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GEOCRES No. 31E-162DIST. 52 REGION \_\_\_\_\_W.P. No. 690-93-01  
6WP: 217-89-00

CONT. No. \_\_\_\_\_

W. O. No. \_\_\_\_\_

STR. SITE No. 42-318HWY. No. 69LOCATION Cranberry Marsh Rd.  
Crossing

No of PAGES - \_\_\_\_\_

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. \_\_\_\_\_

REMARKS: \_\_\_\_\_  
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GEOCRES #31E-16

**Foundation Investigation Report,  
Cranberry Marsh Road Underpass,  
W.P. 690-93-01, Site 42-318  
Highway 69, District 52  
Huntsville, Ontario**

*Revised Final*

*Revised Final  
Report -  
Received on  
March 2, 1999.*

Prepared for:

R.V. Anderson Associates Ltd.  
2001 Sheppard Avenue, East  
Suite 400  
Willowdale, Ontario  
M2J 4Z8

**Trow Consulting Engineers Ltd.**

1595 Clark Boulevard  
Brampton, Ontario L6T 4V1  
Telephone: (905) 793-9800  
Facsimile: (905) 793-0641

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## Preface

Work Project 217-89-00 is one of a series of projects for the four lane expansion of Highway 69. This project is located from 0.4 km south of the Musquash River, northerly 8.9 km to Tower Road, within the MTO Northern Region, District 52, Huntsville.

It is located in the former Townships of Gibson, Freeman and on the Whata Mohawk First Nation Lands in the present Township of Georgian Bay, District of Muskoka. This project includes:

- the construction of new Southbound Lanes
- rehabilitation of the existing highway to divided freeway standards to become the Northbound Lanes
- construction of a replacement bridge over the Musquash River for the Northbound Lanes
- construction of a bridge over the Musquash River for the Southbound Lanes
- construction of a diamond interchange at the intersection of Cranberry Marsh Road and Highway 69
- construction of a bridge over the Moon River for the Southbound Lanes
- construction of associated side roads resulting from the creation of the controlled access highway
- construction of a diamond interchange at the intersection of Muskoka Road 12 and Highway 69

The following report comments on the geotechnical investigation and subsequent recommendations for the Cranberry Marsh Road Overpass at Highway 69.

Other associated Geotechnical, Foundation and Pavement Reports for this project include:

- Foundation Investigation Report, Approach Embankments, Southbound Lanes, Musquash River, MTO Foundation Section, March 1993
- Pavement Design report, Trow Consulting Engineers Ltd., January 1998
- Foundation Investigation Report, Musquash River, Northbound Lanes Replacement Bridge, Site 42-46N, Trow Consulting Engineers Ltd., January 1998
- Foundation Investigation, Musquash River, Southbound Lane Bridge, Site 42-465, MTO Foundation Section, March 1993
- Foundation Investigation Report, Musquash River Bridge, Site 42-465, Trow Consulting Engineers Ltd., January 1998

- Foundation Investigation Report, Moon River, Southbound Lane Bridge, Site 42-265, Trow Consulting Engineers Ltd., January 1998

Forthcoming reports include:

- Foundation Investigation Report, Muskoka Road 12 Interchange, Trow Consulting Engineers Ltd., Spring 1998.
- Supplemental Pavement Design Report, Trow Consulting Engineers Ltd., Spring, 1998.

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## PART 1 Foundation Investigation

### 1.1 Introduction

This submission presents the results of a geotechnical investigation completed by Trow Consulting Engineers Ltd. (Trow) for the proposed crossing at Cranberry Marsh Road and King's Highway 69, WP 690-93-01, Site number 42-318. It is our understanding that a two span structure will be constructed with the central pier located in the median of the proposed King's Highway 69. This report contains factual information (obtained from the field investigation) pertaining to the design parameters required for the bridge foundations and related earthworks.

### 1.2 Site Description and Geological Setting

#### 1.2.1 Site Description

The site is located in Gibson Township along Highway 69 at the Cranberry Marsh Road (approximate Station 21+500 along Highway 69). The Cranberry Marsh Road, at present, is an at grade crossing of the existing Highway 69. The new Highway 69 will be relocated to the west of the existing highway and a new two span bridge will be constructed to carry Cranberry Marsh Road traffic over Highway 69. An 8 m grade increase at abutments is expected in accordance with the new grading plan.

The terrain in the area is relatively flat with many outcrops of gneissic bedrock. Mature trees are present at most of the borehole locations to the west of the existing highway.

#### 1.2.2 Geological Setting

According to OGS Maps 2544 and 2556, the site is located in what is known as the central gneiss belt. The bedrock at the site consists of Precambrian gneisses of metasedimentary origin. As previously noted, the topography in the areas is undulating consisting of bedrock outcrops. As such, the surface soils in the area consist of intervening shallow organic deposits (peat, muck and marl), and glaciofluvial deposits consisting of gravel and sand, including proglacial river and deltaic deposits.

### **1.3 Investigative Procedures**

#### **1.3.1 General**

Part 1 of this report describes the investigative procedures adopted for the geotechnical assessment of the Cranberry Marsh Road interchange at King's Highway 69. Properties of the overburden soils at the site were obtained by in situ and laboratory testing and the procedures employed during the investigation are described below.

#### **1.3.2 Field Investigation**

The fieldwork for the investigation related to the proposed bridge structure was carried out between November 17 and 19, 1997, and consisted of six (6) boreholes (Boreholes 1 to 6) and nine (9) auger probeholes. The boreholes were advanced to depths ranging from 0.46 to 4.02 metres. The probeholes were advanced to refusal depths ranging from 0.15 to 2.13 metres.

Two (2) boreholes were drilled near each of the east and west abutments and the centre pier location. The borehole and probe hole locations are shown on Drawing 1 in Appendix A. All borehole elevations were provided by P. Collie of R.V. Anderson and are referenced to geodetic datum.

The boreholes were advanced through the overburden soils on the site using a truck mounted CME-55 drill rig equipped with solid and hollow stem augers. Soil samples were obtained using a 51 mm O.D. split spoon sampler in conjunction with Standard Penetration Tests (ASTM D1586) at approximately 0.75 metre and 1.5 metre intervals. The Standard Penetration (N) values were recorded and used to provide an assessment of the relative denseness of the overburden soils at the site and the soil samples were used for identification and laboratory testing.

At three (3) of the borehole locations, conventional rock coring techniques were used to advance the boreholes approximately 3.0 to 3.6 metres into the underlying bedrock. B size core barrels and casings were used and core samples of the bedrock were retrieved for rock quality determinations and classification purposes.

#### **1.3.3 Laboratory**

The laboratory testing program for select soil samples consisted of the following:

- Natural Moisture Contents
- Grain Size Distributions



The laboratory test results are summarized on the attached Borehole Logs in Appendix A. The grain size distributions for soil samples from Boreholes 4 and 5 are presented in Appendix B.

#### **1.4 Subsurface Conditions**

The Borehole locations are shown on Drawing 1 of Appendix A and the subsurface information revealed from the boreholes near the proposed bridge location are summarized on the attached Borehole Logs 1 to 6, inclusive and in the Probehole Logs 1 to 9, inclusive. Based on the borehole information, the following soil layers are encountered at this site:

- Topsoil;
- Silty Sand; and
- Bedrock.

A summary for the description of the various soil strata encountered in the boreholes is presented below.

##### **1.4.1 Topsoil**

A thin layer of topsoil, ranging in thickness from 50 to 200 mm, was encountered in Boreholes 2, 3, 4, 5, and 6. The topsoil is generally of a sandy nature, brown in colour and contains rootlets.

##### **1.4.2 Silty Sand**

A layer of brown silty sand is found to underlie the topsoil and the surface of all the boreholes. The sand is generally fine, becoming fine to medium with gravel with depth. The standard penetration resistance 'N' values ranged from 3 to 34 blows/300 mm indicating a loose to dense state of denseness. The thickness of the silty sand ranges from 0.1 m to 1.4 m and moisture contents of the sand varied from 4 to 24 percent.

Grain size analyses of the silty sand are presented in Appendix B (Figures 1 and 2) and on the borehole logs in Appendix A. In general, the sand content varied from 45% to 72% and the fraction less than 75  $\mu$ m varied from 55% to 28%.

### 1.4.3 Bedrock

The bedrock was proven by obtaining B cores in three (3) of the six (6) boreholes, Nos. 1, 4 and 5. The bedrock level was found to range from Elevation 212.1 to 214.4 m at the proposed bridge location based on the findings from coring and probeholes.

Detailed descriptions of the rock are presented in the borehole logs (Appendix A). Generally, the bedrock can be described as a dark grey Hornblende Gneiss. The rock is strong to medium strong, unweathered with variable fracture spacing ranging from close to wide. A possible boulder at the core location was noted in Borehole 1.

Rock core recovery was 100% for all runs and Rock Quality Designation (RQD) values ranged from 88 to 100 %. Water loss was noted in the cores from Borehole 1. In Boreholes 4 and 5, the wash water return was noted to be about 100 percent.

Nine (9) auger probeholes were also put down at the proposed bridge location to estimate the level of the bedrock at the abutment and pier locations. The probeholes suggested that the overburden thickness at the east abutment, centre pier and the west abutment locations are 150 to 300 mm, 1620 mm, and 1160 to 2130 mm, respectively.

### 1.5 Groundwater Conditions

Information regarding the groundwater levels at the site was obtained by measuring the water levels in the open boreholes after the completion of drilling. Free standing water was observed in Boreholes 4, 5 and 6. The water level was noted to be within 150 mm of the ground surface at Boreholes 5 and 6 and 0.73 m at Borehole 4.

## PART 2 Engineering Discussions and Recommendations

### 2.1 General

The following subsections address geotechnical considerations pertaining to the proposed two span bridge over the new King's Highway 69 north of Station 21+500. The east abutment will be situated near where the existing King's Highway 69 is at present. The centre pier will be located between the north and southbound lanes of the new King's Highway 69 with the west abutment to the west of the southbound lanes of the new highway.

### 2.2 Foundations

Based on the subsoil conditions noted in the boreholes, bedrock was noted at relatively shallow depths at this site. As such, the most suitable type of foundation would be spread type foundation founded on the Gneiss bedrock or on a compacted granular pad. Piling is required if integral abutments are considered and provided that the requisite flexibility can be developed given the geometric conditions.

The design parameters for footings founded on the bedrock or granular pad and for piles, if desired, are provided below.

#### 2.2.1 Footings on Bedrock

##### 2.2.1.1 Abutment Locations

At the location of the east and west abutment, there is approximately 200 to 460 mm and 1100 mm of overburden soil, respectively, overlying bedrock. Given the relatively thin layer of overburden sand, placement of new footings directly on Bedrock is recommended. For the purpose of design in accordance with the Ontario Highway Bridge Design Code, the following bearing capacities can be used for spread footings placed directly on the Gneiss bedrock subject to inspection by a qualified geotechnical engineer:

**Table 2-1 Spread Footing Capacity on Bedrock**

	<b>Spread Footing</b>
Factored Geotechnical Resistance at ULS	10000 kPa

The above Factored Geotechnical Resistance at ULS applies to spread footings placed directly on bedrock with a good Rock Mass Quality (RQD>75). The bearing capacity at SLS will not govern for a spread footing founded on bedrock since the loads required to produce unacceptable settlements of the structure will be much larger than the recommended values for the factored capacity at ULS

For the east abutment area, the borehole data (Boreholes 1 and 2) indicate that the construction of spread footings on bedrock would require excavation and removal of approximately 200 to 460 mm of overburden soils. At the west abutment (Boreholes 5 and 6), the overburden soil is approximately 1100 mm thick. The estimated founding level for the footing at this location is near Elevations 213.4 and 213.9 m (at Boreholes 1 and 2, respectively) and 213.1 and 213.2 m (at Boreholes 5 and 6, respectively) for the east and west abutments. The footing bases must be cleared of all loose materials prior to placement of concrete and inspected by a qualified geotechnical engineer to verify the Rock Mass Quality.

As per section 6-8.4.2 of the Ontario Highway Bridge design code, a reduction factor shall be applied to the Ultimate Bearing Resistance at ULS (10,000 kPa) to account for the effects of inclined loading. Table 2-2 contains a summary of reduction factors for inclined loads.

**Table 2-2 Reduction Factors to Account for the Effects of Inclined Loads on the Ultimate Bearing Resistance at ULS\***

<b>Ratio of Horizontal to Vertical Load</b>	<b>Reduction Factor</b>
0.1	0.87
0.2	0.76
0.3	0.66
0.4	0.57

\*As advised by MTO Foundation Section " Although the OBHDC provides resistance reduction due to inclined loadings on bedrock, the OHBDC committee has decided that no such reduction will be required if the footing is constructed on bedrock.

Note: The structural engineer can refer to Figure 6-8.4.2 of the Ontario Highway Bridge Design Code for reduction factors corresponding to ratios of horizontal to vertical loads which are not listed above.

### 2.2.1.2 Centre Pier

The subsurface conditions for the centre pier location are similar to those discussed for the abutments. As a result, spread footings placed directly on bedrock are also recommended for the centre pier given the relative thin overburden of topsoil and silty sand (730 to 1370 mm). Bedrock was encountered at approximate Elevation 213.3 to 213.8 m at the test locations. For the purpose of design base on the Ontario Highway Bridge Design Code, the following bearing capacities can be used for footings placed directly on bedrock subject to geotechnical inspection:

**Table 2-3 Spread Footing Capacity on Bedrock**

	<b>Spread Footing</b>
Factored Geotechnical Resistance at ULS	10000 kPa

The above Factored Bearing Capacity at ULS applies to spread footings placed directly on bedrock with a good Rock Mass Quality (RQD>75). The bearing capacity at SLS will not govern for a spread footing founded on bedrock since the loads required to produce unacceptable settlements of the structure will be much larger than the recommended values for the factored capacity at ULS

### 2.2.1.3 Anticipated Footing Elevations

The following Table summarizes the location and estimated footing base elevations at which the recommended Factored Bearing Capacity (at ULS) is applicable:

**Table 2-4 Location and Estimated Elevation of Footing Bases for Bridge Abutments.**

Location	Boreholes and Probeholes	Overburden Thickness (mm)	Approximate Elevation (m)
East Abutment	Borehole 1	460 to 760*	213.9
	Borehole 2	200 to 460**	213.4
	Probehole AP1	300	213.6
	Probehole AP2	250	213.6
	Probehole AP3	150	214.4
	Probehole AP4	300	213.2
	Probehole AP5	200	213.5
Centre Pier	Borehole 3	760	213.8

**Table 2-4 Location and Estimated Elevation of Footing Bases for Bridge Abutments.**

Location	Boreholes and Probeholes	Overburden Thickness (mm)	Approximate Elevation (m)
	Borehole 4	1370	213.3
	Probehole AP6	1620	212.9
West Abutment	Borehole 5	1070	213.2
	Borehole 6	1070	213.1
	Probehole AP7	1200	212.9
	Probehole AP8	1520	212.9
	Probehole AP9	2130	212.1

\* Possible boulder (~300mm in diameter) near bedrock surface.

\*\* See note on Borehole Log 2 (Appendix A)

The above elevations are for preliminary design purposes and were estimated based on the factual borehole and auger probeholes drilled near the abutment and pier locations. Interpolation between boreholes and probeholes is approximate, and as such, actual footing elevations will depend on the conditions encountered at the time of construction. (The bedrock surface in Northern Ontario is known to be steeply sloping and erratic). The rock surface at the footing base must be cleared of all loosened or highly fractured rock and be inspected by a qualified geotechnical engineer to verify the Rock Mass Quality prior to placement of concrete.

### 2.2.2 Footing on Compacted Granular Pad - East and West Abutments

At the abutment locations it may be desirable to raise the grade in the area and place the footing on a Granular "A" or equivalent compacted granular pad. In order to place the granular pad, the existing topsoil and loose silty sand should be sub-excavated to bedrock level due to the potential for frost heave. The granular pad should extend horizontally a minimum of 1.0 metres beyond the plan limits of the footing and have side slopes no steeper than 1 horizontal to 1 vertical. The granular foundation pad should be compacted to 100 percent Standard Proctor Maximum Dry Density.

The bearing capacities recommended for the abutment footings placed on a compacted granular pad (based on the Ontario Highway Bridge Design Code) are as follows:

**Table 2-5 Spread Footing Capacity on Granular Pad Overlying Bedrock**

Fill Thickness (m)	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at S. L. S.
2.0	500 kPa	500 kPa
4.0	400 kPa	300 kPa
6.0	300 kPa	200 kPa

### 2.2.3 Frost Protection

Frost cover is not required for footings placed directly on bedrock. Due to the nature of bridges, a minimum frost cover of 2 m should be provided for footings placed on the granular pad.

#### 2.2.4 Sliding Resistance

The computation of the sliding resistance of the spread footings shall be carried out in accordance with of O.H.B.D.C. An unfactored friction angle,  $\phi'$ , of 32 degrees can be used for sliding along the bedrock and footing base.

If the factored resistance against sliding failure is inadequate based on friction, then the footing should be anchored into bedrock by means of keys, dowels or sockets. An unfactored coefficient of passive earth pressure,  $K_p'$ , equal to 3.7 can be used for design of a passive resistance key. Given the hardness of the bedrock, sockets and keys will likely be impractical. Developing adequate resistance against sliding of spread footings founded on the sloping bedrock at the site will likely require dowels.

#### 2.2.5 Piled Foundation

It is understood that an integral abutment system is being considered. For the abutment locations, the pile length would be approximately 4 m based on the proposed geometry of the bridge. If an integral system is used, alternatives to satisfy the MTO minimum pile length requirements or alternate systems to provide the requisite flexibility should be explored. The MTO structural office should be contacted for comments with respect to minimum pile length requirements. Piles driven to bedrock at the abutment areas can be designed based on the following Limit States design values in accordance with the O.H.B.D.C.:

**Table 2-6 Design Pile Capacities - West and East Abutments**

	<u>HP 310x79</u>	<u>HP 310x110</u>
Factored Axial Capacity at ULS	1150 kN	1600 kN
Axial Capacity at SLS	825 kN	1150 kN
Ultimate Capacity for Hiley Formula	2475 kN	3450 kN

Based on the attached borehole logs in Appendix A, Table 2-4 shows a summary of the approximate end bearing elevation at the borehole locations at which piles would be expected to be founded. Drawing 2 in Appendix A shows interpreted soil and rock subsurface profiles at the two (2) abutments and pier.

It should be noted that the elevations given in Table 2-4 are approximate. Furthermore, although not experienced in the borings put down at this site, the bedrock elevation in this part of the country is generally highly variable and may change rapidly over a very short distances.



### 2.2.6 Piled Foundations - Construction

All piles should be driven to bedrock. If piles should end above the bedrock surface within the silty sand till, pile driving can be controlled by the Hiley Formula as per MTO standards SS103-10 or SS103-11 using the ultimate pile capacities referred to in Table 2-1.

Since the boreholes indicate that the bedrock elevations are relatively uniform, the potential for irregular steeply sloping bedrock at the foundation locations is considered to be low to moderate. The bedrock in this part of the country, however, is known to be highly variable. As such, some minor problems may arise during pile seating. At some locations, the piles may have a tendency to skip over the bedrock surface resulting in alignment problems and deeper penetration. In the event that this problem occurs, somewhat longer piles may be required and in some cases piles may have to be added or replaced.

To minimize seating difficulties, rock injector points may be considered to facilitate proper seating. As a minimum, all piles must be fitted with reinforcing plates welded to the flanges as per OPSD 3301 to minimize pile damage. It is recommended that, during pile driving and upon initial contact with the bedrock, the pile driving energy should be reduced and subsequently increased incrementally until the piles have been sufficiently seated.

All lateral loads at the abutments should be supported using inclined piles.

If integral abutments are chosen for the Cranberry Marsh Road Underpass, it may be necessary to excavate into the bedrock in order to meet M.T.O. minimum pile length requirements. Given the hardness of the bedrock, blasting may be the most appropriate method for excavating into the rock to provide adequate pile length and flexibility.

## 2.3 Backfill

Backfill to abutments or retaining walls should consist of free draining granular materials such as Granular 'A' and Granular 'B' or rock fill. Computation of earth pressures shall be in accordance with Section 6.7.4 of the Ontario Highway Bridge Design Code. Unfactored properties for backfill materials are provided in the following table.

**Table 2-7 Material Types and Unfactored Properties.**

Material	Friction Angle, $\phi'$	$\gamma$ (kN/m <sup>3</sup> )	$K_a$	$K_p$	$K_o$
Granular A	35 degrees	22.5	0.27	3.7	0.43
Granular B	30 degrees	21.2	0.33	3.0	0.50
Rock Fill	35 degrees	18.0	0.27	3.7	0.43

Note:  $K_a$  is the earth pressure coefficient corresponding to the active state.

$K_p$  is the earth pressure coefficient corresponding to the passive state.

$K_o$  is the earth pressure coefficient at rest.

If rock fill is used as backfill behind abutments, the particle size should be limited to no greater than 300 mm and the backfill must be placed carefully in a manner that does not cause damage to the abutments or other structural components of the bridge.

## 2.4 Excavations

Excavations in the topsoil and sand overburden soil will be required to construct the abutment and pier footings on Bedrock. The overburden sands are classified as Type 3 soils and the maximum depth of excavation anticipated at the site is approximately 2.1 metres. As such, excavations in accordance with the Occupational Health and Safety Regulations for Construction Projects for Type 3 soils will be adequate once the groundwater in the overburden soil is removed. If groundwater seepage is encountered in the sandy soil and appropriate dewatering is not done, the soil would have to be classified as a Type 4 soil and any excavation greater than 1.2 m should be sloped to 3 horizontal to 1 vertical, starting from the base of the excavation.

Some de-watering of excavations below the ground water table may be required. Flows are expected to be small and of a quantity which can be managed using conventional sump pumping techniques in conjunction with perimeter drainage ditches. Excavation work should start from the downstream ends of the excavation to facilitate drainage and enhance the excavation procedures.

## 2.5 Approach Embankments

No stability problems are anticipated for the approach embankments founded on bedrock or on the overlying silty sand soils at the site. All topsoil and compressible organics (if present) must be removed from the plan limit of the approach embankments. If rockfill is used to construct the approach embankments, the side slopes and forward slopes should be constructed a gradient of 1.25H(minimum):1V. If Granular 'A' or Granular 'B', the forward and side slopes should be constructed at 2H(minimum):1V.

## 2.6 General

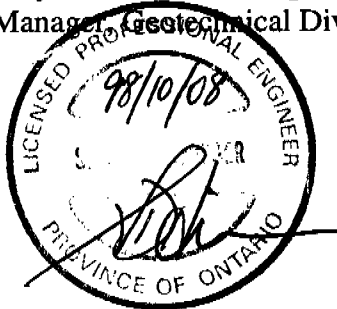
The information presented in this report is based on a limited investigation designed to provide information to support an overall assessment of the current geotechnical conditions at the site of the proposed Cranberry Marsh Road/Hwy. 69 Bridge. The conclusions presented in this report reflect site conditions existing at the time of the investigation. It is noted that the soil boundaries indicated on the Borehole Logs are inferred from discontinuous sampling and observations during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change.

This report has been prepared by Stephen S. M. Cheng and Sean Hinchberger and reviewed by Stan Gonsalves. Chi Ng coordinated the field investigation and Indulis Dumpus performed the fieldwork.

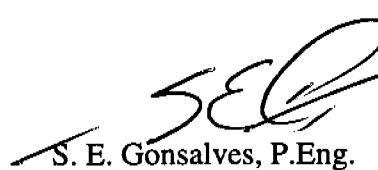
We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

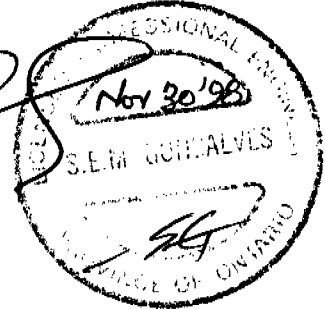
**Trow Consulting Engineers Ltd.**

Lloyd Gonsalves, P.Eng.  
Manager, Geotechnical Division



Sean Hinchberger, Ph.D., P. Eng.  
Project Engineer

  
S. E. Gonsalves, P.Eng.  
Vice-President, GTR



## EXPLANATION OF TERMS AND SYMBOLS

**N VALUE - STANDARD PENETRATION TEST (SPT)** N VALUE IS THE NUMBER OF BLOWS REQUIRED TO DRIVE A STANDARD 51-mm O.D. SPLIT SPOON SAMPLER 0.3 m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3 m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED AS  $\bar{N}$ .

**DYNAMIC CONE PENETRATION TEST - CONTINUOUS PENETRATION OF A CONICAL STEEL POINT** (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

**CONSISTENCY:** COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ( $C_u$ ) AS FOLLOWS:

$C_u$ (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

**DENSENESS:** COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS / 0.3 m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH:

**RECOVERY:** SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

**MODIFIED RECOVERY:** SUM OF THOSE INTACT CORE PIECES, 100mm + IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

**JOINTING AND BEDDING:**

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

### ABBREVIATIONS AND SYMBOLS

#### FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

#### STRESS AND STRAIN

$u$	kPa	PORE WATER PRESSURE
$r_u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
$\nu$	1	POISSON'S RATIO
$E$	kPa	MODULUS OF LINEAR DEFORMATION
$G$	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

#### MECHANICAL PROPERTIES OF SOIL

$m_v$	$kPa^{-1}$	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_{\alpha}$	1	COEFFICIENT OF SECONDARY CONSOLIDATION
$c_v$	$m^2/s$	COEFFICIENT OF CONSOLIDATION
$H$	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
$U$	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_r$	1	SENSITIVITY = $\frac{C_u}{\tau_r}$

#### PHYSICAL PROPERTIES OF SOIL

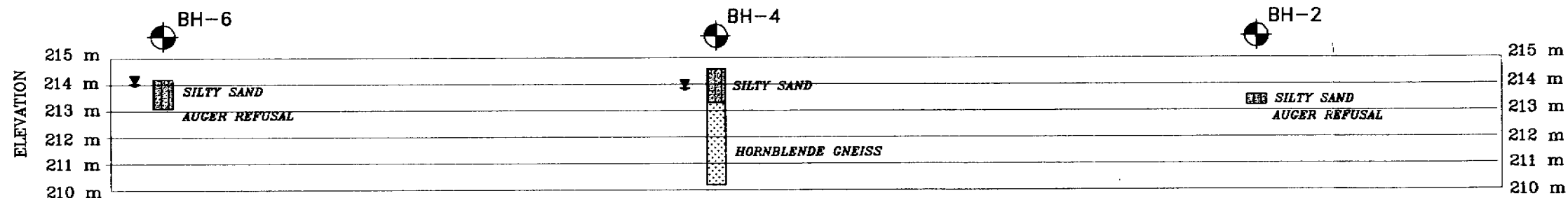
$\rho_s$	$kg/m^3$	DENSITY OF SOLID PARTICLES	$c$	VOID RATIO	$e_{min}$	%	VOID RATIO IN MOST DENSE STATE
$\gamma_s$	$kg/m^3$	UNIT WEIGHT OF SOLID PARTICLES	$n$	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	$kg/m^3$	DENSITY OF WATER	$w$	%	$D$	mm	GRAIN DIAMETER
$\gamma_w$	$kg/m^3$	UNIT WEIGHT OF WATER	$S_r$	%	$D_s$	mm	n PERCENT - DIAMETER
$\rho$	$kg/m^3$	DENSITY OF SOIL	$w_L$	%	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	$kg/m^3$	BULK UNIT WEIGHT OF SOIL	$w_p$	%	$h$	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	$kg/m^3$	DENSITY OF DRY SOIL	$w_s$	%	$q$	$m^3/s$	RATE OF DISCHARGE
$\gamma_d$	$kg/m^3$	UNIT WEIGHT OF DRY SOIL	$I_p$	%	$v$	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	$kg/m^3$	DENSITY OF SATURATED SOIL			$i$	1	HYDRAULIC GRADIENT
			$I_L$	1			
$\gamma_{sat}$	$kg/m^3$	UNIT WEIGHT OF SATURATED SOIL			$k$	m/s	HYDRAULIC CONDUCTIVITY
			$I_c$	1			
$\rho'$	$kg/m^3$	DENSITY OF SUBMERGED SOIL			$j$	$kN/m^2$	SEEPAGE FORCE
$\gamma'$	$kg/m^3$	UNIT WEIGHT OF SUBMERGED SOIL	$e_{max}$	%			



Appendix A:  
Plan Location of Boreholes  
&  
Borehole Logs

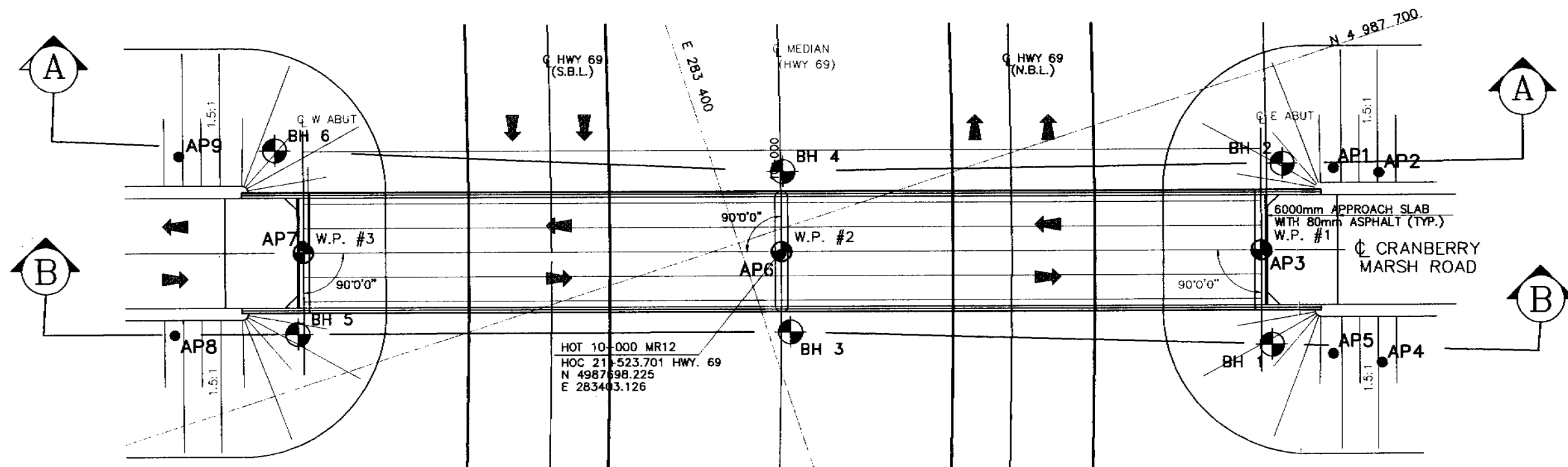
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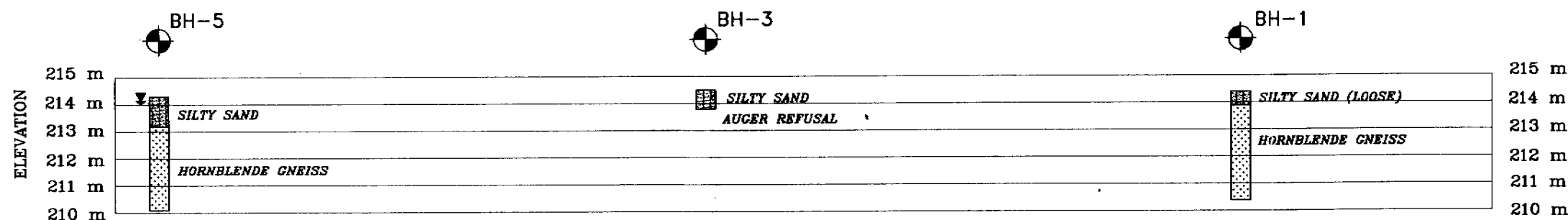
## SECTION A-A

SCALE: HORIZONTAL: 1:400  
VERTICAL: 1:200



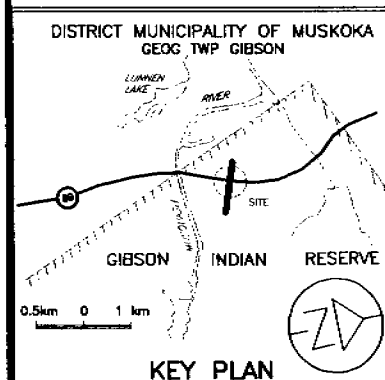
## PLAN

SCALE: 1:400



## SECTION B-B

SCALE: HORIZONTAL: 1:400  
VERTICAL: 1:200

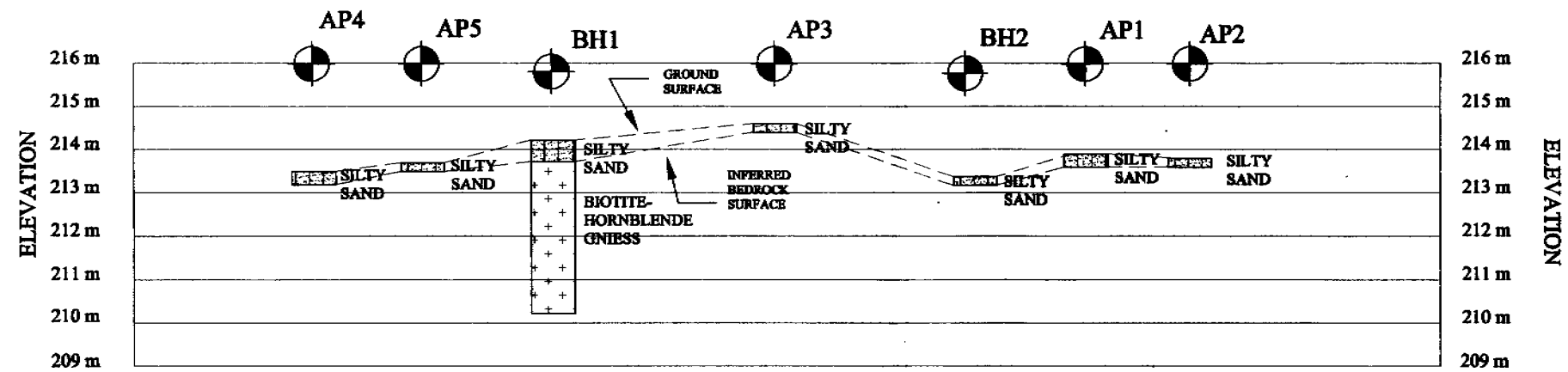


## KEY PLAN

**TROW CONSULTING ENGINEERS LTD.**  
BRAMPTON, ONTARIO

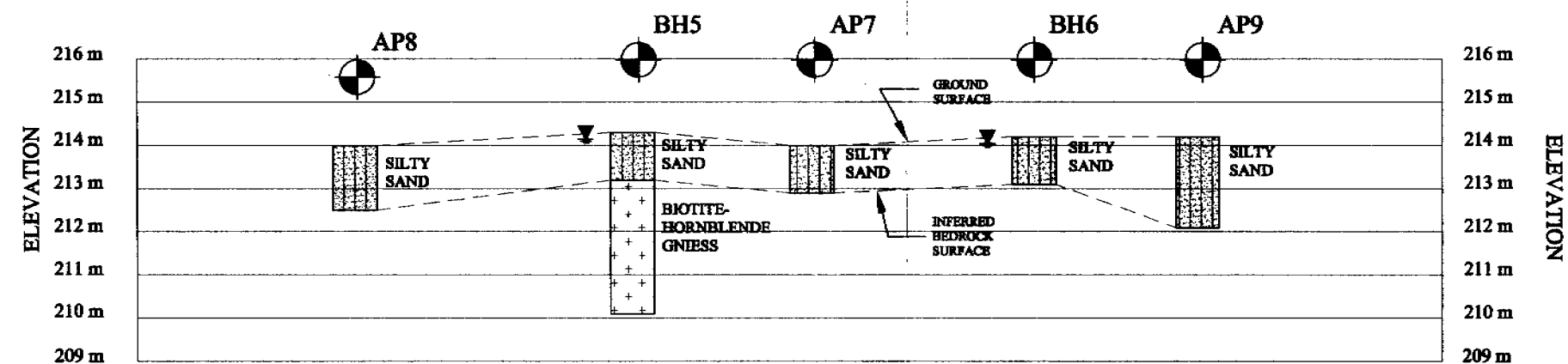
**CRANBERRY MARSH ROAD  
AND HWY 69  
BOREHOLE LOCATION PLAN  
W.P. 690-93-01**  
TORONTO ONTARIO

PROJECT NO.: BR-11546-A/C  
SCALE: AS NOTED  
DRAWN BY: J.I.  
CHECKED BY: S.C.  
DATE: May 28, 1998  
DRAWING NO.: 1

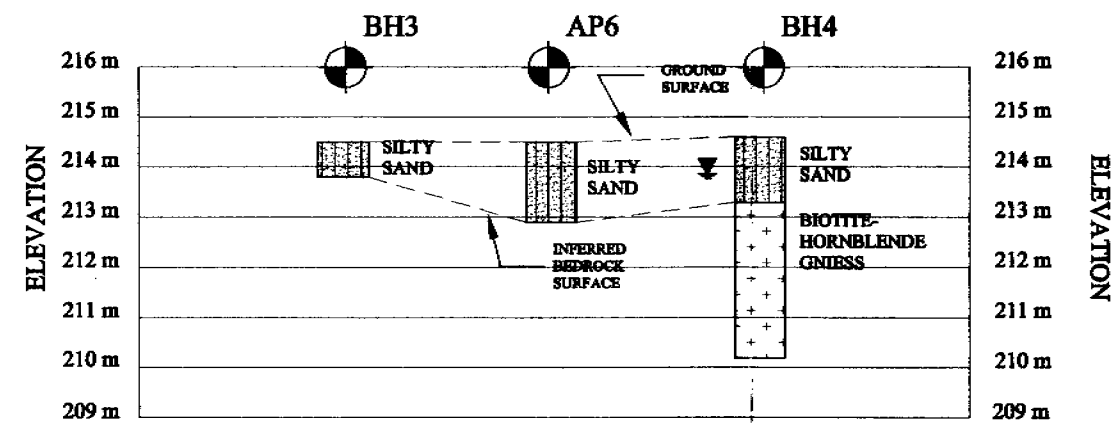


EAST ABUTMENT

NOTE:  
THE STRATIGRAPHY AND SOIL TYPES HAVE BEEN ESTABLISHED AT THE BOREHOLE LOCATIONS ONLY. INTERPOLATION BETWEEN BOREHOLES IS APPROXIMATE, AND AS SUCH, MAY BE SUBJECT TO CONSIDERABLE ERROR.



WEST ABUTMENT



CENTRE PIER

**TROW CONSULTING ENGINEERS LTD.**  
BRAMPTON, ONTARIO

CRANBERRY MARSH ROAD  
HWY 69  
CROSS - SECTIONS  
W.P. 690-93-01

TORONTO

ONTARIO

PROJECT NO.:	BR-11546-A/C
SCALE:	1: 150
DRAWN BY:	J.I.
CHECKED BY:	S.C.
DATE:	MAY 28, 1998
DRAWING NO.:	2

# RECORD OF BOREHOLE 1

## CRANBERRY MARSH ROAD

1 OF 1

**METRIC**

W.P. 690-93-01

LOCATION EAST ABUTMENT - 4 987 678.4N 283 438.6E

ORIGINATED BY I.D.

DIST 52 HWY 69

BOREHOLE TYPE SPT AND B SIZE CORE HOLLOW STEM AUGERS

COMPILED BY S.C.

DATUM GEODETIC

DATE November 17, 1997

CHECKED BY C.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
214.4	GROUND SURFACE														
0.0	SILTY SAND - fine grained, brown, moist, loose.		1	SPT	5										
213.9						214									
0.5	BIOTITE HORNBLLENDE GNEISS		1	RC											REC 100%RQD 90%
						213									
			2	RC		212									REC 100%RQD 88%
						211									
			3	RC											REC 100%RQD 100%
210.4	END OF BOREHOLE														
4.0	NOTE: 1. BOREHOLE DRY UPON COMPLETION OF DRILLING														

# RECORD OF BOREHOLE 2

## CRANBERRY MARSH ROAD

1 OF 1

**METRIC**

W.P. 690-93-01 LOCATION EAST ABUTMENT - 4 987 692.2N 283 444.0E ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE SPT HOLLOW STEM AUGERS COMPILED BY S.C.  
 DATUM GEODETIC DATE November 17, 1997 CHECKED BY S.H.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40					
213.6	GROUND SURFACE													
0.0	75 mm TOPSOIL over SILTY SAND - fine grained, light brown, moist, very loose.		1	SS	3									
213.4	END OF BOREHOLE													
0.2	AUGER REFUSAL ON ASSUMED BEDROCK AT 0.2 M.  NOTE: 1. BOREHOLE DRY UPON COMPLETION OF DRILLING 2. THREE MORE HOLES WERE DRILLED IN THE VICINITY AND THE BEDROCK DEPTH VARIED FROM 0.2 TO 0.5 M													

# RECORD OF BOREHOLE 3

## CRANBERRY MARSH ROAD

1 OF 1

**METRIC**

W.P. 690-93-01 LOCATION CENTRE PIER - 4 987 691.8N ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE SPT HOLLOW STEM AUGERS COMPILED BY S.C.  
 DATUM GEODETIC DATE November 17, 1997 CHECKED BY C.N.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				CONE PENETRATION TEST			PLASTIC LIMIT			NATURAL MOISTURE CONTENT			LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	20	40	60	80	wp	w	wl	20	40	60	80		
214.5	GROUND SURFACE																							
0.0	50 mm TOPSOIL over SILTY SAND - fine grained, brown to light brown, compact.		1	SS	7																			
214																								
213.8	END OF BOREHOLE																							
0.7	AUGER REFUSAL ON ASSUMED BEDROCK AT 0.7 m.  NOTE: 1. BOREHOLE DRY UPON COMPLETION OF DRILLING																							

METRIC

ORIGINATED BY I.D.

COMPILED BY S.C.

CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION								
ELEV.	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80						SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)			
DEPTH								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB VANE															
214.6	GROUND SURFACE																							
0.0	250 mm TOPSOIL over SILTY SAND - fine grained, light brown changing to grey at lower depth, moist, loose to dense.		1	SS	6																			
213.3			2	SS	34																			
1.4	BIOTITE HORNBLLENDE GNEISS		1	RC																				
210.2			2	RC																				
4.5	END OF BOREHOLE																							
NOTE: 1. DEPTH TO FREE STANDING WATER = 0.73 M																								

# RECORD OF BOREHOLE 5

## CRANBERRY MARSH ROAD

1 OF 1

METRIC

W.P. 690-93-01

LOCATION WEST ABUTMENT - 4 987 704.2N 283 363.5E

ORIGINATED BY I.D.

DIST 52 HWY 69

BOREHOLE TYPE SPT AND B SIZE CORE HOLLOW STEM AUGERS

COMPILED BY S.C.

DATUM GEODETIC

DATE November 19, 1997

CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp — $\omega^w$ — wl				
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)				
								UNCONFINED QUICK TRIAXIAL      FIELD VANE LAB VANE				20 40 60 80				
214.3	GROUND SURFACE															
0.0	~150 mm TOPSOIL over SILTY SAND - fine grained, brown, wet, loose.		1	SS	7		214									
			2	SS	6										0 72 28	
213.2	BIOTITE HORNBLENDE GNEISS															
1.1							213									
			1	RC											REC 100%RQD 92%	
							212									

# RECORD OF BOREHOLE 6

## CRANBERRY MARSH ROAD

1 OF 1

**METRIC**

W.P. 690-93-01

LOCATION WEST ABUTMENT - 4 987 719.1N 283 366.5E

ORIGINATED BY I.D.

DIST 52

HWY 69

BOREHOLE TYPE SPT HOLLOW STEM AUGERS

COMPILED BY S.C.

DATUM GEODETIC

DATE November 19, 1997

CHECKED BY C.N.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE BLOWS/0.3m			20	40	60	80	wp	w	wl		
214.2	GROUND SURFACE														
0.0	~200 mm TOPSOIL over SILTY SAND - fine grained, grey to reddish brown, moist to very moist, loose.		1	SS	9										
			2	SS	8										
213.1	END OF BOREHOLE														
1.1	AUGER REFUSAL ON ASSUMED BEDROCK AT 1.1 m.  NOTE: 1. DEPTH TO FREE STANDING WATER = 0.15 M														



# RECORD OF BOREHOLE AP1 CRANBERRY MARSH ROAD

1 OF 1

**METRIC**

W.P. 690-93-01

LOCATION EAST ABUTMENT - 4 987 690.5N 283 448.0E

ORIGINATED BY I.D.

DIST 52

HWY 69

BOREHOLE TYPE PROBE HOLE AUGERS

COMPILED BY S.C.

DATUM GEODETIC

DATE November 17, 1997

CHECKED BY C.N.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION  GR SA (SI & CL)		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa								
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB VANE						WATER CONTENT (%)	w <sub>p</sub>   w <sub>L</sub>
						20	40	60	80	20	40	60	80			
213.9 0.0	GROUND SURFACE		1	AS												
213.6 0.3	END OF PROBEHOLE															
	AUGER REFUSAL ON ASSUMED BEDROCK AT 0.3 m.															
	NOTE: 1. SOIL STRATIGRAPHY INFERRED FROM AUGER CUTTINGS.															

# RECORD OF BOREHOLE AP2 CRANBERRY MARSH ROAD

1 OF 1

METRIC

W.P. 690-93-01

LOCATION EAST ABUTMENT - 4 987 689.0N 283 451.5E

ORIGINATED BY I.D.

DIST 52

HWY 69

BOREHOLE TYPE PROBE HOLE AUGERS

COMPILED BY S.C.

DATUM GEODETIC

DATE November 17, 1997

CHECKED BY C.N.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	CONE PENETRATION TEST						
						20	40	60	80					
213.8	GROUND SURFACE													
0.0	50 mm ORGANICS over SILTY SAND		1	AS										
213.6	END OF PROBEHOLE													
0.3	AUGER REFUSAL ON ASSUMED BEDROCK AT 0.3 m.													
	NOTE: 1. SOIL STRATIGRAPHY INFERRED FROM AUGER CUTTINGS.													

# RECORD OF BOREHOLE AP3 CRANBERRY MARSH ROAD

1 OF 1

**METRIC**

W.P. 690-93-01 LOCATION EAST ABUTMENT - 4 987 685.8N 283 440.6E ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE PROBE HOLE AUGERS COMPILED BY S.C.  
 DATUM GEODETIC DATE November 17, 1997 CHECKED BY C.N.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60					
214.6 0.0	GROUND SURFACE														
214.4 0.2	SILTY SAND		1	AS											
	END OF PROBEHOLE														
	AUGER REFUSAL ON ASSUMED BEDROCK AT 0.2 m.														
	NOTE: 1. SOIL STRATIGRAPHY INFERRED FROM AUGER CUTTINGS.														

# RECORD OF BOREHOLE AP4

## CRANBERRY MARSH ROAD

1 OF 1

**METRIC**

W.P. 690-93-01

LOCATION EAST ABUTMENT - 4 987 674.1N 283 446.8E

ORIGINATED BY J.D.

DIST 52

HWY 69

BOREHOLE TYPE PROBE HOLE AUGERS

COMPILED BY S.C.

DATUM GEODETIC

DATE November 17, 1997

CHECKED BY C.N.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV.	DEPTH	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	CONE PENETRATION TEST							
						20    40    60    80 UNCONFINED QUICK TRIAXIAL    FIELD VANE LAB VANE				wp    —    w <sub>p</sub> —    w <sub>l</sub> WATER CONTENT (%)				20    40    60    80 kN/m <sup>3</sup> GR    SA    (SI & CL)	
213.5															
0.0			1	AS											
213.2															
0.3															

GROUND SURFACE

50 mm ORGANICS over SILTY SAND

END OF PROBEHOLE

AUGER REFUSAL ON ASSUMED BEDROCK AT 0.3 m.

NOTE:

1. SOIL STRATIGRAPHY INFERRED FROM AUGER CUTTINGS.

# RECORD OF BOREHOLE AP5 CRANBERRY MARSH ROAD

1 OF 1

**METRIC**

W.P. 690-93-01 LOCATION EAST ABUTMENT - 4 987 676.1N 283 443.2E ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE PROBE HOLE AUGERS COMPILED BY S.C.  
 DATUM GEODETIC DATE November 17, 1997 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)								
								UNCONFINED QUICK TRIAXIAL		FIELD VANE LAB VANE										
						20	40	60	80	wp	w	wl	20	40	60	80	kN/m³	GR	SA	(SI & CL)
213.7	GROUND SURFACE																			
0.0	50 mm ORGANICS over SILTY SAND		1	AS																
213.5	END OF PROBEHOLE																			
0.2	AUGER REFUSAL ON ASSUMED BEDROCK AT 0.2 m.  NOTE: 1. SOIL STRATIGRAPHY INFERRED FROM AUGER CUTTINGS.																			

# RECORD OF BOREHOLE AP6

## CRANBERRY MARSH ROAD

1 OF 1

**METRIC**

W.P. 690-93-01 LOCATION CENTRE PIER - 4 987 698.0N 283 403.0E ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE PROBE HOLE AUGERS COMPILED BY S.C.  
 DATUM GEODETIC DATE November 17, 1997 CHECKED BY C.N.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT wp	NATURAL MOISTURE CONTENT w	LIQUID LIMIT wl	UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60					
214.5	GROUND SURFACE														
0.0	250 mm ORGANIC SAND over SILTY SAND		1	AS											
212.9															
1.6	END OF PROBEHOLE AUGER REFUSAL ON ASSUMED BEDROCK AT 1.6 m.  NOTE: 1. SOIL STRATIGRAPHY INFERRED FROM AUGER CUTTINGS.														

# RECORD OF BOREHOLE AP7

## CRANBERRY MARSH ROAD

1 OF 1

**METRIC**

W.P. 690-93-01 LOCATION WEST ABUTMENT - 4 987 71.05N 283 366.2E ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE PROBE HOLE AUGERS COMPILED BY S.C.  
 DATUM GEODETIC DATE November 19, 1997 CHECKED BY C.N.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w		
214.0	GROUND SURFACE														
0.0	~150 mm TOPSOIL over SILTY SAND		1	AS											
212.8															
1.2	END OF PROBEHOLE														
	AUGER REFUSAL ON ASSUMED BEDROCK AT 1.2 m.														
	NOTE: 1. SOIL STRATIGRAPHY INFERRED FROM AUGER CUTTINGS.														

# RECORD OF BOREHOLE AP8 CRANBERRY MARSH ROAD

1 OF 1

**METRIC**

W.P. 690-93-01 LOCATION WEST ABUTMENT - 4 987 707.3N 283 354.1E ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE PROBE HOLE AUGERS COMPILED BY S.C.  
 DATUM GEODETIC DATE November 19, 1997 CHECKED BY C.N.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80					
214.4 0.0	GROUND SURFACE															
	300 mm ORGANICS over SILTY SAND															
			1	AS												
212.9 1.5	END OF PROBEHOLE															
	AUGER REFUSAL ON ASSUMED BEDROCK AT 1.5 m.															
	NOTE: 1. SOIL STRATIGRAPHY INFERRED FROM AUGER CUTTINGS.															





# RECORD OF BOREHOLE AP9 CRANBERRY MARSH ROAD

1 OF 1

**METRIC**

W.P. 690-93-01

LOCATION WEST ABUTMENT - 4 987 721.1N 283 359.0E

ORIGINATED BY I.D.

DIST 52 HWY 69

BOREHOLE TYPE PROBE HOLE AUGERS

COMPILED BY S.C.

DATUM GEODETIC

DATE November 19, 1997

CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT wp	NATURAL MOISTURE CONTENT w	LIQUID LIMIT wl	UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80					
214.2	GROUND SURFACE															
0.0	~250 mm TOPSOIL/ORGANICS over SILTY SAND - light brown, wet, compact.															
			1	AS												
212.1																
2.1	END OF PROBEHOLE AUGER REFUSAL ON ASSUMED BEDROCK AT 2.1 m.  NOTE: 1. SOIL STRATIGRAPHY INFERRED FROM AUGER CUTTINGS.															



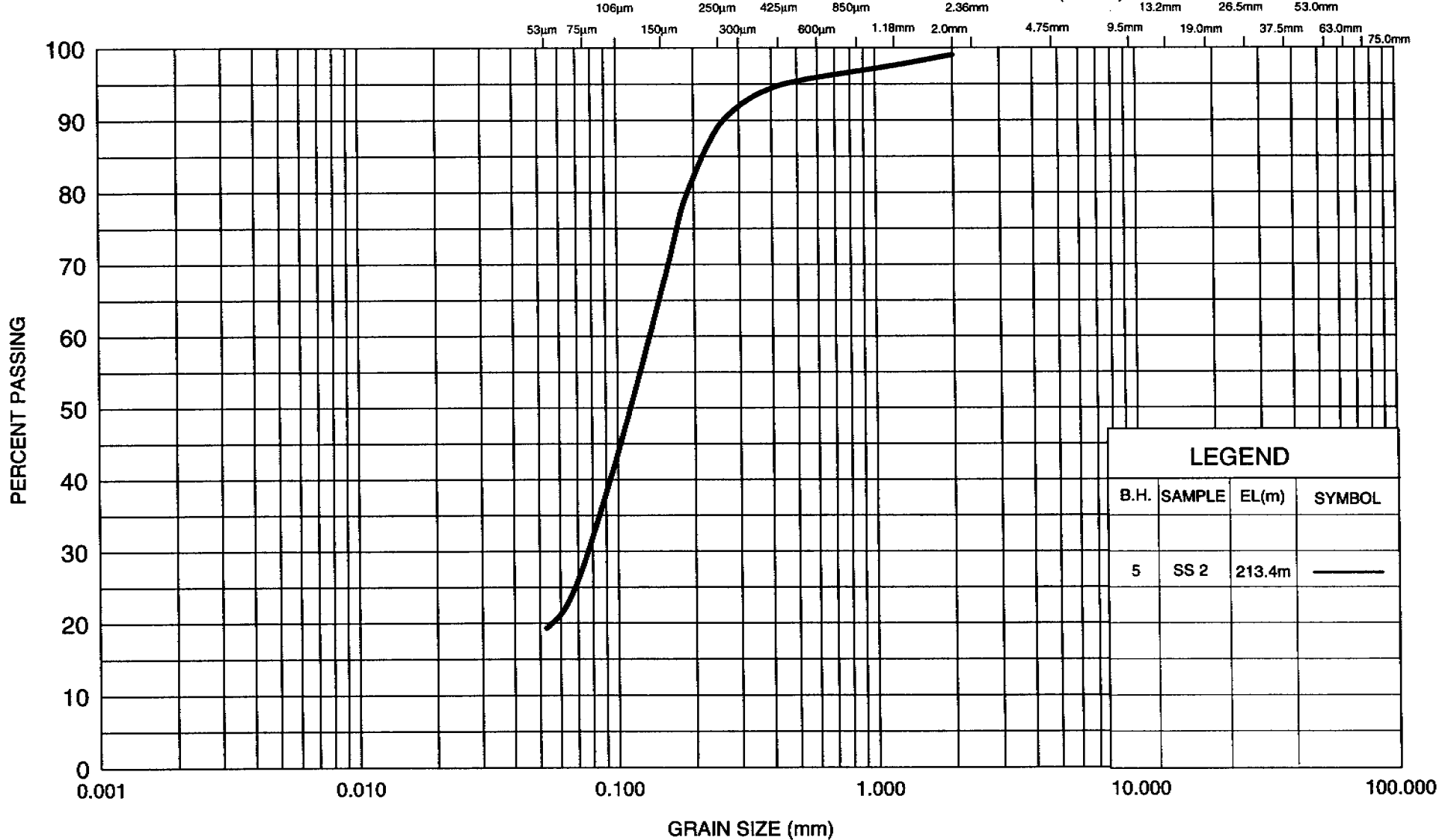
## Appendix B: Laboratory Results

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# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

**METRIC**

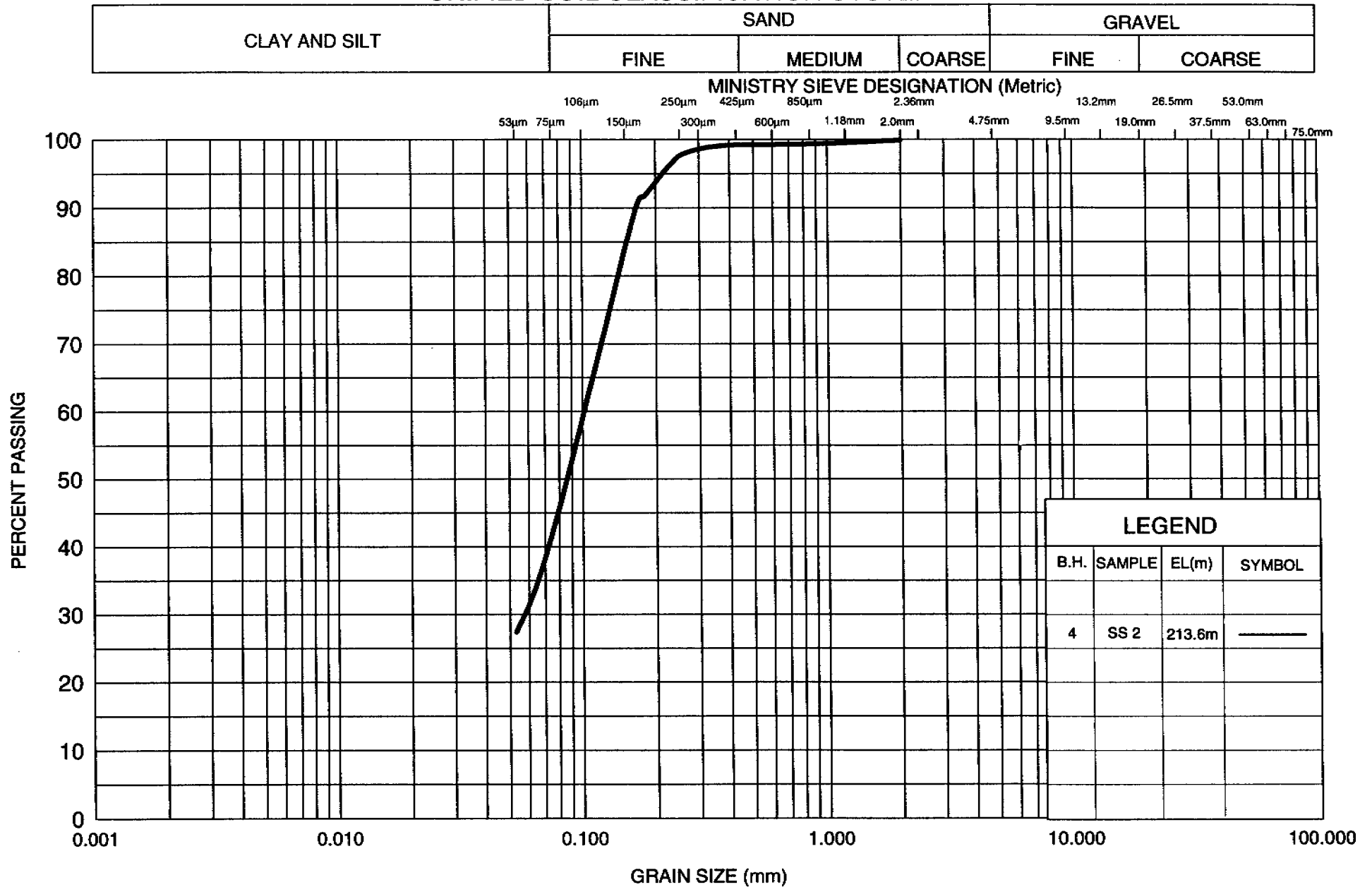
## GRAIN SIZE DISTRIBUTION

B.H. 5 - SAMPLE 2: SAND - some silt, brown

**FIGURE 1**

W.P. 690-93-01

# UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

**METRIC**

## GRAIN SIZE DISTRIBUTION

B.H. 4 - SAMPLE 2: SAND and SILT- brown, moist, compact

**FIGURE 2**

W.P. 690-93-01



# memorandum

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To: Mike Pearsall, P. Eng.  
Senior Project Engineer  
Northern Region

From: Pavements and Foundations Section  
Room 232, Central Building  
Downsview, Ontario

Re: Revised Final Foundation Investigation Reports  
Cranberry Marsh Road Underpass, Site 42-318  
W.P. 690-93-01, Hwy 69  
GWP 217-89-00, District 52, Huntsville

1998 03 22

We have conceptually reviewed the revised final Foundation Investigation Reports dated November 30, 1998 for the above project produced by Trow Consulting Engineers Ltd. for R.V. Anderson Associates Ltd., to determine the consultant's performance in providing the deliverables as would be required by MTO for similar consultant assignments. The accuracy of the subsurface information and the adequacy and the technical aspects of the recommendations remain the responsibility and liability of the consultant. The Ministry assumes no responsibility or liability for these aspects of the reports. These aspects will be reviewed in order to assess the consultant's performance in this assignment upon implementation of the recommendations in the design and upon review of the performance of the foundations for the completed project. Following are our comments:

All the comments made in our memo dated 1998 09 30 are incorporated in the revised final foundation report. We have no further comments.

If you have any questions, please advise.

A handwritten signature in cursive script, appearing to read "K. Ahmad".

K. Ahmad, P. Eng.  
Foundation Engineer

For

T.C. Kim, P. Eng.  
Senior Foundation Engineer

cc: T. Kazmierowski  
P. Stuart  
I. Husain



# memorandum

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To: Mike Pearsall, P. Eng.  
Senior Project Engineer  
Northern Region

1998 09 30

From: Pavements and Foundations Section  
Room 223, Central Building  
Downsview, Ontario

Re: Final Foundation Investigation Reports  
Cranberry Marsh Road Overpass, Site 42-318 Underpass  
W.P. 690-93-01, Hwy 69  
GWP 217-89-00, District 53, New Liskeard

We have conceptually reviewed the final Foundation Investigation Reports dated June 2, 1998 (we received the report on September 24, 1998) for the above project produced by Trow Consulting Engineers Ltd. for R.V. Anderson Associates Ltd., to determine the consultant's performance in providing the deliverables as would be required by MTO for similar consultant assignments. The accuracy of the subsurface information and the adequacy and the technical aspects of the recommendations remain the responsibility and liability of the consultant. The Ministry assumes no responsibility or liability for these aspects of the reports. These aspects will be reviewed in order to assess the consultant's performance in this assignment upon implementation of the recommendations in the design and upon review of the performance of the foundations for the completed project. Following are our comments:

- ✓ 1. Page 8: As per the OHBDC 3<sup>rd</sup> Edition, the term "Type II" is not used anymore for the SLS values. Also, as per the latest version of the OHBDC, the term "Geotechnical resistance" should be used instead of "Bearing Capacity and Axial Capacity". The Consultant should refer to the 3<sup>rd</sup> Edition of OHBDC, 91.
- ✓ 2. Section 2.2.5: The site does not meet the minimum pile length requirements for the integral abutments. If integral abutments are considered for this bridge then, the Structural Office should be contacted for comments.
- ✓ 3. The notes on the map read "Anger refusal". This should be changed to "Auger refusal".

- ✓ 4. The project is called as an Overpass. According to the definition, it should be called an Underpass (a major highway going under a minor highway).

If you have any questions, please advise.



K. Ahmad, P. Eng  
Foundation Engineer

For

T.C. Kim, P. Eng.  
Senior Foundation Engineer

cc: T. Kazmierowski  
P. Stuart  
I. Husain

file:c:\data\wpwin60\2178900.mik4.wpd





# MEMORANDUM

2

**Engineering Materials Office**  
Room 313, Central Building, Downsview  
Tel. (416) 235-3731 Fax. (416) 235-5240

**To:** Paul Lacoarer  
Senior Project Engineer  
Northern Region Planning and Design

**Date:** May 1, 1998

**From:** Pavements and Foundations Section  
Rm. 315, Central Building, Downsview

**Subject:** **Draft Foundation Investigation Reports**  
**1. Crossing at Cranberry Marsh Road**  
**2. Musquast River Bridge NB.**  
**WP 217-89-00, Hwy. 69, Musquach river N'ly 8.9 km to Tower Road**  
**District 52 - Huntsville**

wp. 690-93-01

The preliminary reports submitted by TROW Consulting Engineers Ltd. have been reviewed for format only. The accuracy and completeness of these reports remain the responsibility of the consultant. We have the following comments:

## CRANBERRY MARSH ROAD

1. Groundwater levels should be indicated in the Record of Borehole Sheets. In addition it should be clarified as to whether a dewatering system will be required. In Section 1.5 it is stated that water levels were not stabilized, possibly perched and may not be representative, however section 2.4 recommends that some de-watering may be required. As the native material is susceptible to the water conditions, water levels should be clarified.
2. A profile of the bedrock should be included in the report, in particular if the integral abutment option with pile driving is chosen. Together with the probe hole data this would clarify conditions.
3. Due to the shallow bedrock surface the possibility of utilizing an integral abutment should be thoroughly examined. It should be checked as to whether minimum preaugered hole and pile lengths can be met.
4. Grain size distribution information should be included in the borehole logs and within the subsoil conditions (Section 1.4) as has been included with MTO reports in the past.
5. It is stated in the report that an 8 m grade raise at the abutment locations is expected. However, no comments are provided in regards to slope stabilization. Forward and side slope recommendations should be addressed.

6. In regards to the Footings on Compacted Pad option (Section 2.2.2) the pad thickness and corresponding capacities should be provided. While at the discretion of the report writer, due to the relative shallow nature of the overburden and the susceptible nature of the material to the presence of water it may be prudent to remove the sand down to the bedrock depth and build a pad up to the required elevation.
7. The quality of the footing base should be inspected as correctly stated in the report.
8. For contract purposes an E-Plan as per MTO standards should be included.

## MUSQUASH

1. Grain Size Gradations should be reported for the clay material in the borehole logs.
2. Due to the variable nature of the bedrock, a profile should be provided. Additional borings or probings are required if there are areas where uncertainty exists. The option to conduct an additional investigation was mentioned in the report (Section 2.2.1) however due to the impact the bedrock location has on choosing the appropriate foundation solution, this should be clarified to the greatest extent possible so that an informed decision could be made.
3. The thickness of the surcharge load should be provided (equivalent to 18 kPa).
4. For contract purposes an E-Plan as per MTO standards should be included.

M. Michalek, P. Eng.  
Foundation Engineer  
For:  
T. C. Kim, P. Eng.  
Sr. Foundation Engineer