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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 analyzed

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, ρ_c , (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, γ , (kN/m ³)	Effective Friction Angle, ϕ' , (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, σ'_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/σ'_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/σ'_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	σ'_p (kPa)	$C_v^{O/C}$ (m^2/yr)	$C_v^{N/C}$ (m^2/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

σ'_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
4 m.

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.

how? 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.

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 by
an expert
Consultant*

Sieve Size	Percent Passing (%)
9.5 mm	100
4.75 mm	95-100
2.36 mm	80-100
1.18 mm	50-85
600 μ m	25-60
300 μ m	10-30
150 μ 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[®] 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: $\leq 75\mu$ m

6. Permeability of non-woven filter (ASTM D4491): 1×10^{-4} m/sec.
7. Minimum Width: > 95mm
8. Discharge Capacity (ASTM D4716): 60×10^{-6} m³/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 coarse. 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

*Spacing
is changed*

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

*Spacing
changed*

**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.

Er Cheng

for

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

SE
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Figures

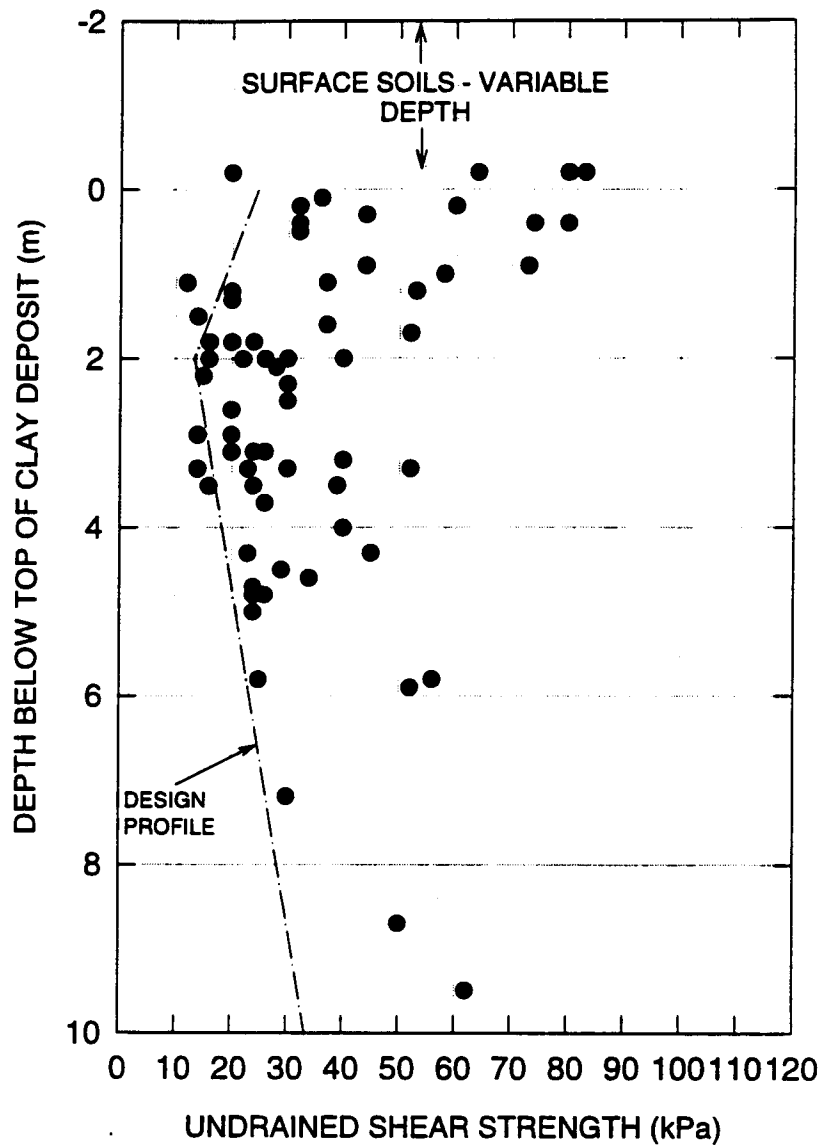
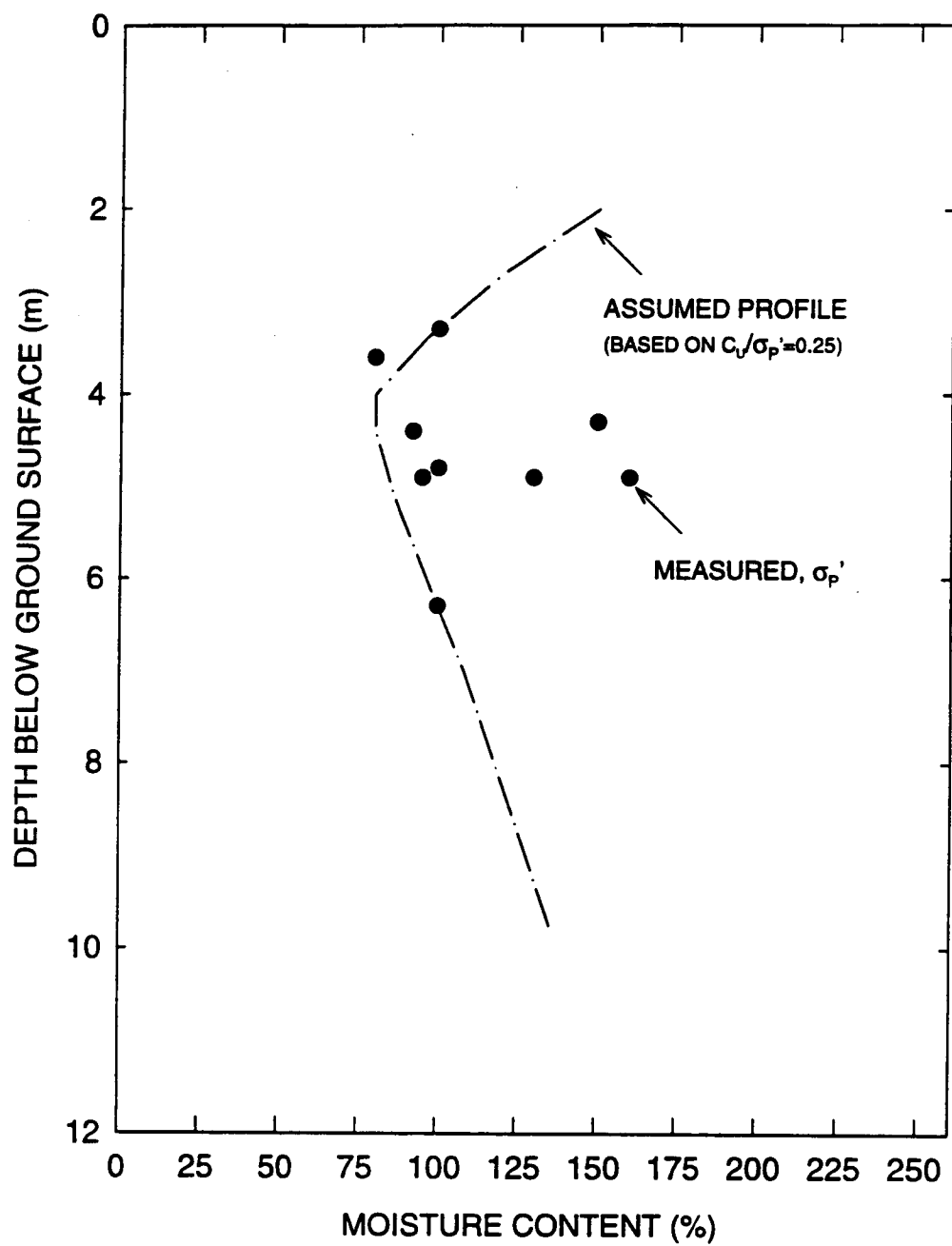


FIGURE 1 SUMMARY OF UNDRAINED SHEAR STRENGTH MEASUREMENTS - HIGHWAY 69



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

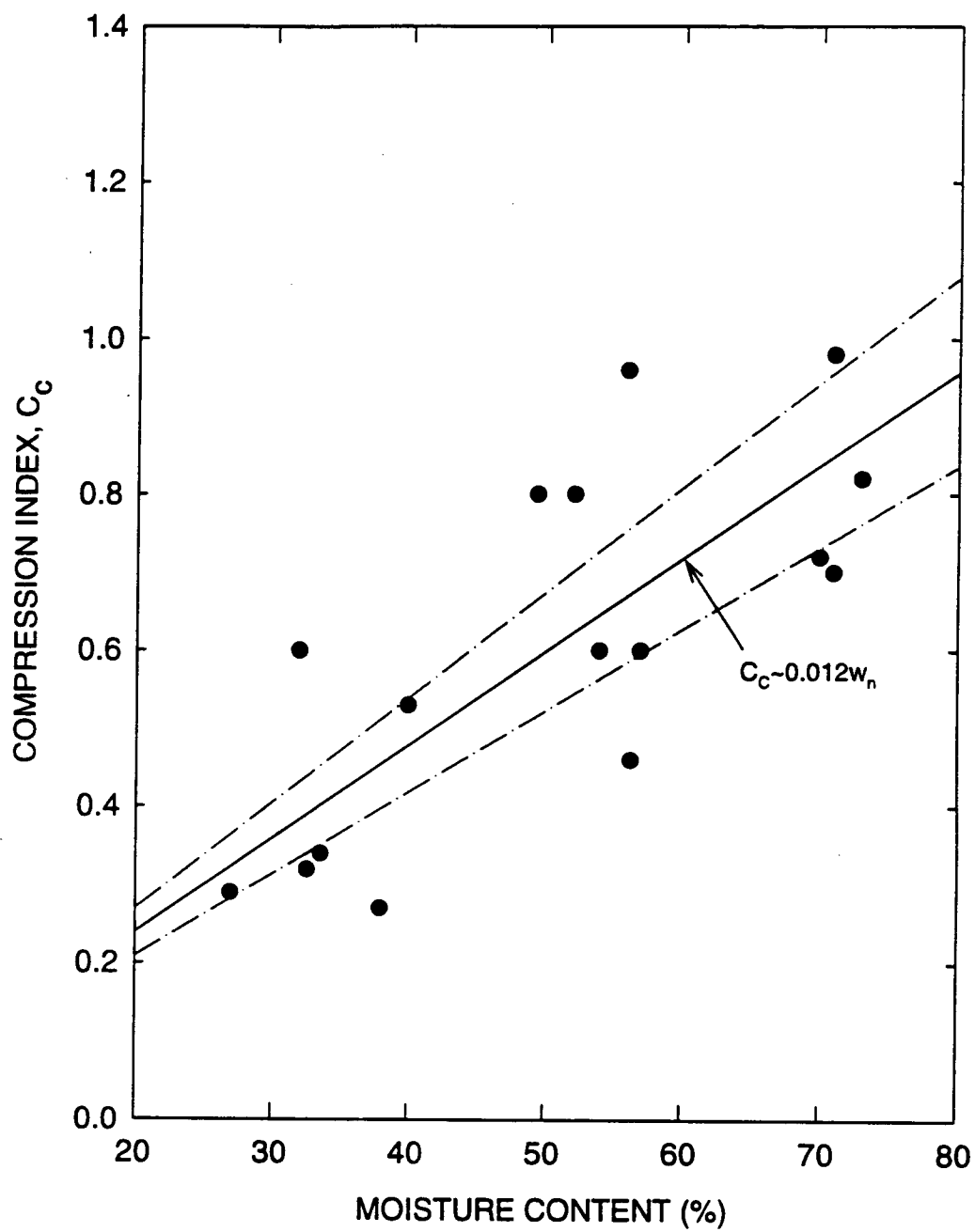


FIGURE 3 RELATIONSHIP BETWEEN MOISTURE CONTENT AND COMPRESSION INDEX, C_c

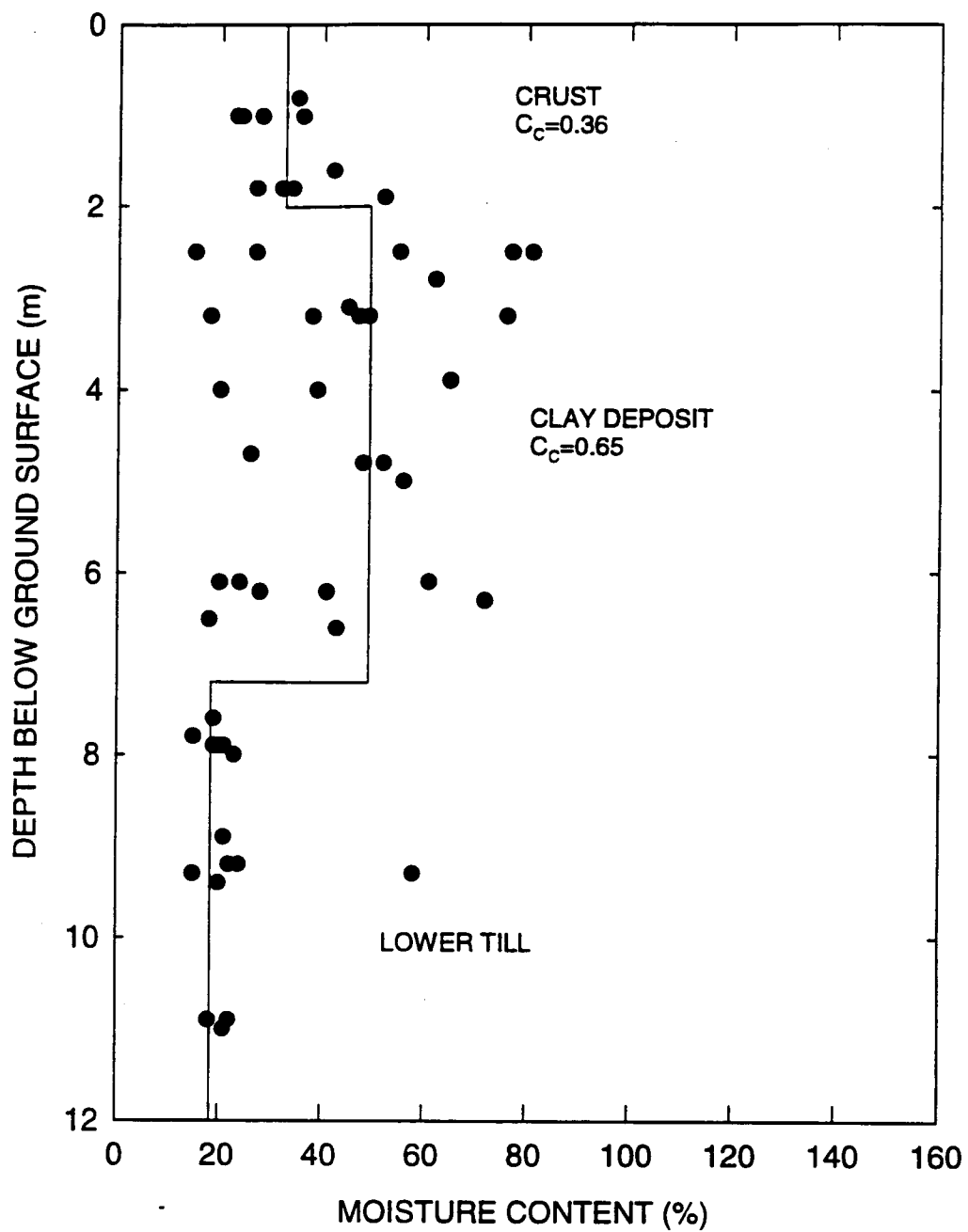


FIGURE 4 SUMMARY OF MOISTURE CONTENTS

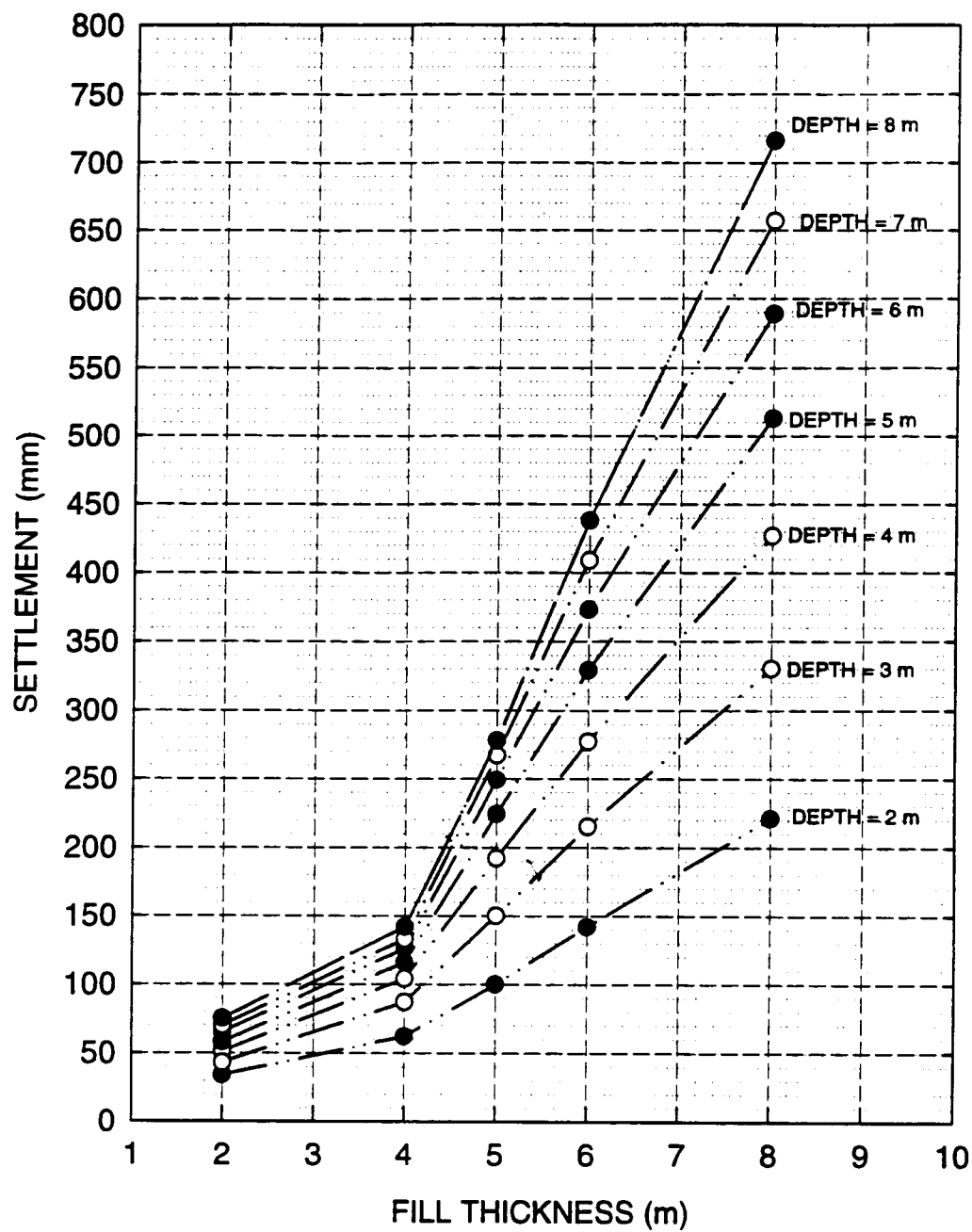


FIGURE 5 CALCULATED SETTLEMENT VERSUS FILL THICKNESS
HIGHWAY 69

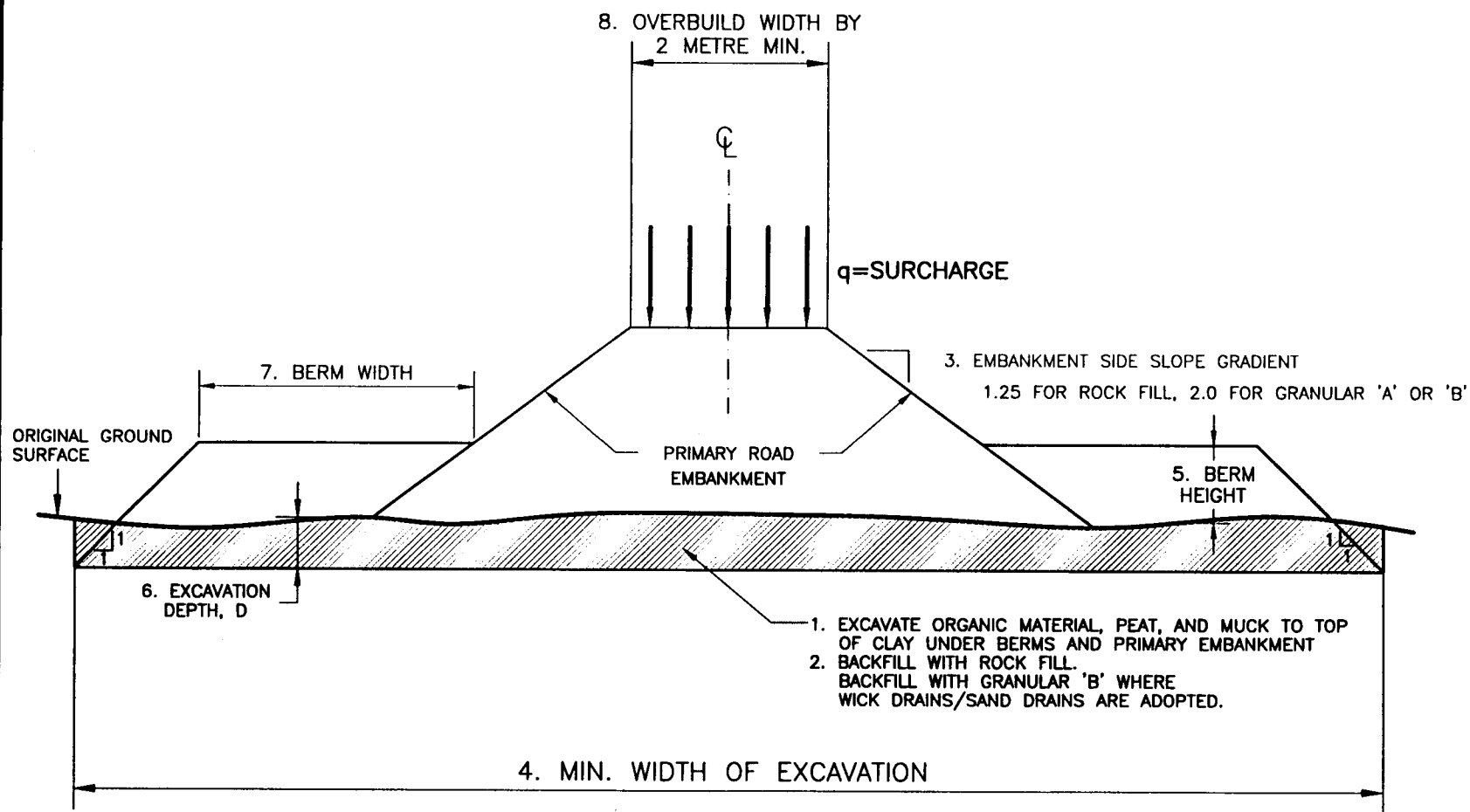


FIGURE 6: GENERALIZED ROAD EMBANKMENT GEOMETRY— HIGHWAY 69, 0.4km SOUTH OF MUSQUASH RIVER, NORTHERLY 8.9 km TO TOWER ROAD.

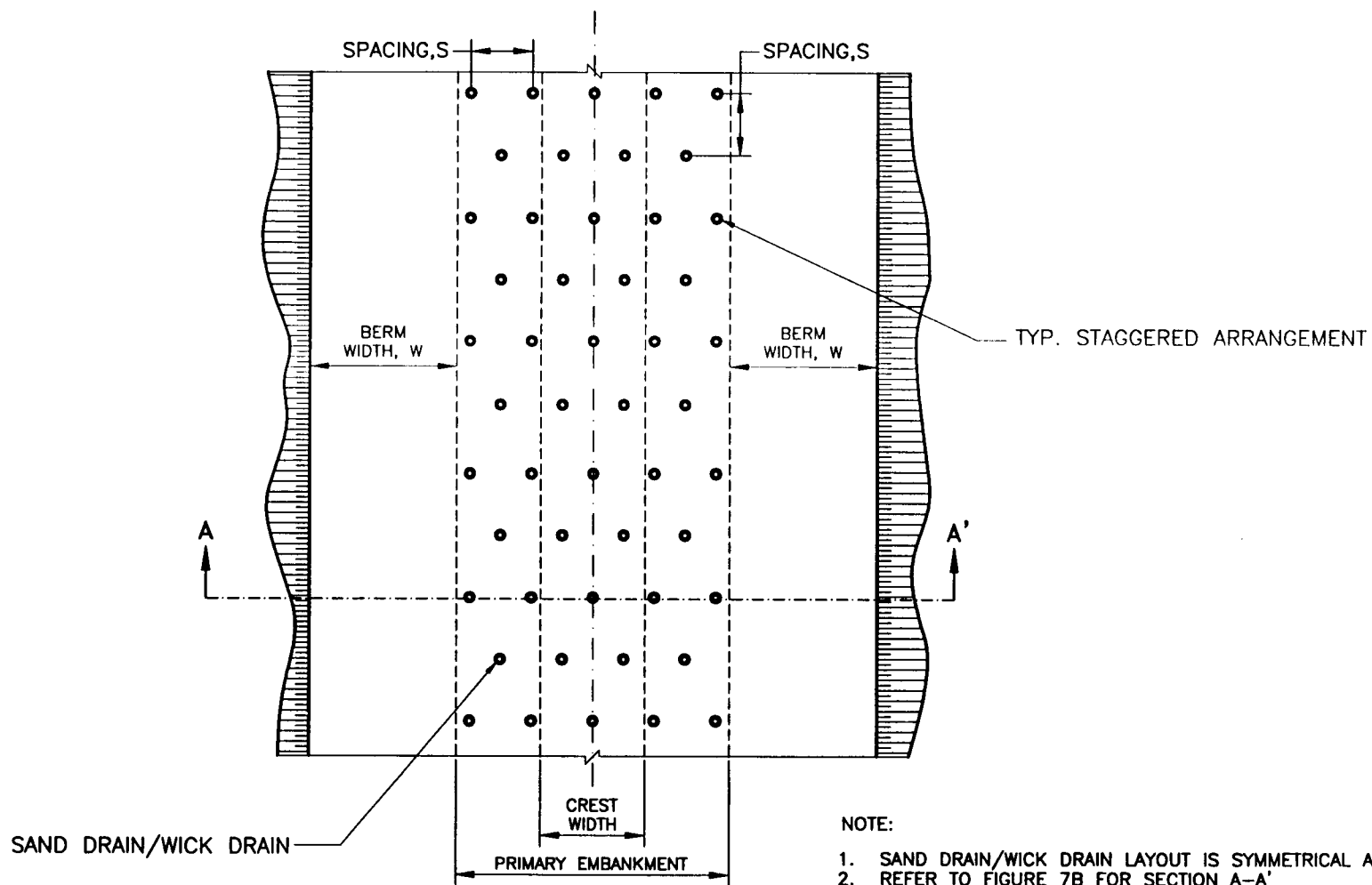


FIGURE 7A: SAND DRAIN/WICK DRAIN LAYOUT. HIGHWAY 69, 0.4km SOUTH OF MUSQUASH RIVER, NORTHERLY 8.9 km TO TOWER ROAD - PLAN VIEW

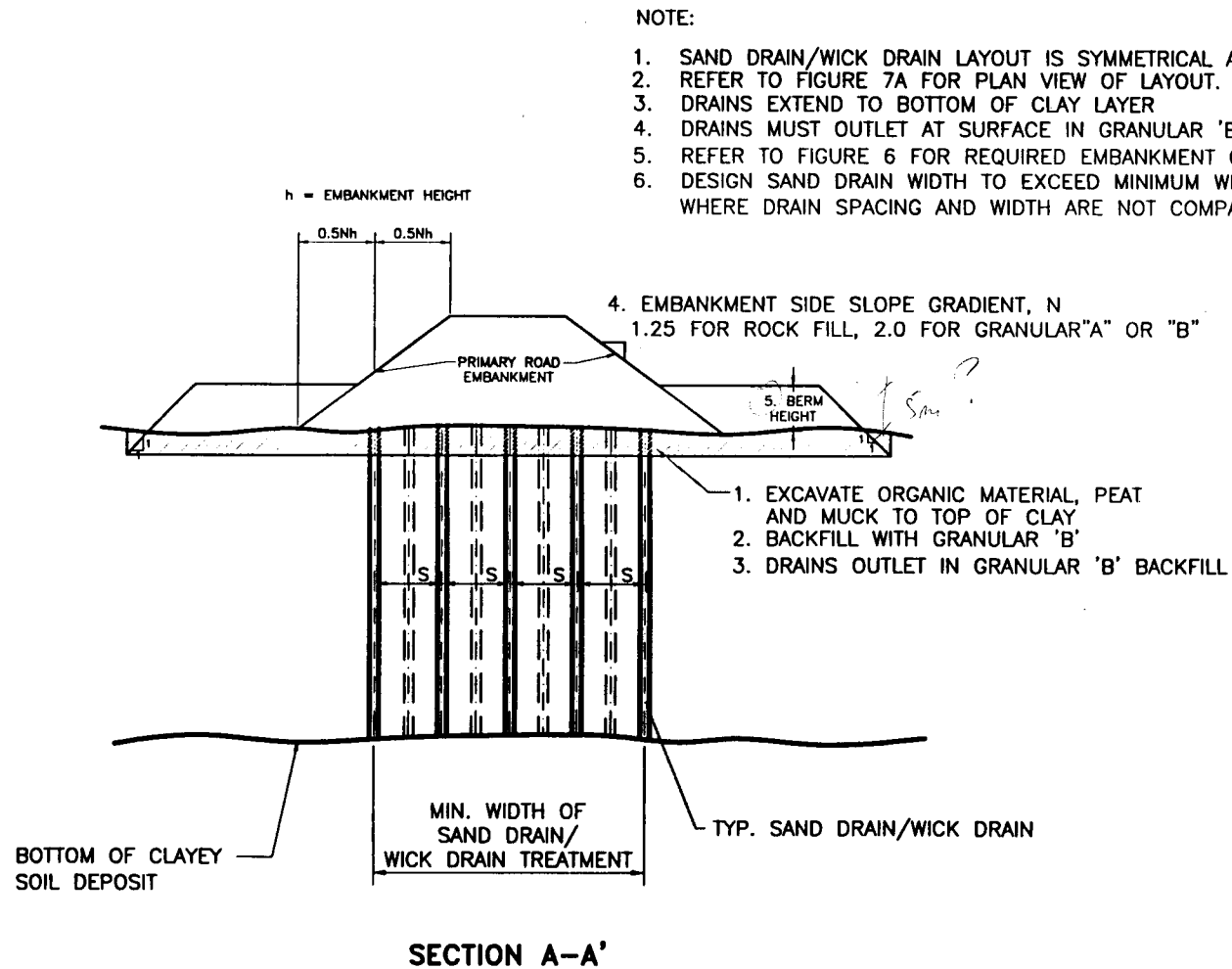


FIGURE 7B: SAND DRAIN/WICK DRAIN LAYOUT. HIGHWAY 69, 0.4km SOUTH OF MUSQUASH RIVER, NORTHERLY 8.9 km TO TOWER ROAD - SECTION VIEW

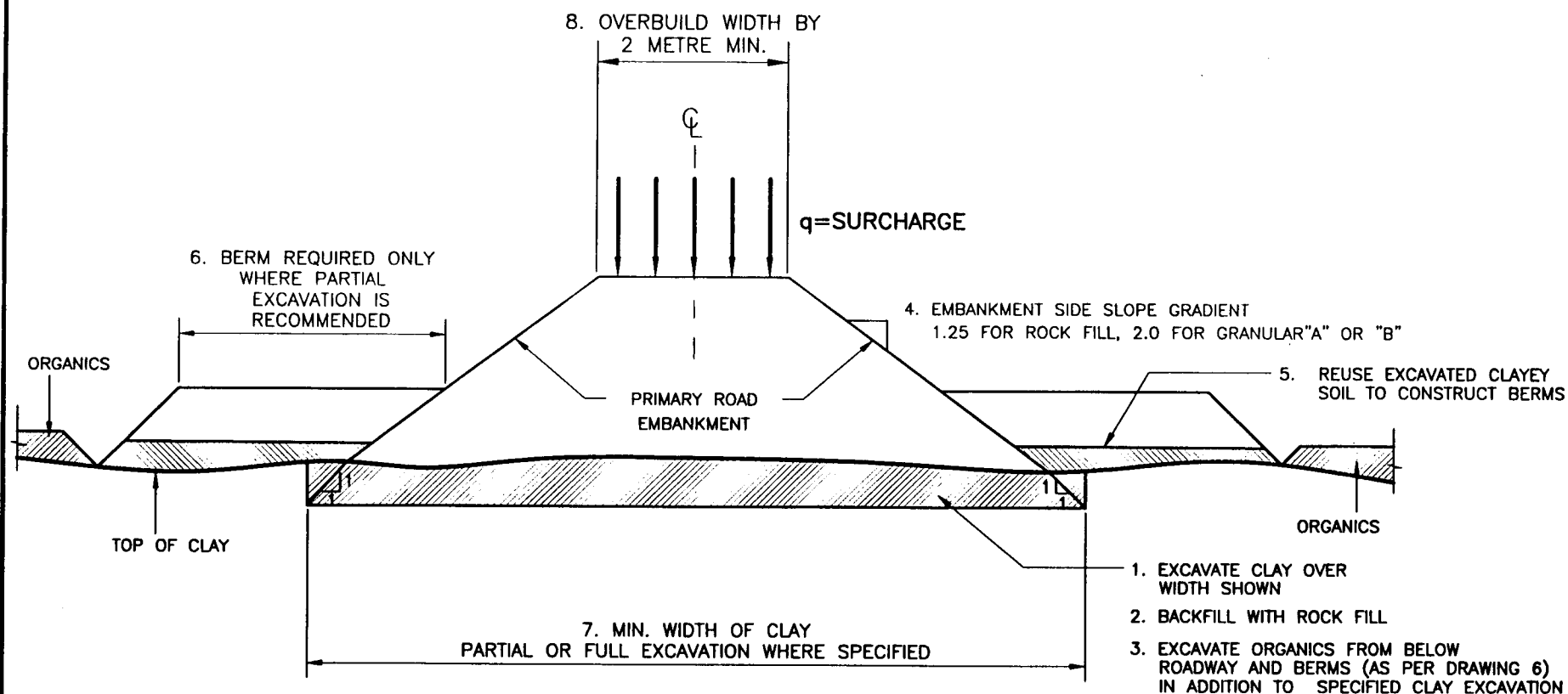


FIGURE 8: ROAD EMBANKMENT GEOMETRY— PARTIAL OR FULL EXCAVATION OF CLAYEY SOIL STATIONS 21+720 TO 21+810, HIGHWAY 69.

Appendix A: Summary of Stability Calculations

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

Station 20+150

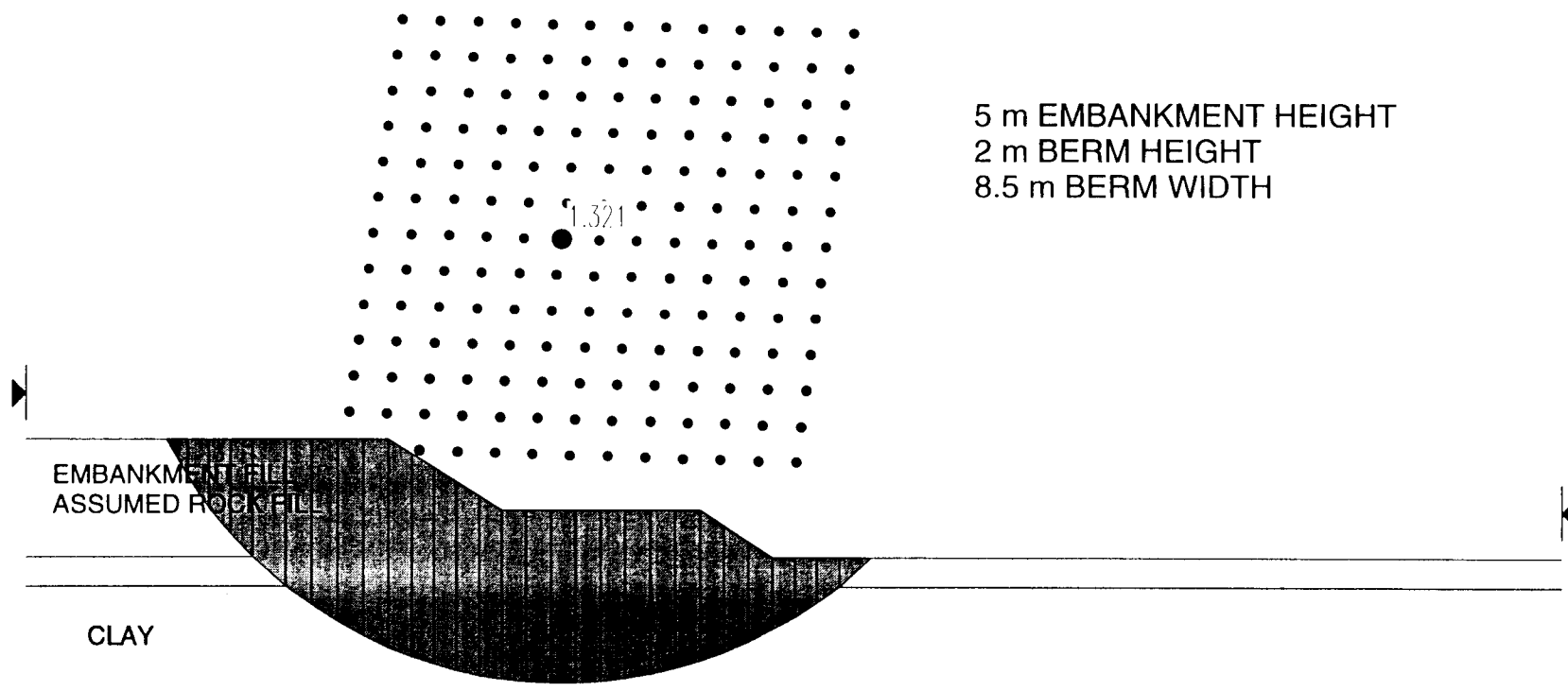


Figure A2: Southbound Lanes Between Stations 20+425 to 20+790

Station 20+750

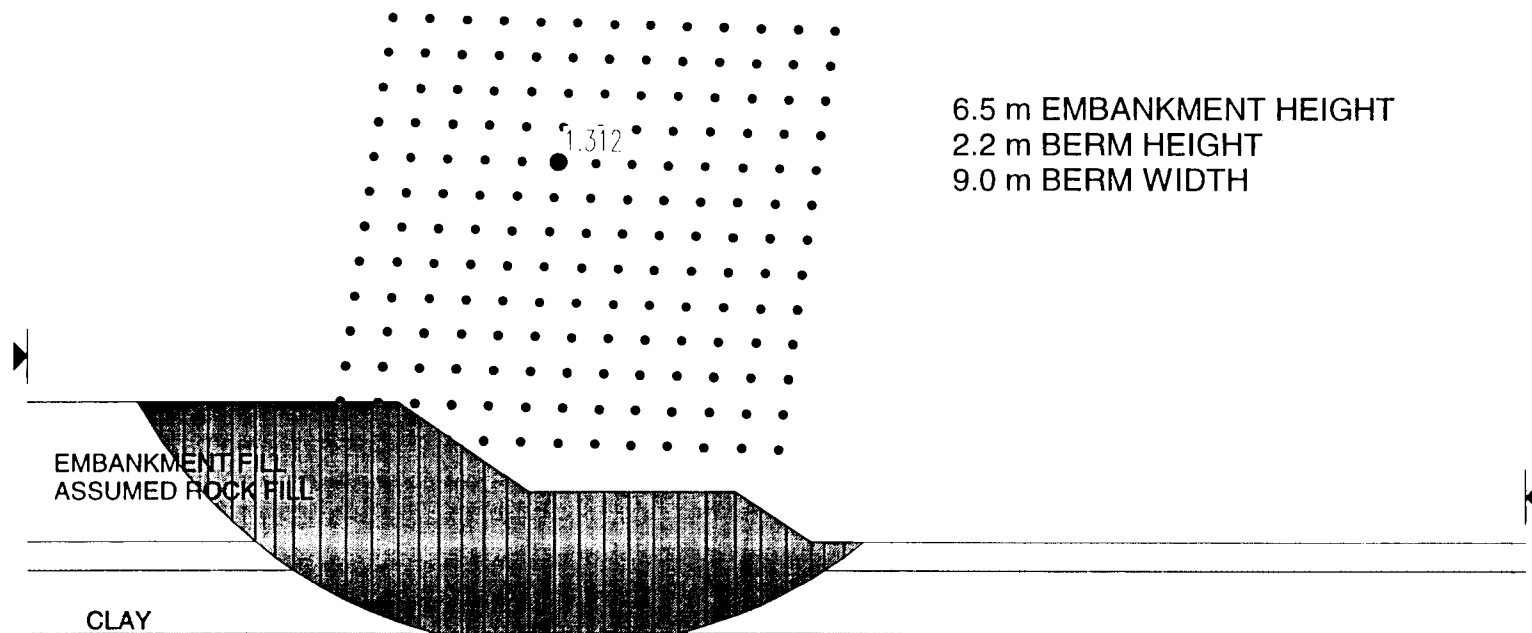


Figure A8: Northbound Lanes Between Stations 20+260 to 20+375

Station 20+300

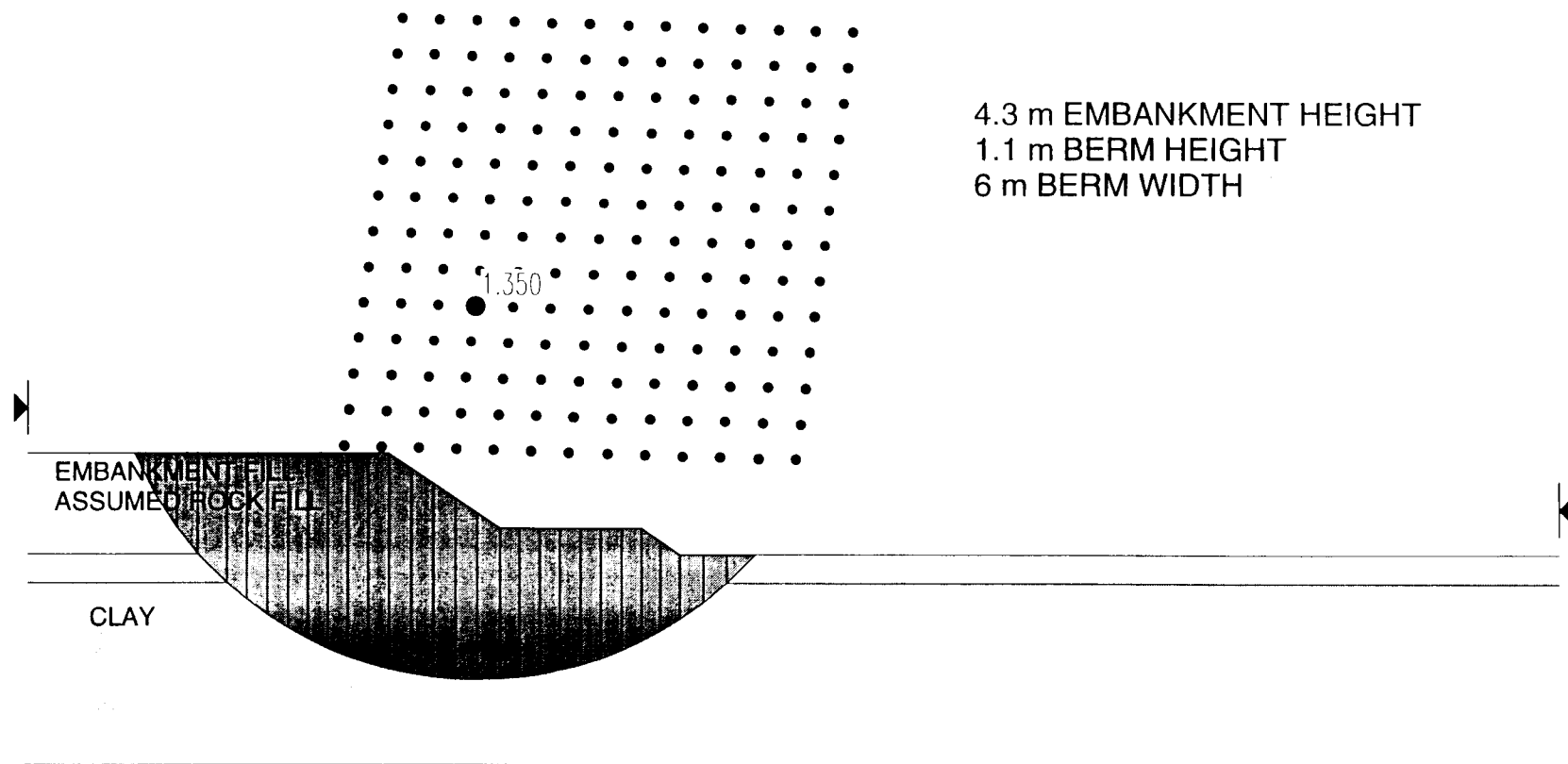
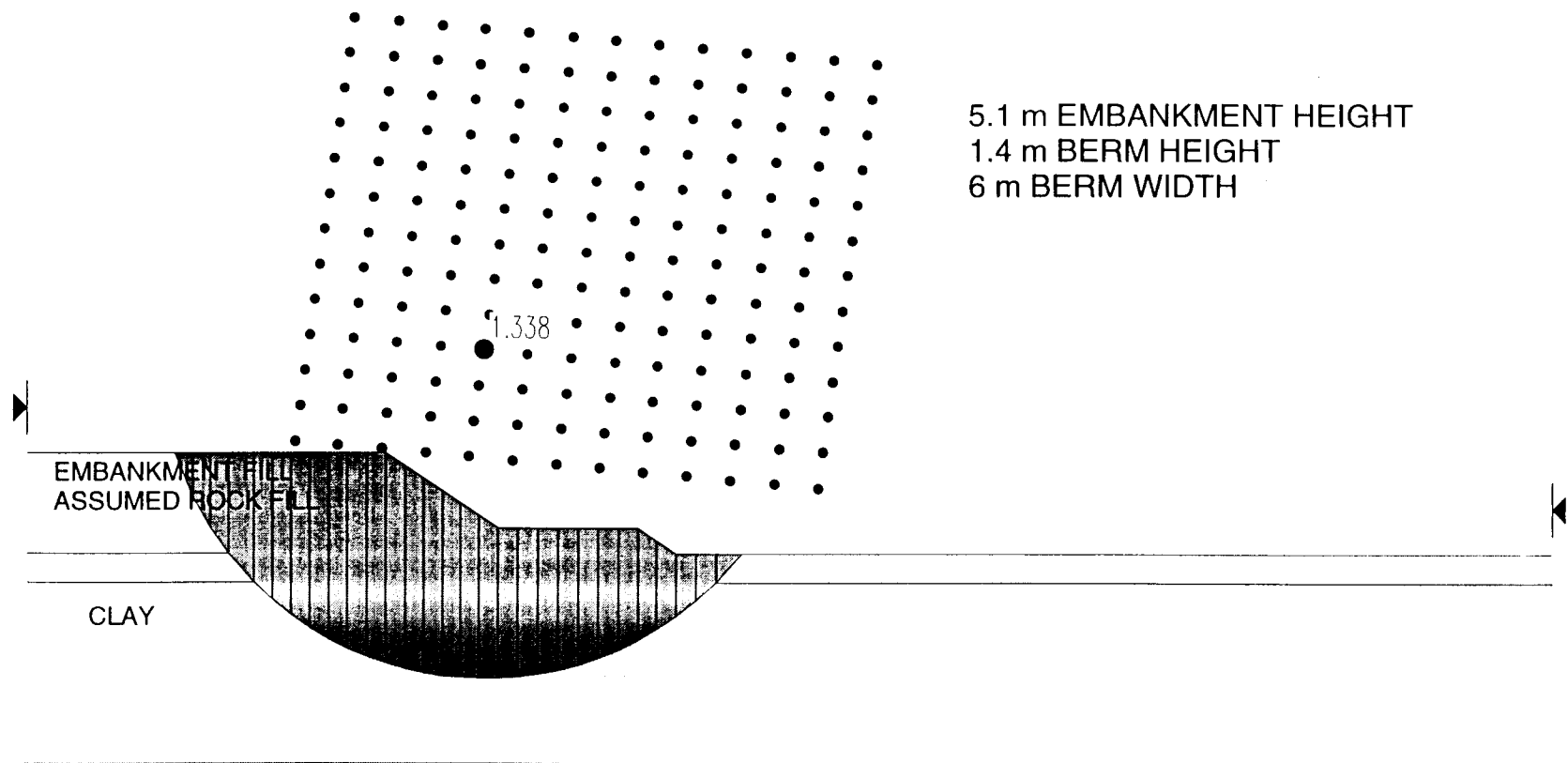


Figure A9: Northbound Lanes Between Stations 20+450 to 20+780

Station 20+600



Appendix B: Borehole Logs Northbound Lanes - Highway 69

METRIC

CONT No
WP 217-89-00

SHEET
B1

LOT 16
CON 10

N 4 986 300

N 4 986 500

N 4 986 500

E 282 800

20+200

20+100

20+000

19+900

1226 X 21.1 CSP

1.0-2.5 RC

BM

480 X 8.7

7.5 RC

3CGR

MUS-1-1

A=350

3-1

3-2

3-3

MUS-1-2

MUS-1-3

MUS-1-4

MUS-1-5

MUS-1-6

4.0 RC

HCP 8690717

8.5 RC

3CGR

P-3104-7

BM

O H&B

3CGR

O H&B

E 282 900

P-3104-6

E 282 900

P-3104-6

LEGEND

⊕ BOREHOLE & CONE
⊕ CONE



TROW CONSULTING ENGINEERS LTD.
SUDBURY, ONTARIO
Trow PROJ. No. S07404G DWG. No. B1

SPARTAN TRAIL

RESERVE No 31

LOT 15
CON 9

WEST LIMIT GIBSON INDIAN

GEOG TWP GIBSON

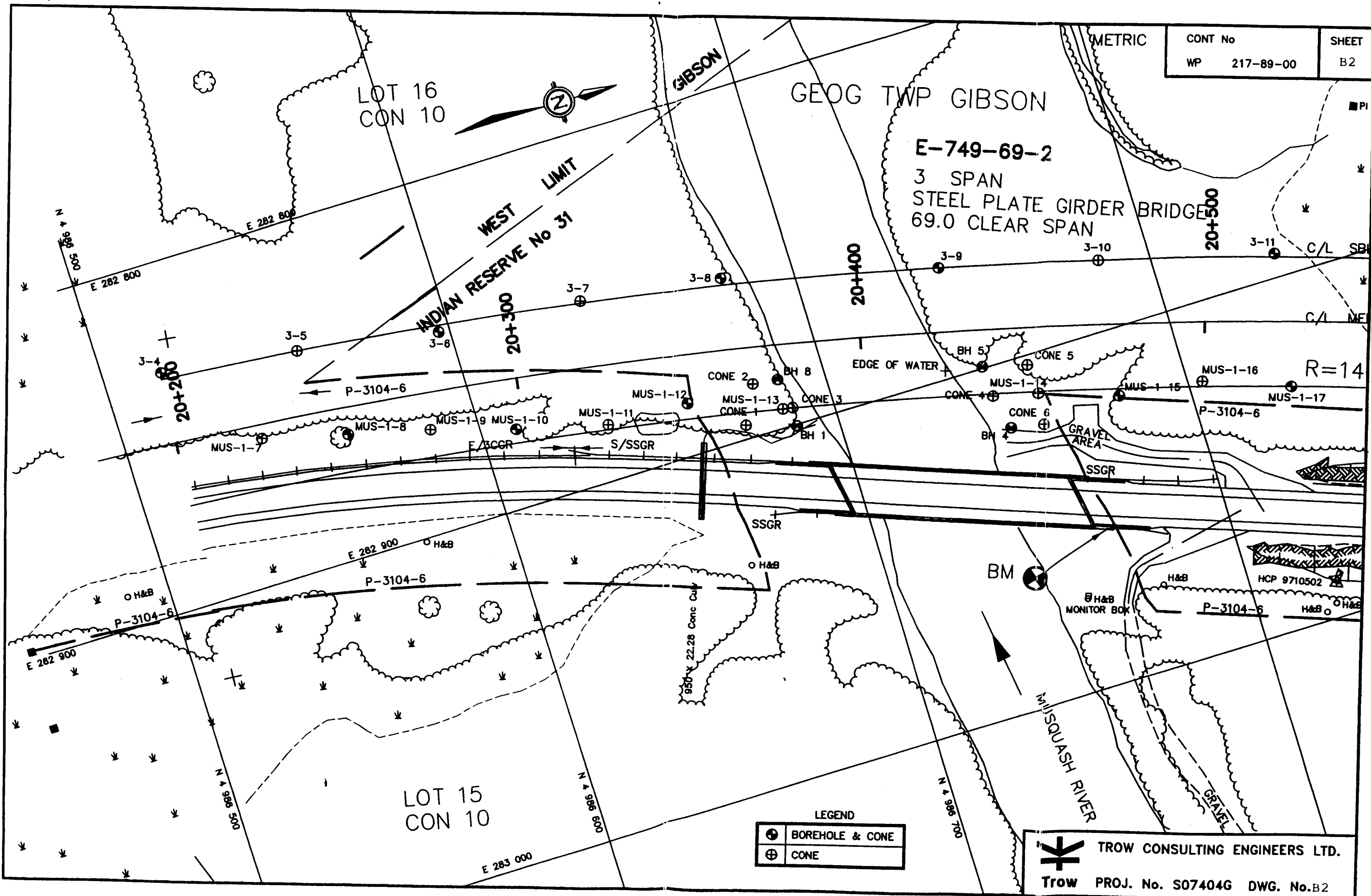
E-749-69-2

3 SPAN
STEEL PLATE GIRDER BRIDGE
69.0 CLEAR SPAN

WEST LIMIT
INDIAN RESERVE No 31

LOT 16
CON 10

LOT 15
CON 10



LEGEND

⊕	BOREHOLE & CONE
⊗	CONE

GEOG TWP GIBSON

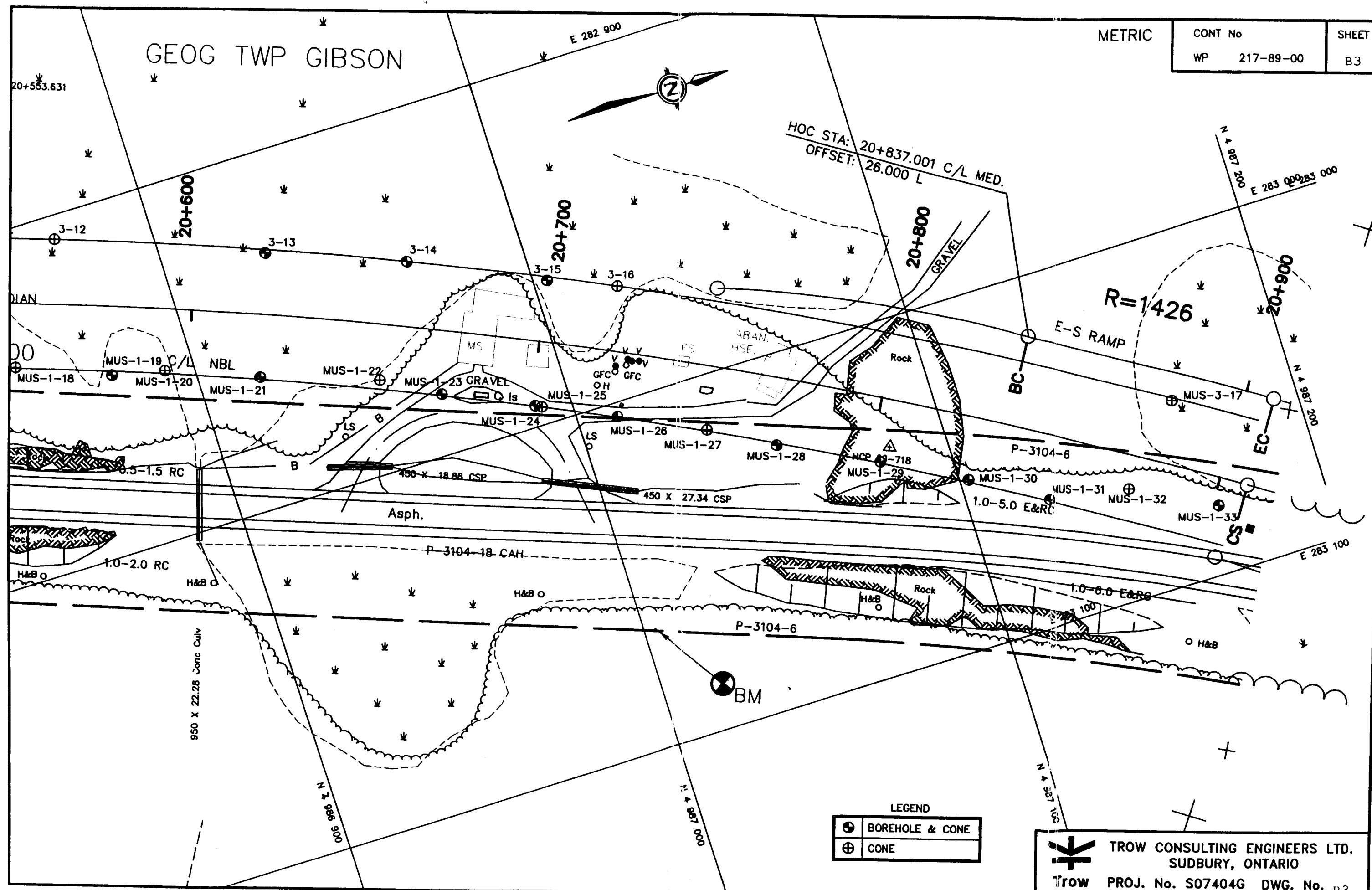
METRIC

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

SHEET

WP 217-89-00

B4

GIBSON INDIAN RESERVE No.31

CON 10
LOT 14HOT STA: 21+247.000 C/L MED.
OFFSET: 26.300 LTHOC STA: 21+247.000 EW-S RAMP
OFFSET: 5.550 RT
N 4987471.556
E 283249.570

A= 354.863

R=1500

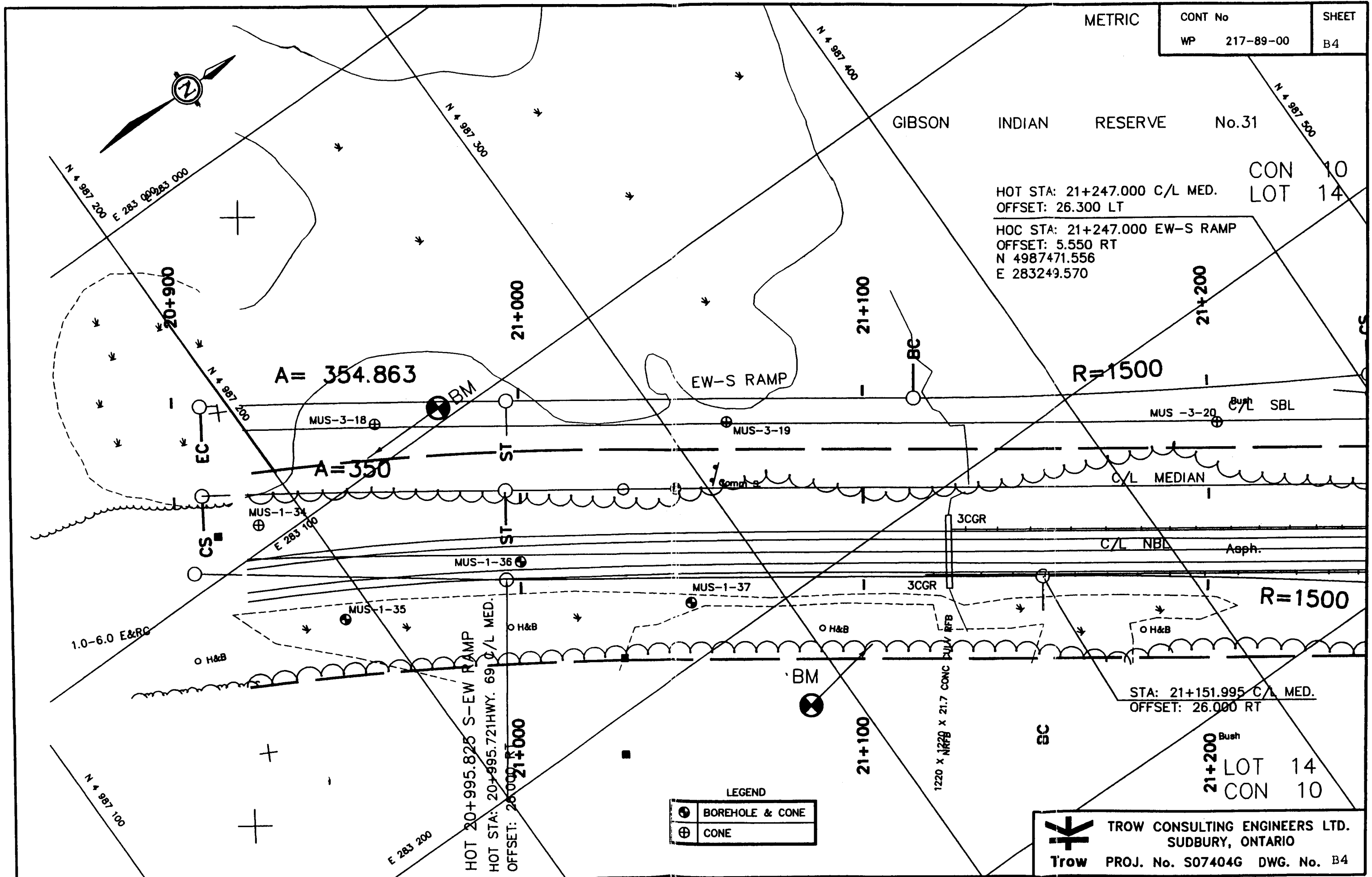
A=350

R=1500

STA: 21+151.995 C/L MED.
OFFSET: 26.000 RT21+200 Bush
LOT 14
CON 10

LEGEND

⊕	BOREHOLE & CONE
⊗	CONE

TROW CONSULTING ENGINEERS LTD.
SUDBURY, ONTARIO
Trow PROJ. No. S07404G DWG. No. B4

RECORD OF BOREHOLE MUS-1-15 1 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			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				wp ——— w ——— wl				
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL FIELD VANE LAB SHEAR				WATER CONTENT (%) 10 20 30 40				
197.36	GROUND SURFACE															
0.00	TOPSOIL, 180 mm over SAND, brown, some silt. (compact)		1	SS	14		197									
195.36			2	SS	23		196									
2.00	SILTY CLAY TO CLAYEY SILT, some silt & fine sand seams, brown to grey. (stiff)		3	SS	5		195									
192.36			4	SS	1		194									
			5	TW			193									
5.00	SILT, some cobble sizes & sand content, grey. (loose)		6	SS	6		192									
190.35							191									
7.01	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER															



RECORD OF BOREHOLE MUS-1-16₁ OF 1

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	80	Wp	W		
196.86 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 1 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) 				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									
						SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)					
						UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR				wp — w — wl					
						20 40 60 80				10 20 30 40					
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_{1 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)				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					
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_{1 OF 1}

METRIC

W.P. 217-89-00 LOCATION Station 20+577.5, offset 1.1 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 30, 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					
197.06	GROUND SURFACE											
0.00	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	AS								
			4	TW								
193.10	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER										16.50	
3.96												



RECORD OF BOREHOLE MUS-1-20_{1 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) 				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					
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	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRATA PLOT NUMBER	TYPE			BLOWS/10 cm	20 40 60 80					
196.49 0.00	GROUND SURFACE											
	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 8.00	SAND, grey, brown. (compact to dense)	7	SS	3								
		8	SS	31								
		9	SS	23								
184.49 12.00	SILTY SAND & GRAVEL TILL, brown	10	SS	76								
183.84 12.65	(dense) END OF BOREHOLE											



RECORD OF BOREHOLE MUS-1-22₁ 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 ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA SI CL			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60						80	20	40
197.90 0.00	GROUND SURFACE																	
195.90 2.00	Possible FILL	F																
	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_{1 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		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w	wl			10
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-1-24₁ OF 1

METRIC

W.P. 217-89-00

LOCATION Station 20+701.2, offset 0.1 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 (N-Value)				PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			CONES PENETRATION TEST 20 40 60 80	UNCONFINED QUICK TRIAXIAL	Cu, KPa	FIELD VANE LAB SHEAR	20 40 60 80	wp	wl	10 20 30 40	10 20 30 40			
198.44	GROUND SURFACE																	
0.00	FILL, sand & gravel with pieces of asphalt & pockets of peat, brown, (compact)																	
196.44			1	SS	3													
2.00	SILTY CLAY TO CLAYEY SILT, seams of silt & fine sand, red/brown & grey. (firm)		2	SS	9													
			3	SS	2													
189.60																		
8.84	END OF SAMPLED BOREHOLE																	
	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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA SI CL
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													
	Probable SILTY CLAY to CLAYEY SILT	H													
191.39 7.00	Probable SANDY SILT to SAND	S													
185.22 13.17	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER														



RECORD OF BOREHOLE MUS-1-26_{1 OF 1}

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 (N-Value) 				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/30.3m	CONE PENETRATION TEST				WATER CONTENT (%)		
						20	40	60	80	wp	w	wl		
						SHEAR STRENGTH: Cu, KPa								
						UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR								
						20	40	60	80	10	20	30	40	
198.02	GROUND SURFACE													
0.00	GRANULAR FILL, 75 mm over SAND & GRAVEL FILL, pieces of wood & other organics, grey. (very loose)	F	1	SS	3									
			2	SS	4									
195.62			3	SS	5									
2.40	SILTY CLAY TO CLAYEY SILT, small seams of silt & fine sand, red/brown & grey. (stiff to firm)		4	SS	2									
			5	TW										
191.77														
6.25	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER													
	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					
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_{1 OF 1}

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60						80	Wp	W
198.09	GROUND SURFACE																	
0.00	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	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER																	
4.33																		



RECORD OF BOREHOLE MUS-1-29₁ OF 1

METRIC

W.P. 217-89-00

LOCATION Station 20+803.3, offset 0.4 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			NATURAL MOISTURE CONTENT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	CONE PENETRATION TEST				WATER CONTENT (%)								
						20	40	60	80	wp	w	wl	10	20	30	40	GR	SA	SI	CL
203.24	GROUND SURFACE																			
203.23	SAND, brown, moist																			
0.25	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER																			



RECORD OF BOREHOLE MUS-1-30^{1 OF 1}

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	REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40						60	80	Wp
199.29 0.00	GROUND SURFACE SILTY SAND, brown, moist.																
197.99 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)		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	30	40	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																
			3	AS																	
195.02	SANDY SILT TO FINE SAND, grey. (loose to compact)		4	SS	5																
3.00																					
192.69	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER																				
5.33																					



RECORD OF BOREHOLE MUS-1-32₁ 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 SHEAR STRENGTH: C_u , KPa ● UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR 20 40 60 80	PLASTIC LIMIT WD — W — WL	NATURAL MOISTURE CONTENT W	LIQUID LIMIT WL	WATER CONTENT (%) 10 20 30 40	UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER TYPE BLOWS/0.3m									
197.72 0.00	GROUND SURFACE											
	Probable SANDY SILT with a trace of clay				197							
					196							
					195							
					194							
					193							
					192							
					191							
					190							
					189							
					188							
188.72 9.00	Probable SAND & GRAVEL											
187.05 10.67	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER											



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.

[illegible]

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 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
199.80	GROUND SURFACE														
0.00	ASPHALT, ~150 mm over SAND & GRAVEL FILL, ~300 mm over														
198.20	SAND, light brown becoming red/brown between ~0.7 to 1.0 m depth then dark brown, moist. (compact)														
1.40															
197.20	SILTY CLAY, some organic content, dark brown, moist (firm)														
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_{1 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	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					
198.02	GROUND SURFACE													
197.72	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-1-37₁ 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 kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp — w — wl				
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL FIELD VANE LAB SHEAR 20 40 60 80				WATER CONTENT (%) 10 20 30 40				
199.70	GROUND SURFACE														GR SA SI CL	
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

GEOG TWP GIBSON

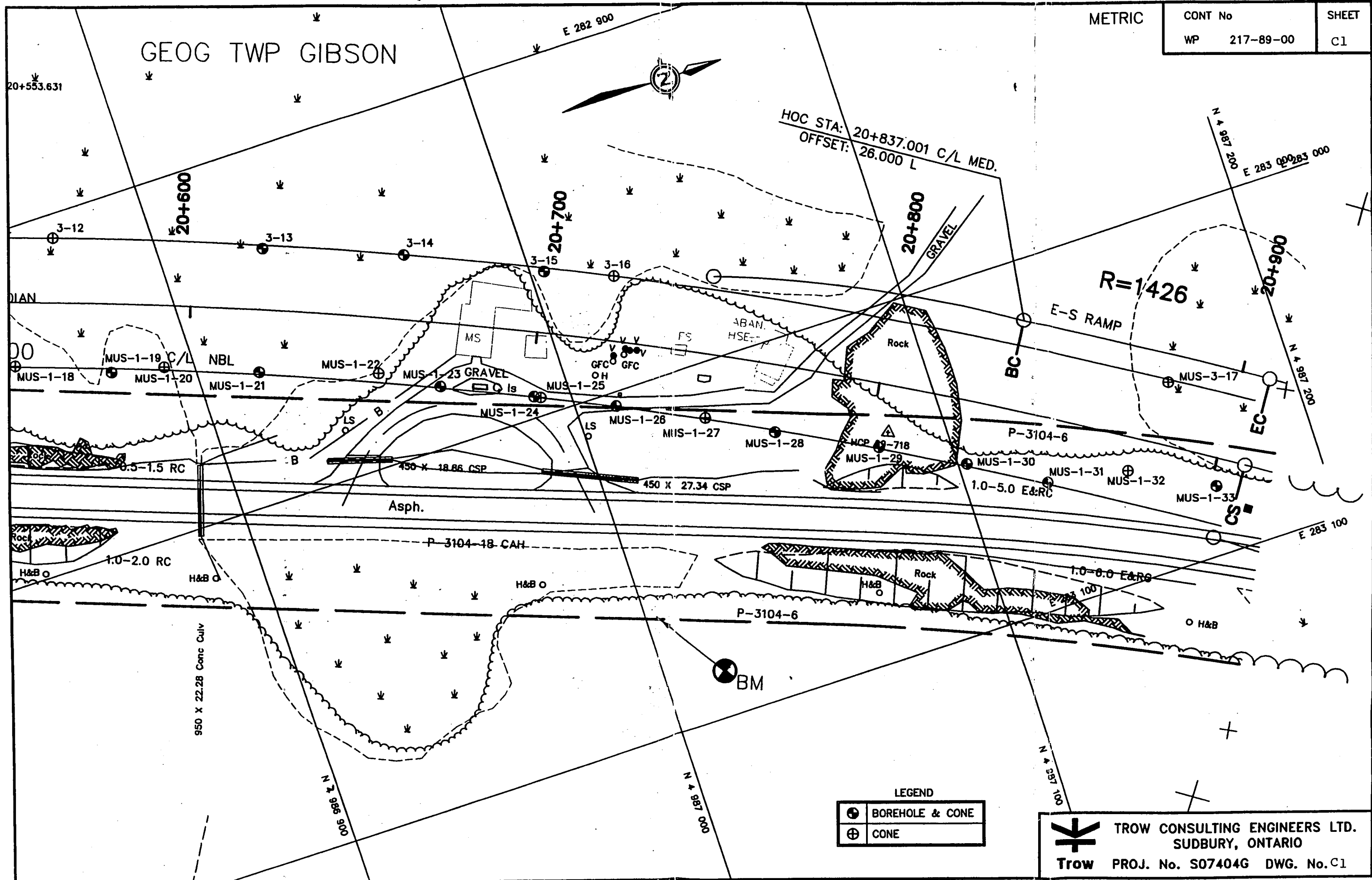
METRIC

CONT No

WP 217-89-00

SHEET

C1



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

R=1500

A=350

R=1500

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

21+200 Bush
LOT 14
CON 10

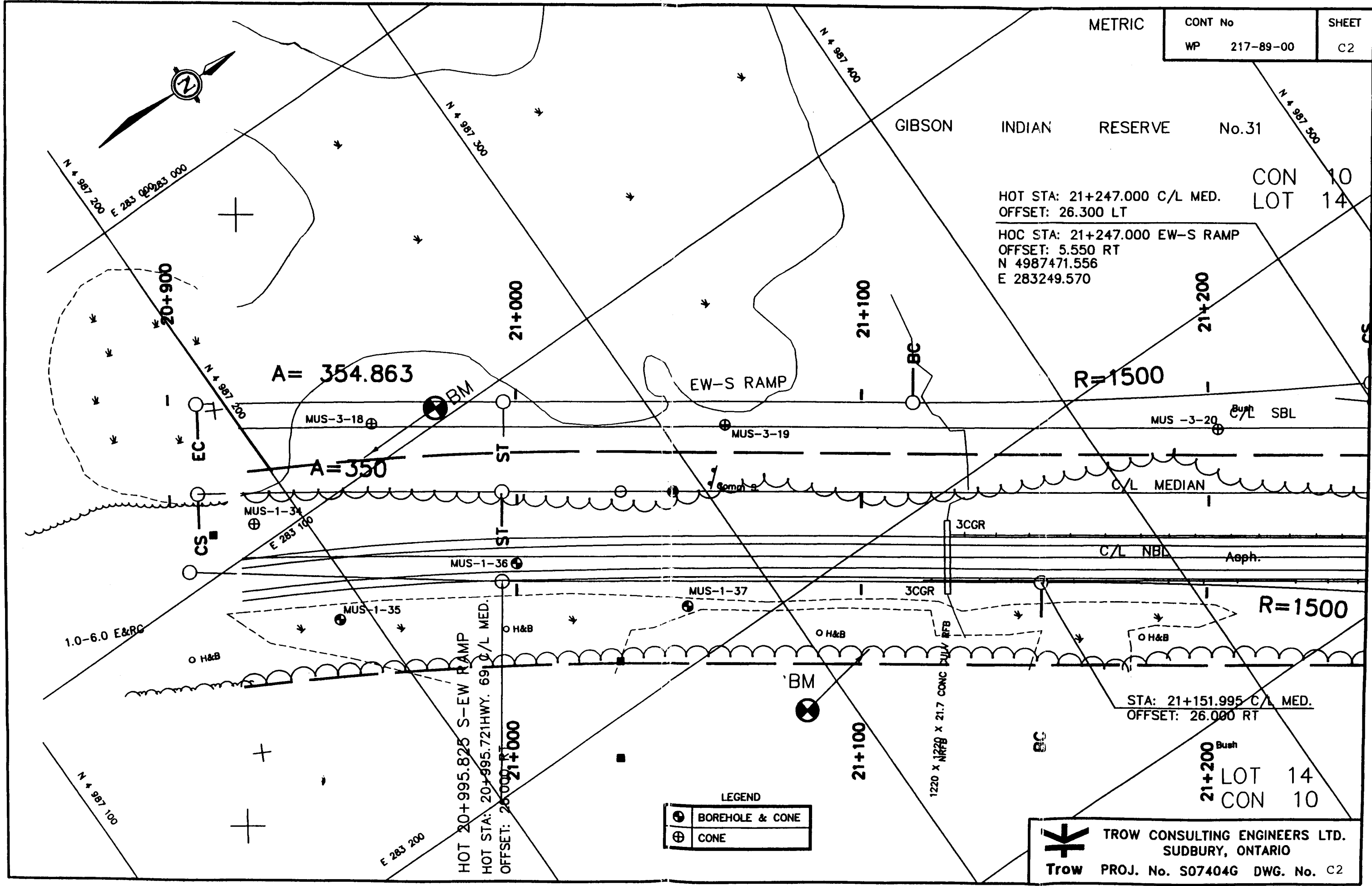
LEGEND

⊕	BOREHOLE & CONE
⊗	CONE



TROW CONSULTING ENGINEERS LTD.
SUDBURY, ONTARIO

Trow PROJ. No. S07404G DWG. No. 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 ³	REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		20	40	60	80					GR	SA	SI	CL
197.64 0.00	GROUND SURFACE																
195.64 2.00	ORGANICS, 250 mm over SILTY CLAY, trace of fine sand, grey/brown. (stiff to firm) 300 mm thick layer of compact SILT & FINE SAND encountered at 0.8 m depth. SILT & FINE SAND, occasional clay seams below 6.0 m depth, grey/brown to grey. (loose to very loose)		1	SS	17								17.00	0	56	44	0
			2	TW										0	7	45	44
			3	SS	4									0	58	42	0
			4	SS	8												
			5	SS	10									0	58	42	0
			6	SS	0												
190.04 7.60	SILT, SAND & GRAVEL TILL, grey, (loose to compact)		7	SS	10									22	29	49	0
189.78 7.86	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER																
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-3-18_{1 OF 1}

METRIC

W.P. 217-89-00 LOCATION Station 20+958.5, offset 0.9 m left of centreline. ORIGINATED BY I.D.
 DIST 52 HWY 69 BOREHOLE TYPE Dynamic Cone / 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 w _p — w — w _L	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60					
198.10	GROUND SURFACE														
197.60	Probable PEAT														
197.55	Probable SILTY CLAY														
197.55															
192.90	Probable SILT & FINE SAND														
192.90															
191.46	Probable SILT, SAND & GRAVEL TILL														
191.46															
6.64	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER														



RECORD OF BOREHOLE MUS-3-19₁ OF 1

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) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w		
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	40										
1.74	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														



RECORD OF BOREHOLE MUS-3-20₁ OF 1

METRIC

W.P. 217-89-00

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

ORIGINATED BY J.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 cm	20	40						60	80	W _p
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

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 51mm O.D. SPT BARREL SAMPLER TO PENETRATE 30mm 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 30mm, 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 1" SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 30mm 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/30mm)	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

MECHANICAL PROPERTIES OF SOIL

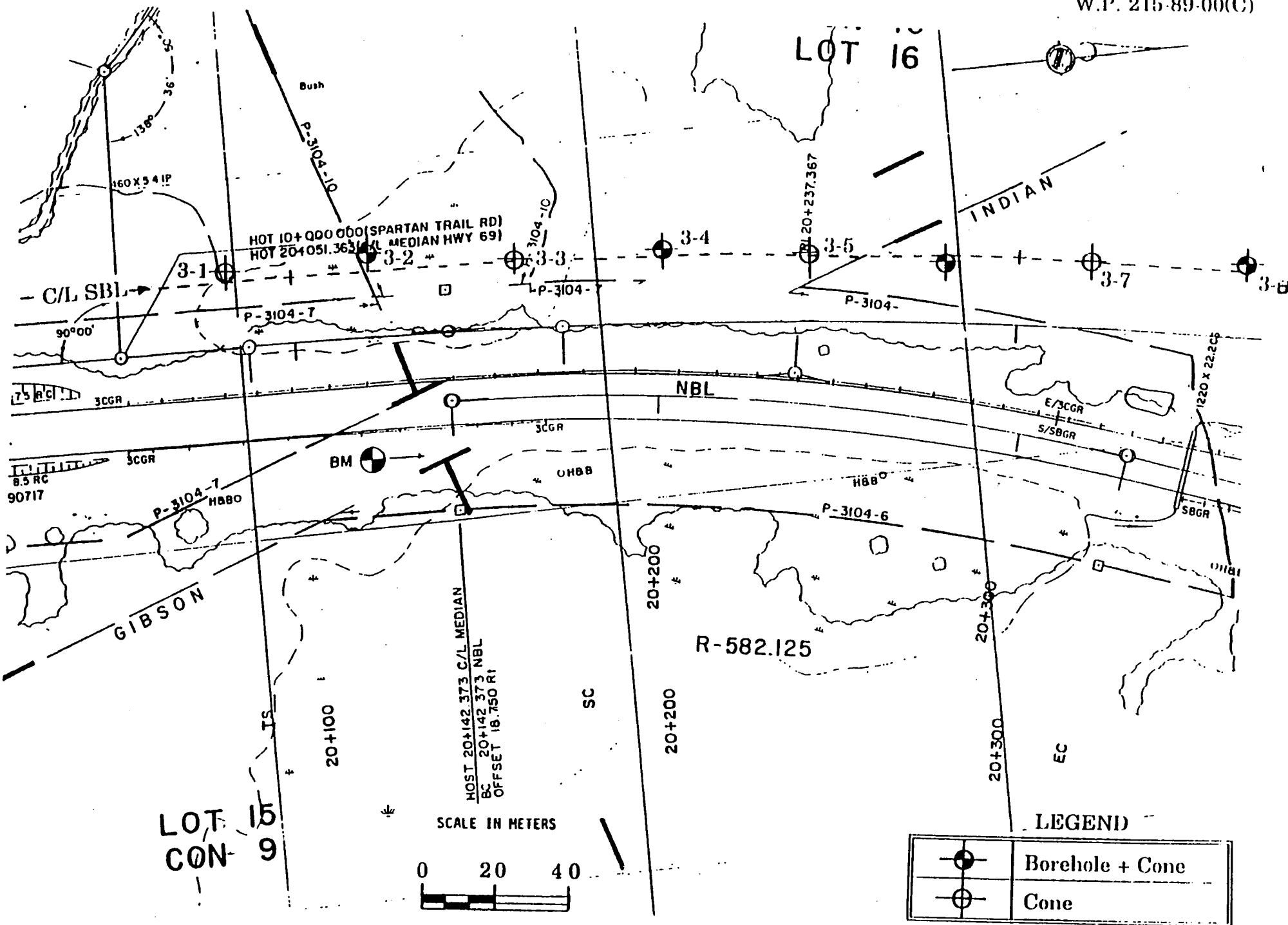
m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c		COMPRESSION INDEX
C_s		SWELLING INDEX
C_α		RATE OF SECONDARY CONSOLIDATION
C_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v		TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{C_u}{\tau_r}$

STRESS AND STRAIN

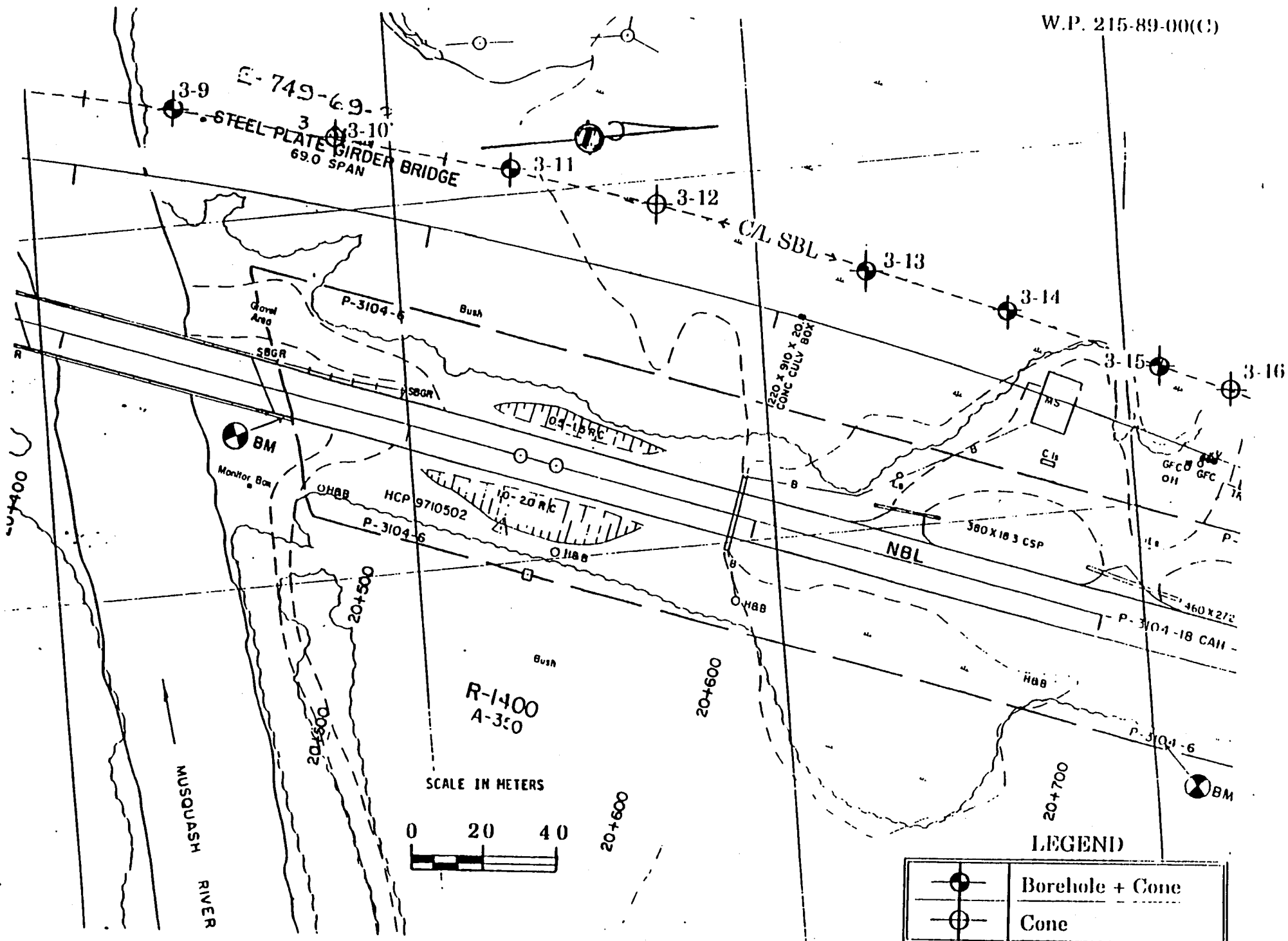
u_w	kPa	PORE WATER PRESSURE
r_u		PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ		COEFFICIENT OF FRICTION

PHYSICAL PROPERTIES OF SOIL

ρ_s	g/cm^3	DENSITY OF SOLID PARTICLES	e	%	VOID RATIO	e_{min}	%	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	g/cm^3	DENSITY OF WATER	w	%	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	g/cm^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	g/cm^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	g/cm^3	DENSITY OF SATURATED SOIL			LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	g/cm^3	DENSITY OF SUBMERGED SOIL				i	kN/m^3	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						



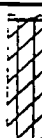

Drawing No: 2158900(C) A



RECORD OF BOREHOLE No 3-1

1 OF 1 METRIC

W.P. 115-89-00(C) LOCATION Site 20-083, 1 - 11 C/A S.B. ORIGINATED BY LE
 DIST HWY 55 BOREHOLE TYPE Cone Test COMPILED BY LB
 DATUM Geodetic DATE April 14 1992 CHECKED BY DD

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCAL	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH κP_c • UNCONFINED - FIELD VANE • QUICK TRIAXIAL • LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
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

LOCATION Sta. 20+72.5 - 11.6' S.B.

ORIGINATED BY JD

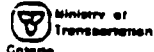
BOREHOLE TYPE Hollow Stem / Cone Test

COMPILED BY

DATE _____ April 2 '59?

CHECKED BY 33

[illegible]



OF : METRIC

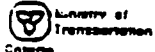
W.P. 215-89-00(C) LOCATION Sta. 21-160 C/A S.B.L. ORIGINATED BY DP
DIST. 11 HWY 69 BOREHOLE TYPE Cone Test COMPILED BY JE
DATUM Geodetic DATE April 12 1992 CHECKED BY DD

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE $\frac{q_{dc}}{\text{MPa}}$	PLASTIC LIMIT w_p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w_L	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES					
196.7	Ground Surface						20 40 60 80 100	20 40 60				
196.7	Probable Peat											
194.0	Probable Silty Clay to Clayey Silt											
188.0	Probable Sandy Silt to Sand											
181.5	End of Cone Test Refusal - Probable Bedrock						120 / 25cm					



W.P. 115-89-00(C) LOCATION Sta. 20-240 C/L S.B.L. ORIGINATED BY DR
DIST 11 HWY 69 BOREHOLE TYPE Cone Test COMPILED BY JS
DATUM Geodetic DATE April - 1992 CHECKED BY CC

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER TYPE 'N' VALUES			20 40 60 80 100	20 40 60 80 100					
196.51	Ground Surface											
196.01	Probable Peat				196							
					194							
	Probable Silty Clay to Clayey Silt				192							
					190							
					188							
	Probable Sandy Silt to Fine Sand				186							
					184							
181.5	End of Cone Test Refusal - Probable Bedrock				182							



1 OF 1

METRIC

LOCATION Sic 33-380 2 - 4. 00 S.B.

ORIGINATED BY JE

BOREHOLE TYPE Shallow Stem - Core Test

COMPILED BY E

DATE April 10 '55

CHECKED BY 20

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			VALUES	20						40	60
196.5	Ground Surface														
195.6	Feat/Organic Clayey Silt Brownish Grey to Black. Soft														
0.9	Silty Clay to Clayey Silt. Some Sand Grey		1	SS	1										
			2	SS	2										
			3	SS	1										
			4	SS	1	C**									
			5	SS											
			6	SS	1										
188.6	Sand and Silt to Silty Fine Sand Grey		7	SS	1	C**									
7.9			8	SS	1										
			9	SS	1	C									
			10	SS	1	C**									
			11	SS	1										
			12	SS	1										
181.6	Loose to Compact Dense to Very Dense														
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														

: OF :

METRIC

LOCATION 4 585 538.4 5 322 847.2

ORIGINATED BY JE

BOREHOLE TYPE Core Test:

COMPILED BY

DATE April 10, 1992

CHECKED BY 33

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE P _{0.1"}	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES		SHEAR STRENGTH kPc • UNCONFINED - FIELD VANE • QUICK TRIAXIAL - LAB VANE 2C 40 60 80 100			
196.21	Ground Surface											
3.01	Probable Peat											
	Probable Silty Clay to Clayey Silt											
	Probable Sand and Silt to Silty Fine Sand											
185.21	End of Cone Test Refusal - Possible Boulder or Bedrock						120 / 18cm					

RECORD OF BOREHOLE No 3-8

1 OF 1 METRIC

W.P. 215-89-0010 LOCATION Ste. 20-352, C.M. 2 - 11 C/L S.B. ORIGINATED BY JS
 DIST HWY 69 BOREHOLE TYPE Hollow Stem Cone Test COMPILED BY JS
 DATUM Geodetic DATE April 9, 1992 CHECKED BY JD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W _n	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
196.6	ce													
196.6	Silty Clay to Clayey Silt		1	SS	1									
	Trace of Sand		2	SS	1									
	Grey		3	SS	1									
	Soft to Firm		4	SS	10**									
			5	TW	1									
			6	SS	1									
189.6														
189.6	Trace of Clay		7	SS	10**									
			8	SS	10**									
	Sandy Silt to		9	SS	1									
	Fine Sand, Some Silt		10	SS	10**									
	Grey to Brownish Grey													
			11	SS	10**									
	Loose to Compact		12	SS	13									
	Dense to Very Dense		13	SS	17									
179.6														
178.8	Probable Bedrock ***													
17.8	End of Borehole													
	* W.L. on April 10, 1992 ** Split spoon sank under weight of hammer one rods *** Auger and Cone likely slipping off rock surface													

: OF :

METRIC

LOCATION

Sic 22-474 C/L S.E.L.

ORIGINATED BY JE

DIST 4WY 65

BOREHOLE TYPE Wallow Stem / Cone Test:

COMPILED BY -E

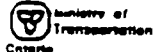
DATUM Geodetic

DATE _____

2021 6 '992

CHECKED BY... 33

[illegible]



: OF : METRIC

W.P. 115-89-0010 LOCATION Sta. 10-470 C.M. S.B.L. ORIGINATED BY JS
DIST 11 HWY 69 BOREHOLE TYPE Cone Test COMPILED BY JS
DATUM Geodetic DATE April 7 1992 CHECKED BY JS

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	SIRAT PLOT	NUMBER	TYPE			'N' VALUES					
196.7	Ground Surface											
0.0	Probable Feet 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					

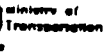
RECORD OF BOREHOLE No 3-11 : OF 1 METRIC

W.P. 215-89-00(C) LOCATION Sta. 20+520.00 - 100' S.B.L. ORIGINATED BY JE
 DIST HWY 55 BOREHOLE TYPE Open Stem / Cone Test COMPILED BY JE
 DATUM Geodetic DATE April 7, 1992 CHECKED BY JD

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIMIT MOISTURE CONTENT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE			20 40 60 80 100		W _p	W		
196.6	Ground Surface									7	
196.0		1	SS		196						
	Silty Clay to Clayey Silt	2	SS		194						
	Brownish Grey to Grey	3	SS		192						
	Firm to Stiff	4	SS		190						
		5	SS								
		6	SS								
189.4	Sandy Silty	7	SS								
188.5	Silty Fine Sand	8	SS								
	Grey, Compact to Dense										
8.1	End of Borehole Refusal - Probable Bedrock										

W.L. on April 14, 1992

75/20cm

[illegible]

1 OF 1

METRIC

W.P. 55-89-00(C)

LOCATION Sta. 30-620; C" - R. 31, S. 31, E. 31;

ORIGINATED BY JB/DR

DIST _____ HWY 55

BOREHOLE TYPE Hollow Stem / Cone Test

COMPILED BY .3

DATUM Geodetic

DATE April 8 '992

CHECKED BY 23

3 5 Number refer to 20

RECORD OF BOREHOLE No 3-15 OF 1 METRIC

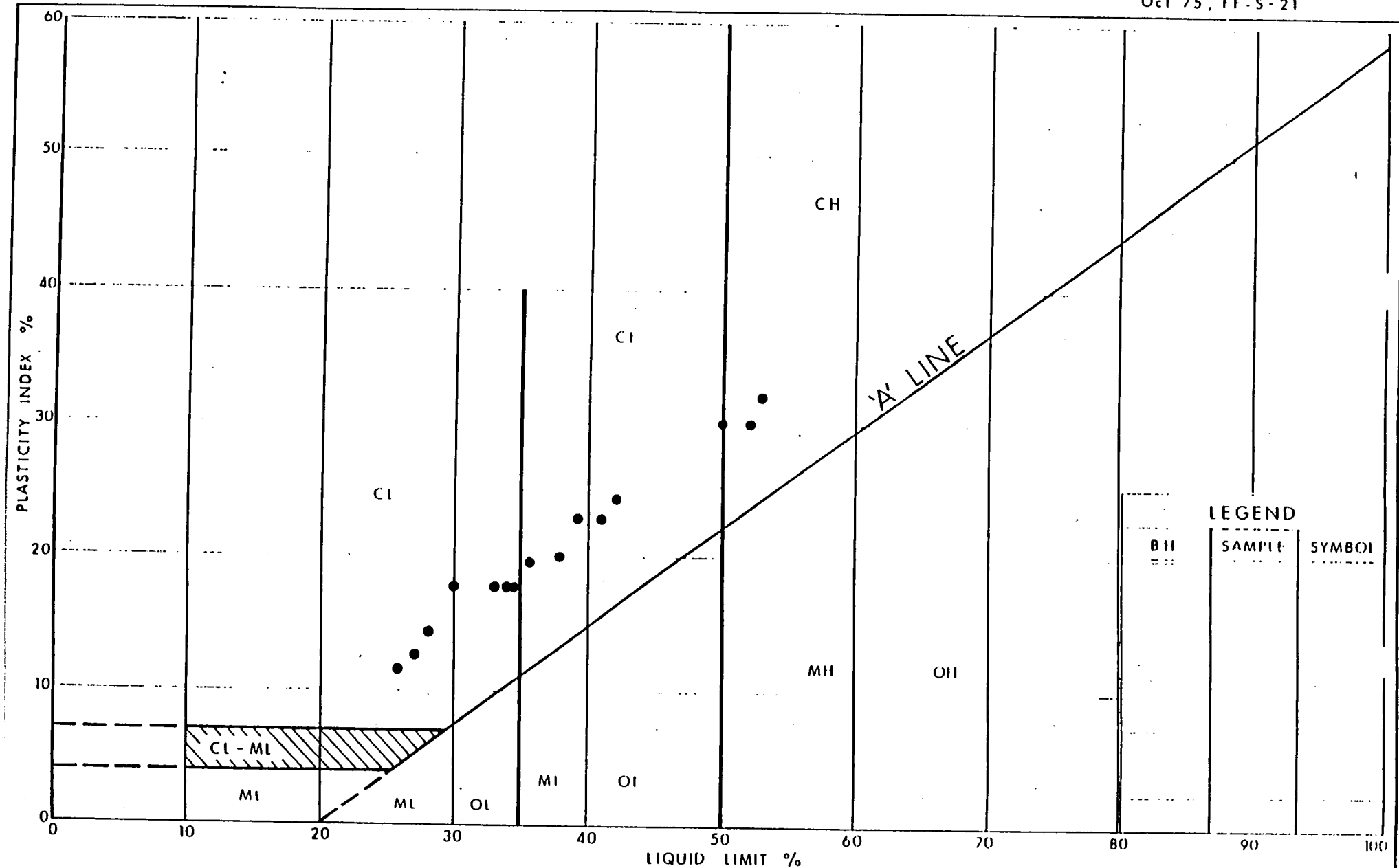
W.P. 115-89-00(C) LOCATION Sta. 20-700.00 - R. 0/L S.B.L. ORIGINATED BY DR
 DIST HWY 69 BOREHOLE TYPE Hollow Stem / Cone Test COMPILED BY JE
 DATUM Geodetic DATE April 10, 1992 CHECKED BY JD

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE			20 40 60 80 100	CO					
196.7	Ground Surface											
C.01 195.91	Peat Black, Soft											
C.81	Silty Clay to Clayey Silt	1	SS	4	196							
	Brownish Grey	2	SS	4	194							
	Stiff to Firm	3	SS	2								
191.5	--- cone lower	4	SS	22	192							
5.21	End of Borehole Refusal - Probable Bedrock • W.L. on April 14, 1992											



W.P. 115-85-CO(C) LOCATION Sta 20-720, 0.7 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 JB

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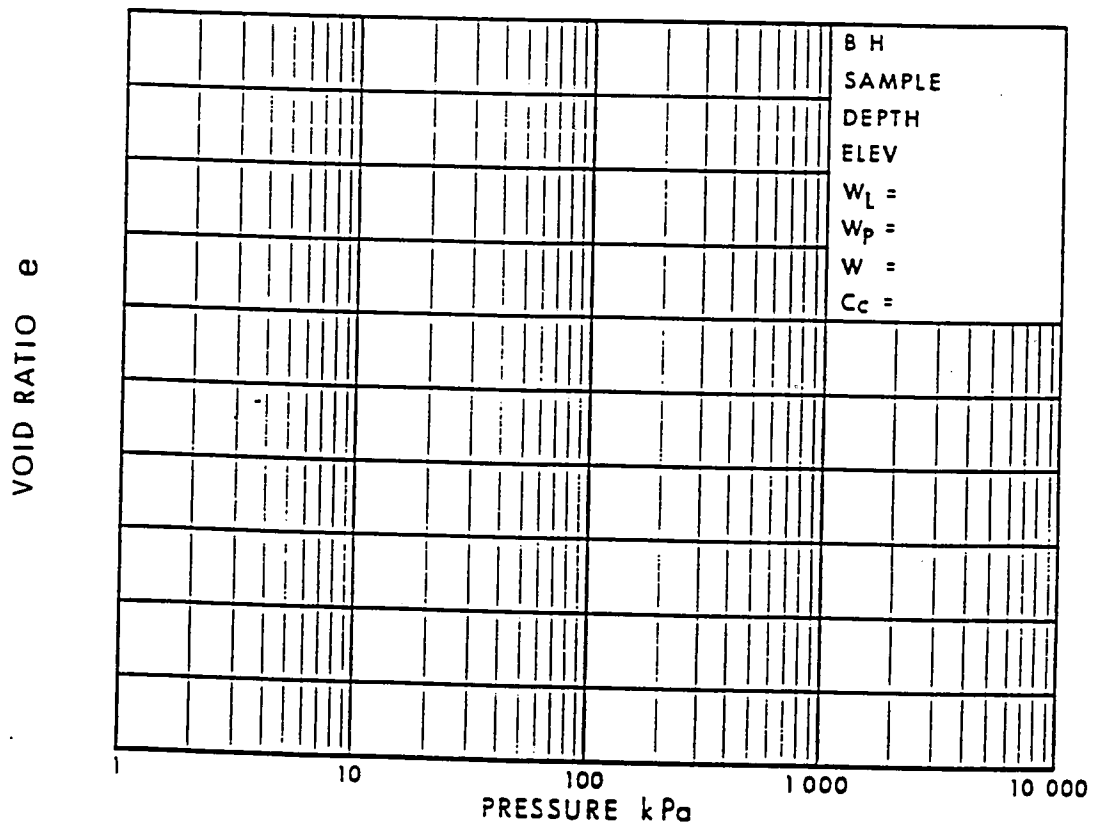
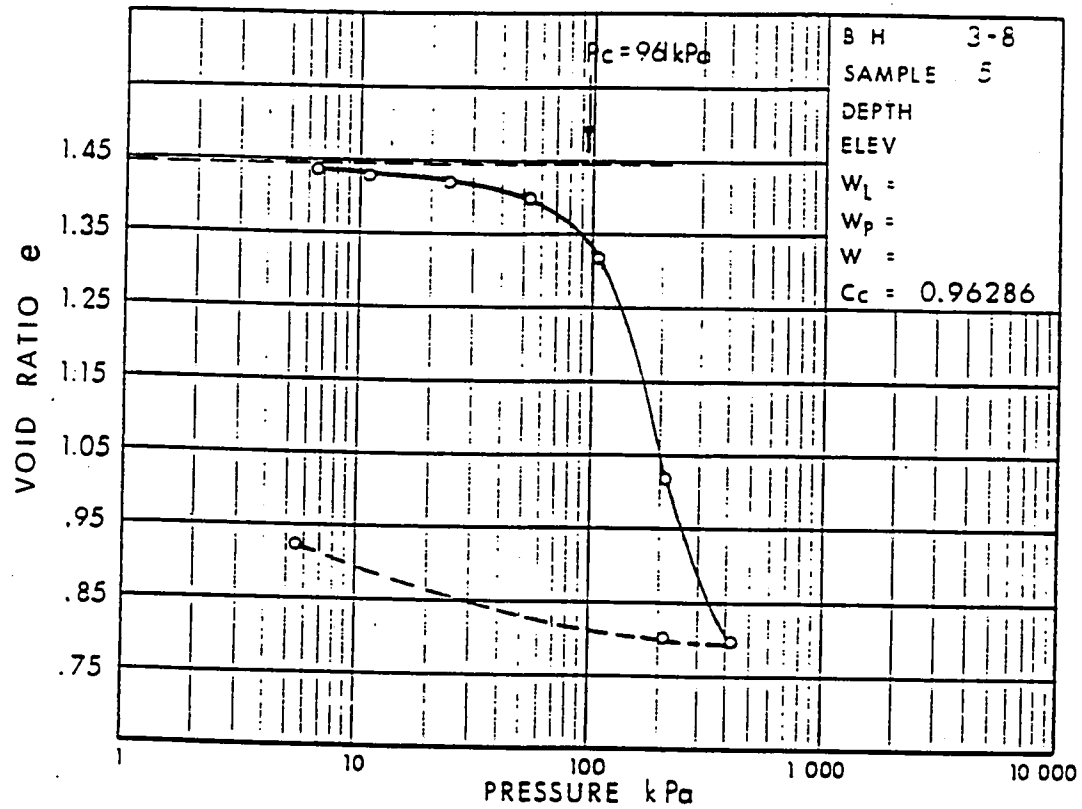
Ministry of
Transportation

PLASTICITY CHART SILTY CLAY TO CLAYEY SILT

FIG No 1

W P 215-89-00 (C)

VOID RATIO - PRESSURE CURVES





GRAIN SIZE DISTRIBUTION
SANDY SILT TO MEDIUM SAND

W P 215-89-00 (C)

Appendix E: Detailed Settlement Calculation Results

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

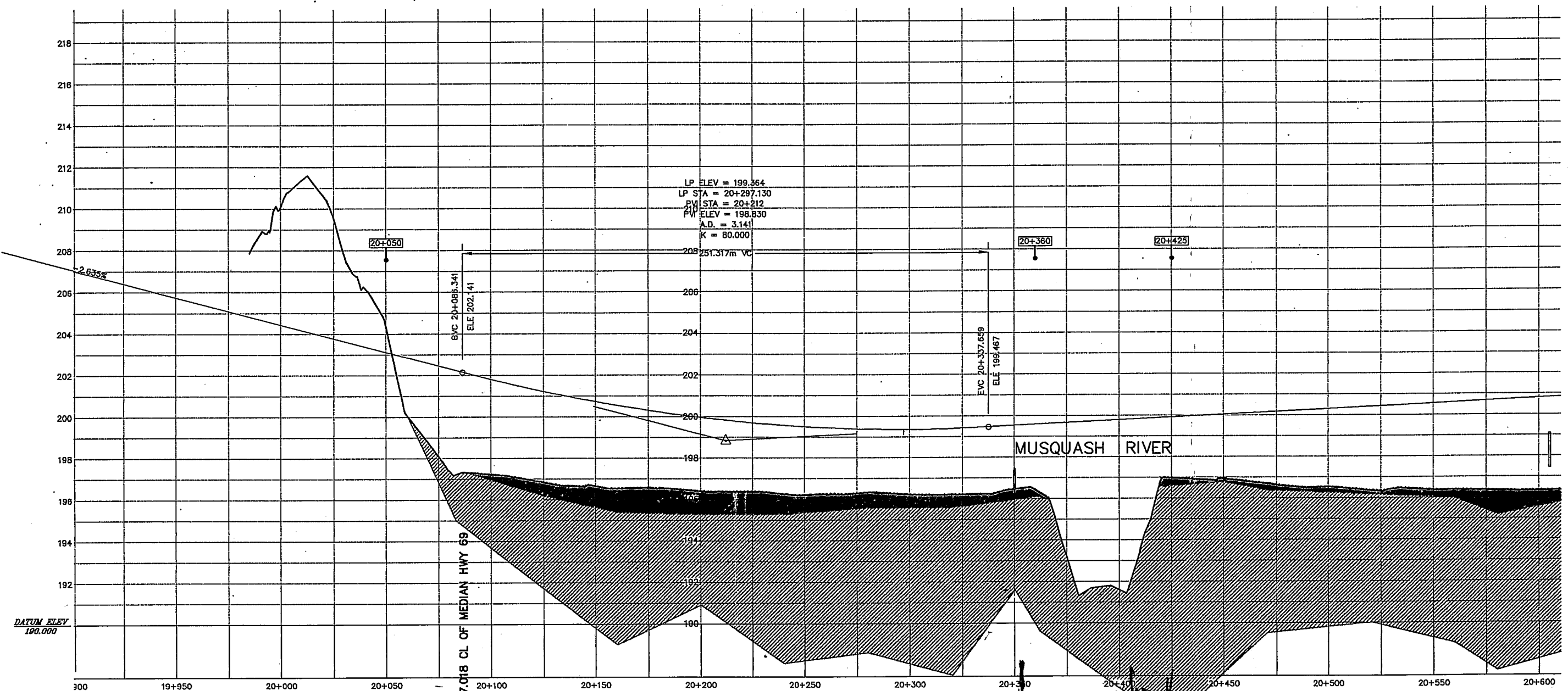
Station	Fill	Clay	Settlement	% Consolidation	Remaining	Req'd
	Thickness	Thickness			Settlement	Additional Fill
	(m)	(m)	(mm)	after 2 months	(mm)	(m)
20+273	0.1	7.2	0	0.22	0	-
20+298	3.3	9.0	120	0.16	101	3.7
20+324	2.9	5.0	85	0.31	59	1.8
20+350	3.3	7.8	115	0.19	93	2.8
20+376	3.8	7.4	102	0.22	80	1.7
20+451	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 remove peat
 - use wide drain or sand drain
 Encase 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.

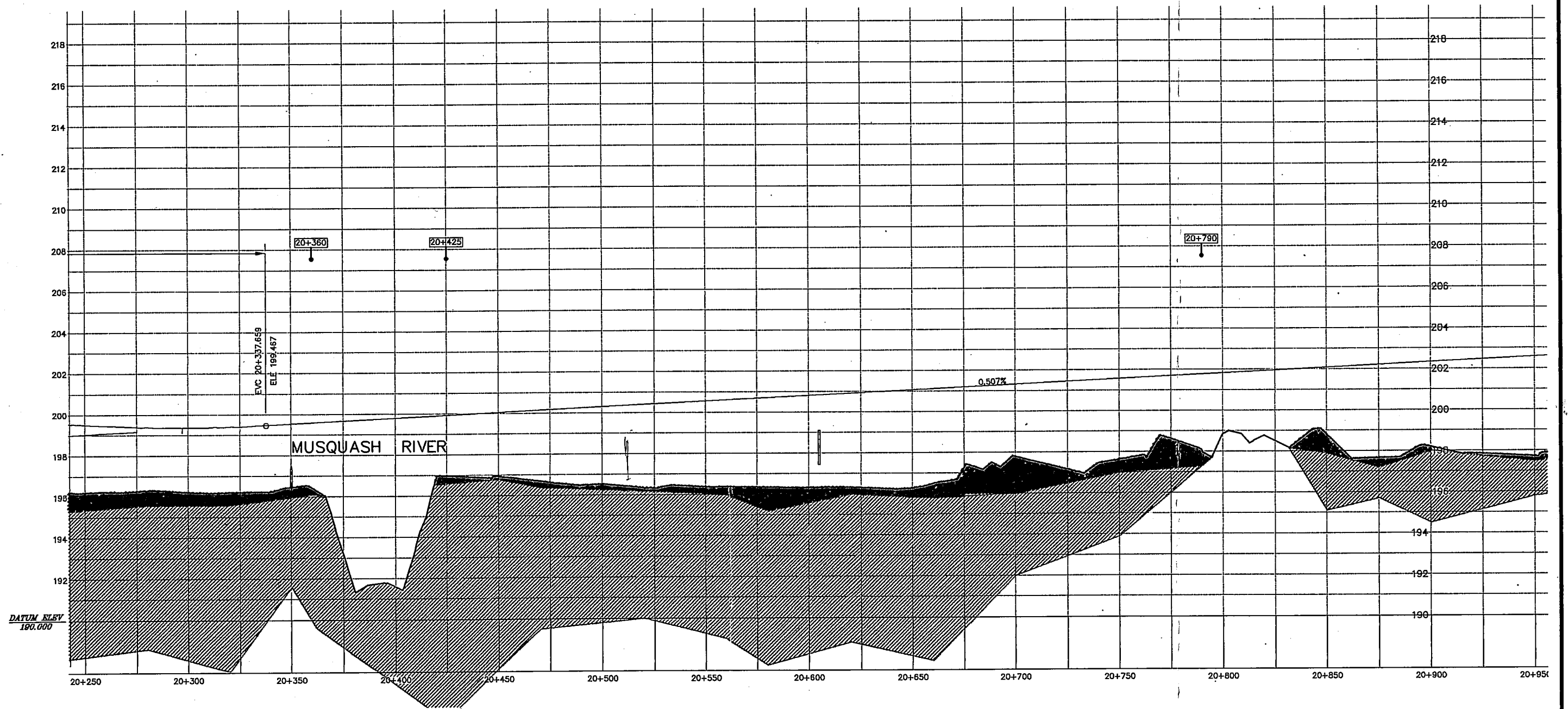
BRAMPTON, ONTARIO

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

PROJECT NO.: BRGE0011546C
 SCALE: AS NOTED
 DRAWN BY: SDH
 CHECKED BY: SDH
 DATE: JUNE 1998
 DRAWING NO.: 1

XX

ONTARIO



LEGEND:



CLAYEY SOILS



SURFICIAL ORGANIC SOILS

20+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.



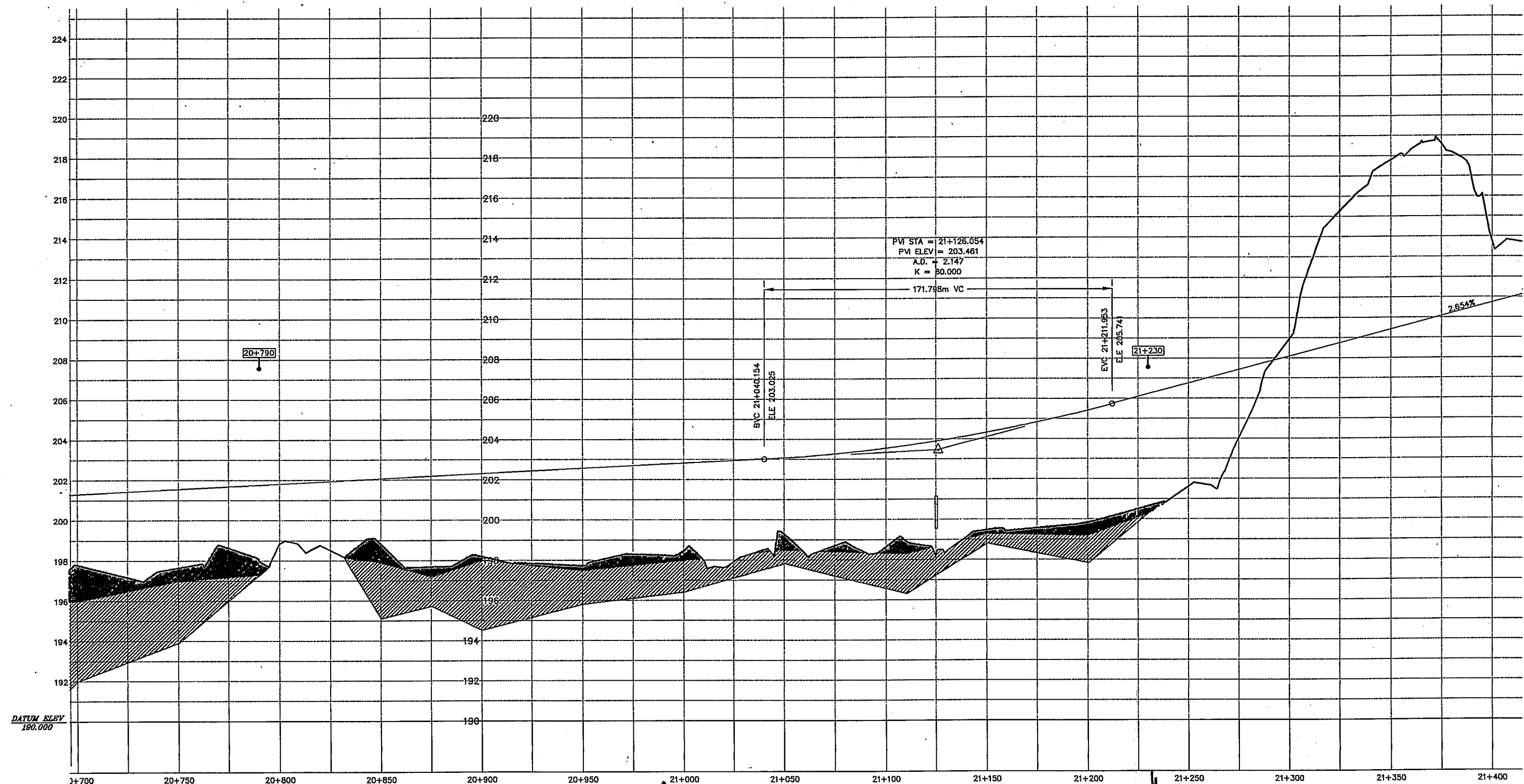
TROW CONSULTING ENGINEERS LTD.
BRAMPTON, ONTARIO

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

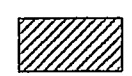


PROJECT NO.: BRGE0011546C
SCALE: AS NOTED
DRAWN BY: SDH
CHECKED BY: SDH
DATE: JUNE 1998
DRAWING NO.: 2

XX

ONTARIO




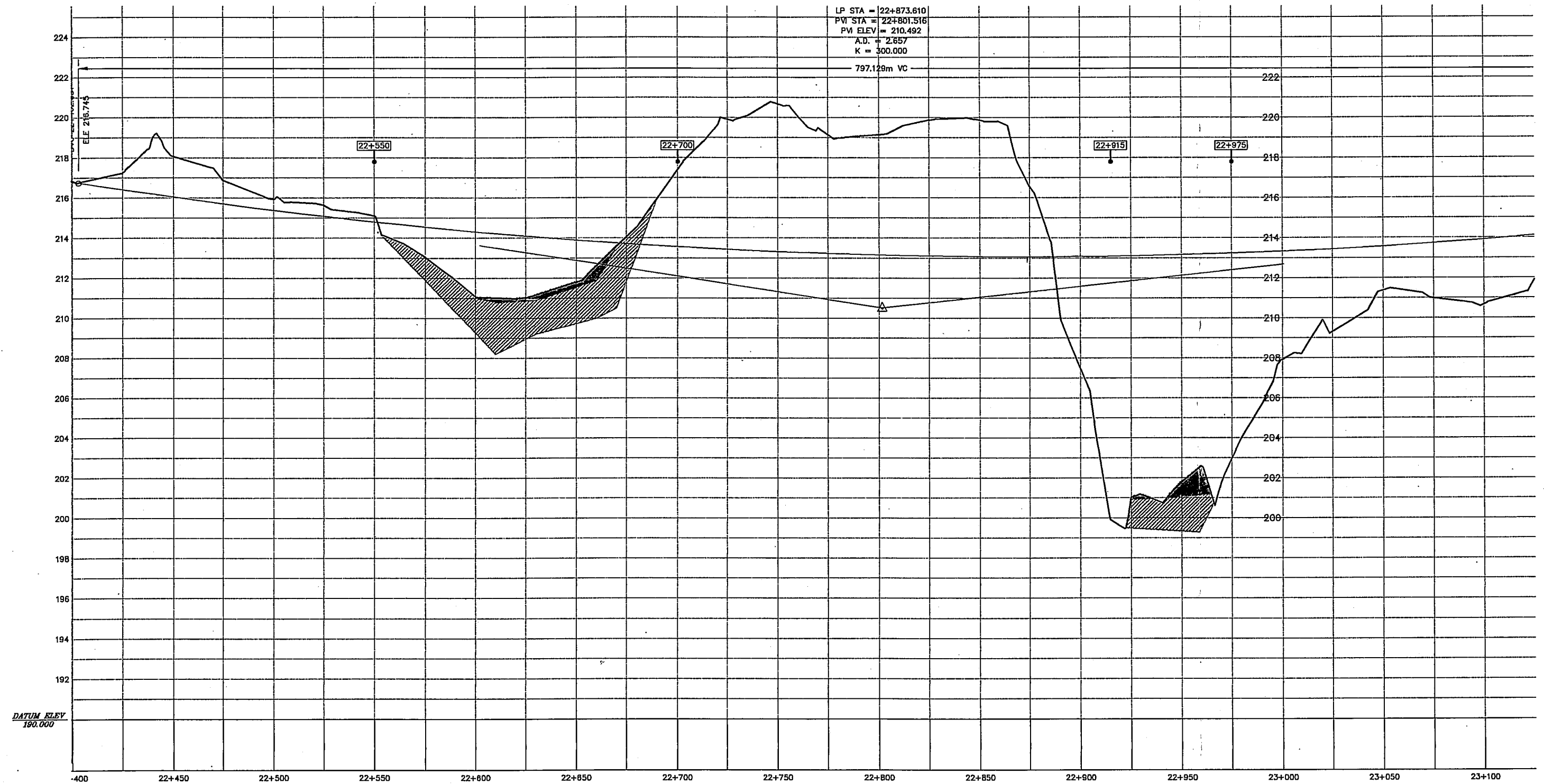
LEGEND:

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

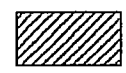

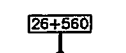
NOTES:

- 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		PROJECT NO.: BR-11546-A	
		SCALE: AS NOTED	
HIGHWAY #69 SOUTHBOUND LANE STATION 20+790 to 21+230		DRAWN BY: SDH	
		CHECKED BY: SDH	
		DATE: JUNE 1998	
ONTARIO		DRAWING NO.: 3	



LEGEND:

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

*excavated
entirely*

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*



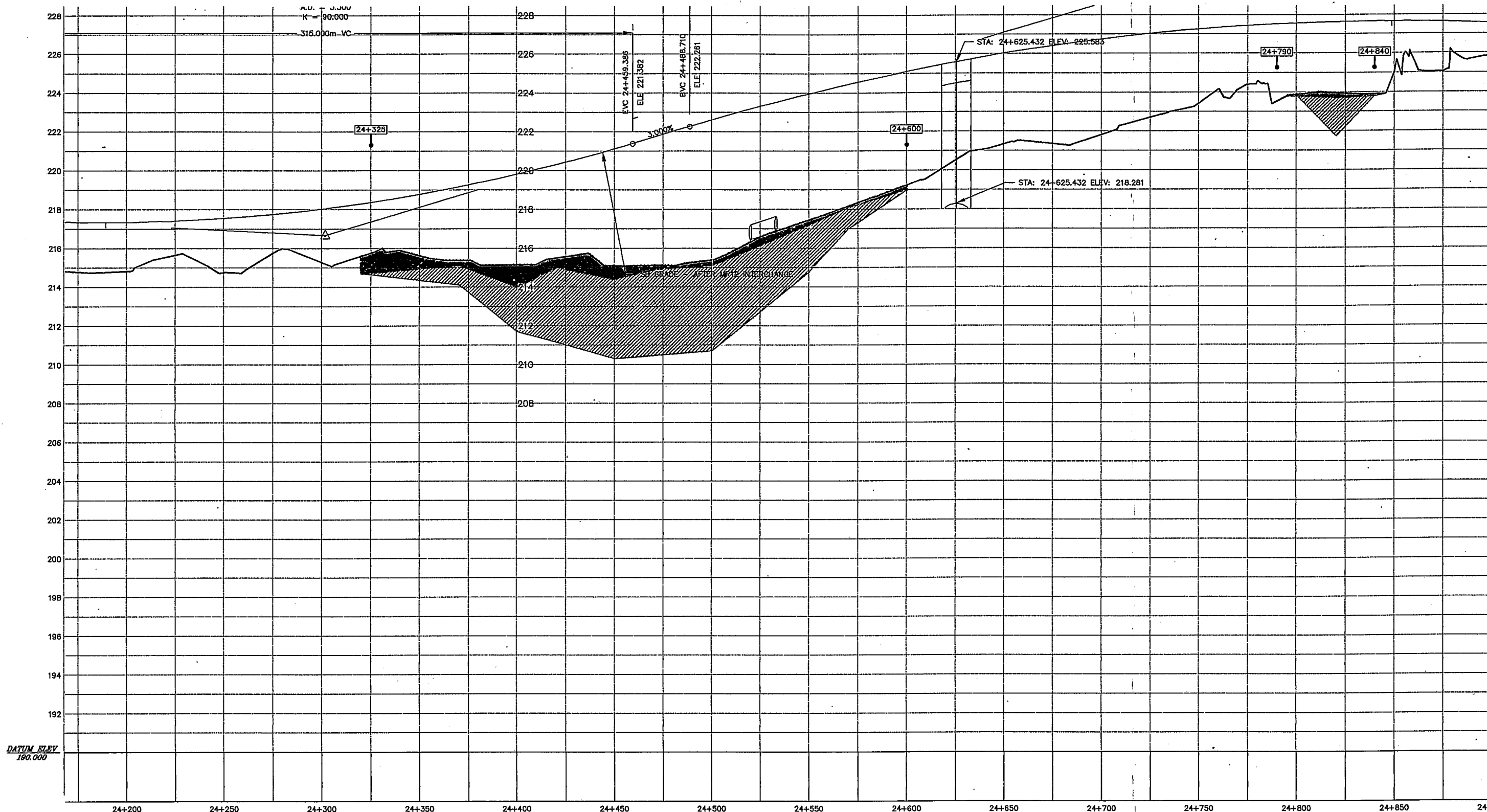
TROW CONSULTING ENGINEERS LTD.
BRAMPTON, ONTARIO

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

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

XX

ONTARIO




DATUM ELEV
190.000

LEGEND:

 CLAYEY SOILS

 SURFICIAL ORGANIC SOILS

 DELIMITATION OF AREA OF INTEREST

NOTES: FOR DETAILED BOREHOLE LOGS REFER TO:

*full excavation
with care*

- 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.



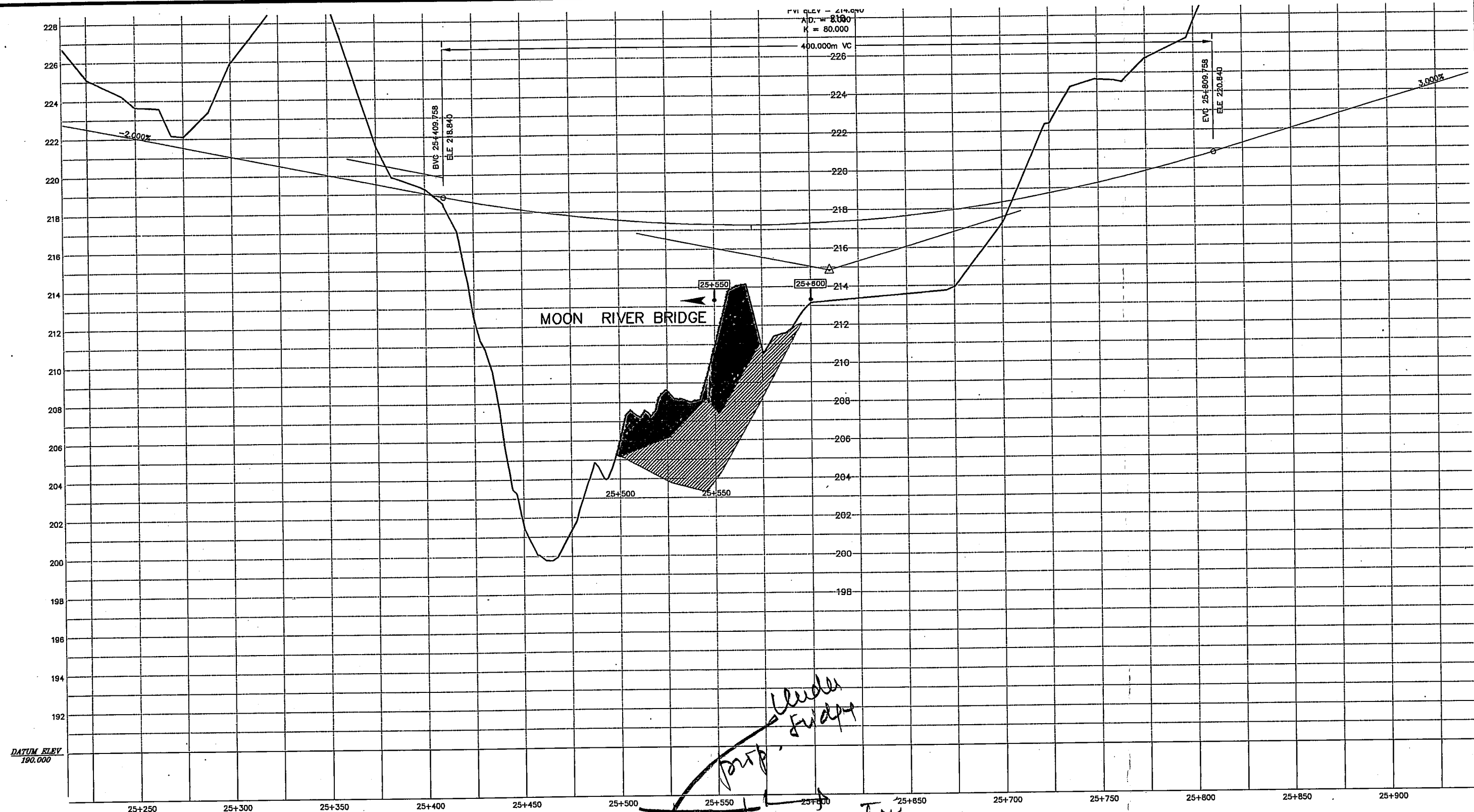
TROW CONSULTING ENGINEERS LTD.
BRAMPTON, ONTARIO

**HIGHWAY #69
SOUTHBOUND LANE
STATION 24+325 to 24+800
24+790 to 24+840**

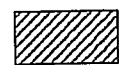
PROJECT NO.: BRGE0011546C
SCALE: AS NOTED
DRAWN BY: SDH
CHECKED BY: SDH
DATE: JUNE 1998
DRAWING NO.: 6

XX

ONTARIO



LEGEND:



CLAYEY SOILS



SURFICIAL ORGANIC SOILS

25+580

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.



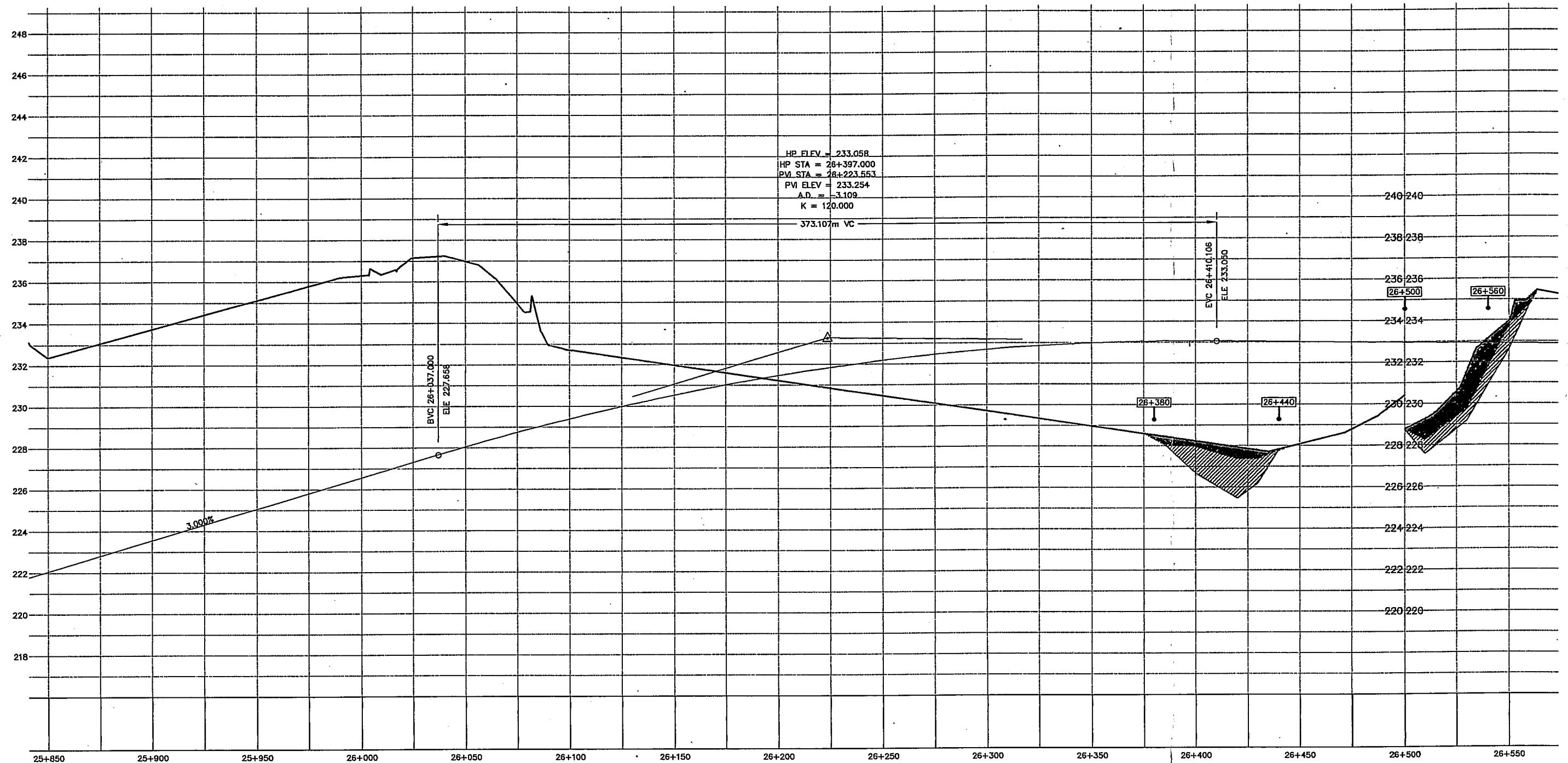
TROW CONSULTING ENGINEERS LTD.
BRAMPTON, ONTARIO

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

PROJECT NO.:	BRGE0011546C
SCALE:	AS NOTED
DRAWN BY:	SDH
CHECKED BY:	SDH
DATE:	JUNE 1998
DRAWING NO.:	7

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.



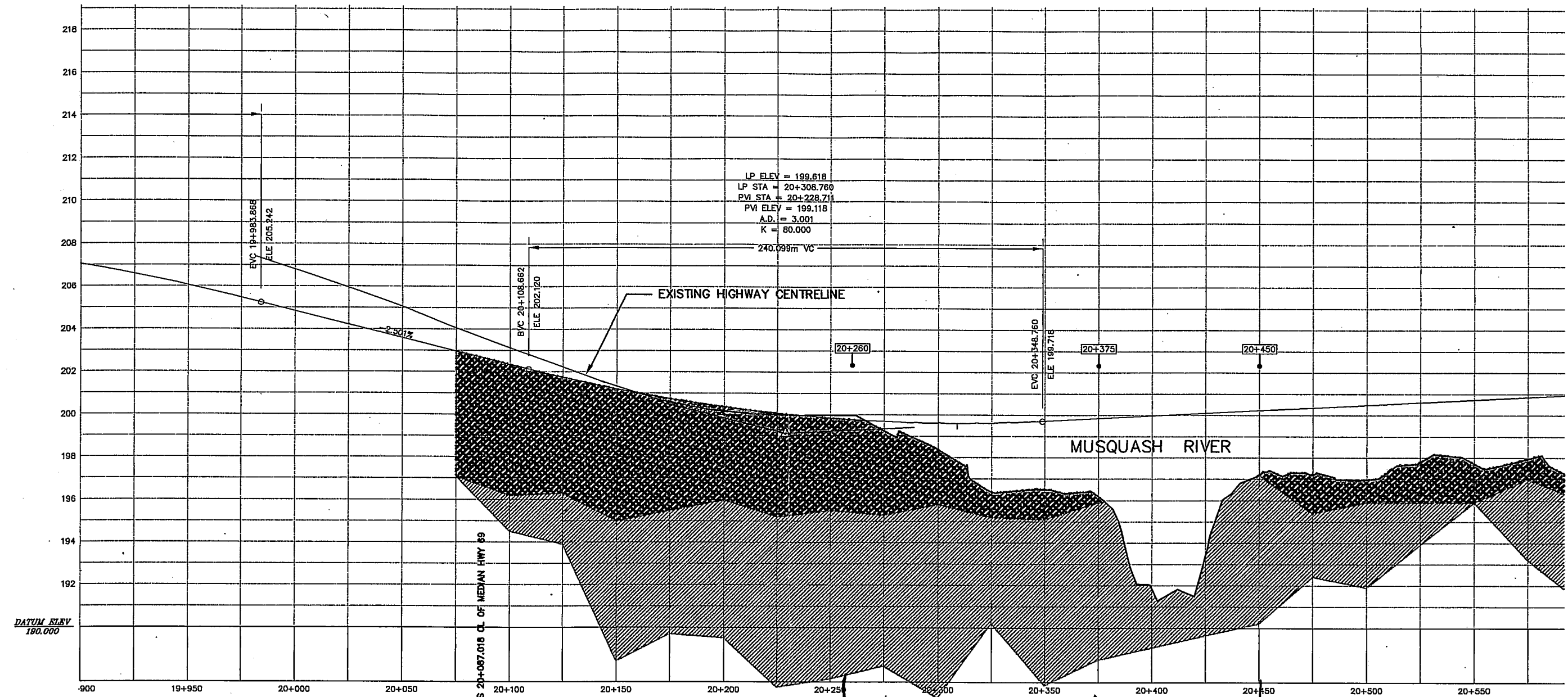
TROW CONSULTING ENGINEERS LTD.
BRAMPTON, ONTARIO

HIGHWAY #69
SOUTHBOUND LANE
STATION 26+380 to 26+440
26+500 to 26+560

PROJECT NO.: BRGE0011546C
SCALE: AS NOTED
DRAWN BY: SDH
CHECKED BY: SDH
DATE: JUNE 1998
DRAWING NO.: 8

XX

ONTARIO



LEGEND:



CLAYEY SOILS



SURFICIAL ORGANIC SOILS

26+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. 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.



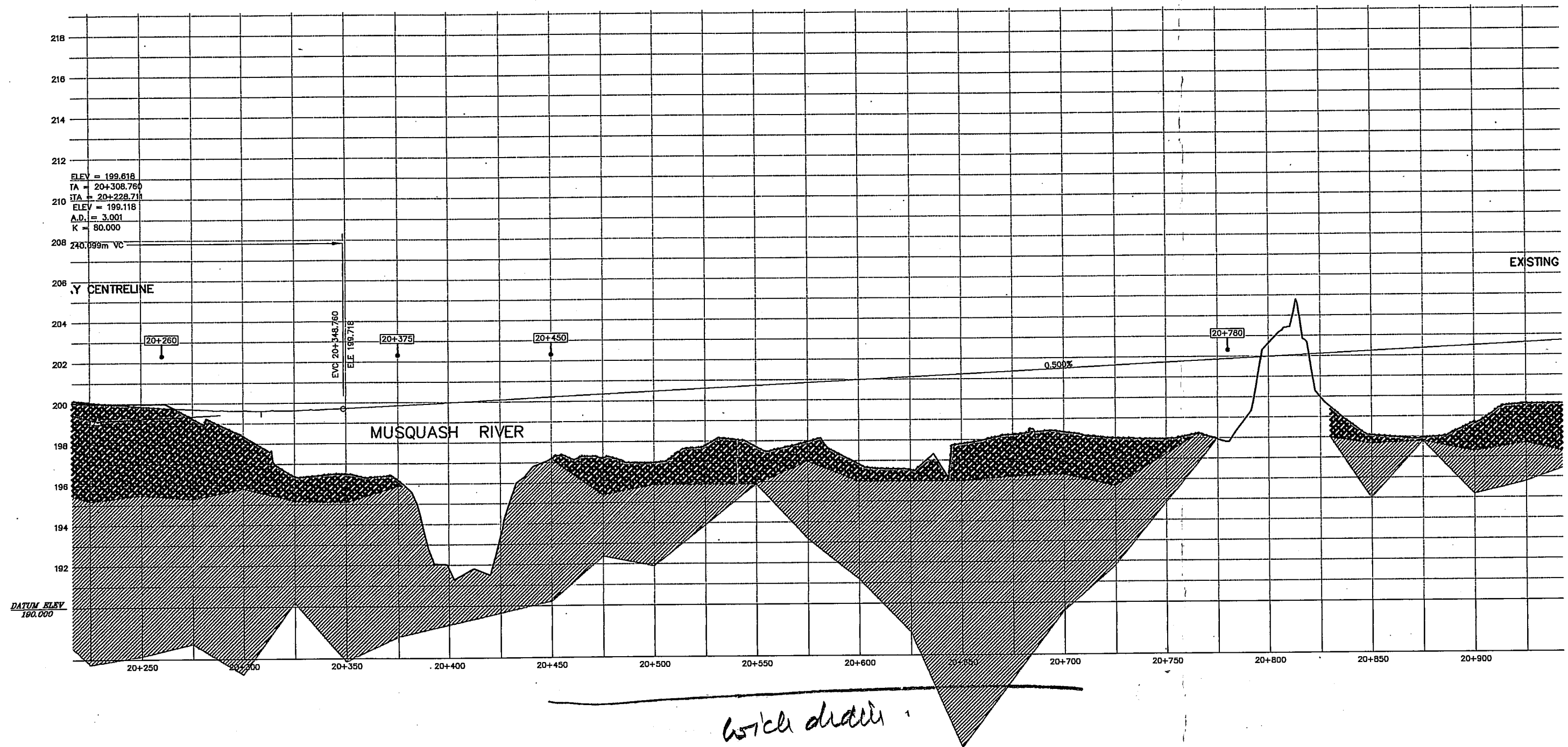
TROW CONSULTING ENGINEERS LTD.
BRAMPTON, ONTARIO

**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
-  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.



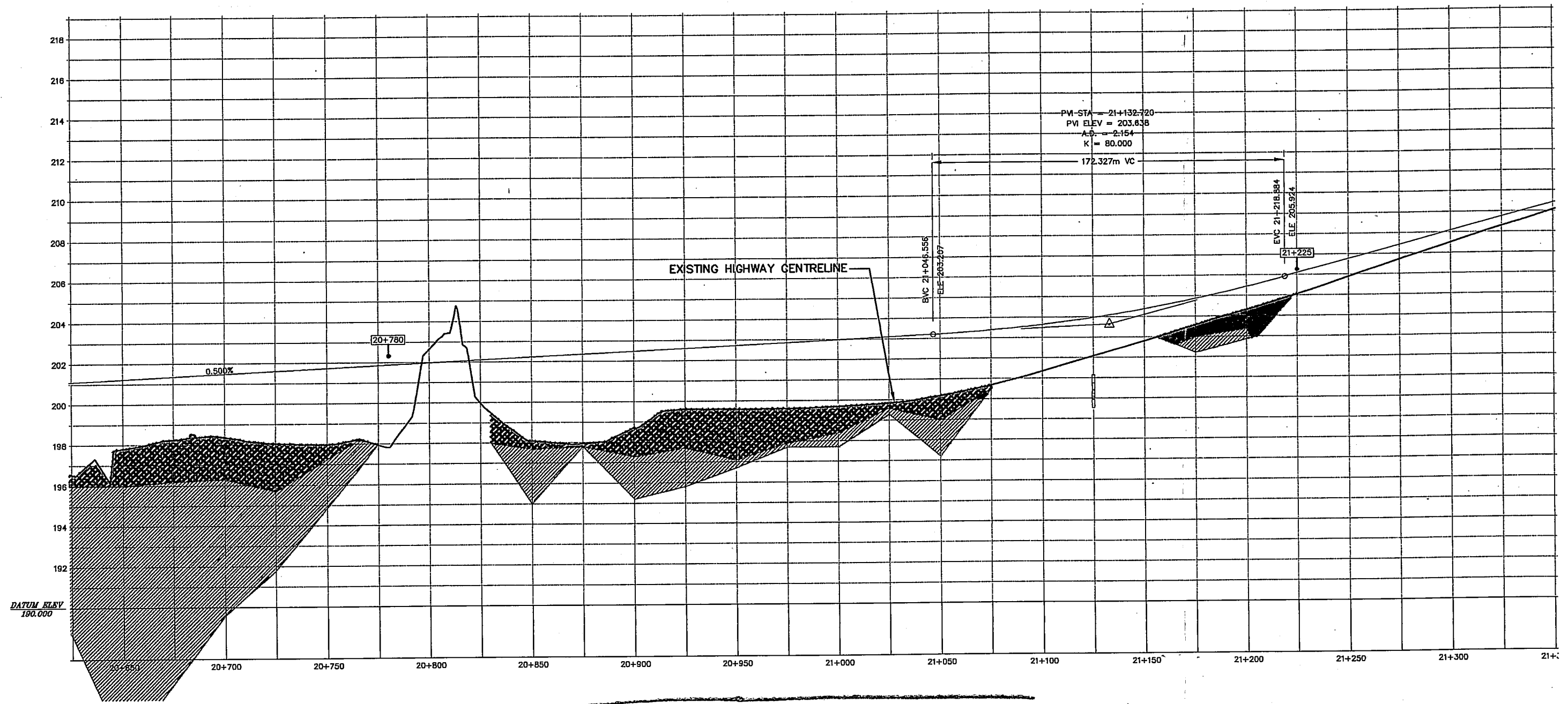
TROW CONSULTING ENGINEERS LTD.
 BRAMPTON, ONTARIO

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

PROJECT NO.: BRGE0011546C
 SCALE: AS NOTED
 DRAWN BY: SDH
 CHECKED BY: SDH
 DATE: JUNE 1998
 DRAWING NO.: 10




XX

ONTARIO



full excavations

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.



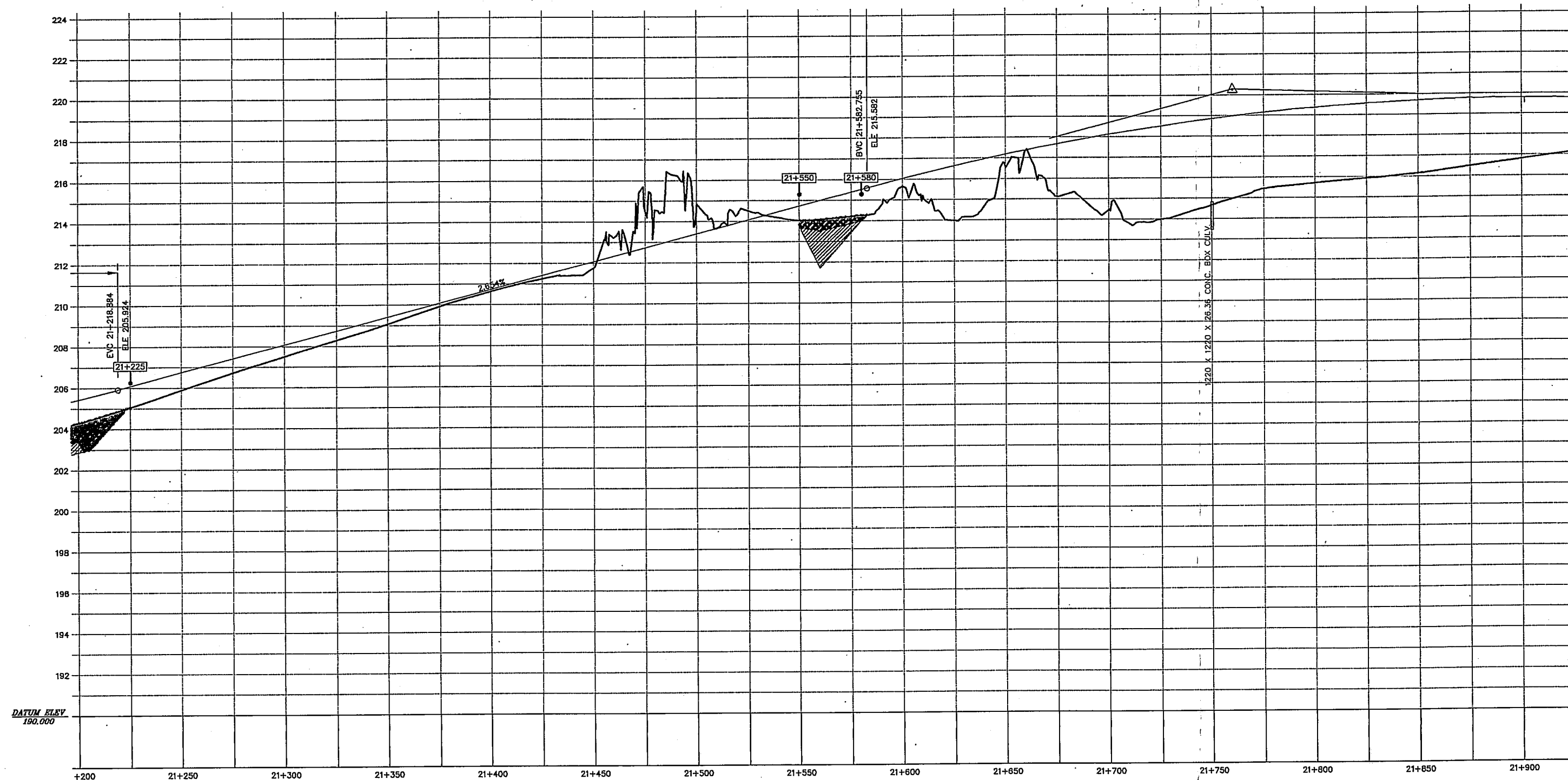
TROW CONSULTING ENGINEERS LTD.
BRAMPTON, ONTARIO

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

PROJECT NO.:	BRGE0011546C
SCALE:	AS NOTED
DRAWN BY:	SDH
CHECKED BY:	SDH
DATE:	JUNE 1998
DRAWING NO.:	11

XX

ONTARIO



DATUM ELEV
190.000

LEGEND:



CLAYEY SOILS



SURFICIAL ORGANIC SOILS

21+550

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.



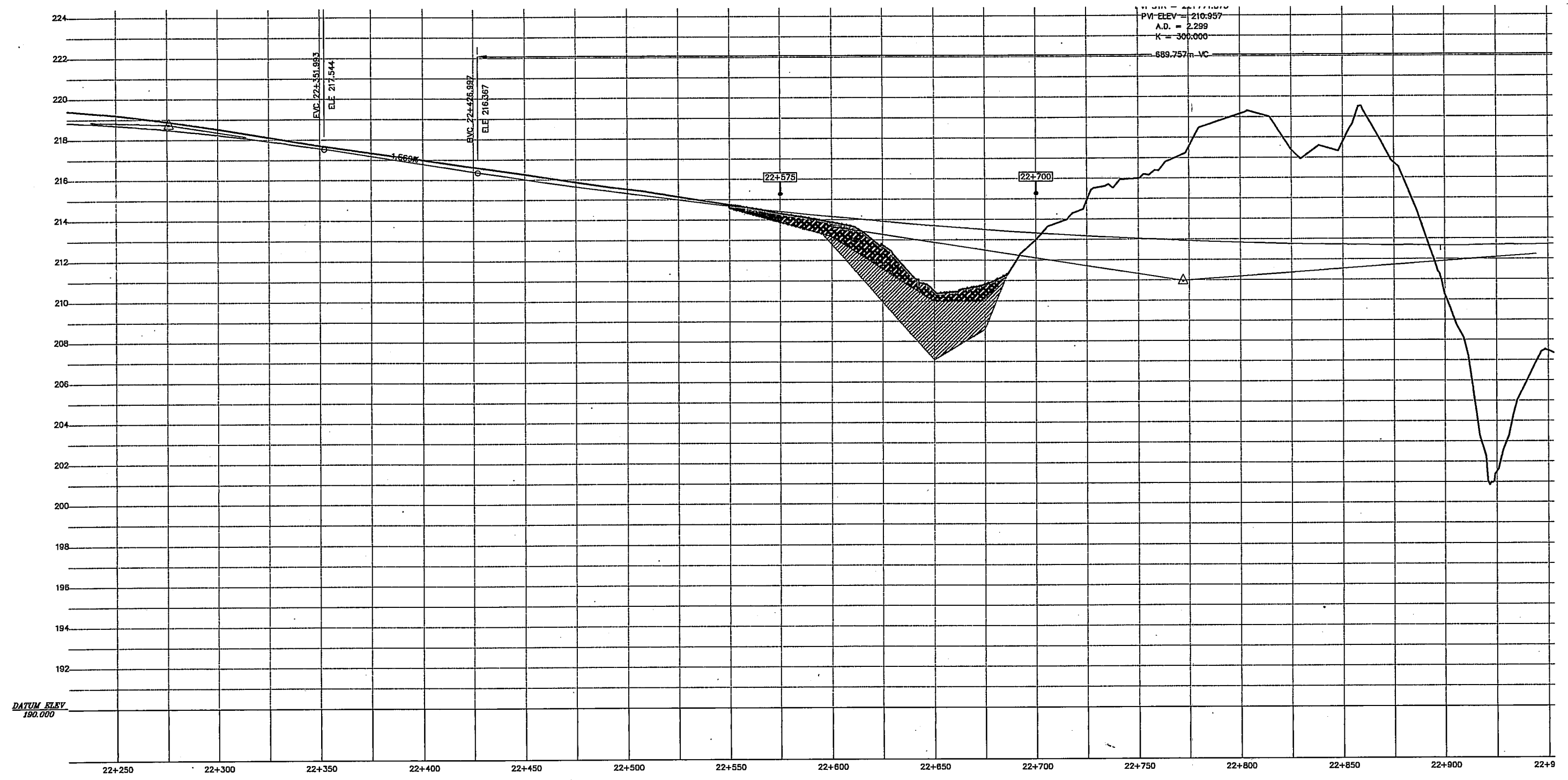
TROW CONSULTING ENGINEERS LTD.
BRAMPTON, ONTARIO

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

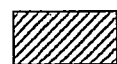
PROJECT NO.:	BRGE0011546C
SCALE:	AS NOTED
DRAWN BY:	SDH
CHECKED BY:	SDH
DATE:	JUNE 1998
DRAWING NO.:	12

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.



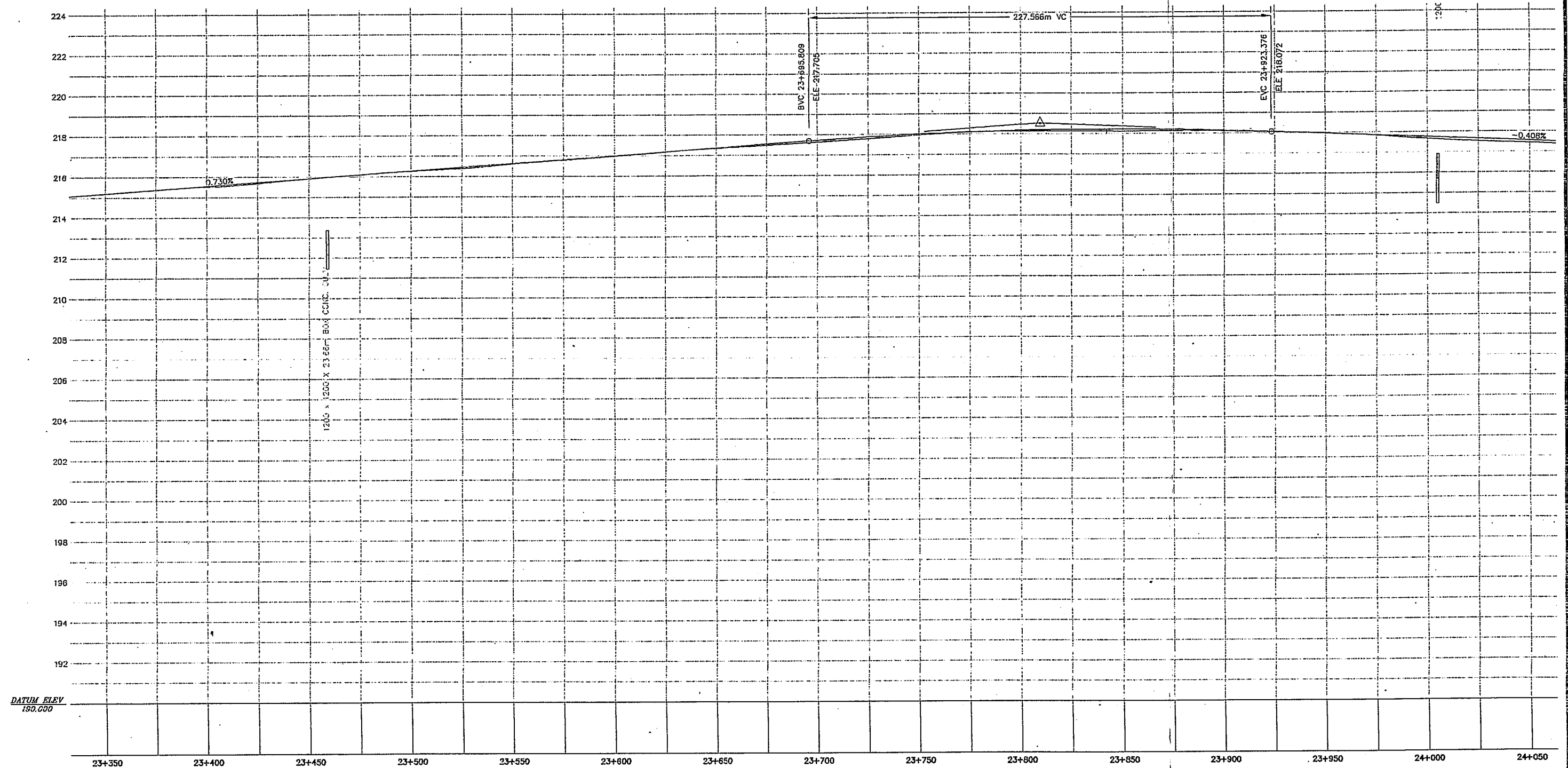
TROW CONSULTING ENGINEERS LTD.
BRAMPTON, ONTARIO

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

PROJECT NO.:	BRGE0011546C
SCALE:	AS NOTED
DRAWN BY:	SDH
CHECKED BY:	SDH
DATE:	JUNE 1998
DRAWING NO.:	13

XX

ONTARIO



LEGEND:

- CLAYEY SOILS
- SURFACIAL ORGANIC SOILS



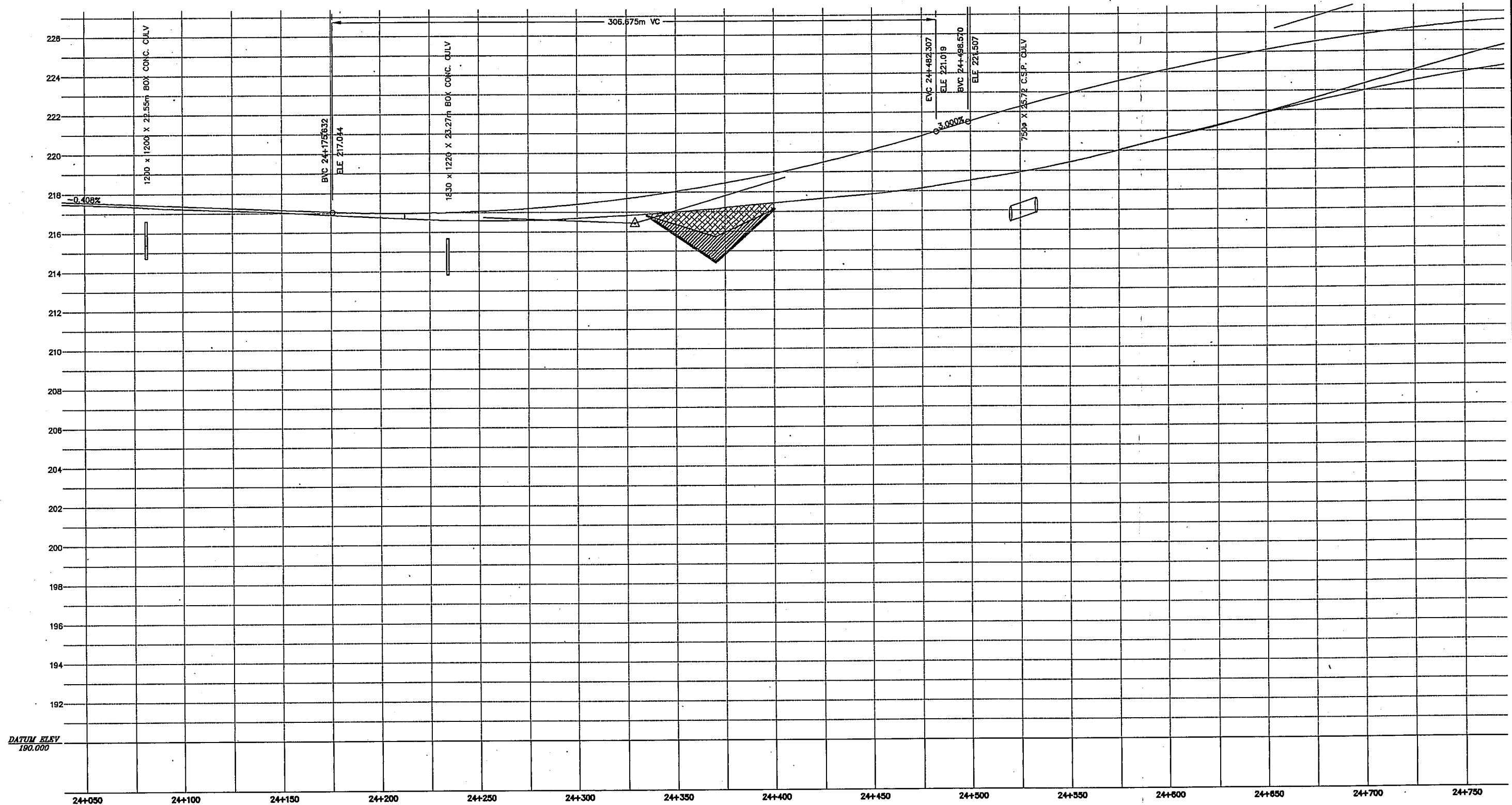
TROW CONSULTING ENGINEERS LTD.
BRAMPTON, ONTARIO

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



LEGEND:

- CLAYEY SOILS
- SURFACIAL ORGANIC SOILS



TROW CONSULTING ENGINEERS LTD.
BRAMPTON, ONTARIO

**HIGHWAY #69
NORTHBOUND LANE
STATION 24+050 - 24+750**

PROJECT NO.:	BR-11546-A
SCALE:	AS NOTED
DRAWN BY:	SDH
CHECKED BY:	SDH
DATE:	JUNE 1998
DRAWING NO.:	15

XX

ONTARIO