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**Embankment Design Report-  
Highway 69 Four Lane  
Construction from 0.4 km South  
of the Musquash River,  
Northerly 8.9 km to Tower Road**

W.P. ? 217-89-00

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# 1. Introduction

## 1.1 General

This report presents the results of a detailed embankment settlement analysis and preliminary embankment design by Trow Consulting Engineers Ltd. (Trow) for R.V. Anderson Associates Ltd. (R.V. Anderson) for the Four Lane Extension of Highway 69 from 0.4 km south of the Musquash River, northerly 8.9 km to Tower Road. The project is located within the Ministry of Transportation, Ontario, (MTO) Northern Region, District 52, Huntsville. A design settlement criteria of less than 25 mm of consolidation settlement after the application of the pavement base course has been established by the MTO for the Four Lane Extension of Highway 69. This report presents the methodology, results and preliminary embankment designs required to achieve the design settlement objectives in accordance with the terms of reference outlined in Section 1.2 below.

The preliminary embankment design recommendations contained herein supercede prior recommendations made in Trow Report: Foundation Investigation Report, Northbound and Southbound Lanes, Approach Embankments, Musquash River, Trow Consulting Engineers Ltd., January, 1998. Section 2 of this report describes the methodology and Section 3 provides a brief summary of subsurface conditions at the site which will affect long term embankment behaviour. Sections 4, 5 and 6 of this report describe the preliminary embankment design options. In particular, Section 4 presents designs based on a six (6) month construction period and Sections 5 and 6 present preliminary design options based on four (4) and two (2) month construction periods, respectively.

## 1.2 Terms of Reference

The terms of reference for this report include the following:

1. Review and compile all existing subsurface information for the current project,
2. Identify soil deposits encountered along the new highway alignment which may be susceptible to long term settlements,
3. Derive reasonable soil properties for settlement susceptible soils based on the available laboratory data,
4. Calculate settlements on a section by section basis,
5. Identify sections of the proposed new highway where calculated settlements exceed the design settlement criteria of less than 25 mm of consolidation settlement after the application of the asphalt base course,



6. Design embankment surcharge schemes, wick drain or sand drain spacings and excavation options such that calculated embankment settlements meet the settlement criteria for this project and
7. Provide design options based on six month, four month and two month construction surcharge periods.

*Trow is not a part*

## 2. Methodology

### 2.1 General

As noted in Section 1 of this report, a design criteria of less than 25 mm of consolidation settlement after the application of the pavement base course has been established for the Highway 69 road embankments from 0.4 km south of the Musquash River, northerly 8.9 km to Tower Road. To meet the imposed settlement criteria, a design procedure was developed which included limit equilibrium calculations to assess embankment stability and standard settlement calculations to estimate clay compression and rate of consolidation.

Section 2.2 below summarizes all reference material used for the present report and the available subsurface information obtained during drilling at the site is summarized on Drawings 1 through 13, inclusive. Figures 1 through 4 summarize the available laboratory test results for the clayey silt to silty clay deposits encountered at the site during drilling. For the purpose of design, it has been assumed that all near surface organic deposits will be excavated and removed from below the plan limits of the roadway and that time dependent settlements will result only from compression of the underlying clayey soil deposits. Refer to Reference Document No. 6 in Section 2.2, below, for detailed subsurface logs and for the depth of organic soils.

The following is a description of all reference materials, measured soil properties and design soil properties used for the embankment analyses. Section 3 of this report summarizes the location of compressible clayey subsurface soils which were identified during drilling and which may exhibit significant time dependent settlements. The preliminary embankment designs and surcharge schemes are described in Sections 4, 5 and 6 of this report.

### 2.2 Reference Documents

The following related reports were used to compile a summary of soil properties at the site:

1. Foundation Investigation Report, Approach Embankments, Southbound Lanes, Musquash River, MTO Foundation Section, March 1993.
2. Foundation Investigation Report, Northbound and Southbound Lanes, Approach Embankments, Musquash River, Trow Consulting Engineers Ltd., January, 1998.
3. Foundation Investigation Report, Musquash River, Northbound Lanes Replacement Bridge, Site 42-46N, Trow Consulting Engineers Ltd., January, 1998.
4. Foundation Investigation Report, Musquash River, Southbound Lanes Bridge, Site 42-46S, MTO Foundation Section, March 1993

5. Foundation Investigation Report, Moon River, Southbound Lanes Bridge, Site 42-26S, Trow Consulting Engineers Ltd., January, 1998
6. Pavement Design Report, Highway 69, From 0.4 km South of the Musquash River, Northerly 8.9 km to Tower Road, Grading, WP-217-89-00, District 52, Huntsville. Volumes 1, 2 and 3, Trow Consulting Engineers Ltd., January, 1998.

### 2.3 Design Undrained Shear Strength Profile

The available field vane shear strength data contained in the reference materials listed in Section 2.2 of this report is summarized in Figure 1. In general, the clayey silt to silty clay soils encountered at the site varied from very soft to stiff based on the range of measured in-situ vane shear strengths. For the purpose of embankment design, however, the design profile shown in Figure 1 was selected and a factor of safety of 1.3 against short term failure was adopted such that the factored undrained shear strength profile encompassed all measured field vane shear strengths.

For the purpose of embankment design, undrained conditions were assumed for all clayey silt to silty clay soils at the site during the initial construction period. To estimate the factor of safety against embankment failure, the upper 2 metres of the clayey silt to silty clay soil deposits was assumed to have an initial undrained shear strength of 25 kPa at the top of the deposit. The undrained shear strength was then assumed to decrease at a rate of 6.0 kPa/m to 13 kPa at a depth of 2 metres. Below the initial 2 metres of clayey silt to silty clay, the undrained shear strength was assumed to increase from 13 kPa at a depth of 2 metres below the top of the deposit to approximately 32 kPa at a depth of 12 metres. Table 2-1 summarizes the undrained shear strength properties adopted for the clayey silt to silty clay soils at the site.

**Table 2-1 Summary of Undrained Shear Strength Profile for Clayey Soils at the Site.**

Depth below top of Clayey Silt to Silty Clay Deposit (m)	Shear Strength at Top of Layer, $C_{u0}$ (kPa)	Rate of Shear Strength Increase, $\rho_e$ (kPa/m)
0.0 – 2.0	25.0	-6.0
2.0 – 10.0	13.0	2.4

### 2.4 Embankment Fill

The Mohr Coulomb failure criterion was used to define failure of the embankment fill. Table 2-2 below summarizes the soil properties adopted for the embankment fill.

**Table 1-2 Shear Strength Properties of Embankment Fill Materials**

Fill Material	Unit Weight, $\gamma$ , (kN/m <sup>3</sup> )	Effective Friction Angle, $\phi'$ , (Degrees)	Effective Cohesion, $c'$ , (kPa)
Rock Fill – Primary Embankment	20.0	35.0	0.0
Granular 'A' Fill	22.0	35.0	0.0
Granular 'B' Fill	21.5	35.0	0.0

## 2.5 Calculation Methods

The factor of safety against collapse of all road embankments was calculated on a section by section basis using the computerized slope stability software Slope/W (GeoSlope International). Slope/W is a slope stability program based on limit equilibrium methods and both Morgenstern Price and Bishop's method of slices were used for the stability calculations. Appendix A contains examples of the calculated factor of safety for select Highway 69 road embankment designs.

Consolidation settlements of the road embankments were calculated based on the  $e$ -log $\sigma$  relationship for clayey soils. Overconsolidated and normally consolidated soil parameters were measured during incremental oedometer consolidation tests. The oedometer consolidation test results were used to estimate a preconsolidation pressure,  $\sigma'_p$ , coefficient of recompression,  $C_r$ , and coefficient of compression,  $C_c$  for the clayey soils at the site.

## 2.6 Design Preconsolidation Pressure Profile

Figure 2 summarizes the available preconsolidation pressures measured during incremental oedometer consolidation tests on clayey silt to silty clay soil samples. In general, it was found that the ratio of undrained shear strength to preconsolidation pressure,  $c_u/\sigma'_p$ , was approximately 0.25 for the clayey soils at the site. The design preconsolidation pressure profile shown in Figure 2 was derived from the vane shear strength profile shown in Figure 1 using a  $c_u/\sigma'_p$  ratio of 0.25. There is generally good agreement between the preconsolidation profile assumed for the design calculations and the measured preconsolidation pressures.

## 2.7 Coefficient of Compression and Consolidation

Figure 3 shows a plot of all measured coefficients of compression,  $C_c$ , and corresponding measured natural moisture contents,  $w_n$ . The best fit relationship between  $C_c$  and  $w_n$  ( $C_c \sim 0.012 w_n$ ) shown in Figure 3 was used in conjunction with the available moisture content measurements to estimate the coefficient of compression for the clayey soils encountered at



the site. Figure 4 summarizes the measured natural moisture contents versus depth below ground surface and shows the distribution of  $C_c$  which was adopted for the design calculations.

Table 2-3 below lists the coefficients of consolidation,  $C_v$ , measured during incremental oedometer tests performed on samples from the Moon and Musquash River Bridge sites. A value of  $C_v = 4 \text{ m}^2/\text{year}$  was assumed for all calculations based on the results presented in Table 2-3 and on the estimated stress range in the field during construction. The majority of clayey silt to silty clay soil layers encountered at the site were found to be underlain by weathered bedrock or sand tills or sand and gravel tills. As a result, two-way drainage was assumed to estimate the time rate of consolidation settlement.

**Table 2-3 Summary of Consolidation Test Data.**

DEPTH (m)	BOREHOLE	$\sigma'_p$ (kPa)	$C_v^{O/C}$ ( $\text{m}^2/\text{yr}$ )	$C_v^{N/C}$ ( $\text{m}^2/\text{yr}$ )	$C_c$	$C_R$	SITE
3.0-3.6	AP-1	85	8.0	3.0	0.6	0.02	Moon River - Southbound
4.0-4.6	BH303	170	9.0	1.25	0.8	0.07	Moon River - Southbound
4.0-4.6	BH303	140	8.0	2.6	0.81	0.04	Moon River - Southbound
3.0-3.6	BH301	100	9.0	5.0	0.32	0.02	Moon River - Southbound
3.0-3.6	BH301	95	10.0	7.5	0.34	0.02	Moon River - Southbound
4.6-5.2	BH302	130	15.0	4.8	0.46	0.05	Moon River - Southbound
3.6	BH 5	80	10.0	2.0	0.04	0.6	Musquash River - Northbound
4.8	BH 8	100	12.0	1.5	0.06	0.6	Musquash River - Northbound

$\sigma'_p$  - Preconsolidation Pressure.

$C_v^{O/C}$  - Coefficient of Consolidation in the over consolidated stress range.

$C_v^{N/C}$  - Coefficient of Consolidation in the normally consolidated stress range.

$C_c$  - Compression Index.

$C_R$  - Recompression Index.

## 2.8 Calculated Embankment Settlements

A summary of the calculated embankment settlements for various fill thicknesses and silty clay to clayey silt thicknesses is summarized in Figure 5. The soil properties described above in Sections 2.3 to 2.7 were used to calculate embankment settlements using Oosterberg's solution to estimate the stress distribution within the foundation soils resulting from the embankment loads.

Appendix E of this report contains a detailed summary of settlement calculations for the section of Highway 69, from 0.4 km South of the Musquash River 8.9 km North to Tower Road.

## 2.9 Design Approach

It is understood that standard MTO practice is to excavate all soft compressible clayey silt to silty clay soils below roadway embankments and to replace the excavated soils with rock fill. The rock fill embankment can then be constructed directly on bedrock or on the compact to dense silty sand to sandy silt tills encountered during drilling in the area. In general, excavation depths of up to approximately 4 metres are considered to be economical and feasible by the MTO for the current project. This approach to embankment construction has the benefit of removing soft compressible foundation soils and thereby eliminating roadway settlements which would otherwise result from foundation soil movements. As such, standard MTO practices were adopted and full excavation and removal of soft compressible clayey silt to silty clay soils has been recommended in this report where required.

Could be  
more than  
4m.

In some very wet areas, however, full excavation of soft clayey soils up to 4 metres deep may require excavations of up to 3 to 4 metres below water due to groundwater collecting in the open excavation. Under the conditions noted above, embankment construction and quality control becomes difficult often resulting in poorly constructed "open-work" rock fills. Also, some soft subgrade soil may remain unexcavated. Poorly constructed rock fills can yield some long term road settlements due to settlement of the rock fill itself. In view of the above discussions and considerations, proper construction controls will be required to ensure that good quality rock fills are constructed within deep excavations.

The approach adopted for the designs presented in this report is summarized below:

1. Settlement calculations were used to identify section of Highway 69 which did not meet the design settlement criteria (less than 25mm of settlement after the application of the pavement base course).
2. If the depth of clayey soils was found to be less than 4 metres, full excavation was recommended in accordance with standard MTO practice.
3. If the depth of clayey soils was found to be greater than 4 metres, the surcharge required to achieve the design settlement criteria was calculated.
4. The configuration of the road embankment during surcharging was then checked for an adequate factor of safety against short term failure.

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5. If the surcharge level required to meet the design settlement criteria resulted in an unstable embankment configuration, the surcharge was reduced to the maximum allowable level and sand/wick drains were adopted to meet the settlement criteria.
6. In some areas there was little or no evidence of a firm crust within the upper zones of the clayey soils. For these areas, the drainage path was reduced using partial excavation due to cost considerations (e.g., Stations 21+720 to 21+810 and 24+325 to 24+600). The depth of excavation for these cases was therefore dependent on the surcharge duration.

## 2.10 Rock Fill Settlements

The current report addresses roadway settlements resulting from the compression of underlying foundation soils, only. For the current project, some settlements of the rock fill road embankments should be expected after the completion of construction. The rate and magnitude of rock fill settlement, however, is not predictable and can be accommodated by overbuilding the rock fill embankments. For the current project, it is understood that the crest width of all road rock fill embankments will be overbuilt by a minimum of 2 metres. This will adequately accommodate all anticipated road embankment settlements.

It is our experience that the primary factor affecting rock fill settlements is construction quality and control. Properly constructed rock fill embankments founded on bedrock or dense to very dense foundation soils (using finer graded material ~ 400mm or less) experience relatively small amounts of post construction movements up to 0.5% of the embankment height. As such, a 6 metre high embankments constructed on bedrock should be expected to settle by as much as 30 mm. If complete excavation and removal of all soft subgrade soils is undertaken, failure to remove all soft subgrade materials and failure to prepare and inspect subgrade conditions prior to placement of rock fill may result in greater roadway settlements.

how? MTO has issued a directive for this project that they would accept settlements that occurred in these fills using MTO Standard Construction Practices. No changes are to be made with respect to maximum rock size in fill areas or the use of a "chinking layer of finer rock as a separation or graded filter between the base rock and overlying granular layer. This approach is to be used in the design.

## 2.11 Sand Drains

The following is a summary of the minimum requirements for vertical sand drain installations:

1. The top of each sand drain should outlet into Granular 'B' fill having a minimum thickness of 600 mm. The Granular 'B' will be required to facilitate drilling operations for the sand drain installation.

2. Installation Procedure: To be submitted by the contractor for review prior to construction.
3. Sand drains must not be installed within 10 metres of a river edge.
4. Minimum Diameter: 300 mm
5. Depth: Sand drains must be continuous and must penetrate full depth of clay layer (to be directed in the field by the engineer).
6. Backfill Material: Well graded sand meeting the following gradation specifications:

*Should be  
reviewed  
an exper  
Consult*

Sieve Size	Percent Passing (%)
9.5 mm	100
4.75 mm	95-100
2.36 mm	80-100
1.18 mm	50-85
600 $\mu$ m	25-60
300 $\mu$ m	10-30
150 $\mu$ m	2-10

## 2.12 Wick Drains

All wick drains are to consist of an outer non-woven geotextile filter and an inner three dimensional relatively stiff geonet drainage material (eg. Coldondrain<sup>®</sup> manufactured by Akzo). The following is a summary of the minimum requirements for wick drains:

1. The top of each wick drain should outlet into Granular 'B' fill having a minimum thickness of 600 mm. The Granular 'B' will be required to facilitate drilling operations for the sand drain installation.
2. Wick drains must not be installed within 10 metres of a river edge.
3. Installation Procedure: As per manufacturer's recommendations.
4. Minimum Tensile Strength at 10% strain (ASTM D4595): 1.0 kN
5. Pore size of non-woven filter:  $\leq 75\mu\text{m}$

6. Permeability of non-woven filter (ASTM D4491):  $1 \times 10^{-4}$  m/sec.
7. Minimum Width: > 95mm
8. Discharge Capacity (ASTM D4716):  $60 \times 10^{-6}$  m<sup>3</sup>/sec
9. Depth: Full depth of clay layer (to be directed in the field by the engineer).

### 3. Subsurface Conditions

#### 3.1 General

The subsurface conditions encountered during drilling for Highway 69 from 0.4 km south of the Musquash River, 8.9 km north to Tower Road are described in detail in the Reference documents listed in Section 2.2 of this report. For the present study, all settlement sensitive soils (clayey soils) encountered during drilling at the site have been identified and are plotted on Drawings 1 through 13, inclusive. Appendix B contains detailed borehole logs drilled by Trow Consulting Engineers Ltd. (Trow) between October 22, 1997, and November 11, 1997, for the northbound lanes of Highway 69, Appendix C contains detailed borehole logs drilled by Trow between November 3, 1997, and November 4, 1997, for the southbound lanes of Highway 69 and Appendix D contains a previous MTO report entitled: Foundation Investigation Report for Approach Embankments (W.P. 215-89-00) which also contains detailed borehole logs for the southbound lanes of Highway 69. More general subsurface information can be found in Reference 6 listed in Section 2.2 of this report.

The following is a description of the subgrade soil conditions for those sections of the highway which are susceptible to long term settlements.

#### 3.2 Southbound Lanes

##### 3.2.1 Station 20+050± to Station 20+360±

Drawing 1 shows the inferred subsurface profile and design centerline elevations for the southbound lanes between Stations 20+050± and 20+360±. The subsurface conditions between Stations 20+050 and 20+360 consist of variable surface soils (eg. peat and organic matter, muck, silty sand etc.) overlying soft to firm clayey silt to silty clay. Vane shear strengths were found to range from 10 kPa to 95 kPa. However, the majority of the vane shear strengths ranged from 15 to 30 kPa indicating that the clayey soils underlying this section of the highway are soft to firm.

##### 3.2.2 Station 20+425± to Station 20+790±

This portion of the southbound lanes is located just north of the Musquash River. Drawing 2 shows the inferred subsurface profile and design centerline elevations for the southbound lanes between Stations 20+425± and 20+790±. Vane shear strengths were found to range from 20 kPa to 85 kPa. The clayey soils encountered during drilling for this section of Highway 69 were encountered below variable surface soils consisting of peat and organic matter, muck, topsoil and silty sand.

### **3.2.3 Station 20+790 to Station 21+230**

Drawing 3 shows the inferred subsurface profile and design centerline elevations for the southbound lanes between Stations 20+790± and 21+230±. The surface soils for this section of highway varied in thickness (see Drawing 3) and consisted of peat, topsoil and muck. Below the surface soils a relatively shallow deposit of clayey silt to silty clay was encountered. Vane shear strengths for the clayey soils found to underlie this section of the southbound lanes were found to range from 21 kPa to 29 kPa indicating that the clayey soils are generally soft to firm.

### **3.2.4 Stations 21+720 to 21+810**

Drawing 4 shows the inferred subsurface information between Stations 21+720 and 21+810. Between Stations 21+720 and 21+810, a localized deposit of soft to stiff silty clay (up to 4 metres deep) was encountered overlain by variable surficial deposits of topsoil, peat and muck.

### **3.2.5 Stations 22+010 to 22+080**

Drawing 4 shows the interpreted foundation soil information between Stations 22+010 and 22+080. In general, clay seams of up to 1 metre thick were encountered during drilling between Stations 22+010 and 22+080. In addition, one vane shear strength was measured (44 kPa) at Station 22+050. The surface soils for this section of the highway consisted of top soil.

### **3.2.6 Stations 22+550 to 22+700**

Firm to stiff silty clay to clayey silt was encountered below variable surface soils (see Drawing 5) during drilling between Stations 22+550 and 22+700. Field vane shear strengths ranging from 44 kPa to 62 kPa were measured at Station 22+610. A soft wet silty clay pocket was encountered at Station 22+630. The surface soils for this section of the highway mainly consisted of topsoil.

### **3.2.7 Stations 22+915 to 22+975**

This section of highway is bounded by a bedrock outcrop to the north and south. The borehole information between Stations 22+915 and 22+975 indicates that there may be some significant clayey seams or pockets (up to 1.0 metres thick) between Stations 22+915 and 22+975.

### **3.2.8 Stations 24+325 to 24+600**

A relatively extensive clayey soil deposit has been encountered below variable surface soils (predominantly topsoil) between Stations 24+325 and 24+600 for the southbound lanes. Drawing 6 summarizes the available subsurface soils information in this area. Based on field vane shear strength testing, the undrained shear strength was found to range from 18 kPa to 64 kPa. In general, however, the vane shear strengths indicate that the majority of the clay deposit is soft. The thickness of the clayey silt to silty clay deposit was observed to vary between 0.5 m thick and 4.3 m thick.

### **3.2.9 Stations 24+790 to 24+840**

A localized deposit of clayey soil was encountered between Stations 24+790 and 24+840. Drawing 6 shows a summary of the subsurface information for this section of the southbound lanes.

### **3.2.10 Stations 25+550 to 25+600**

There is a shallow to moderately deep clayey silt to silty clay deposit located just north of the Moon River. The settlement characteristics of this soil deposit have been addressed in Trow Report BRGE0011546A entitled: "Foundation Design Report – Proposed Bridge Crossing at Moon River". The subsurface conditions consist of some surficial sand and organic soils overlying up to 5 metres of soft to firm clayey silt. It is understood that a four (4) span bridge option has been chosen for the southbound lanes at the Moon River. As a result, only a small section of the road embankment between Stations 25+550 and 25+600 will be underlain by relatively shallow clayey silt to silty clay soils.

### **3.2.11 Stations 26+380 to 26+440**

Soft to stiff clayey silt to silty clay soils were encountered below variable surface soils (peat and organic matter, topsoil and muck) at up to 2.3 metres depth between Stations 26+380 and 26+440. Drawing 8 summarizes the subsurface soil profile for this section of the southbound lanes, Highway 69.

### **3.2.12 Stations 26+500 to 26+560**

Some clayey silt to silty clay soils were encountered below variable surface soils during drilling between Stations 26+500 and 26+560. The extent and depth of this clayey soil deposit is relatively limited. The surface soils were found to be 200mm to 300mm thick consisting of mainly topsoil.



### **3.3 Northbound Lanes**

#### **3.3.1 Stations 20+260 to 20+375**

This section of Highway 69 is located just south of the Musquash River. A surface layer of peat (up to 1 metre thick) was found overlying very soft to soft silty clay to clayey silt during drilling for this section of the northbound lanes, Highway 69. The undrained shear strength of the clayey silt to silty clay was measured using field vane tests and was found to vary between 12 kPa and 25 kPa. Refer to Drawing 9 for a summary of the subsurface soils encountered during drilling for this section of Highway 69.

#### **3.3.2 Stations 20+450 to 20+780**

Drawing 10 summarizes the subsurface conditions encountered during drilling for this section of the northbound lanes, Highway 69. A relatively deep deposit of clayey silt to silty clay was encountered below variable surface soils (peat, organic matter, sand or topsoil) just north of the Musquash River between Stations 20+450 and 20+780. The silty clay to clayey silt deposit was found to be soft to stiff with vane shear strengths ranging from 19 kPa to 72 kPa.

#### **3.3.3 Stations 20+790 to 21+225**

A relatively shallow and extensive deposit of clayey silt to silty clay was encountered during drilling between Stations 20+790 to 21+225. The majority of the clayey silt to silty clay soils were encountered below the existing Highway 69 road embankment and the thickness was found to be variable ranging from 0.6 m to 2.7 m. Drawing 11 shows a summary of the subsurface soils encountered during the drilling program. Clayey soils were not encountered during drilling between Stations 21+075 and 21+160.

#### **3.3.4 Stations 21+550 to 21+580**

A very limited clayey silt to silty clay soil deposit was encountered up to 2.3 metres deep and underlying organic soils (approx. 200 mm thick) during drilling between Stations 21+550 and 21+580. Drawing 12 summarizes the subsurface soil conditions for this section of the northbound lanes, Highway 69.

#### **3.3.5 Stations 22+575 to 22+700**

Drawing 13 summarizes the clayey soil deposit encountered below variable surface soils during drilling between Stations 22+575 to 22+700. This clayey silt to silty clay deposit was encountered at depths ranging from 1.2 m to 2.8 m.

## 4. Preliminary Settlement Design - Six (6) Month Surcharge Period

### 4.1 South Bound Lanes

Detailed settlement calculations were performed for the Highway 69 southbound lanes assuming a construction surcharge period of six (6) months after the initial road embankment construction. The results of detailed settlement calculations are summarized in Appendix E of this report. Drawings 1 through 8 summarize the subsurface soil deposits below the southbound lanes which may exhibit significant time dependent post construction consolidation settlements. All settlement calculations summarized in this report are based on the estimated compression of clayey foundation soils, only. As such, all organic matter, muck and peat must be removed from below the plan limits of the primary road embankment in order to meet the specified design settlement criteria.

The following is a summary of: (i) the section of Highway 69 southbound lanes which were found to be underlain by settlement susceptible soils and (ii) the required embankment designs and surcharge schedules which will limit calculated long term embankment settlements to less than 25mm after application of the pavement base course. Figure 6 illustrates the generalized embankment design geometry adopted for the road embankments discussed below. The road embankment is defined by the height above the original centreline ground surface elevation, the crest width, the side slope gradient and the stabilization berm width. Figure 7 illustrates the recommended sand drain or wick drain layout and Figure 8 shows minimum excavation dimensions for areas where full or partial excavation of clayey silt to silty clay foundation soils has been recommended.

#### 4.1.1 Station 20+050 to Station 20+360

##### Surcharge

To meet the design settlement criteria specified in Section 2 of this report, sand drains or wick drains will be required in conjunction with surcharge loading. Also, all organic matter, peat and topsoil must be removed from below the plan limits of the primary road embankment and berm area (see Figure 6). Table 4-1 shows the surcharge schedule and wick drain spacing/sand drain spacing required to meet the design settlement criteria. For the purpose of detailed design, the sand drains/wick drains should start at Station 20+100. Figure 7 shows the recommended sand drain/wick drain layout and Drawing 1 summarizes the available subsurface information for this section of Highway 69.

**Table 4-1     Surcharge Schedule - Southbound Lanes Between Stations  
20+050 to 20+360**

Station	Design Centreline Road Elevation (m)	Centreline Original Ground Elevation (m)	Embankment Height (m)	Clay Thickness (m)	Calculated Settlement after Surcharge Period (mm)	Surcharge - Rock Fill/Granul ar Fill (m)	Wick Drain Spacing/ Sand Drain Spacing (m)
20+050	203.098	204.183	CUT	N.A.	N.A.	N.A.	N.A.
20+100	201.793	197.058	4.7	3.3	<5	1.0/0.8	N.R.
20+150	200.717	196.724	4.0	5.8	20	1.0/0.8	2.5/3.5
20+200	199.954	196.390	3.6	4.2	20	1.0/0.8	2.5/3.5
20+250	199.503	196.312	3.2	7.0	25	1.0/0.8	2.5/3.5
20+300	199.364	196.259	3.1	7.5	25	1.0/0.8	2.5/3.5
20+350	199.529	196.407	3.1	4.5	10	0.6/0.5	2.5/3.5

**Note:** Embankment height including surcharge must not exceed 6.0 metres for this section of highway.

To ensure an adequate factor of safety against failure during the surcharge period, stabilization berms will be required for this section of Highway 69. For the purpose of detailed design, fifty percent (50%) height berms (based on the final design height) are required for the southbound lanes between Stations 20+050 and 20+360. Table 4-2 below summarizes the required berm dimensions for each of the sections analyzed in Table 4-1. Refer to Figure 6 for the generalized embankment geometry.

**Table 4-2     Required Berm Dimensions**

Station	Design Embankment Height (m)	Berm Height [above OGL] (m)	Berm Elevation (m)	Width (m)
20+050	CUT	N.A.	N.A.	N.A.
20+100	4.7	2.35	199.408	8.5

**Table 4-2 Required Berm Dimensions**

Station	Design Embankment Height (m)	Berm Height [above OGL] (m)	Berm Elevation (m)	Width (m)
20+150	4.0	2.0	198.724	8.5
20+200	3.6	1.8	198.190	8.5
20+250	3.2	1.6	197.912	8.5
20+300	3.1	1.55	197.809	7.5
20+350	3.1	1.55	197.957	6.0

**4.1.2 Station 20+360 to Station 20+425**

This portion of Highway 69 includes the new southbound Musquash River bridge within approximately 20± metres of the north and south abutments. Lightweight fill has been recommended by the MTO for both the north and south bridge abutments (see Foundation Investigation Report, Musquash River, Southbound Lanes Bridge, Site 42-46S, MTO Foundation Section, March 1993). The use of lightweight fill will reduce fill pressures adjacent to the southbound bridge abutments, and as such, one third (1/3) height berms should be adequate for stability of both the north and south Musquash River banks.

For the south approach, the fifty percent height (50%) berms recommended in Table 4-2 for Station 20+360 should be reduced to one third height (1/3) between Station 20+360 and 20+370. For the final design, the one third height berms should then extend to and wrap around the south abutment to the forward slope area.

For the north approach, the forty percent height (40%) berms recommended in Table 4-4 for Station 20+450 should be reduced to one third height (1/3) between Station 20+450 and 20+440.

#### 4.1.3 Station 20+425 to Station 20+790

##### Surcharge

All organic matter, peat and topsoil must be removed from below the plan limits of the primary road embankment as per Figure 6. In addition, to meet the design settlement criteria specified in Section 2 of this report, sand drains or wick drains will be required in conjunction with surcharge loading. Table 4-3 shows the surcharge schedule and wick drain spacing/sand drain spacing required to meet the design settlement criteria for this section of Highway 69. The sand drains/wick drains can be terminated at Station 20+775. (See Drawing 2 for a summary of the available subsurface data).

**Table 4-3 Surcharge Schedule - Southbound Lanes between Stations 20+425 to 20+790**

Station	Design Centreline Road Elevation (m)	Centreline Original Ground Elevation (m)	Embankment Height (m)	Clay Thickness (m)	Calculated Settlement after Surcharge Period (mm)	Surcharge - Rock Fill/Granular Fill (m)	Wick Drain Spacing/ Sand Drain Spacing (m)
20+450	200.036	196.789	3.25	9.2	20	1.0/0.8	2.5/3.5
20+500	200.289	196.485	3.8	6.5	10	1.0/0.8	2.5/3.5
20+550	200.542	196.308	4.2	7.0	<5	1.0/0.8	2.5/3.5
20+600	200.796	196.265	4.5	7.5	25	1.0/0.8	2.5/3.5
20+650	201.049	196.376	4.7	8.1	25	1.0/0.8	2.5/3.5
20+700	201.302	197.225	4.1	4.0	<5	1.0/0.8	2.5/3.5
20+750	201.556	196.073	5.5	3.0	<5	0.6/0.5	2.5/3.5

As previously indicated, Figure 6 shows the generalized embankment geometry for the four lane extension of Highway 69. For the purpose of detailed design, forty percent (40%) height berms (based on the final embankment design height) are required for the southbound lanes between Stations 20+425 and 20+790. Table 4-4 below summarizes the required berm dimensions.

**Table 4-4 Required Berm Dimensions**

Station	Design Embankment Height (m)	Berm Height [above OGL] (m)	Berm Elevation (m)	Width (m)
20+450	3.25	1.3	198.089	6.0
20+500	3.8	1.5	197.985	7.5
20+550	4.2	1.7	198.008	8.5
20+600	4.5	1.8	198.065	8.5
20+650	4.7	1.9	198.276	8.5
20+700	4.1	1.6	198.825	8.5
20+750	5.5	2.2	198.273	9.0

**4.1.4 Station 20+790 to Station 21+230****Excavation**

Drawing 3 summarizes the subsurface soil conditions for this section of Highway 69. Since the clay deposit is shallow between Stations 20+790 and 21+230, all soft to firm clayey subgrade soils should be excavated from below the primary road embankment in accordance with MTO standard practices (see Figure 8 for the required extent of clay excavation). The embankment can be constructed on the underlying till or bedrock which will likely involve limited excavation depths between 0.5m and 3.2m based on the available subsurface information in this area.

**4.1.5 Stations 21+720 to 21+810****Excavation**

Drawing 4 summarizes the condition in this zone. To minimize long term settlements, all soft compressible organic soils should be excavated from below the plan limits of the highway embankment. In addition, the soft to stiff silty clay in this area should also be excavated in accordance with standard MTO practices (see Fig. 8 for extent of clay excavation). The required depth of excavation can be estimated from information provided in Drawing 4.

A moisture content of 26.8% was measured for one clay sample at Station 21+750. Based on this moisture content, the excavated clay will likely be suitable for construction of the side berms in this area of the highway. Some drying of the excavated clayey soils will be required, however, prior to construction of the berms in order to achieve adequate compaction. If the berms are constructed using compacted clay, a Type II non-woven geotextile separator should be provided at the interface between the clay berm and the primary rock fill embankment (assumed to be rock fill). Also, the berms must be revegetated to minimize soil loss (erosion). Figure 8 shows the embankment cross-section which is considered feasible for this section of Highway 69.

#### 4.1.6 Stations 22+010 to 22+080

##### Excavation

Given the relatively limit nature of the clayey soils in this area (see Drawing 4 for summary of subsurface soils), all organic matter, peat, topsoil and soft to firm clayey subgrade soils should be excavated below the primary road embankment in accordance with MTO standard practices. Figure 8 shows the required limits of clay excavation. The rock fill embankment can be constructed directly on the underlying till or bedrock surface after the clay excavation is completed.

#### 4.1.7 Stations 22+550 to 22+700

##### Excavation

Drawing 5 summarizes the subsurface soil conditions for this section of Highway 69. Since the clay deposit is shallow between Stations 22+550 and 22+700, all soft to firm clayey subgrade soils should be excavated from below the primary road embankment in accordance with MTO standard practices. Figure 8 shows the required limits of the required excavation. The embankment for this section of the Highway 69 can be constructed on the underlying till or bedrock which will likely involve limited excavation depths up to 3m based on the available subsurface information in this area.

#### 4.1.8 Stations 22+915 to 22+975

##### Excavation

At Station 22+950 of the southbound lanes, the embankment fill thickness is approximately 12.14 m. The borehole information in this area indicates that there may be some clayey seams or pockets between Stations 22+915 and 22+975. Given the excessive fill thickness and relatively limited clayey soils between Stations 22+915 and 22+975, all soft to firm

clayey soils should be excavated below this section of the southbound Highway 69 lanes. The highway embankment fill can be constructed on the underlying till or bedrock encountered in the area. Removal of the clayey soils between the above referenced Stations may require localized excavation depths of up to 3 metres. Refer to Figure 8 for the required limits of clay excavation.

#### 4.1.9 Stations 24+325 to 24+600

##### Excavation

A relatively extensive clayey soil deposit has been encountered between Stations 24+325 and 24+600 for the southbound lanes. This portion of Highway 69 is located just south of the new Muskoka Road 12 interchange on the east edge of a swamp. Drawing 6 summarizes the available subsurface soils information in this area. Full excavation is proposed to remove organic materials and the underlying soft to firm clay deposits.

Although it should be possible to excavate the full depth of clay in the order of 5 metre or more in this area of the Highway, the deep excavation of this nature, adjacent to a swamp, will likely encounter the following practical problems during construction:

- Up to 5 metres of excavation and rock fill construction below water may be required.
- Sloughing of the side slopes of the excavation should be expected.
- Pumping of the underlying silty sand to sandy silt soils encountered below the clay deposit should also be expected.

In undertaking full excavation for this portion of Highway 69, care should be exercised since it will be difficult to provide good foundation bearing conditions and difficult to construct and provide quality control for rock fill placed below water.

#### 4.1.10 Stations 24+790 to 24+840

##### Excavation

A very shallow and localized deposit of clayey soil was encountered between Stations 24+790 and 24+840. Drawing 6 shows a summary of the subsurface information for this section of the southbound lanes. Given the limited depth (<2.4 m) of clayey soils, excavation and removal is recommended in accordance with standard MTO practice.



#### 4.1.11 Stations 25+550 to 25+600

##### Excavation

There is a shallow to moderately deep clayey silt to silty clay deposit located just north of the Moon River between Stations 25+500 and 25+600. The settlement characteristics of this soil deposit have been addressed in Trow Report BRGE0011546A entitled: "Foundation Investigation Report, Moon River, Southbound Lanes Bridge". The subsurface conditions consist of some surficial sand and organic soils overlying up to 5 metres of soft to firm clayey silt. It is understood that a four (4) span bridge option has been chosen for the southbound lanes at the Moon River and the bridge will span over the clayey soils between Stations 25+500 and 25+550. As a result, only a small section of the road embankment between Stations 25+550 and 25+600 will be underlain by relatively shallow clayey silt to silty clay soils. These soils should be excavated under the plan limits of the approach embankments to the Moon River Bridge in accordance with the recommendations of Trow Report BRGE0011546A entitled: "Foundation Investigation Report, Moon River, Southbound Lanes Bridge". Refer to Drawing 7 for a summary of the subsurface conditions encountered during drilling for this Section of Highway 69.

#### 4.1.12 Stations 26+380 to 26+440

##### Excavation

There is some evidence of clayey silt to silty clay soils between Stations 26+380 and 26+440 based on previous soil borings and probe holes (see Drawing 8). The extent and depth of this deposit is relatively limited, and as such, it is recommended that the soft clayey soils be excavated below the plan limits of the road embankment in accordance with standard MTO practice.

#### 4.1.13 Stations 26+500 to 26+560

##### Excavation

Some clayey silt to silty clay soils were encountered during drilling between Stations 26+500 and 26+560 (see Drawing 8). The extent and depth of this soil deposit is relatively limited, and as such, it is recommended that the soft clayey soils be excavated below the plan limits of the road embankment in accordance with standard MTO practice.

## 4.2 Northbound Lanes

Detailed settlement calculations were also performed for the northbound lanes assuming a surcharge period of six (6) months after the initial road embankment construction. The results of these detailed calculations are summarized in Appendix E and Drawings 9 through 13, inclusive, summarize the location of settlement susceptible subsurface soils. The following is a summary of: (i) the sections of Highway 69 northbound lanes which were found to be underlain by soils which may exhibit significant long term settlements and (ii) the required embankment design options which will limit calculated long term embankment settlements to less than 25mm after application of the pavement base coarse.

### 4.2.1 Stations 20+260 to 20+375

The section of Highway 69 between Stations 20+260 and 20+375 is located just south of the proposed new northbound Musquash River Bridge. The subsurface soil conditions encountered during the field investigation for this project are summarized on Drawing 9 and Table 4-5 below summarizes the calculated settlements and the required surcharge schedule for this section of the northbound lanes. To accelerate the consolidation settlements, wick drains or sand drains will be required at the spacings listed in Table 4-5. In addition, one third height stabilization berms (based on the final design embankment height) must be provided to ensure an adequate factor of safety against failure for the road embankments during construction. All organic matter, peat and topsoil must be removed from below the plan limits of the primary road embankment and berm areas. Table 4-6 summarizes the required berm geometry.

**Table 4-5 Surcharge Schedule - Northbound Lanes Between Stations 20+260 to 20+375**

Station	Design Centreline Road Elevation (m)	Centreline Original Ground Elevation (m)	Embankment Height (m)	Clay Thickness (m)	Calculated Settlement after Surcharge Period (mm)	Surcharge - Rock Fill/Granular Fill (m)	Wick Drain Spacing/ Sand Drain Spacing (m)
20+300	199.623	198.044	~3.3*	9.0	20	1.0/0.8	2.5/3.5
20+350	199.724	196.399	3.3	7.8	20	1.0/0.8	2.5/3.5

\* Note: Embankment Height is based on the estimated ground surface elevation prior to construction of the existing Highway 69 lanes.

**Table 4-6 Required Berm Dimensions**

Station	Design Embankment Height (m)	Berm Height [above OGL] (m)	Berm Elevation (m)	Width (m)
20+300	~3.3	1.1	197.398	6.0
20+350	3.3	1.1	197.499	6.0

**4.2.2 Stations 20+375 to 20+450**

This portion of Highway 69 includes the new northbound Musquash River bridge within approximately 20± metres of the north and south abutments. Lightweight fill has been recommended by Trow Consulting Engineers Ltd. for both the north and south bridge abutments (see Foundation Investigation Report, Musquash River, Northbound Lanes Bridge, Site 42-46S, Trow Consulting Engineers Ltd., October 1998).

For the south approach, the one third (1/3) height berms recommended in Table 4-10 for Station 20+350 extend to and wrap around the south abutment to the forward slope area.

**4.2.3 Stations 20+450 to 20+780**

Just north of the proposed Musquash River Bridge, the section of Highway 69 between Stations 20+450 and 20+780 is underlain by a soft to firm clayey silt to silty clay deposit up to 12 metres deep. This section of highway is bounded by the Musquash River to the south and a bedrock outcrop at approximate Station 20+800. Drawing 10 shows the summarized subsurface profile. Table 4-7 below summarizes the design embankment height, clay thickness, surcharge schedule and Wick Drain Spacing/ sand drain spacing requirements to limit the calculated post construction settlements to less than 25 mm. Also, all organic matter, peat and topsoil must be removed from below the plan limits of the primary road embankment. For the purpose of detailed design, the sand drains/wick drains should start at Station 20+460 and terminate at Station 20+760.

**Table 4-7     Surcharge Schedule - Northbound Lanes Between Stations  
20+450 to 20+780**

Station	Design Centreline Road Elevation (m)	Centreline Original Ground Elevation (m)	Embankment Height (m)	Clay Thickness (m)	Calculated Settlement after Surcharge Period (mm)	Surcharge - Rock Fill/Granular Fill (m)	Wick Drain Spacing/ Sand Drain Spacing (m)
20+450*	200.224	197.317	2.9	6.9	25	1.0/0.8	N.R.
20+500	200.474	197.019	3.5	4.1	15	1.0/0.8	2.5/3.5
20+550	200.724	198.018	2.7	0.0	<5	1.0/0.8	2.5/3.5
20+600	200.974	196.835	4.1	4.8	20	1.0/0.8	2.5/3.5
20+650	201.224	197.855	3.4	13.0	20	1.0/0.8	2.5/3.5
20+700	201.474	198.423	3.1	6.8	25	1.0/0.8	2.5/3.5
20+750	201.724	198.070	3.7	2.3	<5	0.6/0.5	2.5/3.5

\* Note: Calculations base on the use of lightweight fill adjacent to bridge Musquash River abutments.

To ensure an adequate factor of safety against failure of the road embankments during the surcharge period, one third height stabilization berms (based on the final design embankment height) are required. Table 4-8 summarizes the berm requirements for this section of Highway 69, northbound lanes.

**Table 4-8     Required Berm Dimensions**

Station	Design Embankment Height (m)	Berm Height [above OGL] (m)	Berm Elevation (m)	Width (m)
20+450	2.9	1.0	198.317	6.0
20+500	3.5	1.2	198.219	6.0
20+550	2.7	0.9	198.918	6.0

**Table 4-8 Required Berm Dimensions**

Station	Design Embankment Height (m)	Berm Height [above OGL] (m)	Berm Elevation (m)	Width (m)
20+600	4.1	1.4	198.235	6.0
20+650	3.4	1.1	198.955	6.0
20+700	3.1	1.0	199.423	6.0
20+750	3.7	1.2	199.270	6.0

**4.2.4 Stations 20+790 to 21+225****Excavation**

The clayey soils encountered between Stations 20+790 to 21+225 are generally limited in depth and appear to be discontinuous. Also, between Stations 20+900 and 21+225, the clayey soils were encountered below the existing Highway 69 road embankment. All soft to firm clayey subgrade soils should be excavated from below the primary road embankment in accordance with MTO standard practices for this section of Highway 69. Figure 8 shows the required limits of the required excavation.

**4.2.5 Stations 21+550 to 21+580****Excavation**

Drawing 12 shows a summary of the inferred subsurface soil conditions for this section of the northbound lanes. It is evident from Drawing 12 that the depth and extent of the clay deposit is limited. As such, excavation and removal of all soft clayey soil is recommended below the plan limits of the road embankment in accordance with standard MTO practice.

**4.2.6 Stations 22+575 to 22+700****Excavation**

Drawing 13 shows a summary of the inferred subsurface soil conditions for this section of the northbound lanes. It is evident from Drawing 12 that the depth and extent of the clay deposit

is limited. As such, excavation and removal of all soft clayey soil is recommended below the plan limits of the road embankment in accordance with standard MTO practice.

## 5. Preliminary Settlement Design – Four (4) Month Surcharge Period

### 5.1 Southbound Lanes

The embankment designs summarized in this section of the report are suitable for a four (4) month surcharge period. Refer to Appendix E for detailed results of the settlement calculations. As noted in Section 4 of this report, all settlement calculations are based on the estimated compression of underlying clayey soils, only. As such, all organic matter, muck and peat must be removed from below the plan limits of the primary road embankment in order to meet the design settlement criteria. Berm geometry has been designed based on the most critical surcharge loading and Figure 6 shows the generalized road embankment, berm geometry and excavation limits for the removal of organic material, muck and peat. The following is a summary of the preliminary road embankment designs.

#### 5.1.1 Station 20+050 to Station 20+360

##### Surcharge

Table 5-1 shows the revised surcharge schedule and Wick Drain Spacing/ sand drain spacing required to meet the design settlement criteria for the present project. Note that the Wick Drain Spacing/ sand drain spacing between Stations 20+050 and 20+360 has been reduced for the four month surcharge schedule relative to the six month surcharge schedule. For the purpose of detailed design, the wick drains/sand drain treatment should start at Station 20+100. Figure 7 shows the required Wick Drain Spacing/ sand drain layout.

**Table 5-1 Surcharge Schedule - Southbound Lanes between Stations 20+050 to 20+360**

Station	Design Centreline Road Elevation (m)	Centreline Original Ground Elevation (m)	Embankment Height (m)	Clay Thickness (m)	Calculated Settlement after Surcharge Period (mm)	Surcharge - Rock Fill/Granular Fill (m)	Wick Drain Spacing/ Sand Drain Spacing (m)
20+050	203.098	204.183	CUT	N.A.	N.A.	N.A.	N.A.
20+100	201.793	197.058	4.7	2.2	<5	1.0/0.8	2.0/3.0
20+200	199.954	196.390	3.6	4.2	20	1.0/0.8	2.0/3.0

**Table 5-1      Surcharge Schedule - Southbound Lanes between Stations  
20+050 to 20+360**

Station	Design Centreline Road Elevation (m)	Centreline Original Ground Elevation (m)	Embankment Height (m)	Clay Thickness (m)	Calculated Settlement after Surcharge Period (mm)	Surcharge - Rock Fill/Granular Fill (m)	Wick Drain Spacing/ Sand Drain Spacing (m)
20+150	200.717	196.724	4.0	5.8	20	1.0/0.8	2.0/3.0
20+250	199.503	196.312	3.2	7.0	25	1.0/0.8	2.0/3.0
20+300	199.364	196.259	3.1	7.5	25	1.0/0.8	2.0/3.0
20+350	199.529	196.407	3.1	4.5	10	0.6/0.5	2.0/3.0

**Note:** Embankment height including surcharge must not exceed 6.0 metres for this section of highway.

**Table 5-2      Required Berm Dimensions**

Station	Design Embankment Height (m)	Berm Height [above OGL] (m)	Berm Elevation (m)	Width (m)
20+050	CUT	N.A.	N.A.	N.A.
20+100	4.7	2.35	199.408	8.5
20+150	4.0	2.0	198.724	8.5
20+200	3.6	1.8	198.190	8.5
20+250	3.2	1.6	197.912	8.5
20+300	3.1	1.55	197.809	7.5
20+350*	3.1	1.55	197.957	6.0



### 5.1.2 Station 20+360 to Station 20+425

As noted in Section 4.1.2 of this report, this section of Highway 69 includes the new southbound Musquash River bridge within approximately 20± metres of the north and south abutments. In accordance with MTO recommendations (see Foundation Investigation Report, Musquash River, Southbound Lanes Bridge, Site 42-46S, MTO Foundation Section, March, 1993), lightweight fill should be used adjacent to the bridge abutments and the design surcharge recommended in the above referenced report must be maintained for a minimum period of 6 months. As such, a four (4) month surcharge period should not be used for this section of Highway.

### 5.1.3 Station 20+425 to Station 20+790

#### Surcharge

Table 5-3 shows the revised surcharge schedule and Wick Drain Spacing/ sand drain spacing required to meet the design settlement criteria for this section of Highway 69. For the purpose of detailed design, the sand drains/wick drains should be terminated at Station 20+780 based on the available subsurface information.

**Table 5-3 Surcharge Schedule - Southbound Lanes Between Stations 20+425 to 20+790**

Station	Design Centreline Road Elevation (m)	Centreline Original Ground Elevation (m)	Embankment Height (m)	Clay Thickness (m)	Calculated Settlement after Surcharge Period (mm)	Surcharge - Rock Fill/Granular Fill (m)	Wick Drain Spacing/ Sand Drain Spacing (m)
20+450	200.036	196.789	3.25	9.2	20	1.0/0.8	2.0/3.0
20+500	200.289	196.485	3.8	6.5	10	1.0/0.8	2.0/3.0
20+550	200.542	196.308	4.2	7.0	<5	1.0/0.8	2.0/3.0
20+600	200.796	196.265	4.5	7.5	25	1.0/0.8	2.0/3.0
20+650	201.049	196.376	4.7	8.1	25	1.0/0.8	2.0/3.0
20+700	201.302	197.225	4.1	4.0	<5	1.0/0.8	2.0/3.0
20+750	201.556	196.073	5.5	4.0	<5	0.6/0.5	2.0/3.0

Table 5-4 below summarizes the required berm dimensions for each of the sections shown in Table 5-3.

**Table 5-4 Required Berm Dimensions**

Station	Design Embankment Height (m)	Berm Height [above OGL] (m)	Berm Elevation (m)	Width (m)
20+450	3.25	1.3	198.089	6.0
20+500	3.8	1.5	197.985	7.5
20+550	4.2	1.7	198.008	8.5
20+600	4.5	1.8	198.065	8.5
20+650	4.7	1.9	198.276	8.5
20+700	4.1	1.6	198.825	8.5
20+750	5.5	2.2	198.273	9.0

**Note:** Embankment Height and Surcharge Limited at Bridge Location

#### 5.1.4 Station 20+790 to Station 21+230

##### Excavation

In accordance with the recommendations in Section 4.1.4 of this report and with MTO standard practice, all clayey soils between 21+790 to 21+230 must be excavated from below the primary road embankment.

#### 5.1.5 Stations 21+720 to 21+810

##### Excavation

In accordance with the recommendations in Section 4.1.5 of this report and with MTO standard practice. All clayey soils between 21+720 to 21+810 must be excavated from below the primary road embankment.

**5.1.6 Stations 22+010 to 22+080****Excavation**

In accordance with the recommendations in Section 4.1.6 of this report and with MTO standard practice, all clayey soils between 22+010 to 22+080 should be excavated from below the primary road embankment.

**5.1.7 Stations 22+550 to 22+700****Excavation**

In accordance with the recommendations in Section 4.1.7 of this report and with MTO standard practice, all clayey soils between 22+550 to 22+700 should be excavated from below the primary road embankment.

**5.1.8 Stations 22+915 to 22+975****Excavation**

In accordance with the recommendations in Section 4.1.8 of this report and with MTO standard practice, all clayey soils between 22+915 to 22+975 should be excavated from below the primary road embankment.

**5.1.9 Stations 24+325 to 24+600****Excavation**

In accordance with the recommendations in Section 4.1.9 of this report and with MTO practice, all clayey soils between 24+325 to 24+900 should be excavated from below the primary road embankment.

**5.1.10 Stations 24+790 to 24+840**

In accordance with Section 4.1.10 of this report, excavation and removal of all clayey soils encountered between Stations 24+790 to 24+840 is recommended.

**5.1.11 Stations 25+550 to 25+600**

In accordance with Section 4.1.11 of this report, excavation and removal of all clayey soils encountered between Stations 25+550 to 25+600 is recommended.

**5.1.12 Stations 26+380 to 26+440**

In accordance with Section 4.1.12 of this report, excavation and removal of all clayey soils encountered between Stations 26+380 to 26+440 is recommended.

**5.1.13 Stations 26+500 to 26+560**

In accordance with Section 4.1.13 of this report, excavation and removal of all clayey soils encountered between Stations 26+500 to 26+560 is recommended.

## 5.2 Northbound Lanes

The following are embankment design alternatives required to meet the design settlement criteria outlined in Section 2 of this report for a four (4) month surcharge period. Refer to Appendix E for detailed results of the settlement calculations. Berm geometries have been design based on the most critical surcharge loading and Figure 6 shows the general embankment and berm geometry.

### 5.2.1 Stations 20+260 to 20+375

Table 5-9 below summarizes the calculated settlements and surcharge schedule for this section of the northbound lanes. To accelerate the consolidation settlements, wick drains or sand drains will be required at the spacings listed in Table 5-5. In addition, one third height stabilization berms (based on the final design embankment height) are required to ensure an adequate factor of safety against failure for the road embankments during construction (see Table 5-6).

**Table 5-5 Surcharge Schedule - Northbound Lanes Between Stations 20+260 to 20+375**

Station	Design Centreline Road Elevation (m)	Centreline Original Ground Elevation (m)	Embankment Height (m)	Clay Thickness (m)	Calculated Settlement after Surcharge Period (mm)	Surcharge - Rock Fill/Granular Fill (m)	Wick Drain Spacing/ Sand Drain Spacing (m)
20+300	199.623	198.044	~3.3	9.0	20	1.0/0.8	2.0/3.0
20+350	199.724	196.399	3.3	7.0	20	1.0/0.8	2.0/3.0

**Table 5-6 Required Berm Dimensions**

Station	Design Embankment Height (m)	Berm Height [above OGL] (m)	Berm Elevation (m)	Width (m)
20+300	~3.3	1.1	197.398	6.0
20+350	3.3	1.1	197.499	6.0

### 5.2.2 Stations 20+375 to 20+450

As noted in Section 4.2.2 of this report, this section of Highway 69 includes the new northbound Musquash River bridge within approximately 20± metres of the north and south abutments. In accordance with Trow Consulting Engineers Ltd. recommendations (see Foundation Investigation Report, Musquash River, Northbound Lanes Bridge, Site 42-46S, MTO Foundation Section, October 1998), lightweight fill should be used adjacent to the bridge abutments and the design surcharge must be maintained for a minimum period of 6 months. As such, a four (4) month surcharge period is not permitted for this section of Highway.

### 5.2.3 Stations 20+450 to 20+780

Table 5-7 below summarizes the design embankment height, clay thickness, revised surcharge schedule and Wick Drain Spacing/ sand drain spacing requirements to limit the calculated post construction settlements to less than 25 mm between Stations 20+450 and 20+780.

**Table 5-7 Surcharge Schedule - Northbound Lanes Between Stations 20+450 to 20+780**

Station	Design Centreline Road Elevation (m)	Centreline Original Ground Elevation (m)	Embankment Height (m)	Clay Thickness (m)	Calculated Settlement after Surcharge Period (mm)	Surcharge - Rock Fill/Granular Fill (m)	Wick Drain Spacing/ Sand Drain Spacing (m)
20+450	200.224	197.317	2.9	6.9	25	1.0/0.8	N.R.
20+500	200.474	197.019	3.5	4.1	15	1.0/0.8	2.0/3.0
20+550	200.724	198.018	2.7	0.0	<5	1.0/0.8	2.0/3.0
20+600	200.974	196.835	4.1	4.8	20	1.0/0.8	2.0/3.0
20+650	201.224	197.855	3.4	13.0	20	1.0/0.8	2.0/3.0
20+700	201.474	198.423	3.1	6.8	25	1.0/0.8	2.0/3.0
20+750	201.724	198.070	3.7	2.3	<5	1.0/0.8	2.0/3.0

To ensure an adequate factor of safety against failure of the road embankments during the surcharge period, one third height stabilization berms (based on the final design embankment height) are required. Table 5-8 summarizes the berm requirements for this section of highway.

**Table 5-8 Required Berm Dimensions**

Station	Design Embankment Height (m)	Berm Height [above OGL] (m)	Berm Elevation (m)	Width (m)
20+450	2.9	1.0	198.317	6.0
20+500	3.5	1.2	198.219	6.0
20+550	2.7	0.9	198.918	6.0
20+600	4.1	1.4	198.235	6.0
20+650	3.4	1.1	198.955	6.0
20+700	3.1	1.0	199.423	6.0
20+750	3.7	1.2	199.270	6.0

#### 5.2.4 Stations 20+790 to 21+225

In accordance with Section 4.2.4 of this report, excavation and removal of all clayey soils encountered between Stations 20+790 to 21+225 is recommended.

#### 5.2.5 Stations 21+550 to 21+580

In accordance with Section 4.2.5 of this report, all clayey soils should be excavated from below the primary road embankment between Stations 21+550 to 21+580.

#### 5.2.6 Stations 22+575 to 22+700

In accordance with Section 4.2.6 of this report, all clayey soils should be excavated from below the primary road embankment between Stations 22+575 to 22+700.

## 6. Preliminary Settlement Design – Two (2) Month Surcharge Period

### 6.1 Southbound Lanes

The embankment designs summarized in this section of the report are suitable for a two (2) month surcharge period. Refer to Appendix B for detailed results of the settlement calculations. As noted in Section 4 of this report, all settlement calculations are based on the estimated compression of underlying clayey soils, only. As such, all organic matter, muck and peat must be removed from below the plan limits of the primary road embankment in order to meet the design settlement criteria. Berm geometry has been design based on the most critical surcharge loading and Figure 6 shows the generalized road embankment, berm geometry, and required excavation limits for the removal of organic material, much and peat. The following is a summary of the preliminary road embankment designs.

#### 6.1.1 Station 20+050 to Station 20+360

##### Surcharge

Table 6-1 shows the revised surcharge schedule and Wick Drain Spacing/ sand drain spacing required to meet the design settlement criteria for the present project. Note that the Wick Drain Spacing/ sand drain spacing between Stations 20+050 and 20+360 has been reduced for the 2 month surcharge schedule relative to the 4 and 6 month surcharge schedules. For the purpose of detailed design, the Wick Drain Spacing/ sand drain treatment should be started at Station 20+100.

**Table 6-1 Surcharge Schedule - Southbound Lanes between Stations 20+050 to 20+360**

Station	Design Centreline Road Elevation (m)	Centreline Original Ground Elevation (m)	Embankment Height (m)	Clay Thickness (m)	Calculated Settlement after Surcharge Period (mm)	Surcharge - Rock Fill/Granular Fill (m)	Wick Drain Spacing/ Sand Drain Spacing (m)
20+050	203.098	204.183	CUT	N.A.	N.A.	N.A.	N.A.
20+100	201.793	197.058	4.7	2.2	<5	1.0/0.8	1.4/2.1
20+150	200.717	196.724	4.0	5.8	20	1.0/0.8	1.4/2.1



**Table 6-1      Surcharge Schedule - Southbound Lanes between Stations  
20+050 to 20+360**

Station	Design Centreline Road Elevation (m)	Centreline Original Ground Elevation (m)	Embankment Height (m)	Clay Thickness (m)	Calculated Settlement after Surcharge Period (mm)	Surcharge - Rock Fill/Granular Fill (m)	Wick Drain Spacing/ Sand Drain Spacing (m)
20+200	199.954	196.390	3.6	4.2	20	1.0/0.8	1.4/2.1
20+250	199.503	196.312	3.2	7.0	25	1.0/0.8	1.4/2.1
20+300	199.364	196.259	3.1	7.5	25	1.0/0.8	1.4/2.1
20+350	199.529	196.407	3.1	4.5	10	0.6/0.5	1.4/2.1

**Note:** Embankment height including surcharge must not exceed 6.0 metres for this section of highway.

**Table 6-2      Required Berm Dimensions**

Station	Design Embankment Height (m)	Berm Height [above OGL] (m)	Berm Elevation (m)	Width (m)
20+050	CUT	N.A.	N.A.	N.A.
20+100	4.7	2.35	199.408	8.5
20+150	4.0	2.0	198.724	8.5
20+200	3.6	1.8	198.190	8.5
20+250	3.2	1.6	197.912	8.5
20+300	3.1	1.55	197.809	7.5
20+350*	3.1	1.55	197.957	6.0

### 6.1.2 Station 20+360 to Station 20+425

As noted in Section 4.1.2 of this report, this section of Highway 69 includes the new southbound Musquash River bridge within approximately 20± metres of the north and south abutments. In accordance with MTO recommendations (see Foundation Investigation Report, Musquash River, Southbound Lanes Bridge, Site 42-46S, MTO Foundation Section, March, 1993), lightweight fill should be used adjacent to the bridge abutments and the design surcharge recommended in the above referenced report must be maintained for a minimum period of 6 months. As such, a four (4) month surcharge period should not be used for this section of Highway.

### 6.1.3 Station 20+425 to Station 20+790

#### Surcharge

Table 6-3 shows the revised surcharge schedule and Wick Drain Spacing/ sand drain spacing required to meet the design settlement criteria for this section of Highway 69. For the purpose of detailed design, the sand drains/wick drains should be terminated at Station 20+780 based on the available subsurface information.

**Table 6-3 Surcharge Schedule - Southbound Lanes Between Stations 20+425 to 20+790**

Station	Design Centreline Road Elevation (m)	Centreline Original Ground Elevation (m)	Embankment Height (m)	Clay Thickness (m)	Calculated Settlement after Surcharge Period (mm)	Surcharge - Rock Fill/Granular Fill (m)	Wick Drain Spacing/ Sand Drain Spacing (m)
20+450	200.036	196.789	3.25	9.2	20	1.0/0.8	1.4/2.1
20+500	200.289	196.485	3.8	6.5	10	1.0/0.8	1.4/2.1
20+550	200.542	196.308	4.2	7.0	<5	1.0/0.8	1.4/2.1
20+600	200.796	196.265	4.5	7.5	25	1.0/0.8	1.4/2.1
20+650	201.049	196.376	4.7	8.1	25	1.0/0.8	1.4/2.1
20+700	201.302	197.225	4.1	4.0	<5	1.0/0.8	1.4/2.1
20+750	201.556	196.073	5.5	4.0	<5	0.6/0.5	1.4/2.1

Table 6-4 below summarizes the required berm dimensions for each of the sections shown in Table 6-3.

**Table 6-4 Required Berm Dimensions**

Station	Design Embankment Height (m)	Berm Height [above OGL] (m)	Berm Elevation (m)	Width (m)
20+450	3.25	1.3	198.089	6.0
20+500	3.8	1.5	197.985	7.5
20+550	4.2	1.7	198.008	8.5
20+600	4.5	1.8	198.065	8.5
20+650	4.7	1.9	198.276	8.5
20+700	4.1	1.6	198.825	8.5
20+750	5.5	2.2	198.273	9.0

#### 6.1.4 Station 20+790 to Station 21+230

##### Excavation

In accordance with the recommendations in Section 4.1.4 of this report and with MTO standard practice, all clayey soils between 21+790 to 21+230 must be excavated from below the primary road embankment.

#### 6.1.5 Stations 21+720 to 21+810

##### Excavation

In accordance with the recommendations in Section 4.1.5 of this report and with MTO standard practice, all clayey soils between 21+720 to 31+810 must be excavated from below the primary road embankment.

**6.1.6 Stations 22+010 to 22+080****Excavation**

In accordance with the recommendations in Section 4.1.6 of this report and with MTO standard practice, all clayey soils between 22+010 to 22+080 should be excavated from below the primary road embankment.

**6.1.7 Stations 22+550 to 22+700****Excavation**

In accordance with the recommendations in Section 4.1.7 of this report and with MTO standard practice, all clayey soils between 22+550 to 22+700 should be excavated from below the primary road embankment.

**6.1.8 Stations 22+915 to 22+975****Excavation**

In accordance with the recommendations in Section 4.1.8 of this report and with MTO standard practice, all clayey soils between 22+915 to 22+975 should be excavated from below the primary road embankment.

**6.1.9 Stations 24+325 to 24+600****Excavation**

In accordance with the recommendations in Section 4.1.9 of this report and with MTO standard practice, all clayey soils between 24+325 to 24+600 should be excavated below the primary road embankment..

**6.1.10 Stations 24+790 to 24+840**

In accordance with Section 4.1.10 of this report, excavation and removal of all clayey soils encountered between Stations 24+790 to 24+840 is recommended.

**6.1.11 Stations 25+550 to 25+600**

In accordance with Section 4.1.11 of this report, excavation and removal of all clayey soils encountered between Stations 25+550 to 25+600 is recommended.

**6.1.12 Stations 26+380 to 26+440**

In accordance with Section 4.1.12 of this report, excavation and removal of all clayey soils encountered between Stations 26+380 to 26+440 is recommended.

**6.1.13 Stations 26+500 to 26+560**

In accordance with Section 4.1.13 of this report, excavation and removal of all clayey soils encountered between Stations 26+500 to 26+560 is recommended.

## 6.2 Northbound Lanes

The following are embankment design alternatives required to meet the design settlement criteria outlined in Section 2 of this report for a two (2) month surcharge period. Refer to Appendix E for detailed results of the settlement calculations. Berm geometries have been design based on the most critical surcharge loading and Figure 6 shows the generalized embankment and berm geometry.

### 6.2.1 Stations 20+260 to 20+375

Table 6-5 below summarizes the calculated settlements and surcharge schedule for this section of the northbound lanes. To accelerate the consolidation settlements, wick drains or sand drains will be required at the spacings listed in Table 6-9. In addition, one third height stabilization berms (based on the final design embankment height) are required to ensure an adequate factor of safety against failure for the road embankments during construction (see Table 6-6).

**Table 6-5 Surcharge Schedule - Northbound Lanes Between Stations 20+260 to 20+375**

Station	Design Centreline Road Elevation (m)	Centreline Original Ground Elevation (m)	Embankment Height (m)	Clay Thickness (m)	Calculated Settlement after Surcharge Period (mm)	Surcharge - Rock Fill/Granular Fill (m)	Wick Drain Spacing/ Sand Drain Spacing (m)
20+300	199.623	198.044	~3.3*	9.0	20	1.0/0.8	1.4/2.1
20+350	199.724	196.399	3.3	7.0	20	1.0/0.8	1.4/2.1

**Table 6-6 Required Berm Dimensions**

Station	Design Embankment Height (m)	Berm Height [above OGL] (m)	Berm Elevation (m)	Width (m)
20+300	~3.3	1.1	197.398	6.0
20+350	3.3	1.1	197.499	6.0

### 6.2.2 Stations 20+375 to 20+450

As noted in Section 4.2.2 of this report, this section of Highway 69 includes the new northbound Musquash River bridge within approximately 20± metres of the north and south abutments. In accordance with Trow Consulting Engineers Ltd. recommendations (see Foundation Investigation Report, Musquash River, Northbound Lanes Bridge, Site 42-46S, MTO Foundation Section, October 1998), lightweight fill should be used adjacent to the bridge abutments and the design surcharge must be maintained for a minimum period of 6 months. As such, a two (2) month surcharge period is not permitted for this section of Highway.

### 6.2.3 Stations 20+450 to 20+780

Table 6-7 below summarizes the design embankment height, clay thickness, revised surcharge schedule and Wick Drain Spacing/ sand drain spacing requirements to limit the calculated post construction settlements to less than 25 mm between Stations 20+450 and 20+780.

**Table 6-7 Surcharge Schedule - Northbound Lanes Between Stations 20+450 to 20+780**

Station	Design Centreline Road Elevation (m)	Centreline Original Ground Elevation (m)	Embankment Height (m)	Clay Thickness (m)	Calculated Settlement after Surcharge Period (mm)	Surcharge - Rock Fill/Granular Fill (m)	Wick Drain Spacing/ Sand Drain Spacing (m)
20+450	200.224	197.317	2.9	6.9	25	1.0/0.8	N.R.
20+500	200.474	197.019	3.5	4.1	15	1.0/0.8	1.4/2.1
20+550	200.724	198.018	2.7	0.0	<5	1.0/0.8	1.4/2.1
20+600	200.974	196.835	4.1	4.8	20	1.0/0.8	1.4/2.1
20+650	201.224	197.855	3.4	13.0	20	1.0/0.8	1.4/2.1
20+700	201.474	198.423	3.1	6.8	25	1.0/0.8	1.4/2.1
20+750	201.724	198.070	3.7	2.3	<5	1.0/0.8	1.4/2.1

To ensure an adequate factor of safety against failure of the road embankments during the surcharge period, one third height stabilization berms (based on the final design embankment height) are required. Table 6-8 summarizes the berm requirements for this section of highway.

**Table 6-8 Required Berm Dimensions**

Station	Design Embankment Height (m)	Berm Height [above OGL] (m)	Berm Elevation (m)	Width (m)
20+450	2.9	1.0	198.317	6.0
20+500	3.5	1.2	198.219	6.0
20+550	2.7	0.9	198.918	6.0
20+600	4.1	1.4	198.235	6.0
20+650	3.4	1.1	198.955	6.0
20+700	3.1	1.0	199.423	6.0
20+750	3.7	1.2	199.270	6.0

#### **6.2.4 Stations 20+790 to 21+225**

In accordance with Section 4.2.4 of this report, excavation and removal of all clayey soils encountered between Stations 20+790 to 21+225 is recommended.

#### **6.2.5 Stations 21+550 to 21+580**

In accordance with Section 4.2.5 of this report, all clayey soils should be excavated from below the primary road embankment between Stations 21+550 to 21+580.

#### **6.2.6 Stations 22+575 to 22+700**

In accordance with Section 4.2.6 of this report, all clayey soils should be excavated from below the primary road embankment between Stations 22+575 to 22+700.



## 7. General

The preliminary embankment designs presented in this report have been prepared using the subsurface information obtained by Trow Consulting Engineers Ltd. and by others. It should be understood that the subsurface soil conditions summarized in this report have been inferred from auger probe holes, sampled boreholes and cone holes. Also, the space between boreholes is substantial, and as such, actual subsurface soil conditions may vary substantially from those presented in this report. For example, the extent of clayey soils in some areas was inferred from boreholes spaced at up to 50 metres. It may be that during construction, the contractor may not encounter soft soils in some areas shown on the attached Drawings 1 through 13, inclusive (particularly near the estimated extents). In some areas, the contractor may find that the extent of clayey soils is greater than assumed for the present design.

For the purpose of detailed design, it will be necessary to plot the berm elevations specified in the tables contained in Sections 4, 5 and 6 of this report on the design highway centreline profile. The centreline profile can then be offset to create a berm profile. The berm profile should be positioned vertically so that all recommended berm elevation points are less than or equal to the offset centreline profile/new berm profile. Using this approach, there may be some sections with berm heights slightly greater than the recommended berm heights.

Trow Consulting Engineers Ltd. should be provided the opportunity to review the final berm design profile to ensure that the berms will be stable during construction.

We trust that this report is satisfactory. If you have any comments or questions, please contact this office.

**Trow Consulting Engineers Ltd.**

*For Cheng*

*for*

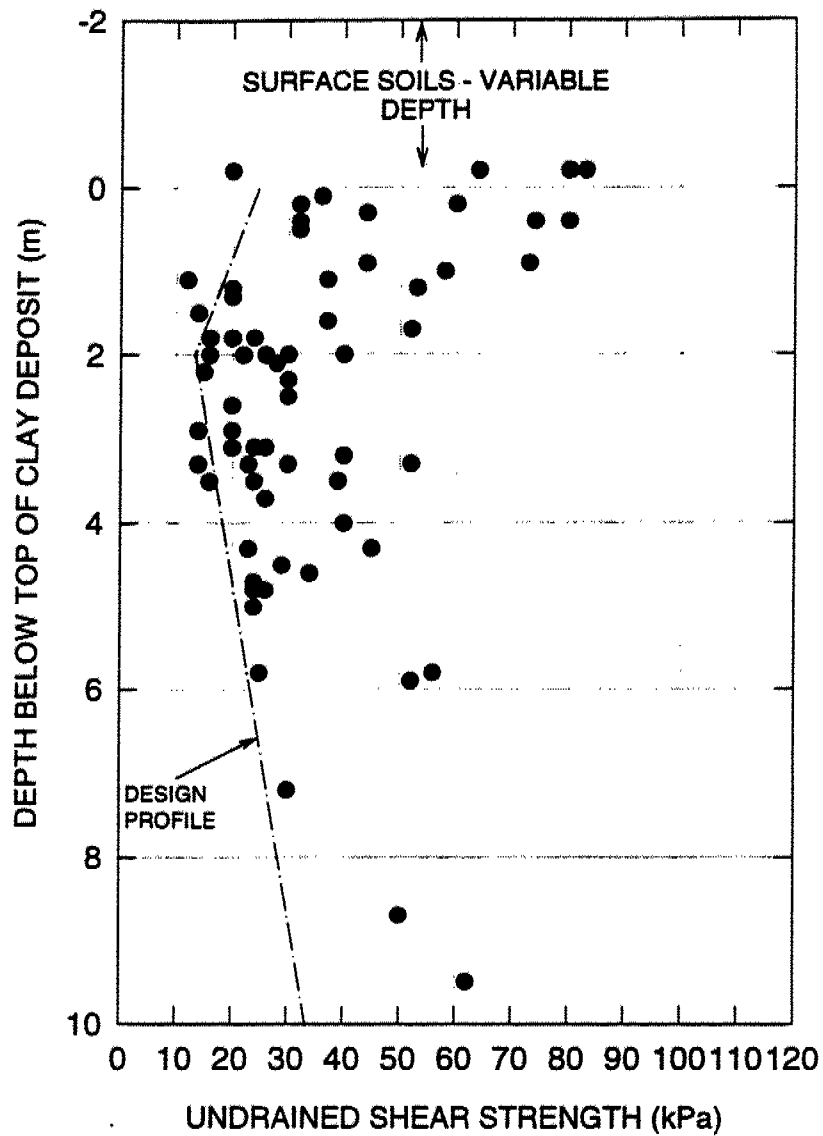
Sean Hinchberger, Ph.D., P.Eng.  
Project Engineer, Geotechnical Division

*SG*  
Stan Gonsalves, P.Eng.  
Principal Engineer

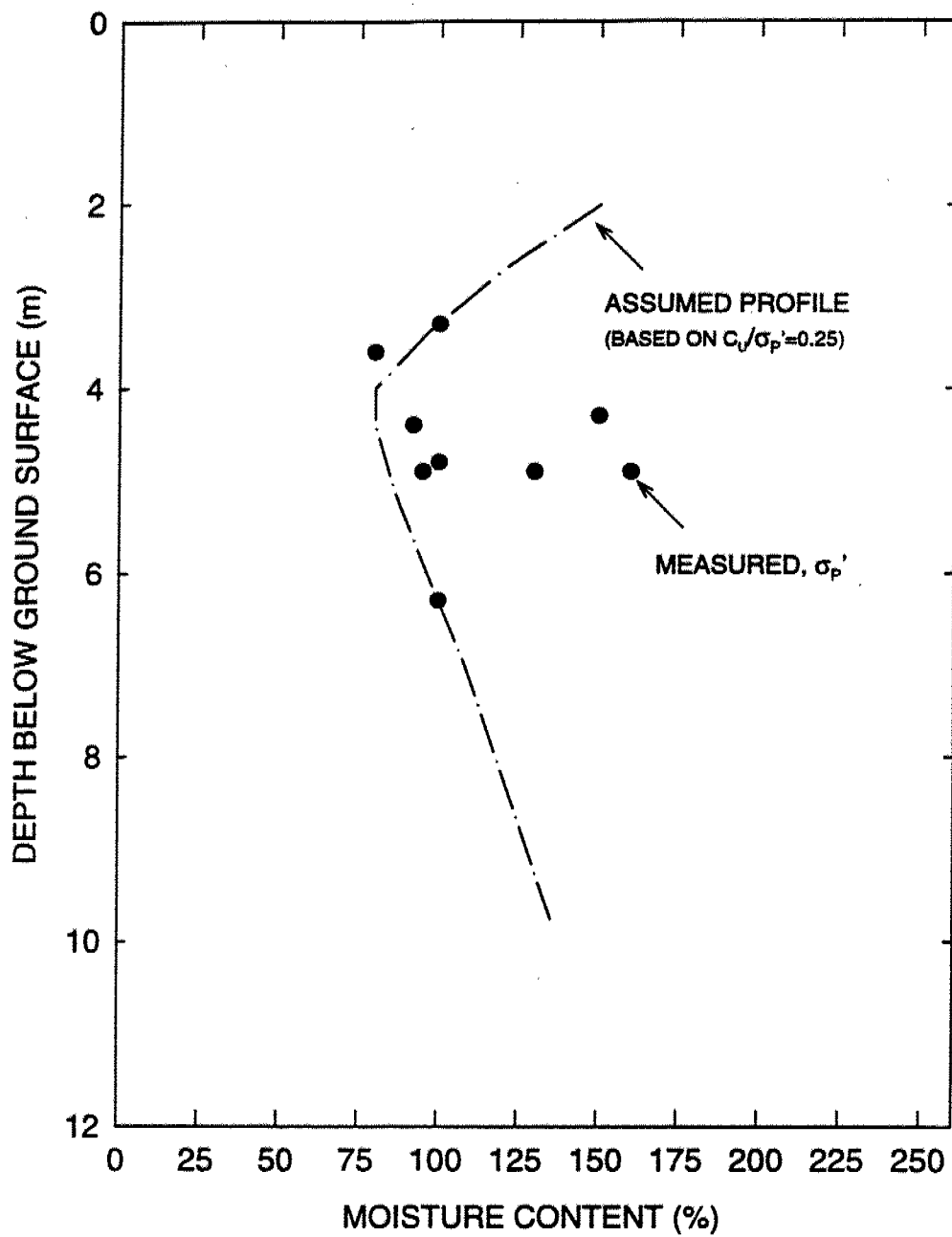


# Figures

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**FIGURE 1 SUMMARY OF UNDRAINED SHEAR STRENGTH MEASUREMENTS - HIGHWAY 69**



**FIGURE 2 COMPARISON OF ASSUMED PRECONSOLIDATION PROFILE AND MEASURE PRECONSOLIDATION PRESSURES**

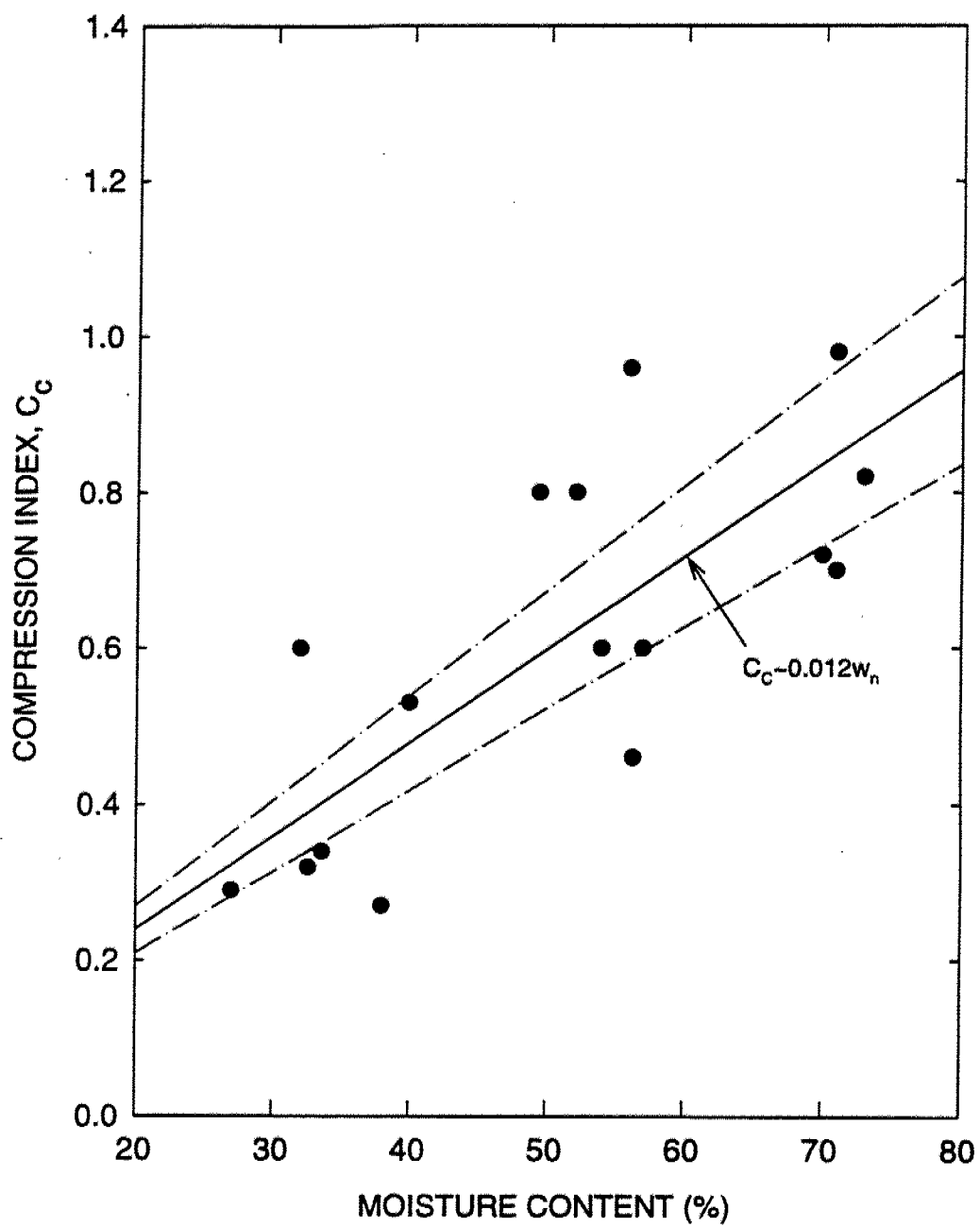


FIGURE 3 RELATIONSHIP BETWEEN MOISTURE CONTENT AND COMPRESSION INDEX,  $C_c$

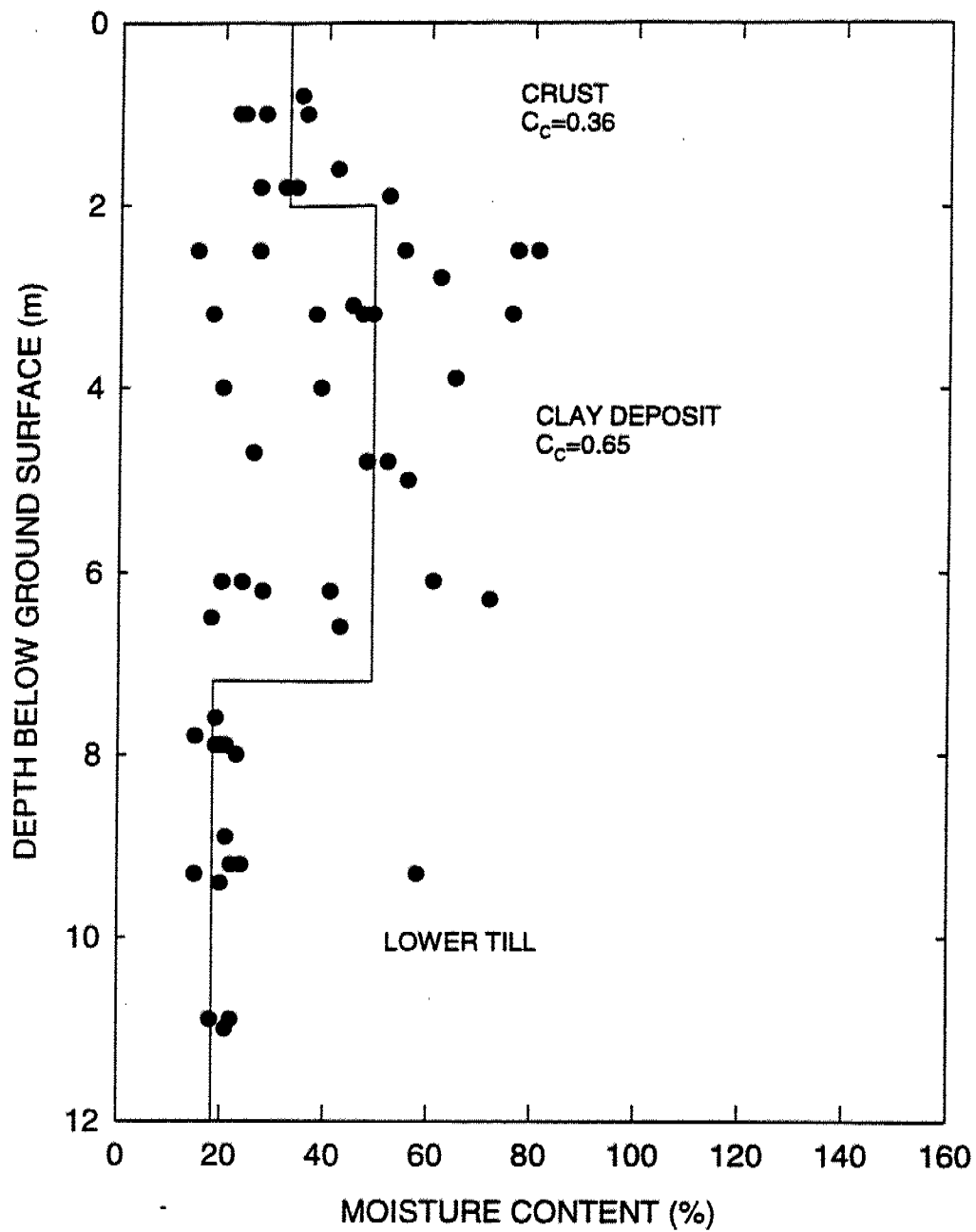


FIGURE 4 SUMMARY OF MOISTURE CONTENTS

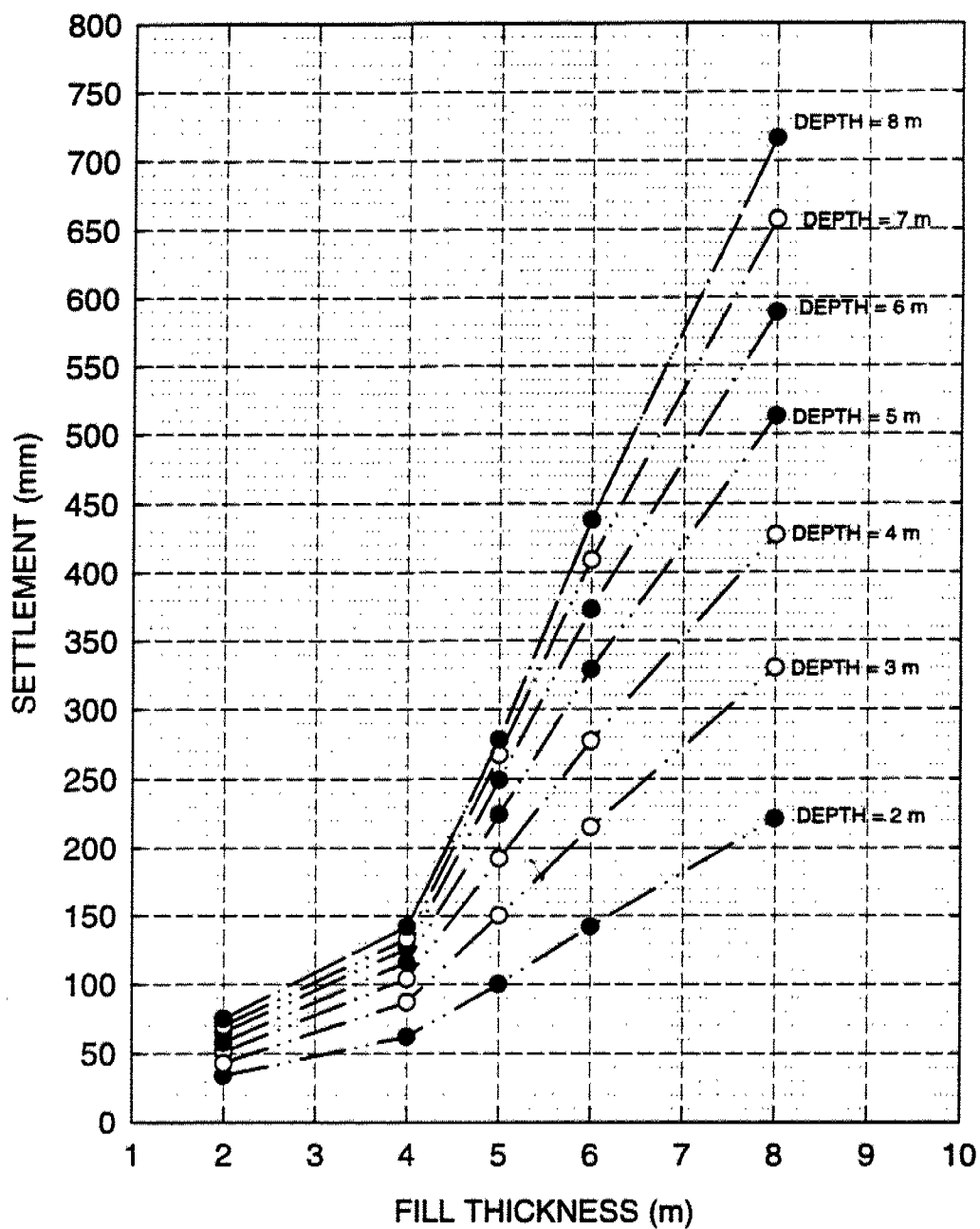


FIGURE 5 CALCULATED SETTLEMENT VERSUS FILL THICKNESS  
HIGHWAY 69



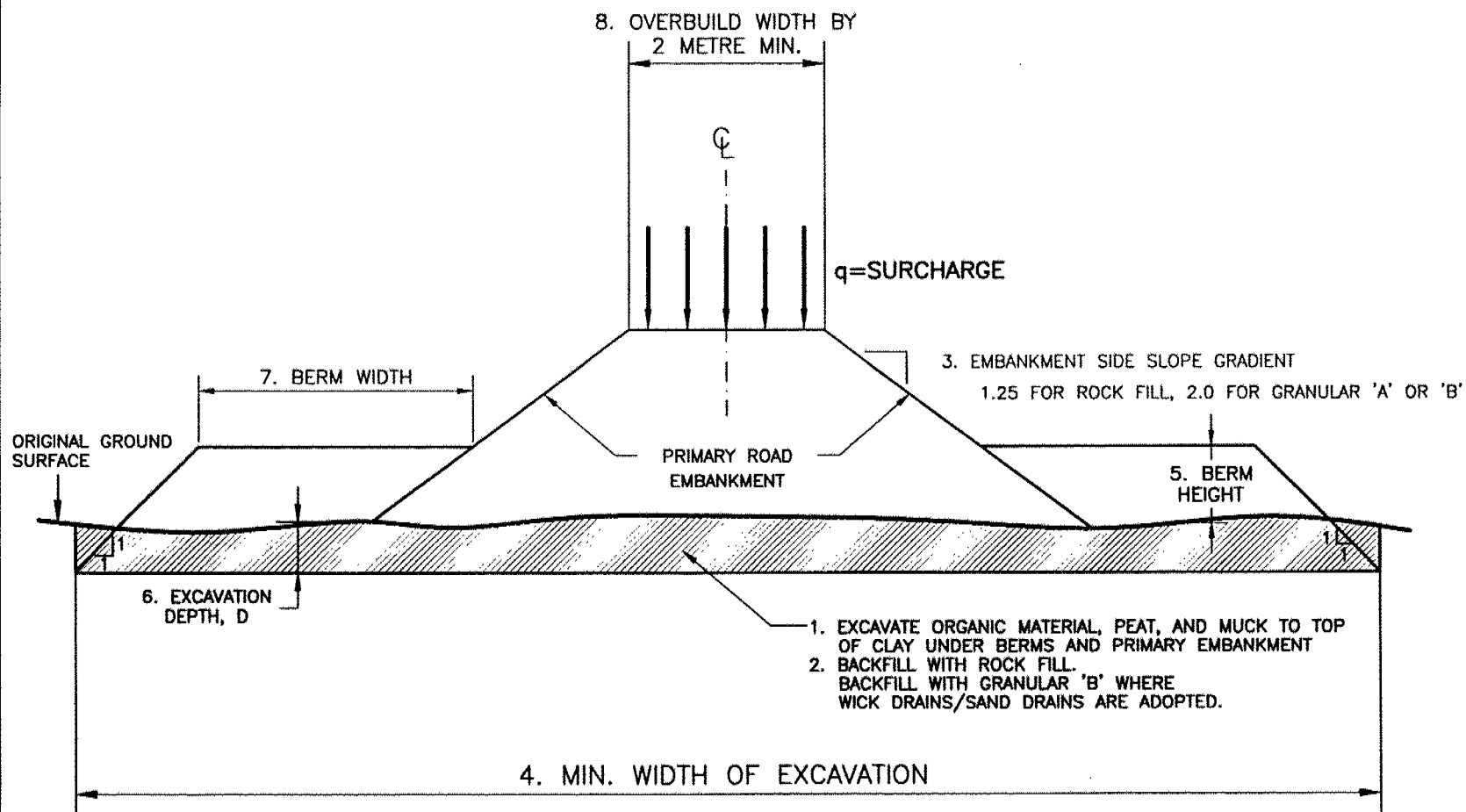
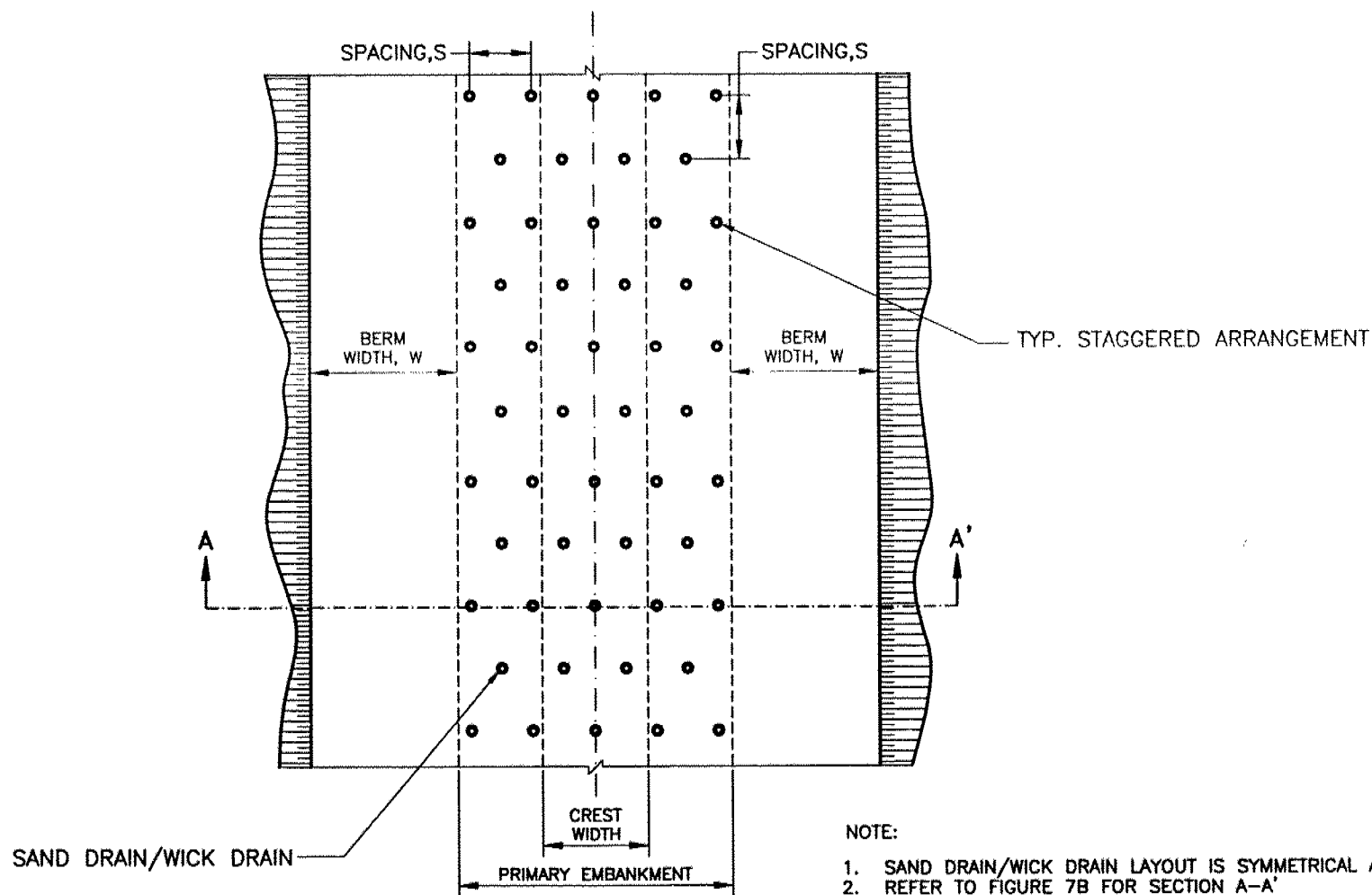


FIGURE 6: GENERALIZED ROAD EMBANKMENT GEOMETRY— HIGHWAY 69, 0.4km SOUTH OF MUSQUASH RIVER, NORTHERLY 8.9 km TO TOWER ROAD.



NOTE:

1. SAND DRAIN/WICK DRAIN LAYOUT IS SYMMETRICAL ABOUT CENTERLINE
2. REFER TO FIGURE 7B FOR SECTION A-A'
3. DRAINS EXTEND TO BOTTOM OF CLAY LAYER
4. DRAINS MUST OUTLET IN GRANULAR 'B' BACKFILL
5. REFER TO FIGURE 6 FOR GENERALIZED EMBANKMENT GEOMETRY

FIGURE 7A: SAND DRAIN/WICK DRAIN LAYOUT. HIGHWAY 69, 0.4km SOUTH OF MUSQUASH RIVER, NORTHERLY 8.9 km TO TOWER ROAD — PLAN VIEW

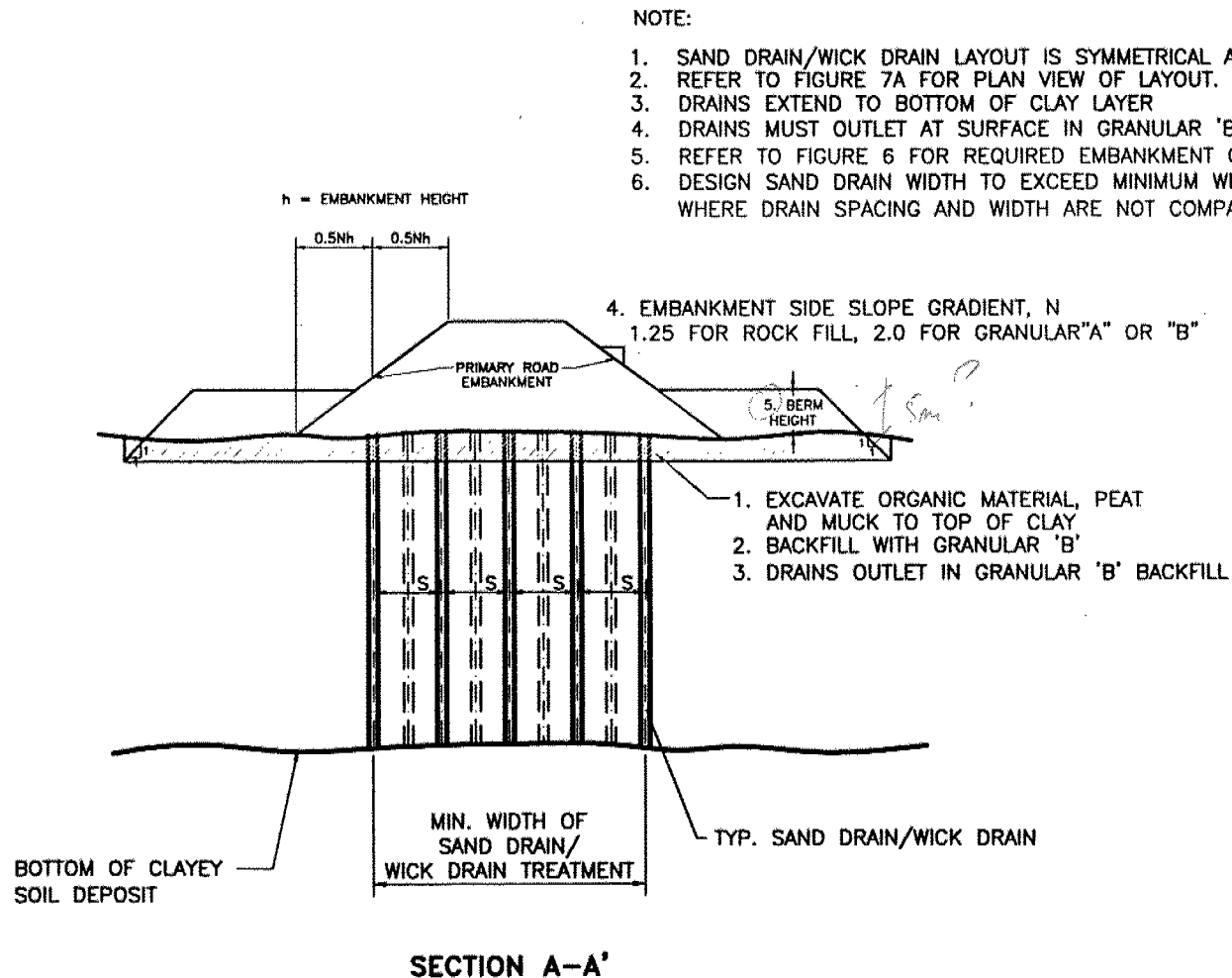


FIGURE 7B: SAND DRAIN/WICK DRAIN LAYOUT. HIGHWAY 69, 0.4km SOUTH OF MUSQUASH RIVER, NORTHERLY 8.9 km TO TOWER ROAD - SECTION VIEW

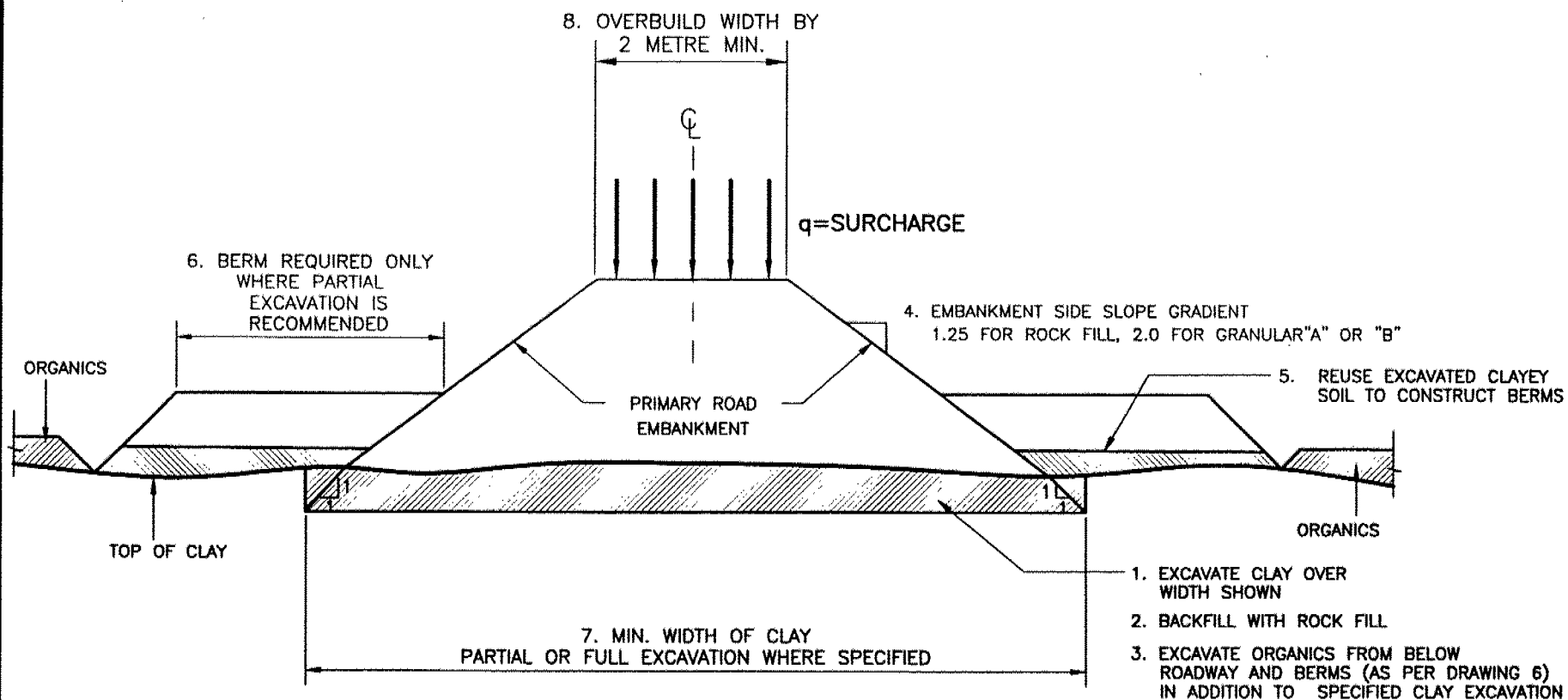


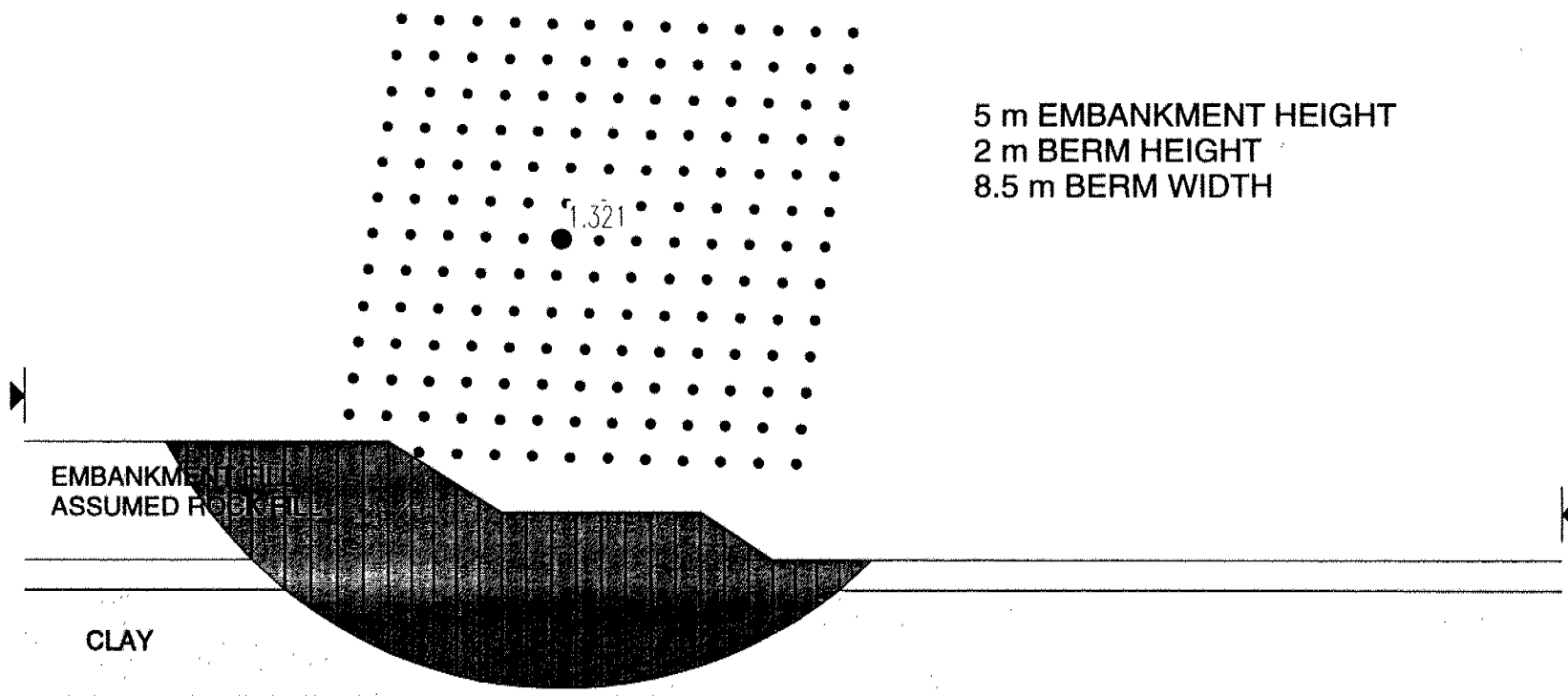
FIGURE 8: ROAD EMBANKMENT GEOMETRY— PARTIAL OR FULL EXCAVATION OF CLAYEY SOIL STATIONS 21+720 TO 21+810, HIGHWAY 69.

## Appendix A: Summary of Stability Calculations

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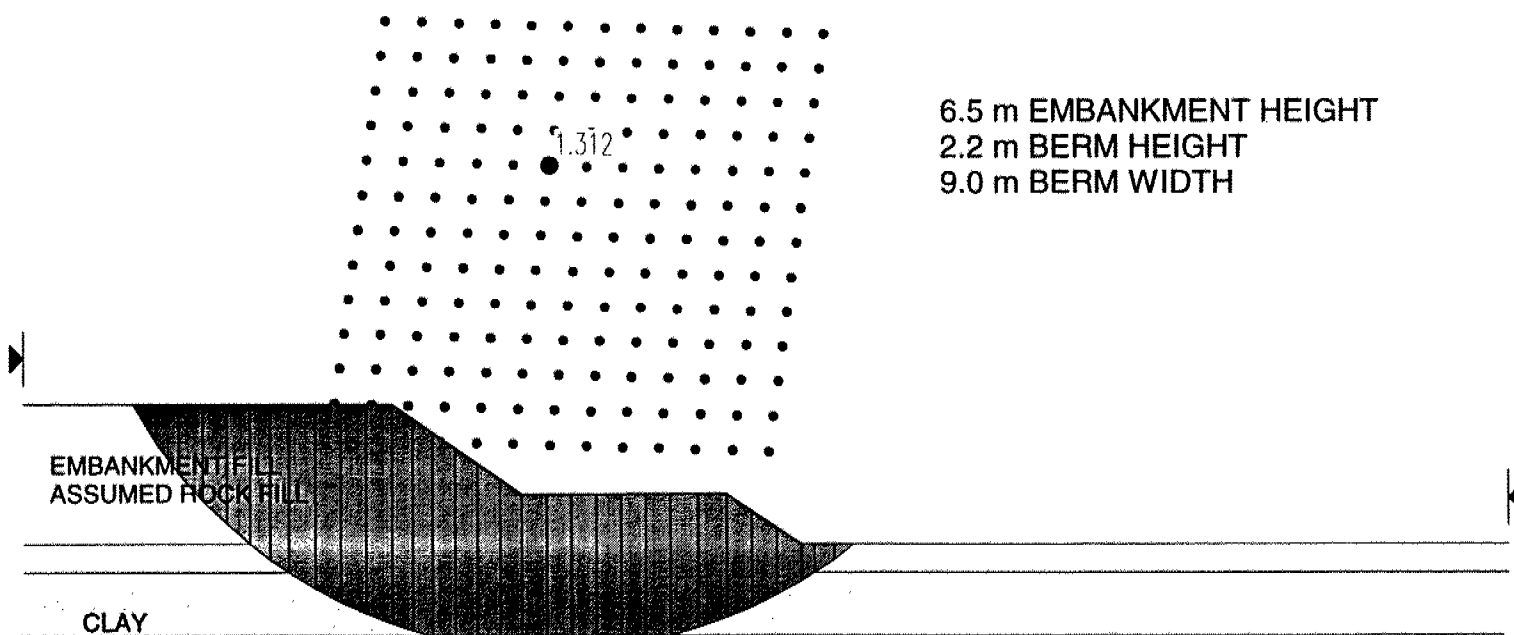
**Figure A1: Southbound Lanes Between Stations 20+050 to 20+350**

**Station 20+150**



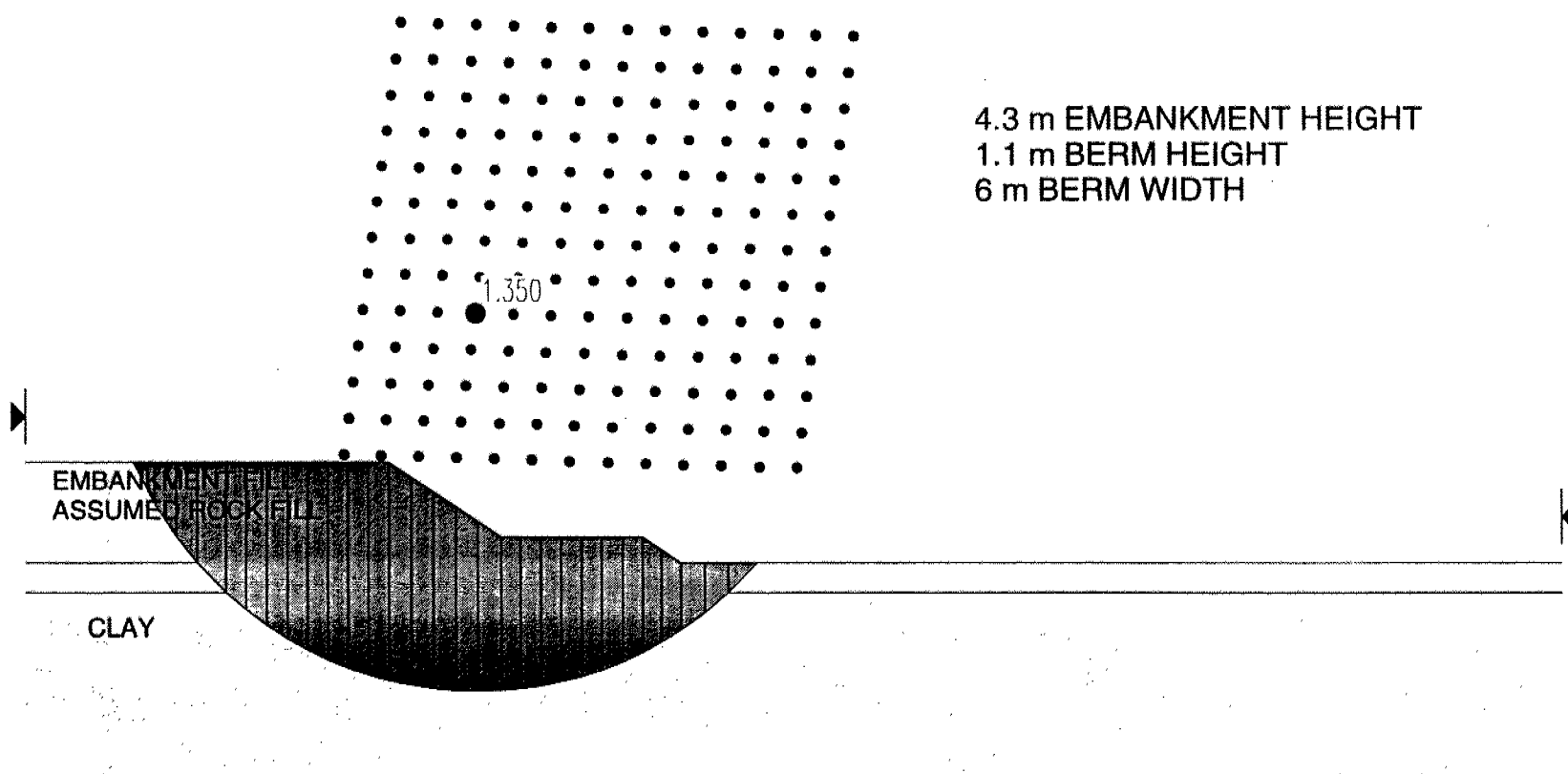
**Figure A2: Southbound Lanes Between Stations 20+425 to 20+790**

**Station 20+750**



**Figure A8: Northbound Lanes Between Stations 20+260 to 20+375**

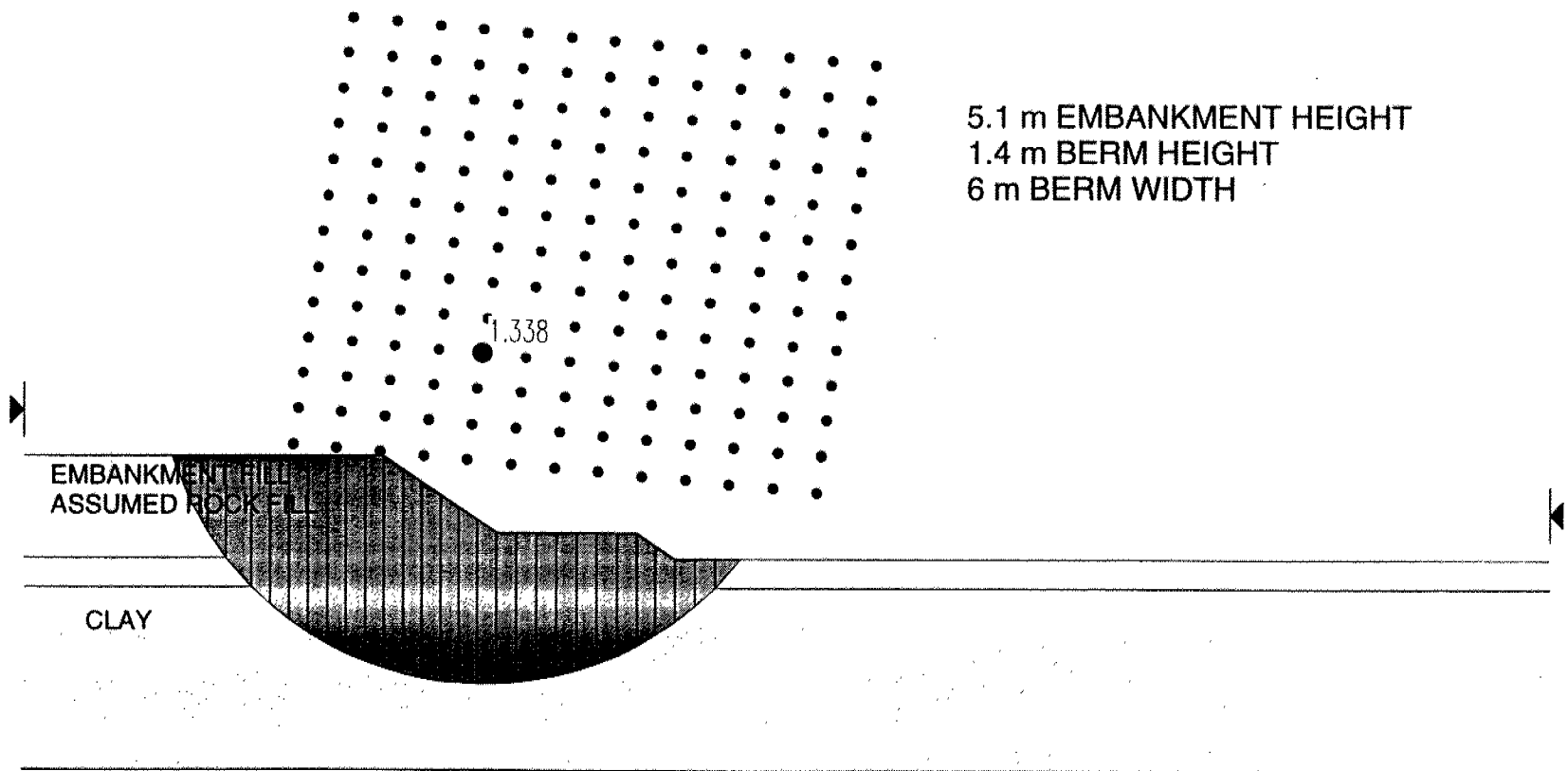
**Station 20+300**





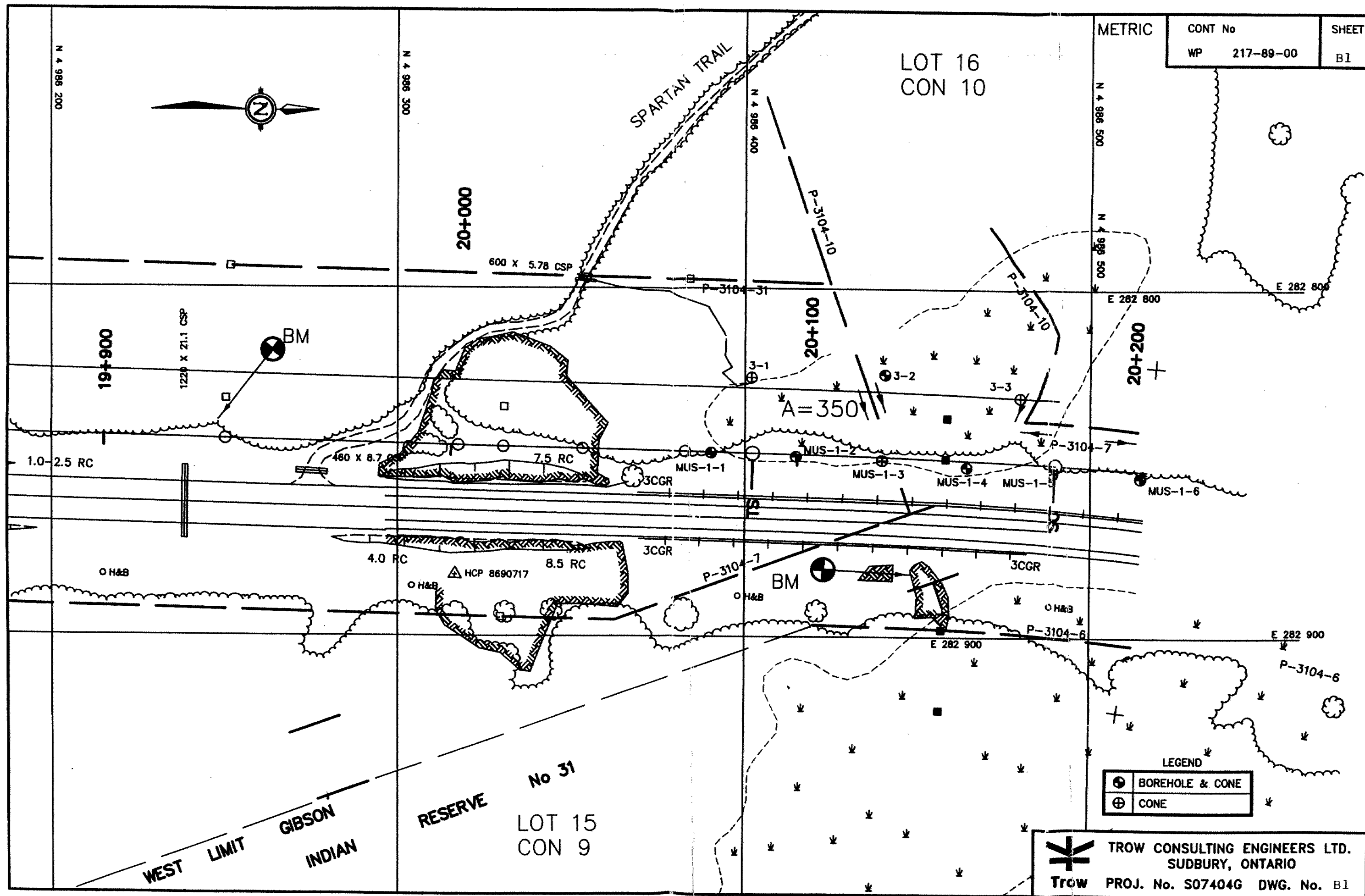
**Figure A9: Northbound Lanes Between Stations 20+450 to 20+780**

**Station 20+600**



## Appendix B: Borehole Logs Northbound Lanes - Highway 69

---





GEOG TWP GIBSON

METRIC

CONT No 217-89-00

SHEET B3

HOC STA: 20+837.001 C/L MED.  
OFFSET: 26.000 L

R=1426

E-S RAMP

Rock

Gravel

Asph.

450 X 18.86 CSP

450 X 27.34 CSP

P-3104-18 CAH

P-3104-6

1.0-2.0 RC

1.0-5.0 E&RC

1.0-6.0 E&RC

950 X 22.28 Conc Culv

N 4 987 200

E 283 100

E 283 000

E 282 900

N 4 987 100

BM

MUS-1-18

MUS-1-19

MUS-1-20

MUS-1-21

MUS-1-22

MUS-1-23

MUS-1-24

MUS-1-25

MUS-1-26

MUS-1-27

MUS-1-28

MUS-1-29

MUS-1-30

MUS-1-31

MUS-1-32

MUS-1-33

MUS-3-17

NBL

C/L

LS

B

H&B

Rock

Gravel

Asph.

450 X 18.86 CSP

450 X 27.34 CSP

P-3104-18 CAH

P-3104-6

1.0-2.0 RC

1.0-5.0 E&RC

1.0-6.0 E&RC

950 X 22.28 Conc Culv

N 4 987 200

E 283 100

E 283 000

E 282 900

N 4 987 100

BM

LEGEND

BOREHOLE & CONE

CONE



TROW CONSULTING ENGINEERS LTD.  
SUDBURY, ONTARIO

Trow PROJ. No. S07404G DWG. No. B3

CONT No  
WP 217-89-00

**SHEET**  
**B3**

### LEGEND

	BOREHOLE & CONE
	CONE



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

Trow PROJ. No. S07404G DWG. No. B3

METRIC

CONT No  
WP 217-89-00

SHEET  
B4

GIBSON INDIAN RESERVE No.31

CON 10  
LOT 14

HOT STA: 21+247.000 C/L MED.  
OFFSET: 26.300 LT

HOC STA: 21+247.000 EW-S RAMP  
OFFSET: 5.550 RT  
N 4987471.556  
E 283249.570

A= 354.863

A=350

R=1500

R=1500

EW-S RAMP

MUS -3-20 C/L SBL

C/L MEDIAN

C/L NBL Asph.

STA: 21+151.995 C/L MED.  
OFFSET: 26.000 RT

21+200 Bush  
LOT 14  
CON 10

LEGEND

⊕ BOREHOLE & CONE  
⊕ CONE



TROW CONSULTING ENGINEERS LTD.  
SUDBURY, ONTARIO

Trow PROJ. No. S07404G DWG. No. B4

HOT 20+995.825 S-EW RAMP

HOT STA: 20+995.721HWY. 69 C/L MED.

OFFSET: 26.000 RT

21+100

1220 X 1220 X 21.7 CONC CURB RFB

BC

3CGR

3CGR

MUS-1-37

MUS-1-36

MUS-1-35

MUS-1-34

MUS-3-19

MUS-3-18

21+000

21+100

21+200

20+900

2

N 4987 200

E 283 000

N 4987 300

N 4987 400

N 4987 500

1.0-6.0 E&RG

N 4987 100

E 283 200

# RECORD OF BOREHOLE MUS-1-15<sub>1</sub> OF 1

METRIC

W.P. 217-89-00

LOCATION Station 20+475.0, offset 2.0 m right of centreline.

ORIGINATED BY I.D.

DIST 52 HWY 69

BOREHOLE TYPE Hollow Stem Augers /

COMPILED BY M.D.

DATUM Depth below grade

DATE October 29, 1997

CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	WATER CONTENT (%)	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL							
197.36	GROUND SURFACE														
0.00	TOP SOIL, 180 mm over SAND, brown, some silt. (compact)		1	SS	14										
195.36			2	SS	23										
2.00	SILTY CLAY TO CLAYEY SILT, some silt & fine sand seams, brown to grey. (stiff)		3	SS	5										
			4	SS	1										
192.36			5	TW											
5.00	SILT, some cobble sizes & sand content, grey. (loose)		6	SS	6										
190.35															
7.01	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														



# RECORD OF BOREHOLE MUS-1-16<sub>1 OF 1</sub>

METRIC

W.P. 217-89-00 LOCATION Station 20+499.4, offset ~1.6 m left of centreline. ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE Dynamic Cone / COMPILED BY M.D.  
 DATUM Depth below grade DATE October 29, 1997 CHECKED BY I.G.

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					
195.86 0.00	GROUND SURFACE Probable SAND														
195.86 1.00															
	Probable SILTY CLAY to CLAYEY SILT														
191.86 5.00															
	Probable SAND														
188.94 7.92	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER														





# RECORD OF BOREHOLE MUS-1-17<sub>1</sub> OF 1

METRIC

W.P. 217-89-00 LOCATION Station 20+525.2, on centreline. ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE Hollow Stem Augers / COMPILED BY M.D.  
 DATUM Depth below grade DATE October 29, 1997 CHECKED BY I.G.

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	20	40	60					
197.97	GROUND SURFACE														
0.00	TOPSOIL, *200 mm over SAND, brown. (compact)		1	SS	22										
195.99			2	SS	37										
1.98	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														



# RECORD OF BOREHOLE MUS-1-18<sub>1</sub> OF 1

METRIC

W.P. 217-89-00 LOCATION Station 20+549.3, offset 0.2 m left of centreline. ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE Dynamic Cone / COMPILED BY M.D.  
 DATUM Depth below grade DATE October 30, 1997 CHECKED BY I.G.

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: $C_u$ , KPa				WATER CONTENT (%)			
						UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR								
						20	40	60	80	wp	w	wl			
197.87 0.00	GROUND SURFACE														
196.27 1.60	Probable ORGANICS														
194.67 3.20	Probable SAND														
	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER														



# RECORD OF BOREHOLE MUS-1-19<sub>1 OF 1</sub>

**METRIC**

W.P. <u>217-89-00</u>	LOCATION <u>Station 20+577.5, offset 1.1 m right of centreline.</u>	ORIGINATED BY <u>I.D.</u>
DIST <u>52</u> HWY <u>69</u>	BOREHOLE TYPE <u>Hollow Stem Augers /</u>	COMPILED BY <u>M.D.</u>
DATUM <u>Depth below grade</u>	DATE <u>October 30, 1997</u>	CHECKED BY <u>I.G.</u>

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					wp ——— w ——— wl			WATER CONTENT (%)						
						SHEAR STRENGTH: Cu, KPa														
						UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR														
						20 40 60 80				10 20 30 40			10 20 30 40							
197.06 0.00	<b>GROUND SURFACE</b>																			
	TOPSOIL, ~200 mm over SILTY CLAY TO CLAYEY SILT, seams of silt & fine sand, grey. (stiff to firm)																			
		1	SS	7																
		2	SS	2																
		3	SS	2																
		4	TW	2																
193.10 3.96	<b>END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER</b>																			



# RECORD OF BOREHOLE MUS-1-20<sub>1</sub> OF 1

METRIC

W.P. 217-89-00

LOCATION Station 20+592.9, offset 0.9 m left of centreline.

ORIGINATED BY I.D.

DIST 52 HWY 69

BOREHOLE TYPE Dynamic Cone /

COMPILED BY M.D.

DATUM Depth below grade

DATE October 30, 1997

CHECKED BY I.G.

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	UNCONFINED QUICK TRIAXIAL						
197.30 0.00	GROUND SURFACE													
196.10 1.20	Probable PEAT													
	Probable SILTY CLAY to CLAYEY SILT													
191.30 6.00	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER													



# RECORD OF BOREHOLE MUS-1-21 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station 20+621.0, offset 0.7 m left of centreline. ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE Hollow Stem Augers / COMPILED BY M.D.  
 DATUM Depth below grade DATE October 30, 1997 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	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	20 40 60 80					
196.49	GROUND SURFACE												
0.00	PEAT, ~460 mm over SILTY CLAY TO CLAYEY SILT, some thin seams of silt & fine sand, grey, (firm/stiff to ~2 m depth then soft/firm)		1	SS	3								
			2	SS	1								
			3	TW									
			4	SS	0								
			5	SS	0								
			6	TW									
188.49			7	SS	3								
8.00	SAND, grey, brown, (compact to dense)		8	SS	31								
			9	SS	23								
184.49			10	SS	76								
12.00	SILTY SAND & GRAVEL TILL, brown												
183.84													
12.65	(dense)												
	END OF BOREHOLE												



# RECORD OF BOREHOLE MUS-1-22 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station 20+655.4, offset ~2.7 m left of centreline. ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE Dynamic Cone / COMPILED BY M.D.  
 DATUM Depth below grade DATE October 30, 1997 CHECKED BY I.G.

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/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/30.3m						20
197.90 0.00	GROUND SURFACE												
195.90 2.00	Possible FILL												
	Probable SILTY CLAY to CLAYEY SILT												
	Probable GRAVEL inclusions below ~15 m depth												
175.50 22.40	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER												



# RECORD OF BOREHOLE MUS-1-23<sub>1</sub> OF 1

METRIC

W.P. 217-89-00 LOCATION Station 20+673.9, offset 0.4 m left of centreline. ORIGINATED BY I.D.  
 DIST 52 HWY 60 BOREHOLE TYPE Hollow Stem Augers / COMPILED BY M.D.  
 DATUM Depth below grade DATE October 31, 1997 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	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					
198.24	GROUND SURFACE											
0.00	SAND & GRAVEL FILL, 300 mm over SAND, organic inclusions, traces of peat, dark brown. (very loose)		1	SS	3							
196.24			2	SS	6							
2.00	SILTY CLAY, small seams of silt & fine sand. (firm)		3	SS	1							
195.24			4	SS	23							
3.00	SAND, grey/brown. (loose to compact)		5	SS	7							
			6	SS	10							
			7	SS	19							
			8	SS	24							
			9	SS	21							
			10	SS	29							
184.07			11	SS	32							
14.17	END OF BOREHOLE											



# RECORD OF BOREHOLE MUS-1-24 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station 20+701.2, offset 0.1 m left of centreline. ORIGINATED BY J.D.  
 DIST 52 HWY 69 BOREHOLE TYPE Hollow Stem Augers / COMPILED BY M.D.  
 DATUM Depth below grade DATE October 31, 1997 CHECKED BY I.G.

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
198.44	GROUND SURFACE														
0.00	FILL, sand & gravel with pieces of asphalt & pockets of peat, brown. (compact)														
196.44															
2.00	SILTY CLAY TO CLAYEY SILT, seams of silt & fine sand, red/brown & grey. (firm)														
			2	SS	9										
			3	SS	2										
189.60	END OF SAMPLED BOREHOLE														
8.84															
	Probable SAND														
186.25															
12.19	END OF CONE TEST														
	Note: Dynamic cone penetration test driven in bottom of sampled borehole, at 9 m depth to refusal at 12.2 m depth.														





# RECORD OF BOREHOLE MUS-1-25 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station 20+703.2, offset 0.1 m left of centreline. ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE Dynamic Cone / COMPILED BY M.D.  
 DATUM Depth below grade DATE October 30, 1997 CHECKED BY I.G.

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		20	40	60	80	wp	w	wl			
198.39 0.00	GROUND SURFACE														
196.39 2.00	Possible FILL	F													
191.39 7.00	Probable SILTY CLAY to CLAYEY SILT														
185.22 13.17	Probable SANDY SILT to SAND														
	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER														



# RECORD OF BOREHOLE MUS-1-26<sup>1 OF 1</sup>

METRIC

W.P. 217-89-00

LOCATION Station 20+725.2, offset 0.2 m left of centreline.

ORIGINATED BY I.D.

DIST 52 HWY 69

BOREHOLE TYPE Hollow Stem Augers /

COMPILED BY M.D.

DATUM Depth below grade

DATE October 31, 1997

CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST IN-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					
198.02	GROUND SURFACE														
0.00	GRANULAR FILL, 75 mm over SAND & GRAVEL FILL, pieces of wood & other organics, grey. (very loose)	TI	1	SS	3										
195.62			2	SS	4										
2.40	SILTY CLAY TO CLAYEY SILT, small seams of silt & fine sand, red/brown & grey. (stiff to firm)		3	SS	5										
			4	SS	2										
			5	TW											
191.77	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
6.29	Note: Chemical and/or gasoline smells were observed in upper 2 m of this borehole.														



# RECORD OF BOREHOLE MUS-1-27 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station 20+762.0, offset ~0.4 m left of centreline. ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE Dynamic Cone / COMPILED BY M.D.  
 DATUM Depth below grade DATE November 3, 1997 CHECKED BY I.G.

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		
198.15 0.00	GROUND SURFACE														
196.65 1.50	Possible ORGANICS & CLAY														
	Probable SAND with GRAVEL														
192.08 6.07	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER														



# RECORD OF BOREHOLE MUS-1-28<sup>1 OF 1</sup>

METRIC

W.P. 217-89-00 LOCATION Station 20+772.6, offset ~0.6 m right of centreline. ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard Augers / COMPILED BY M.D.  
 DATUM Depth below grade DATE October 31, 1997 CHECKED BY I.G.

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	20 40 60 80	wp			w
198.09 0.00	GROUND SURFACE  ORGANICS, ~460 mm over SAND & GRAVEL, some silt with cobbles & possible boulders, grey/brown. (compact)		1	SS	17								
			2	SS	27								
			3	SS	19								
			4	SS	30								
193.76 4.33	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER												



## METRIC

ORIGINATED BY I.D.

COMPILED BY M.D.

CHECKED BY I.G

MT04 7404A 12/12/97



# RECORD OF BOREHOLE MUS-1-30<sub>1 OF 1</sub>

METRIC

W.P. 217-89-00 LOCATION Station 20+829.6, offset 0.7 m right of centreline. ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE Hand Power Augers / COMPILED BY M.D.  
 DATUM Depth below grade DATE October 22, 1997 CHECKED BY I.G.

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			20	40	60	80					
199.29	GROUND SURFACE						199									
0.00	SILTY SAND, brown, moist.															
192.99																
1.30	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BOULDER OR BEDROCK															



# RECORD OF BOREHOLE MUS-1-31 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station 20+853.8, offset 0.5 m left of centreline. ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard Augers / COMPILED BY M.D.  
 DATUM Depth below grade DATE November 3, 1997 CHECKED BY I.G.

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	20	40	60	80	wp	w	wl	10	20	30	40			kN/m <sup>3</sup>	GR
198.02	GROUND SURFACE																					
0.00	TOPSOIL, 200 mm over SILTY CLAY TO CLAYEY SILT, seams of silt & fine sand, brown & grey. (soft)		1	SS	39																	
			2	SS	1																	
195.02			3	AS																		
3.00	SANDY SILT TO FINE SAND, grey. (loose to compact)		4	SS	5																	
192.69																						
5.33	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER																					



# RECORD OF BOREHOLE MUS-1-32 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station 20+876.1, offset ~9.1 m left of centreline. ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE Dynamic Cone / COMPILED BY M.D.  
 DATUM Depth below grade DATE November 3, 1997 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST 20 40 60 80	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT	WATER CONTENT (%) w <sub>p</sub> w <sub>L</sub>	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER							
197.72 0.00	GROUND SURFACE									
188.72 9.00	Probable SANDY SILT with a trace of clay									
187.05 10.67	Probable SAND & GRAVEL									
	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER									





# RECORD OF BOREHOLE MUS-1-33<sup>1</sup> OF 1

METRIC

W.P. 217-89-00

LOCATION Station 20+901.8, offset ~10.9 m left of centreline.

ORIGINATED BY I.D.

DIST 52 HWY 69

BOREHOLE TYPE Standard Augers /

COMPILED BY M.D.

DATUM Depth below grade

DATE November 3, 1997

CHECKED BY I.G.

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
197.73	GROUND SURFACE														
0.00	ORGANICS, ~100 mm over SILTY CLAY TO CLAYEY SILT, seams of silt & fine sand, brown. (firm/stiff)		1	SS	19										
			2	TW											
195.23			3	SS	0										
2.50	SANDY SILT, occasional small seams of silty clay, grey. (loose)		4	SS	0										
			5	SS	5										
			6	SS	8										
			7	SS	4										
188.59															
9.14	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														



## METRIC

ORIGINATED BY I.D.

COMPILED BY M.D

CHECKED BY I.G.



# RECORD OF BOREHOLE MUS-1-35 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station 20+950. ~12.4 m right of centreline. ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE Hand Power Augers / COMPILED BY M.D.  
 DATUM Depth below grade DATE October 22, 1997 CHECKED BY I.G.


SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
199.60	GROUND SURFACE														
0.00	ASPHALT, ~150 mm over SAND & GRAVEL FILL, ~300 mm over					199									
198.20	SAND, light brown becoming red/brown between ~0.7 to 1.0 m depth then dark brown, moist. (compact)					198									
1.40															
197.20	SILTY CLAY, some organic content, dark brown, moist (firm)					197									
2.40															
196.60	SILTY SAND, light brown, moist. (compact)														
3.00	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														



# RECORD OF BOREHOLE MUS-1-36<sub>1</sub> OF 1

METRIC

W.P. 217-89-00 LOCATION Station 20+975, ~6.0 m left of centreline. ORIGINATED BY I.D.  
 DIST 52 HWY 69 BOREHOLE TYPE Hand Power Augers COMPILED BY M.D.  
 DATUM Depth below grade DATE October 22, 1997 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) 				PLASTIC LIMIT wp	NATURAL MOISTURE CONTENT w	LIQUID LIMIT wl	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	CONE PENETRATION TEST 20 40 60 80							
198.02	GROUND SURFACE													
197.02	TOPSOIL ~50 mm over													
0.30	SILTY SAND, dark brown, moist.													
	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER													



# RECORD OF BOREHOLE MUS-1-37 1 OF 1

METRIC

W.P. 217-89-00

LOCATION Station 21+000, offset 5.1 m right of centreline.

ORIGINATED BY I.D.

DIST 52 HWY 69

BOREHOLE TYPE Hand Power Augers /

COMPILED BY M.D.

DATUM Depth below grade

DATE October 22, 1997

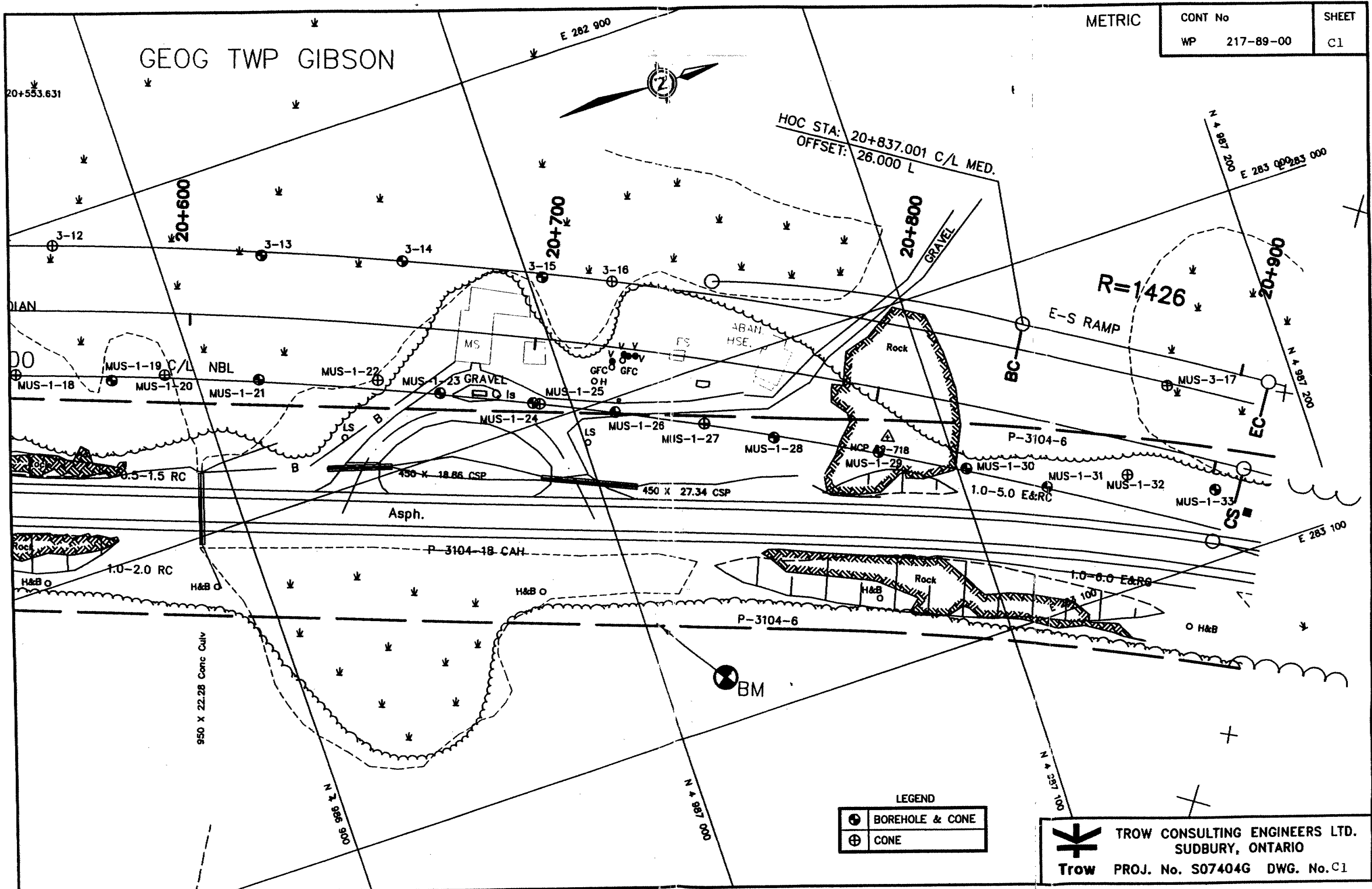
CHECKED BY I.G.

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					
199.70	GROUND SURFACE															
0.00	ASPHALT, 180 mm over SAND & GRAVEL FILL, 270 mm thick then						199									
198.40	SAND, light brown to 0.9 m depth then red/brown. (compact)						198									
1.30																
197.70	SILTY CLAY, dark brown, moist. (firm)						197									
2.00																
196.70	SILTY SAND, dark brown, wet. (compact to 2.4 m depth then loose)															
3.00	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER															



## Appendix C: Borehole Logs Southbound Lanes - Highway 69

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METRIC

CONT No

WP 217-89-00

SHEET

C2

GIBSON INDIAN RESERVE No.31

CON 10  
LOT 14

HOT STA: 21+247.000 C/L MED.  
OFFSET: 26.300 LT

HOC STA: 21+247.000 EW-S RAMP  
OFFSET: 5.550 RT  
N 4987471.556  
E 283249.570

A= 354.863

A= 350

R=1500

R=1500

STA: 21+151.995 C/L MED.  
OFFSET: 26.000 RT

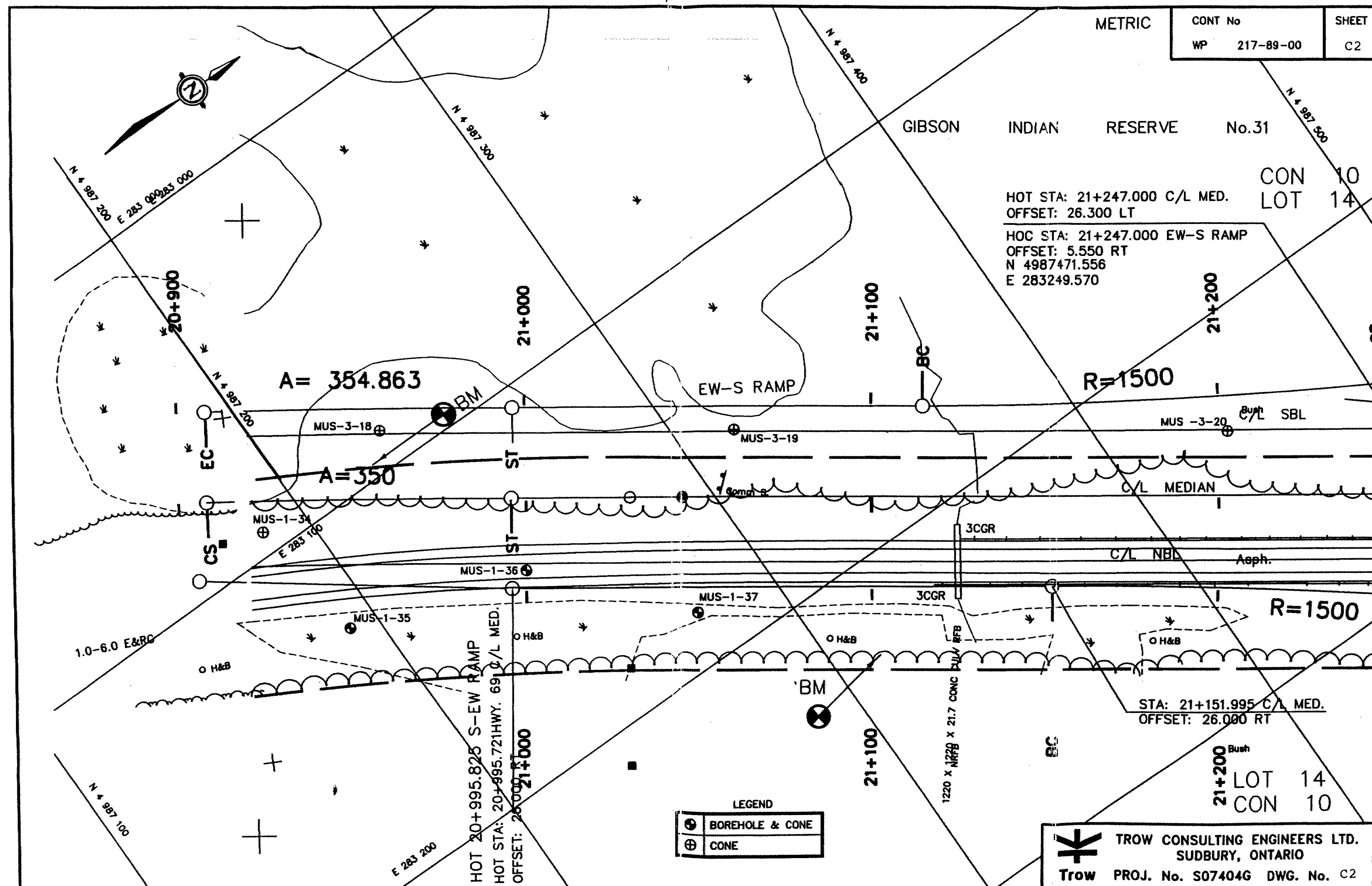
LEGEND

⊕	BOREHOLE & CONE
⊗	CONE



TROW CONSULTING ENGINEERS LTD.  
SUDBURY, ONTARIO

Trow PROJ. No. S07404G DWG. No. C2





# RECORD OF BOREHOLE MUS-3-17 1 OF 1

METRIC

W.P. 217-89-00

LOCATION Station 20+881.3, offset 0.8 m right of centreline.

ORIGINATED BY I.D.

DIST 52 HWY 69

BOREHOLE TYPE Standard Augers

COMPILED BY M.D.

DATUM Geodetic

DATE November 3, 1997

CHECKED BY A.S.

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	wp	w	wl		
197.64	GROUND SURFACE												
0.00	ORGANICS, 250 mm over SILTY CLAY, trace of fine sand, grey/brown. (stiff to firm)		1 SS 17		197							17.00	0 56 44 0
195.64	300 mm thick layer of compact SILT & FINE SAND encountered at 0.8 m depth.		2 TW		196								0 7 45 44
2.00	SILT & FINE SAND, occasional clay seams below 6.0 m depth, grey/brown to grey. (loose to very loose)		3 SS 4		195								0 58 42 0
			4 SS 8		194								
			5 SS 10		193								0 58 42 0
			6 SS 0		192								
					191								
190.04					190								
7.60	SILT, SAND & GRAVEL TILL, grey, (loose to compact)		7 SS 10		Bouncing								27 29 49 0
189.78	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER												
7.86	Notes: 1) A split spoon "N" value of 0 means that the split spoon sampler sank under the weight of the hammer & rods. 2) Atterberg Limits Test Results: Plastic limit = 15% Liquid Limit = 53% Natural Moisture Content = 55%												





# RECORD OF BOREHOLE MUS-3-19<sub>1 OF 1</sub>

METRIC

W.P. 217-89-00

LOCATION Station 21 - 060.4, offset 0.7 m left of centreline.

ORIGINATED BY I.D.

DIST 52 HWY 69

BOREHOLE TYPE Standard Augers

COMPILED BY M.D.

DATUM Geodetic

DATE November 4, 1997

CHECKED BY A.S.

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					
198.07	GROUND SURFACE													
0.00	TOPSOIL, 200 mm over SILT & SAND, trace of gravel, grey, moist then wet. (compact)		1	SS	18									
196.33			2	SS	30									
1.74	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER													



# RECORD OF BOREHOLE MUS-3-20<sub>1</sub> OF 1

METRIC

W.P. 217-89-00

LOCATION Station 21+202.5, offset ~0.7 m right of centreline.

ORIGINATED BY I.D.

DIST 52 HWY 69

BOREHOLE TYPE Dynamic Cone /

COMPILED BY M.D.

DATUM Geodetic

DATE November 4, 1997

CHECKED BY A.S.

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/30 in	20	40	60						80	Wp	W
199.91	GROUND SURFACE																	
0.00	Probable PEAT																	
199.31																		
0.60																		
197.81	Probable SILTY CLAY/CLAYEY SILT																	
2.10																		
	Probable SILT & SAND																	
193.81																		
6.10																		
192.90	Probable SILT, SAND & GRAVEL TILL																	
7.01																		
	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER																	



## Appendix D: Previous MTO Report

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FOUNDATION INVESTIGATION REPORT  
For  
Approach Embankments  
Along Highway 69 New - From Approximately 15.3 km North of  
M.R. #5, Northerly 13 km  
Location 3 - Station 20+080 to 20+800  
(Excluding the Musquash River Bridge)  
W.P. 215-89-00(C)  
Northern Region  
District 11, Huntsville

INTRODUCTION

At the request of the Northern Region, Geotechnical Section, a foundation investigation was carried out for proposed embankments, to be located at the above-captioned site. This report summarizes the factual information obtained from this investigation.

SITE DESCRIPTION AND GEOLOGY

The site is located from approximately 10 m to up to 60 m west of the centreline of the existing Highway 69 embankment, between Stations 20+080 and 20+800, within Lots 15 and 16, Concession X, Gibson Township, District of Muskoka. The investigation includes the area from about 300 m south to 370 m north of the Musquash River, but excludes the river itself (ie. Stations 20+360 to 20+450) and the structure which will be associated with it.

Throughout this area, the existing Highway 69 consists of a roadway with single lanes, running in both the north and south directions. Through a series of rock cuts and embankments, the highway traverses undulating topography consisting of rock knolls of gneissic bedrock separated by low swampy or wooded areas. Drainage is generally towards the west; ie. Georgian Bay.

At this location, Highway 69 has been constructed on a 4.5 to 6 m high embankment (reaching elevations of 198.7 to 204 m). A two-lane, three span bridge crosses the Musquash River.

The site is essentially divided into two separate sections, both of which are covered by swamps. At Site 3A, on the south side of the river, the area is characterized by cattails and small bushes, with occasional sparse (but dead) trees. The area, however, becomes quite wooded at the extreme south end of the site. On the other hand, Site 3B, on the north side of the river, is quite wooded throughout, with dead or dying trees of up to 200 mm in diameter. In most areas, water is pooled at the surface ie. the groundwater table is at or near the existing ground surface.

## PROCEDURES

The fieldwork was carried out, during the period between April 2 and 14, 1992, and consisted of 9 sampled boreholes (Boreholes 3-2, 3-4, 3-6, 3-8, 3-9, 3-11 and 3-13 to 3-15), which were advanced to depths of 5.2 to 18.9 m, using continuous flight, hollow stem augers driven by a bombardier-mounted drilling rig, equipped with standard soil sampling equipment.

Dynamic cone penetration tests were also carried out adjacent to each of the boreholes (often to greater depth than the borehole itself) and at seven other locations (ie. 3-1, 3-3, 3-5, 3-7, 3-10, 3-12 and 3-16).

Soil samples were recovered using a 50 mm OD split spoon sampler, driven into the soil in accordance with the specifications of the Standard Penetration Test (ASTM D 1586). Field vane tests were also carried out wherever soft to firm cohesive soils were encountered in the boreholes.

Groundwater levels, were measured in several of the open boreholes, immediately upon completion of sampling and some of these were left open for at least 24 hours, in order to measure the longer term groundwater conditions.

The boreholes were staked out in the field by the Northern Region Surveys and Plans Office. Small changes in the locations and elevations of the boreholes were determined by our field representatives.

The soil samples, which were obtained in the field, were examined in the laboratory by visual and tactile methods. Moisture content, Unit Weight, Atterberg Limits and Grain Size Distribution tests were carried out on selected soil samples. A consolidation test was carried out on one of the samples.

## SUBSURFACE CONDITIONS

The subsurface conditions, at the boreholes, generally consist of a thin layer of brownish grey to black, soft to firm, peat, topsoil or organic clayey silt, from 0.2 to 1.2 m thick, which is, in turn, underlain by an extensive deposit of brownish grey to grey, soft to firm (or occasionally stiff), silty clay to clay, which reaches depths of up to 13.7 m.

A layer of brown to grey, loose to compact, sandy silt to well-graded sand, from 0.9 to about 13 m thick was encountered beneath the clay, at depths of 5.3 to 13.7 m, in the boreholes. The probable bedrock surface, was encountered below the sandy silt to sand, at depths of 2.7 to 21.3 m, in the boreholes and cone tests.

As indicated previously, the site can generally be divided into two areas; Site 3A on the north and Site 3B, on the south sides of Musquash River, respectively. Although the subsoils, at both areas, are quite similar, the shear strength of the clay is somewhat higher at Site 3B.

The groundwater table was found to be at or close to the existing ground surface (ie. elevations of 196.4 to 196.8 m).

Details of the subsurface information, obtained from this investigation, are included on the borehole logs and on Figures 1 to 3, at the back of this report. Brief descriptions of the individual soil strata and the groundwater conditions which were encountered in the boreholes, are given below.

#### Organic Soils - Peat/Organic Clayey Silt/Topsoil

A layer of brownish grey to black, soft, peat, organic clayey silt, or topsoil, from 0.2 to 1.2 m thick, was contacted at the ground surface, in all of the boreholes.

#### Fine Sand

Beneath a thin (200 mm thick) layer of topsoil, one of the Boreholes (3-9), contacted a layer of loose to compact, fine to silty fine sand, about 1.7 m thick. This sand, likely represents a portion of the fluvial deposits associated with the river.

A moisture content of 19 percent was measured in one of the soil samples obtained.

Since the ground was likely to be still frozen at this depth, the recorded 'N'-value of 14 blows/0.3 m is probably unrepresentative and the soil is likely to be quite loose.

#### Silty Clay to Clayey Silt

Beneath the organic soils (and, at Borehole 3-9, the fine sand deposit described above), at depths of 0.3 to 1.9 m (or elevations of 195.3 to 196.2 m), all of the boreholes contacted a major deposit of brownish grey to grey, soft to firm, silty clay to clayey silt, from 4.4 to 11.8 m thick.

Atterberg limits tests, which were carried out on several samples of soil obtained from this deposit, gave liquid limits and plasticity indices ranging from 26 to 53 (average of 36) and 12 to 32 (average of 20) percent, respectively. These results, which are shown on Figure 1, indicate soils which can generally be classified as silty clay to clayey silt.



Moisture contents, which were measured in several samples obtained from this deposit, ranged from 20 to 64 (average of 45) percent.

'N'-values, measured in these clayey soils, were generally less than 4 blows/0.3 m. At several sampling intervals, the rods and split spoon sampler simply sank under their own self weight. It should be noted that, higher 'N'-values which were recorded in the upper portion of the deposit (ie. at Boreholes 3-2, 3-8, 3-9 and 3-11), are likely to be unrepresentative, since the ground was probably still frozen at those shallow depths, when testing was carried out.

Field vane shear tests gave measured shear strengths ranging from 11 to 120 kPa. South of Musquash River (ie. at Site 3A), shear strengths averaged about 14 kPa. However, somewhat higher values (all greater than 20 kPa), were measured in the clay to the north of the Musquash River (Site 3B). In any case, these results indicate soils of generally soft to firm consistency.

A consolidation test was carried out on a sample of soil obtained from this deposit. The results, shown on Figure 2, indicate that this soil sample was lightly preconsolidated with a compression index of 0.96.

#### Sandy Silt to Medium Sand

A deposit of brown to grey, loose to compact, sandy silt to medium sand, approximately 0.3 to 13.0 m thick, was encountered at depths of 4.9 to 13.7 (or elevations of 183.5 to 191.8 m), in the boreholes.

Grain size distribution tests, carried out on samples obtained from this deposit, and shown on Figure 3, indicate sand, silt and clay-sized particles ranging from 26 to 92, 6 to 68 and 2 to 11 percent, respectively.

Moisture contents of 16 to 26 (average of 22) percent were measured, in several samples obtained from this cohesionless deposit.

'N'-values ranging from 0 to 35 blows/0.3 m indicate generally loose to compact soils with occasional denser zones. The lower 'N'-values', particular those of 0 (ie. the split spoon sampler and rods simply sank under their own self weight) to 3 blows/0.3 m are likely to be unrepresentative. Dynamic cone penetration tests indicate that, in most cases, the soil probably became loosened and disturbed, due to conditions of unbalanced hydrostatic head.

### Bedrock

Probable bedrock was encountered in the boreholes and dynamic cone penetration tests, at depths of 2.7 to 21.0 m (or elevations of 175.0 to 195.0 m). It should be noted, however, that, in some areas, during cone penetration tests, the cone appeared to be skipping off of the steeply-inclined bedrock surface. Therefore the elevations of the bedrock surface, should only be considered to be approximate.

Outcrops adjacent to the swamp, and in the immediate area, indicate that the local bedrock is comprised of a hard, granitic gneiss.

### Groundwater Conditions

The groundwater levels, measured in the open boreholes immediately upon completion of sampling, (or at least 24 hours after completion), were generally found to range from 0 to 0.2 m beneath the existing ground surface or elevations of 196.4 to 196.8 m.

It should be noted, however, that the water level measured in Borehole 3-9, was found to be at a depth of 1.9 m (Elevation of 195.3 m), upon completion of sampling. It is likely, however, that the water level, measured in this borehole, does not reflect the true groundwater table since it did not have sufficient time to adequately stabilize.

## DISCUSSIONS AND RECOMMENDATIONS

### General

The existing Highway 69, which extends through the area of investigation, is comprised of an embankment, from 4.5 to 6 m high, with single lanes running in both the north and south directions. The existing pavement appears to be in relatively good condition, although we understand that, in several areas along Highway 69, the highway has been repaved.

It is proposed to use the existing highway embankment as the northbound lanes and to construct a new sub-parallel embankment, from 2.5 to 4 m high, to the west of the existing embankment, which will be used for the new southbound lanes.

As noted previously, the Musquash river divides the site into two separate sections. On the south side of the river, over the south half of Site 3A, the centreline of the proposed embankment will generally be about 20 m to the west of the new median. However, about 200 m south of Musquash River, the proposed embankment begins to curve to the west, away from the existing one. This also continues on the north side of the river, where, the new embankment will be a maximum of about 60 m away from the existing one, about 200 m north of the river. To the north of this, the embankment once again, begins to curve back (ie. to the east), towards the existing embankment.

### Design

In order to construct the embankments to their proposed heights, it will be necessary to excavate all of the peat and a portion of the underlying clay down to an elevation of about 194.5 m.

In order to maintain adequate stability, the slopes of a rockfill embankment, which is constructed to the south of Musquash River (ie. Site 3A), should be no steeper than 2H:1V with a 7 m wide one-third height berm.

However, at Site 3B, the new embankment may have slopes constructed as steeply as 1.5H:1V, and, in this case, berms are not required.

## Construction Considerations

### Excavations

It is expected that temporary subexcavations for the peat and soft clay, to depths of up to 2 m, will be temporarily stable at slopes of 2H:1V. Subexcavation and backfilling should be carried out concurrently and under water, if necessary.

### Raising the Grade

Rockfill or other fills placed below the groundwater table, may be end-dumped. However, once the material is 0.3 m above the groundwater table, placement and compaction of the fill materials should be carried out according to OPSS standards and MTO practice.

### Settlement

Based on a consolidation test, carried out on one sample from this site, it appears that, at Sites 3A and 3B, settlements of about 0.4 to 0.5 m are expected to occur, primarily due to consolidation of the silty clay deposit and to a lesser extent due to compression of the loose underlying sand. It is expected that 90 percent of the consolidation settlement will extend over a period of about three months, following the completion of construction.

Therefore, to reduce the post construction settlements, of the proposed highway, it is recommended that the embankments to the south and north of the abutments of the proposed bridge be overbuilt by about 0.5 m between Stations 20+160 and 20+364 and 20+420 and 20+770. This excludes the centre span of the proposed structure over the Musquash River (ie. Stations 20+364 to 20+420), the details of which are given in the report under Ref. No. W.P. 208-90-01). It should be noted, however, that in our report, Ref. No. W.P. 208-90-01, it has been recommended to move the south abutment about 5 m further to the south. If this is the case, then the surcharging should be between Stations 20+160 and 20+359, on the south side of the proposed bridge. In any case, the surcharge should be tapered to zero thickness, where bedrock is encountered in the excavation (ie. at approximately Stations 20+080 and 20+780).

The settlement of these embankments should be allowed to take place for as long a period as possible but for a minimum of at least four months. After that time, the embankments may be bladed off and the slopes flattened to their final grades. The surcharge material may consist of either granular material or rockfill.

#### MISCELLANEOUS

The field investigation was supervised by Mr. J. Blair of the Foundation Design Section and Mr. Dan Rothwell of the Northern Region Geotechnical Section, using equipment owned and operated by Atcost Soil Drilling Inc.

This report was written by Mr. J. Blair, Project Foundation Engineer, reviewed by Mr. D. Dundas, Senior Foundation Engineer and approved by Mr. M. Devata, Chief Foundation Engineer.



*John A. Blair*

J. A. Blair, P.Eng.  
Project Foundation Engineer

*M. Devata*  
M. Devata, P.Eng.  
Chief Foundation Engineer

## EXPLANATION OF TERMS USED IN REPORT

**N VALUE:** THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 50mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5 kg FALLING FREELY A DISTANCE OF 0.76m FOR PENETRATIONS OF LESS THAN 0.3m. N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS  $\bar{N}$ .

**DYNAMIC CONE PENETRATION TEST:** CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (50mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 1A 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/0.3m)	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

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

### STRESS AND STRAIN

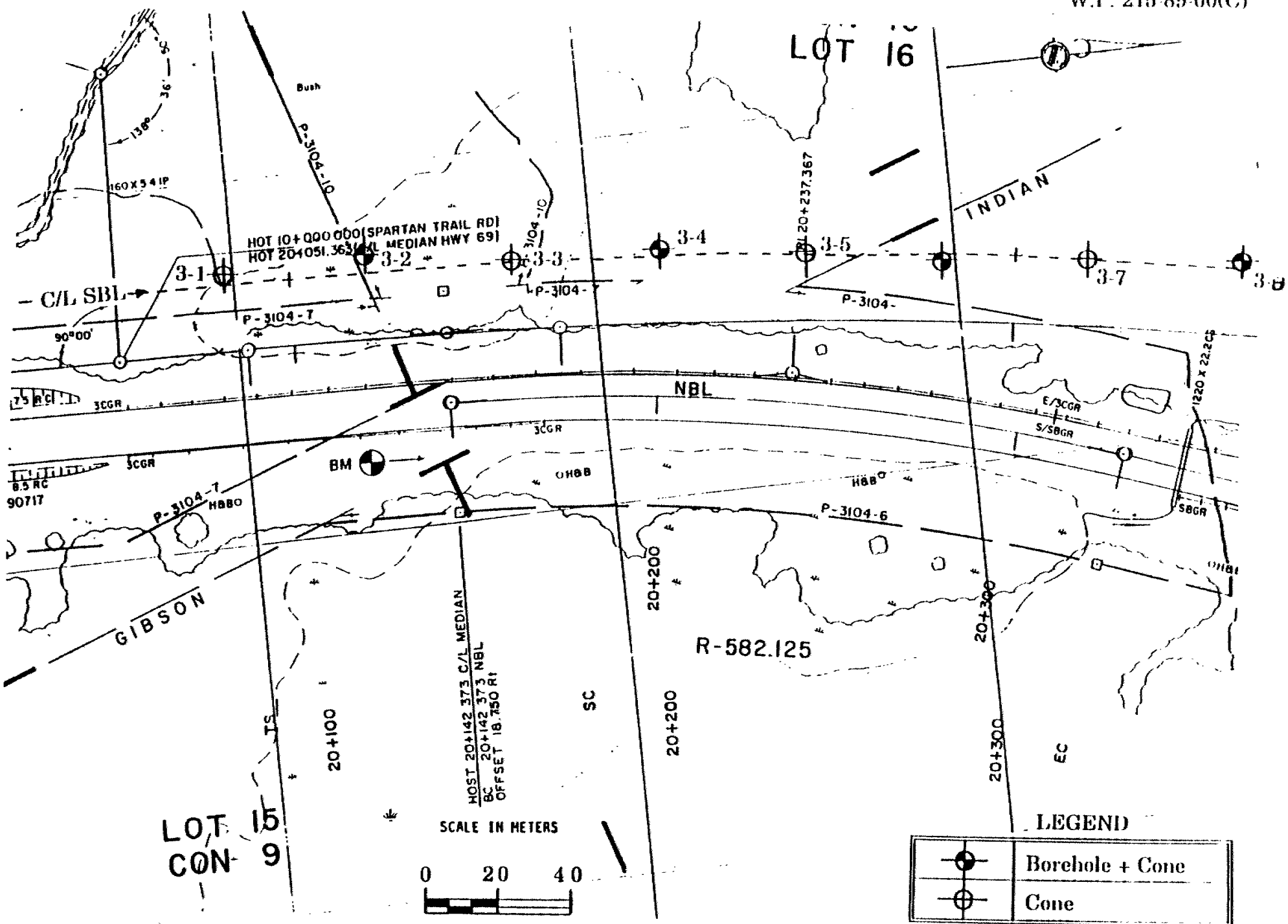
$u_w$	kPa	PORE WATER PRESSURE
$\sigma_v$	kPa	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
$E$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

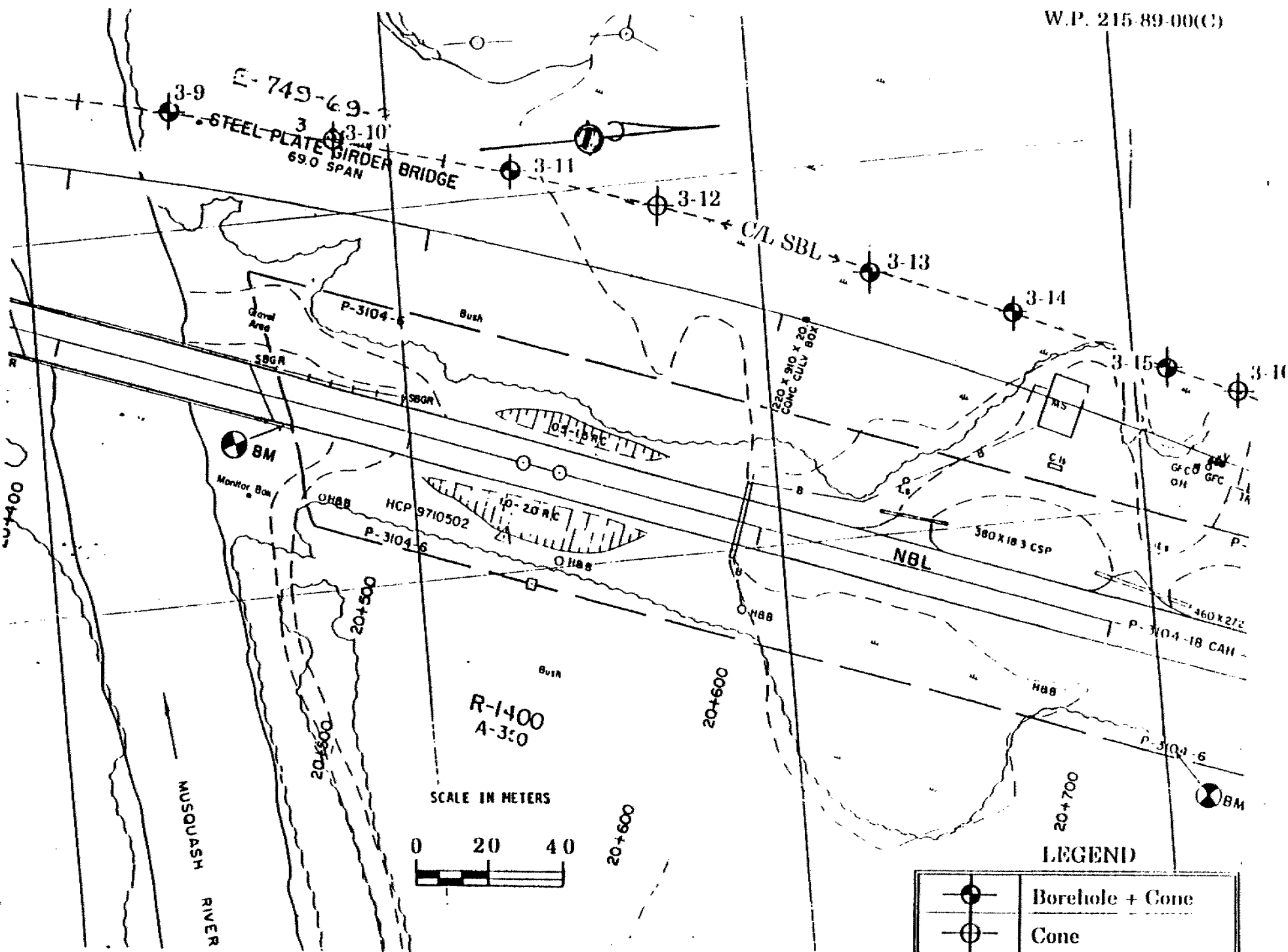
### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$		COMPRESSION INDEX
$C_s$		SWELLING INDEX
$C_\alpha$		RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
$m$		DRAINAGE PATH
$T_v$		TIME FACTOR
$U$	%	DEGREE OF CONSOLIDATION
$\sigma'_{v0}$	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
$\tau_f$	kPa	REMOULDED SHEAR STRENGTH
$S_f$		SENSITIVITY = $\frac{c_u}{\tau_f}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	$e$	%	VOID RATIO	$e_{min}$	%	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	$n$	%	POROSITY	$I_D$		DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	$w$	%	WATER CONTENT	$D$	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$		UNIFORMITY COEFFICIENT
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	$h$	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	$q$	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	$v$	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$L$		LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	$i$		HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$		CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	$k$	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL				$i$	kN/m <sup>2</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						





	Borehole + Cone
	Cone



## METRIC

ORIGINATED BY JE

COMPILED BY CE

CHECKED BY 33

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					
197.7'	Ground Surface											
0.0'	Probable Bedrock											
	Probable Silty Clay to Clayey Silt					197						
195.0'												
2.7'	End of Cone Test Refusal - Probable Bedrock						50/8cm					

## 1 OF 1

METRIC

ORIGINATED BY JE

COMPILED BY H

CHECKED BY 22

60/15cm

# RECORD OF BOREHOLE No 3-3 OF 1 METRIC

W.P. 215-B5-00(C) LOCATION Sta. 22+150, C/A 15E ORIGINATED BY DP  
 DIST HWY 65 BOREHOLE TYPE Cone Test COMPILED BY EB  
 DATUM Geodetic DATE April 12, 1992 CHECKED BY CC

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE $q_{c(10)}$ 20 40 60 80 100	PLASTIC LIMIT NATURAL MOISTURE CONTENT WATER CONTENT (%) 20 40 60	UNIT WEIGHT 7 KN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER TYPE N° VALUES						
196.71	Ground Surface								
0.01	Probable Peat				196				
					194				
	Probable Silty Clay to Clayey Silt				192				
					190				
					188				
	Probable Sandy Silt to Sand				186				
					184				
181.51					182				
15.21	End of Cone Test Refusal - Probable Bedrock					120/25cm			

OF METRIC

ORIGINATED BY CR

COMPILED BY                     

CHECKED BY \_\_\_\_\_

[illegible]



OF 1 METRIC

W.P. 015-89-00(C) LOCATION Sta. 20-740 C/O SBL ORIGINATED BY DR  
DIST 11 HWY 69 BOREHOLE TYPE Cone Test COMPILED BY DR  
DATUM Geodetic DATE April 7, 1992 CHECKED BY DR

[illegible]



## 1 OF 1

## METRIC

**LOCATION**

[illegible]

ORIGINATED BY JE

DIST \_\_\_\_\_

HWY 69

BOREHOLE TYPE

William Green, Long Test:

COMPILED BY       

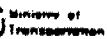
DATUM Geodetic

DATE \_\_\_\_\_

4557

CHECKED BY                      

0 50 29 11



• OF •

METRIC

W.P. 7-9-89-00(C)

LOCATION 1 4 585 548.4 7 322 847.5

ORIGINATED BY CE

DIST \_\_\_\_\_ HWY 66

BOREHOLE TYPE Core Test

COMPILED BY       E      

DATUM Geodetic

DATE April 10 1992

CHECKED BY                     

[illegible]

# RECORD OF BOREHOLE No 3-8

1 of 1

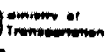
METRIC

W.P. 015-B5-0510 LOCATION Sta. 20+352, C.M. 2 + 11.00, S.E. ORIGINATED BY JE  
 DIST HWY 65 BOREHOLE TYPE Small Stem - Cone Test COMPILED BY JS  
 DATUM Geodetic DATE April 9, 1992 CHECKED BY CD

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER TYPE N° VALUES			20 40 60 80 100	20 40 60 80 100					
196.6	CE											
196.0	Silty Clay to Clayey Silt		1 SS 15		196							
	Trace of Sand		2 SS 1		194							
	Grey		3 SS 1									
	Soft to Firm		4 SS 10**		192						16.3	
189.6			5 TW 24		190							
			6 SS		188							
180.0	Trace of Clay L		7 SS 10**		186							0 26 68 6
	Sandy Silt to		8 SS 10**		184							
	Fine Sand, Some Silt		9 SS 4		182							0 91 6 3
	Gray to Brownish Gray		10 SS 10**		180							
			11 SS 10**									
	Loose to Compact		12 SS 13									
179.6			13 SS 7	/15cm								
178.8	Probable Bedrock ***											
17.8	End of Borehole					120	5cm					

\* W.L. on April 10, 1992  
 \*\* Spill spoon sank under weight of hammer and rods  
 \*\*\* Auger and Cone likely slipping off rock surface





W.P. 215-85-CO(C) LOCATION Site 21-424 2 1/2 S.B.L. ORIGINATED BY JS  
DIST 11 HWY 65 BOREHOLE TYPE Open Stem / Cone Test COMPILED BY JS  
DATUM Geodetic DATE April 6 1992 CHECKED BY JS

[illegible]

# RECORD OF BOREHOLE No 3-10 1 OF 1 METRIC

W.P. MS-89-0010 LOCATION Sta. 22+475 S.D. S.B. ORIGINATED BY JE  
 DIST. 4WY 69 BOREHOLE TYPE Cone Test COMPILED BY JE  
 DATUM Geodetic DATE April 7, 1992 CHECKED BY JE

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 VALUES								
186.7	Ground Surface										
0.0	Probable Fine Sand										
	Probable Silty Clay to Clayey Silt										
189.1	Probable Fine to Medium Sand										
7.6	End of Cone Test Refusal - Probable Bedrock					120/23cm					



## : OF :

W.P. 315-B9-GGIC LOCATION Sta. 10-530, 2" 1" - 1 1/2" S.B. ORIGINATED BY JB  
DIST 4WY 59 BOREHOLE TYPE new Stem - Cone Test COMPILED BY JB  
DATUM Geodetic DATE April 7, 1992 CHECKED BY JB

[illegible]



W.P. 015-89-0210 LOCATION Sta. 20-560; C/L S.B.L. ORIGINATED BY JS  
DIST 1 -WY 69 BOREHOLE TYPE Cone Test COMPILED BY JS  
DATUM Geodetic DATE April 7 1992 CHECKED BY JS

[illegible]

# RECORD OF BOREHOLE No 3-13 1 OF 1 METRIC

W.P. 2-5-89-00(C) LOCATION Sta. 20+632 ON 1 - R. 21.2 S.E.L. ORIGINATED BY JB/DP  
 DIST 1 HWY 55 BOREHOLE TYPE Single Stem Cone Test COMPILED BY JB  
 DATUM Geodetic DATE April 8, 1992 CHECKED BY JD

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 7 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT NUMBER	TYPE			20	40					
196.61	Ground Surface											
0.01	Peat, Black, Soft											
0.61		1	SS		196							
	Sandy	2	SS		194							
	Silty Clay to Clayey Silt	3	SS									
	Grey	4	SS		192							
	Firm to Stiff	5	SS									
188.81		6	SS		190							
7.81		7	SS		188							
	Sandy Silt to Fine Sand											
	Brownish Grey to Grey	8	SS		186							
	Loose to Compact	9	SS		184							
182.91												
13.7	End of Borehole											
	Probable Sandy Silt to Fine Sand											
175.31												
21.31	End of Cone Test Refusal - Probable Bedrock											
	* W.L. on April 14, 1992 ** Silt spoon sank under weight of hammer and rods											

# RECORD OF BOREHOLE No 3-14 1 OF 1 METRIC

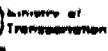
W.P. 215-89-00(C) LOCATION Sta 20+850 CV. S.B. ORIGINATED BY DR  
 DIST HWY 65 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JE  
 DATUM Geodetic DATE April 9 1992 CHECKED BY JD

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLO <sup>2</sup>					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			20	40	60	80	100					
196.6	Ground Surface															
0.0	Peat, Black															
0.8	Silty Clay to Clayey Silt Light Grey to Grey Firm to Stiff		1	SS		196										
			2	SS		194										
	Silt layer		3	SS		192										
			4	SS		190										
			5	SS		188										
	Sandy		6	SS		186										
187.9			7	SS		184										
8.7	Sandy Silt to Fine Sand, Trace Silt Brown to Brownish Grey Loose		8	SS		182										
			9	SS												
			10	SS												
180.8																
15.8	End of Borehole  Probable Sandy Silt to Fine Sand															
175.0																
21.6	End of Cone Test Refusal - Probable Bedrock  * W.L. on April 14, 1992 ** Split spoon sank under weight of hammer and rods											120	18cm			



W.P. 215-89-00(C) LOCATION Sta. 20-700 Off I-40 Rt. 201 S.B. ORIGINATED BY DR  
DIST. 1 HWY 69 BOREHOLE TYPE Howe Stem & Cone Test COMPILED BY JB  
DATUM Geodetic DATE April 10, 1992 CHECKED BY JD

[illegible]



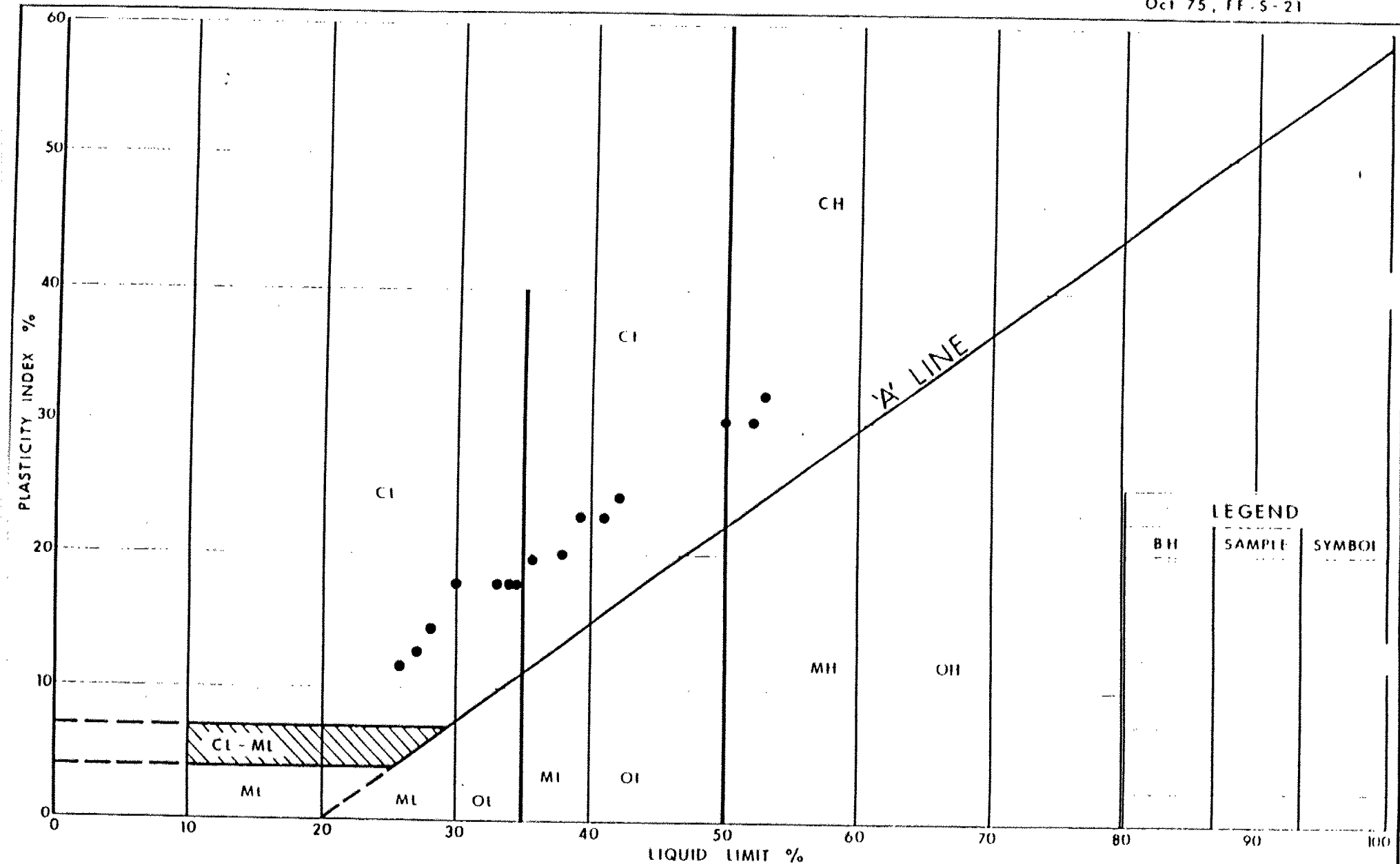
## 1 OF 1

METRIC

W.P. 215-89-CG101 LOCATION Sta 20-722, C.C. SBL ORIGINATED BY JS  
DIST 11 HWY 55 BOREHOLE TYPE Cone Test COMPILED BY JS  
DATUM Geodetic DATE April 14, 1992 CHECKED BY JS

[illegible]





Ontario

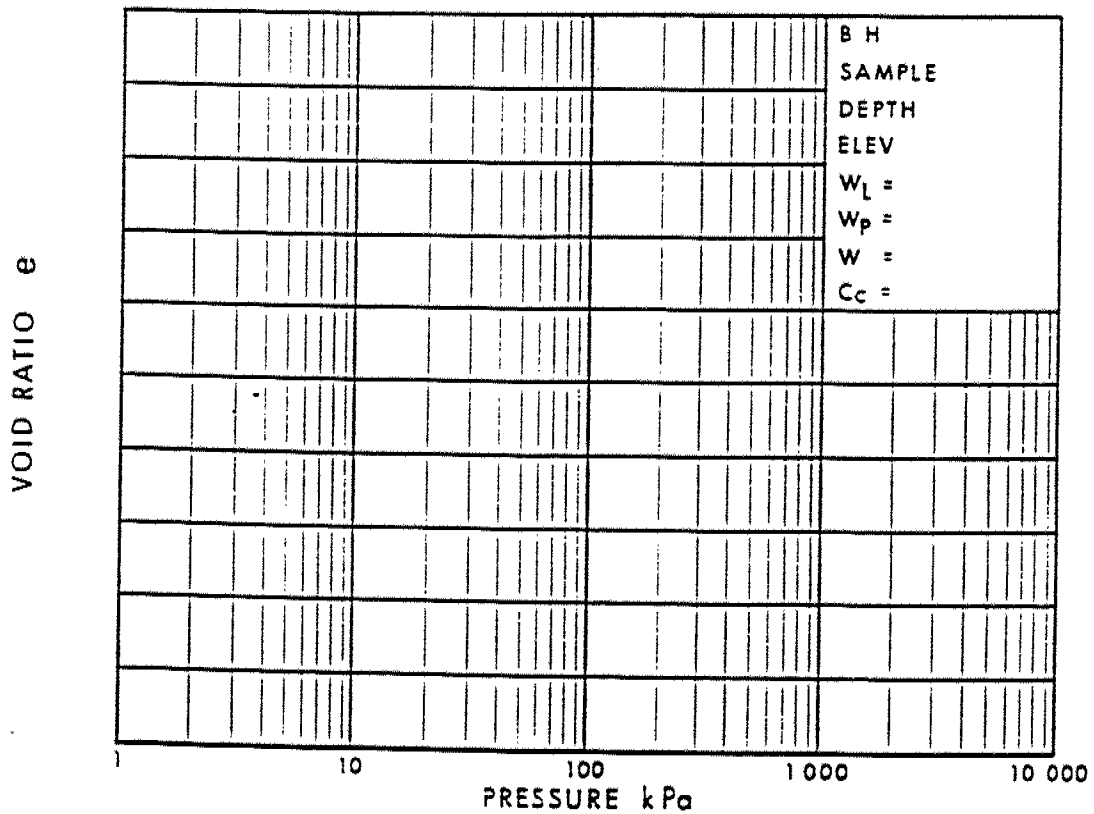
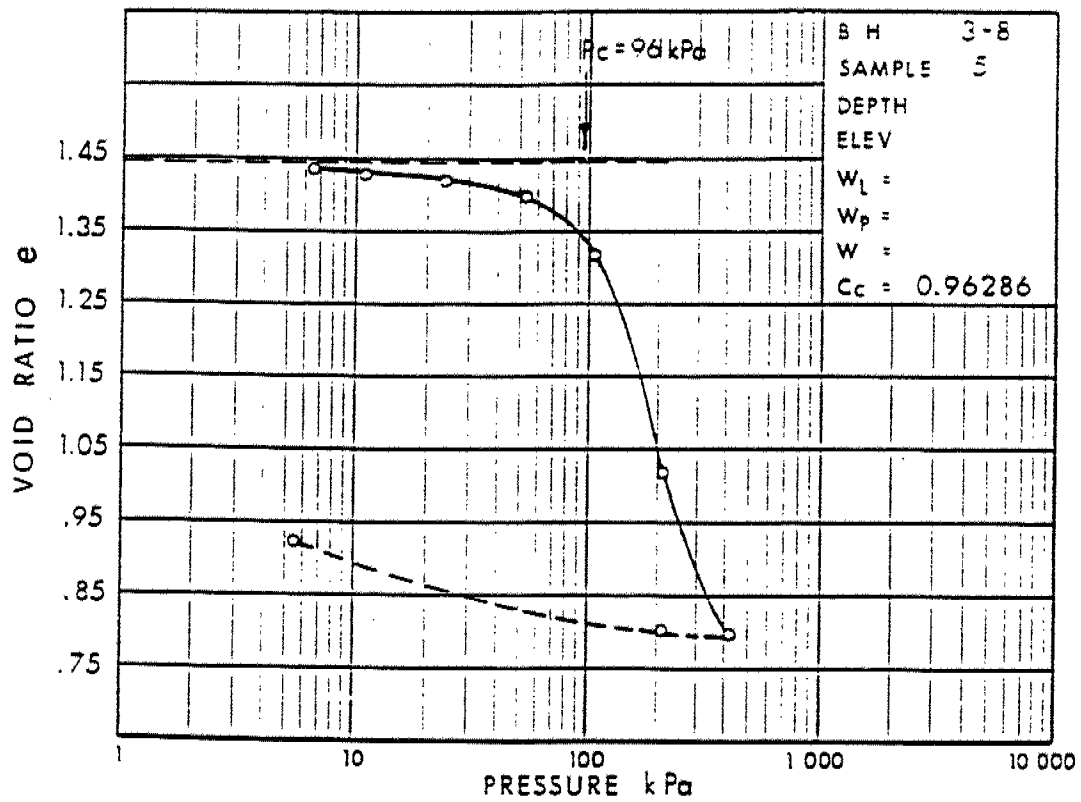
Ministry of  
Transportation

# PLASTICITY CHART SILTY CLAY TO CLAYEY SILT

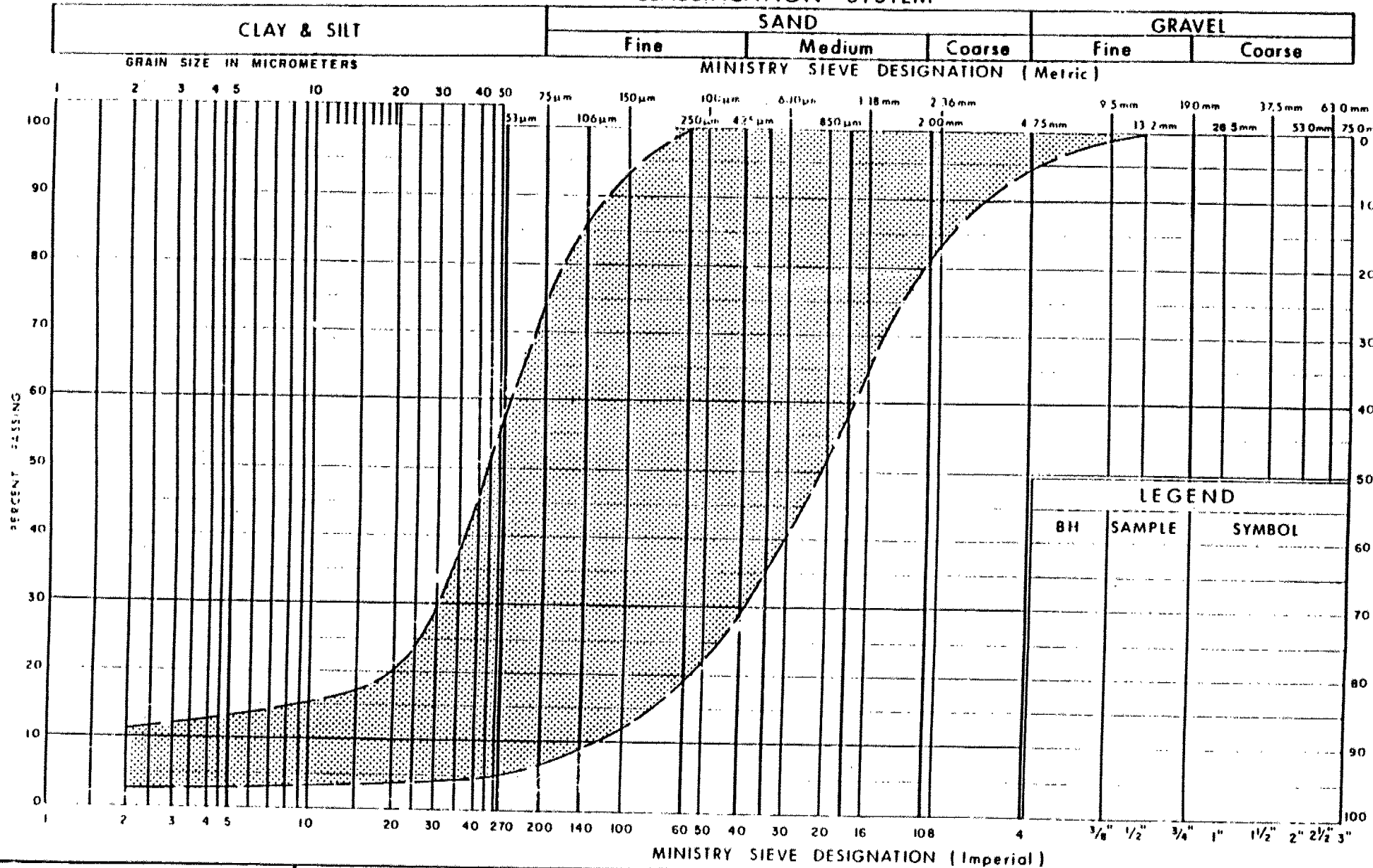
FIG No 1

W P 215-89-00 (C)

# VOID RATIO - PRESSURE CURVES



# UNIFIED SOIL CLASSIFICATION SYSTEM



( Ontario )

Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION  
SANDY SILT TO MEDIUM SAND

FIG No 3

W P 215 - 89 - 00 (C)

## Appendix E: Detailed Settlement Calculation Results

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**SB LANES**  
**2 Month Surcharge**

**Note:** The calculations presented in this Appendix E identify sections of Highway 69 which exceed the design settlement criteria of less than 25 mm after the application of the base pavement course, only. The settlements expected after surcharge and after wick drain/sand drain treatment are tabulated within the main body of this report for each highway section.

Station	Fill Thickness (m)	Clay Thickness (m)	Settlement (mm)	% Consolidation after 2 months	Remaining Settlement (mm)	Req'd Additional Fill (m)
20+100	4.7	3.3	140	0.48	73	1.5
20+150	4.0	5.8	120	0.27	88	1.9
20+200	3.6	4.2	100	0.38	62	1.6
20+250	3.2	7.0	110	0.23	85	2.3
20+300	3.1	7.5	110	0.21	87	2.6
20+350	3.1	4.5	80	0.35	52	1.4
20+450	3.3	9.2	130	0.16	109	4.1
20+500	3.8	6.5	130	0.24	99	2.6
20+550	4.2	7.0	140	0.23	108	2.3
20+600	4.5	7.5	220	0.21	174	4.9
20+650	4.7	8.1	250	0.19	203	6.3
20+700	4.1	4.0	100	0.42	58	0.5
20+750	5.5	3.0	180	0.52	86	1.8

**SB LANES**  
**4 Month Surcharge**

Station	Fill	Clay	Settlement	% Consolidation	Remaining	Req'd
	Thickness	Thickness			Settlement	Additional Fill
	(m)	(m)	(mm)	after 4 months	(mm)	(m)
20+100	4.7	3.3	140	0.66	48	0.4
20+150	4.0	5.8	120	0.38	74	1.0
20+200	3.6	4.2	100	0.54	46	0.9
20+250	3.2	7.0	110	0.31	76	1.7
20+300	3.1	7.5	110	0.29	78	1.9
20+350	3.1	4.5	80	0.49	41	0.9
20+450	3.3	9.2	130	0.25	98	2.4
20+500	3.8	6.5	130	0.34	86	1.6
20+550	4.2	7.0	140	0.31	97	1.4
20+600	4.5	7.5	220	0.29	156	3.4
20+650	4.7	8.1	250	0.27	183	4.0
20+700	4.1	4.0	100	0.56	44	-
20+750	5.5	3.0	180	0.72	50	0.5

**SB LANES**  
**6 Month Surcharge**

Station	Fill	Clay	Settlement	% Consolidation	Remaining	Req'd
	Thickness	Thickness			Settlement	Additional Fill
	(m)	(m)	(mm)	after 6 months	(mm)	(m)
20+100	4.7	3.3	140	0.78	31	-
20+150	4.0	5.8	120	0.47	64	0.6
20+200	3.6	4.2	100	0.64	36	0.6
20+250	3.2	7.0	110	0.38	68	1.4
20+300	3.1	7.5	110	0.38	68	1.5
20+350	3.1	4.5	80	0.60	32	0.4
20+450	3.3	9.2	130	0.29	92	2.1
20+500	3.8	6.5	130	0.43	74	1.1
20+550	4.2	7.0	140	0.38	87	0.9
20+600	4.5	7.5	220	0.38	136	2.2
20+650	4.7	8.1	250	0.34	165	2.8
20+700	4.1	4.0	100	0.66	34	-
20+750	5.5	3.0	180	0.84	29	-

**NB LANES**  
**2 Month Surcharge**

Station	Fill	Clay	Settlement	% Consolidation	Remaining	Req'd
	Thickness	Thickness			Settlement	Additional Fill
	(m)	(m)	(mm)	after 2 months	(mm)	(m)
20+273	0.1	7.2	0	0.22	0	-
20+298	3.3	9.0	120	0.16	101	3.7
20+324	2.9	5.0	85	0.31	59	1.8
20+350	3.3	7.8	115	0.19	93	2.8
20+376	3.8	7.4	102	0.22	80	1.7
20+451	2.9	6.9	100	0.22	78	2.6
20+475	2.9	3.0	61	0.52	29	-
20+499	3.5	4.1	90	0.39	55	1.2
20+577	2.6	3.8	63	0.42	37	0.9
20+593	4.0	4.8	98	0.33	66	1.0
20+621	4.5	7.0	162	0.22	126	3.0
20+655	3.4	13.0	190	0.12	167	5.6
20+674	3.1	10.0	26	0.16	22	-
20+701	3.1	6.8	100	0.22	78	2.5
20+725	3.8	3.9	80	0.39	49	0.7
20+762	3.6	1.5	40	0.90	4	-

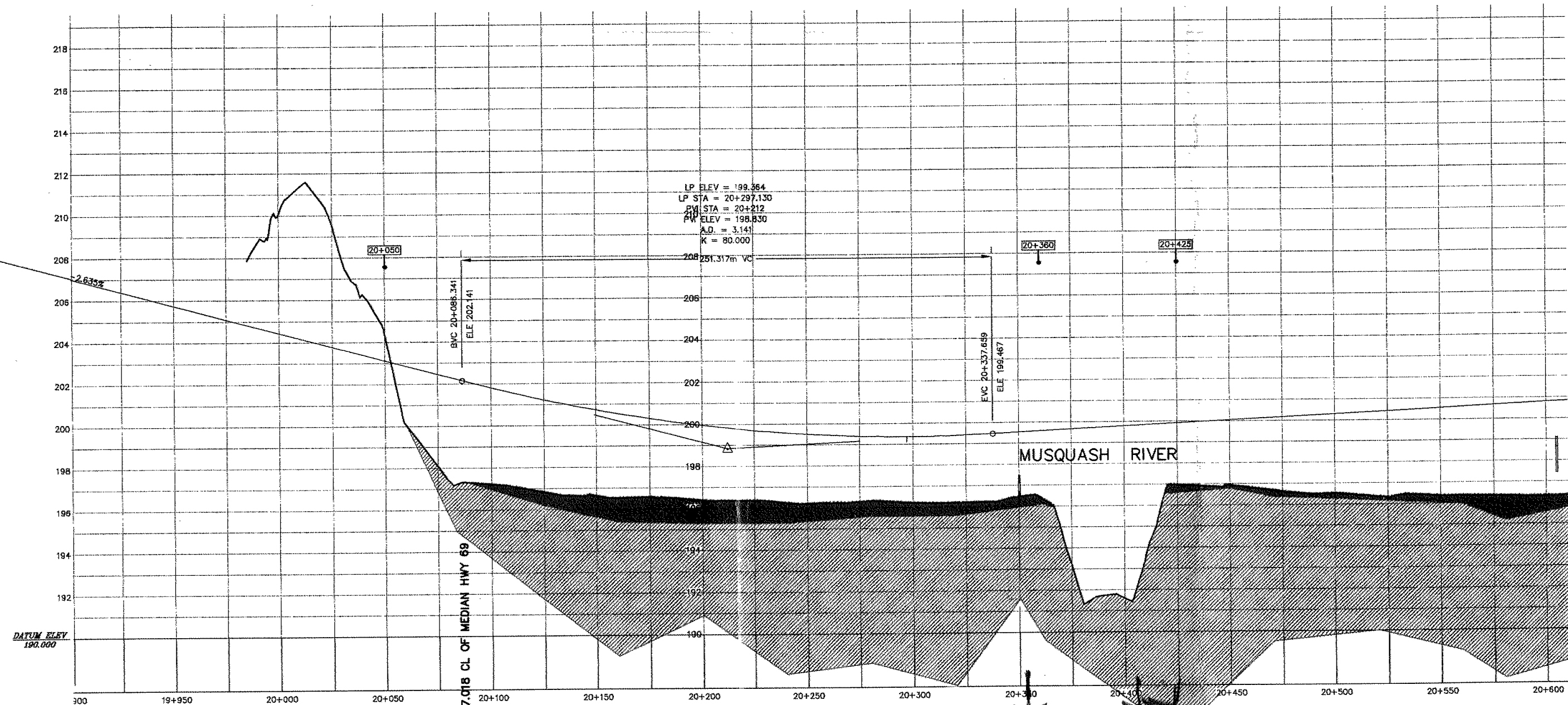


**NB LANES**  
**4 Month Surcharge**

Station	Fill	Clay	Settlement	% Consolidation	Remaining	Req'd
	Thickness	Thickness			Settlement	Additional Fill
	(m)	(m)	(mm)	after 4 months	(mm)	(m)
20+273	0.1	7.2	0	0.31	0	-
20+298	3.3	9.0	120	0.25	90	2.2
20+324	2.9	5.0	85	0.46	46	1.2
20+350	3.3	7.8	115	0.29	82	2.1
20+376	3.8	7.4	102	0.29	72	1.2
20+451	2.9	6.9	100	0.31	69	1.8
20+475	2.9	3.0	61	0.73	16	-
20+499	3.5	4.1	90	0.56	40	0.6
20+577	2.6	3.8	63	0.58	26	-
20+593	4.0	4.8	98	0.48	51	0.4
20+621	4.5	7.0	162	0.31	112	1.9
20+655	3.4	13.0	190	0.16	160	4.3
20+674	3.1	10.0	26	0.22	20	-
20+701	3.1	6.8	100	0.33	67	1.6
20+725	3.8	3.9	80	0.57	34	-
20+762	3.6	1.5	40	0.98	1	-

**NB LANES**  
**6 Month Surcharge**

Station	Fill	Clay	Settlement	% Consolidation	Remaining	Req'd
	Thickness	Thickness			Settlement	Additional Fill
	(m)	(m)	(mm)	after 6 months	(mm)	(m)
20+273	0.1	7.2	0	0.38	0	-
20+298	3.3	9.0	120	0.29	85	1.9
20+324	2.9	5.0	85	0.55	38	0.9
20+350	3.3	7.8	115	0.31	79	1.7
20+376	3.8	7.4	102	0.37	64	0.8
20+451	2.9	6.9	100	0.38	62	1.6
20+475	2.9	3.0	61	0.83	10	-
20+499	3.5	4.1	90	0.67	30	-
20+577	2.6	3.8	63	0.71	18	-
20+593	4.0	4.8	98	0.57	42	0.1
20+621	4.5	7.0	162	0.37	102	1.2
20+655	3.4	13.0	190	0.22	148	3.6
20+674	3.1	10.0	26	0.27	19	-
20+701	3.1	6.8	100	0.41	59	1.2
20+725	3.8	3.9	80	0.68	26	-
20+762	3.6	1.5	40	1.00	0	-



- only reserve pool  
 - use wide drain or sand drain  
 including 0.8m granular  
 light weight fill  
 wide drain

#### LEGEND:



CLAYEY SOILS



SURFICIAL ORGANIC SOILS



DELIMITATION OF AREA OF INTEREST

#### NOTES:

FOR DETAILED BOREHOLE LOGS REFER TO:

1. APPENDIX 'C' AND 'D'
2. PAVEMENT DESIGN REPORT VOLUME 2 (WP-217-89-00) FOR TROW CONSULTING ENGINEERS LTD., JANUARY, 1998.
3. BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.



**TROW CONSULTING ENGINEERS LTD.**

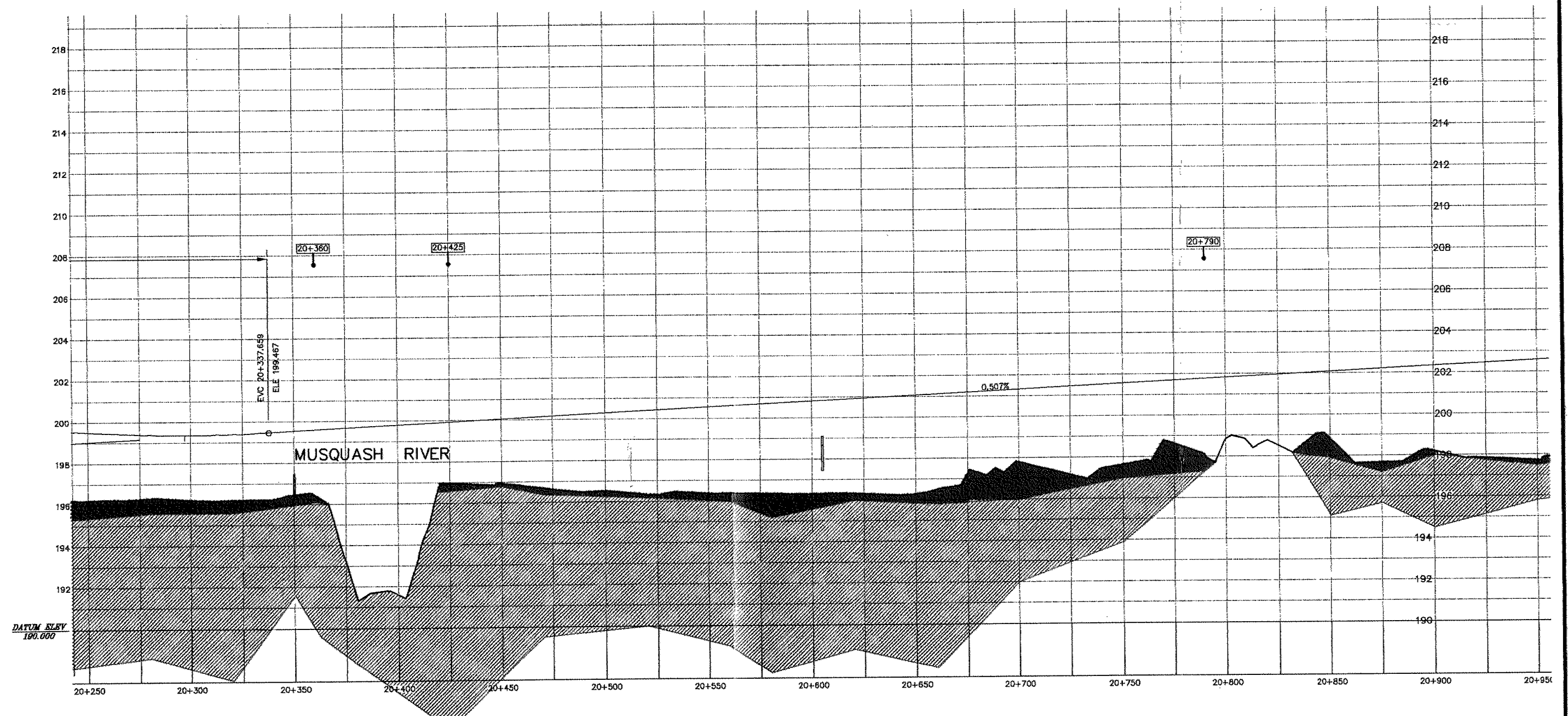
BRAMPTON, ONTARIO

**HIGHWAY #69**  
**SOUTHBOUND LANE**  
**STATION 20+050 to 20+360**

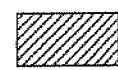
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 SCALE: AS NOTED  
 DRAWN BY: SDH  
 CHECKED BY: SDH  
 DATE: JUNE 1998  
 DRAWING NO.: 1

XX

ONTARIO



#### LEGEND:



CLAYEY SOILS



SURFICIAL ORGANIC SOILS



DELIMITATION OF AREA OF INTEREST

#### NOTES:

FOR DETAILED BOREHOLE LOGS REFER TO:

1. APPENDIX 'C' AND 'D'
2. PAVEMENT DESIGN REPORT VOLUME 2 (WP-217-89-00) FOR TROW CONSULTING ENGINEERS LTD., JANUARY, 1998.
3. BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.



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

**HIGHWAY #69  
SOUTHBOUND LANE  
STATION 20+425 to 20+790**

PROJECT NO.: BRGE0011546C

SCALE: AS NOTED

DRAWN BY: SDH

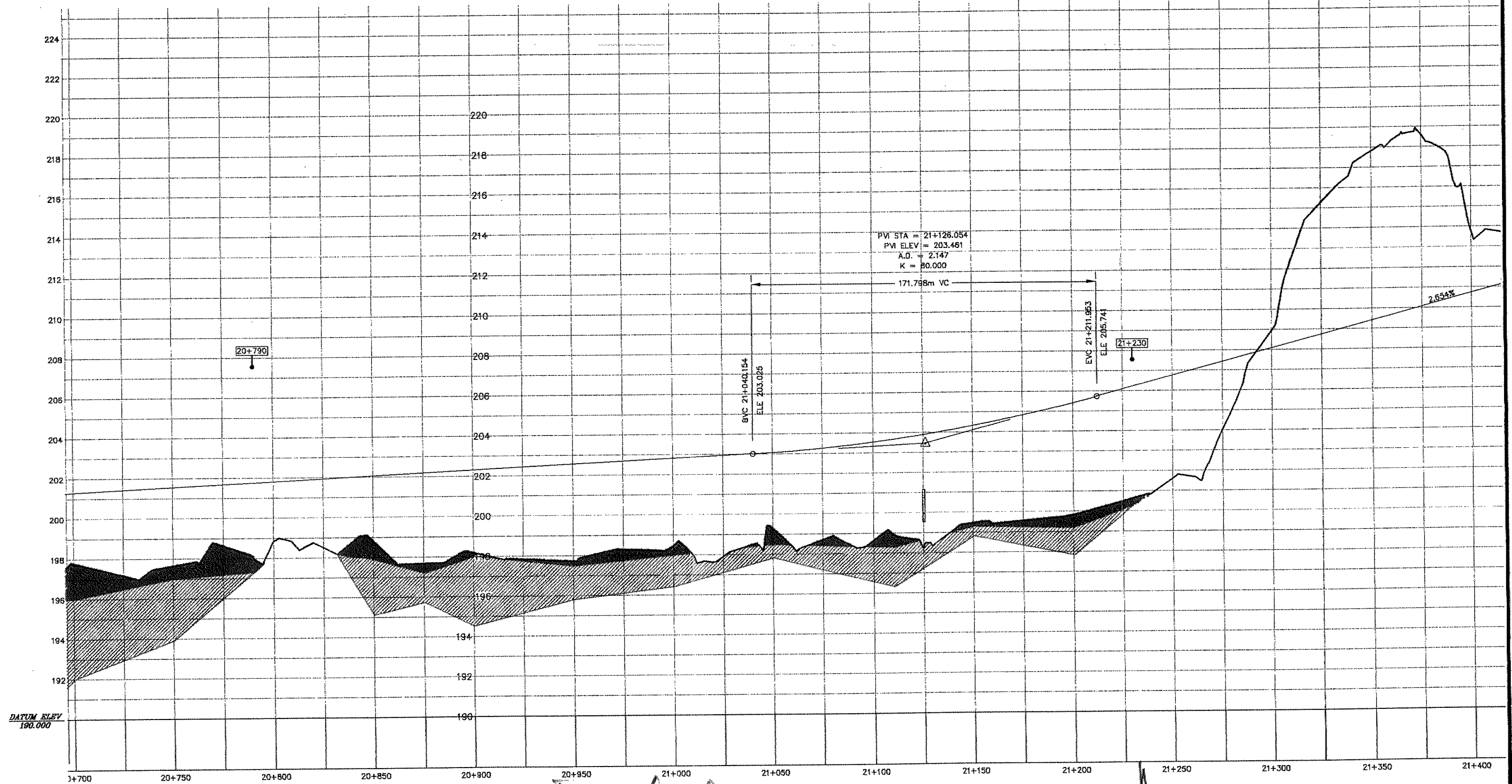
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DATE: JUNE 1998

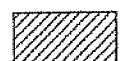
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XX

ONTARIO



# LEGEND:



CLAYEY SOILS



SURFICIAL ORGANIC SOILS

26+560

DELIMITATION OF AREA OF INTEREST

## NOTES:

FOR DETAILED BOREHOLE LOGS REFER TO:

1. APPENDIX 'C' AND 'D'
2. PAVEMENT DESIGN REPORT VOLUME 2 (WP-217-89-00) FOR TROW CONSULTING ENGINEERS LTD., JANUARY, 1998.
3. BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.



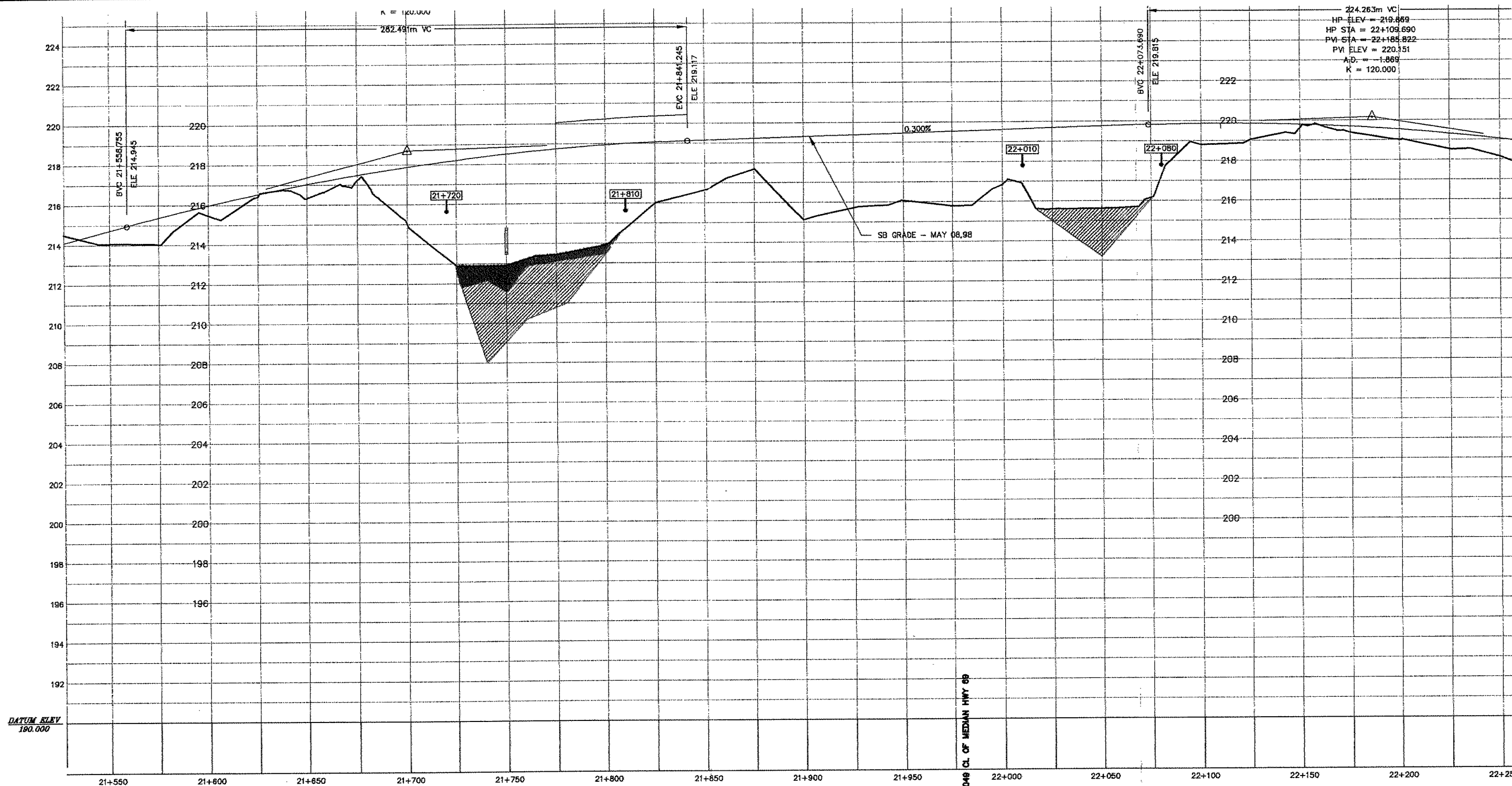
**TROW CONSULTING ENGINEERS LTD.**  
BRAMPTON, ONTARIO

**HIGHWAY #69  
SOUTHBOUND LANE  
STATION 20+790 to 21+230**

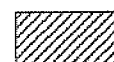
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SCALE: AS NOTED  
DRAWN BY: SDH  
CHECKED BY: SDH  
DATE: JUNE 1998  
DRAWING NO.: 3

XX

ONTARIO



**LEGEND:**



CLAYEY SOILS



SURFICIAL ORGANIC SOILS

21+560

DELIMITATION OF AREA OF INTEREST

**NOTES:**

FOR DETAILED BOREHOLE LOGS REFER TO:

1. APPENDIX 'C' AND 'D'
2. PAVEMENT DESIGN REPORT VOLUME 2 (WP-217-89-00) FOR TROW CONSULTING ENGINEERS LTD., JANUARY, 1998.
3. BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.

*excavated  
entirely*



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

**HIGHWAY #69  
SOUTHBOUND LANE  
STATIONS 21+720 to 21+810  
22+010 to 22+080**

PROJECT NO.: BRGE0011548C

SCALE: AS NOTED

DRAWN BY: SDH

CHECKED BY: SDH

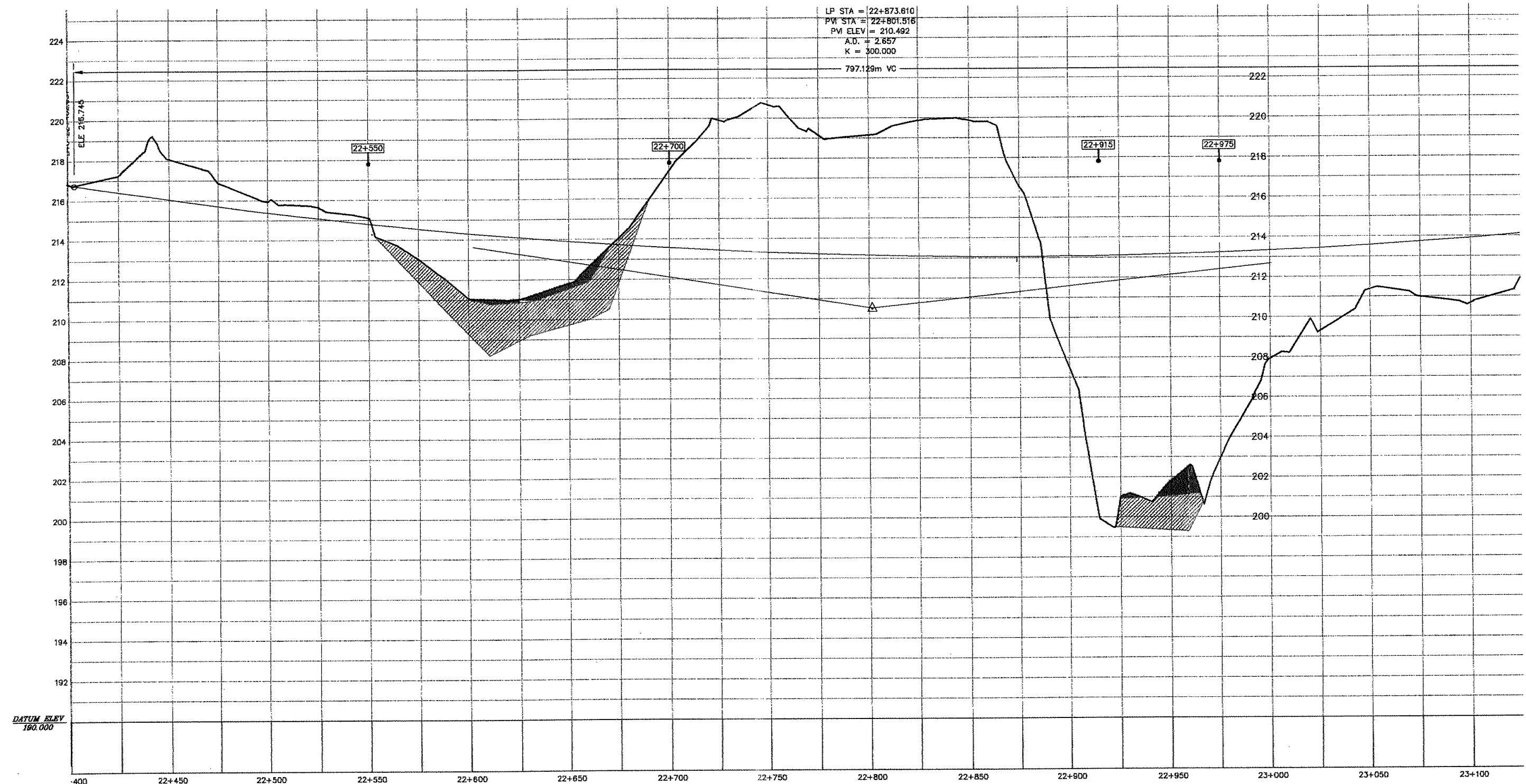
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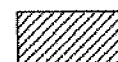
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ONTARIO





# LEGEND:



CLAYEY SOILS



SURFICIAL ORGANIC SOILS

22+550

DELIMITATION OF AREA OF INTEREST

## NOTES:

FOR DETAILED BOREHOLE LOGS REFER TO:

1. APPENDIX 'C' AND 'D'
2. PAVEMENT DESIGN REPORT VOLUME 2 (WP-217-89-00) FOR TROW CONSULTING ENGINEERS LTD., JANUARY, 1998.
3. BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.

*excavate entirely*

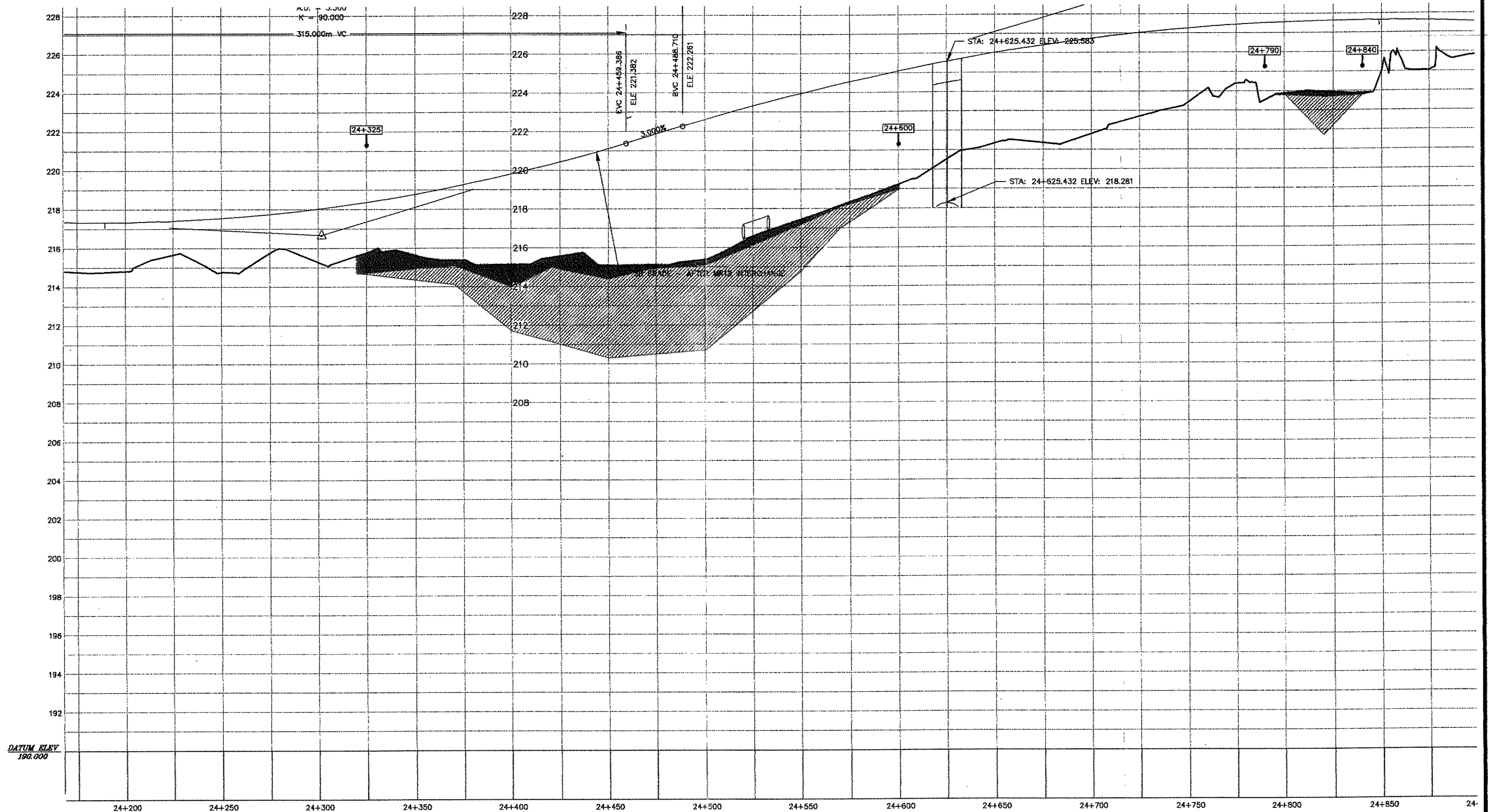


**TROW CONSULTING ENGINEERS LTD.**  
 BRAMPTON, ONTARIO

**HIGHWAY #69**  
**SOUTHBOUND LANE**  
**STATION 22+550 to 22+700**  
**22+915 to 22+975**  
**ONTARIO**

PROJECT NO.: BRGE0011546C  
 SCALE: AS NOTED  
 DRAWN BY: SDH  
 CHECKED BY: SDH  
 DATE: JUNE 1998  
 DRAWING NO.: 5

XX



# LEGEND:

CLAYEY SOILS

SURFICIAL ORGANIC SOILS

DELIMITATION OF AREA OF INTEREST

## NOTES

*full excavation with care*

FOR DETAILED BOREHOLE LOGS REFER TO:

- APPENDIX 'C' AND 'D'
- PAVEMENT DESIGN REPORT VOLUME 2 (WP-217-89-00) FOR TROW CONSULTING ENGINEERS LTD., JANUARY, 1998.
- BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.



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

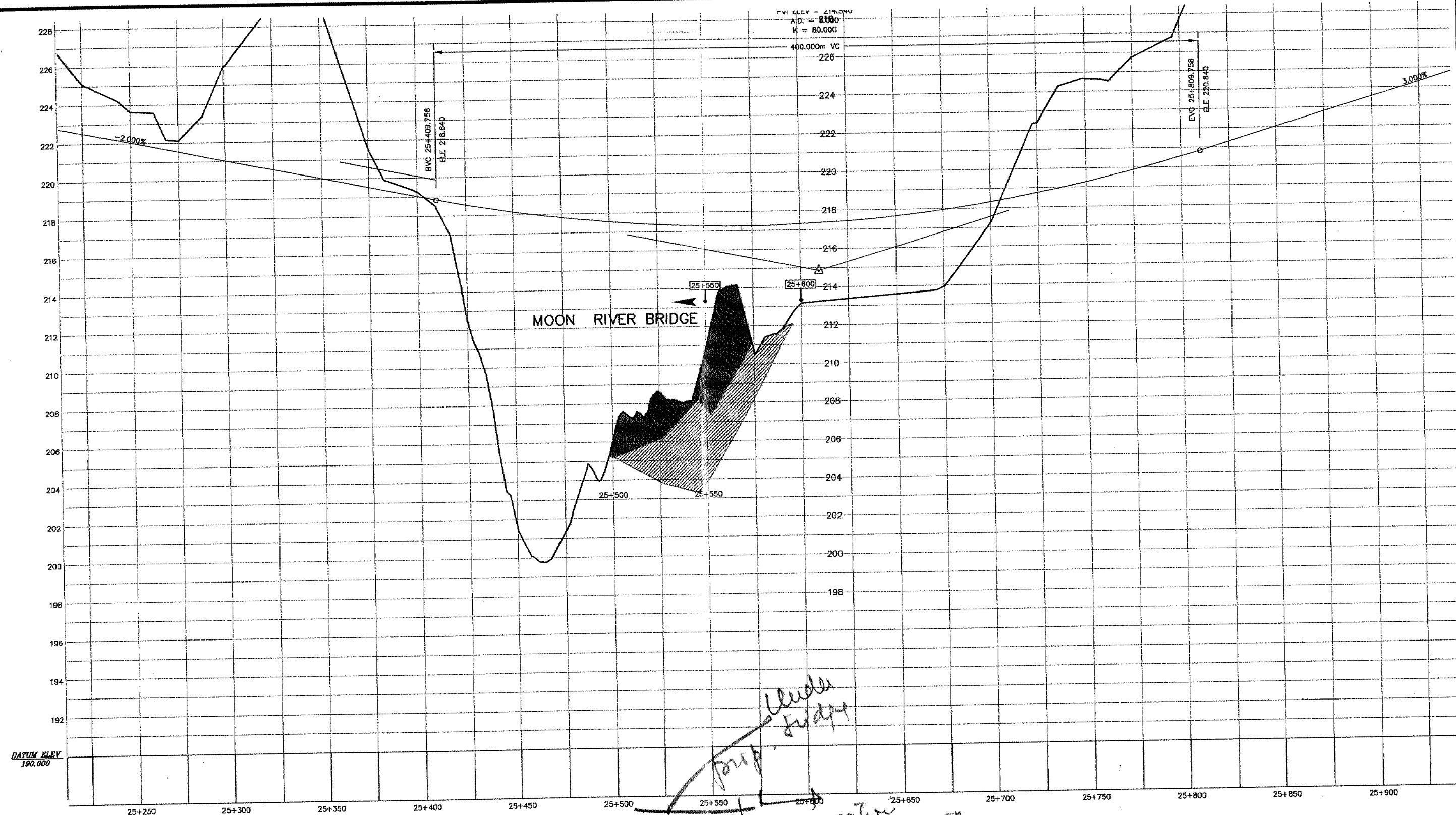
**HIGHWAY #69**  
**SOUTHBOUND LANE**  
 STATION 24+325 to 24+600  
 24+790 to 24+840

PROJECT NO.: BRGE0011546C  
 SCALE: AS NOTED  
 DRAWN BY: SDH  
 CHECKED BY: SDH  
 DATE: JUNE 1998  
 DRAWING NO.: 6

XX

ONTARIO





# **LEGEND:**



CLAYEY SOILS



SURFICIAL ORGANIC SOILS

25+560

DELIMITATION OF AREA OF INTEREST

## **NOTES:**

- FOR DETAILED BOREHOLE LOGS REFER TO:
1. APPENDIX 'C' AND 'D'
  2. PAVEMENT DESIGN REPORT VOLUME 2 (WP-217-89-00) FOR TROW CONSULTING ENGINEERS LTD., JANUARY, 1998.
  3. FOUNDATION INVESTIGATION REPORT, MOON RIVER, SOUTHBOUND LANES, TROW CONSULTING ENGINEERS LTD., OCTOBER, 1998.
  4. BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.
- Handwritten notes: "full excavation" and "25+550"



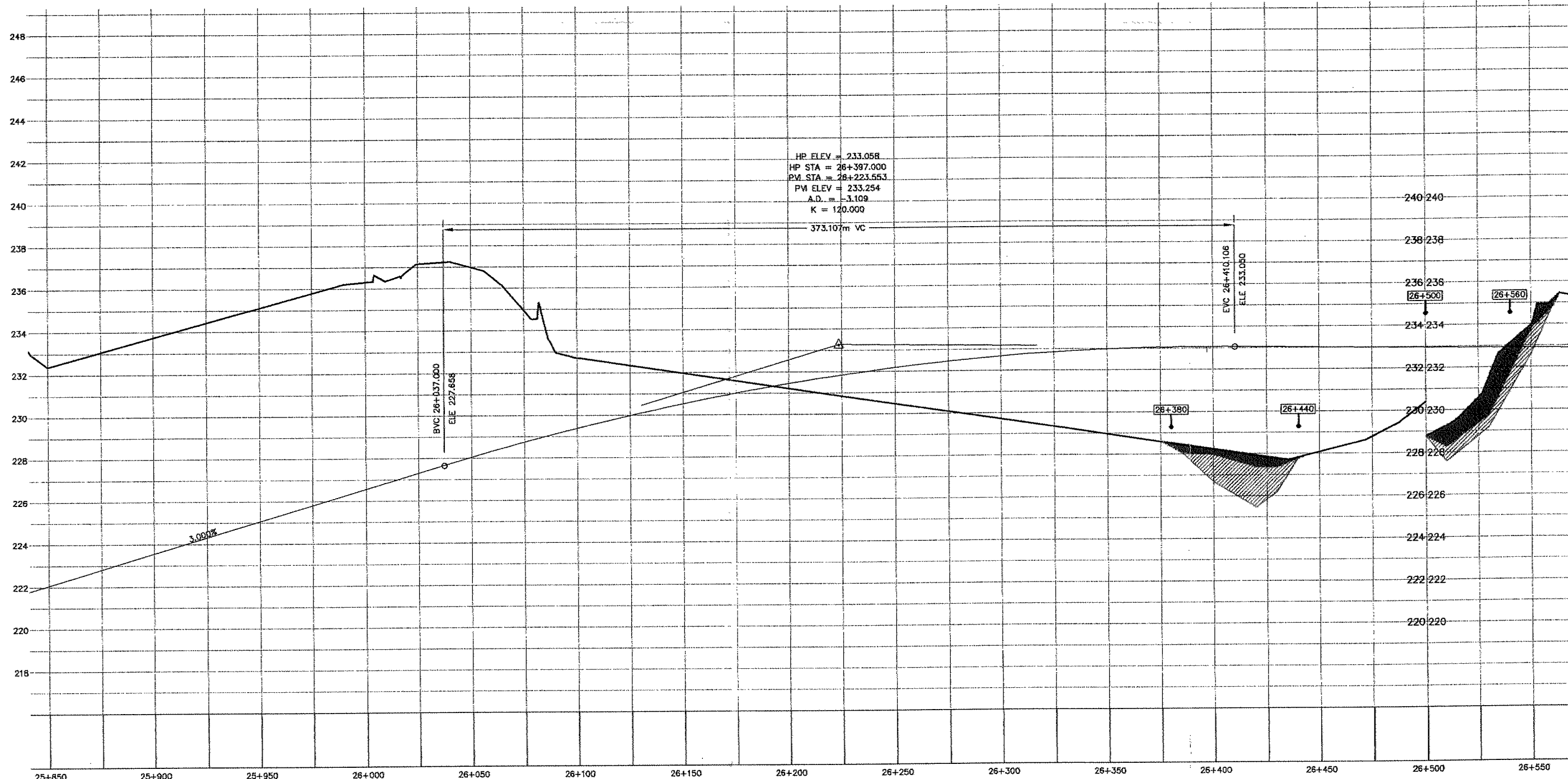
**TROW CONSULTING ENGINEERS LTD.**  
BRAMPTON, ONTARIO

**HIGHWAY #69  
SOUTHBOUND LANE  
STATION 25+550 to 25+800**




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SCALE: AS NOTED  
DRAWN BY: SDH  
CHECKED BY: SDH  
DATE: JUNE 1998  
DRAWING NO.: 7

XX

ONTARIO



**LEGEND:**


-  CLAYEY SOILS
-  SURFICIAL ORGANIC SOILS
-  DELIMITATION OF AREA OF INTEREST

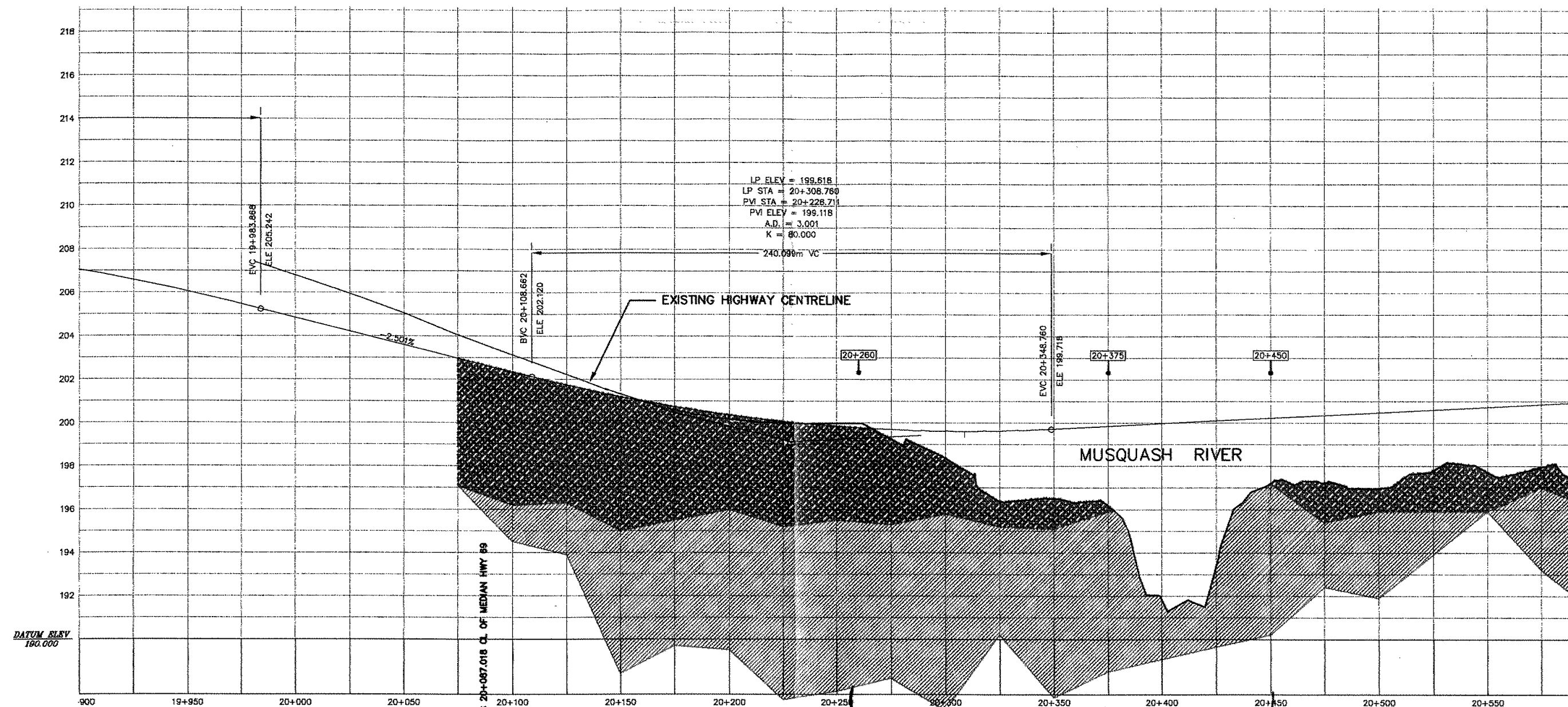
**NOTES:**

- FOR DETAILED BOREHOLE LOGS REFER TO:
1. APPENDIX 'C' AND 'D'
  2. PAVEMENT DESIGN REPORT VOLUME 2 (WP-217-89-00) FOR TROW CONSULTING ENGINEERS LTD., JANUARY, 1998.
  3. BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.

*fullerc*

*fullerc*

 <b>TROW CONSULTING ENGINEERS LTD.</b> BRAMPTON, ONTARIO		PROJECT NO.: BRGE0011546C	
		SCALE: AS NOTED	
<b>HIGHWAY #69</b> <b>SOUTHBOUND LANE</b> <b>STATION 26+380 to 26+440</b> <b>26+500 to 26+560</b>		DRAWN BY: SDH	
		CHECKED BY: SDH	
		DATE: JUNE 1998	
XX	ONTARIO	DRAWING NO.: 8	



# **LEGEND:**



CLAYEY SOILS



SURFICIAL ORGANIC SOILS

26+560

DELIMITATION OF AREA OF INTEREST

## **NOTES:**

FOR DETAILED BOREHOLE LOGS REFER TO:

1. APPENDIX 'B'
2. PAVEMENT DESIGN REPORT VOLUME 2 (WP-217-89-00) FOR TROW CONSULTING ENGINEERS LTD., JANUARY, 1998.
3. FOUNDATION INVESTIGATION REPORT, MUSQUASH RIVER, NORTHBOUND LANES, TROW CONSULTING ENGINEERS LTD., OCTOBER, 1998.
4. BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.



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

**HIGHWAY #69  
NORTHBOUND LANE  
STATION 20+260 to 20+375**

PROJECT NO.: BRGE0011546C

SCALE: AS NOTED

DRAWN BY: SDH

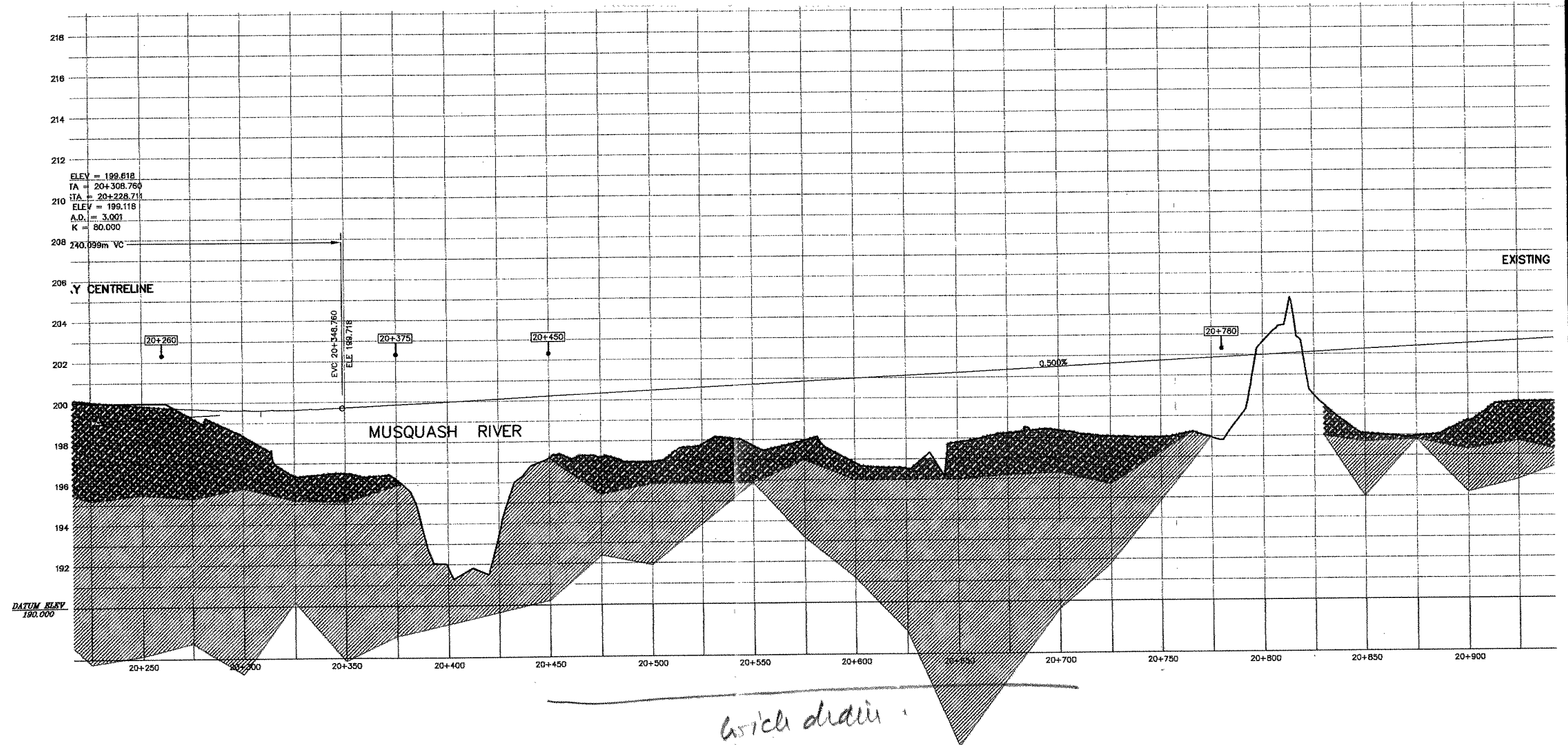
CHECKED BY: SDH

DATE: JUNE 1998

DRAWING NO.: 9

XX

ONTARIO



# LEGEND:



CLAYEY SOILS



SURFICIAL ORGANIC SOILS

26+560

DELIMITATION OF AREA OF INTEREST

## NOTES:

FOR DETAILED BOREHOLE LOGS REFER TO:

1. APPENDIX 'B'
2. PAVEMENT DESIGN REPORT VOLUME 2 (WP-217-89-00) FOR TROW CONSULTING ENGINEERS LTD., JANUARY, 1998.
3. FOUNDATION INVESTIGATION REPORT, MUSQUASH RIVER, NORTHBOUND LANES, TROW CONSULTING ENGINEERS LTD., OCTOBER, 1998.
4. BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.



**TROW CONSULTING ENGINEERS LTD.**

BRAMPTON, ONTARIO

**HIGHWAY #69  
 NORTHBOUND LANE  
 STATION 20+450 tp 20+780**

PROJECT NO.: BRGE0011548C

SCALE: AS NOTED

DRAWN BY: SDH

CHECKED BY: SDH

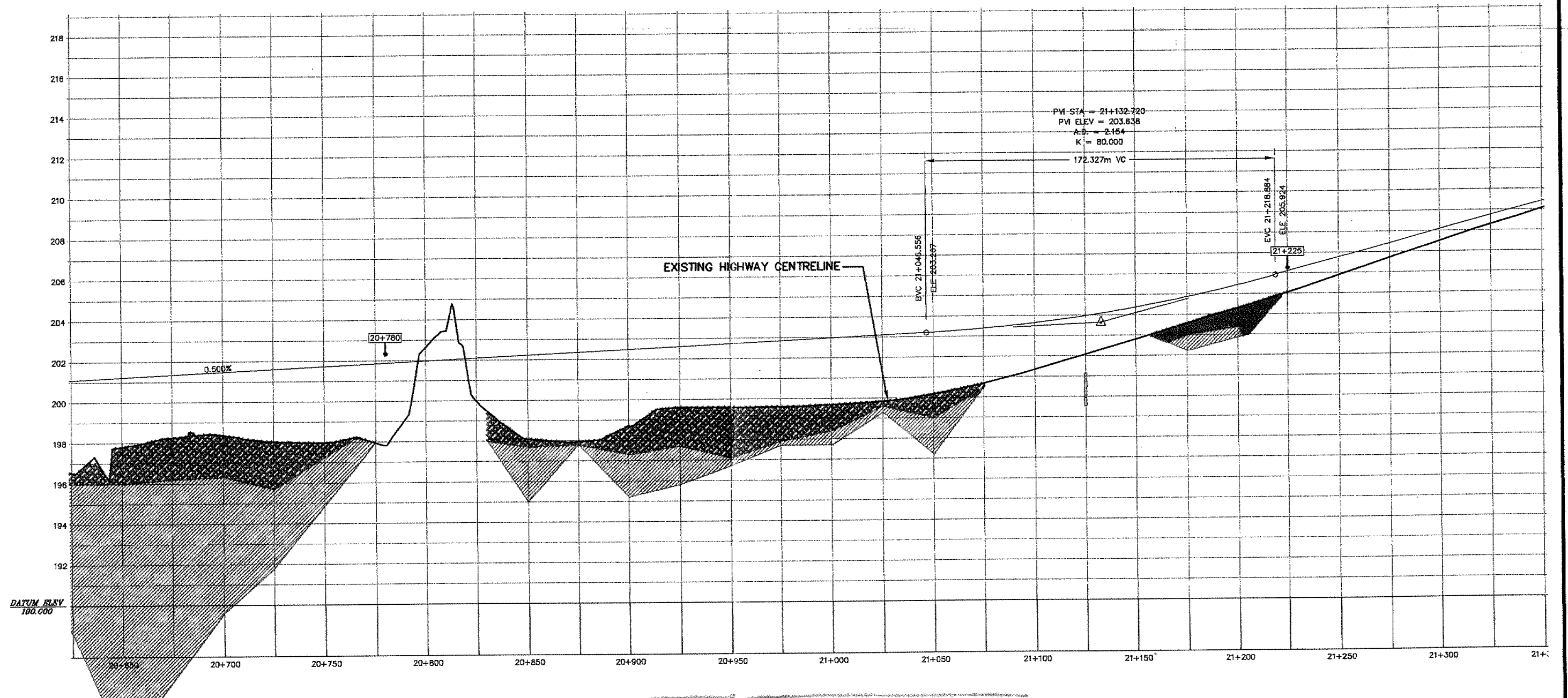
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DRAWING NO.: 10

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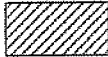


ONTARIO





*full excavation*

**LEGEND:**

-  CLAYEY SOILS
-  SURFICIAL ORGANIC SOILS
-  DELIMITATION OF AREA OF INTEREST

**NOTES:**

- FOR DETAILED BOREHOLE LOGS REFER TO:
1. APPENDIX 'B'
  2. PAVEMENT DESIGN REPORT VOLUME 2 (WP-217-89-00) FOR TROW CONSULTING ENGINEERS LTD., JANUARY, 1998.
  3. BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.



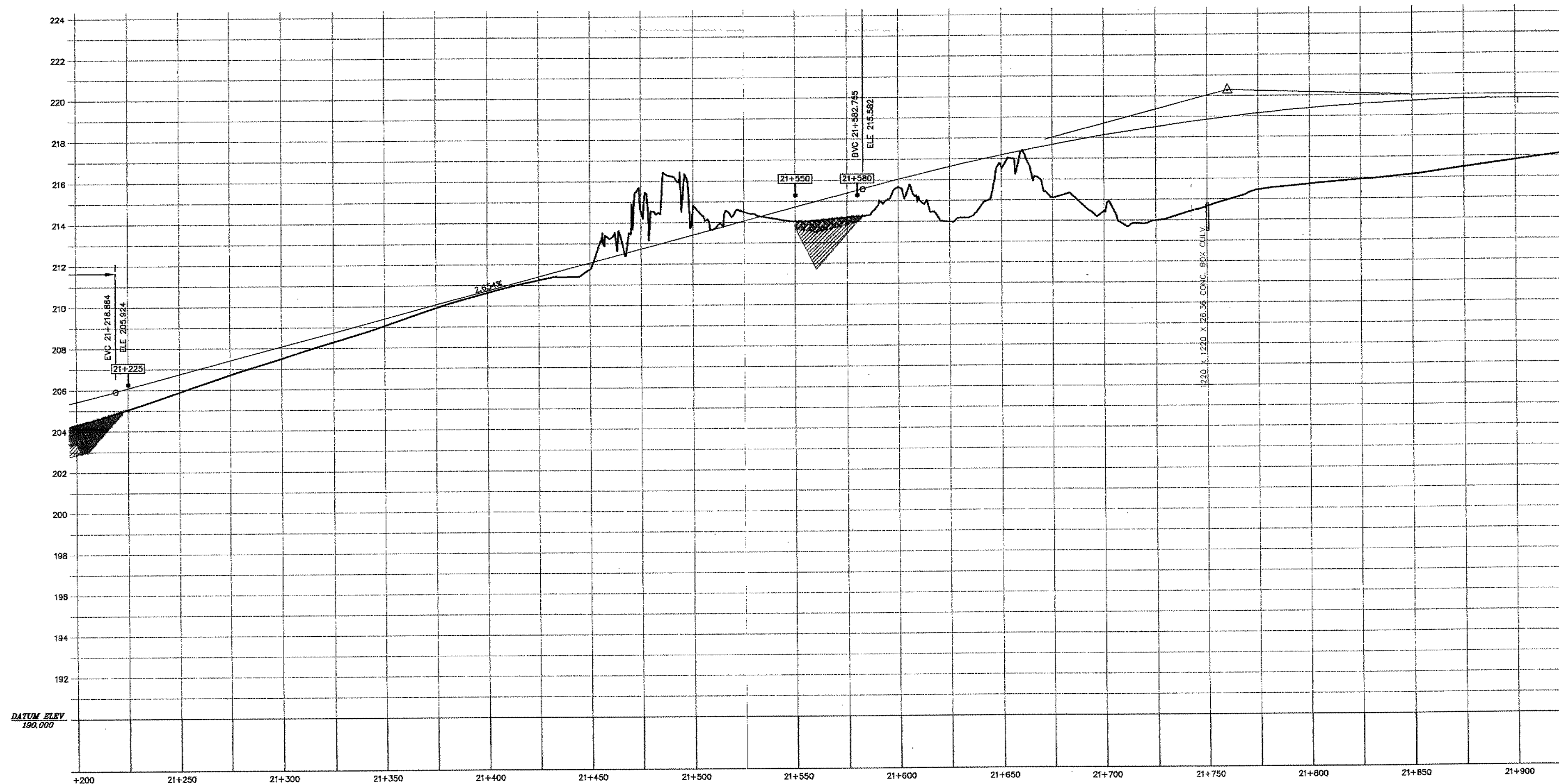
**TROW CONSULTING ENGINEERS LTD.**  
BRAMPTON, ONTARIO

**HIGHWAY #69  
NORTHBOUND LANE  
STATION 20+790 to 21+225**

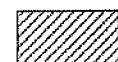
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SCALE:	AS NOTED
DRAWN BY:	SDH
CHECKED BY:	SDH
DATE:	JUNE 1998
DRAWING NO.:	11

XX

ONTARIO



# **LEGEND:**



CLAYEY SOILS



SURFICIAL ORGANIC SOILS

26+560

DELIMITATION OF AREA OF INTEREST

## **NOTES:**

FOR DETAILED BOREHOLE LOGS REFER TO:

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3. BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.



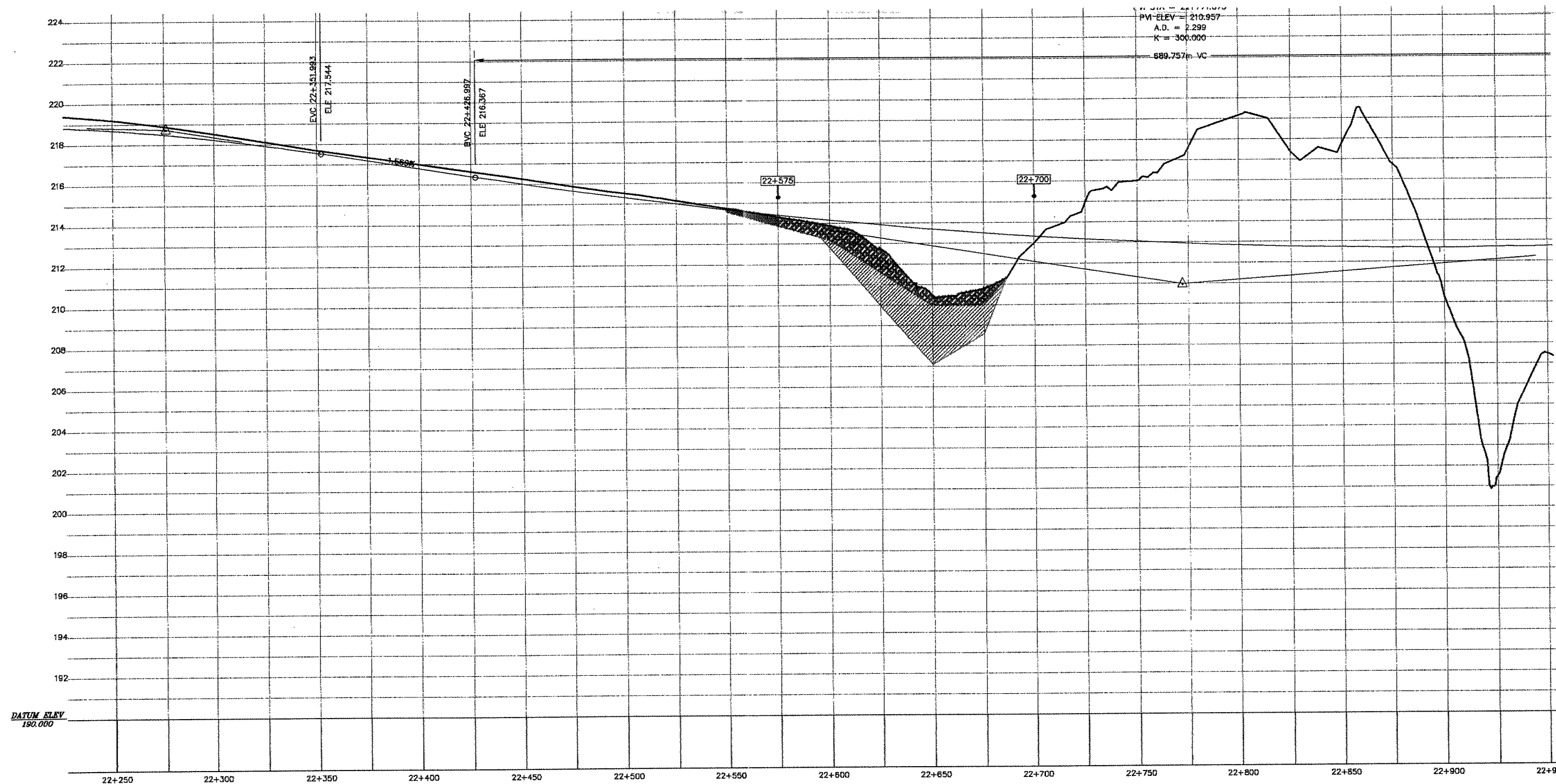
**TROW CONSULTING ENGINEERS LTD.**  
BRAMPTON, ONTARIO

**HIGHWAY #69  
NORTHBOUND LANE  
STATION 21+550 to 21+580**

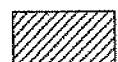
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SCALE: AS NOTED  
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CHECKED BY: SDH  
DATE: JUNE 1998  
DRAWING NO.: 12

XX

ONTARIO



# **LEGEND:**



CLAYEY SOILS



SURFICIAL ORGANIC SOILS

22+580

DELIMITATION OF AREA OF INTEREST

# **NOTES:**

- FOR DETAILED BOREHOLE LOGS REFER TO:
1. APPENDIX 'B'
  2. PAVEMENT DESIGN REPORT VOLUME 2 (WP-217-89-00) FOR TROW CONSULTING ENGINEERS LTD., JANUARY, 1998.
  3. BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.



**TROW CONSULTING ENGINEERS LTD.**

BRAMPTON, ONTARIO

**HIGHWAY #69  
NORTHBOUND LANE  
STATION 22+575 to 22+700**

PROJECT NO.: BRGE0011548C

SCALE: AS NOTED

DRAWN BY: SDH

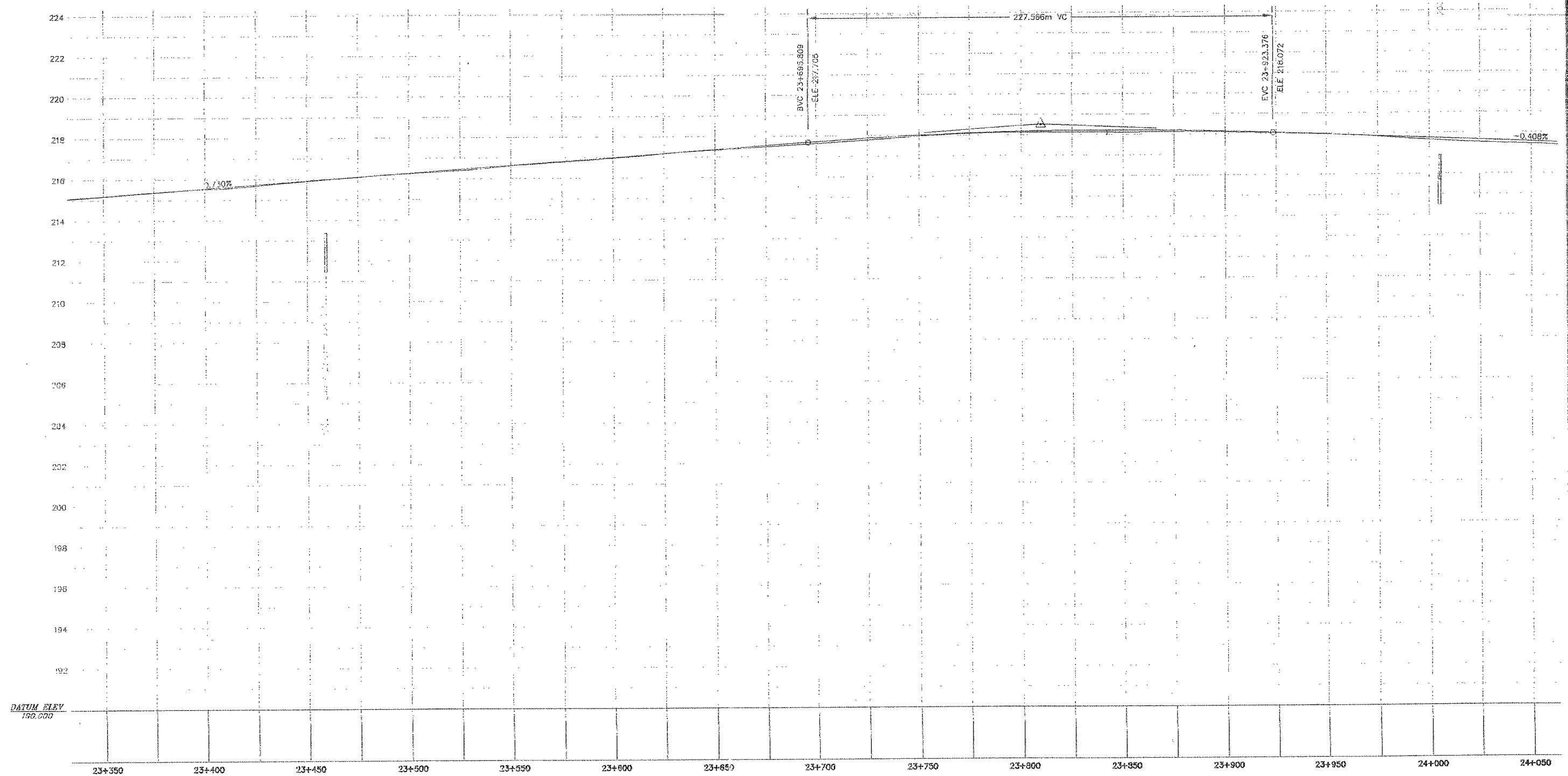
CHECKED BY: SDH

DATE: JUNE 1998

DRAWING NO.: 13

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ONTARIO



LEGEND:

- CLAYEY SOILS
- SURFACIAL ORGANIC SOILS



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

**HIGHWAY #69**  
**NORTHBOUND LANE**  
**STATION 23+350 - 24+050**

PROJECT NO.:	BR-11546-A
SCALE:	AS NOTED
DRAWN BY:	SDH
CHECKED BY:	SDH
DATE:	JUNE 1998
DRAWING NO.:	14

XX

ONTARIO





**Foundation Investigation  
& Design Report  
Proposed Muskoka Road 12  
& Highway 69 NBL Bridge Structure  
(Site 42-319N)  
District 52, Huntsville  
WP-217-89-00**

Prepared for:

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SO7657G/C  
October 26, 1998

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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, in the Northern Region of the MTO, District 52, Huntsville.

This project is located in the former Townships of Gibson and Freeman, and in the present Township of Georgian Bay, District of Muskoka. This work 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 structure 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 diamond interchange at the intersection of Muskoka Road 12 and Highway 69; construction of a structure over the Moon River for the southbound lanes; and rehabilitation of the existing Moon River Bridge for the northbound lanes; construction of two truck lay by areas, and construction of associated side roads resulting from the creation of the controlled access highway.

The following report addresses the foundation investigation and design implications of a proposed three-span bridge for the new northbound lanes of Highway 69 at Muskoka Road 12. Other reports prepared by Trow Consulting Engineers Ltd. address the geotechnical and foundation concerns of the other portions of this project.

## **PART 1 FOUNDATION INVESTIGATION**

### **1.1 Introduction**

This submission presents the results of a foundation investigation completed by Trow Consulting Engineers Ltd. (Trow) for the proposed bridge structure at Muskoka Road 12 and the proposed North Bound Lanes (NBL) of Highway 69, Site 42-319N. It is Trow's understanding that an approximately 36 m long, three-span structure will be designed and constructed at this site. This report contains factual information (obtained from the field investigation) pertaining to the design parameters required for the bridge foundations and related earthworks for the NBL. A similar report addresses the design parameters for the SBL structure, located adjacent to this structure.

### **1.2 Site Description and Geological Setting**

#### **1.2.1 Site Description**

The site is located along the proposed NBL lanes, which currently forms the two lanes of the existing Highway 69 and is approximately 800 m south of the Moon River. It is proposed that Muskoka Road 12 will be realigned, from its present intersection with Highway 69, to a new location, approximately 400 m to the south, and will travel underneath the main lanes of Highway 69.

The site is within Lot 13, Concession 13, of the geographic Gibson Township, in the present Township of Georgian Bay, District of Muskoka. At the proposed overpass, the existing Highway 69 is relatively level and is raised approximately 0.5 m to 0.75 m above the surrounding grade, which comprises numerous intermittent shallow bedrock outcrops. Some blast rock fill is evident on the exposed sides of the road shoulders in a few random sections.

#### **1.2.2 Geological Setting**

Published geological information confirms that the site is located in what is known as the central gneiss belt, with bedrock consisting of Precambrian gneiss. As previously noted, the topography in the area is reasonably level and appears to consist of outcropping bedrock and other areas with shallow bedrock. The overburden, between the intervening shallow bedrock areas, is expected to consist, beneath the surficial organic veneer and existing Highway 69 road construction (asphalt and

sand and gravel or rock fill) of glaciofluvial deposits comprising silt, sand and gravel with cobbles, including proglacial granular deltaic soils.

### **1.3 Investigative Procedures**

#### **1.3.1 General**

Part 1 of this report describes the investigative procedures adopted for the geotechnical assessment of the proposed overpass structure for the Highway 69 (NBL) and Muskoka Road 12 interchange. Properties of the overburden soils were obtained by in-situ and laboratory testing. The procedures, used during the investigation, are described below.

#### **1.3.2 Field Investigation**

The field work for the investigation was carried out between August 26 to August 28, 1998, and consisted of eight(8) sampled boreholes (BH's 5C to 12C), and an additional eight(8) unsampled probes (AP-9C to AP-16C) to define the shallow rock profile. Five explorations (two boreholes and three probes) were completed at each of the two abutment foundation elements, and three explorations (two boreholes and one probe) at each of the two smaller pier elements. The depth of boreholes and probes varied from 1.0 m (shallow refusal level at probe 14C) to a maximum depth of 7.3 m in borehole 8C (cored into bedrock).

The borehole and probe locations are shown on the attached site plan, Drawing 1A, in Appendix A. These locations, as well as the surface elevations, were established by a survey crew from R.V. Anderson Associates Ltd., and are referenced to geodetic datum.

The boreholes and rock probes were advanced through the overburden soils using a track-mounted CME-55 drill, equipped with solid and hollow stem augers, and operated by an approved soils drilling contractor, Master Soil Investigations Ltd. Soil samples were obtained by using a 51 mm O.D. split-spoon sampler in conjunction with standard penetration tests (ASTM D1586) at approximately 0.75 m and 1.5 m intervals. The standard penetration (N) values were recorded and used to provide an assessment of the compactness of the overburden soils. The recovered soil samples were used for identification and laboratory testing.

At four(4) of the borehole locations, i.e. one at each of the four foundation elements, conventional rock coring techniques were used to advance the boreholes approximately 3 metres into the underlying bedrock. A "BQ" size core barrel and casing were used and core samples of the bedrock were retrieved for rock quality determination and classification.

Details of the soil and bedrock conditions encountered in the boreholes are included on the logs in Appendix A. Further details on soil descriptions for classification purposes may be found on Drawings 2A and 2B.

### **1.3.3 Laboratory**

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

- Natural moisture content determinations
- Grain size distribution analyses

The laboratory test results are summarized on the attached borehole logs in Appendix A. The grain size distribution for selected soil samples are presented in Appendix B.

## **1.4 Subsurface Conditions**

The borehole locations are shown on the site plan in Appendix A. Soil sections, longitudinal, as well as at each of the four foundation elements, are plotted on Drawings 1A, 1B and 1C. Based on this information, the following different soil layers were encountered:

- road base and subbase
- fill (subgrade)
- bedrock

A summary of the above soil strata encountered in the boreholes, and inferred from the probes, is presented below.





#### **1.4.1 Road Base and Subbase**

The existing Highway 69, within the driving lanes, consists of approximately 300 mm of asphaltic concrete, based on the results of probes AP-13C to AP-16C, which were drilled close to the centre line of Highway 69. The existing shoulders are known, from previous investigations, to have an approximately 150 mm thick layer of asphaltic concrete over underlying granulars. Beneath the asphaltic concrete and along the outside edge of the existing Highway 69 road shoulders (boreholes 5C to 12C and probes 9C to 12C), a base layer of dense, crushed, sand and gravel, which is an average of 200 mm to 300 mm thick, was encountered.

#### **1.4.2 Fill (Subgrade)**

Beneath the road base and subbase, a fill layer (subgrade) was encountered, consisting of dense sand and gravel, which contains some cobbles sizes, as well as areas of blast rock fill. The depths of the granulars are generally less than 1.5 m thick, although at three locations (borehole 8C and probes 10C and 12C), the sand and gravel stratum is deeper, extending to a depth of approximately 4.5 m.

The compactness of the sand and gravel, based on the standard penetration resistance, "N", values, ranged from 10 blows/300 mm penetration to greater than 60 blows/150 mm penetration, indicating a compact to very dense state. In general, the "N" values exceed 30 blows/300 mm penetration, confirming an overall dense condition. Moisture contents of the deposit are in the 5% to 12% range.

Grain size analyses on samples, secured within the 51 mm O.D. split-spoon sampler, indicate that the "matrix" is a well-graded sand and gravel, with less than 10% silt content. The deposit also contains cobble sizes and blast rock fill boulders, which are not represented on the grain size curves in Appendix B.

#### **1.4.3 Bedrock**

Bedrock was confirmed by retrieving "BQ" size cores in boreholes 6C, 8C, 10C and 12C, i.e. at one location beneath each of the four foundation elements. Based on the borehole and rock probe data, the bedrock level was established at the following depths in the following locations:

- ***North Abutment***

BH 5C, 1.52 m depth, El. 219.7 m\*

BH 12C, 1.22 m depth, El. 220.0 m (proved by rock coring)

AP 9C, 1.4 m depth, ~El. 220.4 m\*

AP 12C, 4.6 m depth, ~El. 216.9 m\*

AP 15C, ~1.4 m depth, ~El. 220. m\*

- ***South Abutment***

BH 8C, 4.3 m depth, El. 216.1 m (proved by rock coring)

BH 9C, 1.1 m depth, ~El. 219.0 m\*

AP 10C, 4.6 m depth, ~El. 215.6 m\*

AP 11C, 1.4 m depth, ~El. 218.7 m\*

AP 14C, 1.0 m depth, ~El. 219.7 m\* (probably blast rock fill)

- ***North Pier***

BH 6C, 1.68 m depth, El. 219.4 m (proved by rock coring)

BH 11C, 1.2 m depth, ~El. 219.8 m\*

AP-13C, 1.3 m depth, ~El. 220.0 m\*

- ***South Pier***

BH 10C, 0.91 m depth, El. 219.6 m (proved by rock coring)

BH 7C, 1.3 m depth, ~El. 219.3 m\*

AP 16C, 0.9 m depth, ~El. 220.0 m\*

Generally, bedrock is expected within 1.5 m of grade, except in the vicinity of AP 12C, north abutment; BH 8C and AP 10C, south abutment, where the rock is locally deeper, i.e. in the order of 4.5 m below grade.

---

\*Bedrock levels were inferred from "grinding" refusal to the machine augers. This refusal could, however, also represent a boulder or blast rock fill obstruction in some instances.

The above elevations were estimated, based on the boreholes and auger probe holes drilled at the pier and abutment locations. Interpolation between boreholes and probe holes 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 erratic and hence may vary between explorations.

Detailed descriptions of the recovered bedrock cores are presented in Table 1-1 in Appendix A. Generally, the bedrock can be described as a pink-grey, Biotite-Hornblende gneiss. The rock is strong, although it appears to be slightly weathered in the upper approximately 1.0 m to 1.5 m in random locations. The joints and fractures are generally very widely spaced, i.e. in the 2 m to 6 m spacing width. The only exception appears to be at borehole 8C, where the fractures and discontinuities are moderately spaced, i.e. 0.2 m to 0.6 m. The fractures occur at angles of 45° to 90° from the vertical.

Rock core recovery was 100% for all four runs. The Rock Quality Designation (RQD) values ranged from 85% to 95% in boreholes 6C, 10C and 12C, i.e. good to excellent rock quality. At borehole 8C, which contains fractures at moderate spacings, the RQD is less, i.e. 55% in the upper approximately 1 m, increasing with depth to 78%, indicating a rock of fair quality, increasing to good quality with depth.

### **1.5 Groundwater Conditions**

No groundwater seepages were encountered during the field work in the dense/very dense, shallow overburden.

## PART 2 ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

### 2.1 General

The following sections address geotechnical considerations pertaining to the proposed three-span bridge which will carry the north bound lanes of Highway 69 over the realigned section of Muskoka Road 12.

The existing grade at the site will be lowered slightly from its present approximate elevation of 221 m down to an approximate elevation of 217.8 m to accommodate Muskoka Road 12 beneath Highway 69. The elevations of the bridge deck, carrying the two north bound lanes of Highway 69 over Muskoka Road 12, will be at approximate elevation 224.5 m.

### 2.2 Foundations

#### 2.2.1 Spread Foundations

Based on the borehole, probe data and proposed grades, it is apparent that the foundations will be in or close to the bedrock. As such, spread foundations established on bedrock are envisaged. 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.

<b>Table 2-1</b> <b>Spread Footing Capacity on Bedrock</b>	
	<b>Spread Footing</b>
<b>Factored Bearing Capacity at ULS</b>	<b>7,500 kPa</b>

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.

Based on the exploration results, it would appear that the major portion of all four foundation elements will be in bedrock, particularly at the piers. Furthermore, in other areas, minimal overburden will need to be removed to expose the rock, i.e. less than 2 m.

The level of bedrock, at the four foundation elements, is summarized below:

- ***North Abutment***

BH 5C, 1.52 m depth, El. 219.7 m\*  
BH 12C, 1.22 m depth, El. 220.0 m (proved by rock coring)  
AP 9C, 1.4 m depth, ~El. 220.4 m\*  
AP 12C, 4.6 m depth, ~El. 216.9 m\*  
AP 15C, ~1.4 m depth, ~El. 220.0 m\*

- ***South Abutment***

BH 8C, 4.3 m depth, El. 216.1 m (proved by rock coring)  
BH 9C, 1.1 m depth, ~El. 219.0 m\*  
AP 10C, 4.6 m depth, ~El. 215.6 m\*  
AP 11C, 1.4 m depth, ~El. 218.7 m\*  
AP 14C, 1.0 m depth, ~El. 219.7 m\* (probably blast rock fill)

- ***North Pier***

BH 6C, 1.68 m depth, El. 219.4 m (proved by rock coring)  
BH 11C, 1.2 m depth, ~El. 219.8 m\*  
AP 13C, 1.3 m depth, ~El. 220.0 m\*

- ***South Pier***

BH 10C, 0.91 m depth, El. 219.6 m (proved by rock coring)  
BH 7C, 1.3 m depth, ~El. 219.3 m\*  
AP 16C, 0.9 m depth, ~El. 220.0 m\*

\*Bedrock levels were inferred from "grinding" refusal to the machine augers. This refusal could, however, also represent a boulder or blast rock fill obstruction in some instances.

Placement of all foundation concrete will be on sound bedrock, which has been cleaned completely of all loose debris and rock shatter, using air hose and water jetting procedures. The footings should preferably be established on a relatively level rock surface, i.e. generally sloping at an angle of say less than  $10^\circ$ . In some instances, footings can be placed on bedrock sloping up to  $25^\circ$  to  $30^\circ$ , provided dowels are incorporated to resist shear. Where the rock slopes at steeper angles, it is normally levelled by benching. If it is necessary to raise the foundation grade, because of low bedrock elevations or possibly resulting from rock "overbreak" during construction, then structural fill, in the form of lean concrete, may be considered.

As per Section 6-8.4.2 of the Ontario Highway Bridge design code, a reduction factor would normally be applied to the Ultimate Bearing Resistance at ULS (7,500 kPa) to account for the effects of inclined loadings. Recent comments, however, received from the Pavement and Foundation Section of MTO, indicate that *"Although, the OHBDC code talks about bearing resistance reduction due to inclined loading for footing on bedrock. The OHBDC committee has decided that no such reduction will be required if the footing is constructed on bedrock."* As such, for spread footings on bedrock, the structural engineer should consult with the Ministry to confirm that a reduction factor for inclined loadings need not be applied.

It is anticipated that the required maximum depth to bedrock, allowing for the lowering of Muskoka Road 12, will be in the order of 2 m. Slopes of intact, sound bedrock, provided any "loose" has been properly scaled off the face, should stand safely at or near a vertical angle for slopes less than 3 m high. It is recommended that the closest edge of the foundation element be established no closer than a set back of 2 m from the face of the sound bedrock cut.

### 2.2.2 Construction Considerations

Removal of any shallow overburden pockets and the existing Highway 69 pavement should be straightforward. Some water seepages into the excavation may occur, particularly during wet periods of the year, however, it should be possible to remove any such seepages using conventional drainage techniques, i.e. pumping from properly filtered sumps and ditches. The existing ditches must be redirected away from the excavation areas.

Most of the overburden soils are expected to consist of sand and gravel, possibly containing cobble sizes, the odd random boulder, and areas of blast rock fill, and can be classified as a Type 2 soil with respect to the Occupational Health and Safety Act for Construction Projects.

Sides of temporary excavations in the overburden should remain stable during the anticipated short construction stage, if cut back at a safe slope of 45 degrees to the horizontal.

Any bedrock excavations will require drilling and blasting techniques. The gneiss bedrock is known to be brittle and it contains some fractures and joints. It will probably be difficult to blast to "neat" lines using conventional drilling and blasting procedures, since problems with "overbreak" are common in this type of bedrock. This potential problem may affect quantities claimed by the contractor for rock excavations, as well as the amount of imported fill required to compensate for "overbreak". The contractor should, therefore, make adequate allowances for these conditions. Consideration should be given to pre-splitting techniques in critical areas in order to reduce potential problems, such as where foundations are designed near bedrock cut slopes. Limiting the depth of sub-drilling to control overbreak beneath the required foundation elements, while still achieving the desired break, is also an important factor that must be considered by the contractor. Overbreak conditions, i.e. rock shatter, will need to be removed down to sound, intact bedrock to ensure the design ULS bearing pressure of 7500 kPa.

### 2.3 Frost Protection

Frost cover is not required for footings placed directly on bedrock.

### 2.4 Lateral Resistance

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

If the factored resistance against sliding failure is inadequate based on friction, then the footings 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 bedrock at the site will likely require dowels.

## 2.5 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.

<b>Table 2-2</b> <b>Material Types and Unfactored Properties</b>					
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	21.1	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.

## 2.6 Approach Embankments

No stability or significant settlement problems are anticipated for the approach embankments established over the competent granular soils and very shallow bedrock. Topsoil and compressible organics (if present) must be removed from the plan limit of the approach embankments. Based on the adjacent borings, it is unlikely that the surficial organics will exceed 300 mm. If rock fill is used to construct the approach embankments, the side slopes and forward slopes should be constructed at a maximum gradient of 1.25H:1V. If Granular "A" or Granular "B" is used, the forward and side slopes should be constructed at 2H (minimum):1V.




## 2.7 General

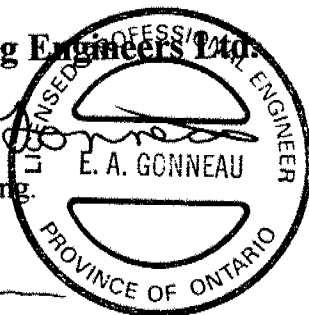
The information presented in this report is based on an investigation designed to provide information to support an overall assessment of the current geotechnical conditions at the site of the proposed Muskoka Road 12, Highway 69 (NBL) bridge structure. The conclusions presented in this report reflect site conditions existing at the time of the investigation. It is noted that the soil boundaries indicated on the 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.

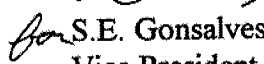
The field investigation was performed by Mr. S. McAuliffe, senior field technician, and supervised by Mr. D.R. Thompson, P.Eng. This report has been prepared by Mr. I.W. Gore, P.Eng., and Mr. E.A. Gonneau, P.Eng., and reviewed by Mr. S.E. Gonsalves, P.Eng.


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.**

  
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IWG:gmw/6.1/Geo/Rpt/7657G-C1

**APPENDIX A**

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# OVERSIZE DRAWING(S)

**NOTES ON SAMPLE DESCRIPTIONS**

1. All descriptions included in this report follow the I.S.S.M.F.E. as suggested in the Canadian Foundation Manual. The laboratory grain-size analysis also follows this classification system. Others may designate the unified classification system as their source; a comparison of the two is shown for your information. Please note that, with the exception of those samples where the grain-size analysis has been carried out, all samples are classified visually and the accuracy of visual examination is not sufficient to differentiate between the classification systems or exact grain sizing.

UNIFIED SOIL CLASSIFICATION	Fines (silt or clay)				Sand			Gravel		Cobbles																				
					Fine	Medium	Coarse	Fine	Coarse																					
	I.S.S.M.F.E. SOIL CLASSIFICATION	Clay	Silt			Sand			Gravel			Cobbles																		
			Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse																			
			Sieve Sizes																											
Particle Size (mm)																														
	0.001	0.002	0.003	0.004	0.006	0.008	0.01	0.02	0.03	0.04	0.06	0.08	0.1	0.2	0.3	0.4	0.6	0.8	1.0	2.0	3.0	4.0	6.0	8.0	10	20	30	40	60	80

2. **FILL:** Where fill is designated on the borehole log, it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of the site fill material. All fills should be expected to contain obstructions such as large concrete pieces of subsurface basements, floors, tanks, etc.; none of these may have been encountered in the borehole. Since boreholes cannot accurately define the contents of fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact and correct composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant on-going and future settlements. Some fill material may be contaminated by toxic waste that renders it unacceptable for deposition in any but designated land fill sites. Unless specifically stated, the fill on this site has not been tested for contaminants that may be considered hazardous. This testing and a potential hazard study can be carried out if you so request. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common but are not detectable using conventional geotechnical procedures.
3. **TILL:** The term till on the borehole logs indicate that the material originates from a geological process associated with glaciation. As a result of this geological process, the till must be considered heterogeneous in composition and, as such, may contain pockets and/or seams of material such as sand, gravel silt or clay. As till often contains cobbles (60 to 200 mm) or boulders (over 200 mm), contractors may encounter them during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size, or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited areas; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till material.

## NOTES ON SAMPLE DESCRIPTIONS (Cont'd)



Project No: S07657G/C

Drawing No: 2B

4. The following table gives a description of the soil based on particle sizes. With the exception of those samples where grain-size analyses have been performed, all samples are classified visually. The accuracy of visual examination is not sufficient to differentiate between this classification system or exact grain size.

Soil Classification		Terminology	Proportion
Clay	< 0.002 mm	"trace" (eg. trace sand)	1% - 10%
Silt	0.002 to 0.06 mm	"some" (eg. some sand)	10% - 20%
Sand	0.06 to 2 mm	adjective (eg. sandy)	20% - 35%
Gravel	2 to 60 mm	and (eg. and sand)	> 35%
Cobbles	60 to 200 mm	noun (eg. boulders)	> 35% and
Boulders	> 200 mm		main fraction

Classification system as suggested in the Canadian Foundation Engineering Manual, 3rd Edition, unless otherwise noted.

The compactness of cohesionless soils and the consistency of cohesive soils are defined by the following:

Cohesionless Soil		Cohesive Soil	
Compactness	Standard Penetration Resistance "N" Blows/0.3 m	Consistency	Undrained Shear Strength (kPa)
Very Loose	0 to 4	Very Soft	< 12
Loose	4 to 10	Soft	12 - 25
Compact	10 to 30	Firm	25 - 50
Dense	30 to 50	Stiff	50 - 100
Very Dense	Over 50	Very Stiff	100 - 200
		Hard	> 200

### 5. Rock Coring

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundness of the rock mass. It is obtained from the rock cores by summing the length of core recovered, counting only those pieces of sound core that are 100 mm or more in length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

RQD Classification	RQD
Very poor quality	< 25
Poor quality	25 - 50
Fair quality	50 - 75
Good quality	75 - 90
Excellent quality	90 - 100

$$\text{Recovery Designation \% Recovery} = \frac{\text{Length of Core Per Run} \times 100}{\text{Total Length of Run}}$$

# RECORD OF BOREHOLE BH-5C 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station +24+636, offset 7 m left of centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 26, 1998 CHECKED BY I.G.

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	wl		
221.27	GROUND SURFACE															
0.00	CRUSHED GRANULAR FILL, 250 mm over SAND FILL, with some gravel & cobbles at lower levels, brown, moist. (compact)	II	1	SS	14	221										
219.75						220	⊗				○				3% 90% 7%	
1.52	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER  Notes: 1) This borehole forms part of the Highway 69, Muskoka Road #12 Interchange, Northbound Lane Bridge Foundation Investigation. 2) Borehole drilled at U.T.M. coordinates 4 990 541.5 N, 282 348.3 E. 3) Borehole was dry & open to 1.1 m depth on completion.															



# RECORD OF BOREHOLE BH-6C 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station +24+627, offset 7 m left of centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 26, 1998 CHECKED BY I.G.

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		20	40	60	80	wp	w	wl		
221.04	GROUND SURFACE													
0.00	CRUSHED GRANULAR ROADBASE, 200 mm over SAND & GRAVEL FILL, with cobbles & blast rock, brown, moist. (compact to dense)	T	1	SS	19									
219.36			2	SS	68									
1.68	BIOTITE HORNBLENDE GNEISS, good rock quality, unweathered.		3	BQ										29% 61% 10%
			4	BQ										RQD 85% Rec. 100%
216.32	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER													RQD 90% Rec. 100%
4.72	Notes: 1) This borehole forms part of the Highway 69, Muskoka Road #12 Interchange, Northbound Lane Bridge Foundation Investigation. 2) Borehole drilled at U.T.M. coordinates 4 990 532.8 N, 282 352.0 E.													



# RECORD OF BOREHOLE BH-7C 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station 24+608, offset 7 m left of centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 26, 1998 CHECKED BY I.G.

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	wp	w			wl	10
220.59	GROUND SURFACE																
0.00	CRUSHED GRANULAR ROADBASE, ~200 mm over SAND & GRAVEL FILL, brown, moist.	T1															
219.32	(compact)		1	SS	11												
1.27	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER																
Notes: 1) This borehole forms part of the Highway 69, Muskoka Road #12 Interchange, Northbound Lane Bridge Foundation Investigation. 2) Borehole drilled at U.T.M. coordinates 4 990 515.9 N, 282 359.2 E. 3) Borehole was dry & open to 0.5 m depth on completion.																	





# RECORD OF BOREHOLE BH-8C 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station ~24+598, offset ~7 m left of centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 27, 1998 CHECKED BY I.G.

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		20	40	60	80	wp	w	wl			
220.34	GROUND SURFACE														
0.00	CRUSHED GRANULAR ROADBASE, ~250 mm over SAND & GRAVEL FILL, with cobbles, some boulders & rock fill. (dense to very dense)		1	SS	22										25% 65% 10%
			2	SS	74										
216.07	BIOTITE HORNBLende GNEISS, fair to good rock quality, slightly weathered to ~4.8 m depth then unweathered.		3	BQ											RQD 54% Rec. 100%
4.27			4	BQ											RQD 78% Rec. 100%
213.02	Notes: 1) This borehole forms part of the Highway 69, Muskoka Road #12 Interchange, Northbound Lane Bridge Foundation Investigation. 2) Borehole drilled at U.T.M. coordinates 4 990 506.5 N, 282 363.1 E. 3) Drill met auger refusal at ~0.8 m depth on first attempt. Drill then made a second attempt ~0.6 m north of BH-8C & met auger refusal at ~0.7 m depth. A third hole was then drilled ~0.6 m south of BH-8C, meeting auger refusal at ~2.7 m depth. Hole was then taken to completion using BW casing & wash boring methods to bedrock before coring.														
7.32	END OF BOREHOLE														



# RECORD OF BOREHOLE BH-9C 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station ~24+595, offset ~7 m right of centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 27, 1998 CHECKED BY I.G.

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	20	40
220.12	GROUND SURFACE																	
0.00	CRUSHED GRANULAR ROADBASE, ~250 mm over SAND & GRAVEL FILL (dense)																	
219.05																		
1.07	END OF BOREHOLE DUE TO REFUSAL ON BEDROCK OR BOULDER																	
	Notes: 1) This borehole forms part of the Highway 69, Muskoka Road #12 Interchange, Northbound Lane Bridge Foundation Investigation. 2) Borehole drilled at U.T.M. coordinates 4 990 509.3 N, 282 377.0 E. 3) Borehole was dry & open to ~0.5 m depth on completion.																	



# RECORD OF BOREHOLE BH-10C 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station "24+604, offset "7 m right of centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 28, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				NATURAL MOISTURE CONTENT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER			TYPE	BLOWS/0.3m	CONE PENETRATION TEST				WATER CONTENT (%)			
						20	40	60	80	wp	w	wl			
220.53	GROUND SURFACE														
0.00	CRUSHED GRANULAR ROADBASE, "200 mm over SAND & GRAVEL FILL (dense)	F	1	SS	60										
219.62															
0.91	BIOTITE HORNBLENDE GNEISS, excellent rock quality, unweathered.		2	BQ											
			3	BQ											
216.57	END OF BOREHOLE														
3.96	Notes: 1) This borehole forms part of the Highway 69, Muskoka Road #12 Interchange, Northbound Lane Bridge Foundation Investigation. 2) Borehole drilled at U.T.M. coordinates 4 990 517.6 N, 282 373.6 E.														



# RECORD OF BOREHOLE BH-11C 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station ~24+623, offset ~6 m right of centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 27, 1998 CHECKED BY I.G.

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/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
220.95	GROUND SURFACE														
0.00	CRUSHED GRANULAR ROADBASE, ~250 mm over SAND & GRAVEL FILL (dense)	F													
219.78		F	1	SS	47	220									38% 57% 5%
1.17	END OF BOREHOLE DUE TO REFUSAL ON BEDROCK OR BOULDER														
Notes: 1) This borehole forms part of the Highway 69, Muskoka Road #12 Interchange, Northbound Lane Bridge Foundation Investigation. 2) Borehole drilled at U.T.M. coordinates 4 990 534.3 N, 282 365.6 E. 3) Borehole was dry & open to ~0.3 m depth on completion.															



# RECORD OF BOREHOLE BH-12C 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station ~24+632, offset ~7 m right of centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 28, 1998 CHECKED BY I.G.

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					wp ——— w ——— wl						
												WATER CONTENT (%)						
							SHEAR STRENGTH: Cu, KPa											
							UNCONFINED QUICK TRIAXIAL      FIELD VANE LAB SHEAR											
							20      40      60      80				10      20      30      40							
221.21	GROUND SURFACE																	
0.00	CRUSHED GRANULAR FILL, ~250 mm over SAND & GRAVEL FILL, some rock pieces, brown, moist. (dense)	F																
219.99																		
1.22	BIOTITE HORNBLLENDE GNEISS, good to excellent rock quality, unweathered.																	
			2	BQ												RQD 95% Rec. 100%		
			3	BQ												RQD 85% Rec. 100%		
216.94	END OF BOREHOLE																	
4.27	Notes: 1) This borehole forms part of the Highway 69, Muskoka Road #12 Interchange, Northbound Lane Bridge Foundation Investigation. 2) Borehole drilled at U.T.M. coordinates 4 990 542.8 N, 282 362.3 E.																	



# RECORD OF BOREHOLE AP-9C 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station 24 + 642, offset 7 m left of centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 26, 1998 CHECKED BY I.G.

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	wl			10
221.80 0.00	GROUND SURFACE																
	Probable SAND & GRAVEL FILL					221											
220.43 1.37	END OF AUGER PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER																
Notes: 1) This auger probe forms part of the Highway 69, Muskoka Road #12 Interchange, Northbound Lane Bridge Foundation Investigation. 2) Auger probe drilled at U.T.M. coordinates 4 990 546.1 N, 282 345.5 E.																	



# RECORD OF BOREHOLE AP-10C 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station 24+592, offset 7 m left of centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 26, 1998 CHECKED BY I.G.

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					
220.13	GROUND SURFACE						220									
0.00	CRUSHED GRANULAR FILL, ~250 mm over															
	Probable SAND & GRAVEL FILL															
218.91							219									
1.22							218									
							217									
	Probable SAND & GRAVEL with COBBLES						216									
215.56	END OF AUGER PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER															
4.57	Notes: 1) This auger probe forms part of the Highway 69, Muskoka Road #12 Interchange, Northbound Lane Bridge Foundation Investigation. 2) Auger probe drilled at U.T.M. coordinates 4 990 501.0 N, 282 365.4 E. 3) Probe hole was dry & open to 2.1 m depth on completion.															



# RECORD OF BOREHOLE AP-11C 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station 24+588, offset 7 m right of centreline of Northbound Lane  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55  
 DATUM Geodetic DATE August 27, 1998

ORIGINATED BY S.M.

COMPILED BY M.D.

CHECKED BY I.G.

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	wp					
220.10	GROUND SURFACE																
0.00	CRUSHED GRANULAR FILL, ~250 mm over Probable SAND & GRAVEL FILL																
218.73																	
1.37	END OF AUGER PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER																
	Notes: 1) This auger probe forms part of the Highway 69, Muskoka Road #12 Interchange, Northbound Lane Bridge Foundation Investigation. 2) Auger probe drilled at U.T.M. coordinates 4 990 502.9 N, 282 379.7 E.																





# RECORD OF BOREHOLE AP-12C 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station ~24+638, offset ~7 m right of centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 27, 1998 CHECKED BY I.G.

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
221.45	GROUND SURFACE														
0.00	CRUSHED GRANULAR FILL, ~250 mm over														
220.54	Probable SAND & GRAVEL FILL														
0.91															
	Probable SAND with GRAVEL, some cobble sizes in lower ~1.5 m depth.														
216.88															
4.57	END OF AUGER PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
	Notes: 1) This auger probe forms part of the Highway 69, Muskoka Road #12 Interchange, Northbound Lane Bridge Foundation Investigation. 2) Auger probe drilled at U.T.M. coordinates 4 990 548.0 N, 282 359.8 E.														



# RECORD OF BOREHOLE AP-13C 1 OF 1

## METRIC

W.P. 217-89-00 LOCATION Station +24+624, offset +1 m right of centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 26, 1998 CHECKED BY I.G.

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					
221.40	GROUND SURFACE															
0.00	ASPHALT, ~300 mm over						221									
220.90	Probable CRUSHED GRANULAR ROADBASE, ~200 mm thick															
0.50	Probable SAND & GRAVEL FILL															
220.05	END OF AUGER PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER															
1.35	Notes: 1) This auger probe forms part of the Highway 69, Muskoka Road #12 Interchange, Northbound Lane Bridge Foundation Investigation. 2) Auger probe drilled at U.T.M. coordinates 4 990 533.3 N, 282 360.1 E.															



# RECORD OF BOREHOLE AP-14C 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station ~24+598, offset ~1 m right of centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 26, 1998 CHECKED BY I.G.

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	wp	w		
220.70	GROUND SURFACE														
0.00	ASPHALT, ~300 mm over														
220.10	Probable CRUSHED GRANULAR ROADBASE ~300 mm thick														
0.60	Probable SAND & GRAVEL FILL														
219.70															
1.00	END OF AUGER PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
Notes: 1) This auger probe forms part of the Highway 69, Muskoka Road #12 Interchange, Northbound Lane Bridge Foundation Investigation. 2) Auger probe drilled at U.T.M. coordinates 4 990 509.5 N, 282 370.4 E.															



# RECORD OF BOREHOLE AP-15C 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station 24+633, offset 1 m left of centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 26, 1998 CHECKED BY I.G.

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/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m		20	40	60	80					
221.60 0.00	GROUND SURFACE														
221.00 0.60	ASPHALT, ~300 mm over Probable CRUSHED GRANULAR ROADBASE, ~300 mm thick Probable SAND & GRAVEL FILL					221									
220.15 1.45	END OF AUGER PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER  Notes: 1) This auger probe forms part of the Highway 69, Muskoka Road #12 Interchange, Northbound Lane Bridge Foundation Investigation. 2) Auger probe drilled at U.T.M. coordinates 4 990 540.8 N, 282 354.7 E.														



# RECORD OF BOREHOLE AP-16C 1 OF 1

METRIC

W.P. 217-89-00 LOCATION Station ~24+606, offset ~1 m left of centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 26, 1998 CHECKED BY I.G.

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					
220.90	GROUND SURFACE														
0.00	ASPHALT, ~325 mm over														
220.45	Probable CRUSHED GRANULAR ROADBASE, 125 mm thick														
0.45	Probable SAND & GRAVEL FILL														
220.00															
0.90	END OF AUGER PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
	Notes: 1) This auger probe forms part of the Highway 69, Muskoka Road #12 Interchange, Northbound Lane Bridge Foundation Investigation. 2) Auger probe drilled at U.T.M. coordinates 4 990 516.1 N, 282 365.4 E.														



S07657G/C

**TABLE 1-1  
ROCK CORE DESCRIPTION**

BH#	Core Recovery				Core Description	
	RC #	Depth (m)	% CR*	% RQD**	Depth (m)	Description
<b>NORTH BOUND LANE BRIDGE STRUCTURE MUSKOKA ROAD 12 INTERCHANGE FOUNDATIONS</b>						
6C	3	1.67 to 3.20	100	85	1.67 to 4.72	<b>Biotite Horneblende Gneiss</b> - dark grey to white, medium to coarse grained, unweathered, fractures very widely spaced, dipping at 45° from vertical, planar, smooth, bedding planes at 45° from vertical
	4	3.20 to 4.72	100	90		
8C	3	4.27 to 5.79	100	55	4.27 to 7.32	<b>Biotite Hornblende Gneiss</b> - light grey-to pinkish white, medium to coarse grained, slightly weathered to 4.8 m depth, then unweathered, fractures moderately spaced, dipping at 45° to 90° from vertical, planar, smooth
	4	5.79 to 7.32	100	78		
10C	2	0.91 to 2.44	100	93	0.91 to 3.97	<b>Biotite Horneblende Gneiss</b> - dark grey/grey-pink, medium to coarse grained, unweathered, fractures very widely spaced, dipping at 45° to 90° from vertical, planar, smooth
	3	2.44 to 3.97	100	90		
12C	2	1.22 to 2.74	100	95	1.22 to 4.27	<b>Biotite Horneblende Gneiss</b> - dark grey to white-pink, medium to coarse grained, unweathered, fractures very widely spaced, dipping at 45° to 90° from vertical, planar, smooth, bedding planes at 45° to 90° from vertical,
	3	2.74 to 4.27	100	85		

\*CR Core Recovery %

\*\*RQD Rock Quality Designation %

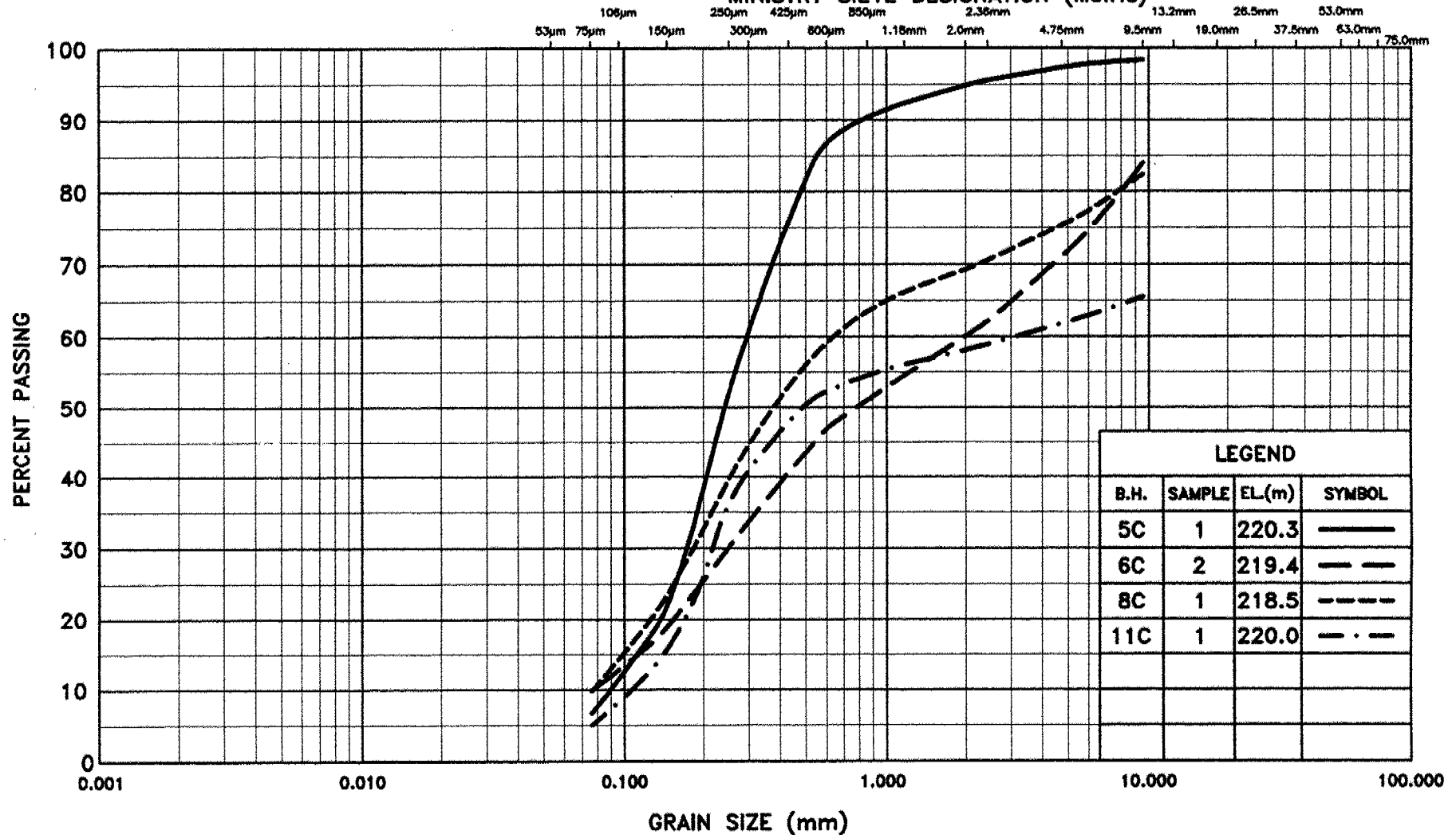
## APPENDIX B

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# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

### GRAIN SIZE DISTRIBUTION

BH-5C, SS-1  
BH-6C, SS-2; BH-8C, SS-1; BH-11C, SS-1

SAND FILL  
SAND & GRAVEL FILL

FIGURE 1

W.P 217-89-00



PROJ. No. S07657GC



**Foundation Investigation  
& Design Report  
Proposed Muskoka Road 12  
& Highway 69 SBL Bridge Structure  
(Site 42-319S)  
District 52, Huntsville  
WP-217-89-00**

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SO7657G/E  
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### Appendix B

Grain Size Analyses

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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, in the Northern Region of the MTO, District 52, Huntsville.

This project is located in the former Townships of Gibson and Freeman, and in the present Township of Georgian Bay, District of Muskoka. This work 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 structure 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 diamond interchange at the intersection of Muskoka Road 12 and Highway 69; construction of a structure over the Moon River for the southbound lanes, and rehabilitation of the existing Moon River Bridge for the north bound lanes; construction of two truck lay by areas, and construction of associated side roads resulting from the creation of the controlled access highway.

The following report addresses the foundation investigation and design implications of a proposed single-span bridge for the new south bound lanes of Highway 69 at Muskoka Road 12. Other reports prepared by Trow Consulting Engineers Ltd. address the geotechnical and foundation concerns of the other portions of this project.

## **PART 1 FOUNDATION INVESTIGATION**

### **1.1 Introduction**

This submission presents the results of a foundation investigation completed by Trow Consulting Engineers Ltd. (Trow) for the proposed bridge structure at Muskoka Road 12 and the proposed Sound Bound Lanes (SBL) of Highway 69, Site 42-319S. It is Trow's understanding that an approximately 30 m long, single-span structure will be designed and constructed at this site. This report contains factual information (obtained from the field investigation) pertaining to the design parameters required for the bridge foundations and related earthworks for the SBL. A similar report addresses the design parameters for the NBL structure, located adjacent to this structure.

### **1.2 Site Description and Geological Setting**

#### **1.2.1 Site Description**

The site is located along the proposed SBL lanes, approximately 35 m to the west of the existing Highway 69 (which will eventually form the NBL of the new four lane Highway 69) and approximately 800 m south of the Moon River. It is proposed that Muskoka Road 12 will be realigned, from its present intersection with Highway 69, to a new location, approximately 400 m to the south, and will travel underneath the main lanes of Highway 69.

The site is within Lot 13, Concession 13, of the geographic Gibson Township, in the present Township of Georgian Bay, District of Muskoka. At the crossing, the terrain is relatively level, with a very gradual overall downward slope towards the south. The area contains mature trees. Some random boulders are strewn over the surface, along with some intermittent bedrock outcrops. A small bedrock knoll outcrops near a portion of the proposed north abutment.

#### **1.2.2 Geological Setting**

Published geological information confirms that the site is located in what is known as the central gneiss belt, with bedrock consisting of Precambrian gneiss. As previously noted, the topography in the area is reasonably level and appears to consist of reasonably shallow areas of bedrock. The overburden, between the intervening shallow bedrock areas, is expected to consist, beneath the

surficial organic veneer, of glaciofluvial deposits comprising silt, sand and gravel with cobbles, including proglacial granular deltaic soils.

### **1.3 Investigative Procedures**

#### **1.3.1 General**

Part 1 of this report describes the investigative procedures adopted for the geotechnical assessment of the proposed structure for the Highway 69 (SBL) and Muskoka Road 12 interchange. Properties of the overburden soils were obtained by in-situ and laboratory testing. The procedures, used during the investigation, are described below.

#### **1.3.2 Field Investigation**

The field work for the investigation was carried out between August 21 to August 25, 1998, and consisted of four(4) sampled boreholes (BH's 1E to 4E), and an additional eight(8) unsampled rock probes (RP-1E to RP-8E) to define the shallow rock profile. Six explorations (two boreholes and four rock probes) were completed at each of the two foundation elements, to a maximum depth of 11.9 m (BH-2E).

The borehole and rock probe locations are shown on the attached site plan, Drawing 1A, in Appendix A. These locations, as well as the surface elevations, were established by a survey crew from R.V. Anderson Associates Ltd., and are referenced to geodetic datum.

The boreholes and rock probes were advanced through the overburden soils using a track-mounted CME-55 drill, equipped with solid and hollow stem augers, and operated by an approved soils drilling contractor, Master Soil Investigations Ltd. Soil samples were obtained by using a 51 mm O.D. split-spoon sampler in conjunction with standard penetration tests (ASTM D1586) at approximately 0.75 m and 1.5 m intervals. The standard penetration (N) values were recorded and used to provide an assessment of the compactness of the overburden soils. The recovered soil samples were used for identification and laboratory testing.

At two (2) of the borehole locations, i.e. one at each of the two foundation elements, conventional rock coring techniques were used to advance the boreholes approximately 3 metres into the

underlying bedrock. A "BQ" size core barrel and casing were used and core samples of the bedrock were retrieved for rock quality determination and classification.

Details of the soil and bedrock conditions encountered in the boreholes are included on the logs in Appendix A. Further details on soil descriptions for classification purposes may be found on Drawings 2A and 2B.

### **1.3.3 Laboratory**

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

- Natural moisture content determinations
- Grain size distribution analyses

The laboratory test results are summarized on the attached borehole logs in Appendix A. The grain size distribution for selected soil samples are presented in Appendix B.

## **1.4 Subsurface Conditions**

The borehole locations are shown on the site plan in Appendix A. Soil sections, longitudinal, as well as at each of the two foundation elements, are plotted on Drawings 1A, 1B and 1C. Based on this information, the following different soil layers were encountered:

- organics
- sand
- sand and gravel
- bedrock

A summary of the above soil strata encountered in the boreholes, and inferred from the probes is presented below.

#### 1.4.1 Organics

An initial layer of organics was observed in all boreholes. These organics, however, are limited to a 150 mm to 250 mm thick surficial veneer of topsoil and roots.

#### 1.4.2 Sand

A thin deposit of compact "loamy" sand, extending to a depth of approximately 1.3 m, was encountered beneath the organics at borehole 2E. A standard split-spoon sample confirmed that the sand is in a compact state ( $N = 12$  blows/300 mm penetration), with a moisture content of less than 5%. Grain size analyses indicate that the deposit is composed of primarily a fine sand, with a silt fraction of approximately 15%. Some roots were also noted in the sand.

#### 1.4.3 Sand and Gravel with Cobbles

This deposit overlies the bedrock at all exploration locations except RP-5E, where overburden is absent (at the northeast corner of the north abutment), and bedrock outcrops. Beneath the remaining area of the north abutment, the sand and gravel is generally less than 2.5 m thick. Beneath the south abutment, the sand and gravel stratum is thicker, extending to depths of approximately 4.5 m (east side) to 8.0 m to 9.0 m (west side).

The compactness of the sand and gravel, based on the standard penetration resistance, "N", values, ranged from 40 blows/300 mm penetration to 60 blows/150 mm penetration, indicating a dense to very dense state. Moisture contents of the deposit are generally less than 5%, although, below the groundwater level (~5 m), the moisture contents increase slightly to between 7% to 11%, based on the soil samples recovered from the deeper borehole 2E.

The deposit contains frequent root and root hairs in the upper ~1.5 m depth.

Grain size analyses on samples, secured within the 51 mm O.D. split-spoon sampler, indicate that the "matrix" is a well-graded sand and gravel, with less than 10% silt content. The deposit also contains cobble sizes and probably random boulders, which are not represented on the grain size curves in Appendix B.



#### 1.4.4 Bedrock

Bedrock was confirmed by retrieving “BQ” size cores in boreholes 2E and 3E, i.e. at one location beneath each of the two foundation elements. Based on the borehole and rock probe data, the bedrock level was established at the following depths in the following locations:

- *North Abutment*

BH 3E, 1.52 m depth, El. 220.3 m (proved by rock coring)

RP 5E, ground level, El. 222.1 m (bedrock outcropping)

BH 4E, ~2.2 m depth, ~El. 218.4 m\*

RP 6E, ~1.3 m depth, ~El. 220.2 m\*

RP 7E, ~2.7 m depth, ~El. 218.2 m\*

RP 8E, ~2.2 m depth, ~El. 219.0 m\*

- *South Abutment*

BH 2E, 8.84 m depth, El. 210.58 m (Proved by rock coring)

BH 1E, ~4.5 m depth, ~El. 215.1 m\*

RP 1E, ~4.4 m depth, ~El. 214.9 m\*

RP 2E, ~8.1 m depth, ~El. 211.1 m\*

RP 3E, ~7.9 m depth, ~El. 211.3 m\*

RP 4E, ~8.5 m depth, ~El. 210.9 m\*

---

\*Bedrock levels were inferred from “grinding” refusal to the machine augers. This refusal could, however, also represent a boulder obstruction in some instances.

As noted above, bedrock either outcrops or is relatively shallow beneath the north abutment, i.e. less than 3 m below grade. Beneath the proposed south abutment, however, the rock is deeper, with levels varying from an inferred elevation of 215 m (~4.5 m below grade) to elevation 210.6 m (8.8 m below grade).

The above elevations were estimated, based on the boreholes and auger probe holes drilled at the abutment locations. Interpolation between boreholes and probe holes 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 erratic and hence may vary between explorations..

Detailed descriptions of the recovered bedrock cores are presented in Table 1-1 in Appendix A. Generally, the bedrock can be described as a pink-grey, Biotite-Hornblende gneiss. The rock is strong, although it appears to be slightly weathered in the upper approximately 1.0 m to 1.5 m, where the bedrock is shallow, i.e. beneath the north abutment (BH 3-E).

Rock core recovery was 100% for all four runs and the Rock Quality Designation (RQD) values ranged from 52% to 97%.

## 1.5 Groundwater Conditions

Very little groundwater was encountered during the field work in the dense/very dense, over-consolidated soils, which have a low permeability.

Based on a careful examination of recovered soil samples and laboratory moisture contents, it is estimated that the groundwater table was at a depth of some 5 m below grade (~El. 214.5 m) at the time of the investigation. This level is subject to seasonal fluctuations and may be higher during wetter periods.

## PART 2 ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

### 2.1 General

The following sections address geotechnical considerations pertaining to the proposed single-span bridge which will carry the south bound lanes of Highway 69 over the realigned section of Muskoka Road 12.

The existing grade at the site will be lowered slightly from its present approximate elevation of 220 m down to an approximate elevation of 218 m to accommodate Muskoka Road 12 beneath Highway 69. The elevations of the bridge deck, carrying the two south bound lanes of Highway 69, will be at approximate elevation 226 m.

### 2.2 Foundations

#### 2.2.1 Spread Foundations

##### 2.2.1.1 *North Abutment*

At the north abutment, bedrock is shallow ( $<2.5$  m below grade). As such, it would be feasible to excavate down to the rock and place the foundations directly on the bedrock surfaces. 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	
	Spread Footing
Factored Bearing Capacity at ULS	5,000 kPa

The above Factored Bearing Capacity at ULS applies to spread footings placed directly on bedrock with a reasonably good Rock Mass Quality (RQD>55). 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 north abutment, the borehole and probe data indicate that the construction of spread footings on bedrock would require excavation and removal of up to approximately 2.5 m of overburden soil. The footing area 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.

Based on the borehole and probe data, bedrock is anticipated at the following levels:

- BH 3E, 1.52 m depth, El. 220.3 m (proved by rock coring)
- RP 5E, ground level, El. 222.1 m (bedrock outcropping)
- BH 4E, ~2.2 m depth, ~El. 218.4 m\*
- RP 6E, ~1.3 m depth, ~El. 220.2 m\*
- RP 7E, ~2.7 m depth, ~El. 218.2 m\*
- RP 8E, ~2.2m depth, ~El. 219.0 m\*

As per Section 6-8.4.2 of the Ontario Highway Bridge design code, a reduction factor would normally be applied to the Ultimate Bearing Resistance at ULS (5,000 kPa) to account for the effects of inclined loadings. Recent comments, however, received from the Pavement and Foundation Section of MTO, indicate that *"Although, the OHBDC code talks about bearing resistance reduction due to inclined loading for footing on bedrock. The OHBDC committee has decided that no such reduction will be required if the footing is constructed on bedrock."* As such, for spread footings on bedrock, the structural engineer should consult with the Ministry to confirm that a reduction factor for inclined loadings need not be applied.

---

\*Bedrock levels were inferred from "grinding" refusal to the machine augers. This refusal could, however, also represent a boulder obstruction in some instances.

## 2.2.1.2 South Abutment

At the south abutment, bedrock is deeper (between approximately 4.5 m to 9.0 m depth). Consequently, it may not be practical to excavate to the deeper bedrock surface. As noted previously, the overburden consists, for the most part, of dense to very dense, sand and gravel with cobble sizes and likely the odd random boulder. Other than the depth of excavation and the possibility of boulder sizes, major excavation problems (related, for example, to groundwater seepages), are not expected. Although some seepages will likely occur in the overburden during excavation, particularly below 5 m, and/or if construction is completed during a wet period of the year, any seepages should not be excessive and probably could be controlled using conventional construction drainage techniques, such as filtered pumps and strategically positioned sumps.

If spread footings to rock are considered to be a feasible and economical option, then 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.

<b>TABLE 2-2</b> <b>Spread Footing Capacity on Bedrock</b>	
	<b>Spread Footing</b>
<b>Factored Bearing Capacity at ULS</b>	<b>7,500 kPa</b>

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.

For the south abutment area, the borehole and probe data indicate that the construction of spread footings on bedrock would require excavation and removal of up to approximately 9 m of overburden soil. The footing area 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.

Based on the borehole and probe data, bedrock is anticipated at the following levels:

- **South Abutment**

- ✓ BH 2E, 8.84 m depth, El. 210.58 m (proved by rock coring)
- BH 1E, ~4.5 m depth, ~El. 215.1 m\*
- RP 1E, ~4.4 m depth, ~El. 214.9 m\*
- RP 2E, ~8.1 m depth, ~El. 211.1 m\*
- RP 3E, ~7.9 m depth, ~El. 211.3 m\*
- RP 4E, ~8.5 m depth, ~El. 210.9 m\*

Because of the potential deep excavation which will be required to reach bedrock, consideration could also be given to establishing the south abutment footing at a higher level in the competent, overconsolidated, dense/very dense, sand and gravel overburden. Provided the foundation is established below the initial weathered zone, i.e. below elevation 217 m, the abutment footing, placed on undisturbed, dense, sand and gravel could be designed using the bearing capacity values specified in Table 2-3, below.

TABLE 2-3 Abutment Footing Capacity on Dense/Very Dense Sand/Sand & Gravel	
	Spread Footing
Factored Bearing Capacity at ULS	450 kPa
Bearing Capacity at SLS	300 kPa

The SLS bearing capacity assumes:

- A foundation width of between 1 m and 4 m
- The anticipated settlement is estimated to be 18 mm. Since the till is essentially granular, most of this settlement is expected to occur during construction, with minimal post construction settlement.

---

\*Bedrock levels were inferred from “grinding” refusal to the machine augers. This refusal could, however, also represent a boulder obstruction in some instances.

## 2.2.2 Piled Foundations

### 2.2.2.1 *Pile Capacities*

Piled foundations established on bedrock may also be considered if an Integral Abutment system is feasible. The bedrock level beneath the north abutment, which is either outcropping or as shallow as 2.5 m below grade, may preclude the use of piled foundations, unless the upper levels of the bedrock are removed to accommodate the structural requirements. From a geotechnical standpoint, end bearing piles are usually feasible, provided the length/width ratio exceeds 10, i.e. a length of 3 m for a 300 mm pile section. Comments on piling capacities have been included for design considerations in Table 2-4, below.

TABLE 2-4 Design Pile Capacities (kN)		
	HP 310x79	HP 310x110
Factored Axial Resistance*	1430	2000
Ultimate Capacity from Hiley Formula (south abutment only)	2475	3450
Note: MTO* = Structural Office Policy Memo 98-01, April 15, 1998		

Based on the attached borehole logs in Appendix A, the previous data in Part 1, Section 1.4.3, of this report, indicates the anticipated approximate level of bedrock. These elevations (except at the two cored boreholes) are approximate and have been interpreted from auger refusal. Furthermore, this type of bedrock can have sharp irregularities in its surface profile, and hence may change significantly over very short distances.

### 2.2.2.2 *Construction of Piled Foundations*

All piles should be driven to bedrock. There may be the occasional pile which may terminate about 1 m above the bedrock surface, i.e. in the very dense sand and gravel, beneath the south abutment, i.e. in the very dense sand and gravel. In this case, pile driving should be controlled by the Hiley

Formula as per MTO Standards SS103-10 or SS103-11, using the ultimate pile capacities referred to previously.

In order to minimize damage, all piles installed on this project must be fitted with reinforcing plates welded to the flanges as per OPSD 3301.

At some locations, the piles may have a tendency to skip over the bedrock surface. In these areas, slight alignment problems may arise and somewhat longer piles may be required. At worst, some of the piles may have to be replaced or added in these areas.

Oslo, or similar rock points, installed and driven in accordance with OPSD 3304 and OPSS 903, respectively, may be considered. Once the locations and orientations of the piles have been determined (i.e. during the preliminary design stage), the use of such methods will be determined and recommendations will be provided by this office.

### 2.2.3 Caisson Foundations

#### 2.2.3.1 *Bearing Capacity*

Where bedrock is deeper beneath the south abutment, a caisson-type foundation system to rock could be considered. Caisson foundations placed directly on bedrock could be designed using the bearing values specified in Table 2-5, below.

<b>TABLE 2-5</b> <b>Caisson Foundation Capacity on Bedrock</b>	
	Spread Footing
Factored Bearing Capacity at ULS	5,000 kPa

The above Factored Bearing Capacity at ULS applies to caisson foundations placed directly on bedrock with a good Rock Mass Quality (RQD>75). The bearing capacity at SLS will not govern for a caisson 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.



### 2.2.3.1 Construction Considerations

Caisson foundations, which are typically large diameter augered elements, which are cased to rock, are not normally feasible in Northern Ontario. The potential for sharp, unknown irregularities in the bedrock surface, difficulties "seating" and sealing casings at the hard strong bedrock contact, as well as the potential excavation difficulties with water seepages in the granular soils and boulders in the overburden, render augered caissons difficult, if not impractical. As such, if caissons are considered at the south abutment, they will likely have to be installed with the assistance of a large backhoe-type excavator, possibly with temporary braced shoring to support the open excavation sides and appropriate dewatering procedures.

## 2.3 Frost Protection

Frost cover is not required for footings placed directly on bedrock. For foundations established on the native granular soil for the south abutment, a minimum frost cover of 2 m should be provided.

## 2.4 Lateral Resistance

The computation of the lateral resistance of the spread footings shall be carried out in accordance with O.H.B.D.C. An unfactored friction angle,  $\phi$ , of 32 degrees can be used for sliding resistance along the bedrock and footing base and 35 degrees for sliding along granular soils (native sand and gravel).

If the factored resistance against sliding failure is inadequate based on friction, then the footings 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 bedrock at the site will likely require dowels.

If piles are used, then lateral loads are normally resisted with battered piles.

## 2.5 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.

<b>Table 2-6</b> <b>Material Types and Unfactored Properties</b>					
<b>Material</b>	<b>Friction Angle, <math>\phi'</math></b>	<b><math>\gamma(\text{kN/m}^3)</math></b>	<b><math>K_a</math></b>	<b><math>K_p</math></b>	<b><math>K_o</math></b>
Granular A	35 degrees	22.5	0.27	3.7	0.43
Granular B	30 degrees	21.1	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.

## 2.6 Excavations

### 2.6.1 Overburden

Excavations of up to 9 m of overburden soil will be required if spread foundations to bedrock are used at the south abutment. The overburden granular soils are classified as Type 2 soils and the maximum depth of excavation anticipated at the site is approximately 9 metres at the west abutment. As such, excavations in accordance with the Occupational Health and Safety Regulations for Construction Projects for Type 2 soils will be adequate, provided the odd seepage, expected below about 5 m depth, is properly controlled.

### 2.6.2 Bedrock

Any removal of bedrock required for the foundations will require drilling and blasting procedures.

## 2.7 Approach Embankments

No stability or significant settlement problems are anticipated for the approach embankments established over the essentially granular soils. Topsoil and compressible organics (if present) must be removed from the plan limit of the approach embankments. Based on the adjacent borings, it is unlikely that the surficial organics will exceed 300 mm. If rock fill is used to construct the approach embankments, the side slopes and forward slopes should be constructed at a maximum gradient of 1.25H:1V. If Granular "A" or Granular "B" is used, the forward and side slopes should be constructed at 2H (minimum):1V.

The geotechnical conditions are such that integral abutment design could be considered (with mechanically stabilized earth retaining walls), if feasible, from structural, practical and economical considerations. It should be noted, however, that the depth to bedrock at the north abutment is currently very shallow, which may preclude an integral system, unless the bedrock level is lowered.

## 2.8 General

The information presented in this report is based on an investigation designed to provide information to support an overall assessment of the current geotechnical conditions at the site of the proposed Muskoka Road 12, Highway 69 (SBL) bridge structure. 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 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.

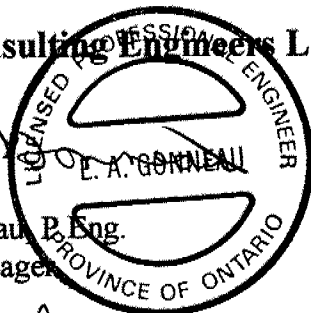
The field investigation was performed by Mr. S. McAuliffe, senior field technician, and supervised by Mr. D.R. Thompson, P.Eng. This report has been prepared by Mr. I.W. Gore, P.Eng., and Mr. E.A. Gonneau, P.Eng., and reviewed by Mr. S.E. Gonsalves, P.Eng.

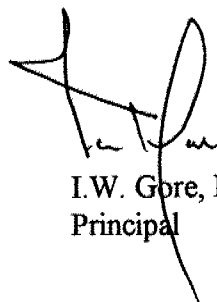
We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

**Trow Consulting Engineers Ltd.**

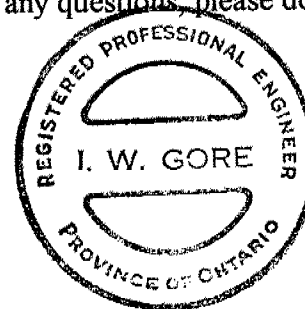


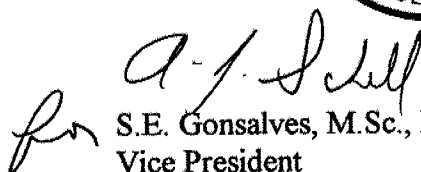
E.A. Gonneau, P.Eng.  
Project Manager





I.W. Gore, M.Sc., P.Eng.  
Principal





S.E. Gonsalves, M.Sc., P.Eng.  
Vice President

Encl.

Dist: R.V. Anderson Associates Ltd.  
Mr. T.H. McColm, P.Eng. (1)  
Project Manager

Ministry of Transportation  
Northern Region  
Mr. M.E. Pearsall, P.Eng. (4)

Ministry of Transportation  
Foundation Section  
Mr. Ken S.Q. Ahmad, P.Eng. (2)

**APPENDIX A**

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# OVERSIZE DRAWING(S)

**NOTES ON SAMPLE DESCRIPTIONS**

- All descriptions included in this report follow the I.S.S.M.F.E. as suggested in the Canadian Foundation Manual. The laboratory grain-size analysis also follows this classification system. Others may designate the unified classification system as their source; a comparison of the two is shown for your information. Please note that, with the exception of those samples where the grain-size analysis has been carried out, all samples are classified visually and the accuracy of visual examination is not sufficient to differentiate between the classification systems or exact grain sizing.

UNIFIED SOIL CLASSIFICATION	Fines (silt or clay)				Sand			Gravel		Cobbles	
					Fine	Medium	Coarse	Fine	Coarse		
I.S.S.M.F.E. SOIL CLASSIFICATION	Clay	Silt			Sand			Gravel			Cobbles
		Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	
	Sieve Sizes										
	Particle Size (mm)										
	0.001 0.002 0.003 0.004 0.006 0.008 0.01 0.02 0.03 0.04 0.06 0.075 0.08 0.1 0.2 0.3 0.4 0.6 0.8 1.0 2.0 3.0 4.0 6.0 8.0 10 20 30 40 60 80										
	200 40 10 4 3/4 3/8 3/16										

- FILL:** Where fill is designated on the borehole log, it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of the site fill material. All fills should be expected to contain obstructions such as large concrete pieces of subsurface basements, floors, tanks, etc.; none of these may have been encountered in the borehole. Since boreholes cannot accurately define the contents of fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact and correct composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant on-going and future settlements. Some fill material may be contaminated by toxic waste that renders it unacceptable for deposition in any but designated land fill sites. Unless specifically stated, the fill on this site has not been tested for contaminants that may be considered hazardous. This testing and a potential hazard study can be carried out if you so request. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common but are not detectable using conventional geotechnical procedures.
- TILL:** The term till on the borehole logs indicate that the material originates from a geological process associated with glaciation. As a result of this geological process, the till must be considered heterogeneous in composition and, as such, may contain pockets and/or seams of material such as sand, gravel silt or clay. As till often contains cobbles (60 to 200 mm) or boulders (over 200 mm), contractors may encounter them during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size, or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited areas; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till material.

## NOTES ON SAMPLE DESCRIPTIONS (Cont'd)



Project No: S07657G/E

Drawing No: 2B

4. The following table gives a description of the soil based on particle sizes. With the exception of those samples where grain-size analyses have been performed, all samples are classified visually. The accuracy of visual examination is not sufficient to differentiate between this classification system or exact grain size.

Soil Classification		Terminology	Proportion
Clay	< 0.002 mm	"trace" (eg. trace sand)	1% - 10%
Silt	0.002 to 0.06 mm	"some" (eg. some sand)	10% - 20%
Sand	0.06 to 2 mm	adjective (eg. sandy)	20% - 35%
Gravel	2 to 60 mm	and (eg. and sand)	> 35%
Cobbles	60 to 200 mm	noun (eg. boulders)	> 35% and main fraction
Boulders	> 200 mm		

Classification system as suggested in the Canadian Foundation Engineering Manual, 3rd Edition, unless otherwise noted.

The compactness of cohesionless soils and the consistency of cohesive soils are defined by the following:

Cohesionless Soil		Cohesive Soil	
Compactness	Standard Penetration Resistance "N" Blows/0.3 m	Consistency	Undrained Shear Strength (kPa)
Very Loose	0 to 4	Very Soft	< 12
Loose	4 to 10	Soft	12 - 25
Compact	10 to 30	Firm	25 - 50
Dense	30 to 50	Stiff	50 - 100
Very Dense	Over 50	Very Stiff	100 - 200
		Hard	> 200

### 5. Rock Coring

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundness of the rock mass. It is obtained from the rock cores by summing the length of core recovered, counting only those pieces of sound core that are 100 mm or more in length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

RQD Classification	RQD
Very poor quality	< 25
Poor quality	25 - 50
Fair quality	50 - 75
Good quality	75 - 90
Excellent quality	90 - 100

$$\text{Recovery Designation \% Recovery} = \frac{\text{Length of Core Per Run}}{\text{Total Length of Run}} \times 100$$



# RECORD OF BOREHOLE BH-1E 1 OF 1

METRIC

W.P. 217-89-00 LOCATION 4 990 505.7 N, 282 337.1 E ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 21, 1998 CHECKED BY I.G.

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		20	40	60	80	wp	w	wl			
219.60	GROUND SURFACE														
0.00	TOPSOIL ~ 100 mm over SAND & GRAVEL, various sized stones with some cobble sizes & possible boulders, brown, moist, some root hairs in upper ~ 1 m depth. (dense to very dense)		1	SS	45										
			2	SS	68										
			3	SS	51										
			4	SS	59										
215.05	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
4.55	Notes: 1) This borehole forms part of the Highway 69, Muskoka Road #12 Interchange, Southbound Lane Foundation Investigation. 2) Borehole drilled at station ~24+608, offset ~6 m right of centreline as referenced to the Southbound Lane. 3) Borehole was dry & open to ~1.0 m depth on completion.														



# RECORD OF BOREHOLE BH-2E 1 OF 1

## METRIC

W.P. 217-89-00 LOCATION 4 990 503.6 N, 282 323.3 E ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / Diamond coring COMPILED BY M.D.  
 DATUM Geodetic DATE August 24, 1998 CHECKED BY I.G.

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	wl		
219.42	GROUND SURFACE															
0.00	TOPSOIL, 75 mm over SAND, brown, moist, some small roots. (compact)		1	SS	12		219									0% 83% 17%
218.12	SAND & GRAVEL, various sized stones with some cobble sizes & possible boulders, brown to 7.5 m depth then grey, wet below 7.5 m depth. (dense)		2	SS	81		218									
1.30			3	SS	42		217									17% 80% 3%
			4	SS	37		216									
			5	SS	52		215									44% 50% 6%
			6	SS	47		214									
			7	SS	38		213									11% 69% 20%
210.58	BIOTITE HORNBLende GNEISS, good to excellent rock quality, slightly weathered to unweathered.		8	BQ			210									Rec. 100% RQD 97%
8.84			9	BQ			209									Rec. 100% RQD 77%
207.53	END OF BOREHOLE						208									
11.89	<b>Notes:</b> 1) This borehole forms part of the Highway 69, Muskoka Road #12 Interchange, Southbound Lane Foundation Investigation. 2) Borehole drilled at station 24+611, offset 7 m left of centreline as referenced to the Southbound Lane.															



# OVERSIZE DRAWING(S)

# RECORD OF BOREHOLE BH-3E 1 OF 1

METRIC

W.P. 217-89-00 LOCATION 4 990 534.2 N, 282 325.5 E ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / Diamond coring COMPILED BY M.D.  
 DATUM Geodetic DATE August 25, 1998 CHECKED BY I.G.

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	wp	w	wl			
221.82	GROUND SURFACE														
0.00	TOPSOIL, ~150 mm over SAND & GRAVEL, brown, moist, small root hair inclusions. (very dense)		1	SS	81										
220.30															
1.52	BIOTITE HORNBLende GNEISS, fair to excellent rock quality, slightly weathered to ~3 m depth then unweathered.		2	BQ											Rec. 98% RQD 52%
			3	BQ											Rec. 100% RQD 90%
217.25	END OF BOREHOLE														
4.57	Notes: 1) This borehole forms part of the Highway 69, Muskoka Road #12 Interchange, Southbound Lane Foundation Investigation. 2) Borehole drilled at station 24+639, offset ~7 m right of centreline as referenced to the Southbound Lane.														



# RECORD OF BOREHOLE BH-4E 1 OF 1

METRIC

W.P. 217-89-00 LOCATION 4 990 533.5 N, 282 310.7 E ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 25, 1998 CHECKED BY I.G.

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			wl	10
220.67	GROUND SURFACE																
0.00	TOPSOIL, ~250 mm over SAND & GRAVEL, some cobbles & possible boulders, brown, moist, some root inclusions to ~1 m depth. (dense)		1	SS	39												
			2	SS	46												
218.43	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER																
2.24	Notes: 1) This borehole forms part of the Highway 69, Muskoka Road #12 Interchange, Southbound Lane Foundation Investigation. 2) Borehole drilled at station ~24+644, offset ~7 m left of centreline as referenced to the Southbound Lane. 3) Borehole was dry & open to 1.3 m depth on completion.																



# RECORD OF BOREHOLE RP-1E 1 OF 1

## METRIC

W.P. 217-89-00 LOCATION 5 990 501.1 N, 282 336.6 E ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 24, 1998 CHECKED BY I.G.

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	20	40
219.35 0.00	GROUND SURFACE  TOPSOIL, ~100 mm over  Probable SAND, GRAVEL & COBBLES																
					219												
					218												
					217												
					216												
214.93 4.42	END OF PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER  Notes: 1) This auger probe forms part of the Highway 69, Muskoka Road #12 Interchange, Southbound Lane Foundation Investigation. 2) Auger probe drilled at station ~24+604, offset ~4 m right of centreline as referenced to the Southbound Lane. 3) Probe hole was dry & open to ~1.9 m depth on completion.				215												



1 OF 1

W.P. 217-89-00

LOCATION 4 990 498.1 N, 282 332.3 E

ORIGINATED BY S.M.

DIST 52 HWY 69

BOREHOLE TYPE Standard augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE August 21, 1998




CHECKED BY I.G.

# RECORD OF BOREHOLE RP-3E

1 OF 1

METRIC

W.P. 217-89-00 LOCATION 4 990 497.2 N, 282 325.7 E ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 21, 1998 CHECKED BY I.G.

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	20	40	60	80	wp			w
219.23	GROUND SURFACE														
0.00	TOPSOIL, ~150 mm over														
	Probable SAND, GRAVEL & COBBLES					219									
						218									
						217									
						216									
						215									
						214									
						213									
					212										
211.31	END OF PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
7.92	Notes: 1) This auger probe forms part of the Highway 69, Muskoka Road #12 Interchange, Southbound Lane Foundation Investigation. 2) Auger probe drilled at station ~24+604, offset ~7 m left of centreline as referenced to the Southbound Lane. 3) Probe hole was dry & open to ~2.2 m depth on completion.														





# RECORD OF BOREHOLE RP-4E

1 OF 1

METRIC

W.P. 217-89-00

LOCATION 4 990 503.2 N, 282 328.7 E

ORIGINATED BY S.M.

DIST 52 HWY 69

BOREHOLE TYPE Standard augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE August 21, 1998

CHECKED BY I.G.

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
219.40	GROUND SURFACE														
0.00	TOPSOIL, ~150 mm over														
	Probable SAND, GRAVEL & COBBLES					219									
						218									
						217									
						216									
						215									
						214									
						213									
						212									
210.87	END OF PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER					211									
8.53	<p>Notes:</p> <p>1) This auger probe forms part of the Highway 69, Muskoka Road #12 Interchange, Southbound Lane Foundation Investigation.</p> <p>2) Auger probe drilled at station ~24+609, offset ~2 m left of centreline as referenced to the Southbound Lane.</p> <p>3) Probe hole was dry &amp; open to ~2.0 m depth on completion.</p>														



## 1 OF 1

ORIGINATED BY S.M.

W.P. 217-89-00

LOCATION 4 990 538.6 N, 282 324.8 E

COMPILED BY M.D.

DIST 52 HWY 69

BOREHOLE TYPE Standard augers / CME-55

CHECKED BY I.G.

DATUM Geodetic

DATE August 25, 1998

# RECORD OF BOREHOLE RP-6E 1 OF 1

METRIC

W.P. 217-89-00 LOCATION 4 990 538.2 N, 282 316.2 E ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 25, 1998 CHECKED BY I.G.

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	w	wl		
221.53 0.00	GROUND SURFACE  TOPSOIL, ~180 mm over  Probable SAND, GRAVEL & COBBLES														GR SA (Si & CL)	
220.26 1.27	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER  Notes: 1) This auger probe forms part of the Highway 69, Muskoka Road #12 Interchange, Southbound Lane Foundation Investigation. 2) Auger probe drilled at station ~24+646, on centreline as referenced to the Southbound Lane. 3) Probe hole was dry & open to full depth on completion.															






# RECORD OF BOREHOLE RP-7E

1 OF 1

METRIC

W.P. 217-89-00 LOCATION 4 990 538.7 N, 282 310.4 E ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 25, 1998 CHECKED BY I.G.

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	20	40	60	80	wp			w
220.94 0.00	GROUND SURFACE														
	TOPSOIL, ~230 mm over														
	Probable SAND, GRAVEL & COBBLES														
218.20 2.74	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
	Notes: 1) This auger probe forms part of the Highway 69, Muskoka Road #12 Interchange, Southbound Lane Foundation Investigation. 2) Auger probe drilled at station 24+649, offset 5 m left of centreline as referenced to the Southbound Lane.														



# RECORD OF BOREHOLE RP-8E 1 OF 1

METRIC

W.P. 217-89-00 LOCATION 4 990 532.4 N, 282 316.6 E ORIGINATED BY S.M.  
 DIST 52 HWY 69 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 25, 1998 CHECKED BY I.G.

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			wl	10	20
221.25 0.00	GROUND SURFACE TOPSOIL, ~200 mm over					221												
	Probable SAND, GRAVEL & COBBLES					220												
219.07 2.18	END OF PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER  Notes: 1) This auger probe forms part of the Highway 69, Muskoka Road #12 Interchange, Southbound Lane Foundation Investigation. 2) Auger probe drilled at station 24+640, offset 2 m left of centreline as referenced to the Southbound Lane. 3) Probe hole was dry & open to full depth on completion.																	



S07657G/E

**TABLE 1-1  
ROCK CORE DESCRIPTION**

BH#	Core Recovery				Core Description	
	RC #	Depth (m)	% CR*	% RQD**	Depth (m)	Description
<b>SOUTH BOUND LANES STRUCTURE AT MUSKOKA ROAD 12 INTERCHANGE FOUNDATIONS</b>						
2-E	8	8.94 to 10.46	100	97	8.94 to 11.99	<b>Biotite Horneblende Gneiss</b> - light grey to pinkish white, medium to coarse grained, unweathered, fractures very widely spaced, dipped at 0 to 45° from vertical, planar, smooth
	9	10.46 to 11.99	100	77		
3-E	2	1.52 to 3.05	100	52	1.52 to 4.57	<b>Biotite Hornblende Gneiss</b> - white to grey-pink, medium to coarse grained, slightly weathered to ~3 m depth, then unweathered, fractures closely spaced, dipping at 45° to 90° from vertical, planar, smooth
	3	3.05 to 4.57	100	90		

\*CR

Core Recovery %

\*\*RQD

Rock Quality Designation %

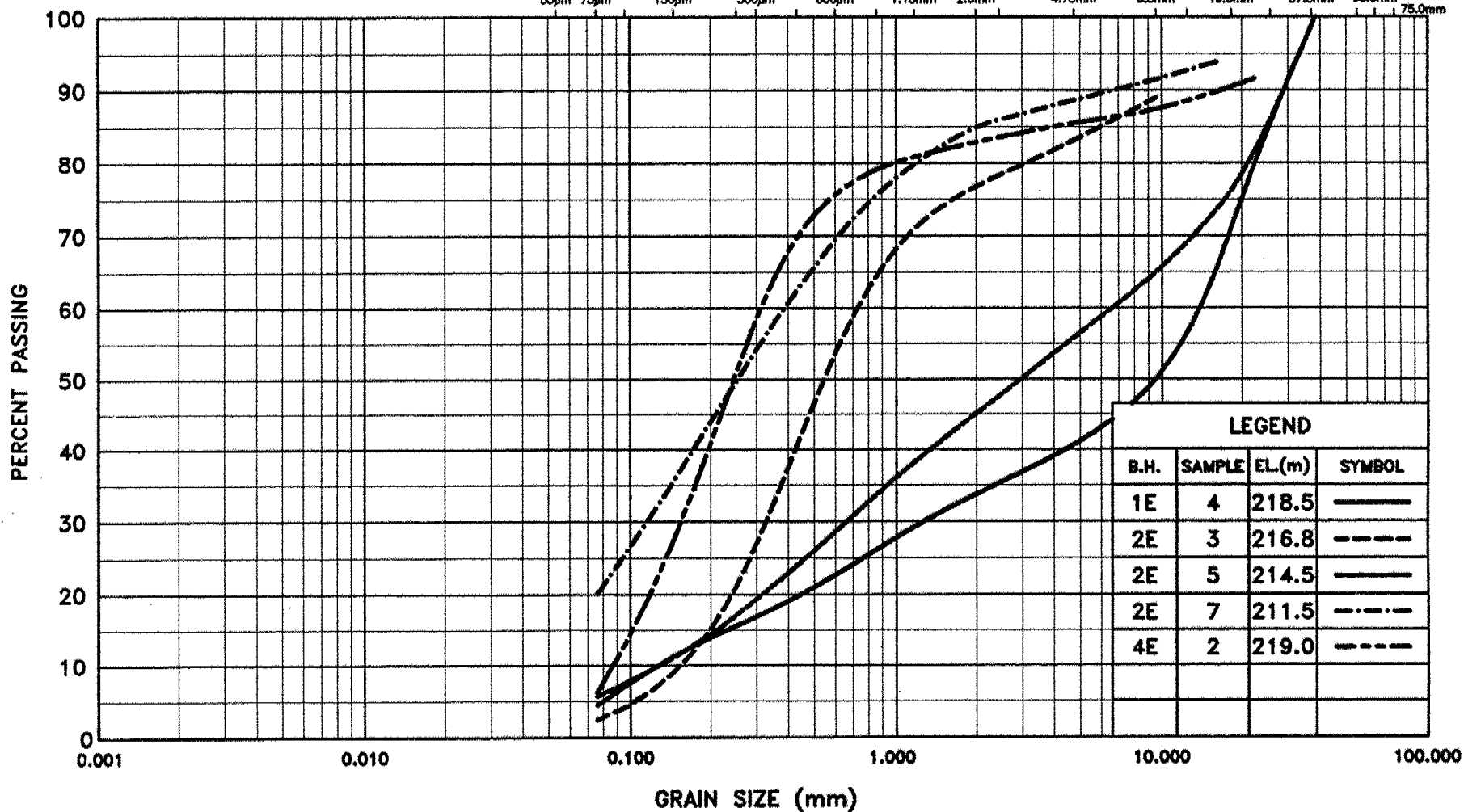
## APPENDIX B

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)

63µm 75µm 106µm 150µm 250µm 300µm 425µm 600µm 850µm 1.18mm 1.18mm 2.0mm 2.36mm 4.75mm 9.5mm 13.2mm 19.0mm 26.5mm 37.5mm 53.0mm 75.0mm



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SAND & GRAVEL

FIGURE 1

W.P 217-89-00



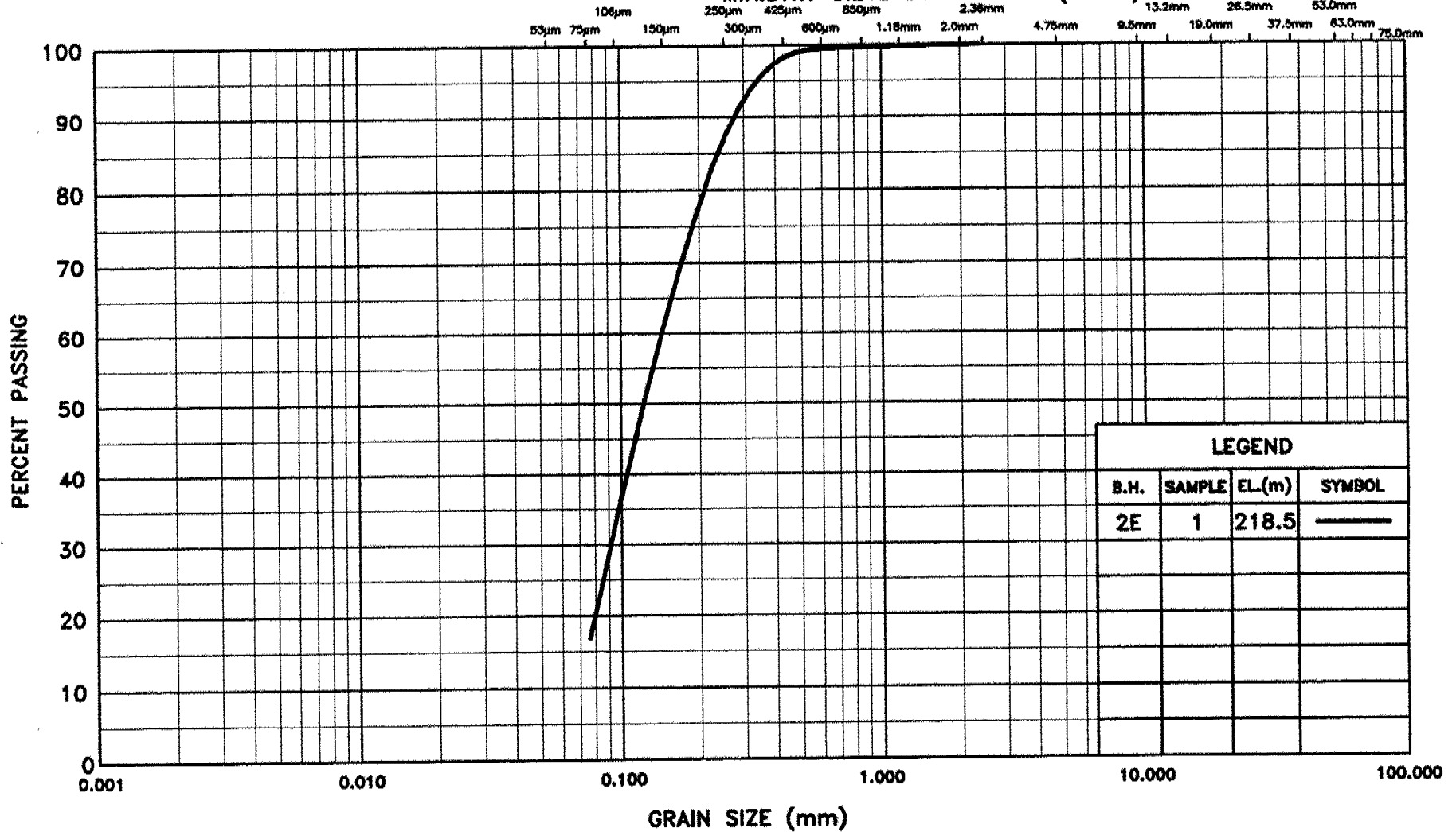
PROJ. No. S07657GE



# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

BH-2E, SS-1

GRAIN SIZE DISTRIBUTION

SAND

FIGURE 2

W.P 217-89-00



PROJ. No. S07657GE



# memorandum

---

To: Mike Pearsall, P. Eng. 1998 11 16  
Senior Project Engineer  
Northern Region

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

Re: Final Foundation Investigation Reports  
Proposed Muskoka Road 12 & Hwy 69 NBL and SBL Bridges  
Site 42-319N and S  
WP 198-98-01 (Northbound Lane)  
WP 199-98-01 (Southbound Lane)  
Hwy 69 Four Laning, from 0.4 km south of the Musquash River, N'ly 8.9km  
GWP 217-89-00, Dist. 52, Huntsville

We have conceptually reviewed the final Foundation Investigation Report 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:

We had made some comments on the draft Foundation report in a memo dated October 19, 1998. We noticed that most of the comments were incorporated in the final Foundation report. We have the following comments:

## Northbound Structure

- Page 7, Para 1: Only the actual bedrock elevation at the borehole locations should be reported. To avoid any contractual claim, the bedrock elevation between the boreholes should not be estimated and reported.

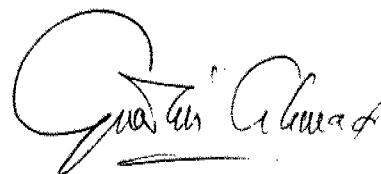
- We understand that all the boreholes remained dry after completion. The note on the groundwater condition appears on some borehole. However, this note should go on all the logs and the drawings.

#### Southbound Structure

- It is not clear from the log that the groundwater was encountered in Borehole BH-2E. If the water level was established in the borehole, then it should be shown on the logs and on the drawing.
- In Section 1.5, Groundwater Level, only the actual water table encountered at the time of the investigation should be reported.
- If it is felt that the piles cannot be driven to the bedrock then it should be clearly recommended to use the Hiley Formula to control the pile driving. This note will then go on the Contract drawings.

The report should be signed and sealed by the MTO designated principal.

If you have any questions, please advise.



K. Ahmad, P. Eng  
Foundation Engineer

For

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

cc: J. McDougall  
T. Kazmierowski



# memorandum

---

To: Mike Pearsall, P. Eng.  
Senior Project Engineer  
Planning and Design Section  
Northern Region

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

Re: General Arrangement Drawings  
Hwy 69, Muskoka Road 12 Crossing  
Northbound and Southbound Structures  
WP 198/199-98-01  
GWP. 217-89-00, Str. Sites: 42-319N/S  
Highway 69, District 52, Huntsville

1998 10 27

We have reviewed the final GA drawing produced by R.V. Anderson Associated Ltd. dated August 1998 for the above projects. Following are our comments:

## Northbound Structure

For the north abutment, consideration should be given to constructing spread footing directly over the bedrock. This may eliminate the need of 1.5H:1V forward slope and result in a rock cut face. If this option is considered then the abutment footing should be located outside an imaginary plane drawn at 1H:1V from the toe of the rock cut slope.

The drawing shows Granular 'B' Type II Modified to be used for the engineered granular pads. MTO standard requires using Granular 'A' for granular pad under the footings.

## Southbound structure

The lower portion of the 1.5H:1V forward slope may be in earth cut. These earth slopes may not be stable at 1.5H:1V.

A note on the drawing suggests placing "loose sand" within the pre-augured holes. The sand should be "Uniformly graded" Ottawa sand or equivalent. The gradation of the sand should meet the following criteria:

NSSP - Backfill to Integral Abutment-Augured Hole

The annular space between the pre-augured oversize hole and the pile shall be backfilled with uniformly graded sand. The gradation for the uniformly graded sand shall be as follows:

<b>MTO SIEVE DESIGNATION</b>	<b>PERCENTAGE PASSING BY MASS</b>
<b>2 mm (#10)</b>	<b>100</b>
<b>600 <math>\mu\text{m}</math> (#30)</b>	<b>80 - 100</b>
<b>425 <math>\mu\text{m}</math> (#40)</b>	<b>40 - 80</b>
<b>250 <math>\mu\text{m}</math> (#60)</b>	<b>5 - 25</b>
<b>150 <math>\mu\text{m}</math> (#100)</b>	<b>0 -6</b>

Alternatively, commercially available material which meets the above gradation may be considered instead of Ottawa sand.

If you have any questions please advise.



K.S.Q. Ahmad, P. Eng.  
Foundation Engineer

For

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

cc: P. Stuart  
T. Kazmierowski



Northern Region  
447 McKeown Avenue  
North Bay, Ontario  
P1B 8L2  
Tel: (705) 497-5526  
Fax: (705) 497-5509

e mail: [stuartpe@mto.gov.on.ca](mailto:stuartpe@mto.gov.on.ca)

October 23, 1998

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

**ATTENTION: H. Vierhuis, P. Eng.**

Dear Sir:

**RE: Preliminary General Arrangement Drawings  
Muskoka Road 12  
Site 42-319N/S, WP 198/9-98-01  
Highway 69**

The road section under these bridges has not been finalized. The following comments are therefore preliminary in nature.

1. Please rotate the plan so that bridge is horizontal. In this way a section projected below it will show the actual span dimensions.
2. The lower portion of some of the 1.5 to 1 slopes under and adjacent to the bridges are in earth cut, probably below the water table in the spring. Please review the stability of these slopes with your Foundation Engineer.
3. Please review the use of Granular "B" under the abutment footings with your Foundation Engineer.
4. It is my understanding that the design will be modified to be semi-integral with the ballast wall cantilevered from the deck and expansion permitted at the abutment bearings.
5. Extend the wingwalls 0.5 m past the top of slope in section 1.
6. Use higher strength concrete in the girders to optimize their design. All other concrete except the footings will be 50 Mpa (HPC).

7. Additional comments are provided on the attached drawing.

Sincerely,

Peter J. Stuart, P. Eng.,  
Senior Structural Engineer,  
Northern Region



# MEMORANDUM

## Engineering Materials Office

Room 233, Central Building, Downsview  
Tel. (416) 235-3732 Fax. (416) 235-5240

To: M. E. Pearsall, P. Eng.  
Sr. Project Engineer, P & D Section  
Northern Region, North Bay

Date: October 14, 1998

From: Pavements and Foundations Section  
Room 223, Central Building

Re: North Bound Lane Structure at Muskoka Road 12  
G. W. P - 217 - 89 - 00; Site No. 42 - 319N  
Highway 69, District 52, Huntsville

W.P. 198-98-01

We have conceptually reviewed the draft Foundation Investigation report submitted by Trow Consulting Engineers Ltd. to decide the performance of the Consultant in providing the deliverables. The assessment of the Consultant in this assignment is required by MTO for similar scope of work in the future. The accuracy of the subsurface information presented in the report and the adequacy of the technical aspects of the recommendations remains the responsibility and liability of the Consultant. This Ministry assumes no responsibility or liability for these aspects of the report.

The report in general meets the terms of reference that would be required for similar type of foundation design and construction. However, our review comments are as follows:

1. The soil strata in probe holes are described as assumed, instead, it should be called as probable to avoid contractual implications.
2. Description of the subsurface under bedrock should include inferred bedrock elevations and any appreciable variations in elevations.
3. Borehole logs should clearly show whether the hole was dry or groundwater was stabilized on completion. This information should also be included in the drawing that will be part of the contract document.
4. It may be advisable to show only the depth at which (elevation) groundwater was encountered rather than estimated depth.
5. The recommendations for the spread footings on bedrock should include the founding

Sam. as SBL.

X. Geotechnical presentation at U.L.S.  
S.L.S.  
\* General site layout (Key plan)



elevation.

6. The report suggests that the information provided is based on a limited investigation. However, the terms of reference clearly say that the site investigation shall be of sufficient scope to verify the design assumptions, provide adequate subsurface descriptions to allow the Consortium/Contractor to plan construction of foundation elements and immediate approaches.

If you have any question or clarification, please contact this office.

c.c: P. Furst  
I. Husain



M. Vasavithasan, P. Eng.  
Foundation Engineer  
for  
Tae C. Kim, P.Eng.  
Sr. Foundation Engineer



# MEMORANDUM

## Engineering Materials Office

Room 233, Central Building, Downsview  
Tel. (416) 235-3732 Fax. (416) 235-5240

To: M. E. Pearsall, P. Eng.  
Sr. Project Engineer, P & D Section  
Northern Region, North Bay

Date: October 14, 1998

From: Pavements and Foundations Section  
Room 223, Central Building

Re: South Bound Lane Structure at Muskoka Road 12  
G. W. P - 217 - 89 - 00; Site No. 42 - 319S  
Highway 69, District 52, Huntsville

W.P. 199-98-01

We have conceptually reviewed the draft Foundation Investigation report submitted by Trow Consulting Engineers Ltd. to decide the performance of the Consultant in providing the deliverables. The assessment of the Consultant in this assignment is required by MTO for similar scope of work in the future. The accuracy of the subsurface information presented in the report and the adequacy of the technical aspects of the recommendations remains the responsibility and liability of the Consultant. This Ministry assumes no responsibility or liability for these aspects of the report.

The report in general meets the terms of reference that would be required for similar type of foundation design and construction. However, our review comments are as follows:

1. The soil strata in probe holes are described as assumed, instead, it should be called as probable to avoid contractual implications.
2. Description of the subsurface under bedrock should include inferred bedrock elevations and any appreciable variations in elevations.
3. Borehole logs should clearly show whether the hole was dry or groundwater was stabilized on completion. This information should also be included in the drawing that will be part of the contract document.
4. It may be advisable to show only the depth at which (elevation) groundwater was encountered rather than estimated depth.
5. The Geotechnical Consultant should decide the feasible and practical options for the design

*1. Geotechnical Resistance at ULS  
SL5  
2. General site layout on plan (Key plan)*

and construction of the foundation based on subsoil information, constructability and economy rather than recommending all the possible options.

6. The recommendations for the spread footings on bedrock should include the founding elevation.

7. The recommendations for spread footings on dense sand and gravel should include the founding elevation, size of footing and anticipated settlement under the SLS load.

8. Geotechnical Consultant should decide whether it is feasible to drive the pile to bedrock. If the pile cannot be driven to bedrock, the probable tip elevation of the pile should be provided.

9. The report suggests that the information provided is based on a limited investigation. However, the terms of reference clearly say that the site investigation shall be of sufficient scope to verify the design assumptions, provide adequate subsurface descriptions to allow the Consortium/Contractor to plan construction of foundation elements and immediate approaches.

If you have any question or clarification, please contact this office.

c.c: P. Furst  
I. Husain



M. Vasavithasan, P. Eng.  
Foundation Engineer  
for  
Tae C. Kim, P.Eng.  
Sr. Foundation Engineer



# memorandum

---

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

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

Re: Proposal for additional Foundation Work  
Single span to Three Span Revision  
Northbound Structure at Muskoka Road 12 Interchange  
WP 198-98-01, Structure Site 42-319N  
GWP 217-89-00, Dist. 52, Huntsville

1998 09 25

We have briefly reviewed Trow's proposal (Table 2, Page 5) for the additional Foundation investigation work at the Hwy 69 and MR 12, interchange. We understand, that the additional work is required due to the change in bridge design. The original scope of work was based on a single span bridge (two foundation elements). Since the design has now changed to a three-span structure, additional foundation investigation is needed.

Based on the proposed 12 boreholes (two cored boreholes and ten boreholes without the core), the total estimate of \$ 14,550 for the additional foundation investigation and design work appears to be reasonable. However, in our opinion the proposed number of boreholes for the investigation at two additional foundation elements is high.

If you have any further questions, please advise.

A handwritten signature in black ink, appearing to read "K. Ahmad".

K. Ahmad, P. Eng  
Foundation Engineer

For

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

cc: T. Kazmierowski

G.I.-30 SEPT. 1976

GEOCRES No. 31E-115

DIST. 11 REGION

W.P. No. 217-89-00(A)

CONT. No.

W. O. No.

STR. SITE No.

HWY. No. 69 New

LOCATION Embankment for Swamp  
Crossings Near Muskegon Rd. 5  
No. of PAGES - (Side 2)

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976

GEOCRES No. 31E-115DIST. 11 REGION W.P. No. 217-89-00(A)CONT. No. W. O. No. STR. SITE No. HWY. No. 69 New

LOCATION Embankment for Swamp  
Crossings Near Muskegon Rd. 5  
No. of PAGES - (Side 2)

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

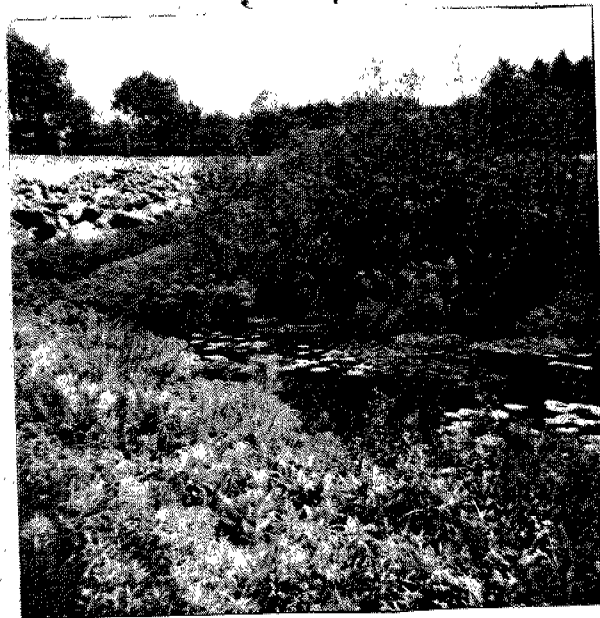


Ministry  
of  
Transportation

**FILE No.** \_\_\_\_\_ **DATE** \_\_\_\_\_

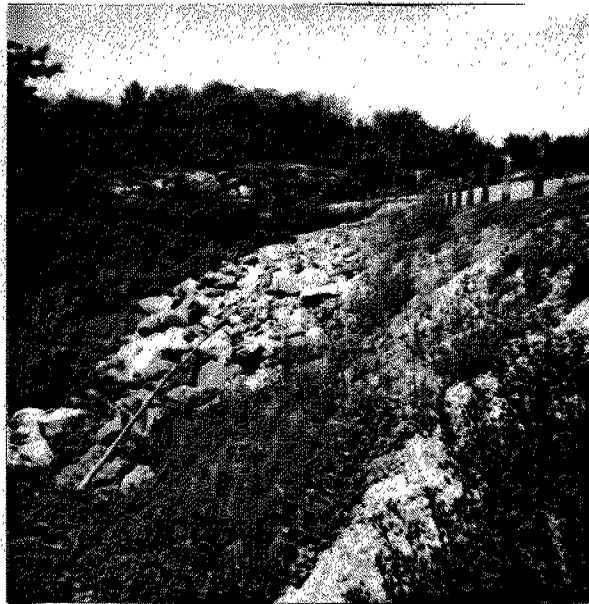
**REMARKS** \_\_\_\_\_

25-50  
89-174



Stern County <sup>southeast</sup> ~~east~~  
River to be crossed

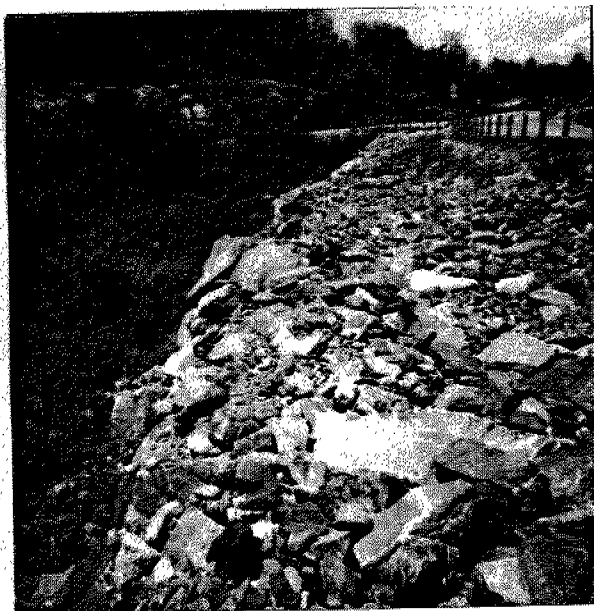




Site 2 looking South  
Showing Rockfill berm which  
was added



Site 2 - looking South  
at east side -  
rockfill berm added.



Site zu Rodeln! Beim  
Locking-Sank



Site 2 - Access to Barn,  
Latham South



**DOMINION SOIL**  
CONSULTING ENGINEERS

DOMINION SOIL INVESTIGATION INC.  
104 CROCKFORD BOULEVARD  
SCARBOROUGH, ONTARIO M1R 3C6  
TEL.: (416) 751-6565  
FAX: (416) 751-7592

March 10, 1992.

**Ref. No. 91-9-5**



Ministry of Transportation of Ontario  
P.O. Box 3030  
447 McKeown Avenue  
North Bay, Ontario  
P1B 8L2

**Attn: Mr. Ron Purdy, P. Eng. - Soils Supervisor West**

**Re: Soil Investigation, Highway 69 New from  
Approx. 15.3 km North of M.R. #5 Northerly 13.0 km  
W.P. 215-89-00**

Dear Sir:

Further to our letter dated March 5, 1992, we are forwarding to you the complete borehole data available within the following sections where, in our opinion, Foundation Investigations are required.

1. - SBL Stations 11+725 to 11+925
2. - SBL Stations 15+000 to 15+030
3. - SBL Stations 17+150 to 17+335
4. - SBL Stations 17+950 to 18+180
5. - SBL Stations 20+060 to 20+360 (Musquash River)
6. - SBL and NBL Stations 22+910 to 23+000

We are also forwarding composite borehole logs immediately beyond the stations listed above, for your information.

.../...

We wish to point out that between Stations 22+580 and 22+650 SBL, a layer of weak clay was encountered. With the available information, we do not foresee instability problems with the embankment which will be constructed here. We feel, however, that it would be prudent to drill additional borehole(s) to confirm this conclusion. Furthermore, as was discussed on our previous letter of January 8, 1992, we were unable to drill boreholes between Stations 9+720 and 9+820 Muskoka Road #32, within our terms of assignment. We recommend however, that this section should be investigated.

We trust that this information is complete for your present requirements. Should you have any questions regarding this letter, please do not hesitate to call this office.

Yours very truly,  
DOMINION SOIL INVESTIGATION INC.



R. Miranda, P. Eng.

RM/dml  
Encl.



# LOG OF BOREHOLE 11+800 C/L SBL

ENCL. No.: 1

REF. No.: 91-9-5	DRILLING DATA
CLIENT: Ministry of Transportation	
PROJECT NAME: Twinning of HWY 69, W.P. 215-89-00	Method: Hand drill
LOCATION: District #11, Huntsville, Ontario	Diameter:
DATUM:	Date: October 22nd, 1991

LABORATORY DATA						SAMPLES			SYMBOL	MATERIAL DESCRIPTION	ELEV. m	DEPTH m	WATER DATA	REMARKS
PL %	NMC %	LL %	UNIT WT kN/m <sup>3</sup>	UNDR Field Vane	STRNG Lab. Comp kPa	No.	TYPE	N <sub>60</sub> Value						
						1	AS			GROUND SURFACE m				
										TOPSOIL 0.2m				
						2	SS	2		grey, v.soft SILTY CLAY		1		W.L. @ 0.4 m upon completion
						3	SS	1				2		
						4	SS	2		black MUCK		3		
						5	SS	<1				4		SS:5 split spoon sampler sunk from 3.0 to 3.8 m by weight of hammer
						6	SS	1*		grey, v.soft SILTY CLAY wet		5		* spoon sampler was driven from 3.8 to 7.8 m where "firm" bottom was encountered and average 'N'-value is 1 blow/0.3 m
												6		
												7		
										END OF BOREHOLE				

Vertical Scale: 1:50



**DOMINION SOIL INVESTIGATION INC.**

Checked: R.M.

SHEET 1 OF 1 HOLE No. 11+800

# LOG OF BOREHOLE 11+900 C/L SBL

ENCL. No.: 2

REF. No.: 91-9-5	DRILLING DATA
CLIENT: Ministry of Transportation	
PROJECT NAME: Twinning of HWY 69, W.P. 215-89-00	Method: Hand drill
LOCATION District #11, Huntsville, Ontario	Diameter:
DATUM:	Date: November 5th, 1991

LABORATORY DATA						SAMPLES			SYMBOL	MATERIAL DESCRIPTION	ELEV. m	DEPTH m	CORRECTION m	REMARKS
PL	NMC	LL	WT	UNIT	UNDR STRNG	Field	Lab.	Moisture						
%	%	%	KN/m3		Vane	Compn		Z <sub>g</sub>	TYPE	Z <sub>g</sub>				
					kPa	kPa								
GROUND SURFACE m														
		119						1	SS	<1				
								2	SS	4				
39	40	68						3	SS	3				
		24						4	SS	1				
25	50	45						5	SS	<1				
PEAT														
grey, soft to v. soft SILTY CLAY interbedded sand seams & peat layers														
grey, v. soft SILTY CLAY with sand seams														
END OF BOREHOLE														
SS:4 split spoon was overdriven														
SS:5 split spoon was pushed manually from 3.0 to 7.2 m and no "firm" bottom was encountered														

Vertical Scale: 1:50



**DOMINION SOIL INVESTIGATION INC.**

Checked: R.M.

SHEET 1 OF 1 HOLE No. 11+900



C/L SBL

ENCL. No.: 3

REF. No.:	91-9-5	DRILLING DATA
CLIENT:	Ministry of Transportation	
PROJECT NAME:	Twining of HWY 69, W.P. 215-89-00	Method: Hand drill
LOCATION	District #11, Huntsville, Ontario	Diameter:
DATUM:		Date: February 27th, 1992

LABORATORY DATA						SAMPLES				SYMBOL	MATERIAL DESCRIPTION	ELEV.	DEPTH	WATER DATA	REMARKS		
PL	NMC	LL	UNIT	UNDR	STRNG	No.	TYPE	Z <sub>10</sub>	Z <sub>100</sub>			m	m	34°C			
%	%	%	WT	Field	Lab.							kN/m <sup>3</sup>	Vane	Compr		kPa	kPa
GROUND SURFACE m																	
						1	SS	1			TOPSOIL 0.3 m						
						2	SS	2			brown SILTY CLAY wet  soft to firm  ----- stiff					1	W.L. @ surface
						3	SS	3								2	
			27													3	
			22													4	
														5			
			62											6			
			80														
			88			4	SS	7			sand seams						
END OF BOREHOLE																	

Vertical Scale: 1:50



**DOMINION SOIL INVESTIGATION INC.**

Checked: **R.M.**

SHEET 1 OF 1 HOLE No. 15+020

**ENCL. No. : 4**

REF. No.:	91-9-5	DRILLING DATA
CLIENT:	Ministry of Transportation	
PROJECT NAME:	Twining of HWY 69, W.P. 215-89-00	Method: Hand drill
LOCATION	District #11, Huntsville, Ontario	Diameter:
DATUM:		Date: November 4th, 1991

LABORATORY DATA						SAMPLES			SYMBOL	MATERIAL DESCRIPTION	ELEV.	DEPTH	WATER DATA	REMARKS
PL %	NMC %	LL %	UNIT WT kN/m3	UNDR Field Vane kPa	STRNG Lab. Compr kPa	No.	TYPE	TEST VALUE			m	m		
GROUND SURFACE										m				
32						1	SS	<1		PEAT			W.L. @ surface upon completion	
27						2	SS	1				1		
60						3	SS	1		dark brown, v.soft SILTY CLAY with peat layers & sand seams, wet		2		
						4	SS	1						
29						5	SS	<1				3		
						6	SS	<1		grey, v.soft CLAYEY SILT trace of sand wet		4		
						7	SS	20/0cm*		END OF BOREHOLE (possible bedrock)				SS:6 split spoon sunk from 3.0 to 4.2 m by weight of hammer  * hammer bouncing @ 4.2 m depth

Vertical Scale: 1:50



**DOMINION SOIL INVESTIGATION INC.**

Checked: **R.M.**

SHEET 1 OF 1 HOLE No. 17+170

# LOG OF BOREHOLE 18+020 C/L SBL

ENCL. No.: 5

REF. No.: 91-9-5	DRILLING DATA
CLIENT: Ministry of Transportation	
PROJECT NAME: Twinning of HWY 69, W.P. 215-89-00	Method: Hand drill
LOCATION: District #11, Huntsville, Ontario	Diameter:
DATUM:	Date: November 28th, 1991

LABORATORY DATA						SAMPLES			SYMBOL	MATERIAL DESCRIPTION	ELEV. m	DEPTH m	WATER DATA	REMARKS
PL %	NMC %	LL %	WT kN/m <sup>3</sup>	Field Vane	Lab. Compr	No. Z <sub>0</sub>	TYPE	NUM Z <sub>0</sub>						
				kPa	kPa									
						1	SS	<1		GROUND SURFACE m				
						2	SS	<1		TOPSOIL 1.2m				W.L. @ 0.1 m above ground surface
16	27	23				3	SS	2		grey, v.soft to soft SILTY CLAY with silt seams				
						4	SS	<1						
						5	SS	20/0cm		END OF BOREHOLE (possible bedrock)				SS:5 split spoon bouncing @ 4.1 m depth

Vertical Scale: 1:50



**DOMINION SOIL INVESTIGATION INC.**

Checked: R.M.

SHEET 1 OF 1 HOLE No. 18+020



# LOG OF BOREHOLE 18+090 C/L SBL

ENCL. No.: 7

REF. No.: 91-9-5	DRILLING DATA
CLIENT: Ministry of Transportation	
PROJECT NAME: Twinning of HWY 69, W.P. 215-89-00	Method: Hand drill
LOCATION: District #11, Huntsville, Ontario	Diameter:
DATUM:	Date: February 26th, 1992

LABORATORY DATA						SAMPLES			SYMBOL	MATERIAL DESCRIPTION	ELEV. m	DEPTH m	WATER CONTENT %	REMARKS
PL %	NMC %	LL %	UNIT WT kN/m <sup>3</sup>	UNDR Field Vane	STRNG Lab. Compr kPa	No.	TYPE	Value						
										GROUND SURFACE m				
						1	SS	<1		black MUCK				
						2	SS	<1				1		
						3	SS	1				2		
										grey, soft to firm SILTY CLAY wet		3		
						4	SS30/15cm			END OF BOREHOLE (possible bedrock)				SS:4 spoon bouncing @ 3.8 m

Vertical Scale: 1:50



**DOMINION SOIL INVESTIGATION INC.**

Checked: R.M.

SHEET 1 OF 1 HOLE No. 18+090

# LOG OF BOREHOLE 18+120 C/L SBL

ENCL. No.: 8

REF. No.: 91-9-5	DRILLING DATA
CLIENT: Ministry of Transportation	
PROJECT NAME: Twinning of HWY 69, W.P. 215-89-00	Method: Hand drill
LOCATION District #11, Huntsville, Ontario	Diameter:
DATUM:	Date: November 28th, 1991

LABORATORY DATA										SAMPLES			SYMBOL	MATERIAL DESCRIPTION	ELEV. m	DEPTH m	WATER DATA	REMARKS
PL	NMC	LL	UNIT	UNDR	STRNG	No.	TYPE	Z Value										
%	%	%	WT	Field	Lab.													
			kN/m <sup>3</sup>	Vane	Compr	kPa	kPa											
GROUND SURFACE    m																		
28						1	SS	<1	TOPSOIL 0.7m						1	W.L. @ 0.2 m above ground surface		
						2	SS	4	grey, firm SILTY CLAY wet									
						3	SS	5										
						4	SS	10/0cm	END OF BOREHOLE (possible bedrock)									
															2	SS:4 spoon bouncing @ 2.0 m		

Vertical Scale: 1:50



**DOMINION SOIL INVESTIGATION INC.**

Checked: R.M.

SHEET 1 OF 1 HOLE No. 18+120

ENCL. No.: 9

REF. No.:	91-9-5	DRILLING DATA
CLIENT:	Ministry of Transportation	
PROJECT NAME:	Twinning of HWY 69, W.P. 215-89-00	Method: Hand drill
LOCATION	District #11, Huntsville, Ontario	Diameter:
DATUM:		Date: February 26th, 1992

[illegible]

# LOG OF BOREHOLE 20+100 C/L SBL

ENCL. No.: 10

REF. No.: 91-9-5	DRILLING DATA
CLIENT: Ministry of Transportation	
PROJECT NAME: Twinning of HWY 69, W.P. 215-89-00	Method: Hand drill
LOCATION: District #11, Huntsville, Ontario	Diameter:
DATUM:	Date: November 9th, 1991

LABORATORY DATA						SAMPLES			SYMBOL	MATERIAL DESCRIPTION	ELEV. m	DEPTH m	WATER DATA	REMARKS
PL	NMC	LL	WT	Field	Lab.	Mo.	TYPE	Mo.						
%	%	%	KN/m3	Vane	Compr									
						kPa	kPa							
GROUND SURFACE m														
						1	SS	<1		black MUCK				
						2	SS	7		grey, v. loose, SANDY SILT, moist				
19										grey, loose				
										SILTY FINE SAND				
										trace of gravel		1		
22						3	SS	3						
						4	SS	<1		grey, v. loose to loose				
										SANDY SILT		2		
										with silty clay seams				
												3		
						5	SS	8		----- silty clay layer -----		4		
										END OF BOREHOLE				

split spoon sampler  
sunk from 1.8 m to  
4.0 m by weight of  
hammer

Vertical Scale: 1:50



**DOMINION SOIL INVESTIGATION INC.**

Checked: R.M.

SHEET 1 OF 1 HOLE No. 20+100



# LOG OF BOREHOLE 20+350 C/L SBL

ENCL. No.: 11

REF. No.: 91-9-5	DRILLING DATA
CLIENT: Ministry of Transportation	
PROJECT NAME: Twinning of HWY 69, W.P. 215-89-00	Method: Hand drill
LOCATION: District #11, Huntsville, Ontario	Diameter:
DATUM:	Date: November 9th, 1991

LABORATORY DATA						SAMPLES			SYMBOL	MATERIAL DESCRIPTION	ELEV. m	DEPTH m	WATER DATA	REMARKS
PL	NMC	LL	WT	Field	STRNG	No.	TYPE	Value						
%	%	%	kN/m <sup>3</sup>	Vane	Compr									
				kPa	kPa									
GROUND SURFACE m														
23						1	SS	4		TOPSOIL 0.2m				
16						2	SS	15		brown, loose SILTY SAND - SANDY SILT moist				
						3	SS	5		brown, compact to loose SANDY SILT with silty clay seams moist to wet		1		
18	35	29				4	SS	3		grey, v. soft to soft SILTY CLAY with silt pockets wet		2		
						5	SS	3		silt pocket		3		
						6	SS	<1				4		split spoon sunk from 3.0 m to 4.9 m by weight of hammer and no "firm" bottom encountered
										END OF BOREHOLE				

Vertical Scale: 1:50



**DOMINION SOIL INVESTIGATION INC.**

Checked: R.M.

SHEET 1 OF 1 HOLE No. 20+350

ENCL. No.: 12

REF. No.:	91-9-5	DRILLING DATA
CLIENT:	Ministry of Transportation	
PROJECT NAME:	Twining of HWY 69, W.P. 215-89-00	Method: SST
LOCATION	District #11, Huntsville, Ontario	Diameter:
DATUM:		Date: February 4th, 1992

LABORATORY DATA				SAMPLES			MATERIAL DESCRIPTION		ELEV.	DEPTH	WATER DATA	REMARKS	
PL	NMC	LL	UNIT	UNDR	STRNG	No.	TYPE	Z-Value	SYMBOL	m	m		
%	%	%	WT	Field	Lab.								
			kN/m3			kPa	kPa	GROUND SURFACE m					
						1	SS	5		TOPSOIL 0.25 m			
						2	SS	9		brown, loose SILTY FINE SAND wet		1	W.L. @ surface
						3	SS	6		brown, loose SANDY SILT with silty clay seams		2	
										grey, firm to stiff SILTY CLAY		3	
										END OF BOREHOLE			

Vertical Scale: 1:50



**DOMINION SOIL INVESTIGATION INC.**

Checked: **R.M.**

SHEET 1 OF 1 HOLE No. 20+520



ENCL. No.: 14

REF. No.:	91-9-5	DRILLING DATA
CLIENT:	Ministry of Transportation	
PROJECT NAME:	Twining of HWY 69, W.P. 215-89-00	Method: Hand drill
LOCATION	District #11, Huntsville, Ontario	Diameter:
DATUM:		Date: November 29th, 1991

LABORATORY DATA						SAMPLES			SYMBOL	MATERIAL DESCRIPTION	ELEV.	DEPTH	WATER DATA	REMARKS
PL	NMC	LL	UNIT	UNDR	STRNG						m	m		
%	%	%	WT	Field	Lab.	No.	TYPE	Value						
			kN/m <sup>3</sup>	Vane	Compr									
				kPa	kPa									
						1	SS	3		TOPSOIL 0.3m				
						2	SS	15		brown, compact SANDY SILT wet		1		MUSKOKA RD. 32
						3	SS	12						
						4	SS	7		brown, firm SILTY CLAY		2		W. L. @ 0.1 m above ground surface
										END OF BOREHOLE				

Vertical Scale: 1:50

**DOMINION SOIL INVESTIGATION INC.**

Checked: **R.M.**

SHEET 1 OF 1 HOLE No. 9+720

Page No. 15

Date: \_\_\_\_\_

Twp: Gibson

W.P. No.: 215-89-00

11+640 C/L

0 - 150  
150 - 600  
600 -

Tps  
Br SiSa some Gr  
NFP (possible Blds)

91MG16 w@400 = 11.4%

11+660 C/L

0 - 50  
50 -

Tps  
BR

11+660 6.0 LT C/L

D-200

0 - Blds

11+660 6.0 RT C/L

D-1.6

0 - 200  
200 - 700  
700 -

Tps  
Br SiSa-SaSi Wet  
NFP (possible BR)

91MG17 w@400 = 39.2%

11+680 C/L

0 - Blds

11+700 C/L

0 - Blds

Hwy. No.: 69

SOUTHBOUND LANES

Engineer: M. Ghinani

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Date: \_\_\_\_\_

Twp: Gibson

W.P. No.: 215-89-00

11+720 C/L

0 - Blds

11+750 C/L

0 - 450	Muck	<u>91MG18</u>
450 - 700	Grey Si Cl Wet	<u>91MG19</u>
700 - 1.3	Muck	<u>91MG20</u>
	WT @ surface	

11+800 C/L

0 - 200	Tps	<u>91MG21</u>
200 - 1.75	Grey Si Cl very soft Wet	<u>91MG22</u>
1.75 - 4.6	Blk Muck	<u>91MG23, 24 and 25</u>
4.6 - 7.8	Grey Si Cl very soft, Wet	<u>91MG26</u>

11+850 C/L

0 - 200	Tps	
200 - 800	Br Sa some Gr Tr Si Damp	<u>91MG27 w@500 = 8.4%</u>

Hwy. No.: 69

SOUTHBOUND LANES

Engineer: M. Ghinani

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Date: \_\_\_\_\_

Twp: Gibson

W.P. No.: 215-89-00

11+900 C/L

0 - 500  
500 - 3.0

Peat 91MG233 w@300 = 118.5%  
Grey Si Cl interbedded 91MG235 w@1.5 = 23.5%  
Sa seams and Peat Layers 91MG234 w@900 = 40.1%  
soft WL = 67.8%  
WP = 38.7%

IP = 29.1 OH  
91MG236 w@2.1 = 49.8%  
WL = 45.2%  
WP = 25.3%  
IP = 19.9 CI

3.0 -7.2

Grey Si Cl with Sa seams 91MG237  
very soft

11+950 C/L

0 - 200  
200 - 750  
750 - 1.1

Tps  
Br Si Cl with Sa seams  
Tr Gr Firm Moist  
Br Sa some Gr Tr Si  
Dense  
WT @ 600

91MG29 w@600 = 27.4%

91MG30

11+990 C/L

0 - 200  
200 - 700  
700 -

Tps  
Br FSa Tr Si and Gr  
Moist  
NFP (possible BR)

91MG31

Hwy. No.: 69

SOUTHBOUND LANES

Engineer: M. Ghinani

**Date:** \_\_\_\_\_

W.P. No.: 215-89-00

0 - 200	Tps
200 - 850	Br Fsa Tr Gr and Si Moist
850 -	NFP (possible BR)

0 - 170	Tps
170 - 1.0	Br FSa Tr Gr and Si
	Moist
1.0 -	NFP (possible BR)

0 - 200 Tps  
200 - 350 Br Fsa Tr Gr and Si, Damp  
350 - NFP (possible BR)

0 - 200	Tps	
200 - 400	Dk Br FSa Tr Gr Si and	
	Cob Damp	<u>91MG32</u>
400 -	NFP (possible BR)	

0 - 150            Tps  
150 - 300        Br Sa some Gr Damp        91MG33 w@200 = 5.9%  
300 -            NFP BR

Engineer: M. Ghinani



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Date: \_\_\_\_\_

Twp: Gibson

W.P. No.: 215-89-00

14+770      8.0 LT C/L                          D+500

0 - 50	Tps
50 -	NFP BR

14+770      8.0 RT C/L      D+3.2

0 - 250	Tps
250 -	NFP BR

14+790 C/L

0 - 200	Tps
200 -	NFP BR

14+810 C/L

0 - 200	Tps	
200 - 400	Br SaSi-SiSa Tr Gr Damp	<u>91MG96</u>
400 -	NFP (possible BR)	

14+830 C/L

0 - 300 Tps  
300 - NFP BR

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SOUTHBOUND LANES

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Date: \_\_\_\_\_

W.P. No.: 215-89-00

0 - 300	Tps
300 -	NFP BR

0 - 300	Tps
300 -	NFP BR

0 - 200	Tps
200 - 800	Br SaSi Wet
800 -	NFP (possible BR)

91MG97 w@500 = 32.6%

0 - 300	Tps	
300 - 700	Br SaSi Wet	<u>91MG98</u>
700 - 750	Br Cl Si Tr Sa Wet	<u>91MG99</u> w@725 = 19.0%
750 -	NFP (Refusal to hand auger)	

91MG98

91MG99 w@725 = 19.0%

0 - 200	Tps	
200 - 700	Br SaSi-SiSa Moist	
700 - 1.2	Br Si Cl Moist	<u>92MG142</u>
1.2 -	NFP (Refusal to hand auger - possible BR)	

92MG142

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Date: \_\_\_\_\_

Twp: Gibson

W.P. No.: 215-89-00

14+950 C/L

0 - 100 Tps  
100 - NFP BR

14+950 18.0 LT C/L

D-2.5

0 - 200 Tps  
200 - 600 Br Cl Si soft, Damp 91MG225  
600 - 2.4 Br Si Cl stiff, Damp 91MG226 w@900 = 25.9%  
91MG227 w@1.5 = 32.7%  
2.4 - 2.6 Br SiSa Tr Gr very Dense  
Wet 91MG229

14+950 10.0 RT C/L

D+1.6

0 - 100 Tps  
100 - NFP BR

14+980 C/L

0 - 150 Tps  
150 - 500 Br SiSa Damp  
500 - NFP (possible Blds)

92MG143

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SOUTHBOUND LANES

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Twp: Gibson

W.P. No.: 215-89-00

15+000 C/L

0 - 200  
200 - 2.2

Tps  
Br Si Cl with Si and Sa seams  
Stiff below 600  
Wet below 1.8

91MG221 w@300 = 25.0%  
WL = 37.7%  
WP = 17.8%  
IP = 19.9 CI  
91MG222 w@900 = 30.7%  
91MG223 w@1.5 = 37.5%  
WL = 58.8%  
WP = 27.7%  
IP = 31.1 CH  
91MG224 w@2.0 = 40.5%

15+020 C/L

0 - 300  
300 - 6.4

Muck  
Br Si Cl Wet Soft-very Soft  
with Sa seams 92MG236, 237, 238  
Cu @ 2.4 = 27 kPa, Cu @ 3.0 = 22  
Cu @ 4.3 = 62, Cu @ 4.9 = 80, Cu @ 5.5 = 88

15+050 C/L

0 - 100  
100 - 1.0  
1.0 - 1.2

Tps  
Br Si Cl Firm Moist  
Br Sa Tr Si compact  
Damp

91MG100

91MG102

15+050 16.0 LT C/L

D+500

0 -

BR

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SOUTHBOUND LANES

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Twp: Gibson

W.P. No.: 215-89-00

15+050      16.0   RT   C/L                      D+400

0 -                      BR

15+080      C/L

0 -                      BR

15+080      8.0   LT   C/L                      D-900

0 -                      BR

15+080      8.0   RT   C/L                      D-2.0

0 -                      Blds

15+100      C/L

0 -                      Sh Rk

15+100      8.0   LT   C/L                      D+1.2

0 -                      Sh Rk

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SOUTHBOUND LANES

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Twp: Gibson

W.P. No.: 215-89-00

17+110 C/L

0 - BR

17+120 C/L

0 - BR

17+120 C/L

0 - BR

17+120 8.0 LT C/L

D-500

0 - 100 Tps  
100 - 400 Br SaSi-SiSa some Gr Damp

17+120 8.0 RT C/L

D-3.85

0 - 150 Tps  
150 - NFP Blds

17+150 C/L

0 - Blds

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SOUTHBOUND LANES

Engineer: M. Ghinani

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Date: \_\_\_\_\_

Twp: Gibson

W.P. No.: 215-89-00

17+170 C/L

0 - 1.0	Peat/Tps	<u>91MG194</u> w@300 = 31.7%
1.0 - 2.1	Br Si Cl with Peat layers and Sa seams, very soft	Org Cont = 6.6%
	Wet	<u>91MG196</u> w@1.5 = 59.5
2.1 - 4.2	Grey Cl Si Tr Sa very soft	
	Wet	<u>91MG198</u> w@2.7 28.7%
4.2 -	NFP (possible BR)	

17+340 C/L

0 - 100	Blk Muck	
100 - 400	Br SiSa Wet	<u>91MG193</u>
400 -	NFP BR	
	WT @ 200 above surface	

17+350 C/L

0 - BR

17+350 8.0 LT C/L D+2.1

0 - BR

17+350 8.0 RT C/L D-3.4

0 - BR

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SOUTHBOUND LANES

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Twp: Gibson

W.P. No.: 215-89-00

17+370 C/L

0 - 200 Tps  
200 - NFP BR

17+370 8.0 LT C/L

D+900

0 - 100 Tps  
100 - 400 Br SaSi-SiSa Damp  
400 - NFP (possible BR)

91MG240

17+370 8.0 RT C/L

D-3.5

0 - 50 Tps  
50 - NFP BR

17+390 C/L

0 - BR

17+390 8.0 LT C/L

D+1.0

0 - BR

17+390 8.0 RT C/L

D-1.1

0 - BR

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SOUTHBOUND LANES

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Twp: Gibson

W.P. No.: 215-89-00

17+870 C/L

0 - 150	Tps	
150 - 1.1	Br SaSi Tr Cl	<u>92MG152</u>
1.1 -	NFP (possible BR)	

17+890 C/L

0 - 150	Tps	
150 - 250	Br Si FSa Damp	<u>91MG257</u>
250 -	NFP (possible BR)	

17+890 8.0 LT C/L D-600

0 - 300	Tps	
300 - 600	Br SiSa-SaSi Moist	<u>91MG258</u>
600 - 1.2	Br Cl Si Damp	<u>91MG259</u>
1.2 -	NFP (Refusal to hand auger)	

17+890 8.0 RT C/L D+200

0 - 200	Tps	
200 -	NFP (possible BR)	

17+910 C/L

0 - 150	Tps	
150 - 500	Br SiSa-SaSi Damp	
500 -	NFP (possible BR)	

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SOUTHBOUND LANES

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Twp: Gibson

W.P. No.: 215-89-00

17+910 8.0 LT C/L

0 - 300 Tps  
300 - NFP BR

17+910 8.0 RT C/L

0 - 200 Tps  
200 - 350 Br SiSa-SaSi Moist  
350 - NFP BR

17+920 C/L

0 - 150 Tps  
150 - NFP BR

17+950 C/L

0 - 200 Tps  
200 - NFP (possible Blds)

17+970 C/L

0 - 200 Tps  
200 - 1.1 Br Si Cl Damp 91MG260  
1.1 - NFP (Refusal to hand auger)

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SOUTHBOUND LANES

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Twp: Gibson

W.P. No.: 215-89-00

18+020 C/L

0 - 1.2  
1.2 - 4.1

Fib Tps  
Grey Si Cl with Si seams  
very soft  
Cu @ 2.6 = 16 kPa

91FBA256

91FBA257 w@1.9 = 27.4%  
WL = 23.0%  
WP = 15.9%  
IP = 7.1 CL

4.1 -

NFP (possible BR)

18+050 C/L

0 - 1.2  
1.2 - 5.6

Muck  
Grey Si Cl Wet very Soft  
Cu @ 2.0, 3.0, 3.8 = 9 kPa  
Cu @ 4.6 = 18, Cu @ 5.3 = 27  
NFP (possible BR)

92MG211, 212

5.6 -

18+070 C/L

D+1.0

0 - 800  
800 - 1.9

Dk Br Muck  
Greyish Br Si Cl Wet  
Wet @ 100 above surface

91FBA1

91FBA2

18+090 C/L

0 - 1.2  
1.2 - 3.8

Muck  
Grey Si Cl Wet with Sa seams  
very Soft  
Cu @ 2.1 = 31 kPa, Cu @ 3.0 = 27  
Cu @ 3.6 = 40  
NFP (possible BR)

92MG209, 210

3.8 -

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SOUTHBOUND LANES

Engineer: M. Ghinani

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Twp: Gibson

W.P. No.: 215-89-00

18+120 C/L

0 - 700  
700 - 2.0  
2.0 -

Fib Tps  
Grey Si Cl Firm Wet  
NFP (possible BR)

91FBA260  
91FBA261 w@1.0 = 28.1%

18+150 C/L

0 - 300  
300 - 2.0

Tps  
Br Si Cl with Sa and Si seams  
Firm

92MG205, 206,  
207, 208

2.0 - 4.7

Br Si Cl Wet very Soft  
Cu @ 2.7 = 27 kPa, Cu @ 3.0, 3.3 = 18  
Cu @ 3.5, 3.9 = 22, Cu @ 4.5 = 35

18+170 C/L

0 - 150  
150 - 550  
550 - 950  
950 -

Tps  
Br SaSi Tr Org and roots  
Damp  
Br SiSa SaSi Tr Gr Moist  
NFP BR

91FBA3  
91FBA4

18+170 10.0 LT C/L

D-800

0 - 150  
150 - 400  
400 - 1.2

Tps  
Br SiSa-SaSi Tr Gr Wet  
Greyish Br Si Cl Sa seams  
Wet  
WT @ 350

91FBA5  
91FBA6

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Twp: Gibson

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18+190 C/L

0 - 100	Tps	
100 - 200	Br SaSi-SaSi Tr Gr Damp	<u>91FBA7</u>
200 -	NFP BR	

18+190 8.0 LT C/L D-1.6

0 - 200	Tps	
200 - 400	Br SiSa-SaSi Tr Gr Damp	
400 - 650	Br Sa some Gr Tr Si	
	Moist	<u>91FBA8</u>
650 -	NFP (possible BR)	

18+190 8.0 RT C/L D+950

0 - 250	Tps	
250 - 700	Br SiSa-SaSi Tr Gr Damp	
700 -	NFP BR	

18+210 C/L

0 - 150	Tps	
150 - 350	Br SiSa-SaSi Tr Gr Moist	
350 -	NFP BR	

18+210 8.0 LT C/L D-1.5

0 - 150	Tps	
150 - 300	Br SiSa-SaSi Tr Gr Damp	
300 -	NFP BR	

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SOUTHBOUND LANES

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Twp: Gibson

W.P. No.: 215-89-00

19+970 C/L

0 - Blds

19+970 8.0 LT C/L

D-1.3

0 - Blds

19+970 8.0 RT C/L

D+800

0 - 200 Tps  
200 - NFP (possible BR)

20+000 C/L

0 - 200 Tps  
200 - 600 Br Sa Tr Sa Damp  
600 - NFP (possible BR)

91MG292

20+000 8.0 LT C/L

D-1.6

0 - 200 Tps  
200 - NFP (possible BR)

20+000 8.0 RT C/L

D-700

0 - 500 Tps  
500 - NFP (possible BR)

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SOUTHBOUND LANES

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Twp: Gibson

W.P. No.: 215-89-00

20+020 C/L

0 - 200	Tps	
200 - 500	Br SiSa-SaSi Damp	<u>91MG293</u>
500 -	NFP (possible BR)	

20+020 8.0 LT C/L D-1.7

0 - BR

20+020 8.0 RT C/L D+800

0 - 250	Tps	
250 - 600	Br SiSa-SaSi Damp	
600 -	NFP (possible BR)	

20+040 C/L

0 - 300	Tps	
300 - 600	Dk Br SiSa-SaSi	
	some Gr Damp	<u>91MG294</u>
600 -	NFP (possible BR)	

20+050 C/L

0 - 50	Tps	
50 -	NFP BR	

Hwy. No.: 69

SOUTHBOUND LANES

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Twp: Gibson

W.P. No.: 215-89-00

20+050 8.0 LT C/L

D-900

0 - 250  
250 - 650  
650 -

Tps  
Br SaSi Tr Gr Damp  
NFP BR

91MG295

20+050 8.0 RT C/L

D-400

0 - 220  
220 -

Tps  
NFP BR

20+060 C/L

0 - 250  
250 - 450  
450 -

Tps  
Dk Br SiSa Damp  
NFP (possible BR)

20+100 C/L

0 - 400  
400 - 600  
600 - 1.0  
1.0 - 4.4

Blk Muck  
Grey SaSi very Loose  
Moist  
Grey Si FSa Tr Gr Loose  
Grey SaSi with Si Cl seams  
very Loose

91MG302

91MG303 w@900 = 18.8%

91MG304 w@1.5 = 22.1%

Hwy. No.: 69

SOUTHBOUND LANES

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Twp: Gibson

W.P. No.: 215-89-00

20+350 C/L

0 - 200	Tps	
200 - 650	Br SiSa-SaSi Loose Moist	<u>91MG296</u> w@300 = 22.8%
650 - 1.5	Br SaSi with Si Cl seams	
	Comp	<u>91MG297</u> w@900 = 15.7%
1.5 - 4.9	Grey Si Cl with Si pockets	
	very soft Wet	<u>91MG298</u> w@2.1 = 34.9%
	Cu @ 2.5 = 95 kPa	WL = 28.7%
	WT @ 700	WP = 17.6%
		IP = 11.1 CL

20+450 C/L

0 - 150	Tps	
150 - 1.4	Br Si Fsa Comp below 600	
	Moist	<u>91MG482</u> and 483
1.4 - 3.0	Br Si Cl with Sa seams	
	Stiff to Firm Moist	<u>91MG484</u> and 485
	grey @ 2.4	<u>91MG486</u>
	WT @ 450	

20+500 C/L

0 - 250	Muck	
250 - 800	Br SiSa Wet	<u>91MG306</u>
800 - 900	Br Si Cl Wet	<u>91MG307</u>
900 - 1.3	Br SaSi with Si Cl seams	<u>91MG308</u>
	WT @ surface	

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Twp: Gibson

W.P. No.: 215-89-00

20+520 C/L

0 - 250	Tps	
250 - 1.1	Br Si FSa Loose Wet	<u>92MG35</u>
1.1 - 2.0	Br SaSi with Si Cl seams Loose	<u>92MG37</u>
2.0 - 3.7	Grey Si Cl Firm to Stiff	
	Cu @ 2.4, 3.0 = 53 kPa, Cu @ 3.4 = 27	
	Cu @ 3.7 = 53	
	WT @ surface	

20+550 C/L

0 - 300	Muck	
300 - 700	Br Si FSa Wet	<u>91MG309</u> w@500 = 23.9%
700 - 1.0	Br Cl Si Wet	<u>91MG310</u> w@850 = 24.7%
1.0 - 1.4	Grey Cl Si Wet	
	WT @ surface	

20+580 C/L

0 - 300	Tps	<u>92MG38</u>
300 - 1.2	Br Si FSa very Loose to Compact	<u>92MG39</u>
1.2 - 1.6	Br SaSi Tr Cl Loose Wet	
1.6 - 4.6	Br Si Cl	<u>92MG40</u>
4.6 - 9.1	Si Cl (inferred)	<u>92MG41</u>
	WT @ surface	<u>92MG42</u>
	Cu @ 2.1 = 89 kPa, Cu @ 3.0 = 49	
	Cu @ 3.8 = 84	

20+600 C/L

0 - 700	Bl Muck	<u>91MG311</u>
700 - 1.0	Grey SiSa-SaSi Wet	<u>91MG312</u>
1.0 - 1.4	Grey Si Cl Wet	<u>91MG313</u>

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22+320 9.0 RT C/L

D-400

0 - 200 Tps  
200 - 1.2 Dk Br SaSi Tr Gr Damp  
1.2 - NFP (possible Blds)

22+370 C/L

0 - 100 Tps  
100 - NFP (possible BR)

22+370 11.0 LT C/L

D-100

0 - 200 Tps  
200 - 600 Dk Br SaSi-SiSa Tr Org  
Moist 91FBA66  
600 - 950 Br Si FSa Tr Gr Wet 91FBA67  
950 - NFP (possible BR)  
WT @ 600

22+370 10.0 RT C/L

D+650

0 - Blds

22+420 C/L

0 - 100 Tps  
100 - 450 Br Si FSa Damp 91FBA68  
450 - 770 Blk Org SaSi Wet 91FBA69  
700 - 1.1 Br Si Cl Moist 91FBA70  
1.1 - NFP (possible BR)

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22+420      11.0   LT   C/L                      D-200

0 - 200	Tps
200 - 400	Br SaSi-SiSa Tr Gr Moist
400 - 600	Br Cl Si Tr Sa seams Wet
600 -	NFP (possible BR)

22+420      11.0   RT   C/L                      D-300

0 - 50	Tps	
50 - 400	Br SaSi-SiSa Tr Gr Damp	
400 - 800	Br Cl Si Tr Sa seams	
	Moist	<u>91FBA71</u>
800 -	NFP (possible Blds)	
	WT @ 600	

22+440      C/L

0 -	BR
-----	----

22+440      8.0   LT   C/L                      D-700

0 - 300	Tps
300 -	NFP BR

22+440      8.0   RT   C/L                      D-900

0 - 300	Tps
300 -	NFP BR

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22+460 C/L

0 - 300	Tps Damp	<u>91FBA72</u>
300 - 1.1	Br SiSa-SaSi some Gr Wet	<u>91FBA73</u> w@700 = 27.4%
1.1 -	NFP (possible BR)	
	WT @ 800	

22+510 C/L

0 - 200	Tps	
200 - 1.0	Br SiSa-SaSi Tr Gr Wet	<u>91FBA74</u>
1.0 -	NFP (Refusal to hand auger)	
	WT @ 200	

22+510 8.0 LT C/L

0 - 200	Tps	
200 - 1.0	Br SiSa-SaSi Tr Gr Wet	
1.0 -	NFP (Refusal to hand auger)	
	WT @ 200	

22+510 8.0 RT C/L

0 - 100	Tps	
100 - 650	Br Sa Tr Si and Gr Wet	<u>91FBA75</u>
	Wet @ surface	
650 -	NFP (Refusal to hand auger)	

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22+560 C/L

0 - 20	Tps	
20 - 500	Br SaSi-SiSa Tr Gr Moist	<u>91FBA76</u> w@250 = 30.5%
500 - 1.3	Br SiSa Tr Gr Wet	<u>91FBA77</u>
1.3 - 1.4	Br Gr Sa Wet	
1.4 -	NFP (Refusals to hand auger)	
	WT @ 400	

22+560 10.0 LT C/L

0 - 100	Tps	
100 - 450	Br SaSi-SiSa Tr Gr Moist	
450 - 1.0	Lt Br SiSa some Gr Wet	
1.0 -	NFP (Refusals to hand auger)	
	WT @ 400	

22+560 10.0 RT C/L D-200

0 - 100	Tps	
100 - 250	Br SaSi-SiSa Tr Gr Moist	
250 - 950	Lt Br SiSa Tr Gr Wet	
950 -	NFP (Refusals to hand auger)	
	WT @ 400	

22+580 C/L

0 - 250	Tps	
250 - 1.3	Br SiSa Tr Gr Wet	<u>92MG155</u>
1.3 -	NFP (possible BR)	

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22+610 C/L

0 - 300  
300 - 1.1  
1.1 - 2.8  
2.8 - 3.0

Tps

Br Si FSa Wet Loose  
Br Si Cl Firm to Stiff  
Cu = 44 to 62 kPa  
Br SiSa Dense

92MG177, 178

92MG179

92MG180

22+630 C/L

0 - 200  
200 - 900  
900 - 2.0  
2.0 - 2.3  
2.3 - 2.9  
2.9 -

Tps

Br SiSa Wet Loose  
Br Si Cl Wet Soft  
Br SiSa some Gr Comp  
Br Si and Gr Wet Comp  
NFP (possible Blds)

92MG172

92MG173, 174

92MG175

92MG176

22+660 C/L

0 - 150  
150 - 650  
650 - 1.45

Tps

Br SaSi-SiSa Tr Gr Moist 91FBA78 w@400 = 19.9%  
Br Sa some Si Tr Gr Wet 91FBA79  
WT @ 1.2

22+660 16.0 LT C/L

D+1.2

0 - 150  
150 - 600  
600 - 1.05

Tps

Br SaSi SiSa Tr Gr Damp  
Lt Br Si some Sa Tr Gr  
Damp

91FBA80

1.05 -

NFP (Refusal to hand auger)

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0 - 200	Tps	
200 - 800	Dk Br SaSi Moist	<u>91FBA81</u>
800 - 1.4	Br Sa some Si Tr Gr Wet	<u>91FBA82</u>
	WT @ 1.1	

```

0 - 100      Tps
100 - 400    Br SaSi-SiSa Tr Gr Damp
400 - 900    Br SiSa-SaSi Tr Gr Wet      91FBA83 w@650 = 30.9%
900 -        NFP (possible BR)

```

0 - Blds

0 - 100	Tps
100 -	NFP BR

0 - 400	Tps
400 -	NFP BR

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22+740 C/L

0 - 50  
50 -

Tps  
NFP BR

22+740 C/L

0 - 250  
250 - 600  
600 -

Tps  
Dk Br SiSa-SaSi  
NFP BR

22+760 C/L

0 - 50  
50 -

Tps  
NFP BR

22+760 8.0 LT C/L

D+100

0 - 50  
50 -

Tps  
NFP BR

22+760 8.0 RT C/L

D+150

0 - 50  
50 -

Tps  
NFP BR

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22+860 C/L

0 - 50	Tps
50 -	NFP BR

22+860 8.0 LT C/L

0 - 100	Tps
100 -	NFP BR

22+860 8.0 LT C/L

0 - 100	Tps
100 -	NFP BR

22+860 8.0 RT C/L

0 -	BR
-----	----

22+880 C/L

0 - 300	Tps
300 -	NFP BR

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22+920 C/L

0 - Blds

22+960 C/L

0 - 50 Tps  
50 - 800 Grey Si FSa Moist 91FBA87 w@425 = 15.6%  
800 - 1.2 Br Sa some Si Tr Gr Moist 91FBA88

22+960 20.0 LT C/L D-2.25

0 - 200 Tps  
200 - 950 Dk Grey SaSi Tr Org Wet 91FBA89  
950 - NFP (possible BR)  
WT @ 250

22+960 20.0 RT C/L D+1.8

0 - 100 Tps  
100 - 500 Br SaSi-SiSa Tr Gr Damp  
500 - 1.2 Lt Br Sa some Si Tr Gr  
Moist 91FBA90

23+000 C/L

0 - 250 Tps  
250 - 800 Br Si FSa Tr Gr Wet 91MG342 w@525 = 29.6%  
800 - NFP (possible BR)

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23+000      10.0   LT   C/L                      D+2.2

0 - 50                      Tps  
50 -                      NFP BR

23+000      8.0   RT   C/L                      D-600

0 - 200                      Tps  
200 - 1.3                      Br SaSi Wet                      91MG343  
1.3 -                      NFP (possible Blds)

23+025      C/L

0 - 1.3                      Dk Br SaSi-SiSa Org Stained (possible Fill)

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23+050 C/L

0 - 150

150 - 350

350 -

Tps

Br SaSi-SiSa some Gr

Damp

NFP BR

91MG344

23+075 C/L

0 -

BR

23+090 C/L

0 - 200

200 - 800

800 -

Tps

Br SiSa Tr Gr Moist

NFP (possible Blds)

92MG156

23+096 C/L

0 - 400

400 - 900

900 -

Tps

Dk Br SaSi-SiSa Wet

NFP (possible BR)

23+100 C/L

0 - 1.4

Tps (possible Fill)

91MG345

23+120 C/L

0 -

Blds

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0 - 200	Tps
200 - 350	Br SaSi-SiSa Tr Gr Damp
350 -	NFP (possible Blds)

0 - 200	Tps
200 -	NFP Bld

22+840      C/L

0 - 700      Tps

700 -      NFP (possible BR)

0 - 250	Tps
250 -	NFP (possible BR)

22+840	8.0	RT	C/L		D-1.3
0 - 150			Tps		
150 - 750			Br SaSi Tr Gr Moist		<u>91MG375</u>
750 -			NFP (possible BR)		

22+860 C/L

0 - BR

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0 - 500 Br SiSa-SaSi Tr Gr some Tps  
500 - NFP BR

0 - BR

0 - 200	Tps	
200 - 550	Dk Br SiSa Tr Gr Wet	<u>91MG376</u>
550 -	NFP BR	

0 - 200	Tps
200 -	NFP BR

0 - BR

0 - 100	Tps
100 - 250	Br SiSa-SaSi Tr Gr Moist
250 -	NFP BR

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22+900 8.0 LT C/L

0 - 50 Tps  
50 - NFP BR

22+900 8.0 RT C/L

D-1.0

0 - 100 Tps  
100 - 700 Dk Br SiSa-SaSi Tr Gr  
Moist  
700 - NFP BR

91MG377

22+950 C/L

0 - 100 Tps  
100 - 850 Br SiSa Tr Gr  
850 - NFP (possible BR)

91MG378 w@500 = 20.3%

22+950 8.0 LT C/L

D-2.4

0 - 100 Tps  
100 - 1.5 Br SaSi-SiSa Tr Gr Wet  
1.5 - NFP (possible BR)

91MG379

22+950 8.0 RT C/L

D+1.2

0 - 150 Tps  
150 - 450 Br SiSa Tr Gr Damp  
450 - NFP (possible BR)

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Twp: Gibson

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23+000 C/L

0 - 150  
150 - 750  
750 -

Tps  
Dk Br SiSa-SaSi Tr Gr  
(possible Fill)  
NFP (possible Blds)

91MG380 w@500 = 36.0%

23+000 8.0 LT C/L

D-400

0 - 450  
450 -

Br SiSa-SaSi Tps mixed  
(possible Fill)  
NFP (possible BR)

91MG381

23+000 8.0 RT C/L

D+400

0 - 450  
450 -

Br SiSa-SaSi Tps mixed  
(possible Fill)  
NFP BR

23+050 C/L

0 - 600  
600 - 1.0  
1.0 -

Br Sa and Gr (Gran)  
Br Sa some Gr (Gran)  
NFP (possible RF)

91MG1001

91MG1002

23+050 8.0 LT C/L

0 -

BR

23+050 8.0 RT C/L

D+500

0 -

Asph

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9+600 C/L

0 - 200	Tps	
200 - 700	Br SaSi Tr Cl Wet	<u>91MG420</u>
700 -	NFP (possible BR)	
	WT @ 300	

9+620 C/L

0 - 200	Tps
200 - 350	Br SiSa Damp
350 -	NFP BR

9+640 C/L

0 - 200	Tps	
200 - 550	Dk Br SaSi some Org Wet	<u>91MG421</u> w@300 = 94.2%
550 -	NFP (possible BR)	

9+660 C/L

0 - 150	Tps
150 -	NFP BR

9+660 5.0 RT C/L

0 - 200	Tps
200 - 600	Dk Br SaSi Tr Org Moist Wet
600 -	NFP (possible BR)

9+660 LT C/L

D+300

0 - 200	Tps
200 -	NFP BR

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MUSKOKA ROAD #32

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Twp. Gibson

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9+680 C/L

0 - 100  
100 - 1.3  
1.3 -

Tps  
Br SiSa Tr Gr Wet  
NFP (possible BR)

91MG422 w@500 = 20.6%

9+700 C/L

0 - 200  
200 - 1.5

Tps  
Br SiSa Occ Gr Moist  
Compact

91MG509

91MG510

91MG511

1.5 - 1.8

Br Si Cl Damp very Stiff

9+720 C/L

0 - 300  
300 - 1.7

Tps  
Br SaSi Wet to Moist  
very Loose above 600  
Compact below 600

91MG512

91MG513 w@900 = 17.3%

91MG514

1.7 - 2.4

Br Si Cl Moist Firm

91MG515 w@2.1 = 31.6%

WT @ 100 above above ground surface

9+820 C/L

0 - 100  
100 - 500  
500 -

Tps  
Br SiSa Tr Gr Wet  
NFP (possible BR)

9+820 5.0 RT C/L

D-700

0 - 350  
350 -

Muck  
NFP (possible BR)  
WT @ 100 above the ground surface

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9+820      5.0      LT      C/L      D+800

0 - 100      Tps  
100 - 600      Br SiSa Tr Gr Wet  
below 300      91MG423  
600 -      NFP (possible BR)

9+840      C/L

0 - 200      Tps  
200 - 1.0      Br SiSa Tr Gr Damp      91MG424  
1.0 -      NFP (possible Blds)

9+840      5.0      RT      C/L      D+1.1

0 - 150      Tps  
150 - 900      Br SiSa some Gr Damp      91MG425  
900 -      NFP (possible Blds)

9+840      5.0      LT      C/L      D-1.3

0 - 750      Dk Br SiSa-SaSi Tr Org      91MG426  
750 -      NFP (possible Blds)  
Wet @ 250 above ground surface

9+860      C/L

0 - 250      Tps  
250 - 800      Br SiSa some Gr Moist  
800 -      NFP (possible Blds)

9+860      5.0      RT      C/L      D-1.1

0 - 200      Tps  
200 -      NFP Blds

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9+860      5.0      LT      C/L      D+800

0 - 100      Tps  
100 -      NFP (possible BR)

9+880      C/L

0 - 200      Tps  
200 - 450      Br SiSa Tr Gr  
450 -      NFP (possible BR)

9+900      C/L

0 - 100      Tps  
100 - 800      Br SaSi Tr Gr Wet      91MG427 w@500 = 36.0%  
800 -      NFP (possible Blds)

9+920      C/L

0 - 150      Tps  
150 - 400      Br SiSa  
400 -      NFP (possible BR)

9+940      C/L

0 - 200      Tps  
200 - 350      Dk Br SiSa-SaSi Tr Org  
Moist to Wet      91MG428

9+960      C/L

0 - 250      Tps  
250 - 400      Br SiSa-SaSi Moist to Wet  
400 -      NFP BR

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MUSKOKA ROAD #32

Engineer: M. Ghinani



Ministry of  
Transportation and  
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## **FOUNDATION DESIGN SECTION**

**foundation  
investigation and  
design report**

ENGINEERING MATERIALS OFFICE  
FOUNDATION DESIGN SECTION

WP 217-89-00(A) DIST 11

HWY 69 STR SITE

Proposed Southbound Embankment  
Station 17+150 to 17+335  
Location No. 2

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FOUNDATION INVESTIGATION REPORT  
FOR  
PROPOSED SOUTHBOUND EMBANKMENT  
STATION 17+150 TO 17+335, HWY 69  
W.P. 217-89-00 (A), LOCATION NO. 2  
DISTRICT 11 (HUNTSVILLE)

INTRODUCTION

This report contains the results of foundation investigations carried out in two stages; from 92 03 30 to 92 04 02 and from 93 02 04 to 93 02 09. The alignment change has necessitated the second investigation. The field work consisted of nine sampled boreholes accompanied with Dynamic Cone Penetration Test without sampling. The borings were advanced by a continuous flight auger machine mounted on a muskeg vehicle and equipped with 82 mm (I.D.) hollow stem augers.

Site Description

The site is located immediately to the west of the existing Highway 69 embankment, between Stations 17+150 and 17+335. The site is adjacent to Muskoka Road 32 and located within Lot 19, Concession V11, Gibson Township, District of Muskoka.

Throughout this area, the existing Highway 69 consists of a roadway with single lanes running in both the north and south directions. Through a series of rock cuts and embankments, the roadway traverses undulating topography consisting of rock knolls of gneissic bedrock separated by low swampy or wooded areas. The direction of flow of several rivers, in this area, indicates that drainage is generally towards the west; ie. Georgian Bay.

At this location, Highway 69 has been constructed on top of an embankment, which is about 4.5 to 5.5 m high (ie. to an elevation of 188.4 to 190.5 m). Swampy areas, characterized by cattails and small bushes, are found on both sides of the embankment and the groundwater table is at or near the existing ground surface.

### Subsurface Conditions

#### General

The natural subsoil at this location was found to consist of a surficial deposit of 4.3 m to 7 m thick organic material (peat), followed by 1.5 m to 10.4 m of silty clay to clay stratum. This is underlain by a zone of silty sand with a minimum thickness of 0.9 m. The boundaries of the different strata encountered at the boring locations together with the obtained field and laboratory test results are shown on the Record of Borehole Sheets contained in the appendix of this report.

Drawing No. 1 shows the locations of the borings in relation to the future southbound lanes and the existing lanes (future northbound lanes).

#### Sand and Gravel - Fill Material

Borehole No. 2 - 10 is located on Muskoka Road #32. The material in the 4.7 m high roadway embankment was found to consist of sand and gravel. The standard penetration test "N" values ranged from 10 blows per 30 cm to 56 blows per 15 cm and may be classified as compact to very dense.

### Peat

This stratum was encountered in all borings immediately below ground level or below the fill material. The thickness varies from 4.3 m (BH #2-1) to 7 m (BB #2-9). The material consists mainly of decayed and undecayed organic substances. The degree of decomposition is increasing with depth. The upper portion (2 - 3 m) of the peat is fibrous, mostly partially decomposed vegetable matter. This deposit has a very low shear strength, high moisture content and extremely high compressible characteristics in-situ shear strength values ranged from 4 kPa to 16 kPa, and the natural moisture content averaged about 415%.

The consistency of the deposit may be described as soft to very soft.

### Silty Clay to Clay, Trace of Sand

This deposit was intersected in all borings and extends from the above described organic material down to El. 176.9 and El. 167.4. The maximum thickness (10.4 m) in BH #2 - 9. The material in the deposit consists mainly of silty clay and clay with trace of sand. Randomly, layers of clayey silt (CL) were also discovered within the main deposit. A plot of plasticity index versus liquid limit (Figure 1) shows the majority of the points to fall within the CI and CH zones.

Physical properties of the main deposit as determined from field and laboratory tests, are as follows:

	<u>Range</u>
Natural moisture content (w)	42 - 122%
Liquid Limit ( $w_p$ )	38 - 61%
Plastic Limit ( $w_L$ )	18 - 27%
Bulk Density ( $\gamma$ )	13.9 - 17.1 kN/m <sup>3</sup>
Field Vane Test ( $c_u$ )	7 - 28 kPa
Sensitivity	2 - 6

For stability analyses, the following undrained shear strength values are recommended:

El. 180 - El. 178.5 ;  $c_u = 18$  kPa

El. 178.5 - El. 175 ;  $c_u = 10$  kPa

El. 175 - El. 173 ;  $c_u = 15$  kPa

El. 173 - El. 171 ;  $c_u = 20$  kPa

The consistency of the overall deposit may be described as very soft to firm.

#### Silty Sand Trace of Clay

This deposit was intersected in most of the boreholes with the exception of BH #2 - 1 0 and # 2 - 7 immediately below the clay to silty clay stratum.. The thickness varies from 0.9 m (BH #2 - 6) to 5.0 m (BH #2 - 3). The lower boundary assumed to be at levels where refusal to conventional boring methods were reached (El. 176 - El. 167.6). In boreholes # 2- 4 and # 2 - 9, the borings were terminated above the refusal level. The material in the deposit consists mainly of sand; 89 - 51%, silt 44 - 5% and trace of clay (8 - 3%).

In BH #2 - 6 only, 13% of gravel was also observed. The natural moisture ranged from 11% to 31%. Standard Penetration Tests, gave "N" values from 1 to 21 blows per 30 cm. In some instances, the split spoon sampler sank under the weight of rods and hammer. Based on the "N" values, the deposit is classified to be in very loose to compact state.

#### Sand and gravel with Some Silt and Trace of Clay

In BH #2 - 7 the clay to silty clay stratum is underlain by an 2.1 m thick sand (51%), Gravel (34%) with some silt (12%) and trace of clay (3%) deposit. The lower boundary assumed to be at El. 170.2. The natural moisture content is in the order of 12%. Standard Penetration Tests "N" values ranged from 11 to 40 blows per 30 cm. The denseness may be described as compact to dense.

#### Groundwater Conditions

The water levels encountered in the boreholes at the time of field investigations are summarized below. The water levels were generally found to be at, or near the elevation of the ground surface in most of the borings.

<u>Borehole</u>	<u>Water Depth/Elevation</u>
2 - 1	0.0 m / 185.1
2 - 2	0.0 m / 185.1
2 - 3	0.2 m / 185.0
2 - 4	0.0 m / 185.3
2 - 6	0.0 m / 184.4
2 - 7	0.9 m / 183.5

- 6 -

2 - 8	Not observed
2 - 9	0.9 m / 183.9
2 - 10	1.4 m / 184.5

It is pointed out that boreholes #2-1 to #2-4 (incl.) were drilled in early spring of 1992 while borings #2-6 to #2-10 (incl.) were put down in the winter of 1993.

No artesian conditions were encountered.

## DISCUSSION AND RECOMMENDATIONS

### General

It is proposed to widen Hwy #69 to four lanes. Presently, the highway constructed under contract No. 55-290 consists of single north and southbound lanes. After completion of the project, the new lanes will be designated as southbound lanes, while the presently existing lanes will serve the northbound traffic.

According to the original proposals, the new lanes would have been located some 42 m west (Ⓔ to Ⓔ ) of the existing roadway. The profile grade was to be 8 m to 10 m above the natural groundsurface. The field investigations have indicated the presence of very soft to soft peat and silty clay to clay deposits to considerable depths. In order to ensure the integrity of the new lanes, some portion of these deposits have to be subexcavated. However, this procedure would undermine the existing Hwy #69. Consequently, it was recommended that the alignment be moved to further west and the profile grade level be lowered.

The new alignment was 15 m to 19 m further to west and the profile was set at 4 m to 5 m above the groundline between Sta. 17+160 and Sta. 17+335.

### Construction

The ground is covered with swamps at the location of the proposed new lanes. In general, swamps are the mixture of decomposed and partially disintegrated organic substances combined with extremely high water content. These types of deposits are highly compressible in nature and consequently subject to immediate and long term

settlements due to surcharge loading. These settlements would significantly affect the performance of a future road. Therefore, it is recommended that the peat completely (horizontal and vertical directions) be removed. This can be achieved by excavation and displacement methods or the combination of the two. However, the displacement method may undermine the stability of the existing lanes. Therefore, this method is not recommended. The only option left is subexcavation. In order, to ensure the integrity of the existing embankment, the excavation should be carried out in strips, commencing not closer than 12 m to the toe of the existing embankment, between Sta. 17+160 and Sta. 17+340. The strip excavation limits (horizontal and vertical) together with the geometry of an individual strip are shown on sketch #1. The excavated strips should be backfilled with non-cohesive (rockfill) material, prior to the commencement of the excavation of the adjacent strip. West of the strip excavation west limit, the subexcavation should be carried out as outlined on sketch #2.

Again, the backfill should consist of non-cohesive (Rockfill) material.

Upon the completion of the subexcavation and backfill operation, the embankment should be built up to the proposed profile. In the case of rockfill, the slopes should not be steeper than 1.25:1. For earth material, 2:1 slopes are recommended. It is estimated that the settlement will be in the order of 0.7 m to 1.5 m. However, the majority of the settlements will take place within 1.5 to 2.0 years. Therefore, it is recommended that embankment be constructed as long as possible ahead of the final paving.

In order to observe the magnitude of settlement, some monitoring will be required. A simple method is to take periodic cross-sections at the same locations.

It is also recommended that the subexcavated material should not be stockpiled within the



site.

### Miscellaneous

The first stage of the field investigation was supervised by Mr. J. Blair of the Foundation Design Section and Mr. Dan Rothwell of the Northern Region Geotechnical Section, using equipment owned and operated by Atcost soil drilling inc.

The second stage of the field investigation was supervised by Mr. Blair and Mr. David Walters of the Foundation Design Section using equipment from both Atcost Soil Drilling Inc. and Master Soil Investigation Inc.

This report was written by Mr. P. Payer, Senior Foundation Engineer with the assistance of Mr. J. Blair, Project Foundation Engineer and approved by Mr. D. Dundas, Chief (Acting) Foundation Engineer.

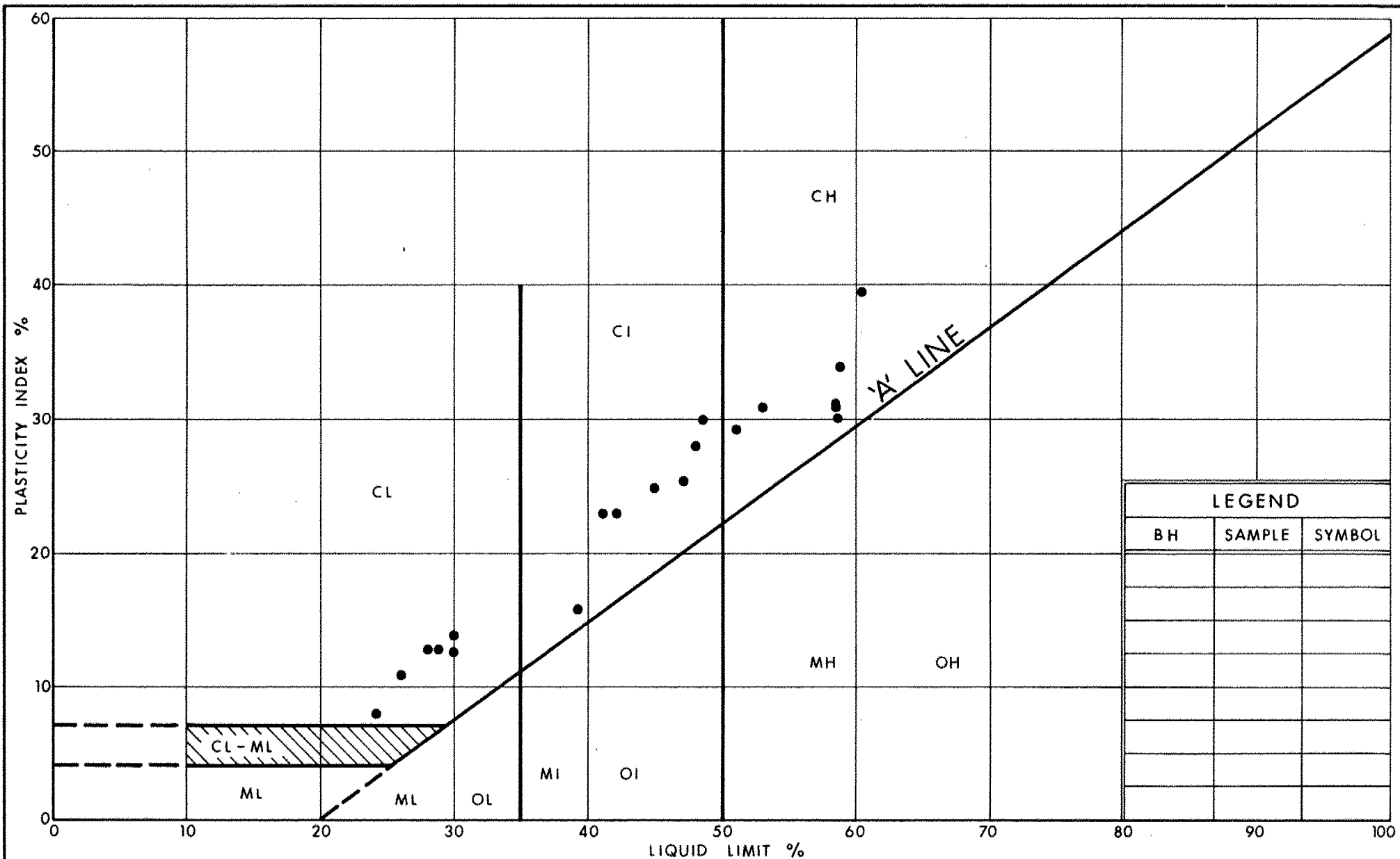


*P. Payer*  
P. Payer, P. Eng.  
Sr. Foundation Engineer



*D. Dundas*  
D. Dundas, P. Eng.  
Chief Foundation Engineer  
(Acting)

## **APPENDIX**



Ministry of  
Transportation

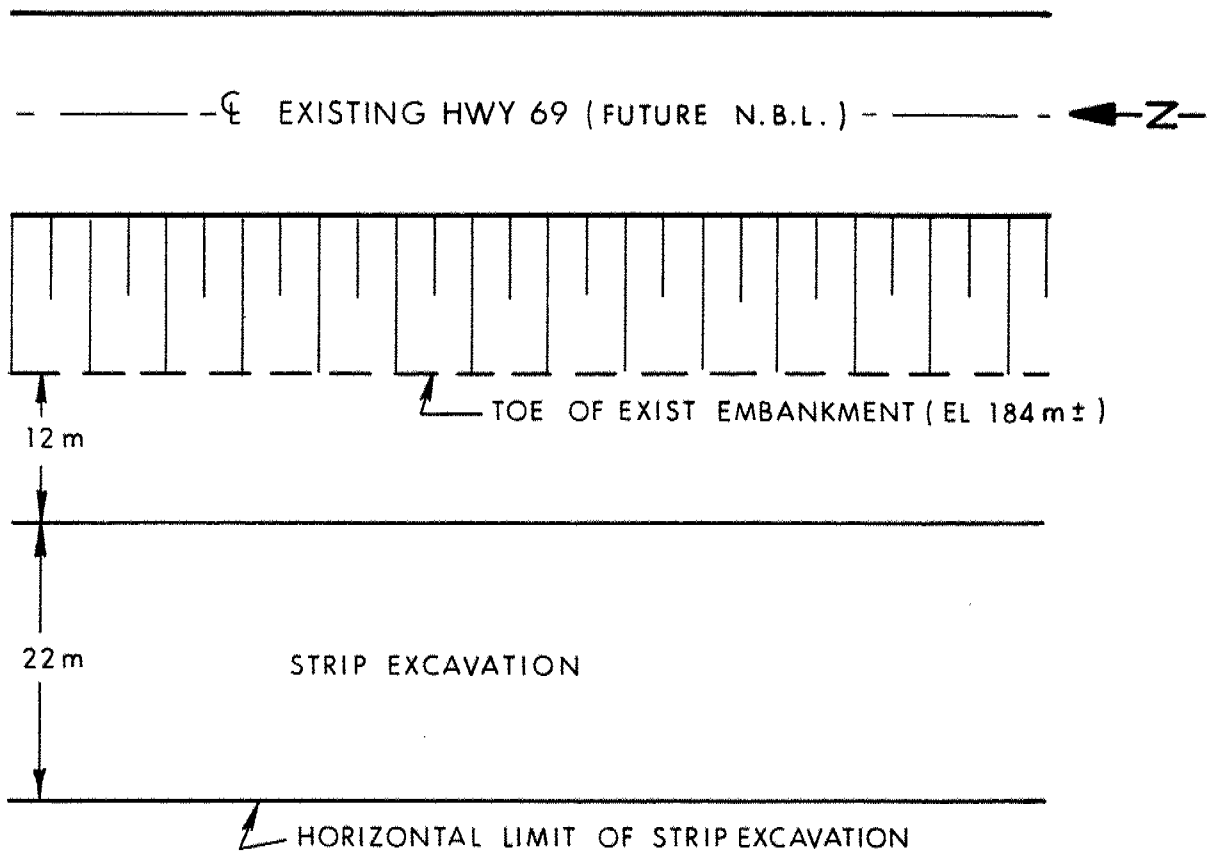
Ontario

# PLASTICITY CHART CLAY TO SILTY CLAY OCC. CLAYEY SILT LAYERS

FIG No 1

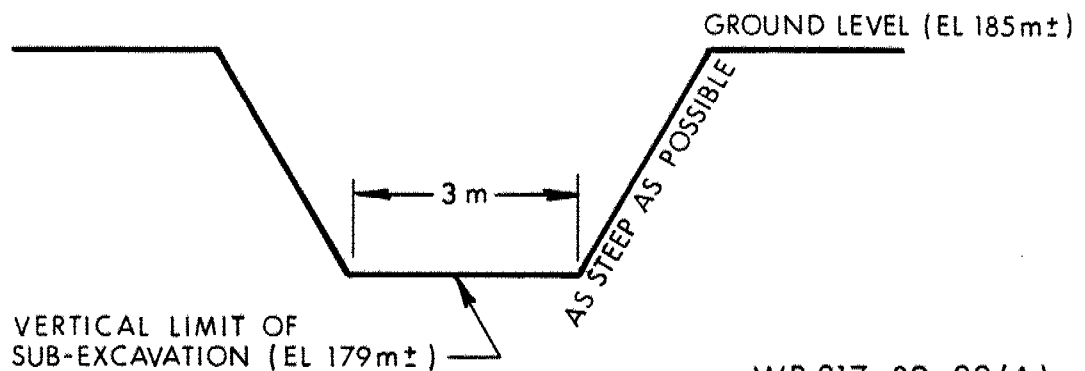
W P 217-89-00(A)

# STRIP EXCAVATION (STA. 17+160 TO STA. 17+340)



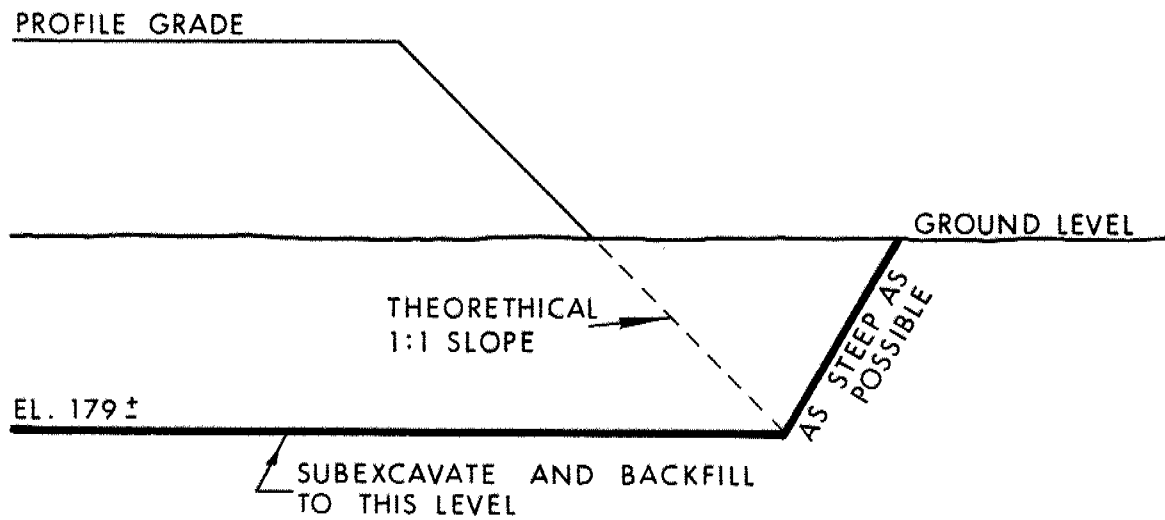
NOT TO SCALE

## STRIP GEOMETRY (PARALLEL TO CL)



WP 217-89-00(A)  
SKETCH No 1

SUBEXCAVATION  
( WEST OF WEST LIMIT OF STRIP EXCAVATION )



NOT TO SCALE

WP 217-89-00(A)

SKETCH No 2

# RECORD OF BOREHOLE No 2-1

1 OF 1

METRIC

W.P. 217-89-00(A) LOCATION Sta. 17+187; 10 m Lt. C/L S.B.L. ORIGINATED BY JB  
 DIST 11 HWY 69 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB/PP  
 DATUM Geodetic DATE April 2, 1992 CHECKED BY PP

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			20 40 60 80 100	20 40 60 80 100					
185.1	Ground Surface													
0.0	Peat Dark Brown to Black Very Soft		1	SS	1		184							
			2	SS	1		182							
180.8			3	SS	1		180							
4.3	Clay to Silty Clay, Trace of Sand Light Gray to Reddish Gray Very Soft to Soft		4	SS	0 **		178							
			5	TW	PH		176							
			6	SS	0 **		174							
176.2			7	SS	0 **		172							
8.9	Silty Sand Gray Very Loose													
171.8														
13.3	End of Borehole Refusal - Probable Bedrock  * Water Level at Ground Surface ** Split Spoon sank under weight of hammer and rods													

# RECORD OF BOREHOLE No 2-2

1 OF 1

METRIC

W.P. 217-89-00(A) LOCATION Sta. 17+221; C/L S.B.L. ORIGINATED BY JB  
 DIST 11 HWY 69 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB/PP  
 DATUM Geodetic DATE April 1, 1992 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER * CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
185.1	Ground Surface													
0.0	Peat Dark Brownish Grey to Black Very Soft to Soft		1	SS	1		184							
			2	SS	1		182							
			3	SS	1		180							
			4	SS	1		178							
179.2	Clay to Silty Clay, Occasional Fine Sand Layers Brownish Grey to Grey Soft to Firm		5	SS	1		176							
5.9			6	SS	0 **		174							
			7	SS	1									
174.9	Silty Sand Grey Very Loose		8	SS	2									
10.2														
172.3														
12.8	End of Borehole Refusal - Probable Bedrock  * W.L. on April 14, 1992 ** Split spoon sunk under the weight of the hammer and rods													

# RECORD OF BOREHOLE No 2-3

1 OF 1

METRIC

W.P. 217-89-00(A) LOCATION Sta. 17+280; 5 m Lt. C/L S.B.L. ORIGINATED BY JB  
 DIST 11 HWY 69 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB/PP  
 DATUM Geodetic DATE March 31, 1992 CHECKED BY PP

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			20	40					
185.2	Ground Surface													
0.0			1	SS	1									
			2	SS	1									
	Peat Dark Brown to Black Very Soft to Soft		3	SS	1									
			4	SS	1									
178.9			5	SS	1									
6.3			6	SS	1									
	Clay to Silty Clay Brownish Grey to Grey Very Soft to Soft		7	TW	PH									
			8	SS	0 **									
172.6			9	SS	1									
12.6			10	SS	0 **									
	Silty Sand Trace of Clay Grey Very Loose		11	SS	1									
167.6			12	SS	1									
17.6	End of Borehole Refusal - Probable Bedrock  • W.L. immediately upon completion of sampling • Split spoon sank under weight of hammer and rods													



# RECORD OF BOREHOLE No 2-4

1 OF 1

METRIC

W.P. 217-89-00(A) LOCATION Sta. 17+298; 2 m Rt. C/L S.B.L. ORIGINATED BY JB/DR  
 DIST 11 HWY 69 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB/PP  
 DATUM Geodetic DATE March 30, 1992 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								20	40							60	80	100
185.3	Ground Surface																	
0.0	Peat Occasional Wood Fragments  Dark Brown to Black Very Soft to Soft		1	SS	17													
			2	SS	1													
			3	SS	1													
			4	SS	1													
			5	SS	1													
179.8																		
5.7	Clay to Silty Clay Trace of Sand Brownish Grey to Grey Very Soft to Soft		6	SS	1													
			7	TW	PH													
			8	SS	1													
			9	SS	1													
			10	TW	PH													
173.0																		
12.3	Silty Sand Trace of Clay Grey Very Loose to Compact		11	SS	21													
			12	SS	0 **													
			13	SS	0 **													
168.5																		
16.8	End of Borehole  Probable Silty Sand																	
161.5																		
23.8	End of Cone Test Refusal - Probable Bedrock  * W.L. immediately upon completion of sampling ** Split spoon sank under weight of hammer and rods									120	5cm							

# RECORD OF BOREHOLE No 2-5

1 OF 1

METRIC

W.P. 217-89-00(A) LOCATION Sta. 17+324; C/L S.B.L. ORIGINATED BY JB/DR  
 DIST 11 HWY 89 BOREHOLE TYPE Cone Test COMPILED BY JB/PP  
 DATUM Geodetic DATE March 30, 1992 CHECKED BY PP

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			20	40					
187.5														
0.0	Probable Sand and Gravel Fill						187							
							185							
	Probable Peat						183							
							181							
	Probable Silty Clay						179							
							177							
							175							
							173							
	Probable Silty Sand						171							
168.6							169							
18.9	End of Cone Test Refusal - Probable Bedrock													

# RECORD OF BOREHOLE No 2-6

1 OF 1

METRIC

W.P. 217-89-00(A) LOCATION Sta. 17+195; 14 m Lt. C/L S.B.L.(Revised) ORIGINATED BY DW  
 DIST 11 HWY 69 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB/PP  
 DATUM Geodetic DATE February 5, 1993 CHECKED BY PP

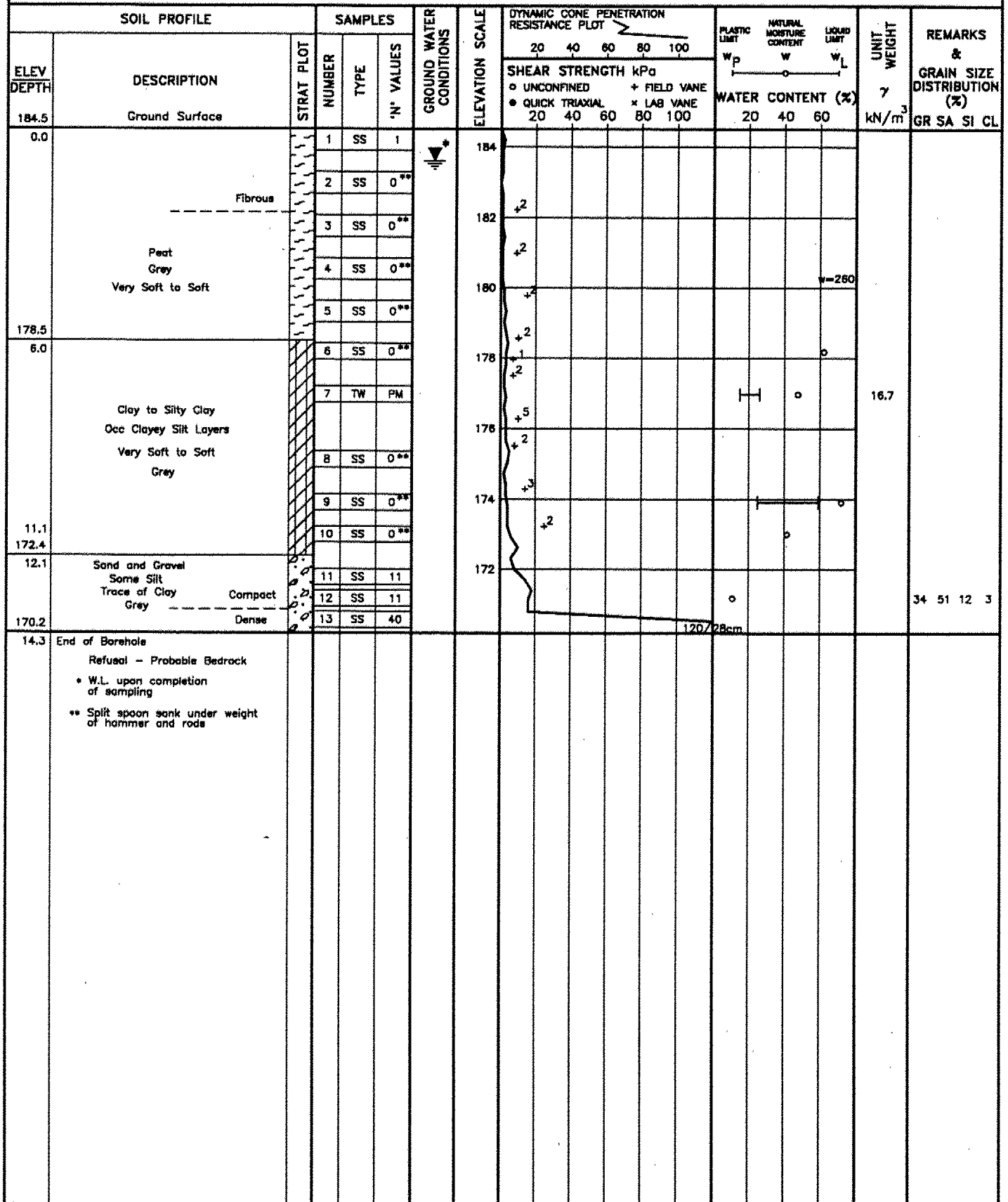
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
184.4	Ground Surface		1	SS	1		184							
0.0			2	SS	1		182							
	Fibrous		3	SS	0**		180							
	Peat		4	SS	0**		178							
	Dark Brown to Black		5	SS	0**									
	Very Soft to Soft		6	SS	0**									
178.4			7	SS	-									
6.0	Clay to Silty Clay													
178.9	Brownish Grey to Grey													
7.5	Very Soft to Firm													
176.0	***													
8.4	End of Borehole													
	Refusal - Probable Bedrock													
	• W.L. immediately upon completion of sampling													
	** Split spoon sank under weight of hammer and rods													
	*** Silty Sand													
	Some Gravel													
	Trace of Clay													
	Compact													

# RECORD OF BOREHOLE No 2-7

1 OF 1

METRIC

W.P. 217-89-00(A) LOCATION Sta. 17+230: C/L S.B.L.(Revised) ORIGINATED BY DW  
DIST 11 HWY 69 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB/PP  
DATUM Geodetic DATE February 5 to 8, 1993 CHECKED BY PP



# RECORD OF BOREHOLE No 2-8

1 OF 1

METRIC

W.P. 217-89-00(A) LOCATION Sta. 17+270: 14 m L/C/L S.B.L. (Revised) ORIGINATED BY JB  
 DIST 11 HWY 69 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB/PP  
 DATUM Geodetic DATE February 4 and 5, 1993 CHECKED BY PP

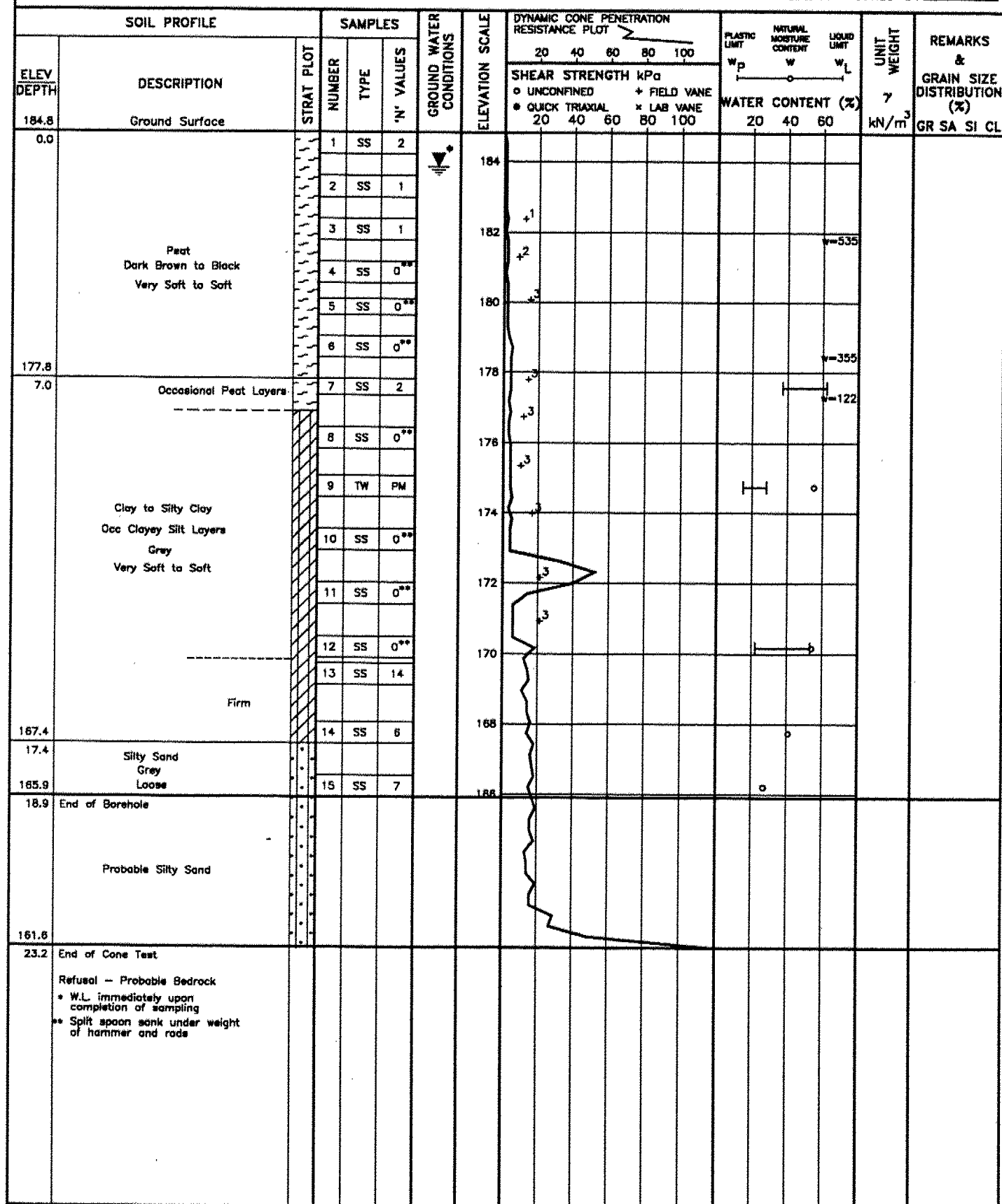
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100					
184.7	Ground Surface													
0.0	Peat		1	SS	1	*	184	+3						
			2	SS	1		182	+2						
	Fibrous		3	SS	1		180	+2						
	Dark Brown to Black		4	SS	0**		178	+4						
	Very Soft to Soft		5	SS	0**		176	+4						
			6	SS	0**		174	+3						
177.8			7	SS	0**		172	+2						
6.9			8	SS	0**		170	+3						
	Clay to Silty Clay		9	TW	PM		168	+3						
	Occ Clayey Silt Layers		10	SS	0**									
	Grey		11	SS	0**									
	Very Soft to Soft		12	SS	0**									
171.4			13	SS	0**									
13.3	Silty Sand		14	SS	2									
	Trace of Clay		15	SS	20									
	Very Loose to Compact													
	Grey													
167.7														
17.0	End of Borehole													
	Refusal - Probable Bedrock													
	* WL Not Observed													
	** Split spoon sand under weight of hammer and rods													

# RECORD OF BOREHOLE No 2-9

1 OF 1

METRIC

W.P. 217-89-00(A) LOCATION Sta. 17+310: C/L S.B.L. ORIGINATED BY DW/JB  
 DIST 11 HWY 69 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB/PP  
 DATUM Geodetic DATE February 5 and 8, 1993 CHECKED BY PP







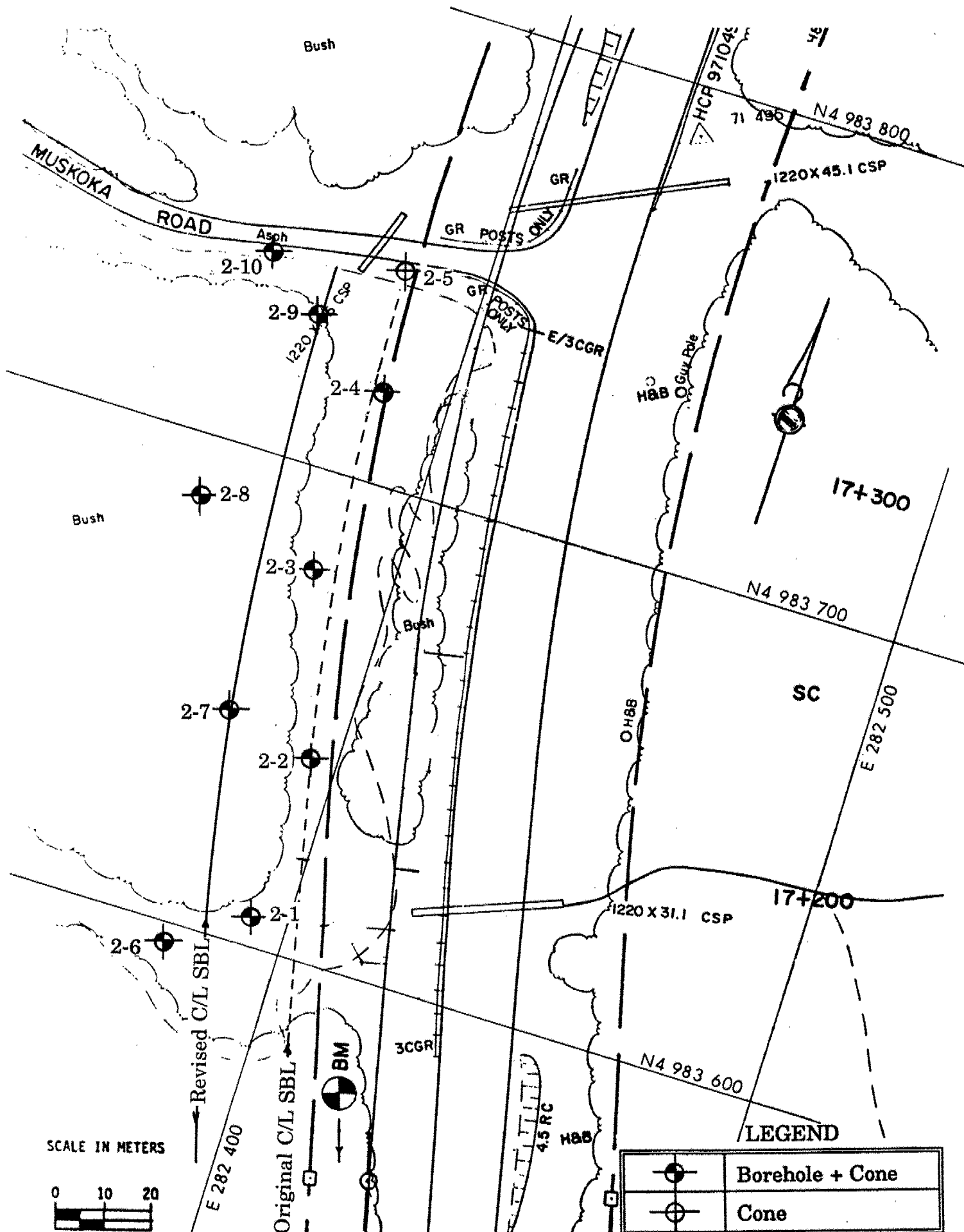
# RECORD OF BOREHOLE No 2-10

1 OF 1

METRIC

W.P. 217-89-00(A) LOCATION Sta. 17+320; 14 m Lt. C/L S.B.L. ORIGINATED BY JB  
 DIST 11 HWY 69 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB/PP  
 DATUM Geodetic DATE February 9, 1993 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								20 40 60 80 100								20 40 60		
185.9	Ground Surface																	
0.0	Sand and Gravel Brown Compact to Very Dense (Fill)		1	SS	18	 28cm 15cm	185											
			2	SS	48													
			3	SS	58													
			4	SS	27													
			5	SS	10													
181.2			6	SS	5													
4.7	Peat Dark Brown to Black Soft to Firm		7	SS	3													
			8	SS	2													
178.6	Clay to Silty Clay Grey Soft to Firm		9	SS	0**													
7.3																		
176.5			10	SS	0		177	2										
9.4	End of Borehole Refusal - Probable Bedrock  * W. L. immediately after completion of sampling  ** Split spoon sank under weight of hammer and rods																	



Dwg. 1

WP 217-89-00(A)



## EXPLANATION OF TERMS USED IN REPORT

**N VALUE:** THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m 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 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS  $\bar{N}$ .

**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/0.3m)	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 (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	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

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

### STRESS AND STRAIN

$u_w$	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
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /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
$S_t$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $\frac{w_L - w_p}{w - w_p}$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL				j	kN/m <sup>2</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE			



# memorandum

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To: Gerry Fletcher  
Contract Administrator  
MTO Construction Office, Huntsville

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

Re: Contract 97-74  
Proposed Hwy 69 Southbound Embankment  
Station 17+150 to 17+335, South of MR. 32  
Hwy 69, District 52, Huntsville

1998 09 15

This memo summarizes our recommendations given in the site meeting. On your request, we visited the above-mentioned site on September 14, 1998. At the time of our site visit the excavation for the embankment construction was taking place in the northwest corner of the proposed embankment at Swamp No. 1. The initial excavation and backfilling with rock fill was in progress to construct a platform parallel to the existing Highway 69. The platform was to be used to carry out the main excavation in strips perpendicular to Highway 69, for the embankment construction. The excavation at the northwest corner had extended slightly below elevation 179m. The soil cutting from the excavation consisted of very soft silty clay mixed with dark brown to black peat.

The Contract Administrator did not have the soil information of the site and therefore, wanted to know if the excavation was to extend below elevation 179m to a firm ground. We advised the Contract Administrator to follow the recommendations given in the Foundation report WP 217-89-00 (A). The recommendation in the Foundation report was for a partial excavation to elevation 179m. The remaining soft material would be displaced by rock fill. Any excavation below Elevation 179m or full excavation of soft material was not recommended in the Foundation report due to possible instability to the existing Highway 69.

We believe the above clarifies the confusion. However, if you have any further questions please advise.

A handwritten signature in black ink, appearing to read 'K. Ahmad', with a stylized flourish at the end.

K. Ahmad, P. Eng  
Foundation Engineer

For

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

cc: J. McDougall  
T. Kazmierowski

file:c:\data\wpwin60\9774 ger.wpd

**From:** Ken Ahmad  
**To:** MTONR.NORTHBAY.McDougal  
**Date:** 5/15/97 12:32pm  
**Subject:** W.P. 183-90-00, Hwy 118 swamp crossing at Port Carling -Reply -Reply

Jim:

You are right the date of our meeting is May 2, 1997. I will make the correction in the memo before I send it to you by mail.

I am forwarding your memo to Tae to comment. Tirechip would not be acceptable at this site because it requires a thick soil cover on it. Also, there is still some environmental concern such as leachate and fire hazard that still requires MOEE approval.

Tae:

Would you please response to Jim on his question of using different type of reinforcement such as geoweb, etc.

Ken

>>>> Jim McDougall 05/15/97 12:10pm >>>>

In the body of your report you have a date June 2, 1997 perhaps this should have been May2,1997? I told George we have abandoned the idea of more property and rock. I tend to agree with you on letting the road settle some more before reexcavating and using a light weight fill, I would prefer sawdust or wood chips over slag but this may not be acceptable to the environmentalists and any future owners. Would you consider rubber tires? and a reinforced geoweb/geogrid cap with a flexible pavement?

>>>> Ken Ahmad 05/15/97 10:59am >>>>

Jim:

Please find the attached recommendations.

Ken

**CC:** Kim

**From:** Ken Ahmad  
**To:** MTONH.HUNTSVILLE.FletchG  
**Subject:** Fw: MTO Contract 97-74, Hwy 69, Pioneer Construction -Forwarded  
-Forwarded -Reply

Gerry:

Further to your E-mail to Tae Kim dated 1998 11 12 and our discussion over the phone today, we understand that the rock fill is placed to the sub-grade level of the proposed southbound lane and now the embankment appears to be stable. In order to expedite settlement and minimize future settlement, the Contractor Tullock Engineering and & Surveying has recommended to place a surcharge of Granular 'B' material 2m high. We understand that as recommended in the Foundation report the rock fill was placed above elevation 179m. Based on the reported amount of settlement in the rock fill it appears that the rock fill has displaced most of the very soft to soft material [Borehole Logs, Foundation Report WP 217-89-00 (A)]. We think that the contractor's proposal for placing the surcharge to expedite settlement is reasonable. However, the surcharge should not be higher than 1m above the final grade of the proposed highway. The height of the surcharge should be 2m or 1m above the final grade of the road, whichever is less. Also, the surcharge should be placed in small lifts of, say 0.5m. If any major settlement or any cracks occur during the surcharge placement, then no more surcharge material should be placed and our office should be contacted for further direction.

Ken Ahmad  
For  
Tae Kim

>>> Tae C. Kim 11/12/98 09:13am >>>  
Dear Ken !

Would you review this surcharge option and discuss with me with your opinion.

Thanks

Tae

**CC:** MTONH.HUNTSVILLE.PepperTo, Kazmiero

FOUNDATION INVESTIGATION REPORT  
For  
A Proposed Embankment  
Along Highway 69 New - From Approximately 15.3 km North of  
M.R. #5, Northerly 13 km  
Location 2 - Station 17+150 to 17+335  
W.P. ~~215-89-00(B)~~ 217-89-00(A)  
Northern Region  
District 11, Huntsville

Introduction

At the request of the Northern Region, Geotechnical Section, a foundation investigation was carried out for a proposed embankment, to be located at the above-captioned site. This report summarizes the factual information obtained from this investigation.

Site Conditions

The site is located immediately to the west of the existing Highway 69 embankment, between Stations 17+150 and 17+335. The site is adjacent to Muskoka Road 32 and located within Lot 19, Concession VII, Gibson Township, District of Muskoka.

Throughout this area, the existing Highway 69 consists of a roadway with single lanes running in both the north and south directions. Through a series of rock cuts and embankments, the roadway traverses undulating topography consisting of rock knolls of gneissic bedrock separated by low swampy or wooded areas. The direction of flow of several rivers, in this area, indicates that drainage is generally towards the west; ie. Georgian Bay.

At this location, Highway 69 has been constructed on top of an embankment, which is about 4.5 to 5.5 m high (ie. to an elevation of 188.4 to 190.5 m). Swampy areas, characterized by cattails and small bushes, are found on both sides of the embankment and the groundwater table is at or near the existing ground surface.

## Fieldwork

The fieldwork was conducted in two stages.

The first stage was carried out, during the period between March 30 and April 2, 1992 and consisted of 4 sampled boreholes (numbered 2-1 to 2-4), which were advanced to depths of 12.8 to 16.8 m. Dynamic cone penetration tests were also carried out adjacent to each of the boreholes and at one other location (ie. 2-5).

The second stage of the investigation, which was conducted between February 4 and 9, 1993, consisted of 5 sampled boreholes (numbered 2-6 to 2-10). Dynamic cone penetration tests were also carried out adjacent to all but one (ie. 2-10) of the boreholes.

All of the boreholes were advanced, using continuous flight, hollow stem augers driven by bombardier-mounted drilling rigs equipped with standard soil sampling equipment.

The locations of all of the borings and cone tests are shown on Drawing No. 2158900B-A.

Wherever soft to firm, cohesive soils were encountered, field vane tests were also carried out.

Soil samples were generally recovered using a 50 mm split spoon sampler, driven into the soil in accordance with the specifications of the Standard Penetration Test (ASTM D 1586). Samples of soft to firm cohesive soils were also obtained in thin-walled (ie. shelly) tubes.

Groundwater levels, were measured in several of the open boreholes, immediately upon completion of sampling. Some of these were left open for at least 24 hours, in order to measure the longer term groundwater conditions.

All boreholes were staked out in the field by the Northern Region Surveys and Plans Office. Small changes in the locations and elevations of the boreholes were determined by our field representatives.

The soil samples, which were obtained in the field, were examined in the laboratory by visual and tactile methods. Moisture content, Unit Weight, Atterberg Limits, Grain Size Distribution, Unconfined Compression and Consolidation tests were carried out on selected soil samples.

## SUBSURFACE CONDITIONS

The subsurface conditions at most of the boreholes, generally consist of a dark brown to black layer of soft to firm, peat or organic clayey silt, from 4.3 to 7.0 m thick, which is in turn, underlain by an extensive deposit of light grey to reddish grey, soft to firm, clay to clayey silt, which reaches depths of up to 17.4 m.

However, at two of the borehole locations, a layer of sand with some gravel to sand and gravel fill from 1.7 to 4.7 m thick overlies the organic soils and soft to firm clay, to clayey silt described above.

At all but one of the borehole locations, a sandy silt to gravelly sand layer, from 0.5 to 11.5 m thick, was found to be sandwiched between the clay to clayey silt layer and the bedrock surface. Gneissic bedrock was found at depths of 8.4 to 23.8 m in the boreholes.

The groundwater table was found to be at or close to the existing ground surface at the time of the investigations with a slight hydraulic gradient to the west.

Details of the subsurface information, obtained from this investigation, are included on the borehole logs and on Figures 1 to 3, at the back of this report. Brief descriptions of the individual soil strata and the groundwater conditions, encountered in the boreholes, are given below.

### Peat/Organic Clayey Silt

A layer of dark brown to black, soft to firm, fibrous peat and organic clayey silt, from 4.3 to 7.0 m thick, was contacted at the ground surface, in all of the boreholes.

Moisture contents, which were carried out on a few samples of soil obtained from this layer, ranged from 175 to 884 (average of 413) percent.

'N'-values, obtained from Standard Penetration tests, carried out in these organic soils, were generally about 1 blow/0.3 m or less (ie. the split spoon sampler simply sank under the weight of the hammer and rods). Field vane tests gave measured shear strengths ranging from 4 to 64 kPa. However, if the two highest and lowest recorded shear strengths are neglected, then the average shear strength was found to be about 15 kPa, indicating soils of generally soft consistency.



### Fill

At Cone Test 2-5 and Borehole 2-10, a sand with some gravel to sand and gravel fill from 1.7 to 4.7 m thick was encountered at the ground surface.

These soils represent a portion of the embankment fills which were used to construct the existing Muskoka Road #32, which crosses the extreme north end of the site.

Although, measured 'N'-values ranged from 10 blows/0.3 m to 56 blows /0.15 m, the higher 'N' values may have been due to cobbles encountered while carrying out the Standard Penetration Test and the soils in this deposit should generally be considered compact.

### Clay to Clayey Silt

At depths of 4.3 to 7.0 m, (or elevations of 177.8 to 180.8 m), all of the boreholes contacted a deposit of reddish or brownish grey to grey, clay to clayey silt, from 1.5 to 10.4 m thick.

Atterberg limits tests, which were carried out on several samples of soil obtained from this deposit, gave liquid limits and plasticity indices ranging from 29 to 62 (average of 48) and 13 to 39 (average of 26) percent, respectively. The results, which are shown on Figure 1, indicate that these soils can generally be classified as clay to clayey silts.

Moisture contents, which were measured in samples obtained from this deposit, ranged from 41 to 122 (average of 74) percent and unit weights ranged from 13.9 to 17.1 (average of 15.8) kN/m<sup>3</sup>

'N'-values, measured in these clayey soils, were generally 1 blow/0.3 m or less (ie. the rods and split spoon sampler simply sank under the weight of the hammer and rods). Field vane tests gave measured shear strengths ranging from 6 to 36 (average of 16) kPa, indicating soils of generally soft, but occasionally firm, consistency.

Consolidation tests were conducted on four representative samples obtained from this deposit. The results, shown on Figure 2 indicate that these clayey soils are slightly preconsolidated with compression indices ranging from 0.16 to 1.84.

### Silty Sand to Gravelly Sand

A deposit of silty sand to gravelly sand, from 2.6 to 11.5 m thick, was encountered at depths of 7.5 to 17.4 m (or elevations of 167.4 to 176.9 m) in all of the boreholes except Borehole 2-10.

Grain size distribution tests, which were carried out on some of the samples obtained from this deposit and shown on Figure 3, measured 0 to 34 percent gravel, 51-89 percent sand, 6 to 44 percent silt and 3 to 9 percent clay-sized particles, respectively. Although some slightly finer zones were found, most of the soils within this deposit ranged from silty sand to gravelly sand.

Moisture contents, measured in samples, obtained from this cohesionless deposit, ranged from 11 to 31 (average of 18) percent.

'N'-values of 0 blows/0.3 m to 40 blows/0.04 m indicate generally loose soils with occasionally compact to very dense zones. It should be noted that 'N-values' of 0 blows/0.3 m, may be unrepresentative and the soil may have become loosened and disturbed, due to conditions of unbalanced hydrostatic head.

### Bedrock

Probable bedrock was encountered in the boreholes and dynamic cone penetration tests, at depths of 8.4 to 23.8 m or elevations of 161.5 to 176.5 m.

Although coring was not carried out at this site, outcrops adjacent to the swamp indicate that the bedrock is likely to be comprised of a hard, granitic gneiss.

### Groundwater Conditions

During the first stage of the investigation (ie. during late March to early April, 1993), the groundwater levels, measured in the open boreholes, immediately upon completion of sampling, (or at least 24 hours after completion), were found to range from the ground surface to depths of 0.2 m beneath the existing ground surface (or elevations of 185.1 to 185.3 m).

However, during the second stage of the investigation (ie. during February 1993), water levels measured in the open boreholes drilled further to the west were found to range from depths of 0.2 to 1.4 m (or elevations of 183.6 to 184.5, immediately after completion of sampling.

The slightly lower water levels (coupled with the fact that the elevation of the ground surface is generally lower), in the boreholes drilled further away from the existing embankment, indicates that the groundwater flow is generally towards the west.

### DISCUSSIONS AND RECOMMENDATIONS

## General

The existing Highway 69, which extends through the area of investigation, is comprised of an existing embankment, from 4.0 to 5.5 m high, with single lanes running in both the north and south directions. The existing pavement appears to be generally good, although occasional cracks were found. These cracks are parallel to the roadway. It should be noted however, that significant settlement has occurred along this stretch of roadway and that a rock berm was placed at the toe of the embankment in an attempt to stabilize it. We understand that the road has since been repaved.

It is proposed to use the existing highway embankment as the new northbound lanes and to construct a new sub-parallel embankment from 3.9 to 5.4 m high, to the west of it. The new embankment will be used to carry the proposed southbound lanes.

It should be noted, however, that when Boreholes 2-1 to 2-5 were drilled (ie. during late March and early April, 1992), the proposed embankment was originally located much closer to the existing embankment.

However, subsequent analyses by this office showed that excavation of the soft soils beneath the proposed embankment could not be carried out without significantly undermining the existing Highway 69 embankment. Therefore the proposed embankment was shifted from 15 to 19 m further to the west. In addition, in order to accomodate a future ramp, it is now also intended to construct an additional 3.5 m wide lane, even further to the west.

As a result of these changes, additional boreholes (ie. 2-6 to 2-10) were drilled in February 1993 to provide supplemental information.

## Design and Construction

Based on the information, which we have obtained from these investigations, it appears that organic soils (up to 7.0 m thick) and other weak cohesive soils extend to depths of up to 17.4 m at this site.

In view of this, several techniques have been investigated in order to construct the embankment to the proposed height. Conventional and non-conventional excavation and displacement as well as the design and construction of embankments using lightweight fill and rockfill are considered.

### **Conventional Excavation and Displacement**

Based on our analyses, if the embankment is constructed of either rockfill or lightweight fill, all of the peat and at least some

portion of the underlying clay must be replaced by rockfill. In all cases, replacement down to an elevation of at least 178.0 m (ie. to a depth of approx. 7.0 to 7.5 m) throughout the full width of the embankment would be required. Although some large backhoes can reach this depth, most could probably only reach to a depth of about 6.5 m.

The additional depth could be reached by other methods such as dredging or by displacement at the toe of the fill. In any case, the replacement of the soft soils should continue for a distance of at least 10 m beyond the toe of the berms in order to maintain adequate stability when the construction is completed.

It should also be noted that replacement of the soft soils also includes any peat and soft clay that still remains beneath the existing embankment for Muskoka Road #32 as indicated by Cone Test 2-5 and confirmed by Borehole 2-10. If not removed, such material would be subject to large additional differential settlement.

The backslope of the excavation should be constructed at 1:1 or flatter from a point at least 10 m beyond the toe of the berm. The subexcavated material should be replaced by rockfill which may be end-dumped below the groundwater table to about 0.3 m above the existing ground surface.

The granular fills excavated for Muskoka Road #32 should not be used for either replacement of the excavated material or in the main construction of the new embankment because of their higher unit weight and lower angle of internal friction. However, some of this material may possibly be reused as granular subbase, provided it is not too silty.

## Embankment Design and Construction

### Conventional Lightweight Fill

In order to maintain adequate stability, the portion of the embankment higher than 0.3 m above the existing ground surface can be constructed of lightweight fill (ie. blast furnace slag). Since our analyses have shown that the stability of the slope is extremely sensitive to the height of the embankment, the finished height must be no greater than 4.5 m above the existing ground surface (Figure 4). Even marginal increases in grade could create an unstable slope.

It should be noted, however, that it appears that the proposed embankment may be as high as 5.5 m at the south end of the site. This is because the surveyors have measured even lower ground surface elevations at the additional boreholes along the new

profile than the elevations which are shown on the old profile with the revised grades. In these areas, the grades would have to be lowered so that the embankment would be no higher than 4.5 m above the existing grade.

In any case, the slopes should be constructed to 2H:1V and 5 m wide mid-height berms will be required which would also be constructed of lightweight fill.

It should also be noted that, in order to construct the embankment in the manner described above, approximately 20,000 m<sup>3</sup> of lightweight fill would be required (assuming the upper 1.2 m of the embankment would consist of the pavement structure). It is recognized that the cost of lightweight fill would be quite significant.

If the embankment height cannot be significantly lowered, constructing the embankment out of lightweight fill would produce the least settlement and problems of routine maintenance of the various methods described.

### Conventional Rockfill

Even if the weak soils were replaced down to an elevation of 178 m, an embankment constructed of rockfill could only realistically be built to a height of about 2.0 to 2.5 m above the existing ground surface. This is because the shear strength of the remaining underlying cohesive soils would still be very low (7.5 to 8 kPa). Even with very large berms, the height of such an embankment would not increase significantly without sacrificing its stability.

However, if the clay can be replaced by rockfill down to an elevation of about 175 m (ie. to a depth of about 10 m), then a rockfill embankment could be built to the proposed grades. This is because the shear strength of the clay increases to at least 14 kPa at this depth. It should be noted that, in the south part of the site, the excavation need only extend to the silty sand to gravelly sand deposit which, in this area, is somewhat higher than elevation 175 m. In any case, the slopes of the embankment should be constructed at 2H:1V with 5 m wide midheight berms.

Although rockfill will provide additional loading, the removal of a greater amount of the clay would ~~likely~~ cause ~~approximately the same~~ settlements and associated maintenance that would be expected if the embankment was constructed of lightweight fill. This, however, would not be the case if the displacement method is used. *only slightly*

### Rockfill and Underfill Blasting

The cost of using conventional techniques to construct embankments up to 4.5 m high using lightweight fill or even rockfill on such weak soils may be prohibitive.

A more practical approach may be to consider blasting methods such as toe-blasting or underfill blasting in order to remove the weak soils to greater depths. For example, a modification of the New Hampshire Method may be used.

Starting from the north and south ends of the embankment where the rock surface is shallowest, the peat and soft clay should be excavated down to the bedrock surface using conventional techniques. When the bedrock surface becomes too deep (ie. beyond the reach of most backhoes), conventional excavation and replacement should continue across the entire site to the maximum practical depth. All of the excavated soils should be cast well away from the new fill to prevent interference of the lateral displacement of the peat and other soft soils. ~~A surcharge of rockfill, about 3.0 m above grade, should be placed on the fill.~~ 76

Holes originating from the toe of the new fill and slightly inclined beneath it should be drilled or jetted to depths of about 10 m at intervals of 3.0 m along either side of the fill.

A second row of holes, a further 3.0 m away, should be drilled or jetted to depths of about 5 m to allow lateral displacement of the liquified materials. Explosives in the shallower set of holes should be detonated slightly after the main set.

The quantity of explosives and the delays required should be decided by a competent blasting contractor. However, if properly carried out, once the charges are detonated, the body of fill should settle into the peat and soft clay, allowing the remainder of the embankment to be constructed to the proposed grades.

~~Such methods~~ <sup>Such methods are likely to be cost effective, but</sup> always leave large pockets of soft clay beneath the fill. Therefore larger differential settlements and longer term maintenance problems should be expected than with the other methods described.

Should this or other blasting methods be considered, we would be happy to discuss them with you.

### Alternative Designs

Alternative designs for embankments involving Tensar reinforcement, are currently being investigated to determine their applicability to this site. However it should be noted that a substantial root mat is not available (mostly small bushes over the site) and such a mat would likely be required. In addition, settlements of the underlying peat and clay would be extremely large and long term (probably years).

In any case, embankments of the order of 200 m long on such soils, have not been attempted using such methods.

## Construction Considerations

### Excavations

It is expected that temporary subexcavations in the peat and soft clay, to depths of up to 7.0 m, will be temporarily stable at only very shallow slopes (ie. 2H:1V) below the water table.

Excavations may be carried out by backhoe, although greater depths can be achieved by dredging. Slumping of the side slopes within the excavation should be avoided. Therefore, backfilling should be carried out as quickly as possible.

### Raising the Grade

Placement and compaction of fill materials, 0.3 m above existing grade, should be carried out according to current MTO practice.

### Settlement

0.7 If the embankment is constructed of lightweight fill, without staged construction and/or surcharging, a total settlement of from ~~1.0~~ to 1.5 m is expected to occur, due to consolidation of the rather extensive deposit of clay to silty clay (up to 5.5 m) that will still remain beneath the embankment, in some areas. It is expected that 90 percent of the consolidation would extend over a period of at least 9 months, following the completion of construction. Similar <sup>settlements</sup> <sup>(up to 2m)</sup> will likely occur if the embankment is conventionally constructed using rockfill. <sup>However,</sup> if displacement or blasting methods are used, somewhat greater settlements should be expected.

In <sup>any</sup> ~~this~~ case, in order to ~~help~~ <sup>over most of its length. However, where</sup> reduce the post construction settlements, of the proposed highway, it is recommended that the embankment be overbuilt by about 0.5 m <sup>with a</sup> ~~until~~ bedrock is encountered in the excavation (ie. between Stations 17+180 and 17+325 - approximately). <sup>high of station</sup> ~~Additional stability analyses (Figure 5) indicate that the slope would be temporarily stable with the added surcharge (ie. to a maximum height of 5.0 m above grade).~~ <sup>the thickness of the surcharge can begin to be reduced in such a way that it is reduced to zero thickness at the ends of the embankment.</sup>

Once the surcharge has been placed, the settlement of the embankments should be allowed to take place over as long a period as possible, but for at least nine months after placement. After that time, the embankment may be bladed off and the slopes flattened to their final grades.

## MISCELLANEOUS

G.I.-30 SEPT. 1976

GEOCRES No. 31E-115DIST. 11 REGION W.P. No. 217-89-00 (B)CONT. No. W. O. No. STR. SITE No. HWY. No. 69 NewLOCATION Embarkment for Swamp  
Crossings Near Marloza Rd 5No of PAGES - (Site #3)=====OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:



FOUNDATION INVESTIGATION REPORT  
For  
Approach Embankments  
Along Highway 69 New - From Approximately 15.3 km North of  
M.R. #5, Northerly 13 km  
Location 3 - Station 20+080 to 20+800  
(Excluding the Musquash River Bridge)  
W.P. 217-89-00(B)  
Northern Region  
District 11, Huntsville

## INTRODUCTION

At the request of the Northern Region, Geotechnical Section, a foundation investigation was carried out for proposed embankments, to be located at the above-captioned site. This report summarizes the factual information obtained from this investigation.

## SITE DESCRIPTION AND GEOLOGY

The site is located from approximately 10 m to up to 60 m west of the centreline of the existing Highway 69 embankment, between Stations 20+080 and 20+800, within Lots 15 and 16, Concession X, Gibson Township, District of Muskoka. The investigation includes the area from about 300 m south to 370 m north of the Musquash River, but excludes the river itself (ie. Stations 20+360 to 20+450) and the structure which will be associated with it.

Throughout this area, the existing Highway 69 consists of a roadway with single lanes, running in both the north and south directions. Through a series of rock cuts and embankments, the highway traverses undulating topography consisting of rock knolls of gneissic bedrock separated by low swampy or wooded areas. Drainage is generally towards the west; ie. Georgian Bay.

At this location, Highway 69 has been constructed on a 4.5 to 6 m high embankment (reaching elevations of 198.7 to 204 m). A two-lane, three span bridge crosses the Musquash River.

The site is essentially divided into two separate sections, both of which are covered by swamps. At Site 3A, on the south side of the river, the area is characterized by cattails and small bushes, with occasional sparse (but dead) trees. The area, however, becomes quite wooded at the extreme south end of the site. On the other hand, Site 3B, on the north side of the river, is quite wooded throughout, with dead or dying trees of up to 200 mm in diameter. In most areas, water is pooled at the surface ie. the groundwater table is at or near the existing ground surface.

## PROCEDURES

The fieldwork was carried out, during the period between April 2 and 14, 1992, and consisted of 9 sampled boreholes (Boreholes 3-2, 3-4, 3-6, 3-8, 3-9, 3-11 and 3-13 to 3-15), which were advanced to depths of 5.2 to 18.9 m, using continuous flight, hollow stem augers driven by a bombardier-mounted drilling rig, equipped with standard soil sampling equipment.

Dynamic cone penetration tests were also carried out adjacent to each of the boreholes (often to greater depth than the borehole itself) and at seven other locations (ie. 3-1, 3-3, 3-5, 3-7, 3-10, 3-12 and 3-16).

Soil samples were recovered using a 50 mm OD split spoon sampler, driven into the soil in accordance with the specifications of the Standard Penetration Test (ASTM D 1586). Field vane tests were also carried out wherever soft to firm cohesive soils were encountered in the boreholes.

Groundwater levels, were measured in several of the open boreholes, immediately upon completion of sampling and some of these were left open for at least 24 hours, in order to measure the longer term groundwater conditions.

The boreholes were staked out in the field by the Northern Region Surveys and Plans Office. Small changes in the locations and elevations of the boreholes were determined by our field representatives.

The soil samples, which were obtained in the field, were examined in the laboratory by visual and tactile methods. Moisture content, Unit Weight, Atterberg Limits and Grain Size Distribution tests were carried out on selected soil samples. A consolidation test was carried out on one of the samples.

## SUBSURFACE CONDITIONS

The subsurface conditions, at the boreholes, generally consist of a thin layer of brownish grey to black, soft to firm, peat, topsoil or organic clayey silt, from 0.2 to 1.2 m thick, which is, in turn, underlain by an extensive deposit of brownish grey to grey, soft to firm (or occasionally stiff), silty clay to clay, which reaches depths of up to 13.7 m.

A layer of brown to grey, loose to compact, sandy silt to well-graded sand, from 0.9 to about 13 m thick was encountered beneath the clay, at depths of 5.3 to 13.7 m, in the boreholes. The probable bedrock surface, was encountered below the sandy silt to sand, at depths of 2.7 to 21.3 m, in the boreholes and cone tests.

As indicated previously, the site can generally be divided into two areas; Site 3A on the north and Site 3B, on the south sides of Musquash River, respectively. Although the subsoils, at both areas, are quite similar, the shear strength of the clay is somewhat higher at Site 3B.

The groundwater table was found to be at or close to the existing ground surface (ie. elevations of 196.4 to 196.8 m).

Details of the subsurface information, obtained from this investigation, are included on the borehole logs and on Figures 1 to 3, at the back of this report. Brief descriptions of the individual soil strata and the groundwater conditions which were encountered in the boreholes, are given below.

#### Organic Soils - Peat/Organic Clayey Silt/Topsoil

A layer of brownish grey to black, soft, peat, organic clayey silt, or topsoil, from 0.2 to 1.2 m thick, was contacted at the ground surface, in all of the boreholes.

#### Fine Sand

Beneath a thin (200 mm thick) layer of topsoil, one of the Boreholes (3-9), contacted a layer of loose to compact, fine to silty fine sand, about 1.7 m thick. This sand, likely represents a portion of the fluvial deposits associated with the river.

A moisture content of 19 percent was measured in one of the soil samples obtained.

Since the ground was likely to be still frozen at this depth, the recorded 'N'-value of 14 blows/0.3 m is probably unrepresentative and the soil is likely to be quite loose.

#### Silty Clay to Clayey Silt

Beneath the organic soils (and, at Borehole 3-9, the fine sand deposit described above), at depths of 0.3 to 1.9 m (or elevations of 195.3 to 196.2 m), all of the boreholes contacted a major deposit of brownish grey to grey, soft to firm, silty clay to clayey silt, from 4.4 to 11.8 m thick.

Atterberg limits tests, which were carried out on several samples of soil obtained from this deposit, gave liquid limits and plasticity indices ranging from 26 to 53 (average of 36) and 12 to 32 (average of 20) percent, respectively. These results, which are shown on Figure 1, indicate soils which can generally be classified as silty clay to clayey silt.

Moisture contents, which were measured in several samples obtained from this deposit, ranged from 20 to 64 (average of 45) percent.

'N'-values, measured in these clayey soils, were generally less than 4 blows/0.3 m. At several sampling intervals, the rods and split spoon sampler simply sank under their own self weight. It should be noted that, higher 'N'-values which were recorded in the upper portion of the deposit (ie. at Boreholes 3-2, 3-8, 3-9 and 3-11), are likely to be unrepresentative, since the ground was probably still frozen at those shallow depths, when testing was carried out.

Field vane shear tests gave measured shear strengths ranging from 11 to 120 kPa. South of Musquash River (ie. at Site 3A), shear strengths averaged about 14 kPa. However, somewhat higher values (all greater than 20 kPa), were measured in the clay to the north of the Musquash River (Site 3B). In any case, these results indicate soils of generally soft to firm consistency.

A consolidation test was carried out on a sample of soil obtained from this deposit. The results, shown on Figure 2, indicate that this soil sample was lightly preconsolidated with a compression index of 0.96.

#### Sandy Silt to Medium Sand

A deposit of brown to grey, loose to compact, sandy silt to medium sand, approximately 0.3 to 13.0 m thick, was encountered at depths of 4.9 to 13.7 (or elevations of 183.5 to 191.8 m), in the boreholes.

Grain size distribution tests, carried out on samples obtained from this deposit, and shown on Figure 3, indicate sand, silt and clay-sized particles ranging from 26 to 92, 6 to 68 and 2 to 11 percent, respectively.

Moisture contents of 16 to 26 (average of 22) percent were measured, in several samples obtained from this cohesionless deposit.

'N'-values ranging from 0 to 35 blows/0.3 m indicate generally loose to compact soils with occasional denser zones. The lower 'N'-values', particular those of 0 (ie. the split spoon sampler and rods simply sank under their own self weight) to 3 blows/0.3 m are likely to be unrepresentative. Dynamic cone penetration tests indicate that, in most cases, the soil probably became loosened and disturbed, due to conditions of unbalanced hydrostatic head.

### Bedrock

Probable bedrock was encountered in the boreholes and dynamic cone penetration tests, at depths of 2.7 to 21.0 m (or elevations of 175.0 to 195.0 m). It should be noted, however, that, in some areas, during cone penetration tests, the cone appeared to be skipping off of the steeply-inclined bedrock surface. Therefore the elevations of the bedrock surface, should only be considered to be approximate.

Outcrops adjacent to the swamp, and in the immediate area, indicate that the local bedrock is comprised of a hard, granitic gneiss.

### Groundwater Conditions

The groundwater levels, measured in the open boreholes immediately upon completion of sampling, (or at least 24 hours after completion), were generally found to range from 0 to 0.2 m beneath the existing ground surface or elevations of 196.4 to 196.8 m.

It should be noted, however, that the water level measured in Borehole 3-9, was found to be at a depth of 1.9 m (Elevation of 195.3 m), upon completion of sampling. It is likely, however, that the water level, measured in this borehole, does not reflect the true groundwater table since it did not have sufficient time to adequately stabilize.

## DISCUSSIONS AND RECOMMENDATIONS

### General

The existing Highway 69, which extends through the area of investigation, is comprised of an embankment, from 4.5 to 6 m high, with single lanes running in both the north and south directions. The existing pavement appears to be in relatively good condition, although we understand that, in several areas along Highway 69, the highway has been repaved.

It is proposed to use the existing highway embankment as the northbound lanes and to construct a new sub-parallel embankment, from 2.5 to 4 m high, to the west of the existing embankment, which will be used for the new southbound lanes.

As noted previously, the Musquash river divides the site into two separate sections. On the south side of the river, over the south half of Site 3A, the centreline of the proposed embankment will generally be about 20 m to the west of the new median. However, about 200 m south of Musquash River, the proposed embankment begins to curve to the west, away from the existing one. This also continues on the north side of the river, where, the new embankment will be a maximum of about 60 m away from the existing one, about 200 m north of the river. To the north of this, the embankment once again, begins to curve back (ie. to the east), towards the existing embankment.

### Design

In order to construct the embankments to their proposed heights, it will be necessary to excavate all of the peat and a portion of the underlying clay down to an elevation of about 194.5 m.

In order to maintain adequate stability, the slopes of a rockfill embankment, which is constructed to the south of Musquash River (ie. Site 3A), should be no steeper than 2H:1V with a 7 m wide one-third height berm.

However, at Site 3B, the new embankment may have slopes constructed as steeply as 1.5H:1V, and, in this case, berms are not required.

## Construction Considerations

### Excavations

It is expected that temporary subexcavations for the peat and soft clay, to depths of up to 2 m, will be temporarily stable at slopes of 2H:1V. Subexcavation and backfilling should be carried out concurrently and under water, if necessary.

### Raising the Grade

Rockfill or other fills placed below the groundwater table, may be end-dumped. However, once the material is 0.3 m above the groundwater table, placement and compaction of the fill materials should be carried out according to OPSS standards and MTO practice.

### Settlement

Based on a consolidation test, carried out on one sample from this site, it appears that, at Sites 3A and 3B, settlements of about 0.4 to 0.5 m are expected to occur, primarily due to consolidation of the silty clay deposit and to a lesser extent due to compression of the loose underlying sand. It is expected that 90 percent of the consolidation settlement will extend over a period of about three months, following the completion of construction.

Therefore, to reduce the post construction settlements, of the proposed highway, it is recommended that the embankments to the south and north of the abutments of the proposed bridge be overbuilt by about 0.5 m between Stations 20+160 and 20+364 and 20+420 and 20+770. This excludes the centre span of the proposed structure over the Musquash River (ie. Stations 20+364 to 20+420), the details of which are given in the report under Ref. No. W.P. 208-90-01). It should be noted, however, that in our report, Ref. No. W.P. 208-90-01, it has been recommended to move the south abutment about 5 m further to the south. If this is the case, then the surcharging should be between Stations 20+160 and 20+359, on the south side of the proposed bridge. In any case, the surcharge should be tapered to zero thickness, where bedrock is encountered in the excavation (ie. at approximately Stations 20+080 and 20+780).

The settlement of these embankments should be allowed to take place for as long a period as possible but for a minimum of at least four months. After that time, the embankments may be bladed off and the slopes flattened to their final grades. The surcharge material may consist of either granular material or rockfill.

#### MISCELLANEOUS

The field investigation was supervised by Mr. J. Blair of the Foundation Design Section and Mr. Dan Rothwell of the Northern Region Geotechnical Section, using equipment owned and operated by Atcost Soil Drilling Inc.

This report was written by Mr. J. Blair, Project Foundation Engineer, reviewed by Mr. D. Dundas, Senior Foundation Engineer and approved by Mr. M. Devata, Chief Foundation Engineer.



*John A. Blair*

J. A. Blair, P.Eng.  
Project Foundation Engineer

*M. Devata*

M. Devata, P.Eng.  
Chief Foundation Engineer



## EXPLANATION OF TERMS USED IN REPORT

**N VALUE:** THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m 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 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS  $\bar{N}$ .

**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/0.3m)	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

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

### STRESS AND STRAIN

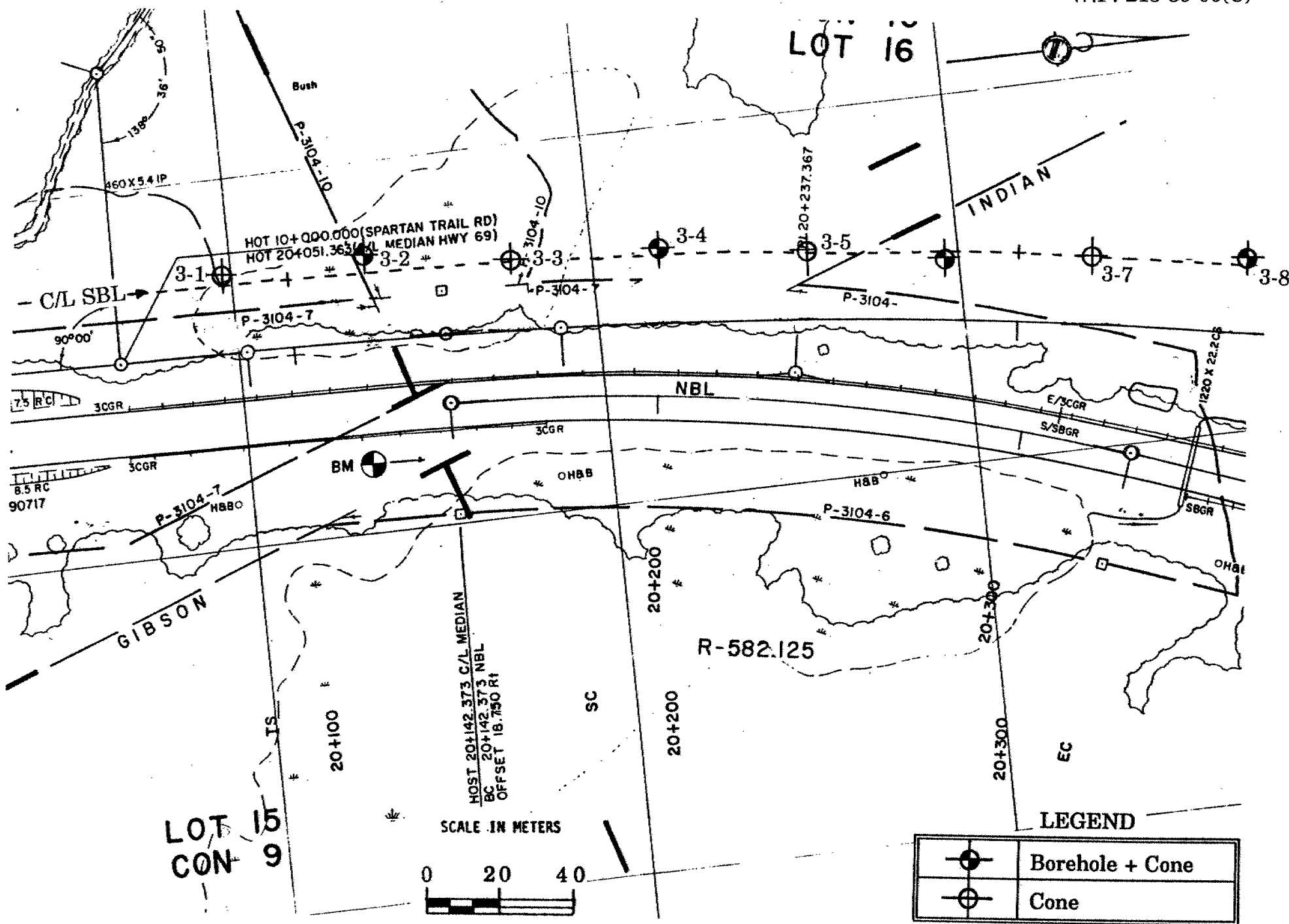
$u_w$	kPa	PORE WATER PRESSURE
$r_u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS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

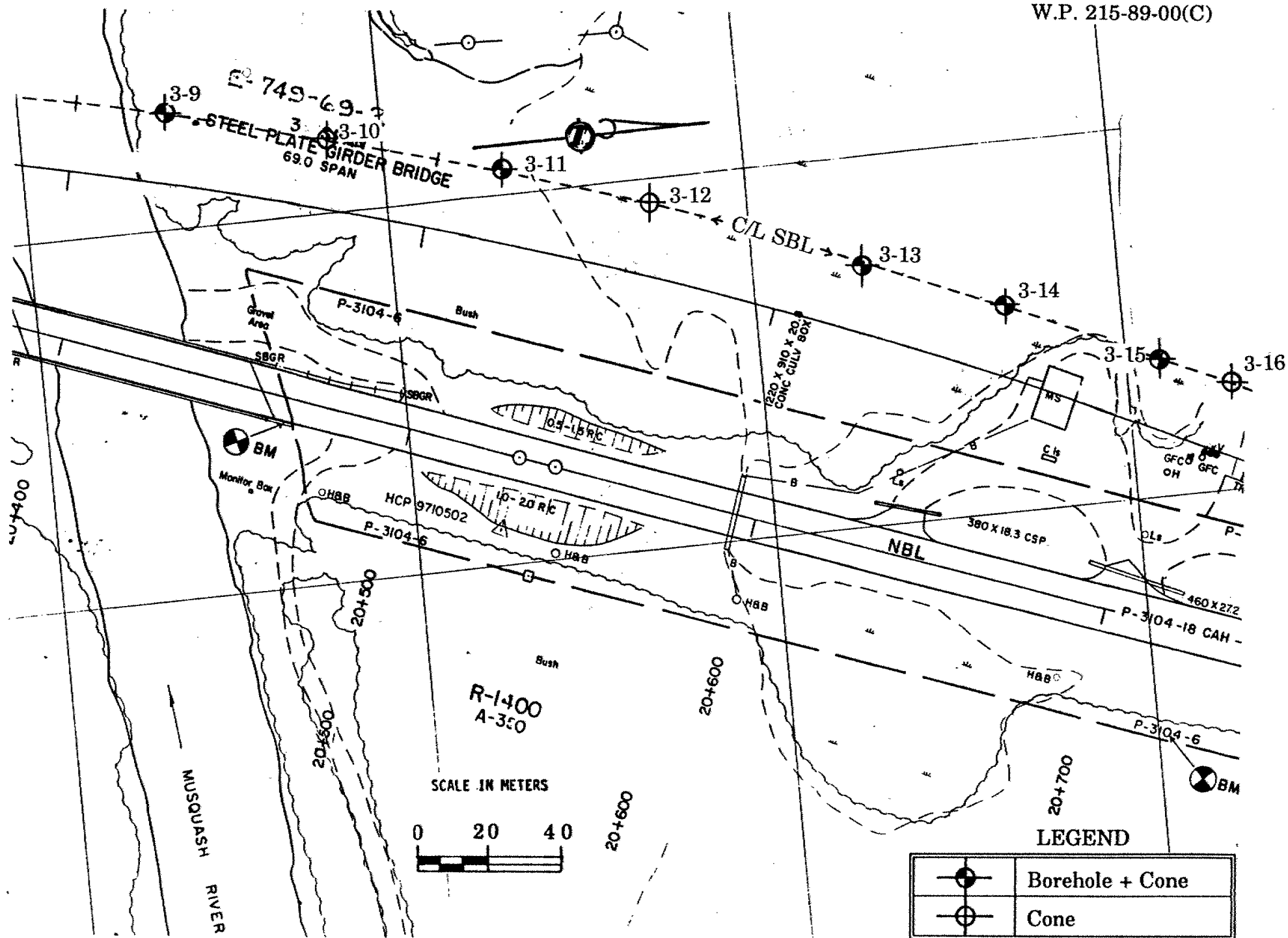
### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{v0}$	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
$S_t$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	kn/m <sup>3</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						





RECORD OF BOREHOLE No 3-1

1 OF 1

METRIC

W.P. 215-89-00(C) LOCATION Sta. 20+083; 3 m LL. C/L S.B.L. ORIGINATED BY JB

DIST 11 HWY 59 BOREHOLE TYPE Cone Test COMPILED BY JB

DATUM Geodetic DATE April 14, 1992 CHECKED BY DD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w <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								
197.7	Ground Surface												
0.0	Probable Feet												
	Probable Silty Clay to Clayey Silt						197						
195.0													
2.7	End of Cone Test Refusal - Probable Bedrock							50/8cm					

## METRIC

SOIL PROFILE			SAMPLES		GROUND WATER • CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20			
197.0	Ground Surface						SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE • QUICK TRIAXIAL * LAB VANE 20 40 60 80 100		7	GR SA SI CL	
							WATER CONTENT (%)		20	40	60
									kn/m <sup>3</sup>		

[illegible]

+3, x<sup>5</sup>: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No 3-3

1 OF 1

METRIC

W.P. 215-89-00(C) LOCATION Sta. 20+160; C/L S.B.L. ORIGINATED BY DR  
 DIST 11 HWY 69 BOREHOLE TYPE Cone Test COMPILED BY JB  
 DATUM Geodetic DATE April 13, 1992 CHECKED BY DD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w <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								
196.7	Ground Surface												
0.0	Probable Peat												
	-----												
	Probable Silty Clay to Clayey Silt												
	-----												
	Probable Sandy Silt to Sand												
181.5													
15.2	End of Cone Test Refusal - Probable Bedrock								120/25cm				

# RECORD OF BOREHOLE No 3-4

1 OF 1

METRIC

W.P. 215-89-00(C) LOCATION Sta. 20+200; 2.0 m Lt. C/L S.B.L. ORIGINATED BY DR  
 DIST 11 HWY 69 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB  
 DATUM Geodetic DATE April 6/7, 1992 CHECKED BY DD

SOIL PROFILE			SAMPLES			GROUND WATER * CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
186.5	Ground Surface												
0.0	Peat Black, Soft		1	SS	3		196						
1.2	Organic Pockets		2	SS	2		194						
	Silty Clay to Clayey Silt		3	SS	1		192						
	Grey		4	SS	1		190						
190.9	Soft to Firm		5	SS	1		188						
5.6			6	SS	5		186						
	Trace of Clay		7	SS	3		184						
	Loose		8	SS	13		182						
	Compact		9	SS	12		180						
	Sandy Silt to Fine Sand		10	SS	10		178						
	Brownish Grey to Grey		11	SS	12								
			12	SS	13								
177.6			13	SS	11								
18.9	End of Borehole												
	Probable Sandy Silt to Fine Sand												
175.5													
21.0	End of Cone Test Refusal - Probable Bedrock												
	* W.L. on April 14, 1992												

# RECORD OF BOREHOLE No 3-5

1 OF 1

METRIC

W.P. 215-89-00(C) LOCATION Sto. 20+240; C/L S.B.L. ORIGINATED BY DR  
 DIST 11 HWY 69 BOREHOLE TYPE Cone Test COMPILED BY JB  
 DATUM Geodetic DATE April 7, 1992 CHECKED BY DD

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			20	40					
196.5	Ground Surface													
0.0	Probable Peat -----													
	Probable Silty Clay to Clayey Silt -----													
	Probable Sandy Silt to Fine Sand													
181.9														
14.6	End of Cone Test Refusal - Probable Bedrock													



# RECORD OF BOREHOLE No 3-6

1 OF 1

METRIC

W.P. 215-89-00(C) LOCATION Sta. 20+280; 2 m Rt. C/L S.B.L. ORIGINATED BY JB  
 DIST 11 HWY 69 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB  
 DATUM Geodetic DATE April 10, 1992 CHECKED BY DD

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					
186.5	Ground Surface												
0.0	Peat/Organic Clayey Silt												
195.6	Brownish Gray to Black, Soft												
0.9			1	SS	3								
			2	SS	2								
	Stiff Firm		3	SS	1								
	Silty Clay to Clayey Silt, Some Sand		4	SS	0 **								
			5	SS	1								
	Grey		6	SS	1								
188.6			7	SS	0 **								
7.9			8	SS	1								
	Sand and Silt to Silty Fine Sand		9	SS	10								
			10	SS	0 **								
	Grey		11	SS	2								
	Loose to Compact Dense to Very Dense												
181.6													
14.9	End of Borehole Auger Refusal - Probable Bedrock												
	* W.L. immediately upon completion of sampling												
	** Split spoon sank under weight of hammer and rods												

# RECORD OF BOREHOLE No 3-7

1 OF 1

METRIC

W.P. 215-89-00(C) LOCATION N 4 986 638.4 E 282 847.2 ORIGINATED BY JB  
 DIST 11 HWY 69 BOREHOLE TYPE Cone Test COMPILED BY JB  
 DATUM Geodetic DATE April 10, 1992 CHECKED BY DD

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 7 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20					
186.2	Ground Surface												
0.0	Probable Peat												
	Probable Silty Clay to Clayey Silt												
	Probable Sand and Silt to Silty Fine Sand												
185.2													
11.0	End of Cone Test Refusal - Possible Boulder or Bedrock												

# RECORD OF BOREHOLE No 3-8

1 OF 1

METRIC

W.P. 215-89-00(C) LOCATION Sta. 20+362; Off. 2 m Lt. C/L S.B.L. ORIGINATED BY JB  
 DIST 11 HWY 69 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB  
 DATUM Geodetic DATE April 9, 1992 CHECKED BY DD

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						
186.6	Ice							20 40 60 80 100						
0.0	0.3m Ice, 0.2m Peat							20 40 60 80 100						
	Silty Clay to Clayey Silt		1	SS	15		196							
	Trace of Sand		2	SS	1		194							
	Grey		3	SS	2									
	Soft to Firm		4	SS	0**		192							
			5	TW	PH									
			6	SS	1		190							
189.6														
7.0	Trace of Clay		7	SS	0**		188							
			8	SS	0**									
	Sandy Silt to		9	SS	4		186							0 26 68 6
	Fine Sand, Some Silt		10	SS	0**		184							
	Grey to Brownish Grey		11	SS	0**		182							0 91 6 3
	Loose to Compact		12	SS	13									
	Dense to Very Dense		13	SS	77	/15cm	180							
179.6														
178.8	Probable Bedrock ***													
17.8	End of Borehole													
	* W.L. on April 10, 1992 ** Split spoon sank under weight of hammer and rods *** Auger and Cone likely skipping off rock surface													

RECORD OF BOREHOLE No 3-9

1 OF 1

METRIC

W.P. 215-89-00(C) LOCATION Sta. 20+424; C/L S.B.L. ORIGINATED BY JB  
DIST 11 HWY 69 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB  
DATUM Geodetic DATE April 6, 1992 CHECKED BY DD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT 7 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
197.2	Ground Surface													
0.0	Fine to Silty Fine Sand Yellowish Brown Loose to Compact		1	SS	14		196							
195.3			2	SS	18		194							
1.9	Silty Clay to Clayey Silt, Trace Sand		3	SS	4		192							
			4	SS	4		190							
	Brownish Grey to Grey		5	SS	0 **		188							
			6	SS	1		186							
	Firm to Stiff		7	SS	0 **		184							
			8	SS	1		182							
			9	SS	0 **									
			10	SS	2									
185.1														
12.1	Medium Sand		11	SS	13									
	Brown													
	Compact to Dense		12	SS	35									5 89 (6)
180.0			13	SS	57	/15cm								
17.2	End of Borehole ***													
179.2														
18.0	End of Cone Test *** Probable Bedrock - Cone possibly skipping off rock surface  * W.L. immediately upon completion of sampling ** Split spoon sank under weight of hammer and rods													

# RECORD OF BOREHOLE No 3-10 1 OF 1 METRIC

W.P. 215-89-00(C) LOCATION Sta. 20+470; C/L S.B.L. ORIGINATED BY JB  
 DIST 11 HWY 69 BOREHOLE TYPE Cone Test COMPILED BY JB  
 DATUM Geodetic DATE April 7, 1992 CHECKED BY DD

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	20					
196.7	Ground Surface												
0.0	----- Probable Peat ----- Probable Fine Sand   Probable Silty Clay to Clayey Silt   ----- Probable Fine to Medium Sand												
189.1	End of Cone Test Refusal - Probable Bedrock												

# RECORD OF BOREHOLE No 3-11

1 OF 1

METRIC

W.P. 215-89-00(C) LOCATION Sta. 20+520; Off. 1 m Lt C/L S.B.L. ORIGINATED BY JB  
 DIST 11 HWY 69 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB  
 DATUM Geodetic DATE April 7, 1992 CHECKED BY DD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	W <sub>P</sub> W W <sub>L</sub>	WATER CONTENT (%)		
186.5	Ground Surface												
0.0	Topsoil		1	SS	13								
	Sand Layer		2	SS	2								
	Silty Clay to Clayey Silt		3	SS	1								
	Brownish Grey to Grey		4	SS	1								
	Firm to Stiff		5	SS	1								
			6	SS	1								
189.4	Sandy		7	SS	27								
7.2	Silty Fine Sand		8	SS	46								
188.5	Grey, Compact to Dense												
8.1	End of Borehole Refusal - Probable Bedrock  + W.L. on April 14, 1992												

# RECORD OF BOREHOLE No 3-12 1 OF 1 METRIC

W.P. 215-89-00(C) LOCATION Sta. 20+560; C/L S.B.L. ORIGINATED BY JB  
 DIST 11 HWY 69 BOREHOLE TYPE Cone Test COMPILED BY JB  
 DATUM Geodetic DATE April 7, 1992 CHECKED BY DD

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	20 40 60 80 100					
196.6	Ground Surface												
0.0	Probable Peat												
	Probable Silty Clay to Clayey Silt												
	Probable Sandy Silt to Fine Sand												
178.6	End of Cone Test Refusal - Probable Bedrock												

# RECORD OF BOREHOLE No 3-13

1 OF 1

METRIC

W.P. 215-89-00(C) LOCATION Ste. 20+620; Off. 1 m Rt. C/L S.B.L. ORIGINATED BY JB/DR  
DIST 11 HWY 59 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB  
DATUM Geodetic DATE April 8, 1992 CHECKED BY DD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT		UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	W <sub>p</sub>	W		
186.6	Ground Surface												
0.0	Peat, Black, Soft												
0.6	Silty Clay to Clayey Silt  Grey  Firm to Stiff	Sandy	1	SS	2		196						
			2	SS	2		194						
			3	SS	1		192						
			4	SS	0**		190						
			5	SS	0**		188						
188.8			6	SS	3		186						
7.8	Sandy Silt to Fine Sand  Brownish Grey to Grey  Loose to Compact		7	SS	13		184						
			8	SS	2								
			9	SS	6								
182.9	End of Borehole												
13.7	Probable Sandy Silt to Fine Sand												
175.3													
21.3	End of Cone Test Refusal - Probable Bedrock												
	* W.L. on April 14, 1992 ** Split spoon sank under weight of hammer and rods												



# RECORD OF BOREHOLE No 3-14 1 OF 1 METRIC

W.P. 215-89-00(C) LOCATION Ste. 20+660; C/L S.B.L. ORIGINATED BY DR  
DIST 11 HWY 69 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB  
DATUM Geodetic DATE April 9, 1992 CHECKED BY DD

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				
186.6	Ground Surface															
0.0	Peat, Black															
0.8	Silty Clay to Clayey Silt Light Grey to Grey Firm to Stiff		1	SS	4											
			2	SS	1											
	Silt Layer		3	SS	2											
			4	SS	0 **											
			5	SS	0 **											
	Sandy		6	SS	6											
187.9			7	SS	6											
8.7	Sandy Silt to Fine Sand, Trace Silt Brown to Brownish Grey Loose		8	SS	5											
			9	SS	0 **											
			10	SS	0 **											
180.8																
15.8	End of Borehole  Probable Sandy Silt to Fine Sand															
175.0																
21.6	End of Cone Test Refusal - Probable Bedrock  * W.L. on April 14, 1992 ** Split spoon sank under weight of hammer and rods															

# RECORD OF BOREHOLE No 3-15 1 OF 1 METRIC

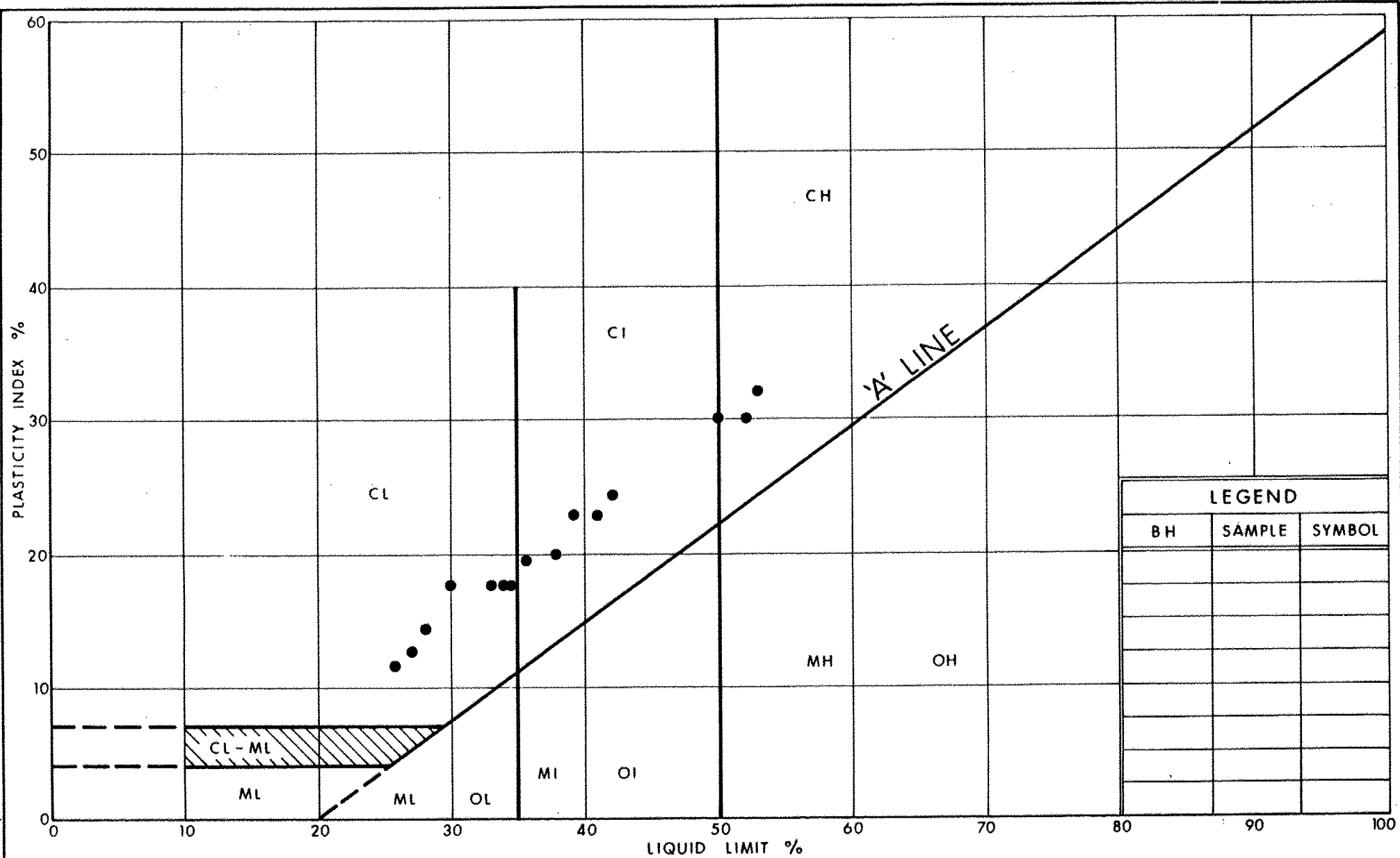
W.P. 215-88-00(C) LOCATION Sta. 20+700; Off. 1 m Rt. C/L S.B.L. ORIGINATED BY DR  
DIST 11 HWY 68 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JB  
DATUM Geodetic DATE April 10, 1992 CHECKED BY DD

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	20 40 60		
196.7	Ground Surface												
0.0	Peat Block, Soft												
195.9													
0.8	Silty Clay to Clayey Silt		1	SS	4								
	Brownish Grey		2	SS	4								
	Stiff to Firm		3	SS	2								
191.5	--- Sand Layer		4	SS	22								
5.2	End of Borehole Refusal - Probable Bedrock • W.L. on April 14, 1992												

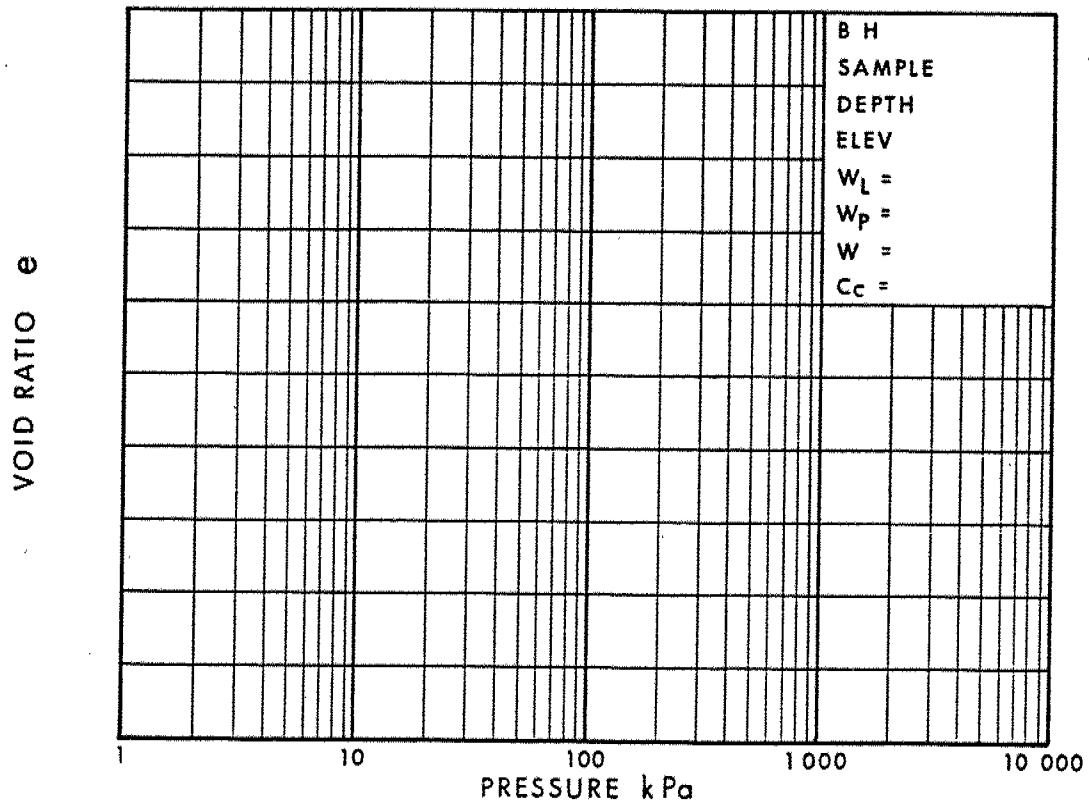
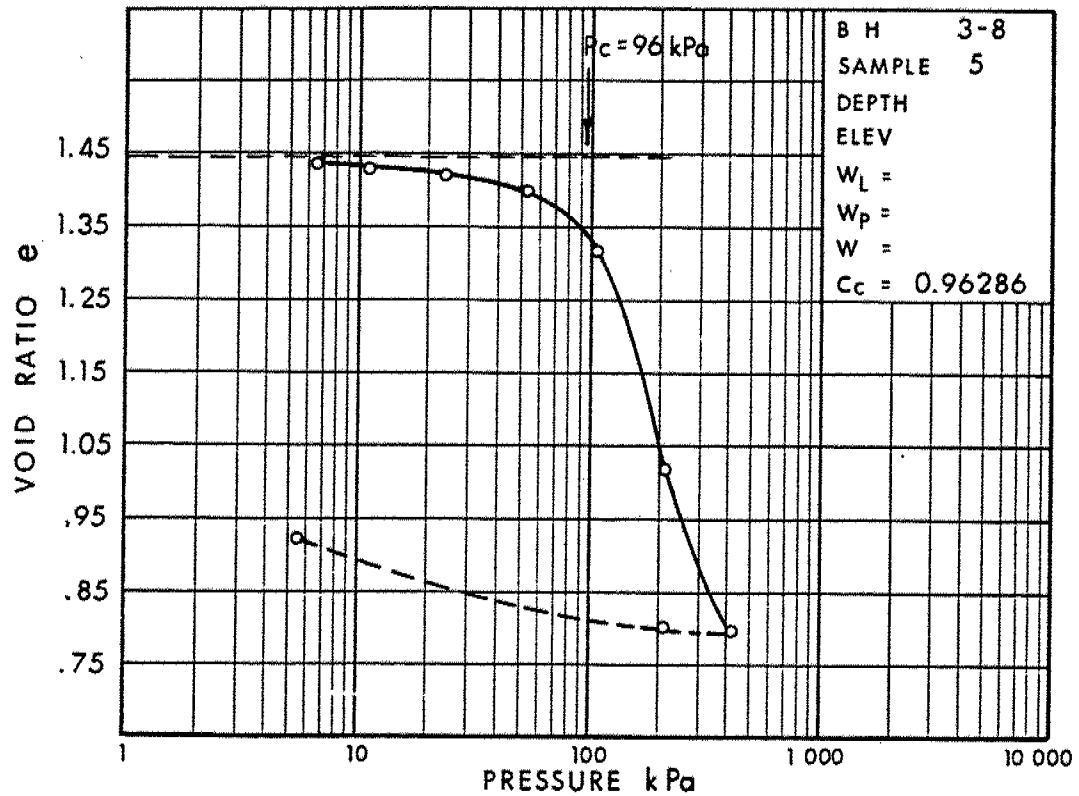
# RECORD OF BOREHOLE No 3-16 1 OF 1 METRIC

W.P. 215-89-00(C) LOCATION Sta. 20+720; C/L S.B.L. ORIGINATED BY JB  
 DIST 11 HWY 69 BOREHOLE TYPE Cone Test COMPILED BY JB  
 DATUM Geodetic DATE April 14, 1992 CHECKED BY DD

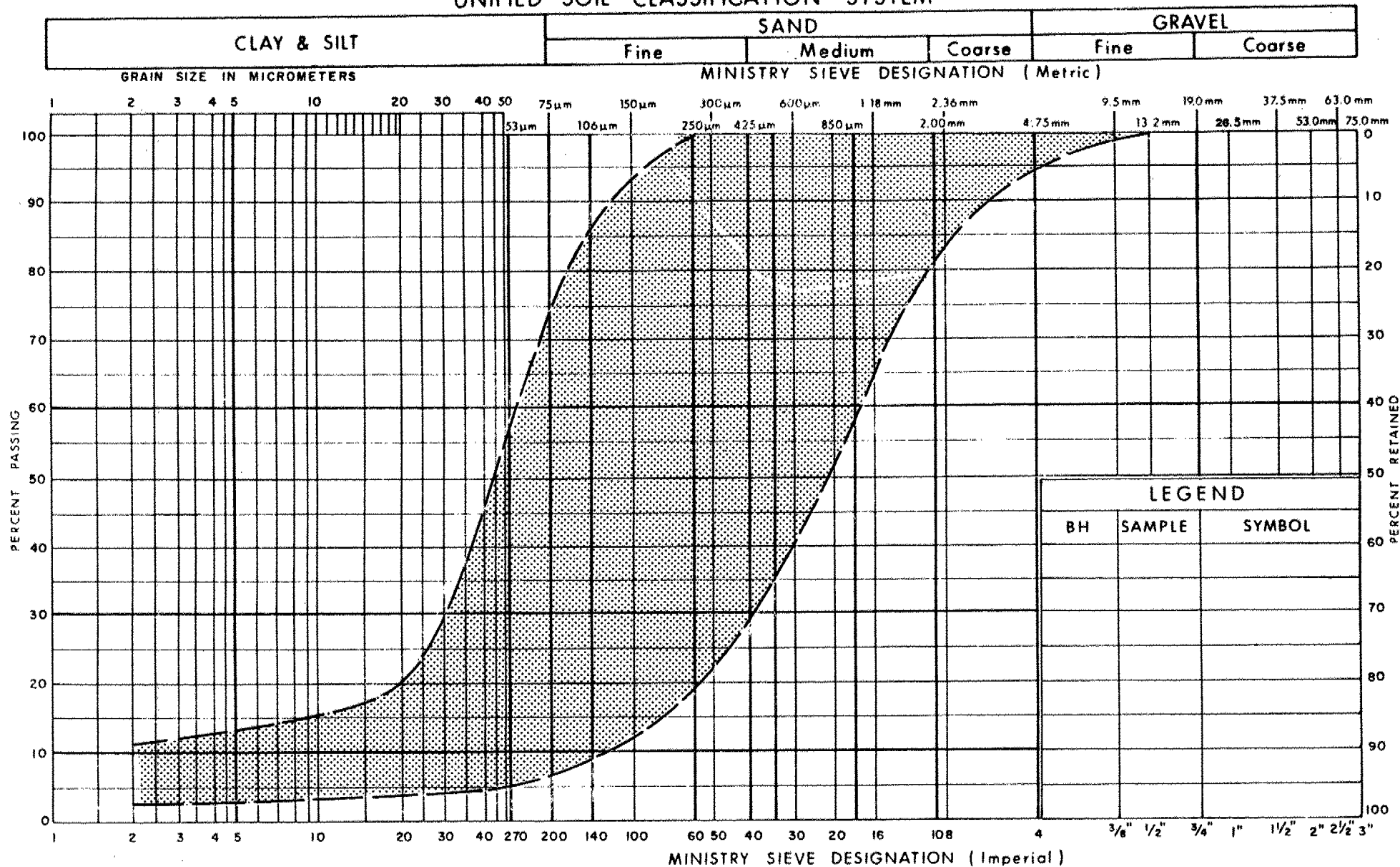
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT 7 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
187.0	Ground Surface													
0.0	Probable Peat													
	Probable Silty Clay to Clayey Silt													
192.8														
4.8	End of Cone Test Refusal - Probable Bedrock													



# VOID RATIO - PRESSURE CURVES



## UNIFIED SOIL CLASSIFICATION SYSTEM

Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION  
SANDY SILT TO MEDIUM SAND

FIG No 3

W P 215-89-00 (C)

G.I.-30 SEPT. 1976

GEOCRES No. 31E-115DIST. 11 REGION W.P. No. 217-89-00 (C)CONT. No. W. O. No. STR. SITE No. HWY. No. 69 NewLOCATION Embarkment for Swamp  
Crossings Near Muskogee Rd. #5No. of PAGES - (Sick 6)

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

File

# MEMORANDUM

(416)235-3731

19940131

To: Mr. J.I. McDougall, P.Eng.  
Head, Geotechnical Section  
Northern Region

Att: Mr. Ron Purdy

From: Foundation Design Section  
Room 315, Central Building  
Downsview, Ontario

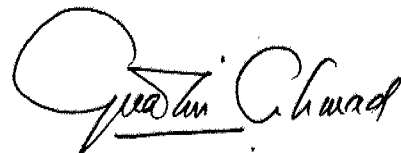
Re: A Proposed Embankment  
Along Highway 69 New From Approximately 15.3 km  
North of M.R. #5, Northerly 13.0 km  
Location 6 - Station 17+950 to 18+180  
W.P. (215-89-00(E)) Geocres No. 31E-115  
Northern Region *NEW WP 217-89-00(c)*  
District 11, Huntsville

The attached report provides recommendations for a proposed embankment for the new southbound lanes along Highway 69 between Stations 17+950 and 18+180.

We believe that this report will be adequate for your purposes. However, should you have any questions regarding it, please do not hesitate to contact this office.

## Distribution

J. McDougall (2)  
P. Furst  
G. Todd  
S. Wilson (2)  
M. Holowka  
G.E. Green  
E.A. Joseph  
D.J. Armatage (Only)  
F. Bacchus (Cover Only)  
File ✓



Ken S.Q. Ahmad, P. Eng.  
Project Foundation Engineer

For

D.H. Dundas, P. Eng.  
Chief Foundation Engineer (Acting)

FEB 01 1994



FOUNDATION INVESTIGATION REPORT  
For  
A Proposed Embankment  
Highway 69 New - From Approximately 15.3 km North of  
M.R. #5, Northerly 13 km  
Location 6 - At Station 17+950 to 18+180  
W.P. 215-89-00(E), Geocres 31E-115  
Northern Region  
District 11, Huntsville

INTRODUCTION

The Northern Region, Geotechnical Section, requested that this office provide recommendations for a proposed embankment, to be located at the above-captioned site. This letter summarizes these recommendations, which have been based on a foundation investigation carried out by a local Consultant.

SITE DESCRIPTION AND GEOLOGY

The site is located immediately to the west of the existing Highway 69 embankment, between Stations 17+950 and 18+180, or about 650 m north of Muskoka Road #32, within Lot 18, Concession VIII, Gibson Township, District of Muskoka.

Throughout this area, the existing Highway 69 consists of a roadway with single lanes in both the north and south directions. Through a series of rock cuts and embankments, the roadway traverses undulating topography consisting of rock knolls of gneissic bedrock separated by low swampy or wooded areas. The flow of several rivers, in this area, indicates that drainage is generally towards Georgian Bay, to the west.

At this site, the existing Highway 69 has been constructed on top of 3.5 m high embankment (ie. to an elevation of 195.5 to 196.5 m). A swampy area, characterized by cattails and grasses, covers most of the area, immediately to the west of the existing embankment. However, the northern portion of this area is partially treed.

In this area, the groundwater table is at or above the existing ground surface.

## SUBSURFACE CONDITIONS

A foundation investigation, dated March 10, 1992, was carried out by Dominion Soil Investigation Inc. under their Ref. No. 91-9-5. Based on this report, it appears that the subsurface conditions, consist of up to 1.2 m of peat (or muck), which is underlain by an extensive deposit of silty clay to clay, reaching depths of up to 5.6 m. Details of the subsurface conditions are included on the borehole logs, which are given in the Appendix.

Based on several vane tests, which were carried out at the site, it appears that the shear strength of the clay, throughout the middle of the site (ie. between Stations 18+040 and 18+080) is only about 9 kPa down to an elevation of about 188.5 m. However, below this, the shear strength generally increases to more than 20 kPa.

The groundwater table is at or slightly above the ground surface (ie. the water is ponded above the ground surface).

## DISCUSSIONS AND RECOMMENDATIONS

### General

The existing Highway 69, which extends through the area of investigation, is comprised of an embankment, about 3.5 m high, with single lanes running in both the north and south directions.

It is proposed to use the existing highway embankment as the northbound lanes and to construct a new parallel embankment, up to about 4 m high, to the west, which will be used for the new southbound lanes.

### Design

Based on the information, which was obtained from the report by Dominion Soil Investigation Inc., it appears that the peat and weak, cohesive soils extend to depths of up to 5.6 m and the groundwater table is at or near the existing ground surface.

In order to construct the embankment to the proposed height, it will be necessary to excavate all of the peat and the underlying clay down to a depth of about 4 m or to an elevation of about 188.5 m. The backslope of the excavation, which will be required to remove the subsoils, may be constructed as steeply as is practicable.

To maintain adequate stability, the slopes of the embankment should be constructed to no steeper than 1.5H:1V, if rockfill is used.

### Construction Considerations

#### Excavations

It is expected that excavations within the peat and soft clay to depths of up to 4 m, below the groundwater table will be temporarily stable at slopes of 2H:1V. Subexcavation and backfilling should be carried out concurrently and under water, if necessary.

#### Raising the Grade

Rockfill or other fills, placed below the groundwater table, may be end-dumped. However, once the material is 0.3 m above the groundwater table, placement and compaction of the fill materials should be carried out according to OPSS standards and MTO practice.

### Embankment Stability

The proposed embankment would be 4 m high. Based on the slope stability analyses, no deep seated stability problem is anticipated if the soft material is removed as recommended.

### Settlement

A settlement of less than 15cm is expected to occur, due to the silty clay which will remain beneath the embankment, in some areas. It is expected that most of this will take place, during the construction period.

However, to ensure that adequate compaction of the submerged fills takes place, it is recommended that the embankment be overbuilt by about 0.5 m. The overbuilt portion should be compacted to MTO standards and then it may be immediately excavated to grade.

### MISCELLANEOUS

This report was written by Mr. J. Blair, Project Foundation Engineer, reviewed by Ken Ahmad, Foundation Engineer and approved by D.H. Dundas, P. Eng., Chief Foundation Engineer (Acting).



A handwritten signature in black ink, appearing to read "K.S.Q. Ahmad".

K.S.Q. Ahmad, P.Eng.  
Project Foundation Engineer



A handwritten signature in black ink, appearing to read "D. Dundas".

D.H. Dundas, P.Eng.  
Chief Foundation Engineer (Acting)

## **APPENDIX**

# LOG OF BOREHOLE 18+020 C/L SBL 1

ENCL. No.: 5

REF. No.: 91-9-5	DRILLING DATA
CLIENT: Ministry of Transportation	
PROJECT NAME: Twinning of HWY 69, W.P. 215-89-00	Method: Hand drill
LOCATION: District #11, Huntsville, Ontario	Diameter:
DATUM:	Date: November 28th, 1991

LABORATORY DATA						SAMPLES			SYMBOL	MATERIAL DESCRIPTION	ELEV. m	DEPTH m	WATER DATA	REMARKS
PL	NMC	LL	WT	UNIT	UNDR	STRNG	No.	TYPE						
%	%	%	KN/m <sup>3</sup>	Field	Lab.									
				Vane	Comp									
				kPa	kPa									
GROUND SURFACE m														
							1	SS	<1	TOPSOIL 1.2m				W.L. @ 0.1 m above ground surface
							2	SS	<1			1		
16	27	23					3	SS	2	grey, v. soft to soft SILTY CLAY with silt seams		2		
							4	SS	<1			3		
							5	SS	10/0cm	END OF BOREHOLE (possible bedrock)		4		SS:5 split spoon bouncing @ 4.1 m depth

Vertical Scale: 1:50



**DOMINION SOIL INVESTIGATION INC.**

Checked: R.M.

SHEET 1 OF 1 HOLE No. 1

## 2

ENCL. No.: 6

REF. No.:	91-9-5	DRILLING DATA
CLIENT:	Ministry of Transportation	
PROJECT NAME:	Twinning of HWY 69, W.P. 215-89-00	Method: Hand drill
LOCATION	District #11, Huntsville, Ontario	Diameter:
DATUM:		Date: February 26th, 1992

LABORATORY DATA							SAMPLES			MATERIAL DESCRIPTION		ELEV.	DEPTH	WATER	REMARKS	
PL %	NMC %	LL %	WT g/m <sup>3</sup>	UNIT	UNDR STRNG	Field Lab.	No.	TYPE	Value	SYMBOL	m	m	LOGIC			
				Vane	Compr	kPa kPa				<b>GROUND SURFACE m</b>						
							1	SS	1		black MUCK					
							2	SS	<1							
							3	SS	<1							
											grey, v.soft to soft <b>SILTY CLAY</b> wet					
								</								

Vertical Scale: 1:50

§

**DOMINION SOIL INVESTIGATION INC.**

Checked: **K.M.**

SHEET 1 OF 1 HOLE No. 2

ENCL. No.: 7

REF. NO.:	91-9-5	DRILLING DATA
CLIENT:	Ministry of Transportation	
PROJECT NAME:	Twinning of HWY 69, W.P. 215-89-00	Method: Hand drill
LOCATION	District #11, Huntsville, Ontario	Diameter:
DATUM:		Date: February 26th, 1992



[illegible]



# LOG OF BOREHOLE 18+120 C/L SBL 4

ENCL. No.: 8

REF. NO.: 91-9-5	DRILLING DATA
CLIENT: Ministry of Transportation	
PROJECT NAME: Twinning of HWY 69, W.P. 215-89-00	Method: Hand drill
LOCATION: District #11, Huntsville, Ontario	Diameter:
DATUM:	Date: November 28th, 1991

LABORATORY DATA						SAMPLES			SYMBOL	MATERIAL DESCRIPTION	ELEV. m	DEPTH m	WATER DATA	REMARKS
PL %	NMC %	LL %	UNIT WT kN/m3	Field Vane kPa	STRNG Lab. Compr kPa	No.	TYPE	N- Value						
GROUND SURFACE										m				
28						1	SS	<1		TOPSOIL 0.7m				W.L. @ 0.2 m above ground surface
						2	SS	4			grey, firm SILTY CLAY wet		1	
						3	SS	5						
							4	SS	10/0cm		END OF BOREHOLE (possible bedrock)		2	

Vertical Scale: 1:50



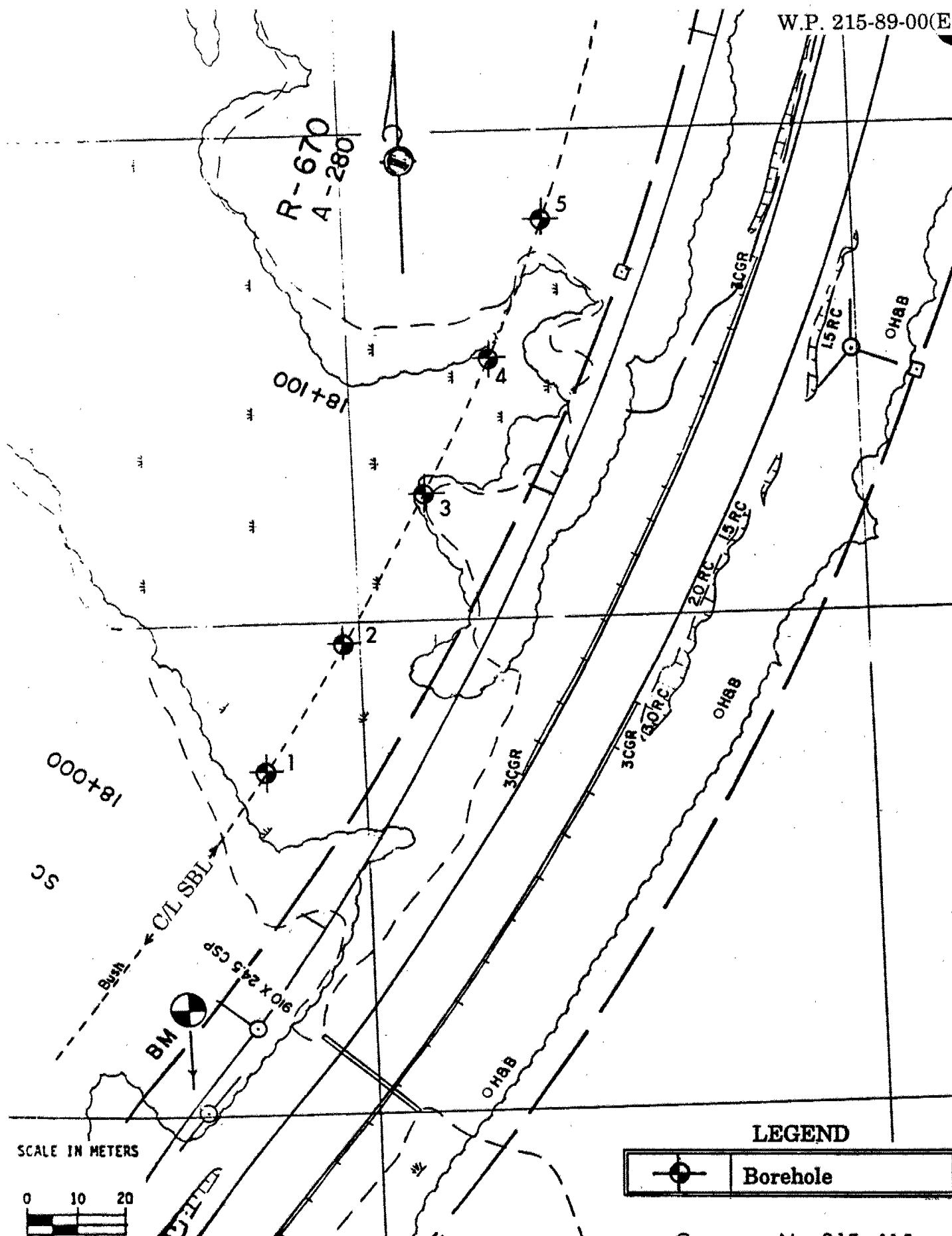
**DOMINION SOIL INVESTIGATION INC.**

Checked: R.M.

SHEET 1 OF 1 HOLE No. 4

ENCL. No.: 9

SHEET 1 OF 1 HOLE No. 5



# LEGEND



Geocres No 31E-115

Drawing No: 2158900(E)-A

## EXPLANATION OF TERMS USED IN REPORT

**N VALUE:** THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m 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 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS  $\bar{N}$ .

**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/0.3m)	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

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

### STRESS AND STRAIN

$u_w$	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
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /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
$S_f$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m <sup>3</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						

GEOCRES No. 31E-115DIST. 11 REGION W.P. No. 217-89-00(D)CONT. No. W. O. No. STR. SITE No. HWY. No. 69 NewLOCATION Em bankment for Swamp  
Crossings Near Markwa Rd #5No. of PAGES - (site 8)

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

File

# MEMORANDUM

(416)235-3731

19940201

To: Mr. J.I. McDougall, P.Eng.  
Head, Geotechnical Section  
Northern Region

Att: Mr. Ron Purdy

From: Foundation Design Section  
Room 315, Central Building  
Downsview, Ontario

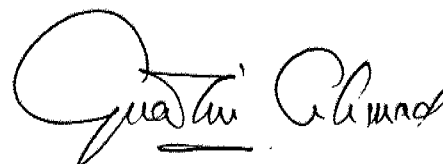
Re: A Proposed Embankment  
Along Muskoka Road #32  
Location 8, Station 9+720 to 9+820  
W.P. 215-89-00 (F) Geocres ~~31B-346~~ 31E-115  
Northern Region 217-89-00(D)  
District 11, Huntsville

The attached report provides recommendations for a proposed embankment between Stations 9+720 and 9+820 for the proposed relocation of Muskoka Road #32.

We believe that this report will be adequate for your purposes. However, should you have any questions regarding it, please do not hesitate to contact this office.

## Distribution

J. McDougall (2)  
P. Furst  
R. Mantha  
S. Wilson (2)  
M. Holowka  
G.E. Greene  
E.A. Joseph  
G. Todd (Cover Only)  
F. Bacchus (Cover Only)  
File ✓



K.S.Q. Ahmad, P. Eng.  
Foundation Engineer

For

D.H. Dundas, P. Eng.  
Chief Foundation Engineer (Acting)

FEB 07 1994

# FOUNDATION INVESTIGATION REPORT

For

A Proposed Embankment

Muskoka Road #32

Location 8 - Station 9+720 to 9+820

W.P. 215-89-00 (F), Geocres 31D-346

Northern Region

District 11, Huntsville

## INTRODUCTION

At the request of the Northern Region, Geotechnical Section, a foundation investigation was carried out for a proposed embankment, to be located at the above-captioned site. This report summarizes the factual information obtained from this investigation.

## SITE DESCRIPTION AND GEOLOGY

The site is located between Stations 9+720 and 9+820, along the proposed relocation of Muskoka Road #32, and is approximately 150 m to the northwest of the intersection of Muskoka Road #38 and the existing Highway 69 embankment, within Lot 22, Concession VII, Gibson Township, District of Muskoka.

The area of investigation consists of fairly large swampy area, approximately 150 m wide, with occasional tree stumps, sticking up through it. The swamp was covered with ice, about 180 mm thick, at the time of the investigation.

Several rivers, in this area, indicate that drainage is generally towards the west; ie. Georgian Bay.

## PROCEDURES

The fieldwork, was carried out, during the period between March 31 and April 4, 1992, and consisted of 3 sampled boreholes (Boreholes 8-2, 8-3 and 8-4), which were advanced to depths of 2.9 to 6.1 m, by casing and wash boring, using tripod-mounted equipment.

Soil samples were obtained using a 50mm Split Spoon Sampler driven into the soil in accordance with the specification of the Standard Penetration Test (ASTM 1586). Field Vane tests were carried out wherever soft to firm cohesive soils were encountered in the boreholes.

Dynamic cone penetration tests were also carried out adjacent to each of the boreholes and at two other locations (Cone Tests 8-1 and 8-5).

The boreholes were staked out in the field by the Northern Region Surveys and Plans Office. Small changes in the locations and elevations of the boreholes, were determined by our field representatives.

The soil samples, which were obtained in the field, were examined in the laboratory by visual and tactile methods. Moisture content, Unit Weight and Atterberg Limits were carried out on selected soil samples.

### SUBSURFACE CONDITIONS

Beneath ice and ponded water, up to 0.6 m deep, the subsurface conditions, at the boreholes, generally consisted of a thin (1.0 to 1.1 m) layer of peat and organic-stained soils which was underlain by an extensive deposit of silty clay reaching depths of up to 5.2 m.

A relatively thin layer (0.9 m) of very dense, fine sand was encountered, immediately above the bedrock surface, in one of the boreholes. The probable bedrock surface was encountered, at depths of 0.9 to 5.2 m, in the boreholes and dynamic cone penetration tests.

Details of the subsurface information, obtained from this investigation, are included on the borehole logs, attached. Brief descriptions of the individual strata and the groundwater conditions, encountered in the boreholes, are given below.

#### Peat/Organic-Stained Silty Clay

Beneath ice and water, up to 0.6 m deep, all boreholes contacted a layer up to 1.1 m thick, of dark brown to black fibrous peat and organic-stained silty clay (ie. containing root fibres and occasional pockets of peat).

#### Silty Clay

At depths of 1.4 to 1.5 m (or elevations of 195.0 to 195.5 m) and beneath the organic soils described above, all of the boreholes contacted a light brown to brownish grey, silty clay, up to 3.8 m thick.



Atterberg limits tests, which were carried out on several samples of soil obtained from this deposit, gave measured liquid limits and plasticity indices ranging from 40 to 50 (average of 46) and 22 to 32 (average of 27) percent, respectively. These results, which are shown on Figure 1, indicate soils which can be generally classified as silty clay.

Moisture contents, which were measured in several samples obtained from this deposit, ranged from 27 to 64 (average of 43) percent.

'N'-values, which ranged from 3 to 21 blows/0.3 and field vane tests gave measured shear strengths ranging from 30 to 88 kPa, indicate soils of generally soft (but occasionally firm to stiff) consistency.

#### Fine to Medium Sand

A relatively thin (0.9 m thick) layer of fine sand was encountered in Borehole 8-2, at a depth of 5.2 m (or an elevation of 191.3 m), above the probable bedrock surface.

A moisture content of 13 percent was measured in a sample, obtained from this cohesionless deposit.

An 'N'-value of 63 blows/0.20 m, measured during Standard Penetration Testing, indicates that this soil is very dense.

#### Bedrock

Probable bedrock was encountered in the boreholes and dynamic cone penetration tests, at depths of 0.9 to 6.1 m, or elevations of 190.4 to 196.8 m. Outcrops adjacent to the swamp indicate that the bedrock is likely to be comprised of a hard, granitic gneiss.

#### Groundwater Conditions

Ice and ponded water at the site, indicates that, at the time of the investigation, the groundwater table was at an elevation of about 196.5 to 196.9 m which formed a 400mm to 600mm deep pond.

## DISCUSSIONS AND RECOMMENDATIONS

### General

Presently, Muskoka Road #32 consists of a narrow, paved and unpaved road which runs to the west from Highway 69 through a series of rock cuts. The existing pavement appears to be in poor condition.

It is proposed to relocate a portion of Muskoka Road #32 (ie. between Stations 9+130 to 10+000), so that it will eventually intersect Highway 69 about 650 m further south or directly adjacent to Muskoka Road #38.

This investigation is concerned with the portion of Muskoka Road #32, between Stations 9+720 and 9+820, where the relocated road must pass over a fairly extensive pond. In this area, it is proposed to construct a two-lane embankment, up to 2.8 m high.

### Design

Based on the information, which we have obtained from this investigation, it appears that, beneath the ponded water, soft peat, compact sand and/or soft to stiff, silty clay underlies the area of investigation, to depths of up to 6.1 m.

In order to construct the embankment to the proposed height, it will be necessary to excavate out the peat as well as the upper organic-stained portion of the silty clay down to an elevation of about 195.0 m (about 1m below ground surface).

To maintain adequate stability, the slopes of the new rockfill embankment may be constructed as steeply as 1.25H:1V.

### Construction Considerations

#### Excavations

Excavations, to depths of up to 1 m, below the bottom of the pond, will be temporarily stable at slopes of 2H:1V. Subexcavation and backfilling should be carried out concurrently and under water, if necessary.

### Raising the Grade

Rockfill or other fills placed below the groundwater table, may be end-dumped. However, once the material is 0.3 m above the groundwater table, placement and compaction of the fill should be carried out according to OPSS standards and MTO practice.

### Settlement

A settlement of less than 10cm is expected to occur, due to the silty clay which will remain beneath the embankment, in some areas. It is expected that most of the settlement will take place, during the construction period.

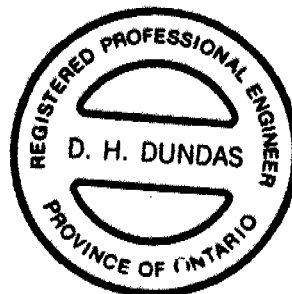
### MISCELLANEOUS

The field investigation was supervised by J. Blair and Dan Rothwell, using equipment owned and operated by Atcost Soil Drilling Inc.

This report was written by J. Blair, Project Foundation Engineer, reviewed by K. Ahmad, Foundation Engineer and approved by D.H. Dundas, P. Eng., Chief Foundation Engineer (Acting).

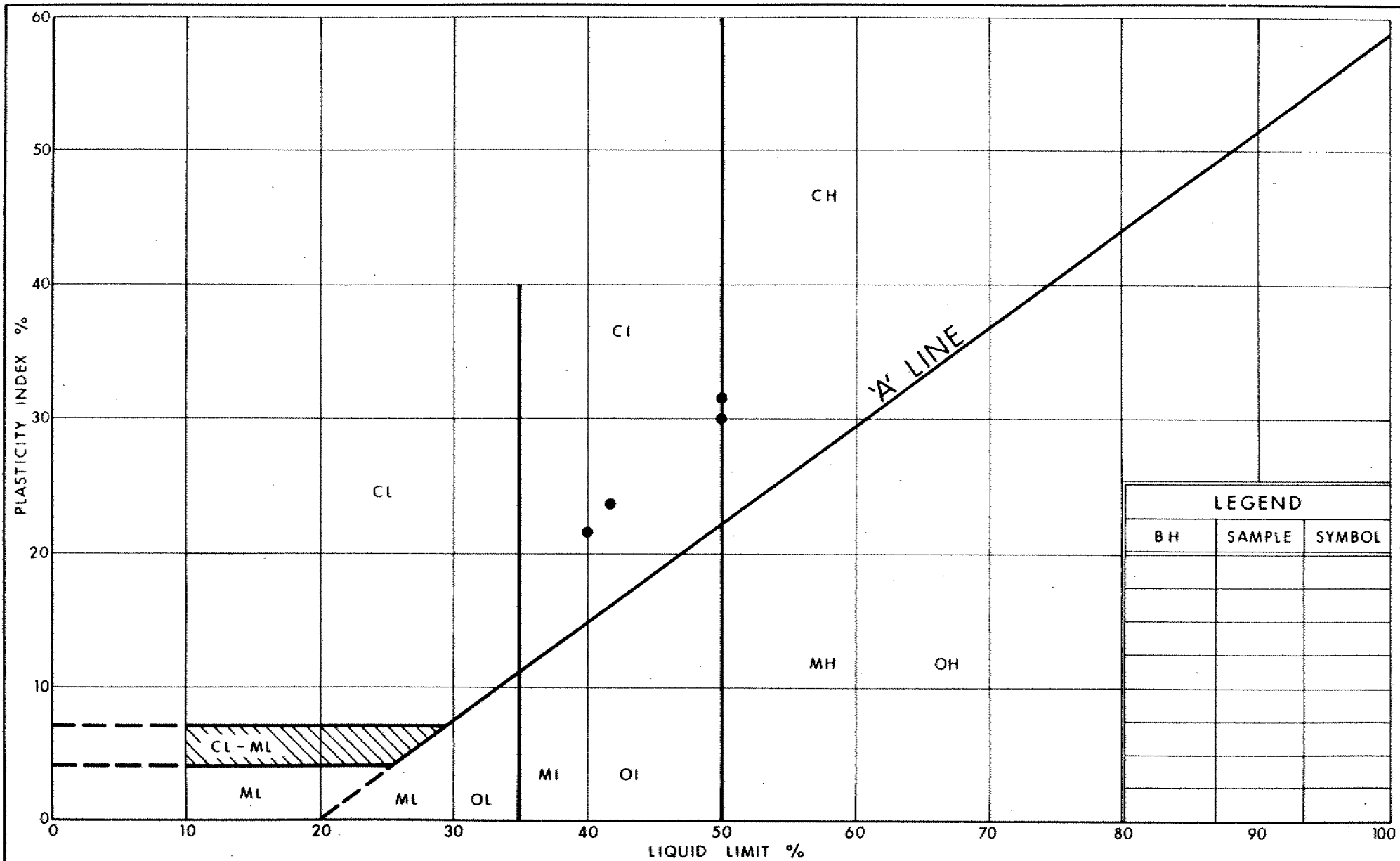


K.S.Q. Ahmad, P.Eng.  
Foundation Engineer



D.H. Dundas, P.Eng.  
Chief Foundation Engineer (Acting)

## **APPENDIX**



Ministry of  
Transportation

## PLASTICITY CHART SILTY CLAY

FIG No 1

W P 215-89-00 (F)

# RECORD OF BOREHOLE No 8-1

1 OF 1

METRIC

W.P. 215-89-00(F) LOCATION Sta. 9+715; Off. 1.0 m Lt C/L ORIGINATED BY DR  
DIST 11 HWY 69 BOREHOLE TYPE Cone Test COMPILED BY JB  
DATUM Geodetic DATE March 31, 1992 CHECKED BY DD

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL * LAB VANE 20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	WATER CONTENT (%) 7 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE								
197.1	Ground Surface											
0.0	Probable Topsoil											
	Possible Sandy Silt to Silty Sand											
	Probable Silty Clay											
194.1												
3.0	End of Cone Test Refusal - Probable Bedrock											

# RECORD OF BOREHOLE No 8-2

1 OF 1

METRIC

W.P. 215-89-00(F) LOCATION Sta. 9+740; Off. 1.0 m Rt C/L ORIGINATED BY DR  
DIST 11 HWY 69 BOREHOLE TYPE Wash Boring / Cone Test COMPILED BY JB  
DATUM Geodetic DATE March 31, 1992 CHECKED BY DD

SOIL PROFILE		SAMPLES			GROUND WATER # CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT 7 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	W <sub>p</sub> W W <sub>L</sub>	20 40 60			
196.5	Ice **												
0.0	180 mm Ice, 230 mm Water Pockets of Peat, Occasional Rod Fibres  Silty Clay  Light Brown to Brownish Grey  Firm to Stiff		1	SS	10								
			2	SS	8								
			3	SS	3								
			4	TW	PH								
191.3			5	SS	3								
5.2 190.4	Fine Sand, Some Silt Grey, Very Dense		6	SS	63	/23cm							
6.1	End of Borehole Refusal - Probable Bedrock  * W.L. upon completion of Sampling  ** Surveyed elevations are with respect to top of ice												

# RECORD OF BOREHOLE No 8-3

1 OF 1

METRIC

W.P. 215-89-00(F) LOCATION Sta. 9+770; Off. 1.0 m Lt C/L ORIGINATED BY DR  
DIST 11 HWY 69 BOREHOLE TYPE Wash Boring / Cone Test COMPILED BY JB  
DATUM Geodetic DATE April 4, 1992 CHECKED BY DD

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL * LAB VANE					
196.8	Ice **												
0.0	180 mm Ice, 230 mm Water												
	Peat Pockets of Peat, Occ. Rootlets		1	SS	14								
	Silty Clay Brownish Grey		2	SS	15								
194.0	Stiff to Very Stiff		3	SS	21								
2.9	End of Borehole Refusal - Probable Bedrock * W.L. upon completion of Sampling ** Surveyed elevations are with respect to top of ice												



# RECORD OF BOREHOLE No 8-4

1 OF 1

METRIC

W.P. 215-89-00(F) LOCATION Sta. 9+800; Off. 1.0 m Rt C/L ORIGINATED BY DR  
DIST 11 HWY 69 BOREHOLE TYPE Wash Boring / Cone Test COMPILED BY JB  
DATUM Geodetic DATE April 02, 1992 CHECKED BY DD

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ KN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)		
196.8	Ice **												
0.0	180 mm Ice; 430 mm Water Peat Layers, Traces of Root Fibres		1	SS	8								
	Silty Clay Brown to Brownish Grey Firm to Stiff		2	SS	11								
193.1			3	SS	19								
3.7	End of Borehole Refusal - Probable Bedrock												
	• W.L. upon completion of Sampling												
	** Surveyed elevations are with respect to top of ice												

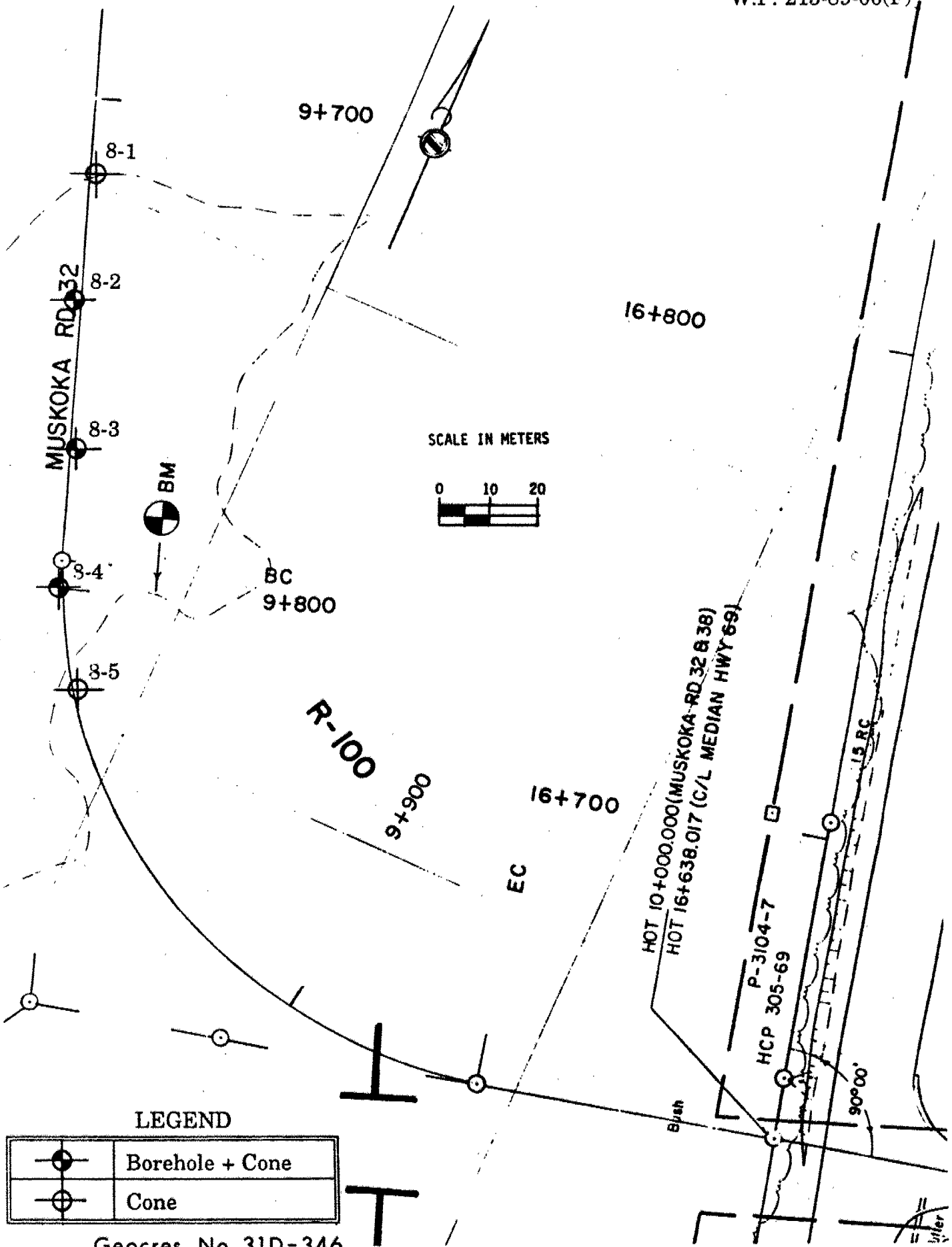
# RECORD OF BOREHOLE No 8-5

1 OF 1

METRIC

W.P. 215-89-00(F) LOCATION Sta. 9+820; Off. 1.0 m Lt C/L ORIGINATED BY DR  
DIST 11 HWY 69 BOREHOLE TYPE Cone Test COMPILED BY JB  
DATUM Geodetic DATE April 02, 1992 CHECKED BY DD

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									
197.7								20	40	60	80	100					
0.0	Probable Peat							○ UNCONFINED									
196.8	Probable Silty Clay						197	○ UNCONFINED									
								• QUICK TRIAXIAL									
								20	40	60	80	100					



LEGEND

	Borehole + Cone
	Cone

Geocres No 31D-346

Drawing No: 2158900(F)-A

## EXPLANATION OF TERMS USED IN REPORT

**N VALUE:** THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63 kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS  $\bar{N}$ .

**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/0.3m)	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

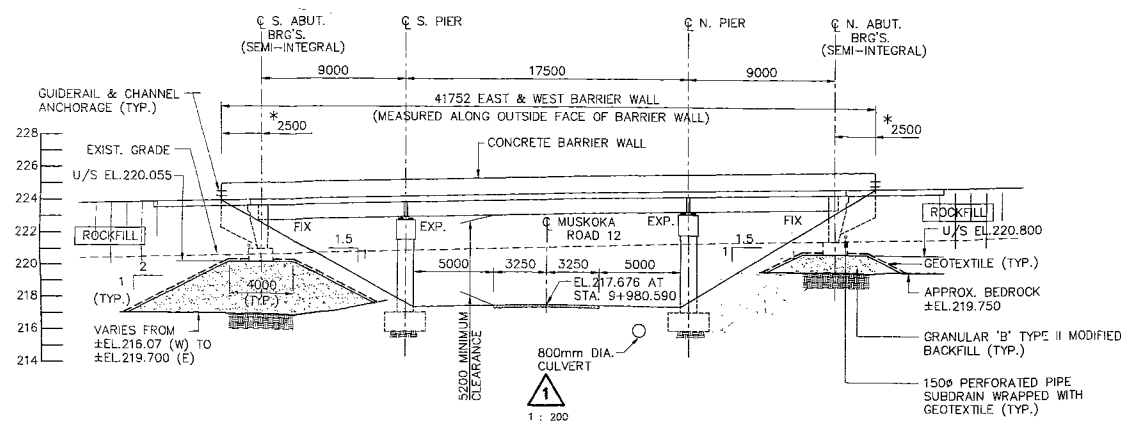
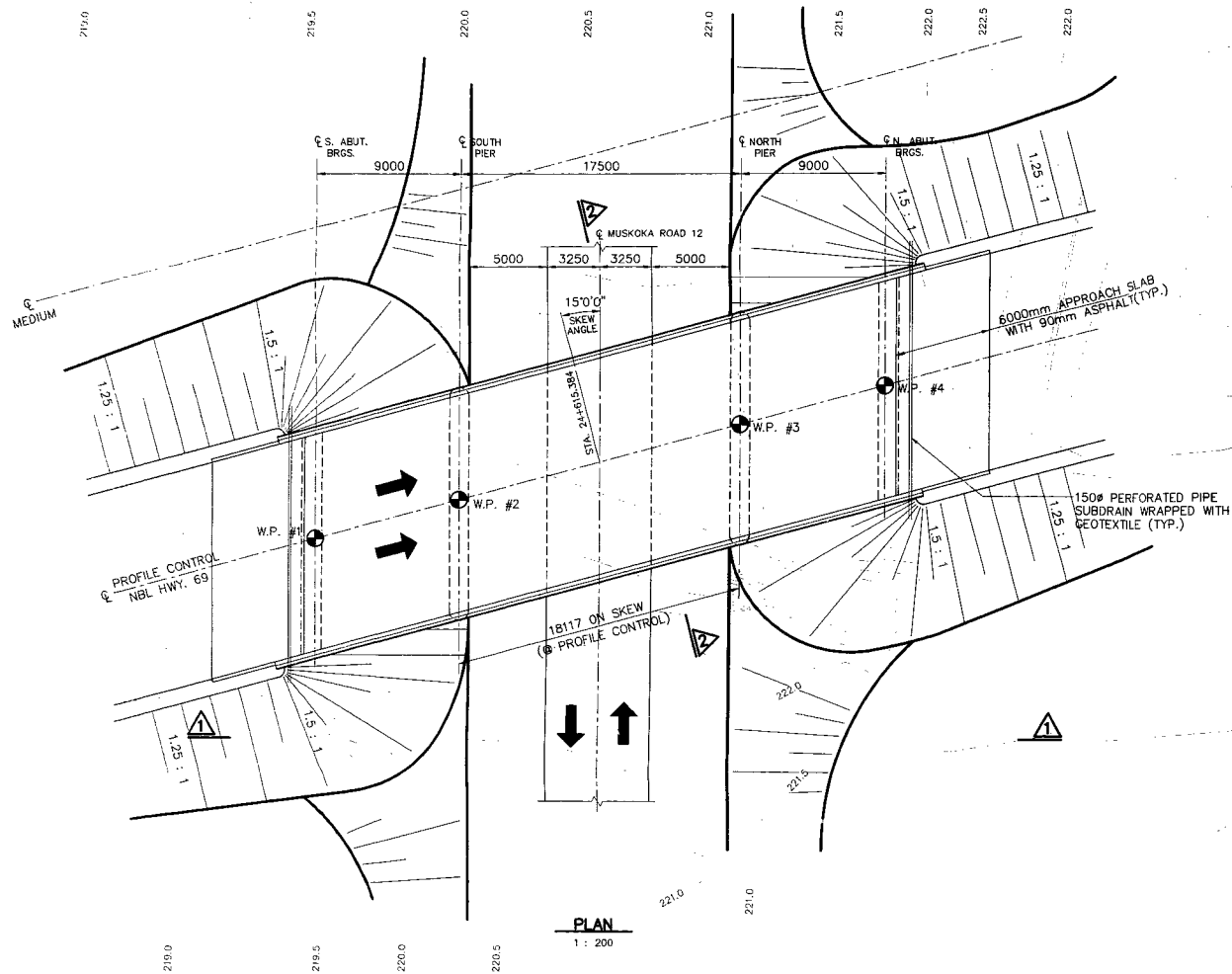
$u_w$	kPa	PORE WATER PRESSURE
$r_u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS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

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /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
$S_t$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m <sup>3</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						

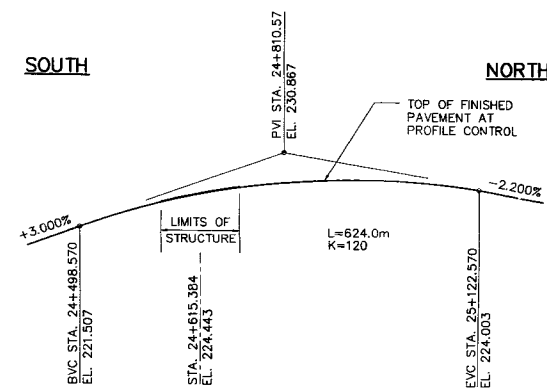


NOTE:  
ROCK ELEVATIONS ARE APPROXIMATE WITH REFERENCE TO  
SOIL INVESTIGATION REPORT S07657G/C DATED OCTOBER 5, 1998.

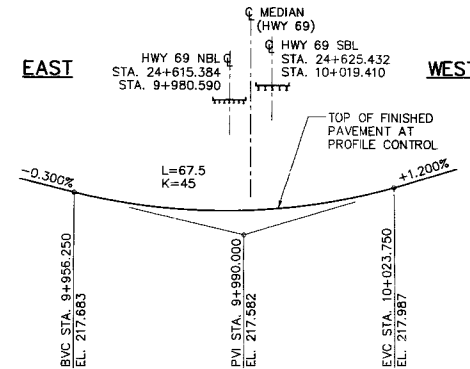
**PRELIMINARY**

OCT 14 1998

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

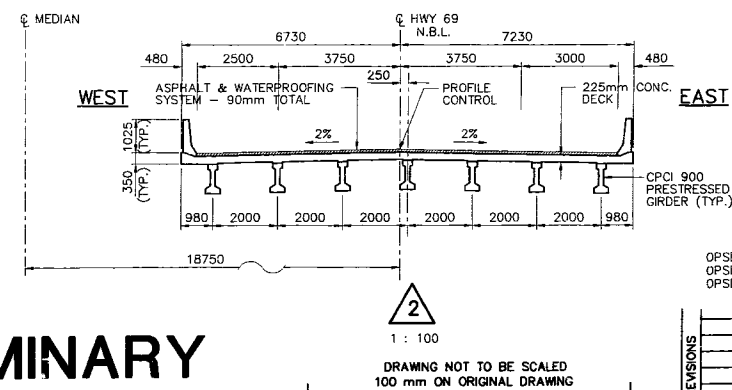


**PROFILE CONTROL - NORTHBOUND LANES**  
N.T.S.



**PROFILE CONTROL - MUSKOKA ROAD 12**  
N.T.S.

WORKING POINTS			
STATION AT Q ROAD	T/A ELEVATIONS	COORDINATES	
W.P. #1	24+597.007	224.056	N 4990508.260 E 282369.832
W.P. #2	24+606.325	224.256	N 4990516.813 E 282366.135
W.P. #3	24+624.442	224.623	N 4990533.443 E 282358.947
W.P. #4	24+633.760	224.801	N 4990541.996 E 282355.251



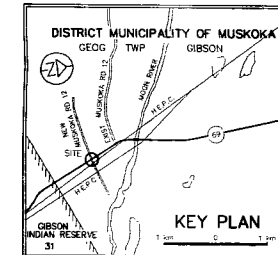
DIST HWY 69  
CONT No  
WP No 198-98-01



MUSKOKA ROAD 12 CROSSING  
HWY 69 NORTHBOUND STRUCTURE  
GENERAL ARRANGEMENT

SHEET

**R.V. Anderson Associates Limited**  
consulting engineers, architects, technology managers 4971



#### GENERAL NOTES

- CLASS OF CONCRETE:  
-PRECAST GIRDERS..... 40 MPa  
-REMAINDER..... 30 MPa
- CLEAR COVER TO REINFORCING STEEL:  
-FOOTINGS..... 100±25  
-DECK: TOP..... 70±20  
BOTTOM..... 40±10  
-REMAINDER..... 70±20  
UNLESS OTHERWISE NOTED.
- REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED.  
OTHERWISE SPECIFIED, TENSION LAP LENGTHS NOT INDICATED ON THE CONTRACT DRAWINGS SHALL BE CLASS B.  
HOOKS AND BENDS FOR REINFORCING STEEL SHALL BE DETAIL ACCORDING TO OHBDC-91, UNLESS SHOWN OTHERWISE. THE FOLLOWING SHALL APPLY:  
a) STANDARD HOOKS WITH MINIMUM BEND DIAMETERS SHALL BE USED FOR STIRRUPS AND TIES, ACCORDING TO CLAUSE B-14.1.  
b) OTHER BARS SHALL HAVE STANDARD HOOKS WITH BEND DIAMETERS ACCORDING TO CLAUSE C-8-14.1.

#### CONSTRUCTION NOTES

- THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.
- NO BACKFILL TO BE PLACED BEHIND ABUTMENTS UNTIL CONCRETE IN DECK HAS REACHED 75% OF ITS SPECIFIED STRENGTH.
- BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND ABUTMENTS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN THE BACKFILL HEIGHTS BE GREATER THAN 500mm.

#### LIST OF DRAWINGS

- GENERAL ARRANGEMENT
- FOUNDATION LAYOUT AND FOOTING REINFORCEMENT
- NORTH ABUTMENT
- NORTH WINGWALLS
- SOUTH ABUTMENT
- SOUTH WINGWALLS
- PRESTRESSED GIRDERS AND BEARINGS
- DECK DETAILS
- BARRIER WALL
- 6000mm APPROACH SLAB
- STANDARD DETAILS

#### APPLICABLE STANDARD DRAWINGS

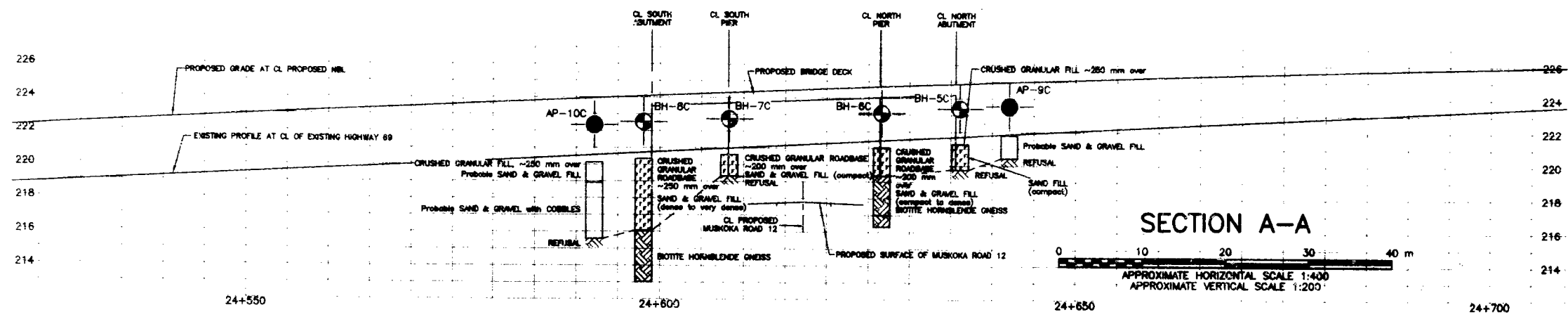
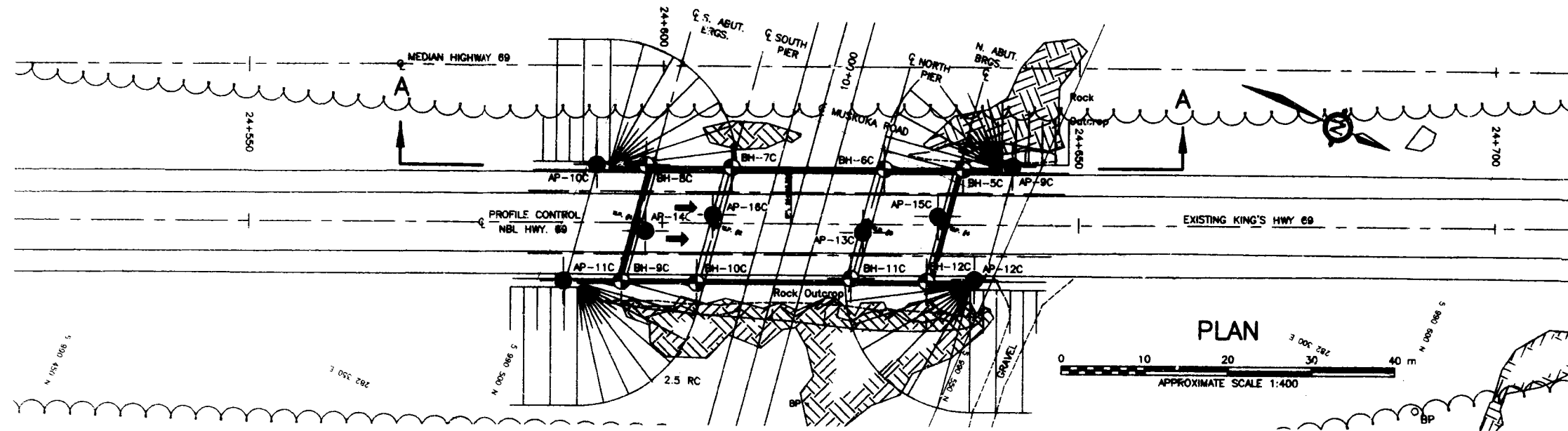
OPSD 3501.00	GRANULAR BACKFILL REQUIREMENTS - ABUTMENTS
OPSD 3505.00	ROCK BACKFILL REQUIREMENTS - ABUTMENTS
OPSD 4010.00	GUIDE RAIL AND CHANNEL ANCHORAGE

REVISIONS		DESCRIPTION	
DESIGN	KN	CHK	CODE OHBDC-91
DRAWN	HC	CHK	SITE 42-319N
			STRUCT 1
			SCHEME
			DATE NOV 1998
			IDWG 1

PLATE No  
 DRAWING No  
 CONT No  
 WP No

217-89-00

SHEET



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

TROW CONSULTING ENGINEERS LTD.  
 SUDBURY, ONTARIO  
 PROJ. No S076570C DWG. No. 1A

MINISTRY OF TRANSPORTATION  
 ENGINEERING OFFICE  
 SURVEYS AND PLANS SECTION

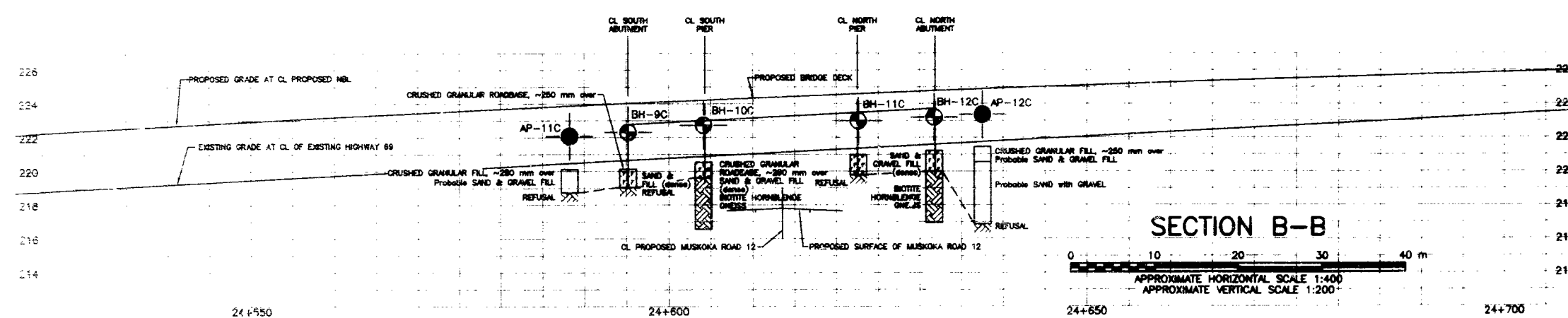
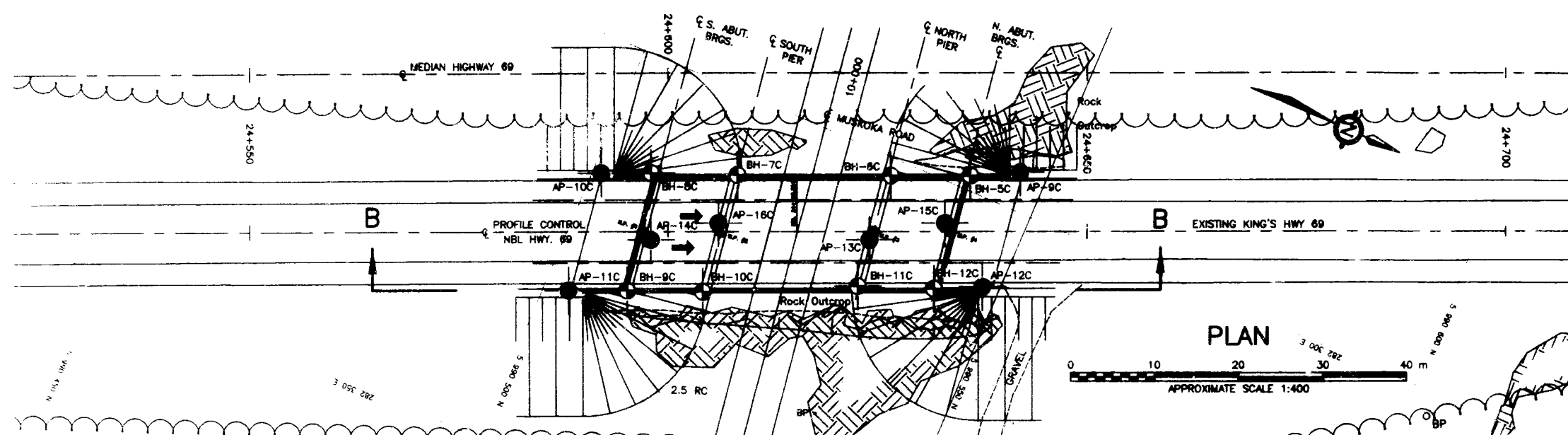
SITE PLAN & CROSS SECTION A-A  
 PROPOSED CROSSING  
 AT  
 MUSKOKA ROAD 12  
 AND  
 NBL OF HIGHWAY 69  
 OGDG TWP GIBSON DIST OF MUSKOKA  
 LOT 13 CSD 13

SCALE AS SHOWN	DISTRICT S2, HUNTSVILLE	REGION NORTHERN
ETB		
SURVEY DATE	PLAN DATE	98/10
SITE	PLAN	

PLAN No.  
 SHEET No.  
 CONT No  
 WP No

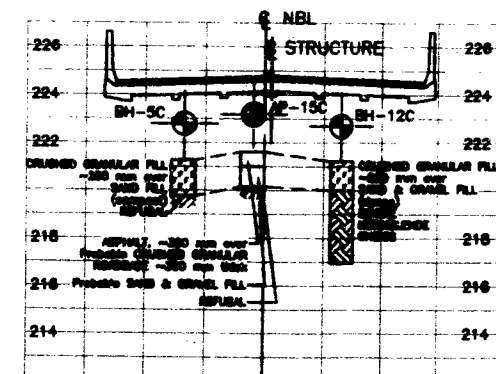
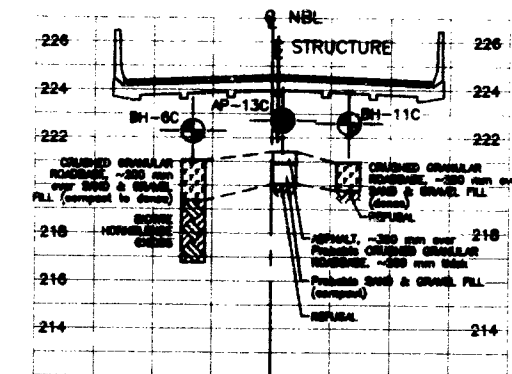
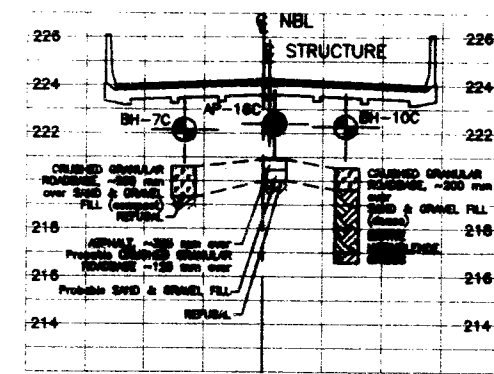
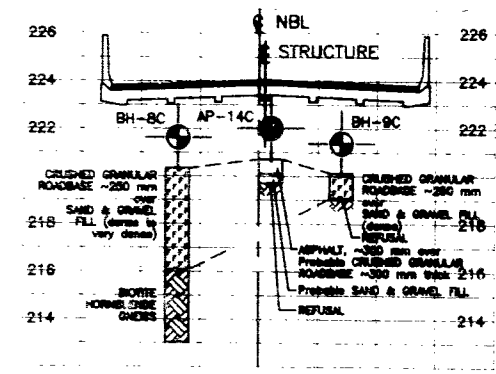
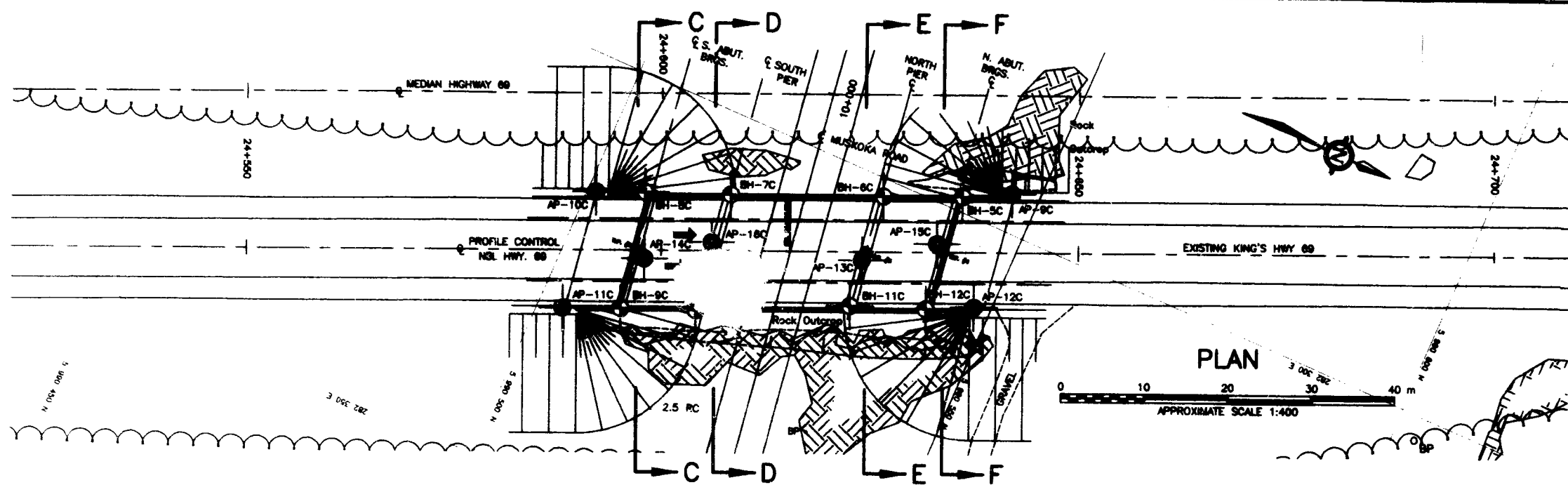
217-89-00

SHEET



METRIC  
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 AND/OR MILLIMETRES  
 UNLESS OTHERWISE SHOWN

TROW CONSULTING ENGINEERS LTD. SUDBURY, ONTARIO PROJ. No. S07657GC DWG. No. 1B		
MINISTRY OF TRANSPORTATION ENGINEERING OFFICE SURVEYS AND PLANS SECTION		
<b>SITE PLAN &amp; CROSS SECTION B-B</b> PROPOSED CROSSING AT MUSKOKA ROAD 12 AND NBL OF HIGHWAY 69		
GSDC TWP. GIBSON LOT 13	DIST. OF MUSKOKA CO. 13	
SCALE AS SHOWN	DISTRICT S2, HURONVILLE	REGION NORTHWEST
SURVEY LINE		PLAN DATE 08/10
SITE		PLAN



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

**TROW CONSULTING ENGINEERS LTD.**  
SUBSIDIARY, ONTARIO  
PROJ. No. S078570C Dwg. No. 1C

**SECTIONS C-C, D-D, E-E & F-F**

**PROPOSED CROSSING**  
**AT**  
**MUSKOKA ROAD 12**  
**AND**  
**NBL OF HIGHWAY 69**

DATE OF DESIGN: 12/12/88 DATE OF REVISION: 01/12/89

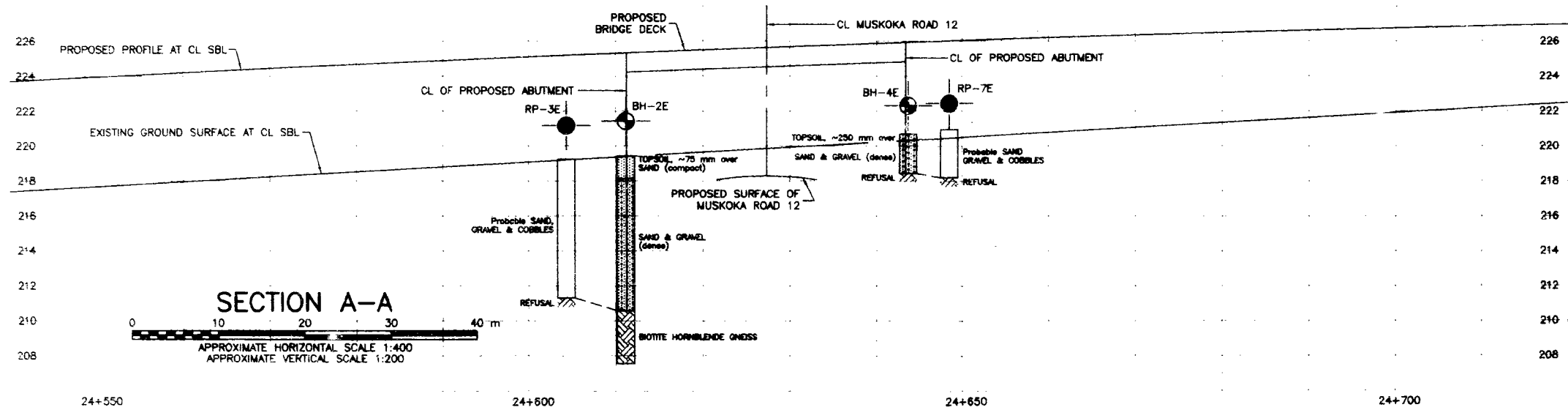
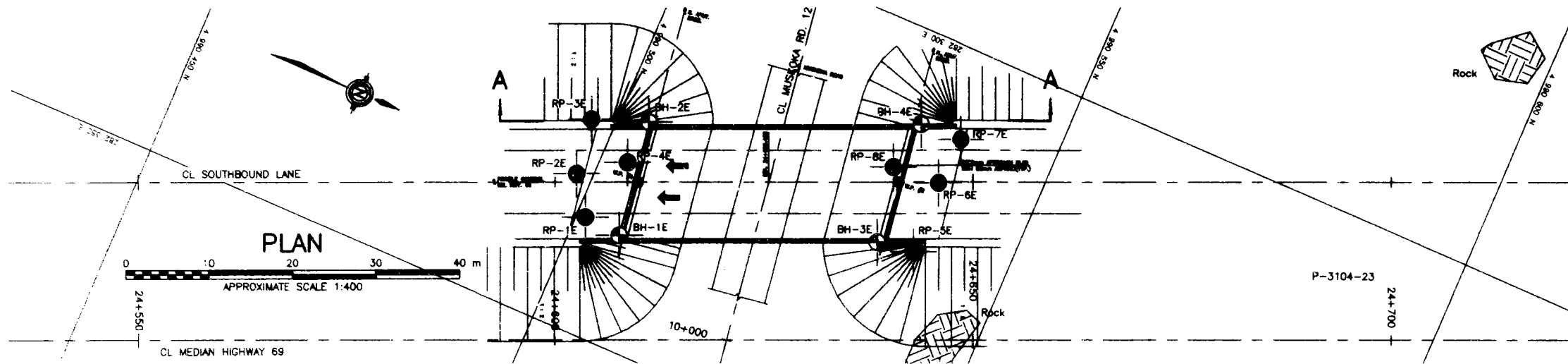
SCALE AS SHOWN	DISTRICT AL. HARRISVILLE	REGION HARRISVILLE
SURVEY DATE: PLAN DATE: 02/79		
SHEET: PLAN		



PLATE No  
 DRAWING No  
 CONT No  
 WP No

217-89-00

SHEET

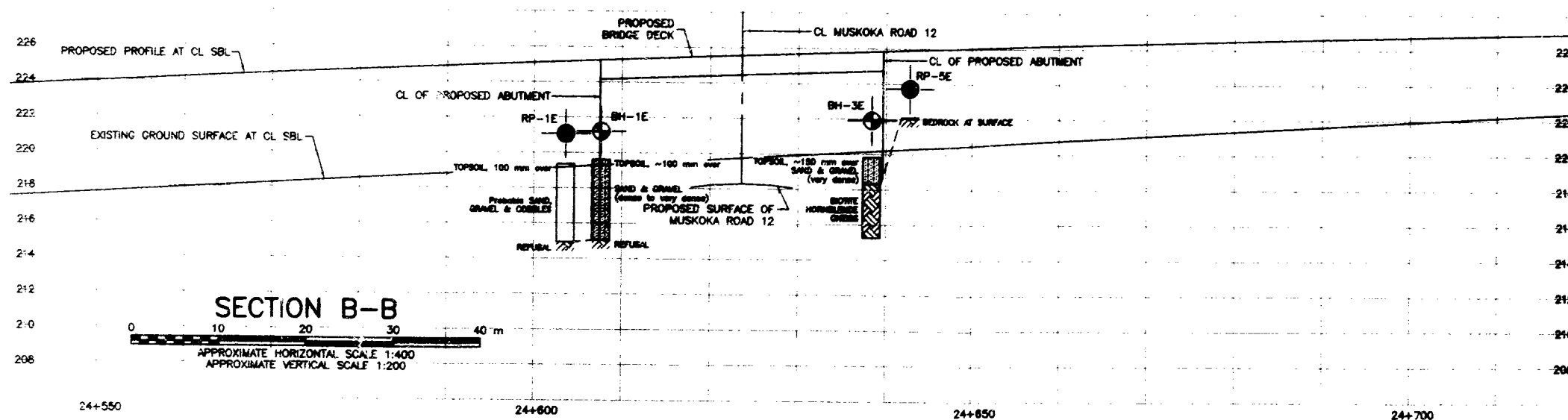
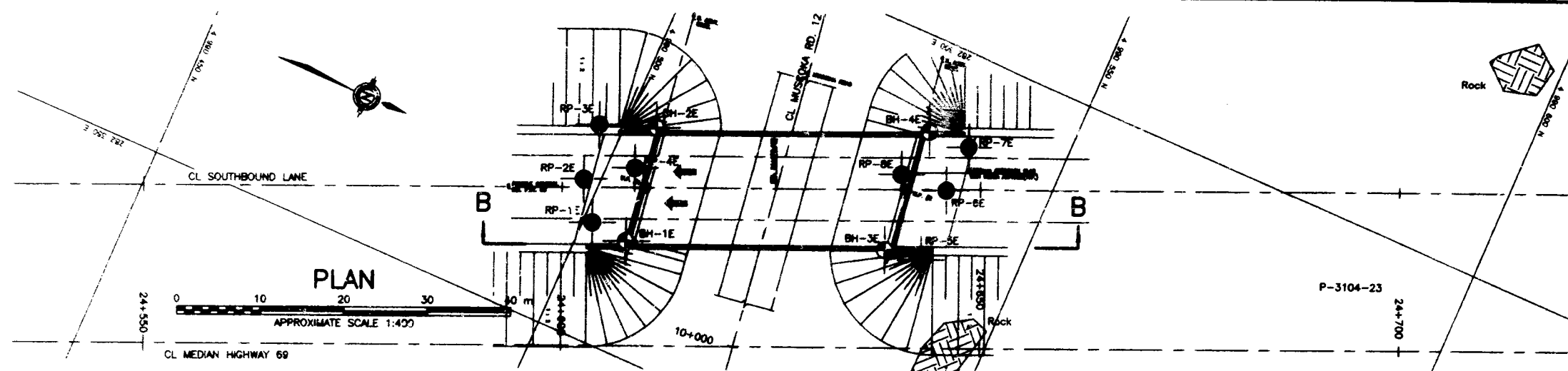


METRIC  
 DIMENSIONS ARE IN METRES  
 AND/OR MILLIMETRES  
 UNLESS OTHERWISE SHOWN

TROW CONSULTING ENGINEERS LTD. SUDBURY, ONTARIO PROJ No S07657GE DWG No. 1A		
MINISTRY OF TRANSPORTATION ENGINEERING OFFICE SURVEYS AND PLANS SECTION		
SITE PLAN & SECTION A-A		
PROPOSED CROSSING AT MUSKOKA ROAD 12 AND SBL OF HIGHWAY 69		
GEOG. TYP. OF GIBSON LOT 14		DIST. OF MUSKOKA COR 14
SCALE AS SHOWN	DISTRICT 52, HURTSVILLE	REGION NORTHERN
SURVEY DATE		PLAN DATE 98/08
SITE		PLAN

PLATE No.  
 DRAWING No.  
 CONT No  
 WP No 217-89-00

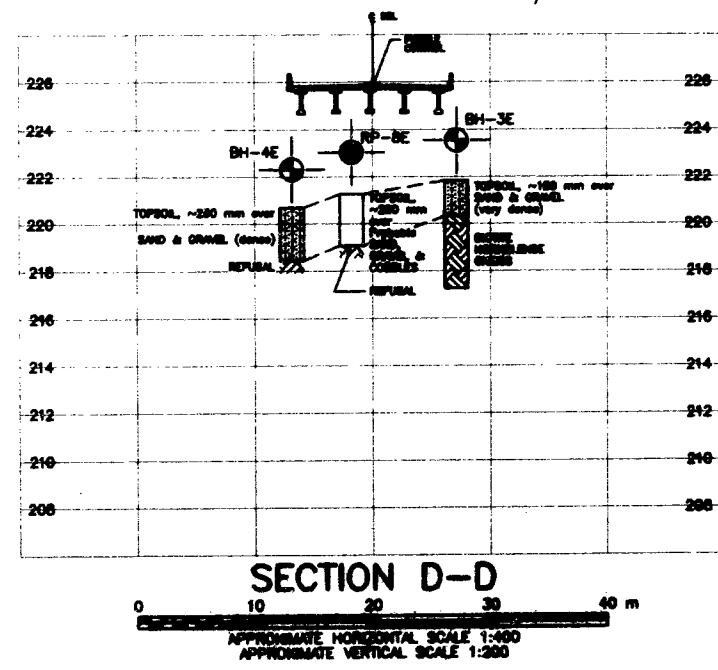
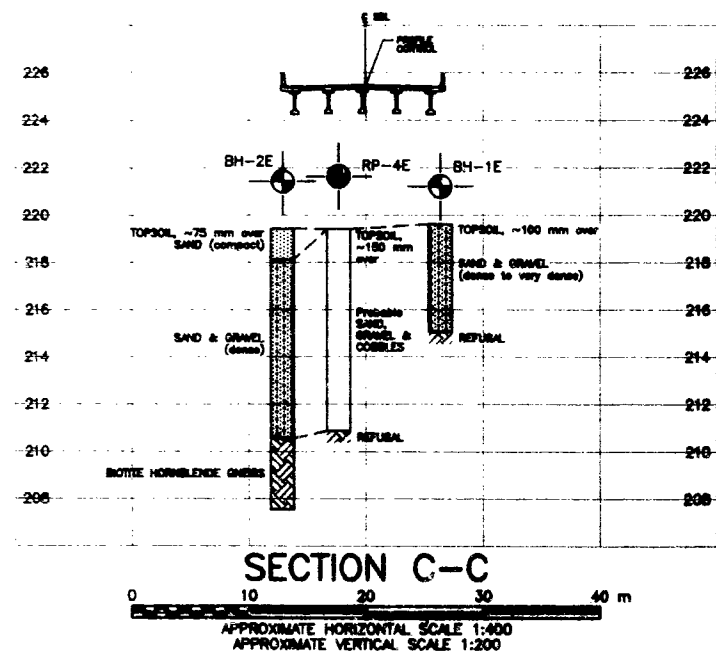
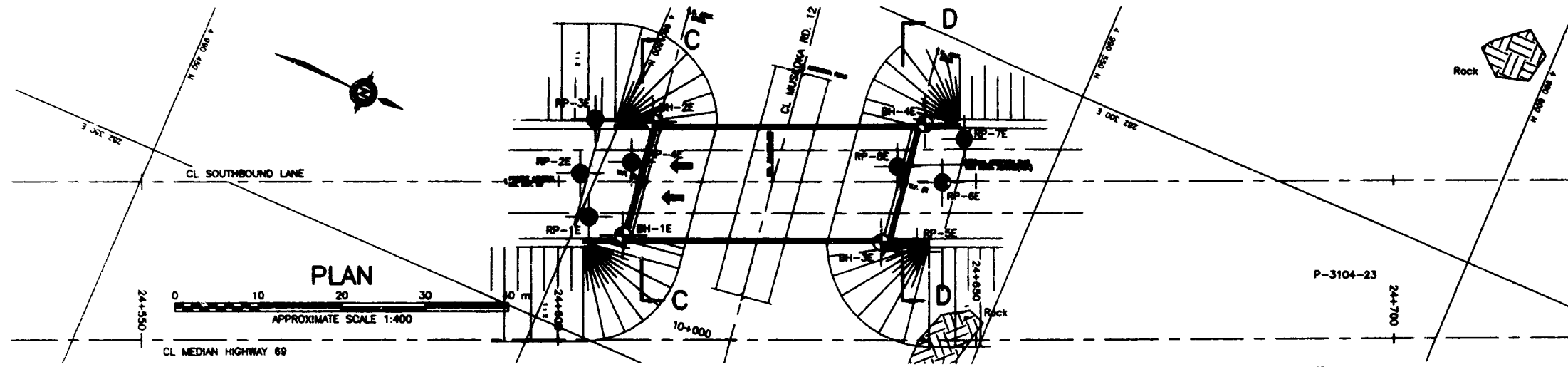
SHEET



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

TROW CONSULTING ENGINEERS LTD. SUDBURY, ONTARIO PROJ. No. S07857GE DWG. No. 18		
MINISTRY OF TRANSPORTATION SURVEYS AND PLANS SECTION		
SITE PLAN & SECTION B-B		
PROPOSED CROSSING AT MUSKOKA ROAD 12 SBL OF HIGHWAY 69		
CROSSING OF GIBSON LOT 14		DIST. OF MUSKOKA GIB. 14
SCALE AS SHOWN	DISTRICT S2, HARTVILLE	REGION NORTHERN
SURVEY DATE		PLAN DATE 09/98
SITE		PLAN

SHEET



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

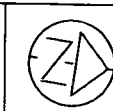
TROW CONSULTING ENGINEERS LTD.  
 SUDBURY, ONTARIO  
 PROJ. No. S078570E Dwg. No. 1C

SITE PLAN, SECTION C-C & D-D  
 PROPOSED CROSSING  
 AT  
 MUSKOKA ROAD 12  
 AND  
 SBL OF HIGHWAY 69

SCALE AS SHOWN	DATE 12/1/89	REVISION 01/01
SHEET NO. 1 OF 1 PLAN		

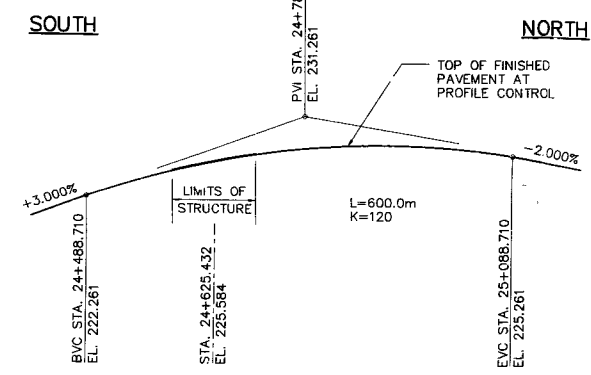
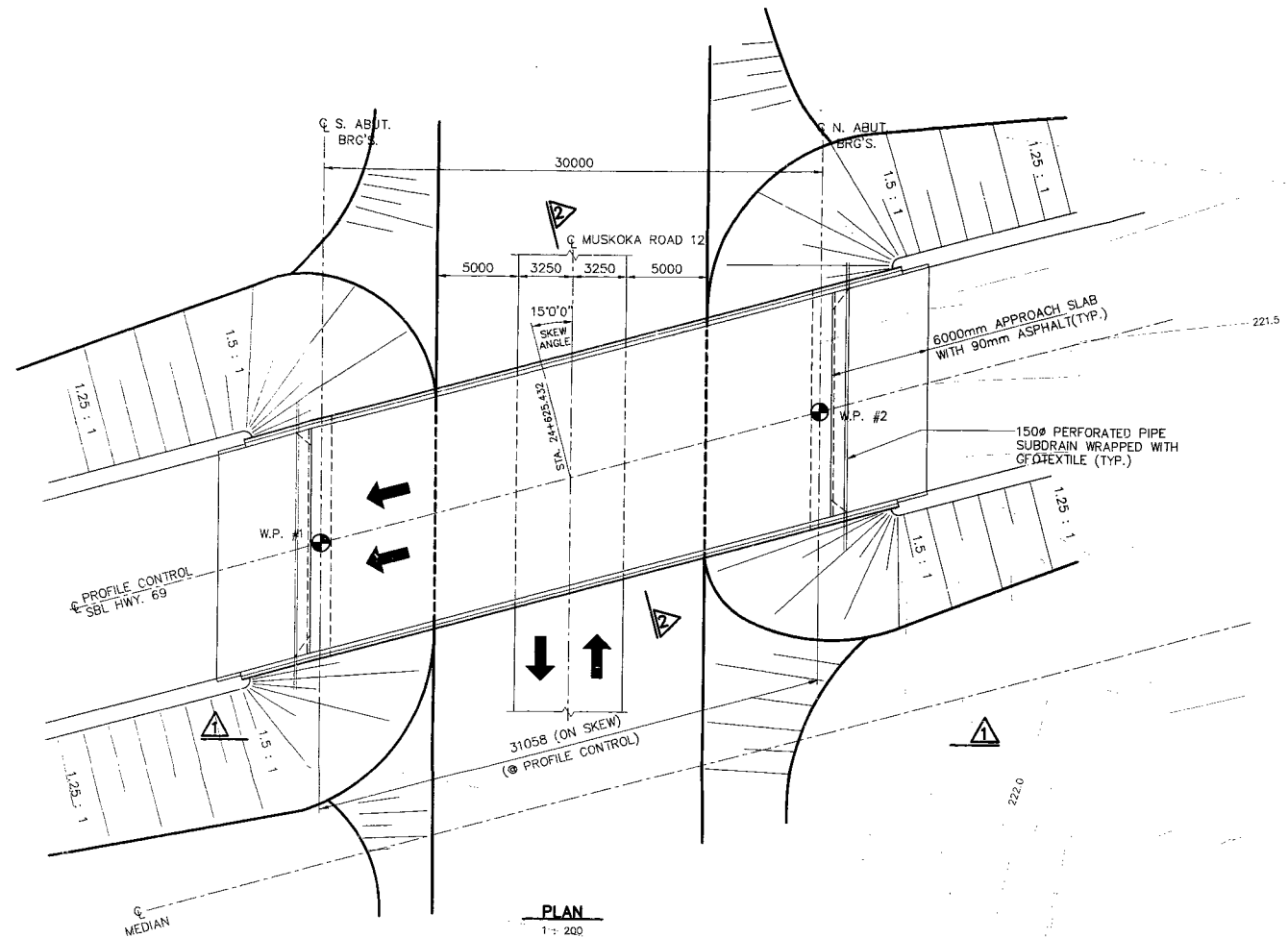
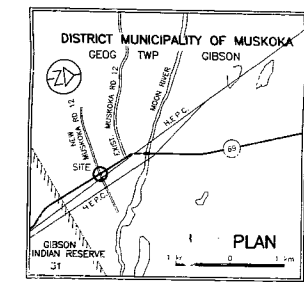
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AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

DIST HWY 69  
CONT No  
WP No 199-98-01

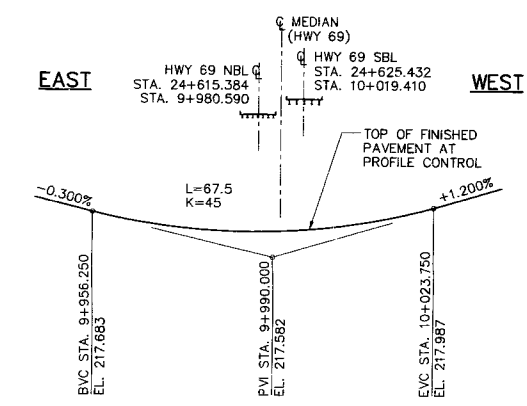


MUSKOKA ROAD 12 CROSSING  
HWY 69 SOUTHBOUND STRUCTURE  
GENERAL ARRANGEMENT

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consulting engineers, architects, technology managers 4971



PROFILE CONTROL - SOUTHBOUND LANES  
N.T.S.



PROFILE CONTROL - MUSKOKA ROAD 12  
N.T.S.

GENERAL NOTES

- CLASS OF CONCRETE:  
-PRECAST GIRDERS..... 40 MPa  
-REMAINDER..... 30 MPa
- CLEAR COVER TO REINFORCING STEEL:  
-FOOTINGS..... 100±25  
-DECK: TOP..... 70±20  
BOTTOM..... 40±10  
-REMAINDER..... 70±20  
UNLESS OTHERWISE NOTED.
- REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. UNLESS SHOWN OTHERWISE, TENSION LAP LENGTHS NOT INDICATED ON THE CONTRACT DRAWINGS SHALL BE CLASS B. HOOKS AND BENDS FOR REINFORCING STEEL SHALL BE DETAIL ACCORDING TO OHBDC-91. UNLESS SHOWN OTHERWISE, THE FOLLOWING SHALL APPLY:  
a) STANDARD HOOKS WITH MINIMUM BEND DIAMETERS SHALL BE USED FOR STIRRUPS AND TIES, ACCORDING TO CLAUSE B-14.1.  
b) OTHER BARS SHALL HAVE STANDARD HOOKS WITH BEND DIAMETERS ACCORDING TO CLAUSE C-8-14.1.

CONSTRUCTION NOTES

- THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.
- NO BACKFILL TO BE PLACED BEHIND ABUTMENTS UNTIL CONCRETE IN DECK HAS REACHED 75% OF ITS SPECIFIED STRENGTH.
- BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND ABUTMENTS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN THE BACKFILL HEIGHTS BE GREATER THAN 500mm.

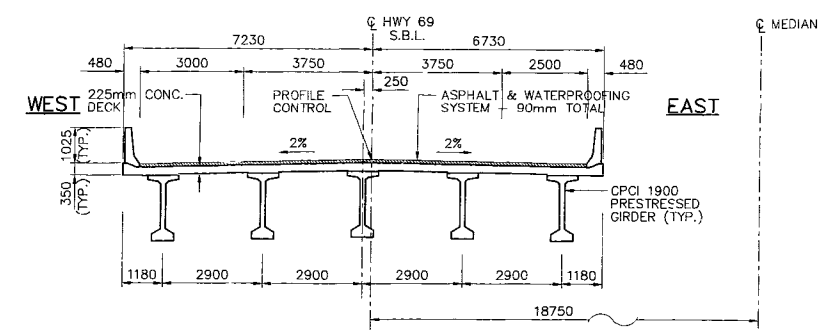
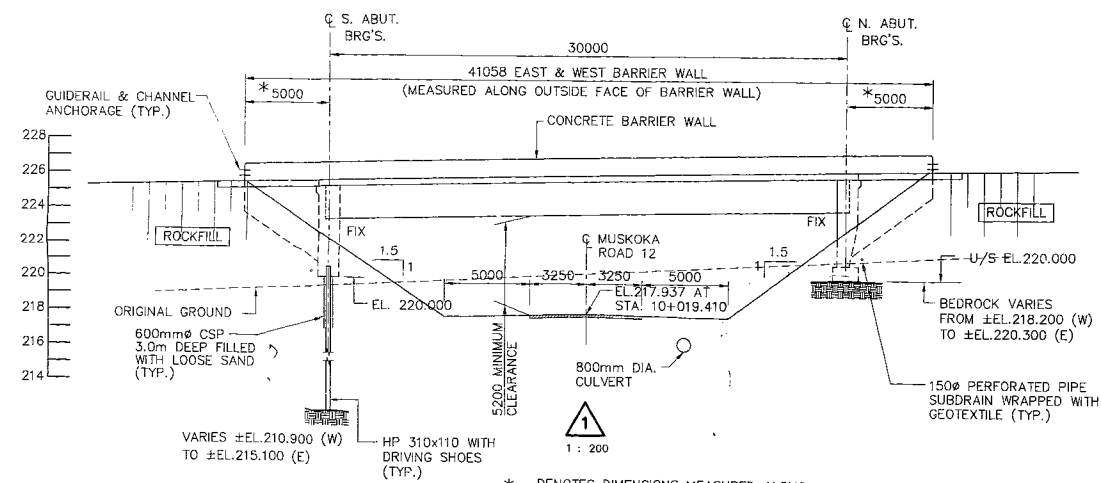
LIST OF DRAWINGS

- GENERAL ARRANGEMENT
- FOUNDATION LAYOUT AND FOOTING REINFORCEMENT
- NORTH ABUTMENT
- NORTH WINGWALLS
- SOUTH ABUTMENT
- SOUTH WINGWALLS
- PRESTRESSED GIRDERS AND BEARINGS
- DECK DETAILS
- BARRIER WALL
- 6000mm APPROACH SLAB
- STANDARD DETAILS

APPLICABLE STANDARD DRAWINGS

OPSD 3501.00	GRANULAR BACKFILL REQUIREMENTS - ABUTMENTS
OPSD 3505.00	ROCK BACKFILL REQUIREMENTS - ABUTMENTS
OPSD 4010.00	GUIDE RAIL AND CHANNEL ANCHORAGE

WORKING POINTS			
STATION AT ROAD	T/A ELEVATIONS	COORDINATES	
W.P. #1	24+609.903	225.285	N 4990505.219 E 282330.294
W.P. #2	24+640.961	225.863	N 4990533.728 E 282317.971



PRELIMINARY  
OCT 14 1998



DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

- NOTES:
- ALL PILES TO BE DRIVEN TO BEDROCK.
  - ROCK ELEVATIONS ARE APPROXIMATE WITH REFERENCE TO SOIL INVESTIGATION REPORT 5076576/E DATED SEPTEMBER 23, 1998.

REVISIONS	DESCRIPTION
DESIGN KN	CHK
DRAWN HC	CHK
CODE OHBDC'91	LOAD OHBDC'91
SITE 42-3195	STRUCT 1
SCHEME	DWG 1
DATE	AUG 1998