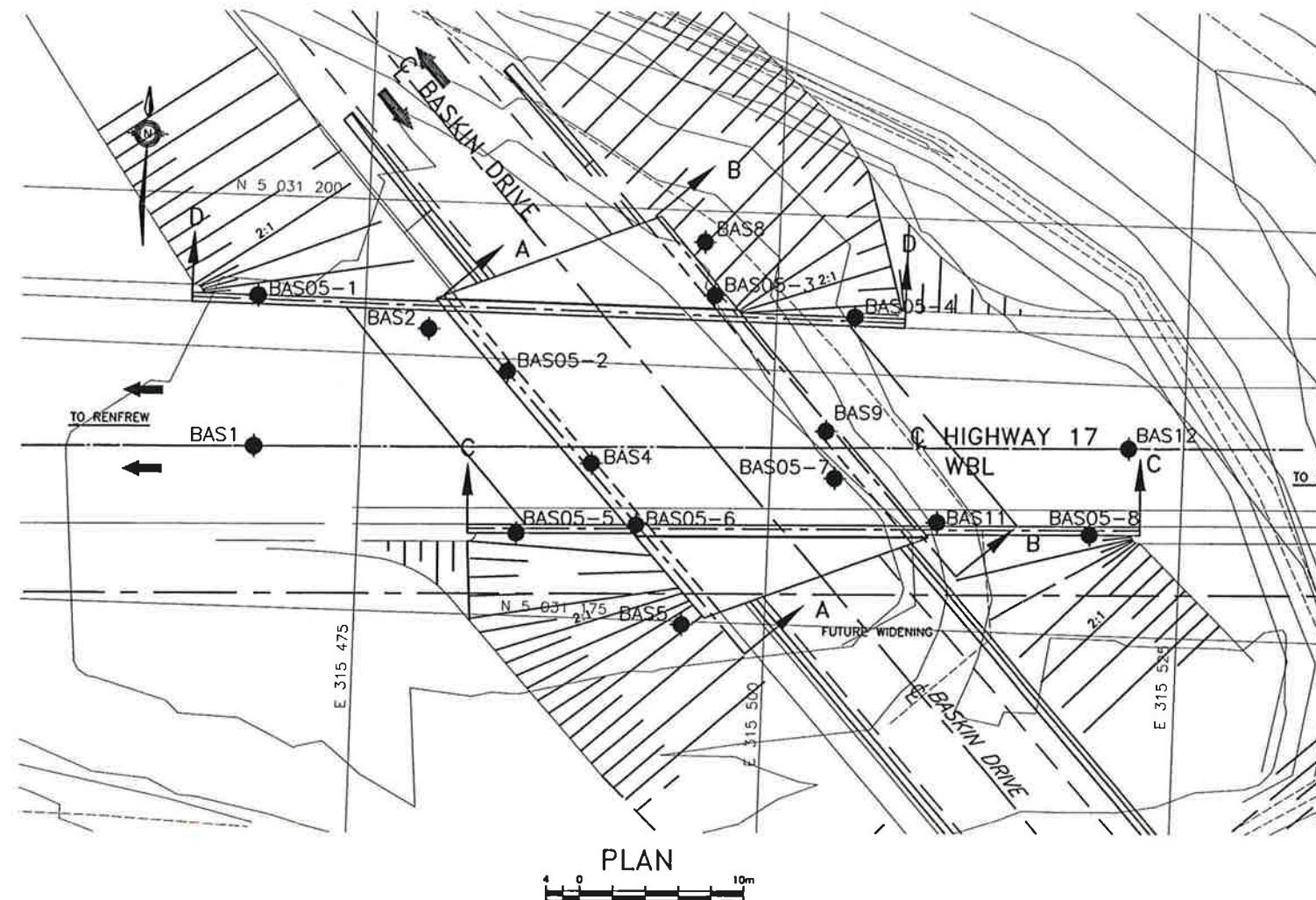
Baskin Drive Overpass (Westbound Lanes)  
Highway 17/417 Twinning, Arnprior

---

**Appendix F**

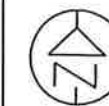
**Drawing**





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

HWY.17  
GWP NO. 647-92-00



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

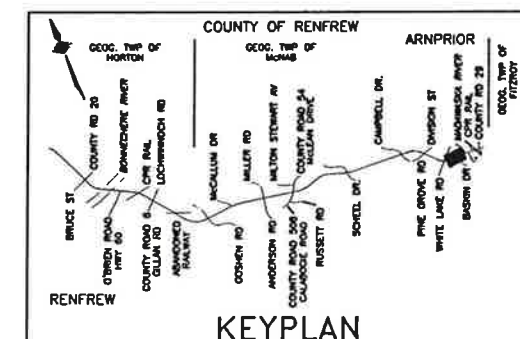
SHEET



McCORMICK RANKIN  
CORPORATION



THURBER ENGINEERING LTD.



# LEGEND

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

NO	ELEVATION	NORTHING	EASTING
BAS1	106.2	5031184.8	315468.5
BAS2	106.2	5031192.3	315478.9
BAS4	106.3	5031184.5	315489.1
BAS5	106.2	5031174.8	315494.9
BAS8	105.1	5031198.1	315495.5
BAS9	105.5	5031186.9	315503.3
BAS11	105.2	5031181.6	315510.2
BAS12	104.6	5031186.5	315521.7
BAS05-1	106.1	5031194.0	315468.4
BAS05-2	106.2	5031189.9	315483.8
BAS05-3	105.7	5031194.9	315496.2
BAS05-4	104.7	5031193.9	315504.8
BAS05-5	106.5	5031180.1	315484.8
BAS05-6	106.3	5031180.8	315492.0
BAS05-7	105.8	5031184.1	315503.9
BAS05-8	104.7	5031181.2	315519.5

## NOTE

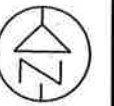
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION
FEB. 06	SP	FINAL (REVISED)	
SEP. 04	SP	FINAL	
FEB. 04	SP	ISSUED AS DRAFT FOR REVIEW	
DESIGN	SP	CHK PKC	CHBDC 2000
DRAWN	SS	CHK SP	SITE29-423/2
			LOAD
			DATE FEB 2006
			DWG. 19-1351-82-1



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

HWY.17  
GWP NO. 647-92-00



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

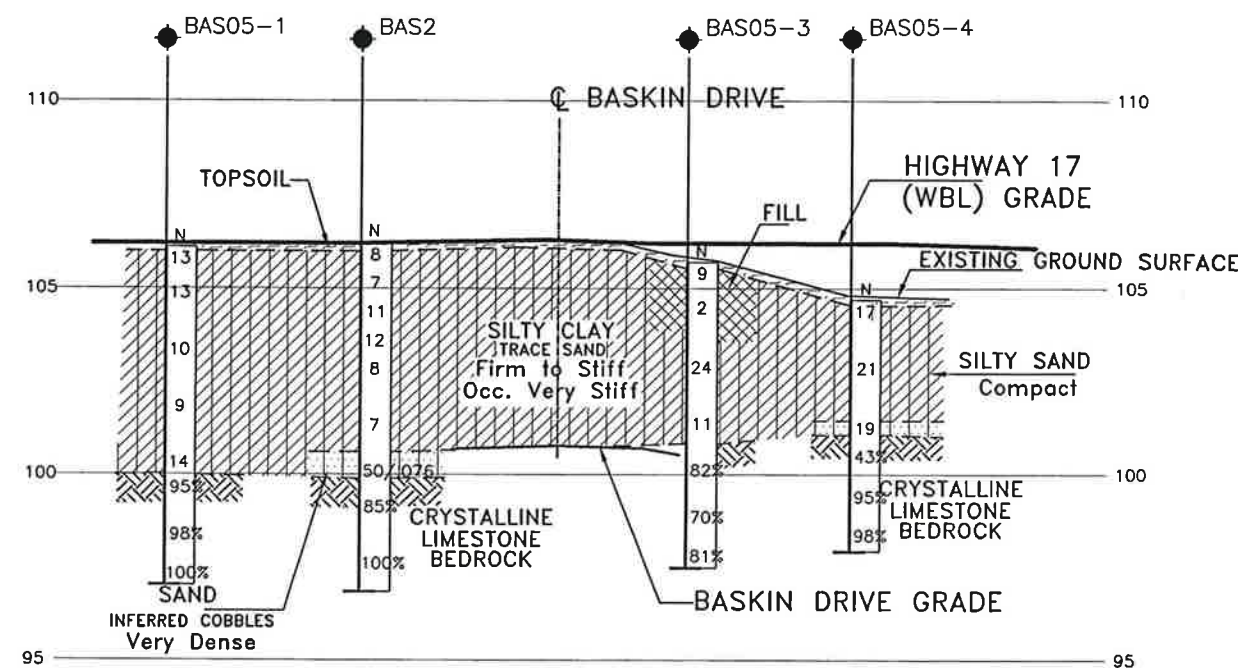
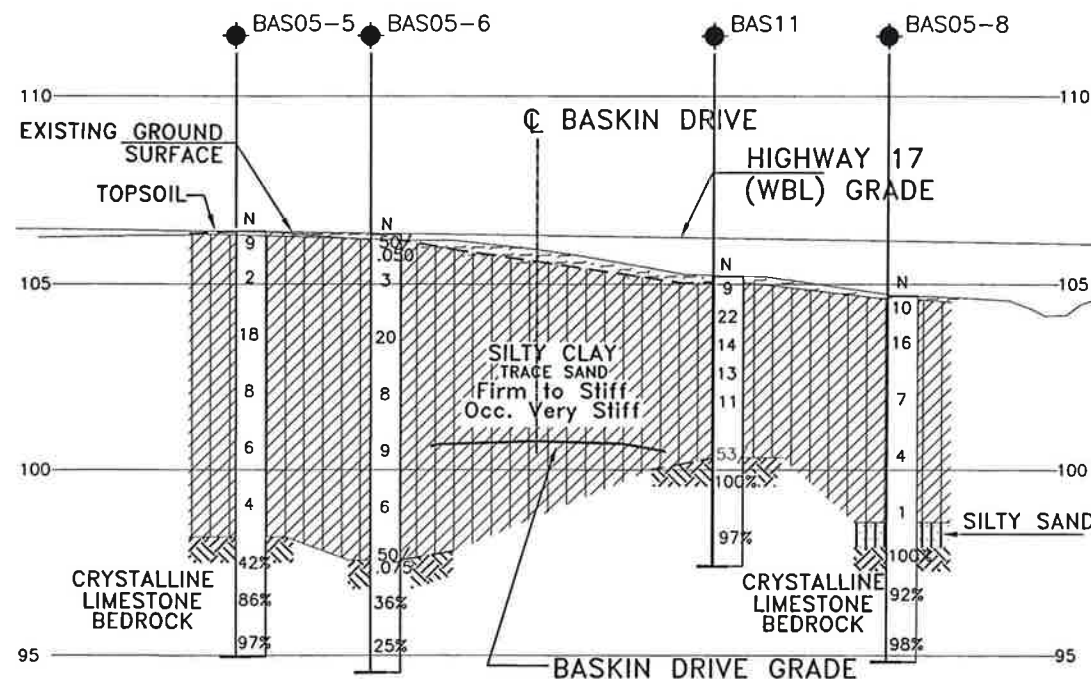
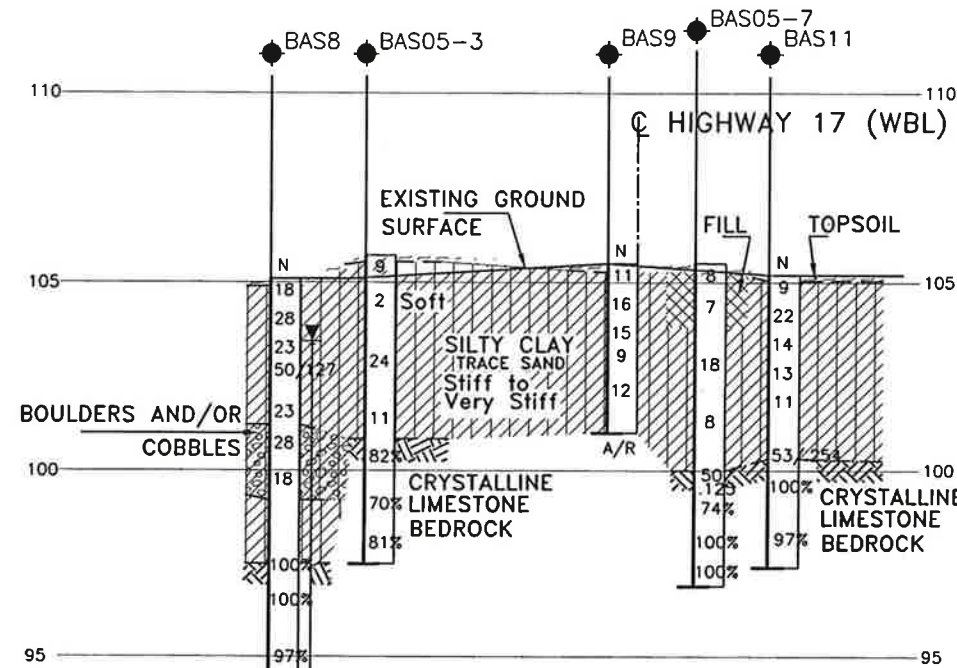
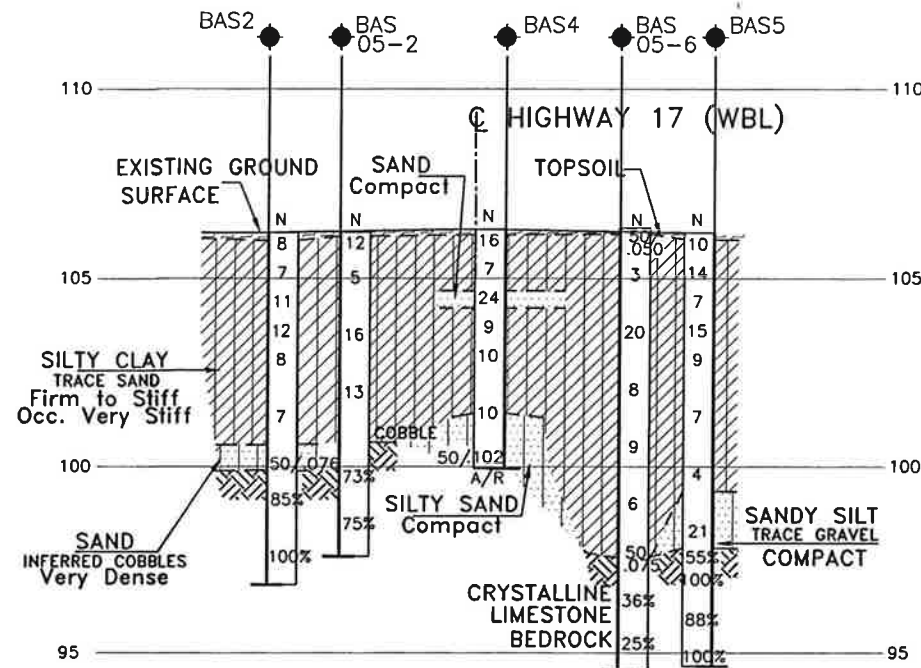
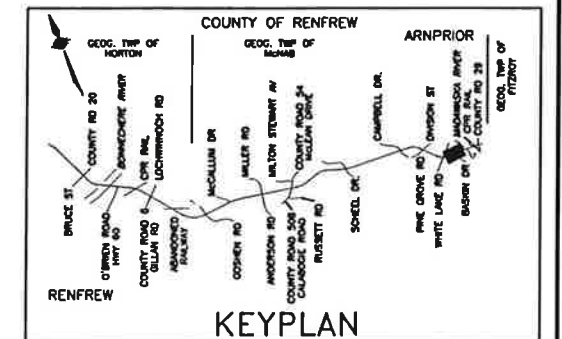
SHEET



McCORMICK RANKIN  
CORPORATION



THURBER ENGINEERING LTD.



# LEGEND

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

NO	ELEVATION	NORTHING	EASTING
BAS1	106.2	5031184.8	315468.5
BAS2	106.2	5031192.3	315478.9
BAS4	106.3	5031184.5	315489.1
BAS5	106.2	5031174.8	315494.9
BAS8	105.1	5031198.1	315495.5
BAS9	105.5	5031186.9	315503.3
BAS11	105.2	5031181.6	315510.2
BAS12	104.6	5031186.5	315521.7
BAS05-1	106.1	5031194.0	315468.4
BAS05-2	106.2	5031189.9	315483.8
BAS05-3	105.7	5031194.9	315496.2
BAS05-4	104.7	5031193.9	315504.8
BAS05-5	106.5	5031180.1	315484.8
BAS05-6	106.3	5031180.8	315492.0
BAS05-7	105.8	5031184.1	315503.9
BAS05-8	104.7	5031181.2	315519.5

## NOTE

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



REVISIONS	DATE	BY	DESCRIPTION
FEB. 06	SP	FINAL (REVISED)	
SEP. 04	SP	FINAL	
FEB. 04	SP	ISSUED AS DRAFT FOR REVIEW	
DESIGN	SP	CHK PKC	CHBDC 2000
DRAWN	SS	CHK SP	SITE29-423/2 STRUCT
			DATE FEB 2006
			DWG. 19-1351-82-2