FINAL
GEOTECHNICAL INVESTIGATION AND DESIGN REPORT HIGHWAY 61, 0.4 KM NORTH OF CLOUD RIVER ROAD TOWNSHIP OF CROOKS STATION 23 + 075 TO 23 + 150 RT MTO GEOCRES NO.: 52A-153 GWP NO.: 6210-10-00

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Table of Contents

Appendices

Enclosures

List of Tables

List of Figures

GEOTECHNICAL INVESTIGATION FACTUAL REPORT
HIGHWAY 61, 0.4 KM NORTH OF CLOUD RIVER ROAD, TOWNSHIP OF CROOKS,
STATION 23 + 075 TO STATION 23 + 120 RT
AGREEMENT NO.: 6012-E-0047
GWP NO. 6210-10-00
GEOCRES NO. 52A-153

FACTUAL INFORMATION

1. INTRODUCTION

DST Consulting Engineers Inc. (DST) had been retained by Ainley Group to conduct additional geotechnical investigation and provide an updated factual report and design recommendations for remedial works to address embankment slope movement along Highway 61, 0.4 km North of Cloud River Road, Township of Crooks, Station 23 + 075 to Station 23 + 150 Rt. This work has been completed under Agreement No. : 6012-E-0047, GWP 6210-10-00.

Previously DST Consulting Engineers Inc. (DST) had been retained by the Ministry of Transportation (MTO), Geotechnical Section, Northwestern Region to conduct a geotechnical investigation and provide a factual report for a slope failure along Highway 61, 0.4 km north of Cloud River Road, Township of Crooks, Station 23 + 075 to Station 23 + 150 Rt. This work was carried out under Agreement No.: 6009-E-0005 - Geotechnical Retainer, Assignment # 8, GWP 2012-11001 and completed in November 2011.

This report addresses the field investigation, laboratory test program and provides a factual report on ground conditions at the site.

2. SITE DESCRIPTION

The site is located on Highway 61, 0.4 km north of Cloud River Road, Township of Crooks. The investigated section is between Station 23 + 075 and Station 23 + 150.

An indication of embankment movement was found on the northbound lane. A crack on the asphalt paving surface was observed at a section approximately 2 m wide x 50 m long (Figure 2.1). The total height of the existing road embankment and natural valley is approximately 8.0 m and the overall slope of the embankment is at an approximate gradient of 2.3H:1.0V (Figure 2.4). A ditch beside the northbound lane of the highway has a lower elevation than the ditch beside the southbound lane. A culvert exists at the northern extent of the investigated embankment and drains water from the west to the east side of the embankment.

Figure 2.1 Investigated embankment along Hwy 61 (looking northbound)

Figure 2.2 Investigated embankment northbound lane along Hwy 61 (looking northbound)

Figure 2.3 Investigated embankment along Hwy 61 (looking southbound)

Figure 2.4 Embankment slope northbound lane (looking west)

Figure 2.5 Concrete culvert outlet at northbound lane (looking south).

3. INVESTIGATION PROCEDURES AND LABORATORY TESTING

Site work was carried out between May 27th and June 2nd, 2011 utilizing a truck mounted CME 55 drill rig as well as hand equipment operated by DST personnel. A total of three (3) foundation boreholes, using hollow stem augers, and two (2) pedo boreholes using hand equipment were advanced. Foundation boreholes were advanced to a depth of 9.6 to 12.5 m and pedo boreholes were advanced to a depth of 2.5 m. Additional Boreholes 4 and 5 drilling works were carried out between 13th November 2013 and 14th November 2013. Borehole 4 was advanced up to 17.5 m and Borehole 5 was advanced up to 16.5 m depths.

Boreholes were advanced using hollow stem augers. One hydraulically powered borehole was advanced at the edge of the northbound lane and two hydraulically powered boreholes were advanced at the edge of the southbound lane. A monitoring standpipe was installed in Borehole 3 for the purpose of determining the groundwater level.

Borehole locations and stratigraphic sections are shown on the Borehole Location Plan, and Drawings 1, 2 and 3. The numbers, locations and depths of all boreholes were specified by MTO in consultation with DST.

The centreline of the more severe movement zone was determined to be Station 23 + 125, as indicated on the base drawings provided by the MTO. The ground surface elevations at the borehole locations were surveyed by DST personnel. Elevations were measured at the borehole locations at the slope failure site and reference to a temporary benchmark located at approximately Station 23 + 157, approximately 13.5 m right of the road centreline with an assigned elevation of 100.00 m. Borehole elevations and benchmark elevations were subsequently surveyed by Delta Survey on October 7, 2011 and borehole elevations were converted with reference to the datum elevations (208.92 m). Borehole 4 and 5 locations were surveyed on November 13, 2013 by DST personnel referencing the same benchmark used in earlier works. Table 3.1 summarizes the borehole locations elevations and depths.

Fieldwork was supervised on a full-time basis by DST personnel who located the boreholes in the field, performed sampling, in-situ testing and logged of the boreholes. In-situ tests included Standard Penetration Tests (SPT) and Field Vane Tests (FVT). Soil samples collected during drilling were identified in the field, placed in labelled containers and transported to DST's laboratory in Thunder Bay for further analysis.

All boreholes were abandoned using a suitable abandonment barrier as described in Ontario

Regulation 903 and its amendments. Boreholes were decommissioned by backfilling to the bottom of the road base with cuttings and bentonite chips. From the bottom of the road base, granular materials were replaced to the bottom of the surface treatment. Cold mix asphalt was then placed to the surface.

Classification and index tests were subsequently performed in the laboratory on samples collected from the boreholes to aid in the selection of engineering properties. Laboratory tests included moisture contents, sieve analyses, and Atterberg limits. A total of sixty four (64) moisture contents, fifteen (15) particle size analyses and eleven (11) Atterberg limits were carried out for this assignment. Laboratory test results are presented in the Boreholes Logs (Enclosures 1 to 7), and Graphical Plots (Enclosures 8 and 12).

Table 3.1 Details of borehole locations

Borehole ID	Station	Elevation (m)	Depth (m)	Offset (m)
BH1	23+125	210.5	12.5	4.6 Rt
BH2	23+125	210.8	9.6	4.6 Lt
BH3	23+150	210.6	12.1	4.6 Lt
BH4	23+100	210.6	17.5	4.6 Rt
BH5	23+150	210.6	16.5	4.6 Rt
HA1	23+150	204.0	2.5	16.0 Lt
HA2	23+125	204.0	2.5	16.0 Lt

4. DESCRIPTION OF SUBSURFACE CONDITIONS

The subsurface conditions at the failure locations are presented based on the data obtained during field and laboratory testing.

The generalized stratigraphy of the existing embankment, based on the conditions encountered in Boreholes 1 through 5, consists of asphalt overlying a fill material with deeper fill encountered near the existing culvert. Underlying the fill material is a silty clay, which is again underlain by silt and sand. This sand and silt is underlain by a lower silty clay layer. Within the silty clay, silt with some clay was encountered in the Boreholes 4 and 5 at depths of 13.1 to 15.6 m and 11.8 to 13.3 m respectively beneath the lower silty clay. The generalized stratigraphy at the toe of the embankment, based on conditions encountered in hand auger holes HA1 and HA2, consists of native silty clay. Groundwater was observed in the boreholes during drilling at elevations between 202.1 m to 205.0 m. Groundwater levels are expected to vary with precipitation events. At the time of the investigation, water was observed in the ditches at the toe of the embankment.

Table 4.1 Summary of subsurface soil profile under the pavement

Layer	Depth (m)*	Elevation (m)**	Comments
Asphalt	0.04 to 0.2	210.8(210.5) to 210.7(210.4)	
Fill (sand with gravel and gravelly sand)	0.04(0.2) to 1.3(7.1) m	210.7(210.4) to 209.5(203.5)	Deep fill encountered in BH4 adjacent culvert
Upper Silty Clay	1.3(2.9) to 5.5(7.0) m	209.5(207.6) to 205.3(203.6)	Not including BH4
Silt (Sandy Silt or Silt and Sand)	5.5(7.0) to 8.0(11.3) m	205.3(203.6) to 202.6(199.3)	Not including BH4
Silt -some Clay	11.8(13.1) to 13.3(15.6) m	198.8(197.5) to 197.3(195.0)	
Lower Silty Clay or Clay and Silt	8.0(11.3) to >16.5(18.3) m	197.3(195.0) to <194.1(192.3)	BH4 advanced to 18.3 m BH5 advanced to 16.5 m

*Depth (m) presented as upper boundary min (max) to lower boundary min (max)
** Elevation (m) presented as upper boundary min (max) to lower boundary min (max)

Cross-sectional profiles of the site can be found in Drawings 2 and 3. Grainsize distributions and Atterberg Limit tests of material are reported in the Borehole Logs (Enclosures 1 to 7) and Plots (Enclosures 8 through 12).

4.1 Asphalt

Asphalt was encountered in Boreholes 1, 2, 3, 4 and 5 from surface with thicknesses of 100 mm, 100 mm, 100 mm, 40 mm and 200 mm respectively.

4.2 Sand with Crushed Gravel to Gravelly Sand (Fill)

Very loose to compact sand with crushed gravel to gravelly fill layer was encountered in all boreholes from ground surface down to depths between 0.1 and 1.3 m (209.2 and 209.2 m elevations) and 2.1 and 2.9 m (208.4 and 207.6 m elevation) in Borehole 1, between 0.1 and 1.3 m (210.7 and 209.5 m elevation) in Borehole 2, between 0.1 and 0.6 m (210.5 and 210.0 m elevation) in Borehole 3, between 0.04 and 0.8 m (210.6 and 209.9 m elevation) and 2.5 and 3.8 m (208.1 and 206.8 m elevation) in Borehole 4, and between 0.2 and 1.5 m (210.4 and 209.3 m elevation) in Borehole 5.

SPT 'N' values obtained in this material ranged from 4 to 27 blows per 0.3 m penetration indicating a very loose to compact condition. Gradation analyses conducted on samples from Boreholes 1 and 3 indicate gravel, sand, and fine contents from approximately 27 to 37 %, 51 to 53 % and 10 % to 22 % respectively. This material does not meet a strict adherence to OPSS Granular A specifications as material percentages passing the 9.5 mm, 4.75 mm and 75 µm sieves were too high in some samples by approximately 11 %. The moisture content of samples was between 3 and 15 %. The results of the laboratory tests are summarized in Table 4.2.

Table 4.2 Summary of sand and gravel fill sieve analyses

Laboratory Results - Sieve Analyses	
Gravel %	27 to 37
Sand %	51 to 53
Fines %	10 to 22

4.3 Sand (Fill)

A loose to compact sand fill layer was encountered at depths between 1.3 and 2.1 m (209.2 m and 208.4 m elevation) in Borehole 1, between 0.6 and 2.0 m (210.0 m and 208.6 m elevation) in Borehole 3, between 0.8 and 2.5 m (209.9 and 208.1 m elevation), 3.8 and 5.3 m (206.8 and 205.3 m elevation) and 6.3 and 7.1 m (204.3 and 203.5 m elevation) in Borehole 4, and between 1.5 and 2.4 m (209.1 and 208.2 m elevation) in Borehole 5.

SPT 'N' values obtained in this material ranged from 5 to 21 blows per 0.3 m penetration indicating a loose to compact condition. Gradation analyses conducted on samples from Boreholes 1, 3 and 4 indicate gravel, sand, and fine contents of approximately 5 to 7 %, 64 to 74 % and 19 to 31 % respectively. This material does not meet OPSS Granular B, Type I or II specifications. The moisture content of the samples was between 6 to 17 %. The result of the laboratory tests are summarized in Table 4.3.

Table 4.3 Summary of sand fill sieve analyses

Laboratory Results - Sieve Analyses	
Gravel %	5 to 7
Sand %	64 to 74
Fines %	19 to 31

4.4 Upper Silty Clay

An upper soft to firm silty clay layer was encountered at depths between 2.9 and 6.4 m (207.6 m and 204.1 m elevation) in Borehole 1, between 1.3 and 5.5 m (209.5 m and 205.3 m elevation) in Borehole 2, between 2.0 and 7.0 m (208.6 m and 203.6 m elevation) in Borehole 3, between 7.1 and 8.6 m (203.5 and 202.0 m elevation) in Borehole 4 and between 2.4 and 6.3 m (207.0 and 203.1 m elevation) in Borehole 5.

Atterberg limits tests carried out on samples indicate this clay varies from low to high plasticity with liquid limit and plasticity index of 23 to 57 % and 9 to 35 % respectively. In-situ field vane tests carried out indicate undrained shear strengths of 80 to 155 kPa with sensitivity of 2 which indicates a stiff to very stiff condition. Moisture contents of samples range from 8 to 44 %. The result of the laboratory tests are summarized in Table 4.4.

Table 4.4 Summary of laboratory tests for upper clay

Atterberg Limits	
Liquid Limit	23 to 57
Plastic Limit	14 to 24
Plasticity Index	9 to 35

4.5 Silt to Silt and Sand

Very loose to loose sandy silt to silt and sand layers were encountered at depths between 6.4 and 8.5 m (204.1 m and 202.0 m elevation) in Borehole 1, between 5.5 and 9.3 m (205.3 m and 201.5 m elevation) in Borehole 2, between 7.0 and 11.3 m (203.6 m and 199.3 m elevation) in Borehole 3, between 8.6 and 9.6 m (202.0 and 201.0 m elevation) in Borehole 4 and between 6.3

and 8.0 m (204.3 and 202.6 m elevation) in Borehole 5.

SPT 'N' values obtained in this material range from 2 to 7 blows per 0.3 m penetration indicating a very loose to loose condition. Gradation analyses conducted on samples from Boreholes 2, 3, 4 and 5 indicate gravel, sand, and fine contents of approximately 0 to 1 %, 20 to 48 % and 52 to 80 % respectively. The moisture content of the samples was between 19 to 30 %. The result of the laboratory tests are summarized in Table 4.5.

Table 4.5 Summary of sand and gravel fill sieve analyses

Laboratory Results - Sieve Analyses	
Gravel %	0 to 1
Sand %	20 to 48
Fines %	52 to 80

4.6 Lower Silty Clay to Clay and Silt

A lower soft silty clay layer was encountered at depths below 8.5 m (202.0 m elevation) in Borehole 1, below 9.3 m (201.5 m elevation) in Borehole 2, below 11.3 m (199.3 m elevation) in Borehole 3, between 9.6 and 13.1 m (201.0 and 197.5 m elevation) and below 15.6 m (195.0 m elevation) in Borehole 4 and between 8.0 and 11.8 m (202.6 and 198.8 m elevation) and below 13.3 m (197.3 m elevation) in Borehole 5. The thickness of this stratum is not defined in Boreholes 1, 2, 3, 4 and 5 as terminus of borehole sampling was reached at a depths of 12.5 m (198.0 m elevation), 9.6 m (201.2 m elevation), 12.5 m (198.0 m elevation), 18.3 m (192.3 m elevation) and 16.5 m (194.1 m elevation) prior to the bottom of the stratum respectively.

Atterberg limits tests carried out on samples indicate the silty clay has low plasticity with liquid limit and plasticity index of 25 to 32 % and 6 to 11 % respectively. In-situ field vane tests carried out indicate undrained shear strengths of 30 to 120 kPa with sensitivity varying between 1 and 4 which indicates a firm to very stiff condition. Moisture contents of samples ranged from 20 to 36 %. The result of the laboratory tests are summarized in Table 4.6.

Table 4.6 Summary of laboratory tests for lower clay to clay and silt

Laboratory Results - Sieve Analyses	
Gravel %	0
Sand %	0 to 3
Silt %	70 to 74
Clay %	24 to 30
Atterberg Limits	
Liquid Limit	25 to 32
Plastic Limit	15 to 22
Plasticity Index	6 to 11

4.7 Silt with Some Clay

Stiff to very stiff silt with some clay layer was encountered at depths between 13.1 and 15.6 m (197.5 m and 195.0 m elevation) in Boreholes 4 and between 11.8 and 13.3 m (198.8 m and 197.3 m elevation) in Borehole 5.

Atterberg limits tests carried out on samples indicate this silt has low plasticity with liquid limit and plasticity index of 19 to 25 % and 1 to 3 % respectively. In-situ field vane tests carried out indicate undrained shear strengths of 75 to 110 kPa with sensitivity varying between 2 and 3 which indicates a stiff to very stiff condition. Moisture contents of samples ranged from 26 to 28 %. The result of the laboratory tests are summarized in Table 4.7.

Table 4.7 Summary of laboratory tests for silt

Laboratory Results - Sieve Analyses	
Gravel %	0
Sand %	0 to 1
Silt %	83 to 89
Clay %	10 to 13
Atterberg Limits	
Liquid Limit	19 to 25
Plastic Limit	18 to 22
Plasticity Index	1 to 3

4.8 Groundwater

Temporary water standpipes were installed in the boreholes and water level elevations were measured. Groundwater levels can be expected to vary with season and precipitation events. The measured depths of groundwater levels below the ground surface elevation are given in Table 4.8.

Table 4.8 Elevations of water table at boreholes

Borehole ID	Borehole Ground Surface Elevation (m)	Groundwater table elevation (m)	Date
BH 1	210.5	204.5	2 June 2011
BH 2	210.8	205.0	2 June 2011
BH 3	210.6	204.4	2 June 2011
BH 4	210.6	202.7	13 November 2013
BH 5	210.6	203.0	13 November 2013
HA 1	204.0	202.1	2 June 2011

5. MISCELLANEOUS

Original site work was carried out between May 27th and June 2nd, 2011 utilizing a truck mounted CME 55 drill rig as well as hand equipment operated by DST personnel. Additional site work, including Boreholes 4 and 5 were carried out between November 13th and 14th, 2013. Fieldwork was supervised on a full time basis by Joe Forgues who located the boreholes in the field, performed sampling, in-situ testing and logged the boreholes. Soil samples collected during drilling were identified in the field, placed in labelled containers and transported to DST's laboratory in Thunder Bay for further analysis. Interpretation of the data and preparation of the report was completed by Tun Lwin, P.Geo and Wes Saunders, P.Eng and reviewed by Prof. Myint Win Bo, P.Eng a designated principal contact for MTO projects.

GEOTECHNICAL INVESTIGATION FACTUAL REPORT
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PART 2: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

6. PROJECT DESCRIPITION

DST Consulting Engineers Inc. (DST) had been retained by Ainley Group to conduct additional geotechnical investigation and provide an updated factual report and design recommendations for remedial works to address embankment slope movement along Highway 61, 0.4 km north of Cloud River Road, Township of Crooks, Station 23 + 075 to Station 23 + 150 Rt. This work has been completed under Agreement No. : 6012-E-0047, GWP 6210-10-00.

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This report provides the recommendations for remedial works to address embankment slope movement.

7. SOIL PARAMETERS FOR STABILITY ANALYSIS

A representative stratigraphy has been interpreted from the borehole data of Highway 61 from Station 23 + 125. A ground model was prepared based on the results interpreted from the field and laboratory data obtained.

The generalized stratigraphy of the existing embankment, based on the conditions encountered in Boreholes 1 through 5, consists of asphalt overlying a fill material with deeper fill encountered near the existing culvert. Underlying the fill material is a silty clay, which is again underlain by sand and silt. This sand and silt is underlain by a lower silty clay layer. Within this silty clay, silt with some clay was encountered in the Boreholes 4 and 5 at depths of 13.1 to 15.6 m and 11.8 to 13.3 m respectively beneath the lower silty clay. The generalized stratigraphy at the toe of the embankment, based on conditions encountered in Hand Auger Holes HA1 and HA2, consists of native silty clay.

The groundwater level was found at elevations between 202.1 m to 205.0 m during the site investigation (May 2011) and 202.7 m and 203.0 m in Boreholes 4 and 5 respectively (November 2013).

Soil properties interpreted based on in-situ and laboratory test results are shown in Table 6.1 which were used in the modelling. Several different conditions were considered to assess the physical stability of the embankment. The short term stability of the slopes was checked using total stress (undrained) parameters, while the long term stability was checked using effective stress (drained) parameters.

Table 7.1 Soil parameters used in the slope stability analysis

Material	Density (kN/m^3)	Drained Angle of Internal Friction (degrees)	Undrained Cohesion (C) kPa
Granular Fill (Sand and Crushed Gravel)	21	30 – 34 (30)	-
Granular Sand Fill	19	28 – 33 (30)	-
Clay – Intermediate to High Plasticity	18	24 – 30 (24)	80 – 155 (80)
Sandy Silt (Cohesionless)	19	30 (30)	-
Clay – Low Plasticity	18	28 – 30 (28)	30 - 120 (30)
Silt (Cohesive)	18	28 – 30 (28)	75 – 110 (75)
Rock Fill	19	45 (45)	-

* Values in the bracket are design parameters used for stability analysis.

8. SLOPE STABILITY ANALYSIS

8.1 Existing Conditions

Potential mechanisms of movement of the embankment were analyzed, including deep circular failures through the underlying silty clay soils, and shallow slides through the fill materials. Analyses were performed with high water levels to reflect seasonal variations.

Analyses for circular failures, both deep and shallow, were carried out for the critical cross-section located at Station 23+125 for total and effective stress conditions using Slope/W software developed by Geo-slope International. Total stress and effective stress condition analyses were carried out. As Morgenstern & Price's method satisfies force equilibrium, overall moment equilibrium and inter slice moment equilibrium as well as providing consistent results for all groundwater conditions, this method was applied and factors of safety from this method have been reported here. In each analysis two line loads of 50 kN were applied in the north bound lane to represent truck traffic.

The following table shows factors of safety of the selected cases under total stress and effective stress conditions for the current embankment configuration. Slope stability analysis outputs can be found in Appendix C, Figures C1 through C3. A cross section of the existing slope is provided in Drawing 3.

Table 8.1 Slope stability analysis result for existing embankment

Embankment	Groundwater Condition	Existing Embankment Slope	Factor of Safety	
			Total Stress Analysis	Effective Stress Analysis
Existing Embankment	High water level	2.3H:1V	1.2	1.0

Stability analyses of the existing embankment profile and present soil parameters, estimated from the recent site investigation, shows a factor of safety of 1.2 for the total stress condition analysis. However, stability analysis with effective stress condition resulted in a factor of safety close to unity.

This indicates that the embankment has a reasonable factor of safety for total stress condition. However, this is not the case for the effective stress condition which reflects long term conditions.

The long term condition only has a factor of safety of unity. This indicates that the embankment has a long term instability problem. This also indicates that the input parameters used in the slope stability analyses reasonably represent the average overall strength parameters of the in situ soil mass and the parameters are very close to the critical state soil parameters. Therefore, stabilization measures are required to maintain long term embankment stability.

8.2 Stability Analyses with Steepened Upper Slope and Counterweight Berm

The embankment stability analyses were carried out with a remedial profile of a steepened embankment constructed of rock fill materials placed over a properly benched 1H: 1V backslope excavated from the existing centreline of road down to 203.5 m elevation. This profile was analysed assuming a high creek water level of 204.0 m at the toe of the embankment with an elevated groundwater level within the embankment.

The steepened rock fill embankment with upper and lower sides of 1.25H: 1V respectively constructed with a counterweight rock berm with a bench elevation at 205.5 m and width of 2.0 m was considered.

This option resulted in acceptable factors of safety of greater than or equal to 1.3. For these analyses, the total stress and effective stress parameters were used. The following table shows factors of safety of the selected cases under total stress and effective stress conditions. Examples of slope stability analysis outputs can be found in Appendix C, Figures C.4 through C.6. Plan view and cross sections of the proposed remedial methods are provided in Appendix B, Drawing 4.

Table 8.2 Slope stability analyses of steepened embankment conditions

Remedial Option	Groundwater Condition	Proposed Embankment Slope		Bench Elevation, (m)	Minimum Bench Width, (m)	Factor of Safety	
		Upper	Lower			Total Stress Analysis	Effective Stress Analysis
Steepened Rock Fill Embankment with Counterweight Berm	High water	1.25H:1.0V	1.25H:1.0V	205.5	2.0	>1.5	1.3

8.3 Stability Analyses with Counterweight Berm at Toe of Slope

The embankment stability analyses were carried out with a remedial profile of the existing embankment constructed of sand fill and native clay with a counterweight berm placed over a properly benched 1H: 1V backslope excavated from approximately 13 m right of the existing centreline down to 203.0 m elevation. This profile was analysed assuming a high creek water level of 204.0 m at the toe of the embankment with an elevated groundwater level within the embankment.

The existing embankment with granular fill foreslopes of 2H: 1V and with rock fill counterweight berm side slope of 1.25H: 1V, a bench elevation at 207.5 m and width of 5.5 m was considered..

This option resulted in acceptable factors of safety of greater than or equal to 1.3. For these analyses, the total stress and effective stress parameters were used. The following table shows factors of safety of the selected cases under total stress and effective stress conditions. Examples of slope stability analysis outputs can be found in Appendix C, Figures C.7 through C.9. Plan view and cross sections of the proposed remedial methods are provided in Appendix B, Drawing 5.

Table 8.3 Slope stability analyses of steepened embankment conditions

Remedial Option	Groundwater Condition	Proposed Embankment Slope		Bench Elevation, (m)	Bench Width, (m)	Factor of Safety	
		Upper (Existing)	Lower (Rock Fill)			Total Stress Analysis	Effective Stress Analysis
Existing Embankment with Counterweight Berm	High water	2H:1.0V	1.25H:1.0V	207.5	5.5	>1.5	1.3

9. RECOMMENDATIONS

All recommended remedial options must include the evaluation of the existing site hydrology and improvement of the existing roadside drainage.

To remediate the slope movement at this location, it is recommended that the limits of remediation extend from approximately Station 23+075 to Station 23+150 for all remedial options.

9.1 Steepened Upper Slope and Counterweight Berm

Construction methodology must be completed in accordance with all relevant Ministry guidelines. It is assumed the existing embankment would be excavated and properly benched at a 1H:1V slope from approximately the centreline of the existing alignment to an elevation of approximately 203.5 m and constructed with benching as per OPSD 208.010.

With a properly benched 1H:1V backslope underlying new rock fill construction, the minimum embankment required to stabilize the embankment with a factor of safety greater than 1.3 is a rock fill embankment with upper and lower side slopes of 1.25H:1.0V constructed with a counterweight rock berm with a bench elevation at 205.5 m and width of 2.0 m. This corresponds to a minimum height of rock fill at the bench of approximately 2.0 m. All rock fill materials should meet requirements of Rip-Rap R-10 as per OPSS 1004. The proposed configuration of this remediation is shown in Drawing 4 in Appendix B.

The anticipated required embankment should extend from approximately Station 23+075 to Station 23+150 and be tapered into the existing slope at the end locations.

It is recommended that all existing organics be removed prior to the placement of materials on the slope. Upon completion of fill placement, topsoil should be placed to support vegetative growth and the slope should be vegetated with native saplings.

A non-woven geotextile should also be placed between the rock materials and any contacting materials including the underlying clay and potentially overlying topsoil. The geotextile will help to prevent the movement of fines, and provide additional erosion resistance. The non-woven geotextile should conform to OPSS 1860.07.05.01 Class II and have a filtration opening size (FOS) less than or equal to 135 µm.

9.2 Counterweight Berm at Toe of Slope

Construction methodology must be completed in accordance with all relevant Ministry guidelines. It is assumed the existing embankment would be excavated and properly benched at a 1H:1V slope from approximately 13 m right of the existing centreline and elevation of 207.5 m to an elevation of approximately 203.0 m or existing ground level whichever is higher at the base, and constructed with benching as per OPSD 208.010. This treatment method will require the rerouting of the existing drainage flow an additional 6 m right of centreline.

With a properly benched 1H:1V backslope underlying the new counterweight berm construction, the minimum rock fill counterweight berm required to stabilize the embankment with a factor of safety greater than 1.3 is a counterweight rock berm with side slopes of 1.25H:1.0V, a bench elevation at 207.5 m and width of 5.5 m or less to minimum of 1.25 m where reduces width is required due to space constrains. All rock fill materials should meet requirements of Rip-Rap R-10 as per OPSS 1004. The proposed configuration of this remediation is shown in Drawing 5 in Appendix B.

The anticipated required embankment should extend from approximately Station 23+075 to Station 23+150 and be tapered into the existing slope at the end locations.

It is recommended that all existing organics (including all tree growth) be removed prior to the placement of rock materials on the slope. Upon completion of fill placement, topsoil should be placed to support vegetative growth and the slope should be vegetated with native saplings.

A non-woven geotextile should also be placed between the rock materials and any contacting materials including the underlying clay and potentially overlying topsoil. The geotextile will help to prevent the movement of fines, and provide additional erosion resistance. The non-woven geotextile should conform to OPSS 1860.07.05.01 Class II and have a filtration opening size (FOS) less than or equal to 135 µm.

9.2.1 Culvert Realignment

As an alternative of the provided remedial option above, realignment of the existing culvert north of the embankment failure such that water flow is directed toward the downstream area right of the roadway alignment at approximately Station 23+075 should be considered. This would require a skewed culvert with inlet at approximately Station 23+145 and outlet at approximately Station 23+090 and would prevent the flow of water parallel to the roadway alignment at the location of the soil movement.

9.3 General Recommendations

9.3.1 Site Drainage

The accumulation of water at the top of the embankment during a significant storm event may affect the stability of the embankment. It is recommended to review the existing discharge capacity of roadside drainage and structures in the vicinity to be able to discharge water from storm events.

During the site inspection on 18 October 2011, DST noticed that water is flowing underneath the existing culvert (see Figure 2.6). This may cause internal erosion below the existing culvert in the long run and should be prevented. A clay seal or alternative should be used upstream to prevent the movement of fine material.

9.3.2 Erosion Protection

To protect the remedial measures constructed, it is recommended that erosion protection is installed beyond the low water level. The size of rip rap for erosion protection should be selected based on the hydraulic characteristics of the predicted water flows.

10. CLOSURE

Based on the information collected from field investigation, parameters interpreted from laboratory test results and groundwater monitoring data, the present embankment condition is stable under the total stress condition which is considered as short term stability soon after construction. However, movement at the site is likely to have resulted from movement in the underlying clay layer which occurred under the long term condition where effective stress parameters play a key role. This movement is likely due to the insufficient resisting forces or moments for the particular geometry of the slope under effective stress conditions and influenced by changes in the slope geometry as a result of erosion or change in environmental conditions. The presence of a potentially high groundwater level during spring thaw and heavy rainfall events combined with inadequate drainage could further reduce the factor of safety of the embankment.

Remedial options discussed in Section 9 include the construction of a steepened rock fill embankment with upper and lower side slopes of 1.25H:1.0V with a counterweight toe berm as well as the construction of a rock fill counterweight toe berm with a side slope of 1.25H:1.0V while maintaining much of the existing embankment geometry for the upper slope.

Stability analyses were carried out for the critical cross section, however, slope gradients of the embankment become gentler when moving away from the centreline of the embankment. Table 10.1 summarizes the advantages and disadvantages of the considered options.

Table 10.1 Remedial option advantages and disadvantages comparison

Remedial Option	Advantages	Disadvantages
Steepened Rock Fill Embankment with Counterweight Berm	• Certain effectiveness • Construction contained within original site geometry • Construction is out of the water	• Requires partial removal of existing embankment, • Requires detour or traffic control during construction work • Higher construction cost • Larger quantity of material requirements
Existing Embankment with Counterweight Berm	• Certain effectiveness • Ease of construction • Erosion resistant • Potential lowest construction cost • Minimal traffic control required	• Requires removal of vegetation • May require in or near water works and drainage relocation • May require acquisition of additional land or more land space for construction

All remedial options provided are considered certain in effectiveness and constructability. Primary concerns relating to the remediation of the embankment movement include in water works, land acquisition, future drainage consideration and future culvert replacement. The selected option should address all these concerns.

DST is of the opinion that the best suited remedial option is the construction of a rock fill counterweight berm while maintaining much of the existing embankment geometry and materials. This option is recommended as it combines a high degree of certainty in effectiveness while potentially allowing for the continuation of traffic flow.

11. LIMITATIONS OF REPORT

A description of limitations which are inherent in carrying out site investigation studies is given in Appendix 'A', and this forms an integral part of this report.

For DST CONSULTING ENGINEERS INC.

Prepared by:

Deep Bensal, P.Eng.
Project Manager

Bernardo Villegas, MASc
Sector Head

Reviewed by:

Dr. M. W. Bo, PhD., P.Eng., P.Geo., Int. PE,
C.Geol, C.Eng., Eur Geol., Eur Eng.
Senior Vice President/Senior Principal

APPENDIX 'A'

LIMITATIONS OF REPORT

LIMITATIONS OF REPORT

GEOTECHNICAL STUDIES

The data, conclusions and recommendations which are presented in this report, and the quality thereof, are based on a scope of work authorized by the Client. Note that no scope of work, no matter how exhaustive, can identify all conditions below ground. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the specific locations tested, and conditions may become apparent during construction which were not detected and could not be anticipated at the time of the site investigation. Conditions can also change with time. It is recommended practice that DST Consulting Engineers be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavation, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

Unless otherwise noted, the information contained herein in no way reflects on environmental aspects of either the site or the subsurface conditions.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs, e.g. the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

Any results from an analytical laboratory or other subcontractor reported herein have been carried out by others, and DST Consulting Engineers Inc. cannot warrant their accuracy. Similarly, DST cannot warrant the accuracy of information supplied by the client.

A P P E N D I X 'B'

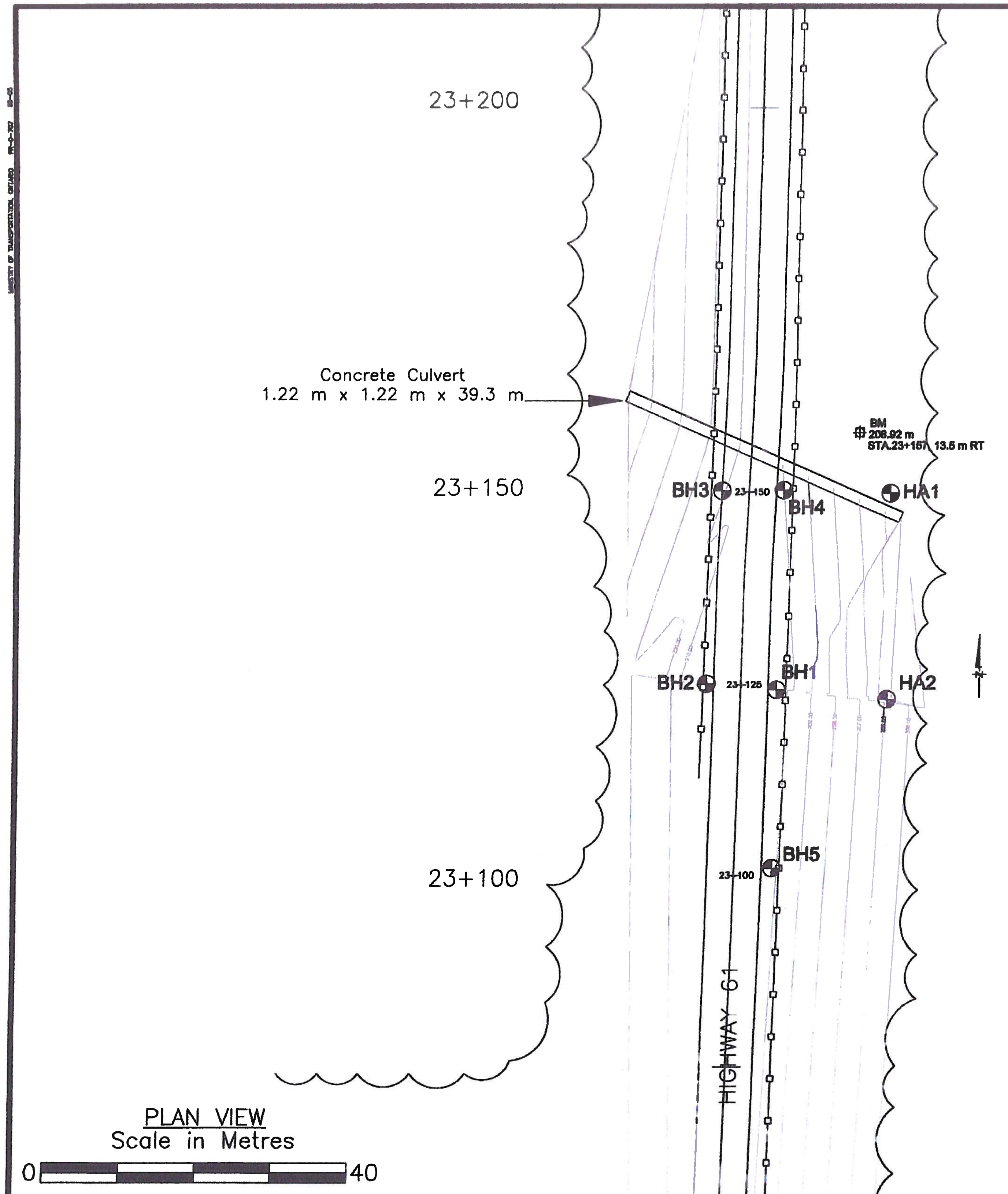
DRAWINGS

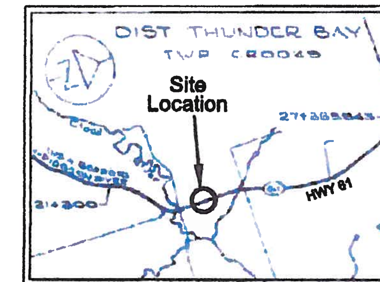
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES

CONT	No	6012-E-0047
GWP	No	6210-10-00
GeoCres	No	52A-153

GEOTECHNICAL INVESTIGATION
HIGHWAY 61
Crooks Township
BOREHOLE LOCATION PLAN

SHEET





KEY PLAN
0 400
SCALE IN METRES

LICENSED PROFESSIONAL ENGINEER
D. BANSAL
100178485
March 20, 2014
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
20th March 2014
M. W. BO
100129834
PROVINCE OF ONTARIO

LEGEND

- Borehole (DST 2013)
- Borehole (DST 2011)
- Dynamic Cone Penetration Test (DCPT)
- Rock Probe
- 'N' Blows/0.3m (Std. Pen Test, 475 J/Blow)
- Water level at time of investigation.
- Benchmark

Fill, Organics, Topsoil, Till, Bedrock, Sand, Silt, Clay, Sand & Gravel, Boulders

No.	Elevation	Northing	Easting	Station	Offset
BH1	210.47	5332723	317011	23+125	4.5 m RT
BH2	210.76	5332724	317002	23+125	4.5 m LT
BH3	210.60	5332761	317003	23+150	4.0 m LT
HA1	204.02	5332765	317025	23+150	10.0 m RT
HA2	204.01	5332722	317010	23+125	10.0 m RT
BH4	210.60	5332761	317014	23+150	4.5 m RT
BH5	210.60	5332702	317011	23+100	4.5 m RT

COORDINATE REFERENCE: MTM NAD83 ZONE 16

NOTE:
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed by interpolation and may not represent actual conditions.

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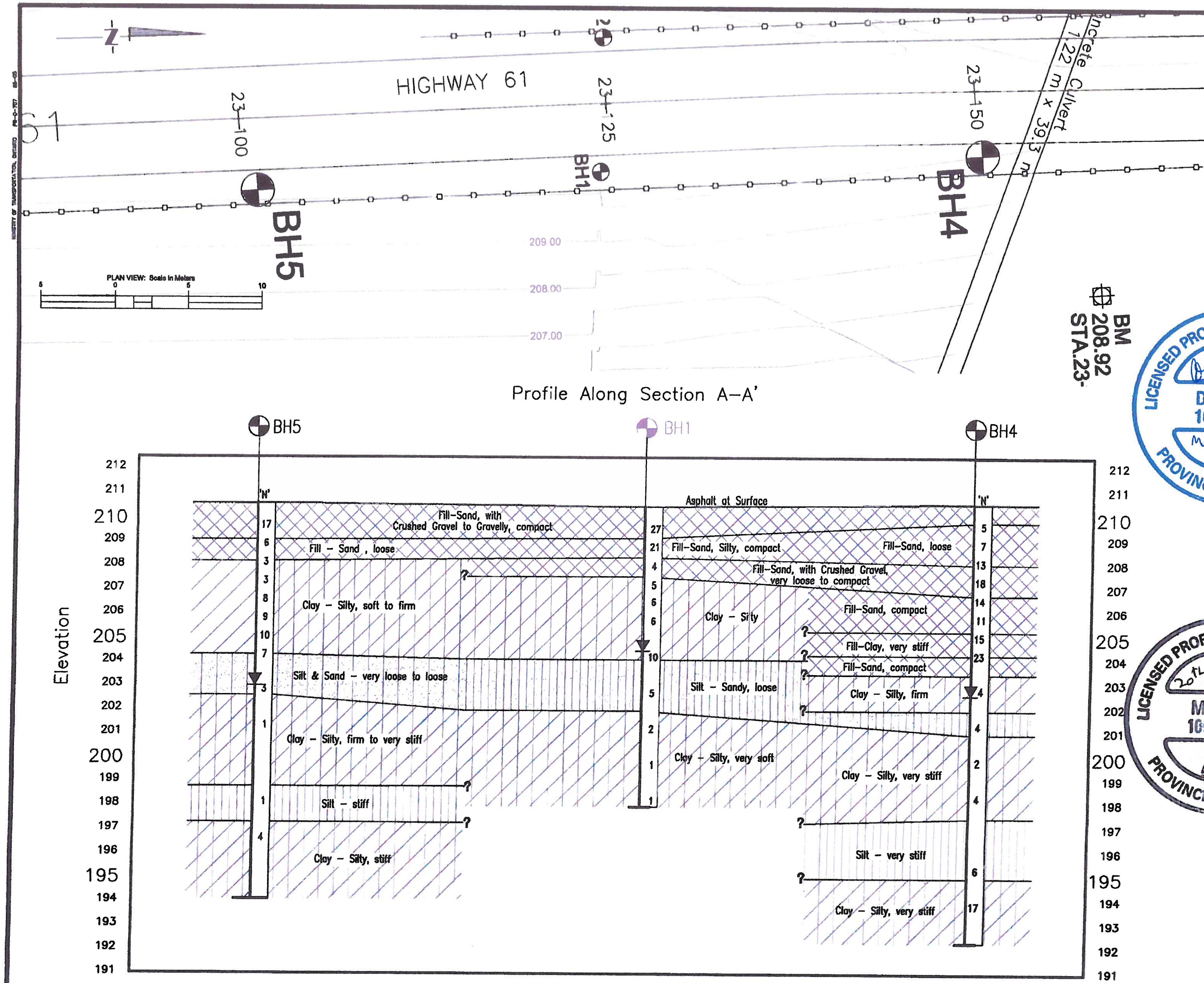
DRAWING 1



CONT No 6012–E–0047
GWP No 6210–10–00
GeoCres No 52A–153

GEOTECHNICAL INVESTIGATION
HIGHWAY 61
Crooks Township
BOREHOLE STRATIGRAPHY

SHEET

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100178485
March 20, 2014
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
20th March 2014
M. W. BO
100129834
PROVINCE OF ONTARIO

LEGEND

- Borehole (DST 2013)
- Borehole (DST 2011)
- Dynamic Cone Penetration Test (DCPT)
- Rock Probe
- 'N' Blows/0.3m (Std. Pen Test, 475 J/Blow)
- Water level at time of investigation.
- Benchmark

Fill, Organics, Topsoil, Till, Bedrock, Sand, Silt, Clay, Sand & Gravel, Boulders

No.	Elevation	Northing	Easting	Station	Offset
BH1	210.47	[illegible]	317611	23+125	4.5 m RT
BH2	210.76	[illegible]	[illegible]	23+125	4.6 m LT
BH3	210.[illegible]	[illegible]	[illegible]	23+1[illegible]	4.6 m LT
HA1	204.02	[illegible]	[illegible]	23+1[illegible]	10.0 m RT
HA2	204.01	[illegible]	[illegible]	23+1[illegible]	10.0 m RT
BH4	210.[illegible]	[illegible]	[illegible]	23+1[illegible]	4.6 m RT
BH5	210.[illegible]	[illegible]	317611	23+100	4.6 m RT

COORDINATE REFERENCE: MTM NAD83 ZONE 16

NOTE: The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed by interpolation and may not represent actual conditions.

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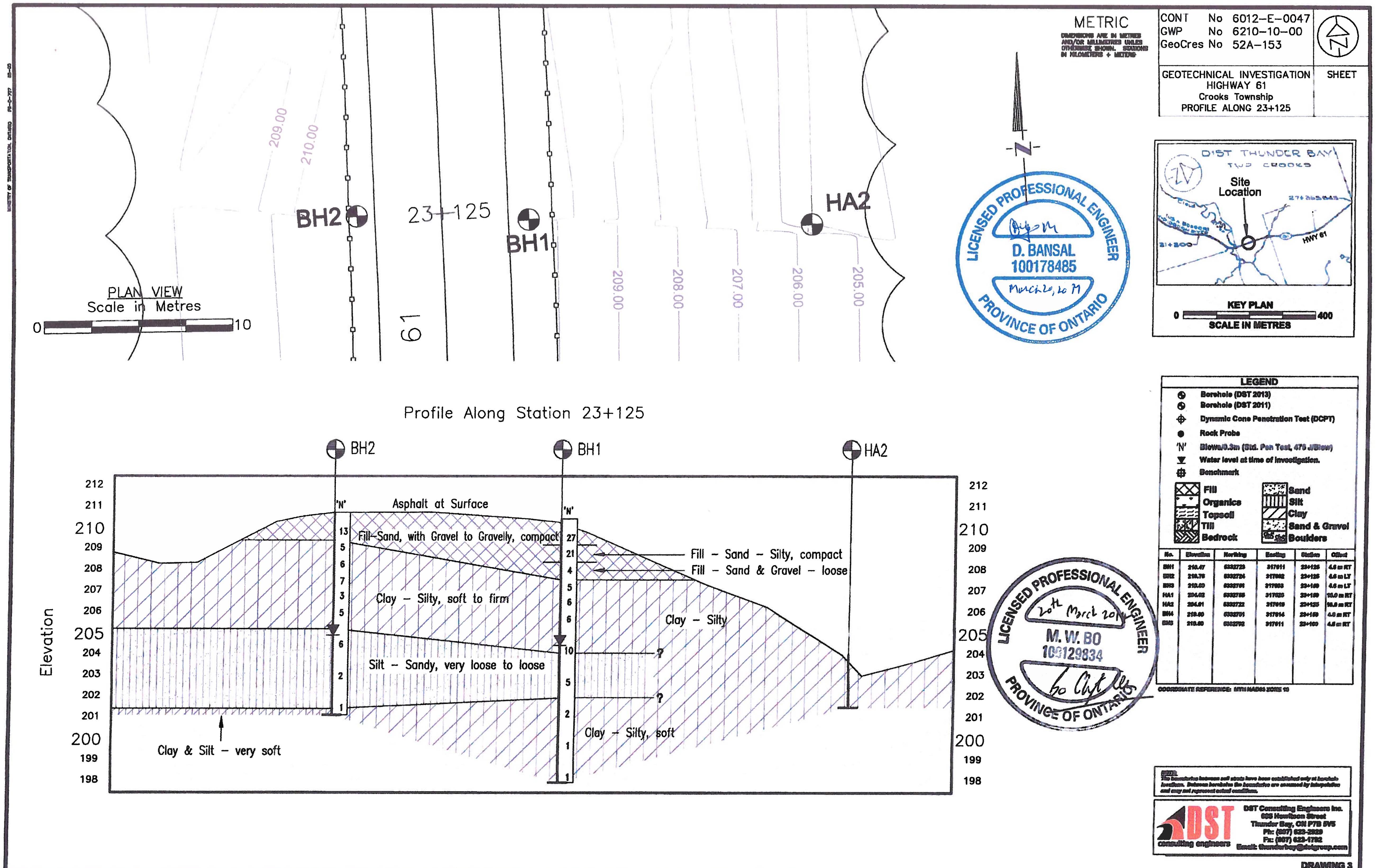
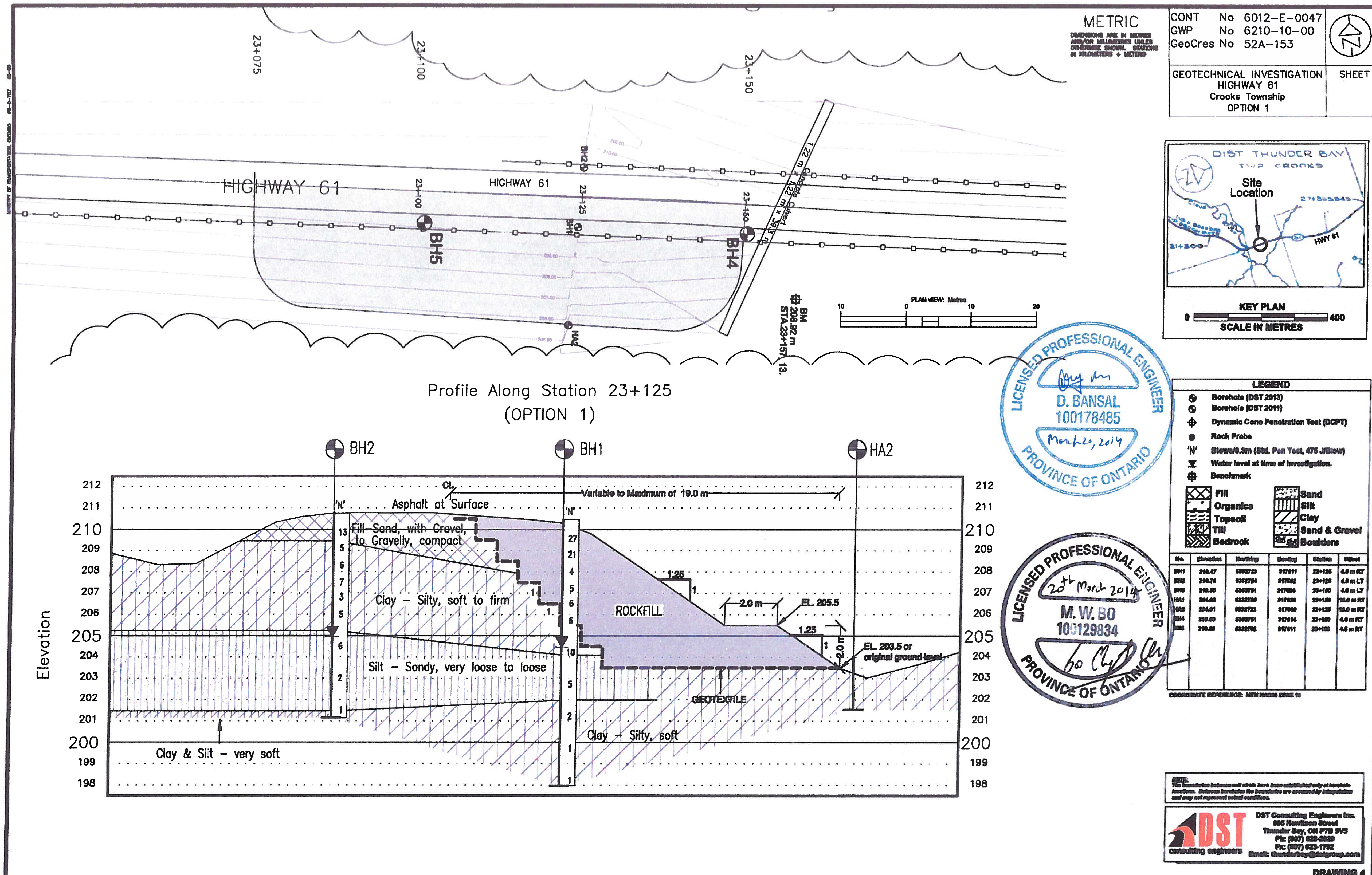
DRAWING 2


METRIC
CONT No 6012–E–0047
GWP No 6210–10–00
GeoCres No 52A–153
GEOTECHNICAL INVESTIGATION
HIGHWAY 61
Crooks Township
PROFILE ALONG 23+125
SHEET
DIST THUNDER BAY
TWP CROOKS
Site Location
HWY 61
KEY PLAN
0
400
SCALE IN METRES
PLAN VIEW
Scale in Metres
0
10
BH2
BH1
HA2
23+125
61
209.00
210.00
205.00
206.00
207.00
208.00
LICENSED PROFESSIONAL ENGINEER
D. BANSAL
100178485
PROVINCE OF ONTARIO
Profile Along Station 23+125
Elevation
Asphalt at Surface
Fill–Sand, with Gravel to Gravelly, compact
Fill – Sand – Silty, compact
Fill – Sand & Gravel – loose
Clay – Silty, soft to firm
Clay – Silty
Silt – Sandy, very loose to loose
Clay – Silty, soft
Clay & Silt – very soft
'N'
LEGEND
Borehole (DST 2013)
Borehole (DST 2011)
Dynamic Cone Penetration Test (DCPT)
Rock Probe
Blows/0.3m (Std. Pen Test, 475 J/Blow)
Water level at time of investigation.
Benchmark
Fill
Organics
Topsoil
Till
Bedrock
Sand
Silt
Clay
Sand & Gravel
Boulders
LICENSED PROFESSIONAL ENGINEER
M. W. BO
100129834
PROVINCE OF ONTARIO
DST
consulting engineers
DRAWING 3


METRIC
CONT No 6012-E-0047
GWP No 6210-10-00
GeoCres No 52A-153
GEOTECHNICAL INVESTIGATION
HIGHWAY 61
Crooks Township
OPTION 1
SHEET
DIST THUNDER BAY
Site Location
HWY 61
KEY PLAN
0
400
SCALE IN METRES
HIGHWAY 61
23+075
23+100
23+125
23+150
BH1
BH2
BH4
BH5
HA2
Concrete Culvert
1.22 m x 1.22 m x 39.3 m
BM
208.92 m
STA.23+157 13.
PLAN VIEW: Metres
Profile Along Station 23+125
(OPTION 1)
Elevation
Asphalt at Surface
Fill Sand, with Gravel, to Gravelly, compact
Clay - Silty, soft to firm
Silt - Sandy, very loose to loose
Clay & Silt - very soft
Clay - Silty, soft
ROCKFILL
GEOTEXTILE
Variable to Maximum of 19.0 m
EL. 205.5
EL. 203.5 or original ground level
2.0 m
1.25
LEGEND
Borehole (DST 2013)
Borehole (DST 2011)
Dynamic Cone Penetration Test (DCPT)
Rock Probe
'N' Blows/0.3m (Std. Pen Test, 475 J/Blow)
Water level at time of investigation.
Benchmark
Fill
Organics
Topsoil
Till
Bedrock
Sand
Silt
Clay
Sand & Gravel
Boulders
No.
Elevation
Northing
Easting
Station
Offset
LICENSED PROFESSIONAL ENGINEER
D. BANSAL
100178485
PROVINCE OF ONTARIO
LICENSED PROFESSIONAL ENGINEER
M. W. BO
100129834
PROVINCE OF ONTARIO
DST
consulting engineers
DST Consulting Engineers Inc.
DRAWING 4

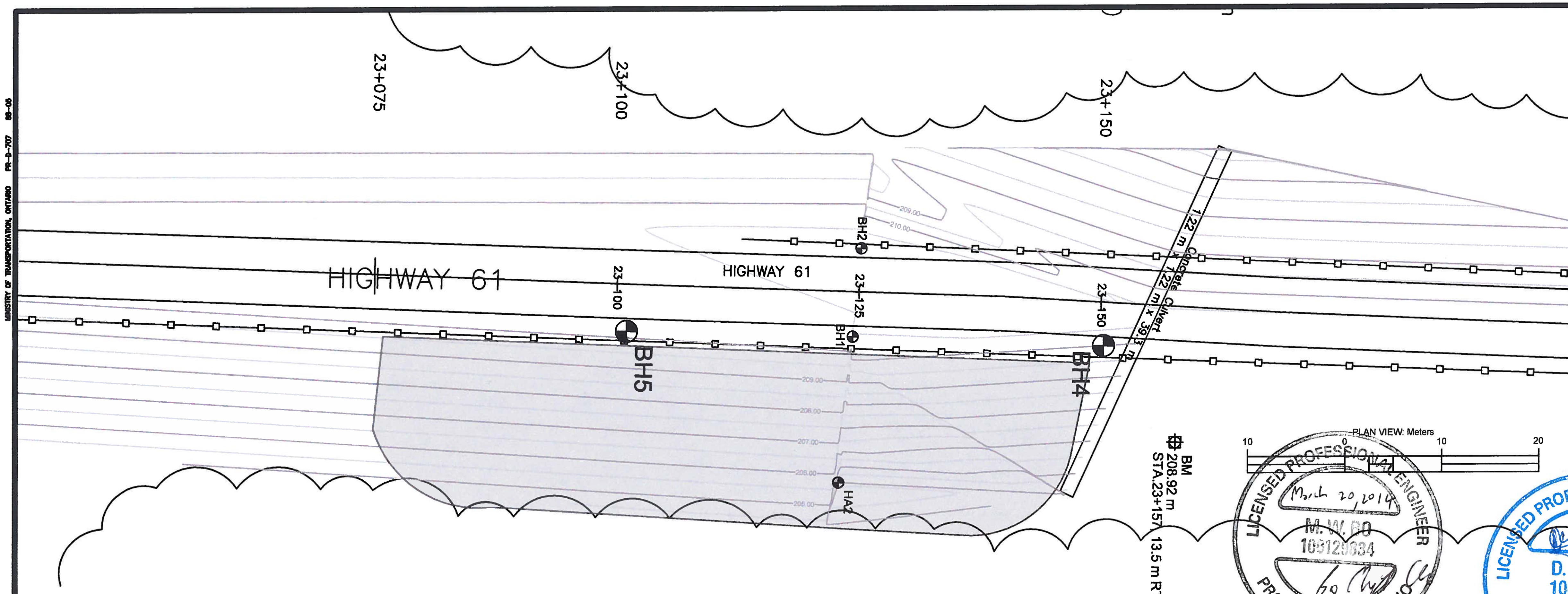
METRIC

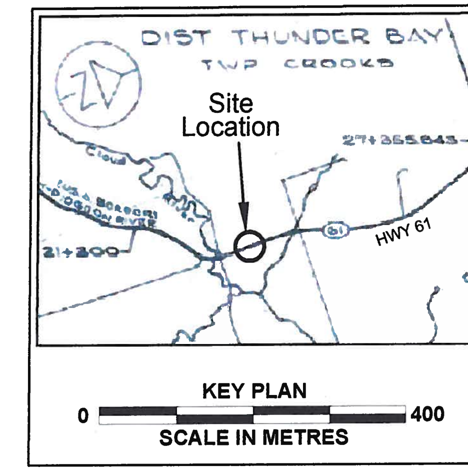
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETERS + METERS

CONT No 6014–XXXX
GWP No 6210–10–00
GeoCres No 52A–153

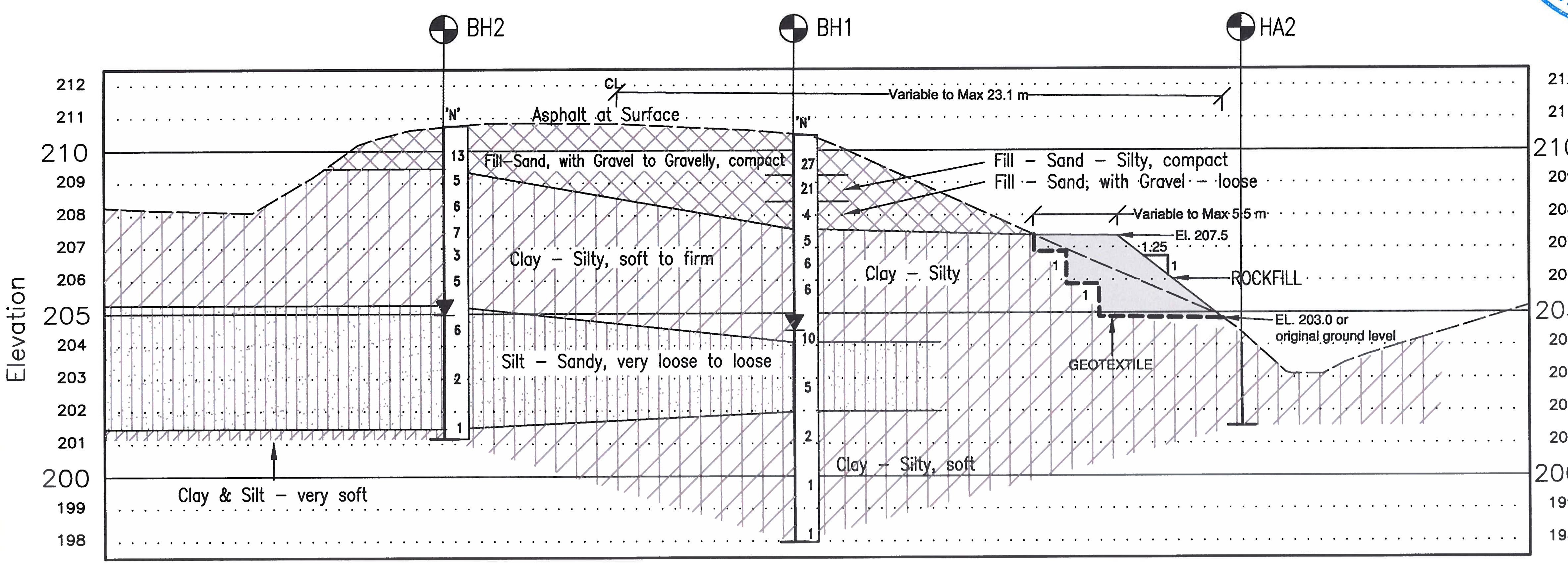
GEOTECHNICAL INVESTIGATION HIGHWAY 61 Crooks Township (Option 2)

SHEET 45





Profile Along Station 23+125



LICENSED PROFESSIONAL ENGINEER M. W. BO 100129334 PROVINCE OF ONTARIO March 20, 2014

LICENSED PROFESSIONAL ENGINEER D. BANSAL 100178485 PROVINCE OF ONTARIO March 20, 2014

LEGEND

- Borehole (DST 2013)
- Borehole (DST 2011)
- Dynamic Cone Penetration Test (DCPT)
- Rock Probe
- 'N' Blows/0.3m (Std. Pen Test, 475 J/Blow)
- Water level at time of investigation.
- Benchmark
- Fill
- Organics
- Topsoil
- Till
- Bedrock
- Sand
- Silt
- Clay
- Sand & Gravel
- Boulders

No.	Elevation	Northing	Easting	Station	Offset
BH1	210.47	5332723	317611	23+125	4.6 m RT
BH2	210.76	5332724	317602	23+125	4.6 m LT
BH3	210.60	5332751	317603	23+150	4.6 m LT
HA1	204.02	5332755	317626	23+150	16.0 m RT
HA2	204.01	5332722	317619	23+125	16.0 m RT
BH4	210.60	5332751	317614	23+150	4.6 m RT
BH5	210.60	5332702	317611	23+100	4.5 m RT

COORDINATE REFERENCE: MTM NAD83 ZONE 16

NOTE: *The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed by interpolation and may not represent actual conditions.*

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DRAWING 5

A P P E N D I X 'C'

STABILITY ANALYSIS RESULTS

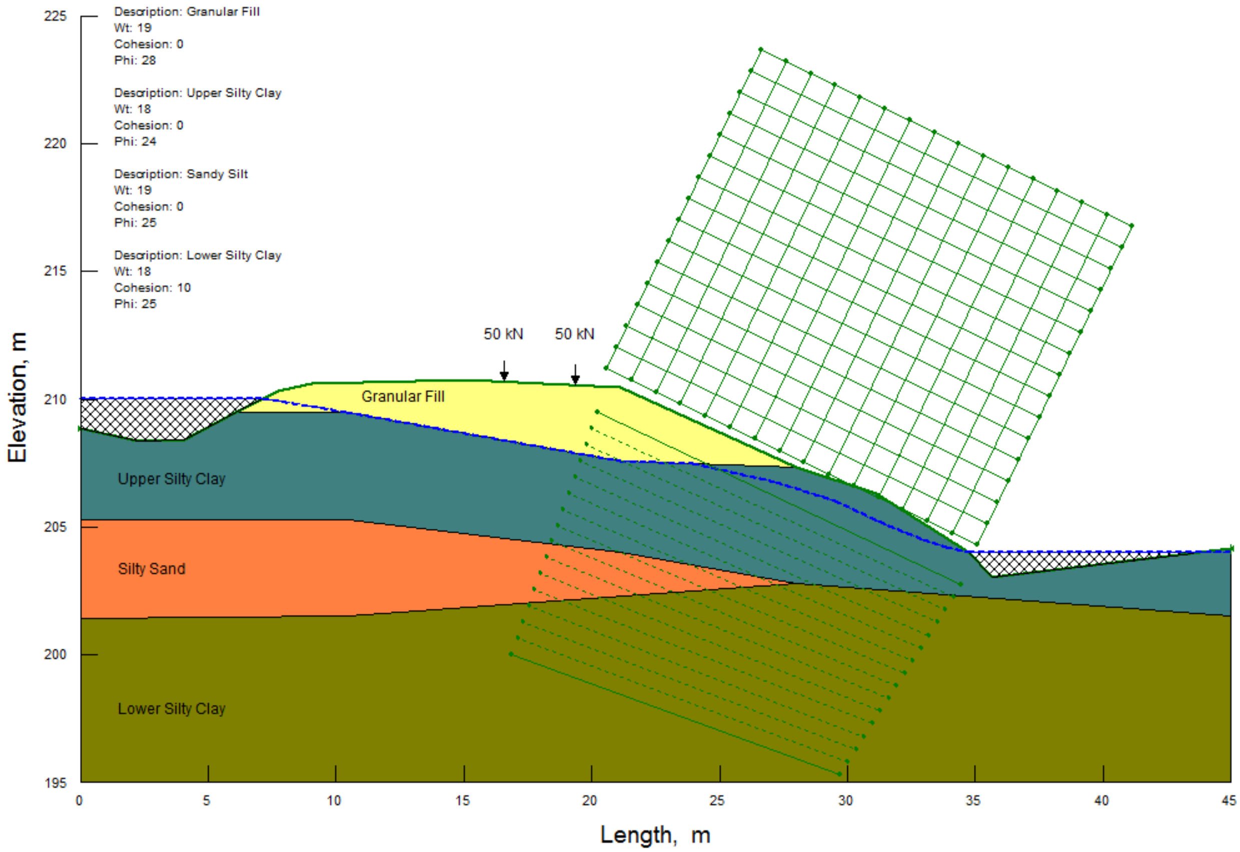


Figure C.1 Existing embankment, profile with high water

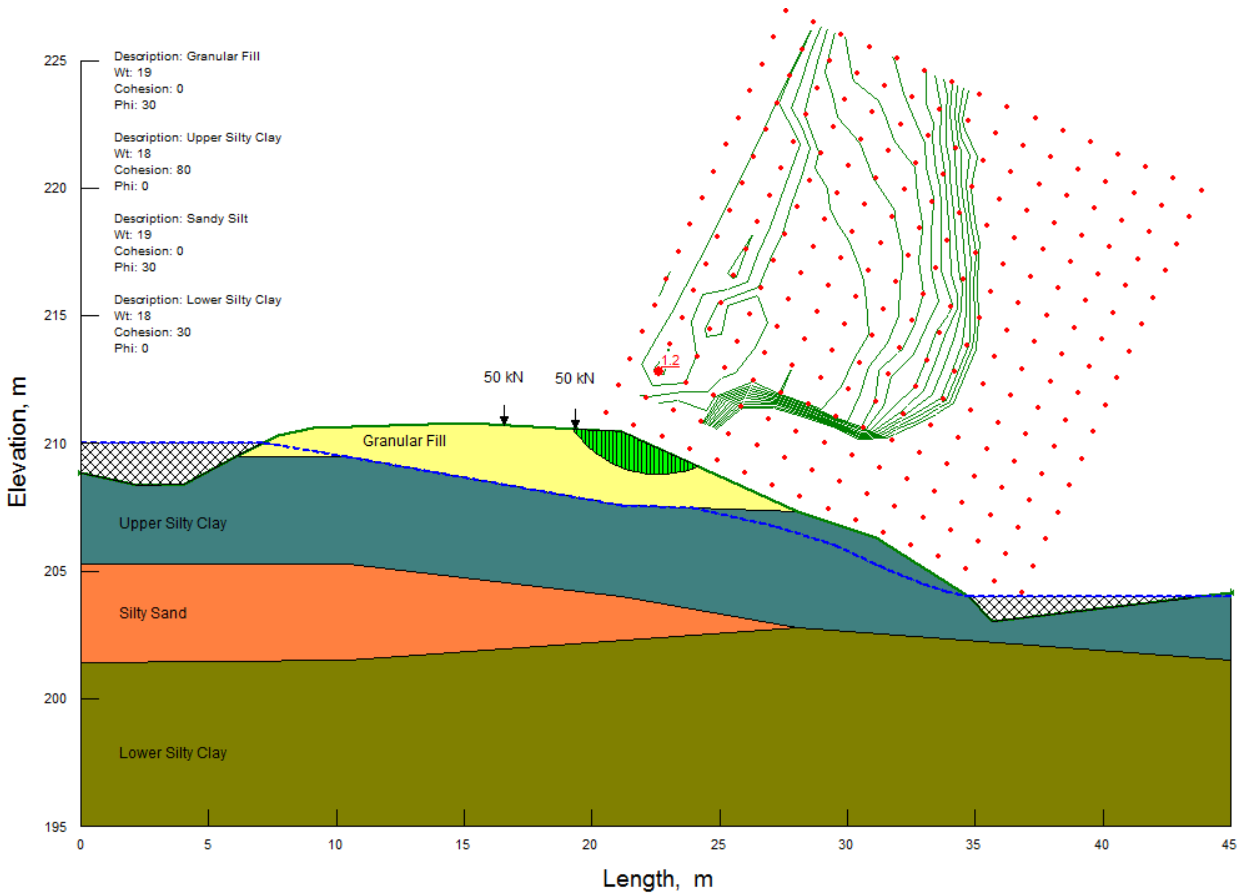


Figure C.2 Existing embankment, total stress analysis with high water

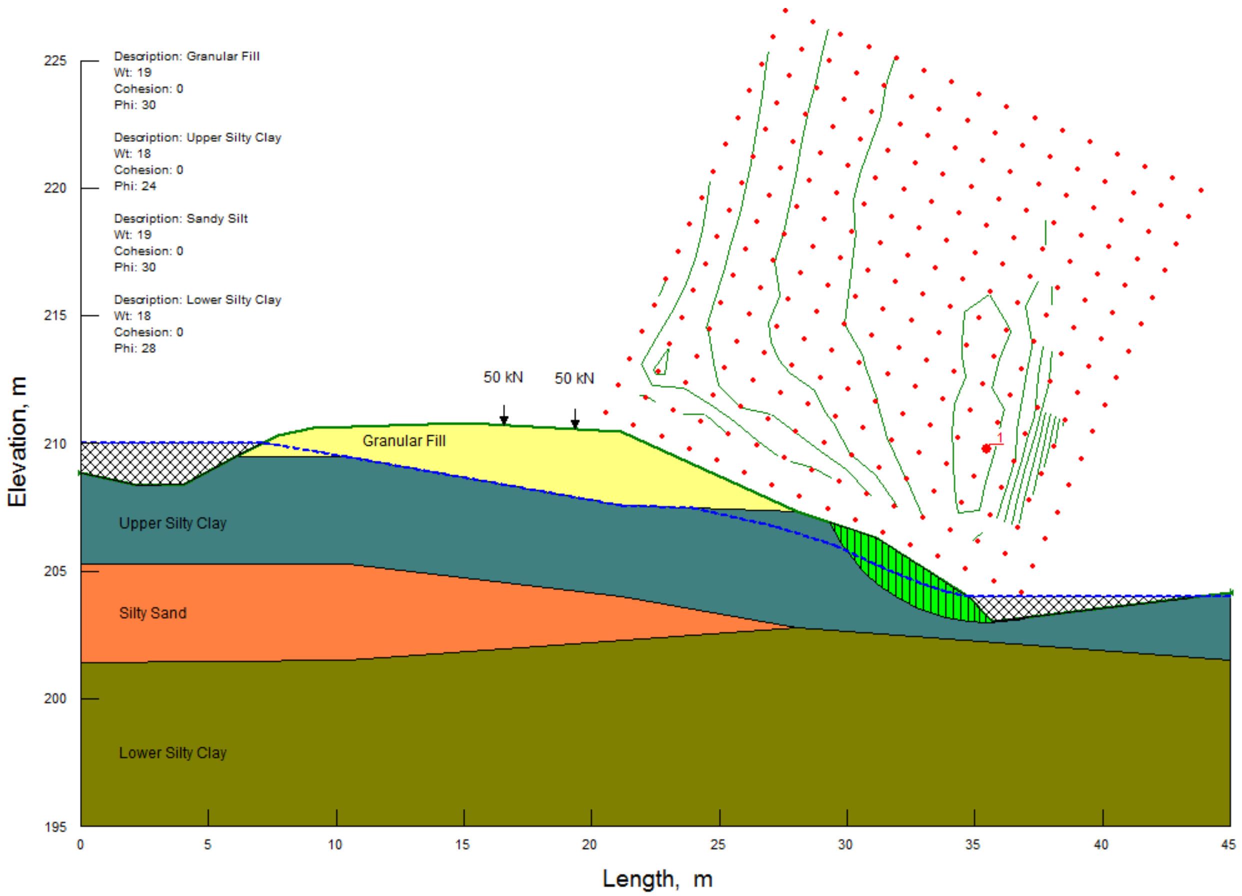


Figure C.3 Existing embankment, effective stress analysis with high water

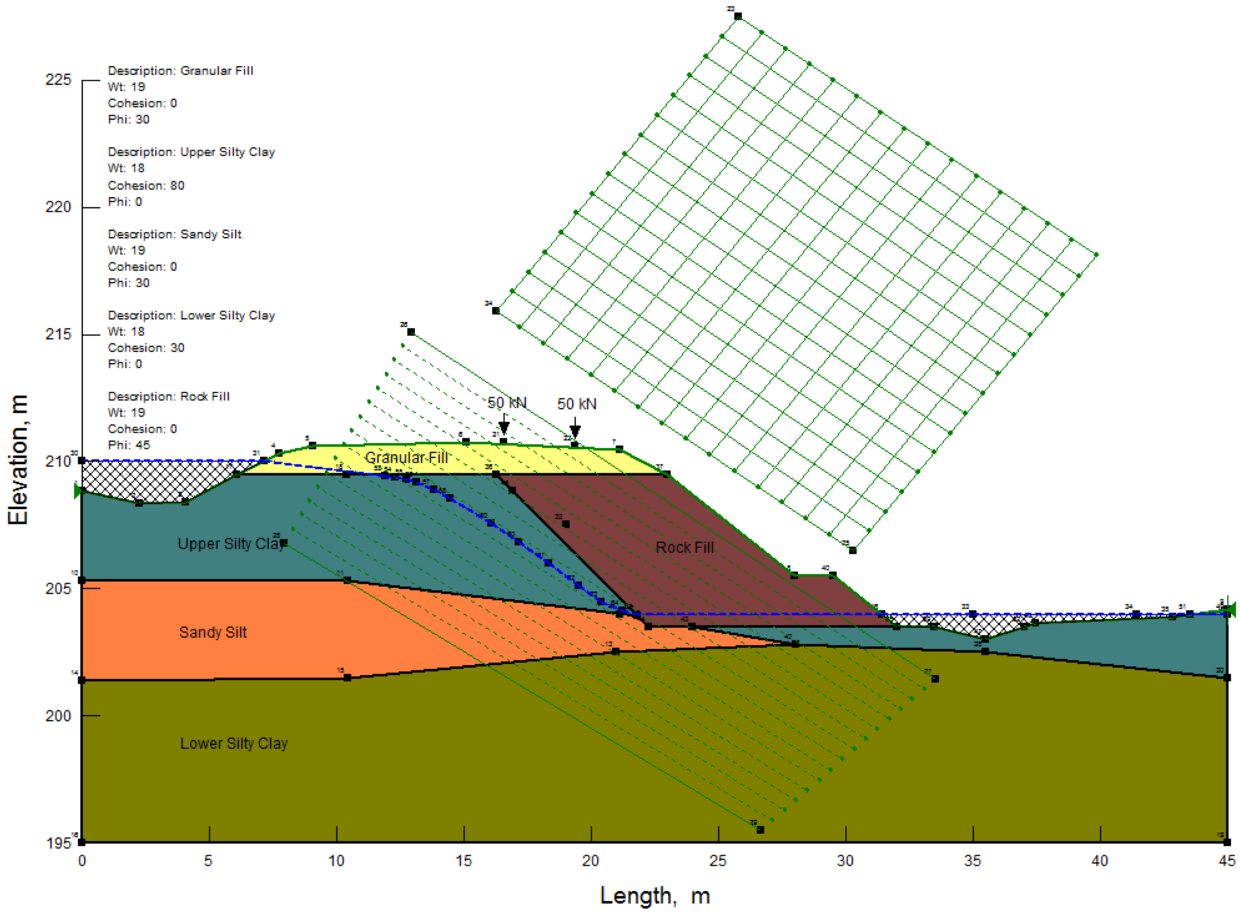


Figure C.4 Steepened rock fill embankment with counterweight berm, profile with high water

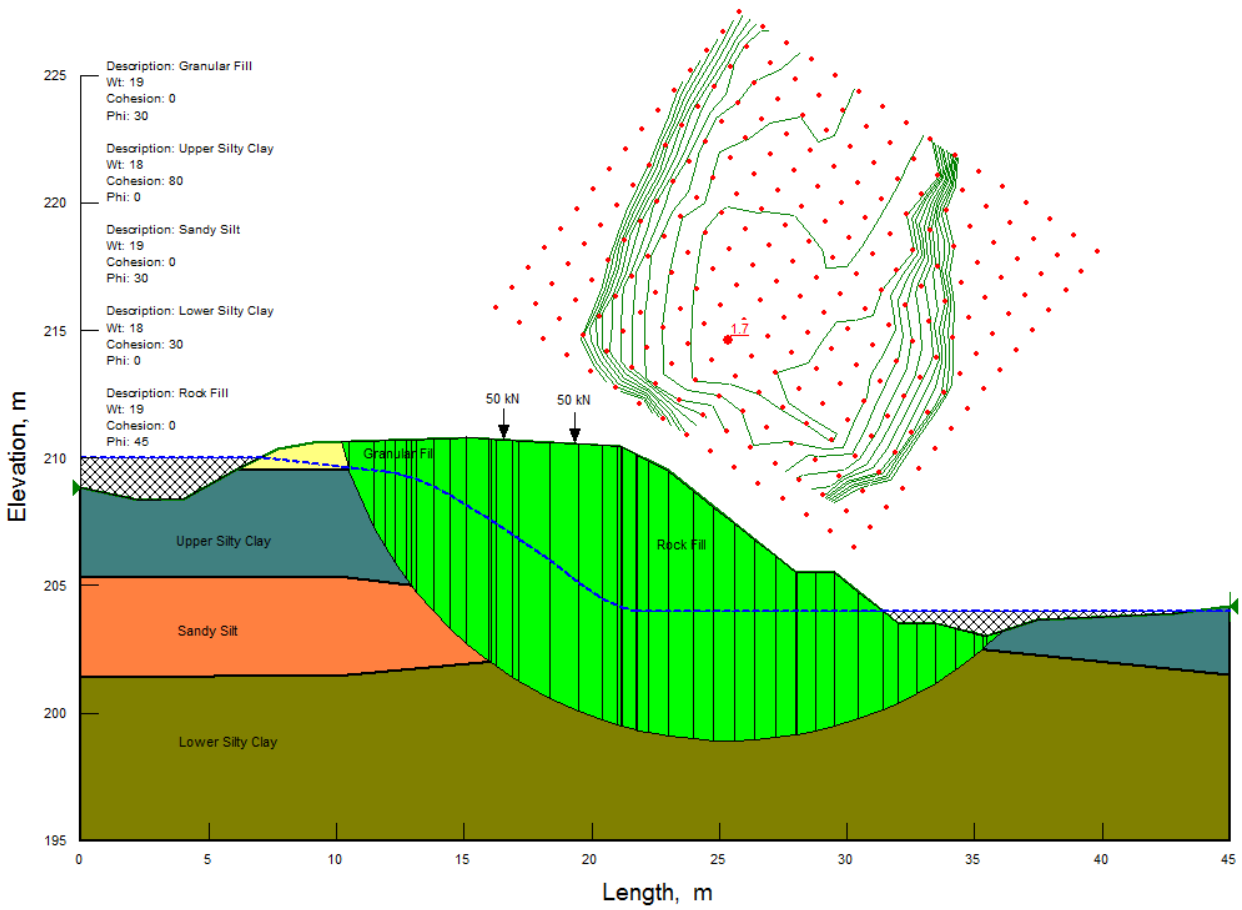


Figure C.5 Steepened rock fill embankment with counterweight berm, total stress analysis with high water

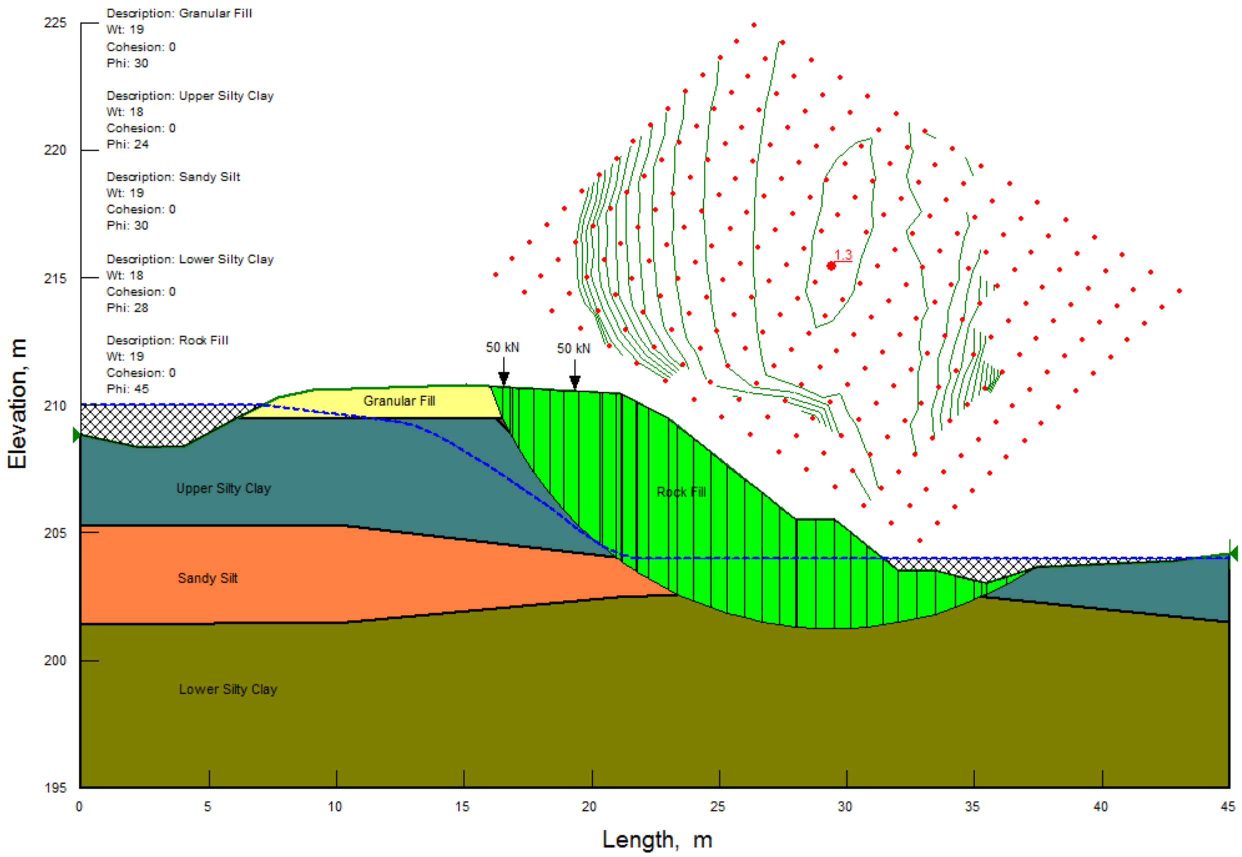


Figure C.6 Steepened rock fill embankment with counterweight berm, effective stress analysis with high water

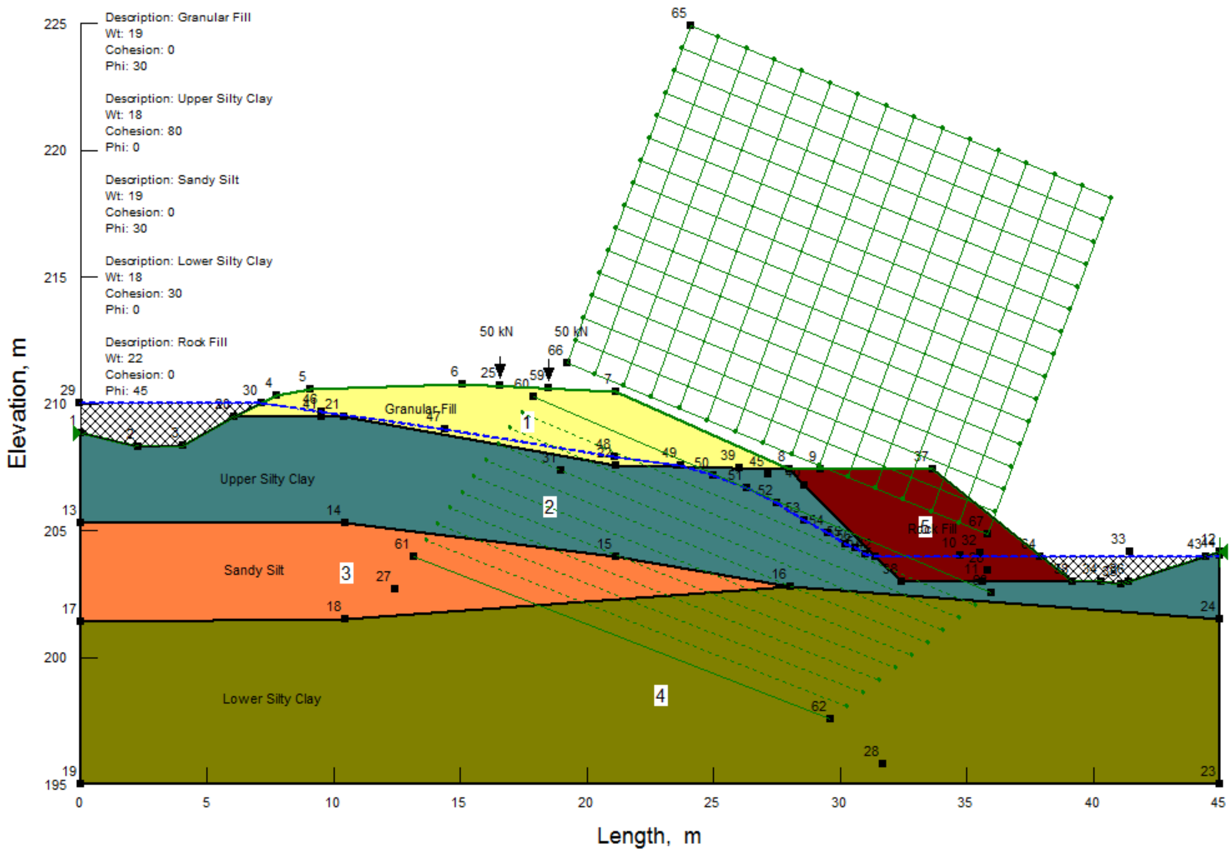


Figure C.7 Existing embankment with counterweight berm, profile with high water

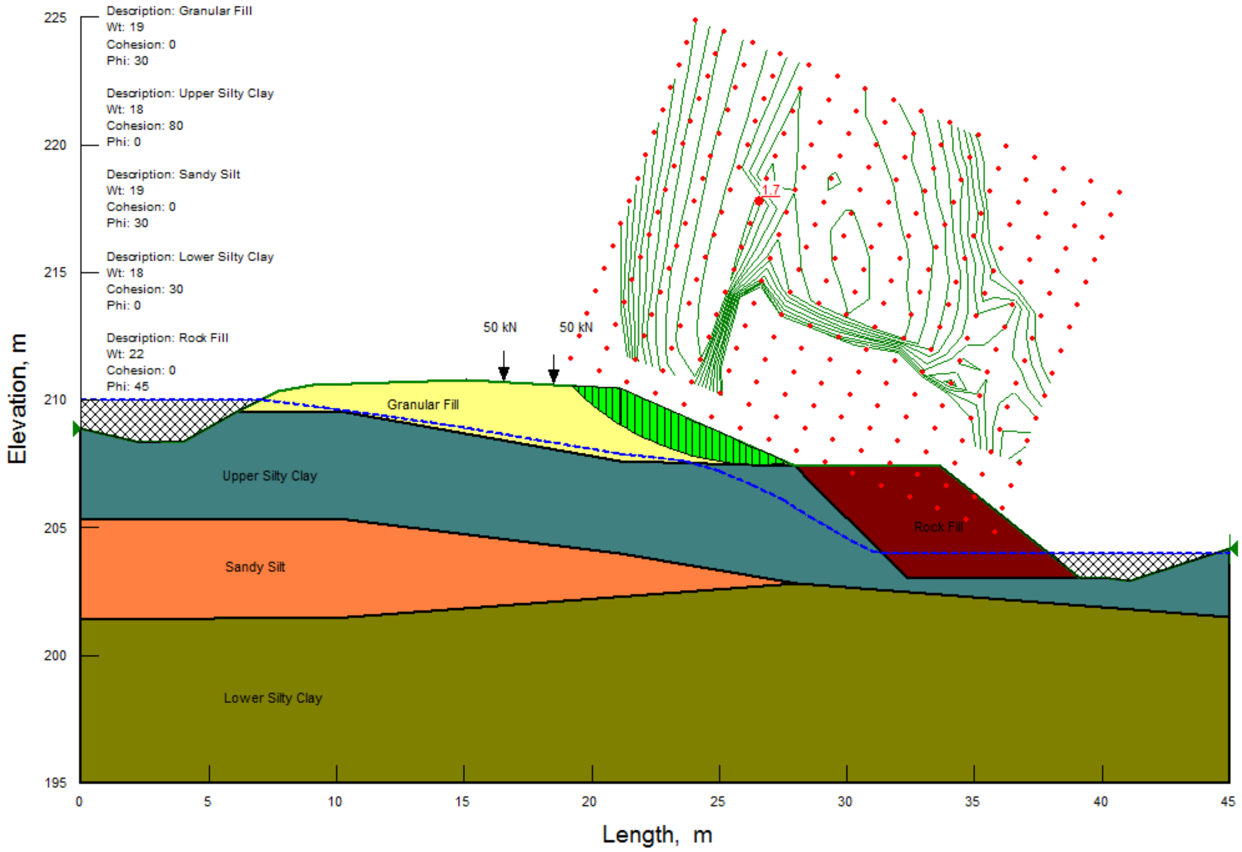


Figure C.8 Existing embankment with counterweight berm, total stress analysis with high water

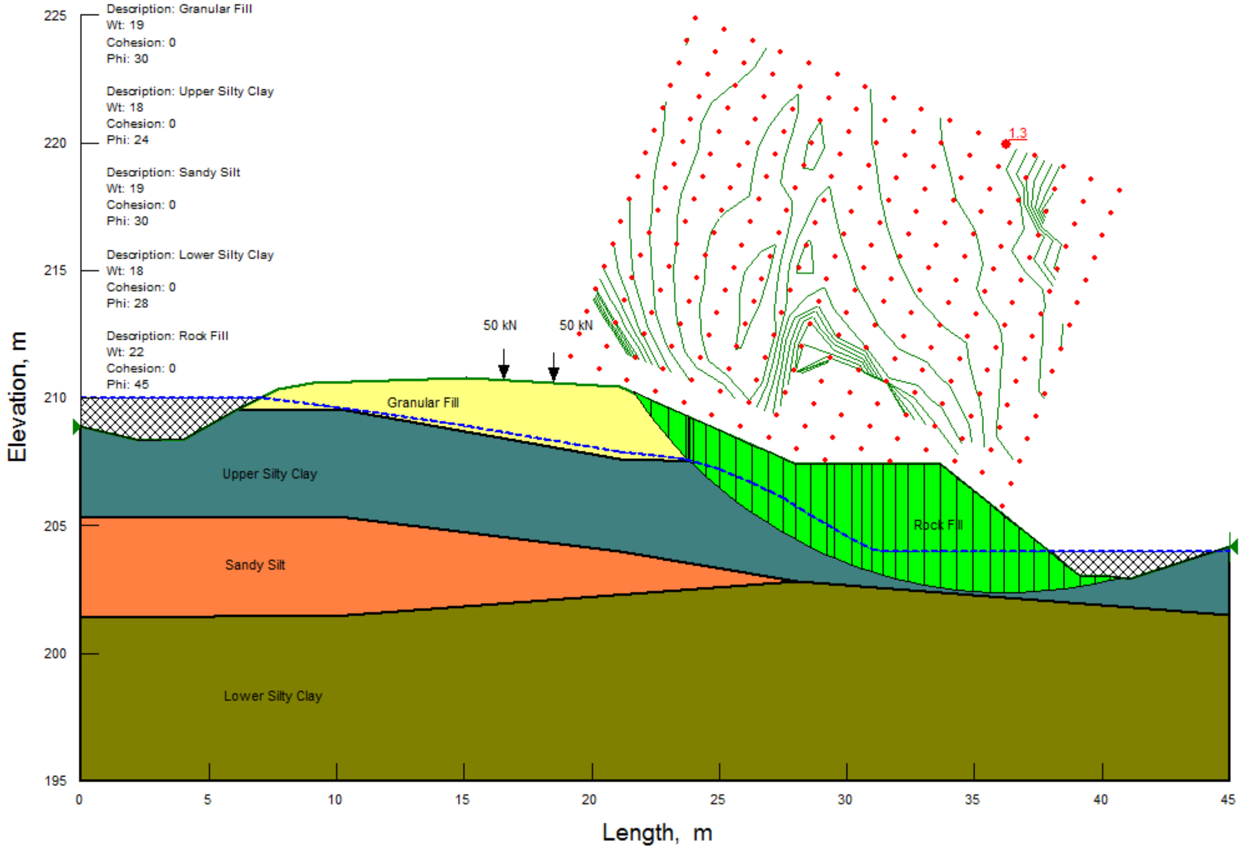


Figure C.9 Existing embankment with counterweight berm, effective stress analysis with high water

A P P E N D I X ‘D’

EXPLANATION OF TERM USED IN REPORT

EXPLANATION OF TERMS USED IN REPORT

SPT 'N' VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE OF THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51 mm O.D. SPLIT BARREL SAMPLES TO PENETRATE 0.3 m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5 kg, FALLING FREELY A DISTANCE OF 0.76 m. FOR PENETRATION OF LESS THAN 0.3 m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS Ñ.

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS

TEXTURAL CLASSIFICATION OF SOILS

BOULDERS	COBBLES	GRAVEL	SAND	SILT	CLAY
GREATER THAN 200 mm	75 TO 200 mm	4.75 TO 75 mm	0.075 TO 4.75 mm	0.002 TO 0.075 mm	LESS THAN 0.002 mm

COARSE GRAIN SOIL DESCRIPTION (50% GREATER THAN 0.075 mm)

TERMINOLOGY	TRACE OR OCCASIONAL	SOME	WITH	ADJECTIVE (e.g. SILTY OR SANDY)	AND (e.g. SAND AND SILT)
	LESS THAN 10%	10 TO 20%	20 TO 30%	30 TO 40%	40 TO 60%

CONSISTENCY*: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (C_U) AND SPT 'N' VALUES AS FOLLOWS

C_U (kPa)	0 – 12	12 – 25	25 – 50	50 - 100	100 - 200	> 200
N (BLOWS / 0.3 m)	<2	2 - 4	4 - 8	8 - 15	15 - 30	>30
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

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

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

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

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

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100 mm+ IN LENGTH EXPRESSED AS A PERCENTAGE OF THE LENGTH OF THE CORING RUN.

THE ***ROCK QUALITY DESIGNATION*** *(R.Q.D)* FOR MODIFIED RECOVERY IS:

R.Q.D (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

LEGEND OF RECORDS FOR BOREHOLES: SYMBOLS AND ABBREVATIONS FOR SAMPLE TYPE

SS	SPLIT SPOON SAMPLE	WS	WASH SAMPLE
TW	THIN WALL SHELBY TUBE SAMPLE	AS	AUGER (GRAB) SAMPLE
PH	SAMPLER ADVANCED BY HYDRAULIC PRESSURE	TP	THIN WALL PISTON SAMPLE
WH	SAMPLER ADVANCED BY SELF STATIC WEIGHT	PM	SAMPLER ADVANCED BY MANUAL PRESSURE
SC	SOIL CORE	RC	ROCK CORE
▼	WATER LEVEL	$SENSITIVITY = \frac{UNDISTURBED\ SHEAR\ STRENGTH}{REMOLDED\ SHEAR\ STRENGTH}$	

*HIERARCHY OF SOIL STRENGTH PREDICTION: **1)** LABORATORY TRIAXIAL TESTING. **2)** FIELD INSITU VANE TESTING. **3)** LABORATORY VANE TESTING. **4)** SPT VALUES. **5)** POCKET PENETROMETER.

E N C L O S U R E S

RECORD OF BOREHOLE No BH1

METRIC

W.P. 6009-E-0005 LOCATION STA. 23+125 - 4.6 m RT, 5332723 m N, 317609 m E ORIGINATED BY JF/RW

DIST HWY 61 BOREHOLE TYPE Hollow Stem Auger (80 mm ID) COMPILED BY ML

DATUM DATE 2011 05 27 CHECKED BY WS/BV

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
210.5	GROUND SURFACE					
210.4 / 0.1	ASPHALT - 100 mm	AS1	AS			Water level at 6.0 m on completion
	FILL - SAND & CRUSHED GRAVEL - trace silt, brown, compact					37 53 (10)
		SS2	SS	27		
209.2 / 1.3	FILL - SAND - Silty, trace gravel, brown, compact	SS3	SS	21		6 70 (24)
208.4 / 2.1	FILL - SAND & GRAVEL - trace silt and clay, brown, loose	SS4	SS	4		
207.6 / 2.9	CLAY - Silty, trace sand, gravel and organics, brown/grey, firm, high plasticity	SS5	SS	5		
		SS6	SS	6		
		SS7	SS	6	∇	
204.1 / 6.4	SILT - Sandy, grey, loose	SS8	SS	10		
		SS9	SS	5		
202.0 / 8.5	CLAY - Silty, trace sand, grey, very soft, low plasticity	SS10	SS	2		
		SS11	SS	1		
		SS12	SS	1		
198.0 / 12.5	End of Borehole at 12.5 m					

STRAT PLOT | ELEVATION SCALE: 210, 209, 208, 207, 206, 205, 204, 203, 202, 201, 200, 199, 198

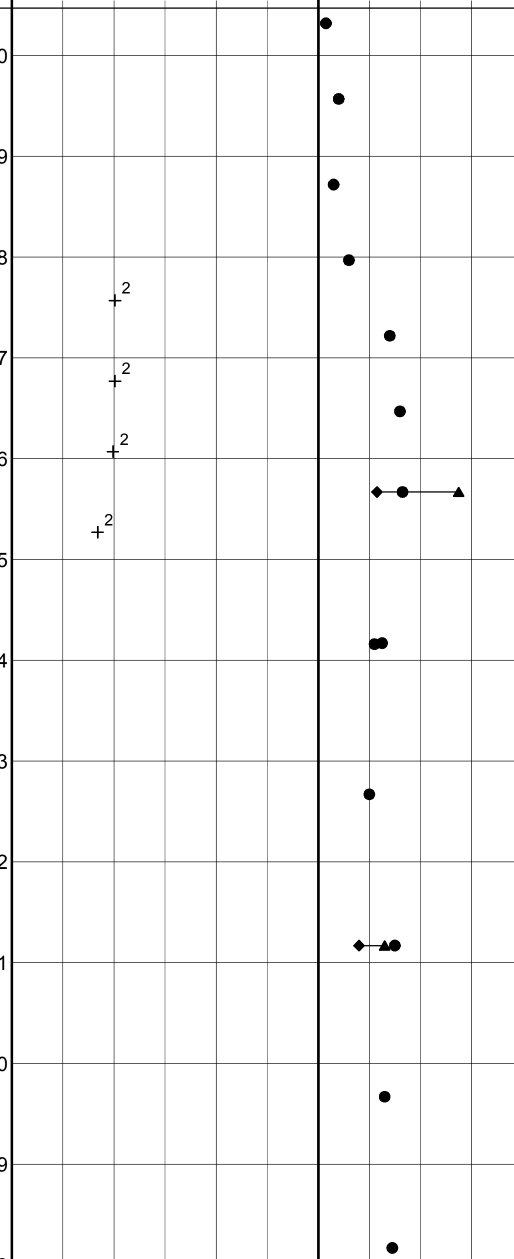
DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE □ QUICK TRIAXIAL × LAB VANE — 50 100 150 200 250

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³



+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH2

METRIC

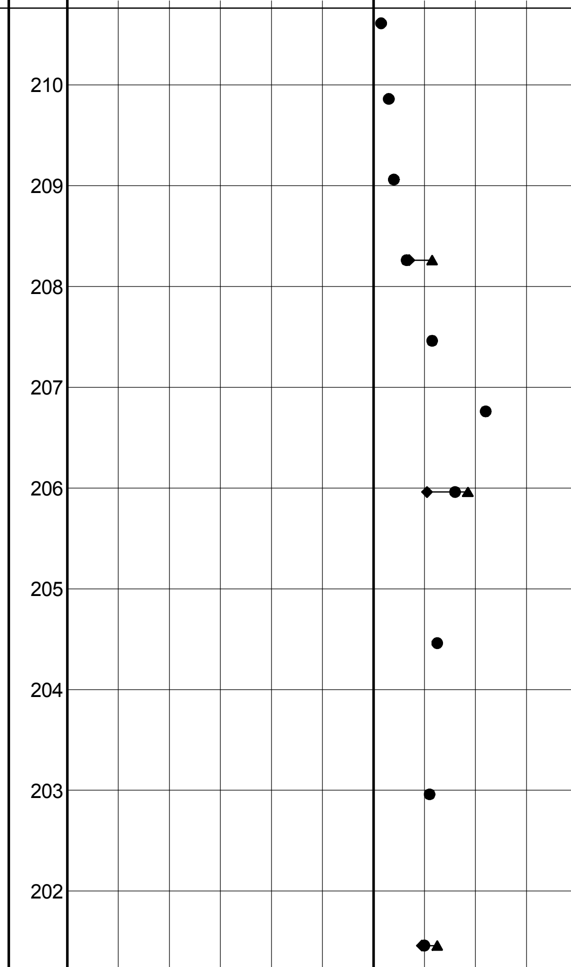
W.P. 6009-E-0005 LOCATION STA. 23+125 - 4.6 m LT, 5332751 m N, 317603 m E ORIGINATED BY JF/RW

DIST HWY 61 BOREHOLE TYPE Hollow Stem Auger (80 mm ID) COMPILED BY ML

DATUM DATE 2011 05 30 CHECKED BY WS/BV

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
210.8	GROUND SURFACE					
210.7 / 0.1	ASPHALT - 100 mm	AS1	AS			Water level at 5.8 m on completion
	FILL - SAND & CRUSHED GRAVEL - trace silt, brown, compact	SS2	SS	13		
209.5 / 1.3	CLAY - Silty, trace sand and gravel, brown, soft to firm, low to intermediate plasticity	SS3	SS	5		
		SS4	SS	6		
		SS5	SS	7		
	- - - - - trace organics	SS6	SS	3		
		SS7	SS	5		
205.3 / 5.5	SILT - Sandy, grey, very loose to loose				∇	
		SS8	SS	6		0 34 (66)
		SS9	SS	2		
201.5 / 9.3	CLAY & SILT - trace sand, grey, very soft, low plasticity	SS10	SS	1		0 3 (97)
201.2 / 9.6	End of Borehole at 9.6 m					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, □ QUICK TRIAXIAL, × LAB VANE (50 100 150 200 250); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL).



+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ON_MOT-HIGH VANES GS-TB-013228 MTO - CULVERTS HWY 61.GPJ DST_MIN.GDT 12/17/13

Ministry of Transportation Ontario

RECORD OF BOREHOLE No BH3

METRIC

W.P. 6009-E-0005 LOCATION STA. 23+150 - 4.6 m LT, 5332726 m N, 317600 m E ORIGINATED BY JF/RW

DIST HWY 61 BOREHOLE TYPE Hollow Stem Auger (80 mm ID) COMPILED BY ML

DATUM DATE 2011 05 30 CHECKED BY WS/BV

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
210.6	GROUND SURFACE					
210.5 / 0.1	ASPHALT - 100 mm	AS1	AS			27 51 (21)
	FILL - SAND & CRUSHED GRAVEL - trace silt, brown					Water level at 6.2 m on completion
210.0 / 0.6	FILL - SAND - Silty, trace gravel, brown, loose to compact				210	
		SS2	SS	8		
		SS3	SS	10	209	5 64 (31)
208.6 / 2.0	CLAY - Silty, trace sand and gravel, brown, firm to very stiff, high plasticity					
		SS4	SS	7	208	
		SS5	SS	5	207	
		SS6	SS	7		
	---- - trace organcis	SS7	SS	12	206	
					205	
		SS8	SS	19	204	
203.6 / 7.0	SILT - Sandy, grey, very loose to loose					
		SS9	SS	7	203	
					202	
		SS10	SS	2	201	0 23 (77)
		SS11	SS	2	200	0 20 (80)
199.3 / 11.3	CLAY & SILT - trace sand, grey, soft, low plasticity				199	
		SS12	SS	1		
198.1 / 12.5	End of Borehole at 12.5 m					

Plot headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (50, 100, 150, 200, 250): ○ UNCONFINED, + FIELD VANE, □ QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20, 40, 60; UNIT WEIGHT γ kN/m^3; GROUND WATER CONDITIONS; STRAT PLOT.

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH4

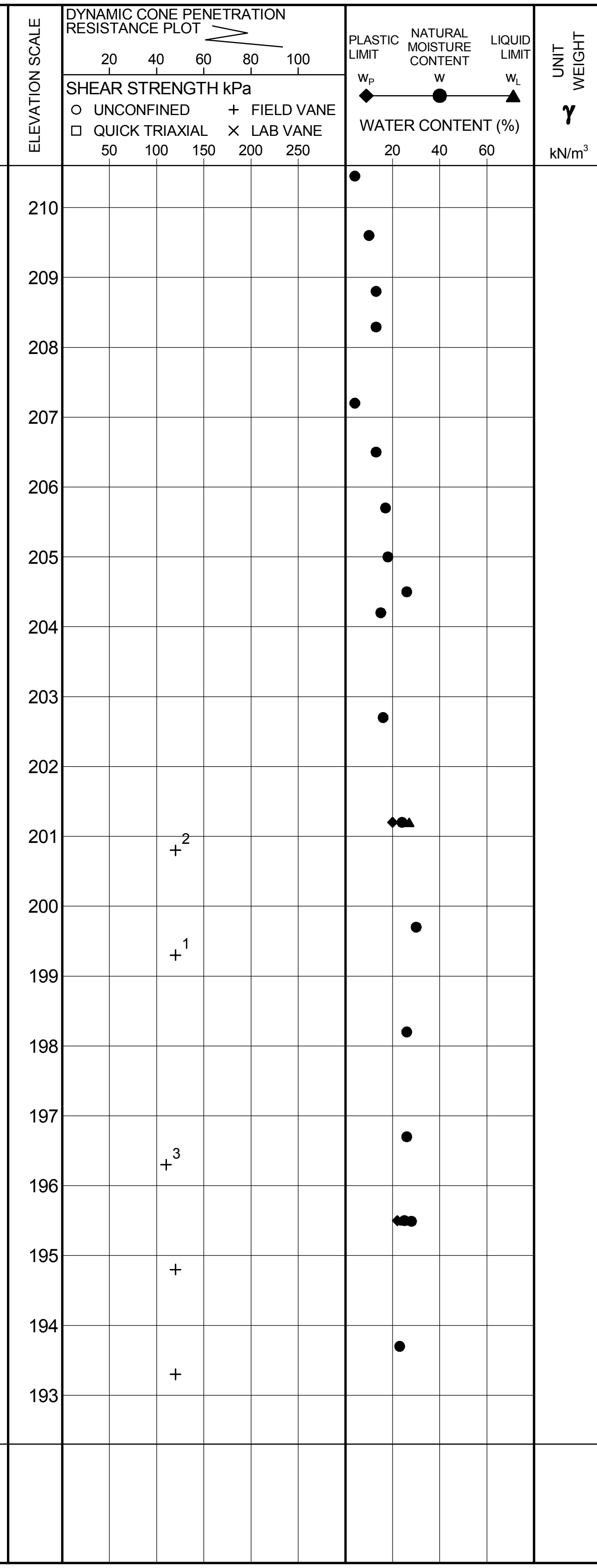
METRIC

W.P. 6012-E-0047 | LOCATION STA. 23+150, 4.6 m RT (UTM 16U - 5332751 m N, 317614 m E) | ORIGINATED BY JS

DIST ___ HWY 61 | BOREHOLE TYPE Hollow Stem Auger (80 mm ID) | COMPILED BY ML

DATUM Local | DATE 2013 11 13 | CHECKED BY WS

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
210.6	GROUND SURFACE					
210.6 / 0.0	ASPHALT - 40 mm	AS1	AS			Water level at 8.0 m on completion
	FILL - SAND & CRUSHED GRAVEL - trace silt, brown					
209.9 / 0.8	FILL - SAND - some silt, trace gravel, brown, loose	SS2	SS	5		
		SS3	SS	7		
208.1 / 2.5	FILL - SAND & CRUSHED GRAVEL - some silt, grey, compact	SS4	SS	13		
		SS5	SS	18		
206.8 / 3.8	FILL - SAND - some silt, trace clay and gravel, brown, compact	SS6	SS	14		7 74 (19)
		SS7	SS	11		
205.3 / 5.3	FILL - CLAY - Silty, trace sand and gravel and organics, brown, very stiff	SS8	SS	15		
204.3 / 6.3	FILL - SAND - with gravel, some silt, trace organics, brown/black, compact	SS9	SS	23		
203.5 / 7.1	CLAY - Silty, trace sand and gravel, brown/grey, firm	SS10	SS	4	∇	
202.0 / 8.6	SILT - Sandy, trace gravel and clay, grey, loose	SS11	SS	4		1 30 (69)
201.0 / 9.6	CLAY - Silty, trace sand, brown, very stiff					0 0 70 30
		SS12	SS	2		
		SS13	SS	4		
197.5 / 13.1	SILT - some clay, grey, very stiff	ST14	ST			
		SS15	SS	6		0 0 87 13
195.0 / 15.6	CLAY - Silty, brown, very stiff	SS16	SS	17		
192.3 / 18.3	End of Borehole at 18.3 m					



+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No BH5

METRIC

W.P. 6012-E-0047 LOCATION STA. 23+100, 4.5 m RT (UTM 16U - 5332702 m N, 317611m E) ORIGINATED BY JS

DIST HWY 61 BOREHOLE TYPE Hollow Stem Auger (80 mm ID) COMPILED BY ML

DATUM Local DATE 2013 11 13 CHECKED BY WS

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
210.6	GROUND SURFACE					Water level at 7.6 m on completion
210.4 / 0.2	ASPHALT - 200 mm					
0.2	FILL - SAND & CRUSHED GRAVEL - trace silt, brown, compact	AS1	AS			
		SS2	SS	17		
209.1 / 1.5	FILL - SAND - some silt, trace gravel, brown, loose	SS3	SS	6		
208.2 / 2.4	CLAY - Silty, trace gravel, trace sand and organics, brown/grey, soft to stiff	SS4	SS	3		
		SS5	SS	3		
		SS6	SS	8		
		SS7	SS	9		
		SS8	SS	10		
204.3 / 6.3	SILT & SAND - trace clay and organics, brown/grey, very loose to loose	SS9	SS	7		
					∇ (203)	0 48 (52)
202.6 / 8.0	CLAY - Silty, trace sand and gravel, brown/grey, firm to very stiff	SS10	SS	3		
		SS11	SS	1		
		ST12	ST			
198.8 / 11.8	SILT - some clay, brown/grey, stiff	SS13	SS	1		0 1 89 10
197.3 / 13.3	CLAY - Silty, brown, stiff	SS14	SS	4		
		SS15	SS	11		0 0 74 26
194.1 / 16.5	End of Borehole at 16.5 m					

Plot columns: ELEVATION SCALE (210 to 195); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, □ QUICK TRIAXIAL, × LAB VANE; 50, 100, 150, 200, 250); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³.

Elevation (approx.)	Field vane (sensitivity)
200.8	+ 4
199.3	+ 2
197.6	+ 2
196.1	+ 3
194.6	+ 2

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ON_MOT-HIGH VANES GS-TB-016596 MTO #6012-E-0047 - HWY 61.GPJ DST_MIN.GDT 12/17/13

Ministry of Transportation Ontario

RECORD OF BOREHOLE No HA1

METRIC

W.P. 6009-E-0005 LOCATION STA. 23+150 - 16.0 m RT, 5332755 m N, 317626 m E ORIGINATED BY JF/RW

DIST HWY 61 BOREHOLE TYPE Hand Auger COMPILED BY ML

DATUM DATE 2011 06 02 CHECKED BY WS/BV

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (50 100 150 200 250)	WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
204.0	GROUND SURFACE										
204.0 / 0.0	TOPSOIL - 20 mm CLAY - Silty, trace sand, gravel and organics, occasional boulder, brown					∇	203 202				Water level at 1.6 m on completion
201.5 / 2.5	End of Borehole at 2.5 m										

SHEAR STRENGTH legend: ○ UNCONFINED, + FIELD VANE, □ QUICK TRIAXIAL, × LAB VANE

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L

+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ON_MOT-HIGH VANES GS-TB-013228 MTO - CULVERTS HWY 61.GPJ DST_MIN.GDT 12/17/13

RECORD OF BOREHOLE No HA2

METRIC

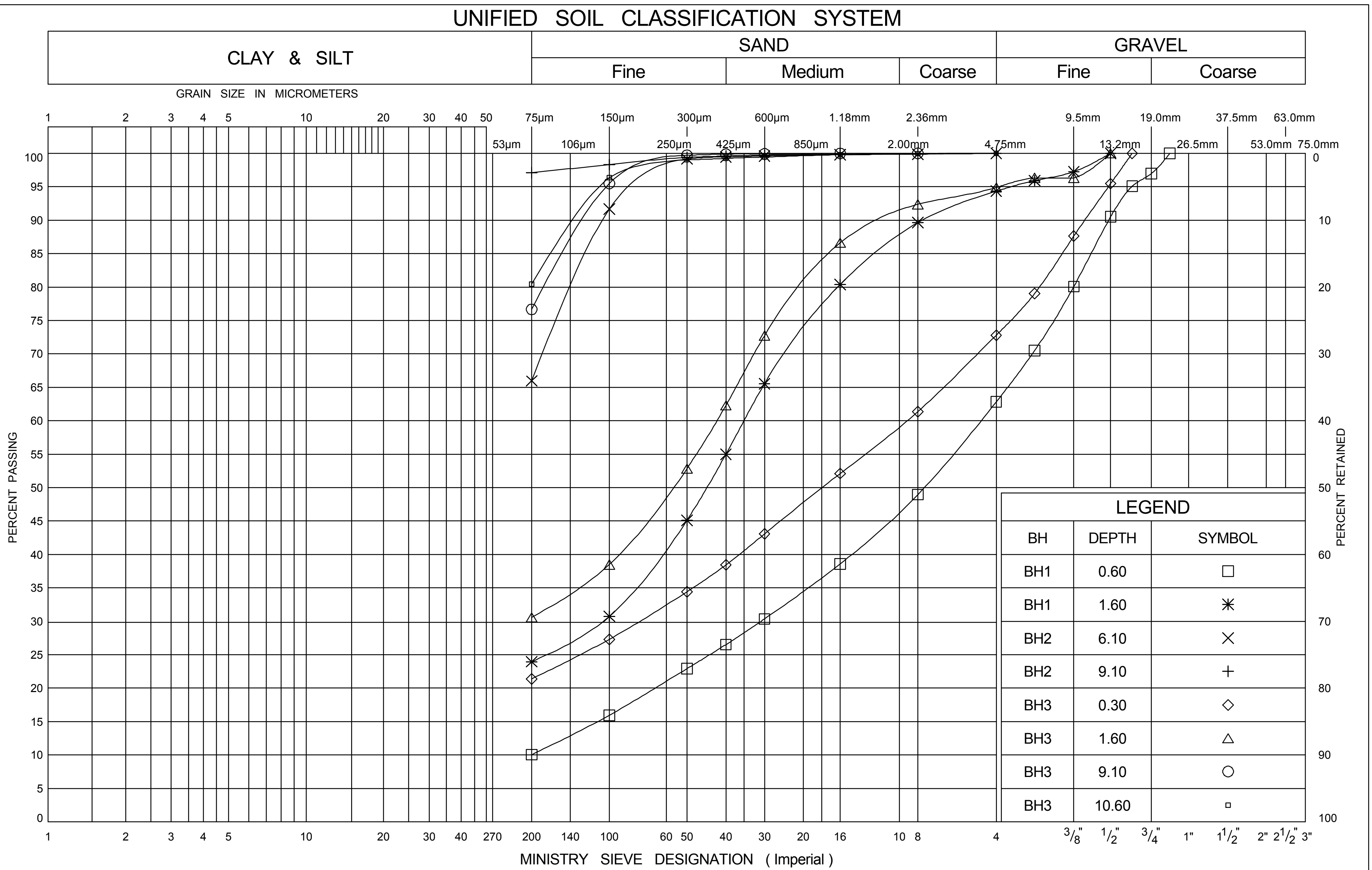
W.P. 6009-E-0005 LOCATION STA. 23+125 - 16.0 m RT, 5332722 m N, 317619 m E ORIGINATED BY JF/RW

DIST HWY 61 BOREHOLE TYPE Hand Auger COMPILED BY ML

DATUM DATE 2011 06 02 CHECKED BY WS/BV

ELEV / DEPTH	SOIL PROFILE - DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, □ QUICK TRIAXIAL, × LAB VANE) 50 100 150 200 250	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
204.0	GROUND SURFACE										
204.0 / 0.0	TOPSOIL - 40 mm										
	CLAY - Silty, trace sand, gravel and organics, brown/red						203				
							202				
201.5 / 2.5	End of Borehole at 2.5 m										

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE



BH	DEPTH	SYMBOL
BH1	0.60	□
BH1	1.60	✳
BH2	6.10	×
BH2	9.10	+
BH3	0.30	◇
BH3	1.60	△
BH3	9.10	○
BH3	10.60	▫

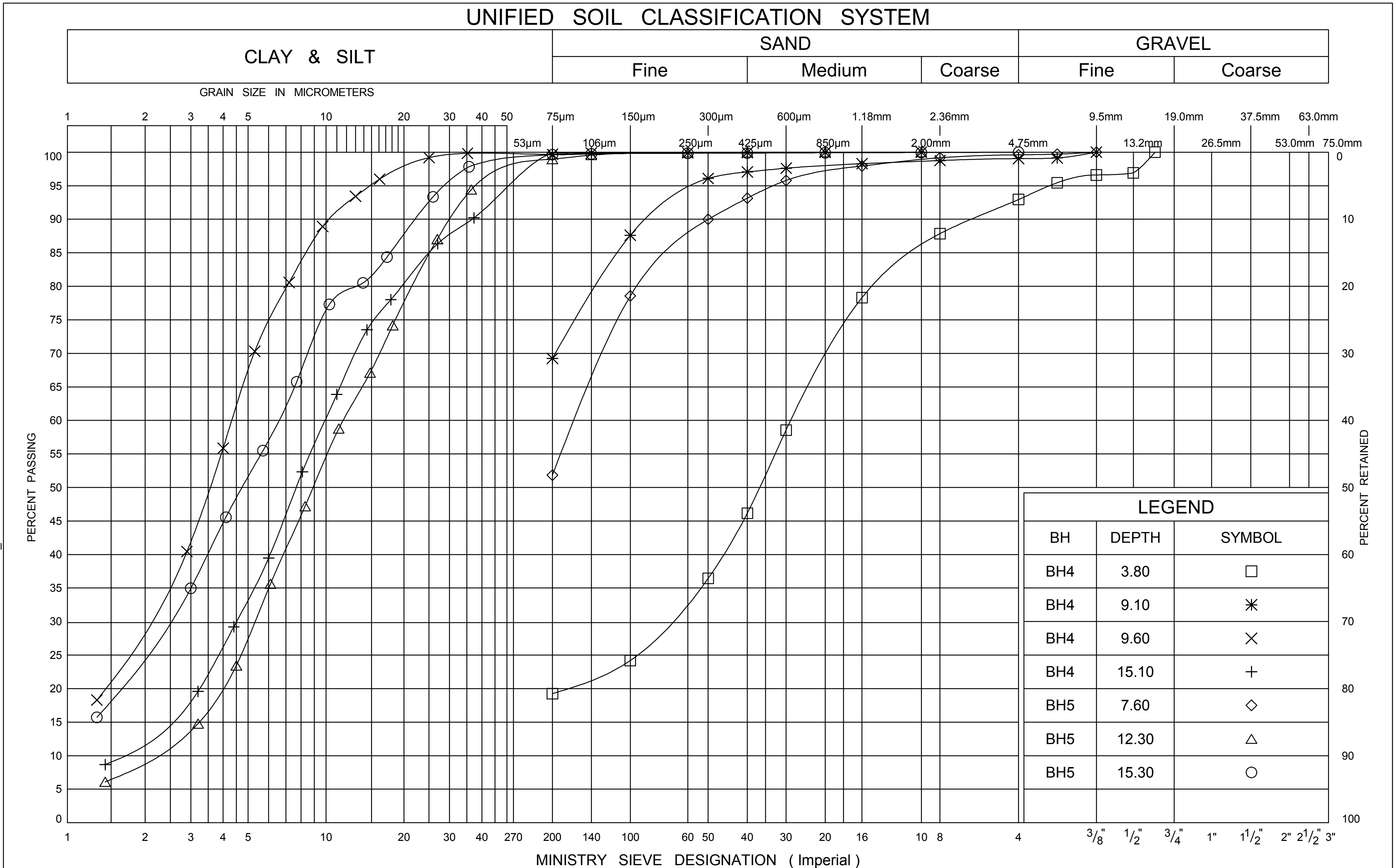
Ministry of Transportation Ontario

GRAIN SIZE DISTRIBUTION

ENCLOSURE 8

W P 6012-E-0047

HIGHWAY 61



GRAIN SIZE DISTRIBUTION

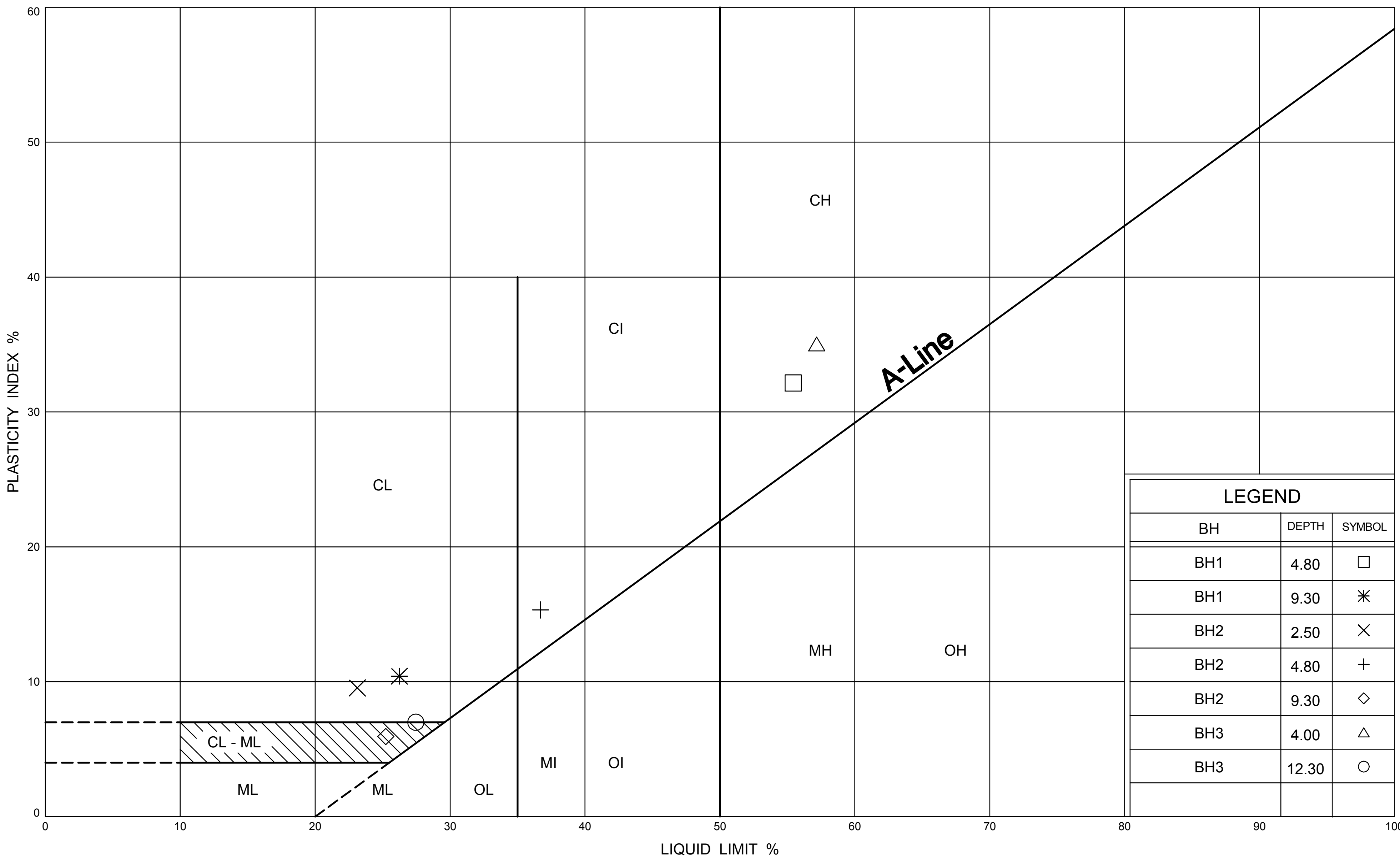
Ministry of Transportation
Ontario

ENCLOSURE 9

W P 6012-E-0047

HIGHWAY 61

ONTARIO MOT GRAIN SIZE GS-TB-016596 MTO #6012-E-0047 - HWY 61.GPJ DST_MIN.GDT 12/2/13



LEGEND		
BH	DEPTH	SYMBOL
BH1	4.80	□
BH1	9.30	✳
BH2	2.50	×
BH2	4.80	+
BH2	9.30	◇
BH3	4.00	△
BH3	12.30	○

ONTARIO MOT PLASTICITY CHART GS-TB-013228 MTO - CULVERTS HWY 61.GPJ DST_MIN.GDT 12/20/13

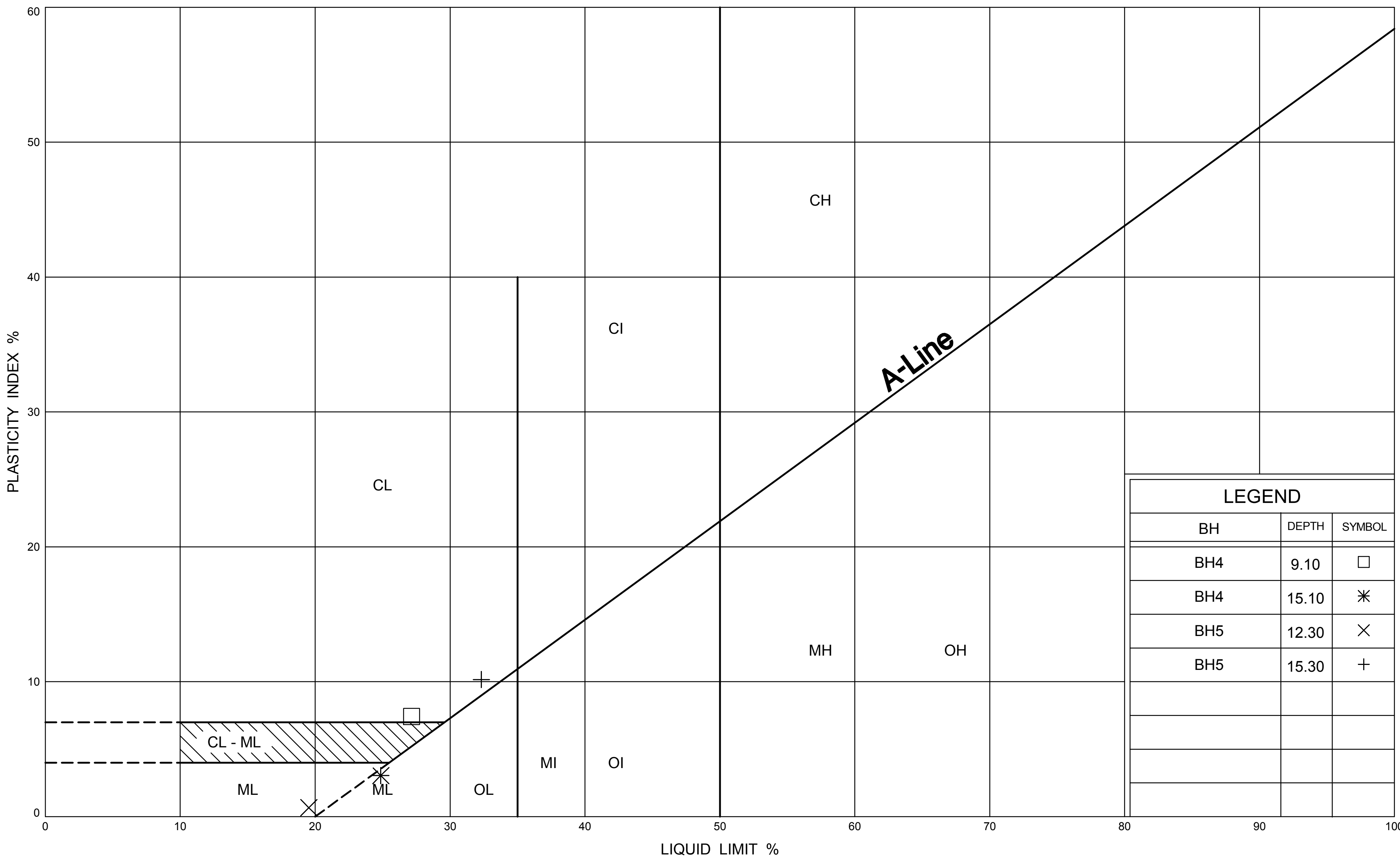
Ministry of Transportation Ontario

PLASTICITY CHART

ENCLOSURE 10

W P 6012-E-0047

HIGHWAY 61



PLASTICITY CHART

Ministry of Transportation Ontario

ENCLOSURE 11

W P 6012-E-0047

HIGHWAY 61

ONTARIO MOT PLASTICITY CHART GS-TB-016596 MTO #6012-E-0047 - HWY 61.GPJ DST_MIN.GDT 12/2/13