

**FOUNDATION INVESTIGATION AND DESIGN REPORT
BLACK CREEK ROAD EMBANKMENT INSTABILITY
WEST OF HIGHWAY 11, ONTARIO**

MTO CONTRACT NO. 2007-5188

Geocres Number: 31E-302

Report to

MMM Group

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

August 17, 2010

File: 19-5161-73

H:\19\5161\73 Hwy 11 Black Ck Instability\Report\Final Report\Black Creek Road
Embankment Instability FIDR_FINAL_Aug_17_2010.docx



TABLE OF CONTENTS

SECTION	PAGE
1 INTRODUCTION	1
2 SITE DESCRIPTION AND BACKGROUND	1
3 SITE INVESTIGATION	3
3.1 Site Inspection	3
3.2 Drilling Investigation and Field Testing	4
4 LABORATORY TESTING	6
5 DESCRIPTION OF SUBSURFACE CONDITIONS	6
5.1 General.....	6
5.2 Surcharge Fill.....	6
5.3 Select Subgrade Material (SSM) Fill.....	7
5.4 Backfill	7
5.5 Silty Clay	8
5.6 Sand and Silt.....	10
5.7 Piezometric Levels.....	10
5.8 Surface Water	11
6 EMBANKMENT SETTLEMENT	12
6.1 General.....	12
6.2 Black Creek Road - Sta. 8+925 to Sta. 9+060 (Embankment Instability Area)	12
6.3 Black Creek Road - Sta. 9+240 to Sta. 9+650 (East of Stirling Creek).....	12
6.4 Estimate of Post Construction Settlement	13
6.5 Measures to Control Settlement.....	13
7 EMBANKMENT STABILITY	14
7.1 General.....	14
7.2 Potential Modes of Instability.....	14
7.2.1 Rotational Slip Surface in Silty Clay.....	15
7.2.2 Translation in the Granular Backfill.....	15
7.3 Limit Equilibrium Analysis	15
7.4 Stabilizing Measures for Reconstruction	16
8 RECOMMENDATIONS	18
8.1 General.....	18
8.2 Comparison of Reconstruction Options.....	18
9 MISCELLANEOUS	21

REFERENCES

Figures

Figure 1	Summary of Subsurface Conditions
Figure 2	Monitoring Data - Vibrating Wire Piezometers and Standpipe Piezometer
Figure 3	Monitoring Data - Settlement Pins (Cross-Sections B-B and D-D)
Figure 4	Monitoring Data - Settlement Rods (between Sta. 9+240 and 9+650)
Figure 5	Daily Rainfall Data in Burk's Falls, Ontario

Tables

Table 1	Summary of Consolidation Test Results
Table 2	Summary of Dissipation Test Results
Table 3	Summary of Settlement Analysis
Table 4	Comparison of Alternatives

Drawings

Drawing 1	Site Plan
Drawings 2 - 3	Embankment Cross-sections and Soil Strata

Appendices

Appendix A	Record of Borehole Sheets
Appendix B	Laboratory Test Results
Appendix C	Cone Penetration Test – CPTU Plots
Appendix D	Cone Penetration Test – Dissipation Test Plots
Appendix E	Site Photos
Appendix F	Slope Stability Analysis Results - Selected Runs

**FOUNDATION INVESTIGATION
AND DESIGN REPORT
BLACK CREEK ROAD EMBANKMENT INSTABILITY
WEST OF HIGHWAY 11, ONTARIO**

MTO CONTRACT NO. 2007-5188

Geocres Number: 31E-302

1 INTRODUCTION

This report presents the results of a foundation investigation and instrumentation monitoring program undertaken for the assessment and remediation of embankment instability along a section of Black Creek Road west of Highway 11, Ontario. This embankment section is part of the Highway 11, Black Creek Road and Robins Road Interchange (Contract 2007-5188) currently under construction. The purpose of the investigation was to explore the subsurface conditions beneath the failed embankment to assess the probable causes of the embankment instability and provide recommendations for remediation of the failed embankment.

Thurber Engineering Ltd. (Thurber) carried out the investigation as a sub-consultant to MMM Group (MMM). The work was carried out in accordance with our proposal letter dated July 22, 2009. Authorization to proceed with the work was provided in an electronic message by MMM on October 13, 2009.

2 SITE DESCRIPTION AND BACKGROUND

The site is located on Black Creek Road approximately 1 km west of Highway 11 and is part of the Highway 11 / Black Creek Road / Robins Road Interchange under construction. The site location is shown in the key plan on Drawing 1. The embankment discussed herein is at the western section of the interchange between Rodeo Road and Stirling Creek (Sta. 8+800 to 9+150 – 350 m long). The design embankment height at the site is 2 m, and was therefore not within the original scope for the foundation investigation.

The Black Creek Road embankment is situated on a low-lying, flat, wet and poorly drained area. The area to the south is an open field sparsely covered with scrub and to the north is a thick vegetation cover of mature trees and brush. An overhead hydro line runs parallel along the north side of Black Creek Road. Stirling Creek crosses Black Creek Road in an approximate north-south orientation. Drainage in the general area is to the south and is mainly controlled by Stirling Creek and Bernard Creek. This area has been subjected to flooding associated with heavy rainfall.

The embankment construction started on December 3, 2008 and proceeded as summarized in Table 2.1 until it failed on July 3, 2009.

Table 2.1 – Summary of Embankment Construction Activities

Activity (between Sta. 8+800 and 9+150)	Date/Time
Removal of organic soils immediately followed by placement of Granular B Type 2 Backfill (~Elev 312.2 m*)	Dec 3, 2008 to Dec 17, 2008
SSM Placement and Compaction (~Elev 313.5 m*)	Jun 1, 2009 to Jun 8, 2009
Ditch Excavation in Granular B Type 2 Backfill	Jun 9, 2009 to Jun 15, 2009
Granular B Type 2 Fill Placement and Compaction up to top of pavement (~Elev 314 m*)	Jun 22, 2009 to Jun 25, 2009
Granular B Type 2 Surcharge Placement was completed on Friday July 3, 2009 at approx. 2:30pm (~Elev 316 m*)	Jun 26, 2009 to Jul 3, 2009 (~2:30pm)
The Embankment between Sta. 8+925 and 9+060 failed on Friday, July 3, 2009 at approx. 4:00pm	Jul 3, 2009 (~4:00pm)

*Note: Elevations referenced to Sta. 9+000

The following summarizes the notable observations collected during the embankment construction by Tulloch Consulting Group (Tulloch), the Contract Administrator (CA) for Contract 2007-5188:

- The removal of organic soils, varied in depth from 1 to 2 m, and was carried out in partial frozen condition;
- Majority of the backfill material, Granular B Type 2 was placed in water and no compaction was carried out in the submerged backfill;
- Compaction was carried out on the backfill placed above surface water level by a vibratory roller. No quality control or quality assurance compaction testing was completed on the backfill;
- Select subgrade material (SSM) fill was placed in lifts (± 450 mm) and compacted by a vibratory roller;
- Above the SSM, Granular B Type 2 was placed in lifts (± 450 mm) to the top of pavement level and compacted by a vibratory roller;
- Granular B Type 2 surcharge was end-dumped to the top of surcharge level and no compaction was carried out.

The failed embankment is approximately 135 m in length (i.e. Sta. 8+925 to 9+060) and its east end is located approximately 150 m west of Stirling Creek. Immediately before failure, the embankment height was 3m increasing to 4m at the eastern end. The embankment consisted of 1m to 2m of SSM fill overlain by 2m of Granular B Type 2 Surcharge. Subsurface conditions before the embankment construction are not known in detail as the foundation investigation carried out at the interchange was limited to culvert and bridge sites and where embankments higher than 2m to the top of pavement were proposed. A pavement investigation was carried out at the subject site, but was not of sufficient depth to delineate the embankment foundation conditions. The scope of

the foundation investigation did not include areas where the design embankment height was less than 2 m and therefore this location was not investigated during the foundation investigation

On July 6, 2009, Tulloch surveyed the embankment and the survey data indicated that the top of embankment centerline had dropped approximately 1.0m to 1.2m. Thurber was requested by MTO to carry out a site inspection to review the instability and develop options for reconstruction. The site inspection took place on July 8, 2009. The observations made during the inspection are shown in the following “Site Investigation” section. Subsequent to the site inspection, Thurber was requested to carry out a subsurface investigation and a monitoring program to obtain further information to assist in assessing the most likely cause of the instability and design suitable remedial measures.

Thurber’s scope of work involved advancing of eight testholes and installing instrumentation to check the excess pore pressure beneath the site of the instability. A monitoring program was also implemented using the following settlement instruments as outlined in a memorandum dated July 14, 2009:

- Eight settlement pins (SP) at the section west of Stirling Creek. The purpose of monitoring these SPs is to study the foundation settlement response subsequent to the embankment instability.
- Six settlement rods (SR) at the area (Sta. 9+240 to 9+650) east of Stirling Creek where the surcharge construction has not yet taken place. The purpose of monitoring these SRs is to study the foundation settlements in response to the relatively small loading due to low embankment (~1 to 2m in height). The settlement results will assist in determining whether the 2m surcharge can be deferred or possibly eliminated for this section of Black Creek Road.

The installations of SRs and SPs were completed by Coffey Geotechnics as a sub-contractor to Carillion Construction Inc., the general Contractor for Contract 2007-5188, on September 24, 2009 and September 30, 2009 respectively.

3 SITE INVESTIGATION

The site investigation was carried out in two stages as follows:

- Site inspection on July 8, 2009
- Drilling investigation and field testing between October 26 and 29, 2009

3.1 Site Inspection

The site inspection was carried out on July 8, 2009 by Paulo Branco, P.Eng. of Thurber accompanied by Tony Sangiuliano, P.Eng. of MTO Foundations, Steve Cunningham of MTO Northeast Region and Scott Houlton of Tulloch. The following observations were made during the visit:

- Length of instability: ~130 to 140m
- Height of embankment (post-failure): ~3.0m
- Both sides of the embankment, north and south, slid away from the embankment centreline.
- Approximately four significant cracks were noted near the centreline of embankment (up to 1m wide and in the order of 1m deep)
- Some tension cracks were noted along the flattened SSM slope
- Toe bulging in the order of 1m at the ditches was noted at both sides blocking the ditch flow
- Two hydro poles located just north of the ditches tilted away from the road embankment (i.e. to the north)

The above observed features are shown in Figure E1 to E6 in Appendix E.

During the investigation program, multiple loose sand boils were noted on ground surface possibly indicating the occurrence of liquefaction, as discussed later. These features were documented in photographs shown in Figures E7 to E12 in Appendix E.

3.2 Drilling Investigation and Field Testing

The drilling investigation and field testing for this project, carried out between 26th to 29th of October 2009, consisted of 5 piezocones tests (CPTU) and 3 sampled boreholes. Two of the testholes were located outside of the instability (i.e. to the east) to furnish information on the horizontal variation of the properties of the undisturbed subsoil. All testholes were advanced to depths of 14.3 to 24.5 m. The testhole locations are shown on the Site Plan (Drawing 1) and Embankment Cross-sections and Soil Strata (Drawings 2 and 3). The total depth and final elevation of each of the testholes are summarized below:

Location	Testhole No.	Total Depth (m)	Testhole Termination Elevation (m)
Sta. 9+063.4 o/s L0.9m (Intact Section)	BH09-01	18.3	297.3
Sta. 8+998.2 o/s L3.7m (Instability)	BH09-02	16.8	298.2
Sta. 8+971.8 o/s R11.4m (Instability)	BH09-03	14.3	298.3
Sta. 9+063.4 o/s L0.9m (Intact Section)	CPTU09-1	24.5	291.2
Sta. 8+999.5 o/s L10.5m (Instability)	CPTU09-2	20.3	292.5
Sta. 9+003.4 o/s R10.9m (Instability)	CPTU09-3	15.0	297.6
Sta. 8+973.6 o/s L11.6m (Instability)	CPTU09-4	18.6	293.9
Sta. 8+973.5 o/s R2.6m (Instability)	CPTU09-5	20.2	294.6

The drilling investigation was carried out using a track-mounted CME45 drill rig supplied and operated by George Downing Estate Drilling Ltd. of Ottawa, Ontario. Hollow stem auger drilling techniques were used to advance the boreholes in the overburden. Samples of the fill and native materials were obtained at selected intervals throughout the entire depth of each borehole using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). In the cohesive deposits, undisturbed soil samples were collected using thin-walled piston tube samplers. The undrained shear strength and sensitivity of the cohesive material were measured in-situ by means of field vane tests (MTO N-Vane).

In Situ Engineering Ltd. supplied and operated the piezocone equipment, mobilizing from Snohomish, Washington. The piezocone consists of a steel rod equipped with electronic sensors at its tip, which is continuously pushed into the ground to measure the cone depth, tip resistance, sleeve friction and dynamic pore water pressure. A stratigraphic profile of the inferred soil properties is produced based on the test results. The piezocone testing included pore pressure dissipation tests at selected depths in cohesive deposits in order to obtain horizontal coefficient of consolidation values. Since coarse granular fill materials were encountered at the surface, pre-augering and then the piezocone was advance through the underlying native soils. In these cases, the conditions within the fill materials were logged from the conventional SPT sampled boreholes. The piezocone data and inferred stratigraphic data are included in Appendix C, The pore pressure dissipation test results are provided in Appendix D.

A standpipe piezometer, consisting of 19 mm PVC pipe with a slotted tip, was installed in BH09-3 to monitor the groundwater level. Two vibrating wire piezometers, VWP-1 and VWP-2, were installed in BH09-1 and BH09-2, respectively, to measure the pore pressure in the foundation soils. The vibrating wire piezometers (VWP) were RST Standard Vibrating Wire Piezometer (Model VW2100-0.35), suitable for pressures from 0 to 350 kPa. The VWP includes a thermistor for the measurements of groundwater temperature. The VWP locations are shown schematically in Drawing 2 and are summarized below:

Vibrating Wire Piezometer	Ground Surface Elevation	Sensor Tip Elevation
VWP-1 (in BH09-1)	315.61 m	302.20 m
VWP-2 (in BH09-2)	314.93 m	306.09 m

The coordinates and elevations of the testholes were surveyed by Tulloch and are provided on the Site Plan and Embankment Cross-section and Soil Strata drawings and on the individual Record of Borehole Sheets in Appendix A.

A member of Thurber's engineering staff supervised the piezocone testing, borehole drilling and sampling operations on a full time basis. The inspector logged the boreholes and the recovered samples and processed them for transport to Thurber's Oakville office. All boreholes were grouted on completion of the drilling program in accordance with O'Reg 903 (as amended by O'Reg 372/07).

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A.

Selected samples were subjected to gradation analysis (sieve and hydrometer) and Atterberg Limit tests and the results are shown on the Record of Borehole sheets in Appendix A and on the charts in Appendix B. Selected samples were also subjected to one-dimensional consolidation testing.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

5.1 General

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil stratigraphy are presented on these records and on the attached Embankment Cross-section and Soil Strata drawings. A generalized description of the stratigraphy is given in the following paragraphs for summary purposes. However, the borehole logs should be referenced in preference to the generalized description for interpretation of the subsurface conditions. It should be recognized that soil conditions may vary between and beyond borehole locations.

In general terms, the stratigraphy of the site area comprises of embankment fill materials overlying a deep deposit of glacial lake sediments (glaciolacustrine origin), which is in turn underlain by sand and silt. The fill materials generally consist of Granular B Type 2 surcharge fill underlain by SSM embankment fill which overlies Granular B Type 2 backfill. The glaciolacustrine deposit consists of plastic silty clay with varying thickness.

The subsurface conditions at the project site are summarized schematically in Figure 1. More detailed descriptions of the individual strata are presented below.

5.2 Surcharge Fill

A layer of granular fill used as surcharge material was encountered in the testholes advanced from the top of the embankment. The surcharge fill was described as sand and gravel, trace silt and was described as dry to moist and grey.

Based on recorded SPT N-values ranging from 8 to 30 blows for 0.3 m of penetration, the surcharge fill is described as loose to compact with increasing density with depth.

The natural moisture content of the samples obtained from the sand layer ranged from 2 to 5%. The results of laboratory tests carried out on one grab sample (GS#2) were as follows:

Particle Size	Gravel	Sand	Silt and Clay
(%)	78	19	3

The grain size distribution curve for the sample tested lies within the typical range of MTO specified Granular B Type 2 and is shown in Figure B1 in Appendix B.

The depth to the base of the surcharge fill ranged from 2.3 m (Elevation 313.3m) to 2.9 m (Elevation 311.6m).

5.3 Select Subgrade Material (SSM) Fill

Immediately below the surcharge fill, SSM fill was encountered in the testholes advanced from the top and on the slope of the embankment. The SSM fill was described as sand, trace to some gravel, trace to some silt and was described as moist to wet and was brown.

The SSM fill within the central portion of the embankment (i.e. where covered directly by surcharge fill) is described as compact to dense based on SPT N-values of 18 and 32 blows for 0.3 m of penetration. Where underneath the embankment slopes, the SSM fill is described as loose based on recorded SPT N-values ranging from 7 to 8 blows for 0.3 m of penetration.

The natural moisture content of the samples obtained from the SSM fill ranged from 3 to 14%.

The results of laboratory tests carried out on four samples were as follows:

Particle Size	Gravel	Sand	Silt and Clay
(%)	3 to 20	70 to 84	3 to 20

The grain size distribution curves for the samples tested lie within the typical range of MTO specified SSM and are shown in Figure B2 in Appendix B.

The elevation of the base of the SSM fill ranged from 311.9 m to 310.4m. The thickness ranged from 0.6 m to 1.4 m.

5.4 Backfill

A layer of sand and gravel backfill was encountered below the SSM fill extending to the native soils. The sand and gravel backfill contained trace to some silt and was described as moist to wet and grey. This is consistent with the construction records which indicate that the native organic soil was excavated and replaced with the sand and gravel backfill.

The backfill is described as loose to compact based on SPT values of 3 and 27 blows for 0.3 m of penetration with decreasing density towards its bottom.

The natural moisture content of the samples obtained from the backfill ranged from 10 to 27%.

The results of laboratory tests carried out on six samples were as follows:

Particle Size	Gravel	Sand	Silt and Clay
(%)	38 to 62	33 to 48	4 to 14

The grain size distribution curves for the samples tested, as shown in Figure B3 in Appendix B, generally lie within the typical range of MTO specified Granular B Type 2 except the sample in BH09-2 SS#4 at 4.27 m. This grain size curve appears to contain finer sand material which lies above the upper limit of the Granular B Type 2. The gradation tests on the backfill samples showed that the fines content typically decreased whereas the gravel content increases with depth.

The sand boils from the backfill was sampled for laboratory testing and a summary of the gradation distribution is shown below:

Particle Size	Gravel	Sand	Silt and Clay
(%)	43	47	10

The grain size distribution curve for the sand boil sample shown in Figure B4 in Appendix B lies in the finer boundary of the range of MTO specified Granular B Type 2.

The elevations of the base of the backfill ranged from 311.0 m to 309.3 m. The thickness ranged from 0.9 m to 2.9 m. Further variations in thickness may occur between or beyond the testholes.

5.5 Silty Clay

A deposit of silty clay was encountered below the backfill in all testholes extending to the bottom of the sampled boreholes at depths ranging from 18.3 m to 24.5 m. The silty clay contained trace silt and was described as moist to wet and grey.

The results of in-situ strength testing indicate that the upper 1.2 m of this deposit within the embankment loaded zone is described as being stiff. In this upper loaded zone, SPT N-value was 6 blows for 0.3 m of penetration and the inferred SPT N-values by piezocone ranged from 8 to 15 blows for 0.3 m of penetration. Undrained shear strength values were generally in the order of 50 to 100 kPa as measured by in-situ vane tests and inferred from

piezocone data. The undrained shear strength in this upper zone generally decreased with depth.

Below the embankment footprint, the results of in-situ strength testing indicate that the silty clay is described as being soft to firm. SPT N-values ranged from Weight of Hammer (WH) to 4 blows, typically from WH to 1, for 0.3 m of penetration. The inferred SPT N-values from piezocone data ranged from 1 to 4 blows for 0.3 m of penetration. Undrained shear strength values, generally increasing with depth, ranged from 13 to 50 kPa as measured by in-situ vane tests and inferred from piezocone data. Each piezocone sounding detected one to two stiff to very stiff silty clay interlayers. The thickness of the interlayers ranged from 0.2 to 0.6m and the inferred undrained shear strength were in the order of 50 to 198 kPa.

The sensitivity of the silty clay, as measured in remoulded vane shear tests, ranged from 3 to 6 indicating the deposit has low to medium sensitivity.

The natural moisture contents of the samples recovered from the silty clay deposit ranged from 29 to 72% with the higher values closer to the bottom of the deposit.

The results of laboratory tests carried out on eleven samples were as follows:

Particle Size	Gravel	Sand	Silt	Clay
(%)	0 to 3	0 to 10	41 to 67	29 to 59

Atterberg Limits	(%)
Liquid Limit	29 to 44
Plastic Limit	22 to 25

The results of these tests indicate that the silty clay deposit has low to medium plasticity and is classified according to the Modified Unified Soil Classification System as follows:

- Low plastic silty clay (CL) in the upper portion of the deposit with 29 to 40% clay content
- Medium plastic silty clay (CI) in the lower portion of the deposit with 42 to 59% clay content.

The grain size distribution curves for the samples tested are shown in Figures B5 to B7 in Appendix B. Atterberg Limit test results are shown in Figure B8 to B9 in Appendix B.

The consolidation tests carried out on two undisturbed silty clay samples, both collected from BH09-2, are presented in Figures B10 and B11 in Appendix B. The consolidation tests results are summarized on the Record of Borehole sheets in Appendix A and in Table 1 (Summary of Consolidation Test Results). In Table 1, the consolidation test

results were compared between the current investigation and the previous investigation for the Highway 11 Black Creek Road / Robins Road Interchange Project. Although the sampling locations are more than 600 m apart, the consolidation test results are quite similar.

The results of the consolidation tests, insitu strength testing and piezocone data indicate that within the embankment footprint the silty clay is over-consolidated in the upper 1 to 2 m and becomes normally consolidated with depth.

Horizontal coefficient of consolidation (C_h) values were inferred from the pore water dissipation tests conducted at various depths in every piezocone, as summarized in Table 2. The C_h values, along with the vertical coefficients of consolidation (C_v) values obtained from the consolidation tests, are plotted with elevations in Figure 1.

The silty clay deposit was not completely penetrated in all testholes. The elevation of the underside of the silty clay deposit ranged from less than 291.2 m at CPTU09-1 to 296.3 m at CPTU09-4. The thickness of the silty clay deposit ranged for 11.1 m to greater than 18.6 m. It appears that the silty clay deposit along Black Creek Road becomes thicker and deeper towards Highway 11.

5.6 Sand and Silt

The silty clay deposit was penetrated at CPTU09-2, CPTU09-4 and CPTU09-5. Based on the inferred stratigraphy from the piezocone test, the underlying soil is described as sand and silt. The inferred SPT N-values from piezocone data ranged from 6 to 20 indicating that the sand and silt deposit has loose to compact density.

Where encountered, the elevations of the top of this deposit ranged from 296.3 m to 292.8 m.

5.7 Piezometric Levels

The initial and subsequent groundwater depths and elevations monitored in the standpipe piezometer in BH09-3 are shown in the following table.

Borehole	BH09-3	
	Depth below top of backfill (m)	Elevation (m)
October 30, 2009	0.80	311.81
November 11, 2009	0.50	312.11
November 20, 2009	0.78	311.84
December 8, 2009	0.69	311.92
December 17, 2009	0.84	311.77
December 23, 2009	0.92	311.70
January 12, 2010	1.01	311.60
February 1, 2010	0.90	311.72

February 19, 2010	1.10	311.51
February 25, 2010	1.12	311.49
March 9, 2010	0.95	311.66
March 18, 2010	0.75	311.86
April 13, 2010	0.86	311.75

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level will be influenced by the Stirling Creek level and may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall. A plot of the water level variation is shown in Figure 2.

The piezometric level monitoring using VWPs was started on October 29, 2009. A plot of the measured piezometric levels is included in Figure 2. It is shown that VWP-1, located underneath the intact embankment, has a piezometric head approximately 2.7 to 3.5 m higher than the level measured in BH09-3 over the interval where data was collected. The excess pore pressure (EPP) at VWP-1 appears to dissipate slowly (~1.5 to 1.6 kPa per month) and approximately 27 kPa remains in the foundation silty clay on April 13, 2010. The piezometric levels measured in VWP-2, located underneath the crest of the embankment, are (~0 to 0.4 m) lower than the conditions measured at the toe of the embankment, and do not show evidence of significant EPP or dissipation over time at these locations.

5.8 Surface Water

Precipitation record of Burk's Falls Station, as presented in Figure 5, was obtained from Environment Canada to study the temporal fluctuation in surface water. The precipitation data indicates that the site area experienced the highest daily rainfall of 16.6 mm and 17.0 mm on July 2 and 3, 2009, respectively, during that summer. Observations from piezometers installed at adjacent sites indicate a water-table variation of approximately 0 to 0.25 m concurrent with this storm.

6 EMBANKMENT SETTLEMENT

6.1 General

The original embankment design for these two sections of Black Creek Road comprises of application of a 2 m high surcharge placed on a 1 to 2 m high embankment. The scope of the foundation investigation did not include areas where the design embankment height was less than 2 m and therefore this location was not investigated during the foundation investigation. The instability between Sta. 8+925 and 9+060 occurred immediately after placement of the surcharge fill to approximately 2 m above the final pavement grade. The instability resulted in a drop of elevation of the top of surcharge by about 1 m and corresponding uplift of the ditch at the toe of the embankment.

The surcharge was originally incorporated into the design to reduce potential post-construction settlement, and therefore an assessment of the estimated foundation settlements was carried out for the subject embankment and for adjacent portions of Black Creek Road where the surcharge has yet to be constructed. The long term settlements were estimated based on varying heights of surcharge and consolidation times. Selection of the preferred surcharge option will depend on the schedule and long term settlement requirements for Black Creek Road.

6.2 Black Creek Road - Sta. 8+925 to Sta. 9+060 (Embankment Instability Area)

- The existing height of this section of the embankment measured from top of surcharge to the top of pavement, following the instability, is irregular and varies from 0.8 to 1.5 m.
- Back-analysis of the measured settlements from the Settlement Pins, as shown in Figure 3, indicates that approximately 300 to 360 mm of settlement had taken place from July 3, 2010 to April 7, 2010 (approximately 9 months). The starting time for the analysis ($t = 0$) is assumed to be the completion date of surcharge placement (i.e. July 3, 2010).

6.3 Black Creek Road - Sta. 9+240 to Sta. 9+650 (East of Stirling Creek)

- The embankment was constructed to top of pavement elevation in this area which ranges from 1.0 to 2.1 m above original ground surface. No surcharge was applied at this location pending review of the stability and settlement conditions.
- Back-analysis of the measured settlements from the Settlement Rods, as shown in Figure 4, indicates that approximately 190 to 260 mm of settlement had taken place from late July 2010 to April 7, 2010 (approximately 8 months). The starting time for analysis ($t = 0$) is assumed to be in late July, 2010 when the embankment fill placement was completed.

6.4 Estimate of Post Construction Settlement

The anticipated post-construction settlements were estimated based on recent settlement data for the site. Time-dependent settlements have been estimated based on the following methodologies:

- Settlement analysis using Settle3D (Version 2.006) based on the one-dimensional finite difference calculation method. Consolidation parameters were selected based on the geotechnical data presented in Figure 1.
- Curve fitting techniques (Sridharan, 1987 and Asaoka, 1978) using survey readings collected from the settlement pins (SP) and settlement rods (SR), as shown in Figures 3 and 4, for consolidation settlement extrapolation.

The results of the above projected settlements are compared with those measured in the foundation monitoring program of the Highway 11 / Black Creek Road / Robins Road Interchange. For the site of the instability, and assuming the embankment is re-constructed back to final design pavement grade by April, 2010, the remaining settlements will be:

- Primary Consolidation Settlement: 0 to 50 mm in 15 months
- Secondary Consolidation Settlement: 100 to 150 mm in 20 years from end of primary consolidation (EOP)

For the area east of Stirling Creek, assuming the embankment is constructed to final design pavement grade in April, 2010, the outstanding settlements are:

- Primary Consolidation Settlement: 90 to 130 mm in 22 months
- Secondary Consolidation Settlement: 110 to 160 mm in 20 years after EOP

Provided that the outstanding primary consolidation settlement is complete, the estimated long term settlements (secondary consolidation settlements) are less than the previously accepted design values (i.e. < 200mm). However, the estimated waiting times to reach the end of primary consolidation are in excess of the time allowed under the contract. The rate of primary consolidation settlement can be controlled by means of surcharging to accelerate settlement or by using light weight fill to reduce settlement, as discussed in the subsequent sections.

6.5 Measures to Control Settlement

In order to reduce the post-construction settlements as discussed above, the following measures have been considered:

a. Placement of Surcharge

The additional height of surcharge required to meet the foundation performance requirements of time-dependent settlements was assessed. The results of the settlement

analysis for various surcharge heights indicate that the time to reach end of primary consolidation and remaining post-construction settlements decrease as the surcharge height increases, as shown in Table 3. Due to the weak foundation conditions, the surcharge options would require stabilizing measures to satisfy the required factor of safety as described in the later section.

b. Light Weight Fill

Another alternative for reducing the post-construction settlement would be to construct the embankment with light weight fill material. Depending on the degree of load reduction required, light weight fill could consist of: blast furnace slag, cellular concrete or expanded polystyrene fill (EPS). Typically, unit rates for light weight fill materials are more costly than standard granular and rock fills.

7 EMBANKMENT STABILITY

7.1 General

This section presents an interpretation of the geotechnical data and assessment of the potential causes of the embankment instability. Based on the soil parameters derived from back analysis of the instability and construction settlement monitoring at the site, options for reconstruction of the embankment including assessment of the long term settlement performance for each option have been developed.

The analysis of the additional data obtained from the investigation of the instability and the collection of settlement data from the instrumentation at this location presents a potential opportunity to refine the embankment design at the site of the instability (Sta. 8+925 to 9+060) and at the swamp east of Stirling Creek (Sta. 9+240 to 9+650). This additional information can be used to evaluate potential stability improvements and assess the surcharge requirements and settlement rates in these areas.

The embankment stability at final pavement grade is sufficient to maintain stability, and the embankments have successfully been constructed to these elevations. Construction of additional embankment height for surcharge requirements has caused instability at one location and therefore stability analysis was carried out to assess this condition.

7.2 Potential Modes of Instability

The instability at St. 8+925 to 9+060 occurred after completion of the 2 m high surcharge. A review of the results of the field and laboratory investigation programs considers two potential failure mechanisms that might have caused the embankment failure: i) rotational slip surface through the silty clay or ii) translational slide through the granular backfill. Details for each of these options are presented below.

7.2.1 Rotational Slip Surface in Silty Clay

The following factors indicate that the embankment failure may have occurred through the silty clay:

- The piezocone and SPT data presented in Appendix C show zones within the top 1m to 4m of the silty clay where the undrained shear strength ranges from 13kPa to 18kPa, indicating soft consistency.
- A visual inspection of the piston samples showed that the samples within the top 3m of the silty clay were more disturbed, possibly due to the embankment failure.
- The pore pressures measured with VWP 09-2, installed in the top 3m of the silty clay in the zone where the embankment failed, indicated that there were no excess pore pressures in this area. VWP09-1, installed in the section of the embankment that did not fail, measured excess pore pressures in the order of 35 kPa, equivalent to a 1.5m high embankment. This may indicate that the excess pore pressures where VWP09-2 was installed may have dissipated because of the disturbance of this zone during the embankment failure.

7.2.2 Translation in the Granular Backfill

The site investigation within the embankment fill materials encountered compact to loose sand and gravel with SPT values generally decreasing at greater depths (N=27 to 3).

Because of the presence of sand boils at the site, an assessment of the liquefaction potential of the backfill was carried out using a screening method suggested by Tsuchida (1970) as illustrated in Figures B3 and B4 in Appendix B. The bold lines indicate the grain size distribution for the boundaries potentially liquefiable soils and most liquefiable soils. The gradation curves for all backfill samples do not fit within these boundaries and therefore static liquefaction is not considered a likely trigger of the instability.

7.3 Limit Equilibrium Analysis

Limit equilibrium analyses were carried out in order to assess whether the embankment failure may have been caused by the presence of soft silty clay underlying the granular backfill. Analysis was carried out using G-Slope software Version 4.12 (Mitre Software Corporation) and Bishop's Modified method.

The analysis was carried out in terms of both total and effective stresses in the foundation soils and was based on assumptions outlined in Appendix F. The results of the analysis are presented in Appendix F and summarized as follows:

Case	Factor of Safety	Figure in Appendix F
Effective Stress Analysis	0.99	F1
Total Stress Analysis	1.01	F2

The above results indicate that the embankment fails through a rotational slip surface mechanism that is likely associated with the presence of a soft zone in the silty clay foundation. Accordingly, stabilizing measures to allow reconstruction to the original design grades or to alternate surcharge heights have been developed based on this information.

7.4 Stabilizing Measures for Reconstruction

Three surcharge cases have been considered (no surcharge, 1m surcharge and 2m surcharge) for the purpose of assessing waiting time and remaining post-construction settlement as described previously. Each surcharge option will require the stabilizing measures presented below, to satisfy the required factors of safety during and after construction.

The existing SSM subgrade has been disturbed in the area where the instability has occurred, and therefore the existing disturbed material should be sub-excavated down to original ground elevation (about EL 312.5 m) followed by re-construction of the embankment up to the final grade or to top of surcharge as required. Sub-excavation of the disturbed material is recommended to ensure a properly compacted embankment during its re-construction.

a. No Further Treatment (additional surcharge construction not implemented)

This option involves proceeding directly with removal of the displaced/ disturbed surcharge material and SSM base down to original ground elevations (about EL 312.5 m), followed by reconstruction of the embankment to the final grade as per the original design. The embankment stability considering a maximum 0.5 m deep ditch, under long term conditions provides a minimum factor of safety of 1.5.

b. Stabilizing Berms (for surcharge re-construction)

Stabilizing berms are considered a viable option to maintain sufficient stability during construction of surcharge up to 1 m or 2 m above the final grade. The dimensions of the berms vary with the proposed surcharge height and the location as shown in the following table.

Location	Surcharge Height	Berm Dimensions (width x height)	Stability Results in Appendix F
Black Creek Road Sta. 8+925 to 9+060 (Site of Instability)	1 m	4.0 m x 1.0 m	F5, F6
	2 m	7.5 m x 1.0 m	F3, F4
Black Creek Road Sta. 8+240 to 9+650	1 m	4.0 m x 1.0 m	F5, F6
	2 m	7.5 m x 1.0 m	F3, F4

The berm geometry is designed based for a factor of safety of 1.3. The height of berm should be measured from original ground surface and the berm should be sloped no steeper than 1.5H:1V. The stabilizing berms should be constructed on each side of the embankment prior to construction of the embankment to final elevation.

Granular B Type 2 is recommended for construction of the berms. The stabilizing berms can be removed following surcharge removal. Compaction of the berms per OPS 501 is recommended if the berms are to be left in place.

c. Geogrid Reinforcement (for surcharge re-construction)

Construction of the embankment up to top of surcharge is also feasible using high strength uni-axial geogrid instead of the berms described above. The geogrid should be placed across the width of the embankment at the original ground elevation (~EL 312m). The embankment would then be reconstructed as per the original design.

Tensar UX1600MSE uniaxial geogrid with ultimate tensile strength of 58kN/m at 5% strain per layer is recommended. The number of layers required is designed based on a factor of safety of 1.3, strain compatibility and varies with the proposed height of surcharge. The grid strength and number of layers required are summarized in the following table. The geogrid should extend a minimum length of 6.5 m to each side of centreline.

Location	Surcharge Height	Geogrid (width x height)	Stability Results in Appendix F
Black Creek Road Sta. 8+925 to 9+060 (Site of Instability)	1 m	1 layers UX1600	F10, F11
	2 m	2 layers UX1600	F8, F9
Black Creek Road Sta. 8+240 to 9+650	1 m	1 layers UX1600	F10, F11
	2 m	2 layers UX1600	F8, F9

d. Light Weight Fill (No surcharge)

The “Light Weight Fill” option is not considered economically feasible within the existing contract terms, and alternate methods available meeting the current schedule requirements are preferred as discussed in the subsequent section .

8 RECOMMENDATIONS

8.1 General

The subject site is underlain by soft, compressible soil. Review of the recent subsurface information collected following the instability indicates that the likely instability mechanism involves circular failure in soft clay related to excess pore pressure generated during construction. Settlement monitoring carried out during embankment construction in this area and in adjacent areas provides additional information which can be used to improve estimates of long term settlement rates following construction to design grades. The following options are proposed for re-construction and control of post-construction settlement.

8.2 Comparison of Reconstruction Options

The embankments are currently constructed to the elevation of the final pavement grade or to 0 to 1 m above this grade (at Sta. 8+925 to 9+060). The settlement rates have been monitored under these conditions for a period of 6 months, and the predicted long term settlements have been derived as shown in the following table, assuming the reconstruction is completed by April, 2010 (i.e. “Construct Now” option).

Surcharge Thickness	Stabilizing Berm (Width x Height,)	Geosynthetic Reinforcement	Waiting Time before Paving	Settlement during waiting period	Settlement Long Term (after 20 yrs)
Black Creek Road – Sta. 8+925 to Sta. 9+060 (Site of Instability)					
0 m	-	-	15 months	0 to 50 mm	100 to 150 mm
1 m	4m x 1m	-	9 months	50 to 110 mm	60 to 100 mm
	-	60 kN/m			
2 m	7.5m x 1m	-	5.5 months	100 to 200 mm	40 to 80 mm
	-	120 kN/m			
Black Creek Road – Sta. 9+240 to Sta. 9+650 (East of Stirling Creek)					
0m	-	-	22 month	90 to 130 mm	110 to 160 mm
1 m	4 m x 1 m	-	13.5 months	150 to 200 mm	70 to 110 mm
	-	60 kN/m			
2 m	7.5m x 1m	-	7.5 months	200 to 300 mm	50 to 80 mm
	-	120 kN/m			

The long term settlement rates for the *No additional Surcharge Option* significantly exceed the design standards set for this project, and is not feasible without significant schedule delay.

A significant reduction in the long term settlement magnitude can be achieved by application of a temporary surcharge for a sufficient waiting period to allow completion of primary consolidation. Increasing the surcharge height would provide increased consolidation rates thereby reducing the wait time if required to meet schedule requirements.

Construction of a surcharge would require stabilizing measures as described in the preceding sections. Preliminary cost analysis indicates that the construction of stabilizing berms would be more cost effective than utilization of geogrid reinforcement options for the extensive treatment length. However, localized application of geogrid where property limits preclude the use of berms is considered feasible. The available footprint area for construction of the berms may require some variation in the berm dimensions. Additional analysis may be required to adjust berm dimensions as required.

In addition, after discussion with the Project Team, it is understood that the Contract has flexibility to extend the Black Creek Road closure to until late Spring or early Summer of 2011. Therefore, aside from the “Construct Now” option, a “Do Nothing” option has also been assessed as follows:

Black Creek Road – Sta. 8+925 to Sta. 9+060 (Site of Instability)

- The current embankment height at the instability site is approximately 1m above the design pavement grade which is equivalent to about 1m surcharge above the design embankment grade.
- Leave the existing failed embankment until late Spring / early Summer of 2011 to allow further primary consolidation settlement to take place for approximately 1 year or as long as possible.
- The estimated long term settlement (20 years after paving), based on the above constraints, would be in the order of 60 to 100 mm.

Black Creek Road – Sta. 9+240 to Sta. 9+650 (East of Stirling Creek)

- Approximately 190 to 260 mm settlement has taken place up to April 2010.
- It is recommended that the embankment be raised to 250 mm above the final design pavement grade to allow for additional settlement during the waiting period.
- Leave the existing embankment until late Spring / early Summer of 2011 to allow primary consolidation settlement to take place and allow approximately 1 year for primary settlement to occur.

- The estimated long term settlement (20 years after paving) under the above conditions would be in the order of 150 to 220 mm

The above estimated long term settlement values are consistent with the previously calculated design settlements (i.e. 200 mm) for the adjacent interchange and Stirling Creek Culvert embankments. Therefore the “Do Nothing” option is considered feasible for reconstruction of these portions of Black Creek Road.

The selection of the preferred option for these locations will depend on the budget and schedule requirements of the construction contract. Monitoring of the existing instrumentation would be required to confirm that primary consolidation has been completed prior to road paving.

The constructed geometry of the embankment must be overbuilt to take account of the predicted settlements such that the design platform width is achieved after consolidation settlement is complete and the surcharge is stripped to subgrade elevation. The quantity estimates must take account of the additional material required to overbuild the embankments to compensate for settlement.

During construction, the Contract Administrator should employ experienced high complexity geotechnical staff to implement the geotechnical monitoring program and to observe construction activities related to foundation remediation.

9 MISCELLANEOUS

Surveying of the locations of the testholes was carried out by staff from Tulloch.

George Downing Estate Drilling Inc. of Ottawa, Ontario supplied and operated a track-mounted CME-45 drill rig to conduct the drilling, sampling and in-situ testing operations.

Piezocene testing and equipment was provided and operated by In Situ Engineering Ltd. of Snohomish, Washington.

Full time supervision of field activities was carried out by Mr. Tony Harte, M.Sc. of Thurber.

Overall supervision of the field program, interpretation of the data and engineering analysis, preparation of the report were carried out by Mr. Steven Sather, P.Eng. and Mr. Jason Lee, P.Eng..

The report was reviewed by Dr. Paulo Branco, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Jason P. Lee, P.Eng., M.Sc.
Geotechnical Engineer



Steven Sather, P.Eng., M.Eng.
Senior Geotechnical Engineer

Report reviewed by:
Paulo J. Branco, P.Eng., Ph.D.
Review Principal

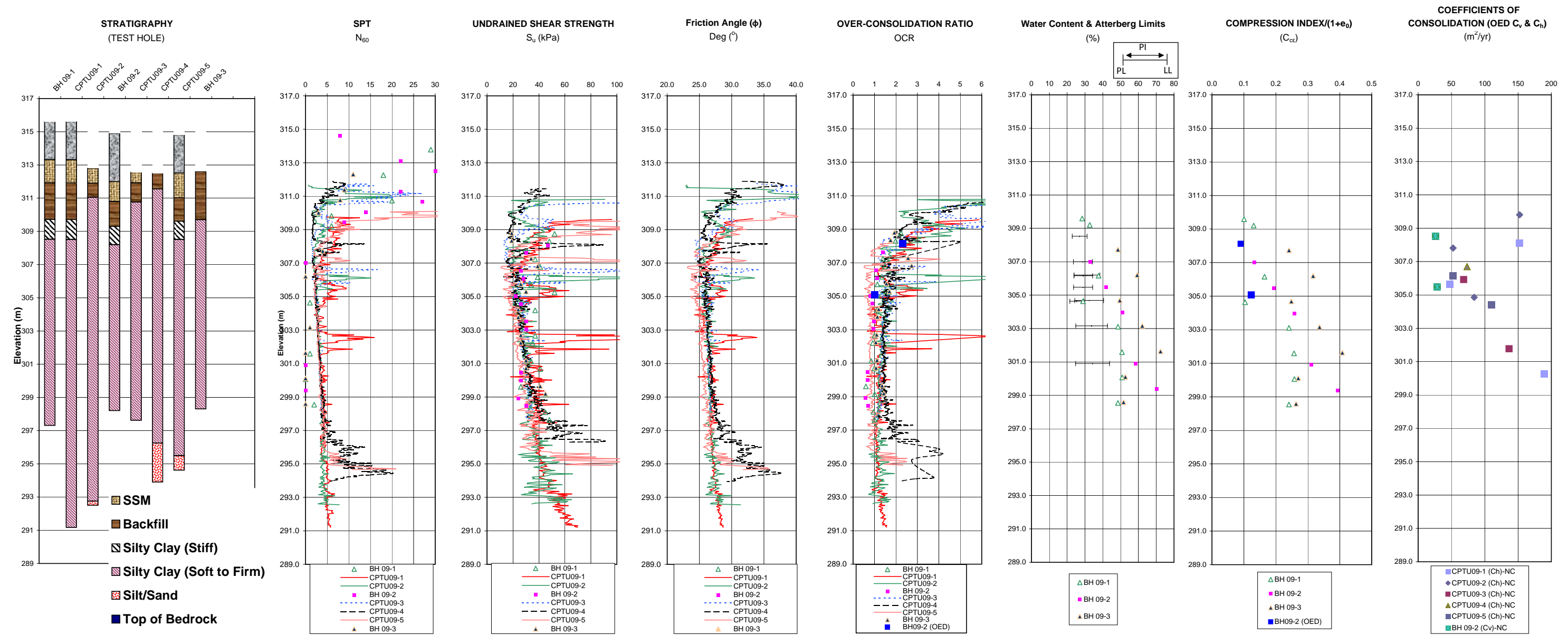


REFERENCES

- Fourie, A.B. and Tshabalala, L. (2005), Initiation of static liquefaction and the role of K_0 consolidation. Canadian Geotechnical Journal, Vol. 42: 892-906
- Mahmoud, M, Woeller, D. and Robertson, P.K. (1999), Detection of shear zones in a natural clay slope using the cone penetration test and continuous dynamic sampling. Canadian Geotechnical Journal, Vol. 37: 652-661
- Terzaghi, K., Peck, R.B., and Mesri, G. (1996), Soil Mechanics in Engineering Practice, John Wiley and Sons, 549 pages.
- Sasitharan, S., Robertson, P.K., Sego, D.C. and Morgenstern, N.R. (1993), Collapse behaviour of sand. Canadian Geotechnical Journal, Vol. 20: 569-577
- Kulhawy, F.H. and Mayne, F.H. (1990), Manual on Estimating Soil Properties for Foundation Design. Electric Power Research Institute EL-6800, Research Project 1493-6
- Sridharan, A., Murthy, N.S. and Prakash, K. (1987), Rectangular Hyperbola Method of Consolidation Analysis. Geotechnique 37, No. 3, 355-368
- Asaoka, A. (1978), Observation Procedure of Settlement Prediction. Soils Foundations 18, No.4, 87-101
- Tsuchida H. (1970), Prediction and Countermeasure Against the Liquefaction in Sand Deposits, in abstract of the Seminar in the Port and Harbor Research Institute 1970 pp 3.1-3.33.

Figures

BLACK CREEK ROAD EMBANKMENT INSTABILITY (FROM STA. 8+925 to 9+064)
SUMMARY OF SUBSURFACE CONDITIONS



MASTER PLOT

FIGURE 1

19-5161-73 Highway 11 Black Creek Road Embankment Instability MONITORING DATA: PIEZOMETRIC LEVEL vs. TIME PLOT

Vibrating Wire Piezometer VWP-1, 2 / Standpipe Piezometer SPP-1

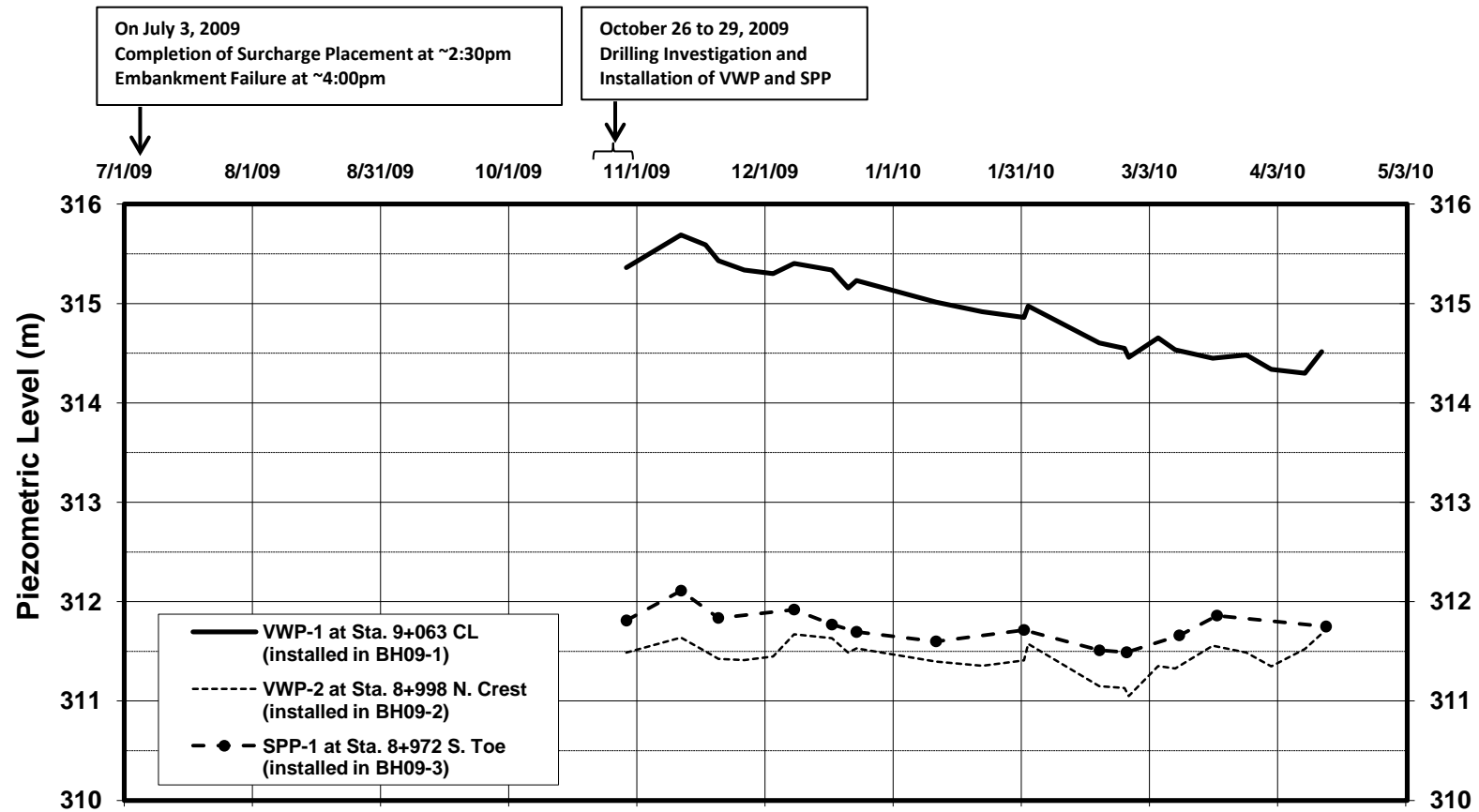


FIGURE 2

19-5161-73 Highway 11 Black Creek Road Embankment Instability

MONITORING DATA: SETTLEMENT (SP) vs. TIME PLOTS

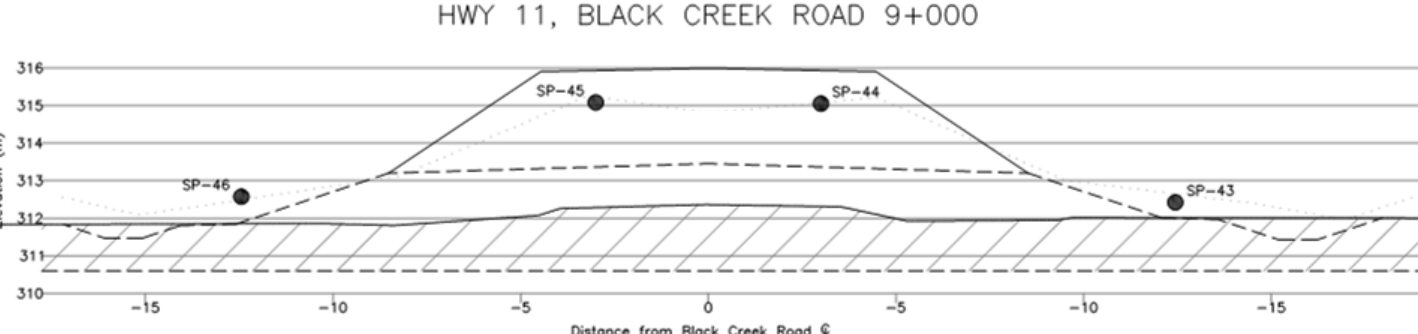
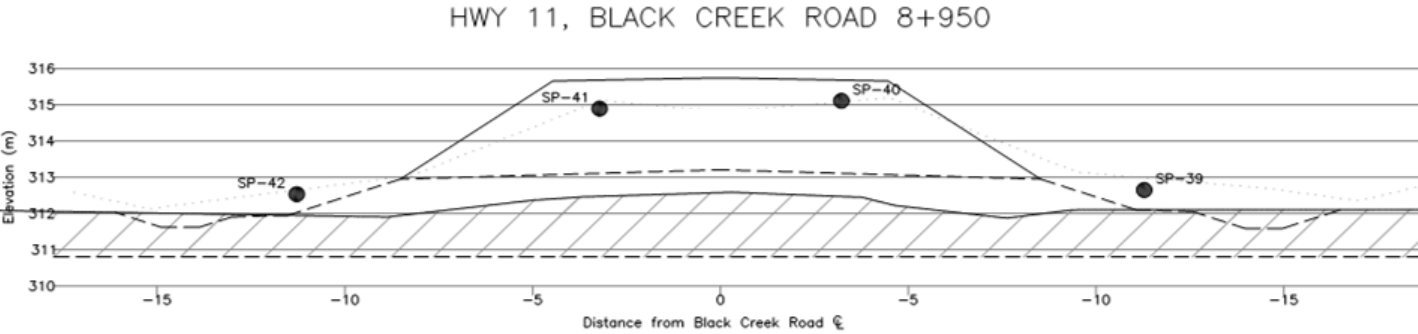
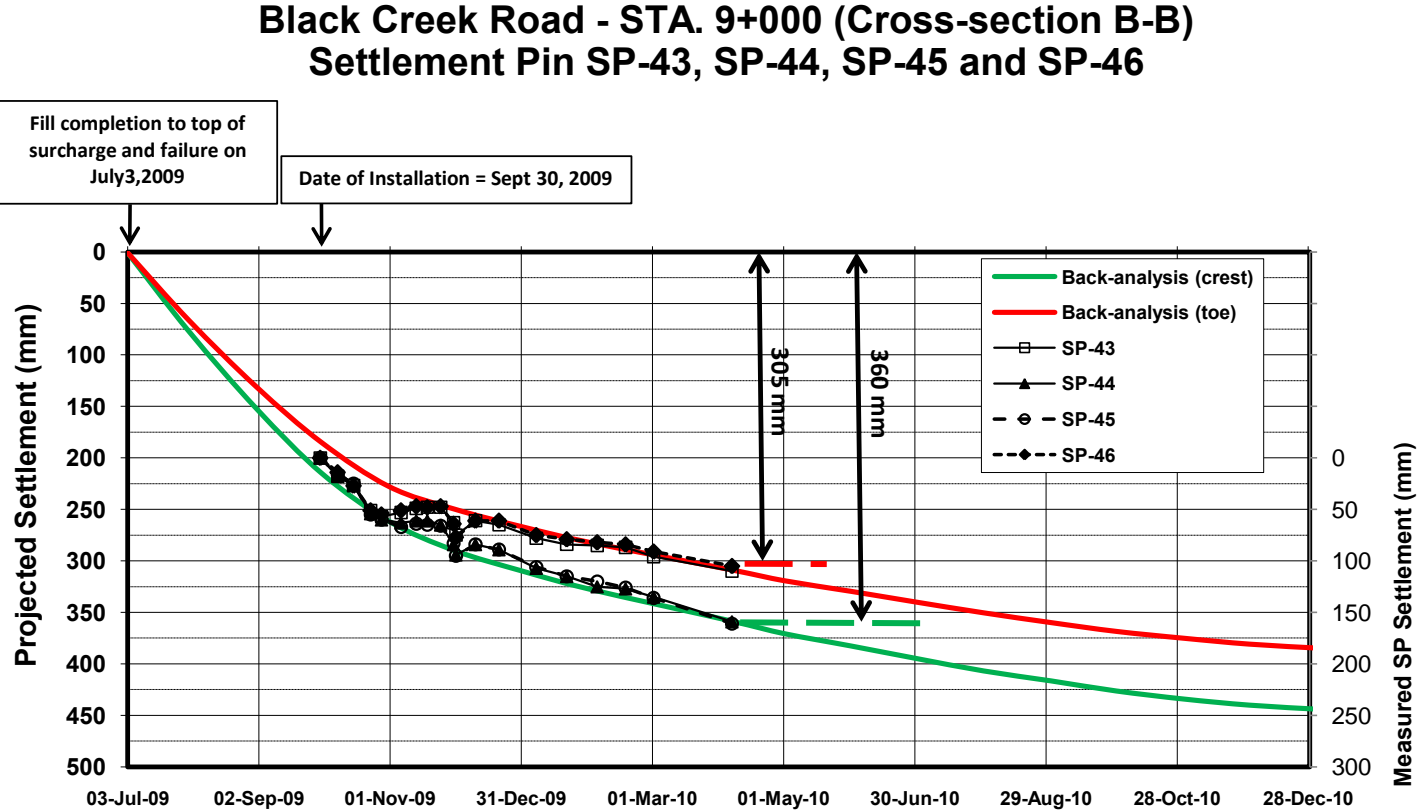
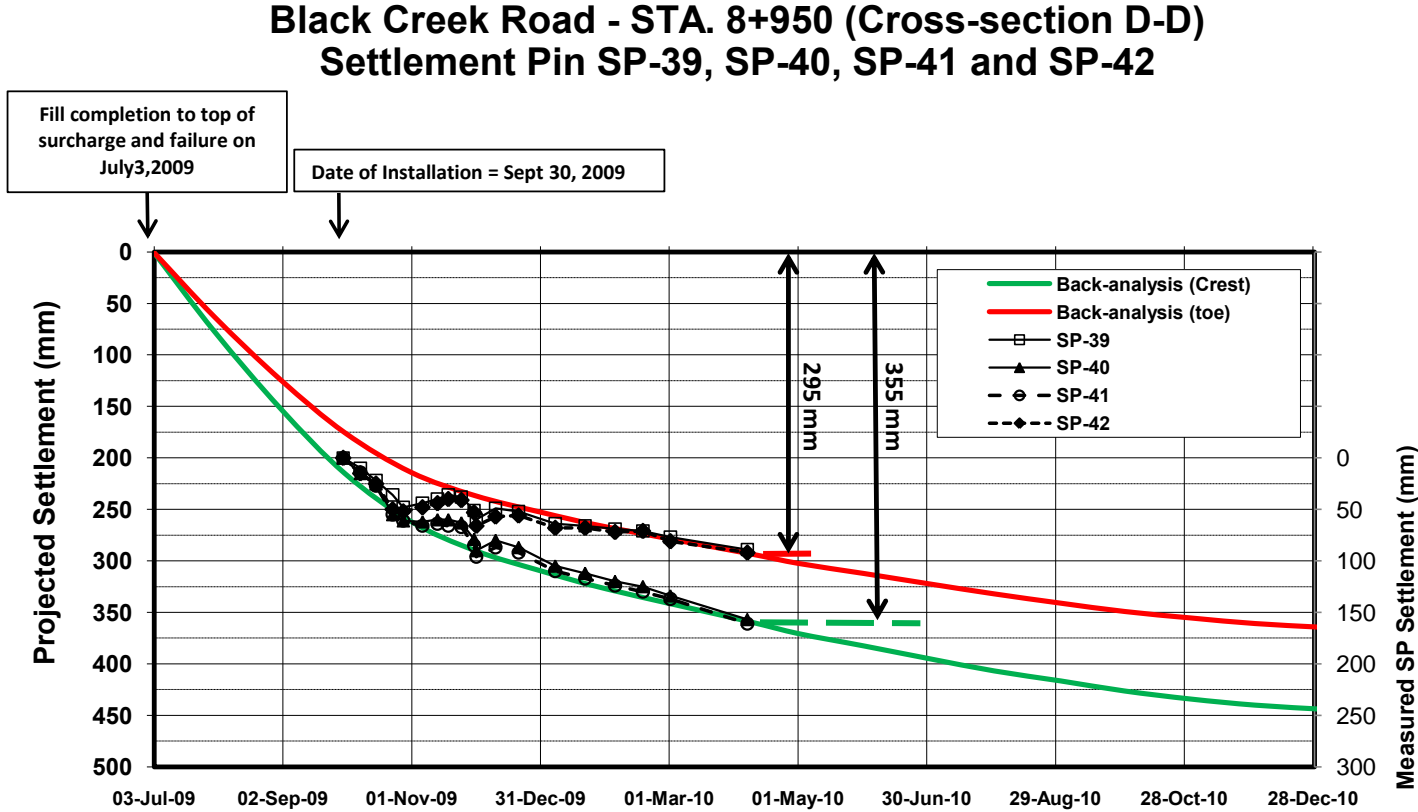


FIGURE 3

19-5161-73 Highway 11 Black Creek Road Embankment Instability MONITORING DATA: SETTLEMENT (SR) vs. TIME PLOTS

Black Creek Road - STA. 9+250 to 9+650 Settlement Rod SR-49 to SR-54

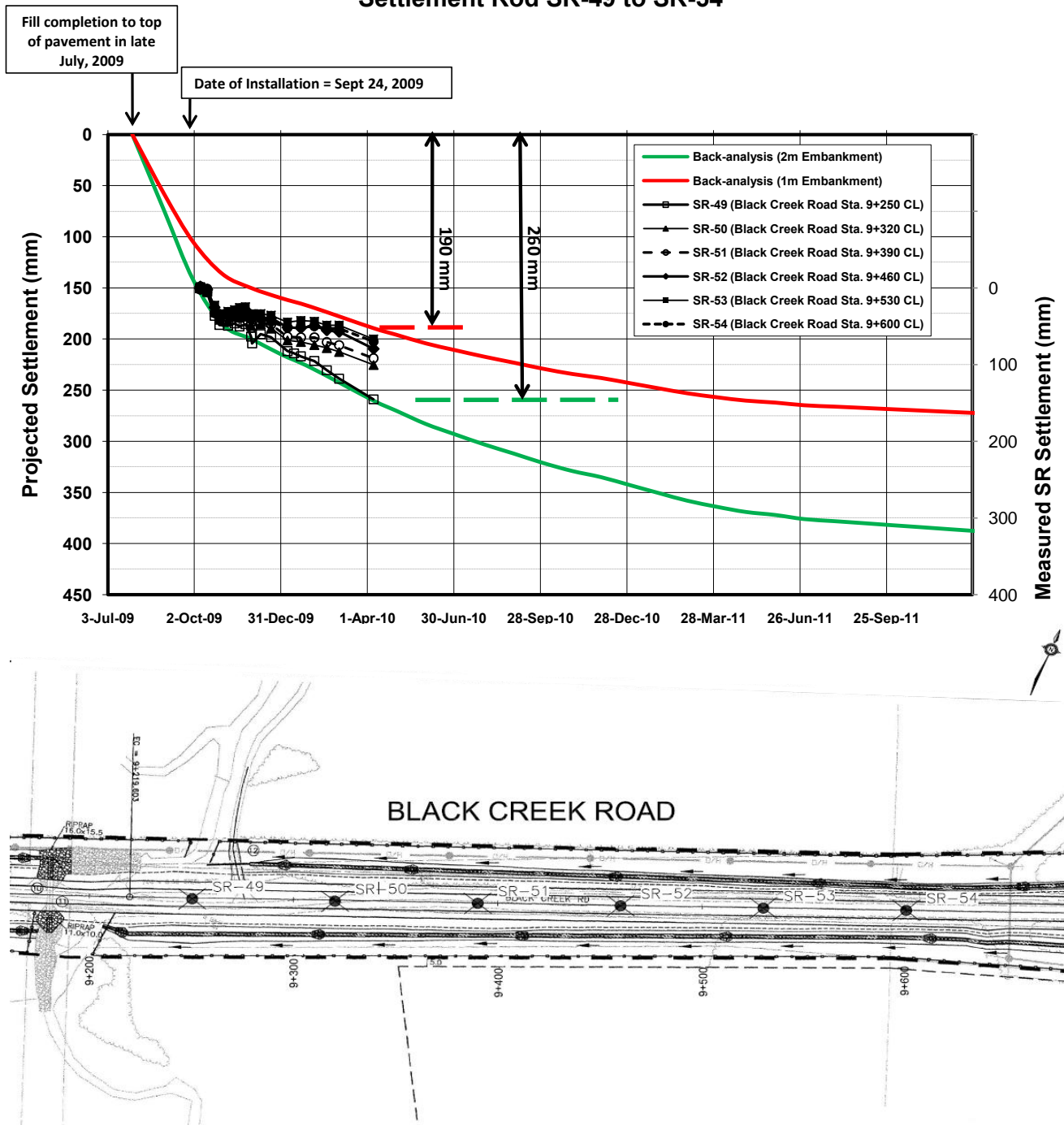


FIGURE 4

**19-5161-73 Highway 11 Black Creek Road Embankment Instability
DAILY RAINFALL IN BURK'S FALLS, ONTARIO (2009 SUMMER)**

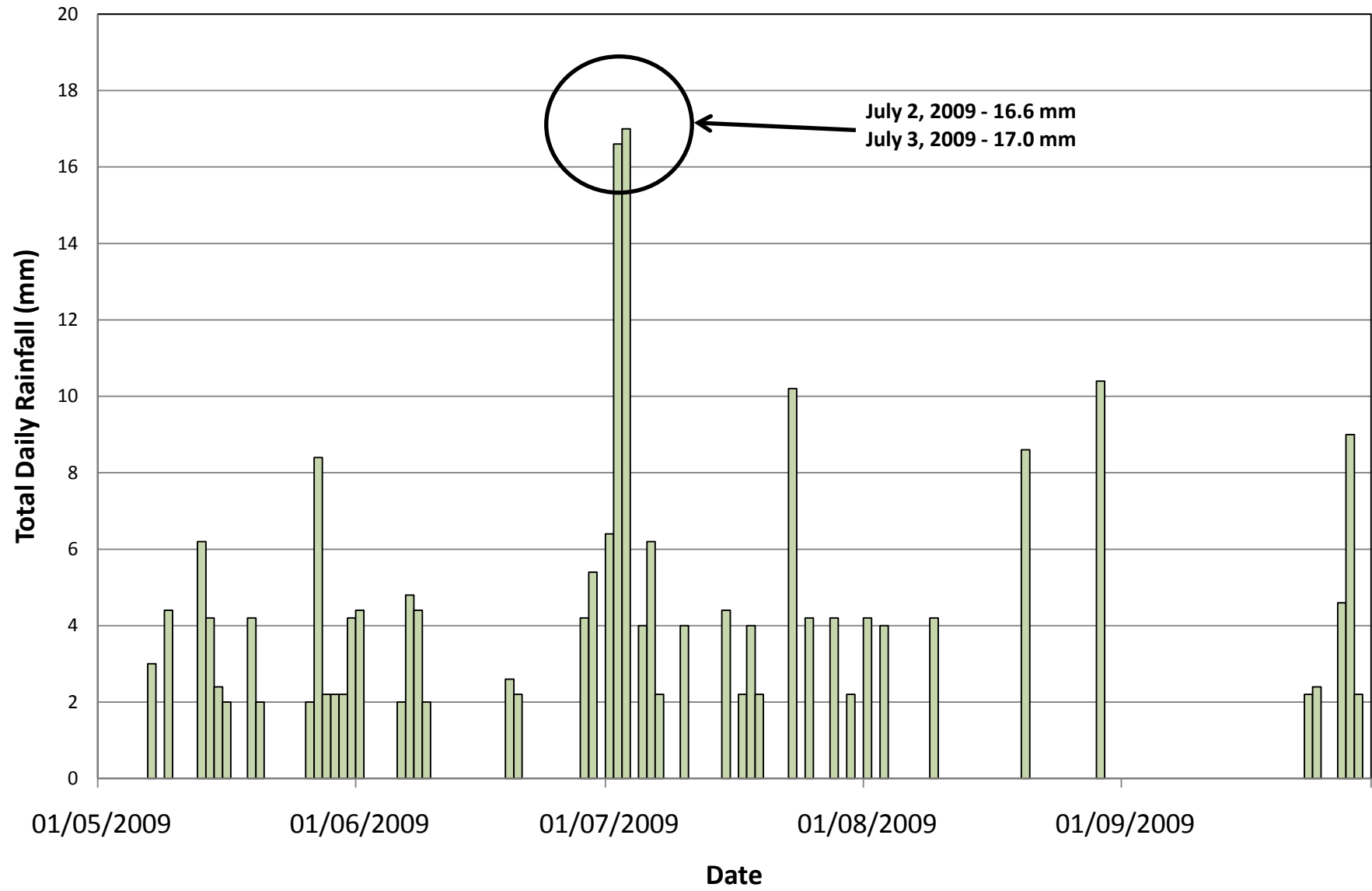


FIGURE 5

Tables

19-5161-73 Highway 11 Black Creek Road Embankment Failure Investigation

SUMMARY OF CONSOLIDATION TEST RESULTS

	Current Investigation		Previous Investigation (*)		
Borehole	BH09-2	BH09-2	BH 24-3	BH 24-3	BH 24-3
Location	Black Creek Road Sta. 8+998		Black Creek Road Sta. 9+625		
Sample No.	ST#1	ST#2	ST#1	ST#2	ST#3
Depth (m)	6.4	9.5	9.4	20.1	30.8
Elevation (m)	308.5	305.4	302.8	292.1	281.4
Soil Type	Silty Clay (CL)	Silty Clay (CL)	Silty Clay (CL)	Silty Clay (CL)	Silty Clay (CL)
Clay Content (%)	34	37	30	26	56
Moisture Content (%)	30.8	41.7	46.6	38.2	57.2
Liquid Limit (%)	31.2	34.3	32.8	33.4	52.0
Plasticity Index (%)	8.4	10.7	10.9	12.2	27.6
γ - Unit Weight (kN/m ³)	18.9	17.6	17.3	18.3	16.4
G _s - Specific Gravity	2.7	2.7	2.78	2.75	2.78
e ₀ - Initial Void Ratio	0.84	1.132	1.28	1.063	1.62
P' ₀ - In situ effective vertical stress (kPa)	60	75	86	171	256
P' _c - Preconsolidation Pressure (kPa)	136	75	86	171	256
OCR - Overconsolidation Ratio	2.3	1.0	1.0	1.0	1.0
C _c - Compression Index	0.166	0.262	0.160	0.225	0.940
C _{ce} - Modified Compression Index	0.090	0.123	0.070	0.109	0.359
C _r - Recompression Index	0.026	0.045	0.040	0.050	0.090
C _{re} - Modified Recompression Index	0.014	0.021	0.018	0.024	0.034

Note (*): Data obtained from Foundation Investigation and Design Report, Embankments along Highway 11 - Sta.13+260 to Sta.14+600, Approach Embankments and Access Ramps, Highway 11, Nurk's Falls to South River, Ontario. July 6, 2004. GWP 742-93-00 WP 757-93-01, Site 44-421, Geocres Number: 31E-234

TABLE 1

19-5161-73 Highway 11 Black Creek Road Embankment Failure Investigation

SUMMARY OF PORE PRESSURE DISSIPATION TESTS

Test Date: October 26 to 28, 2009

CPT Sounding	Ground Surface Elevation	Test Depth (m)	Test Elevation (m)	Test Duration (s)	Estimated Equilibrium (m)	Water Table (m)*	t ₅₀ (s)	c _h (cm ² /min) **	c _h (m ² /yr) **
CPTU09-1	315.61	7.50	308.11	600	3.70	3.80	360	2.9	153.5
		9.95	305.66	570	6.15	3.80	1280	0.9	45.0
		15.35	300.26	840	11.55	3.80	335	3.6	187.5
		24.50	291.11	660	20.70	3.80	120	10.6	556.0
CPTU09-2	312.81	3.00	309.81	1110	2.00	1.00	340	2.9	150.2
		5.00	307.81	990	4.00	1.00	1000	1.0	54.6
		7.95	304.86	960	6.95	1.00	690	1.6	85.2
CPTU09-3	312.57	6.65	305.92	1560	5.89	0.76	870	1.2	65.2
		10.80	301.77	1290	10.04	0.76	445	2.6	137.3
		15.00	297.57	1230	14.24	0.76	280	4.3	226.2
CPTU09-4	312.50	5.80	306.70	1170	5.11	0.69	780	1.3	70.8
		15.20	297.30	1200	14.51	0.69	170	7.1	371.1
CPTU09-5	314.81	8.65	306.16	1140	6.07	2.58	1200	1.0	51.2
		10.40	304.41	870	7.82	2.58	575	2.0	106.8

* Assumed water table based on site observations

**c_h calculations as described by Robertson et al., 1992

TABLE 2

19-5161-73 Highway 11 Black Creek Road Embankment Instability

SUMMARY OF SETTLEMENT ANALYSIS

EXISTING CONDITIONS: NO ADDITIONAL SURCHARGE

Site	Time for Primary Consolidation (months)	Total Settlement from Primary Consolidation (mm)	Settlement to April 7, 2010 (mm)	Remaining Time for Primary Consolidation (months)	Remaining Primary Settlement (mm)	Long Term Settlement (20 years after paving)
Black Creek Rd. 8+925 to 9+060	24	250 to 400	300 to 350	15	0 to 50	100 to 150
Black Creek Rd. 9+240 to 9+650	30	280 to 400	190 to 260	22	90 to 130	110 to 160

CONSTRUCT SURCHARGE TO 1m ABOVE FINAL GRADE

Site	Preload Time for Primary Consolidation * (months)	Total Settlement from Primary Consolidation ** (mm)	Settlement to April 7, 2010 (mm)	Remaining Preload Time for Primary Consolidation (months)	Remaining Primary Settlement (mm)	Long Term Settlement (20 years after paving)
Black Creek Rd. 8+925 to 9+060	18	370 to 450	300 to 350	9	50 to 110	60 to 100
Black Creek Rd. 9+240 to 9+650	21	370 to 450	190 to 260	13.5	150 to 200	70 to 110

CONSTRUCT SURCHARGE TO 2m ABOVE FINAL GRADE

Site	Preload Time for Primary Consolidation* (months)	Total Settlement from Primary Consolidation ** (mm)	Settlement to April 7, 2010 (mm)	Remaining Preload Time for Primary Consolidation (months)	Remaining Primary Settlement (mm)	Long Term Settlement (20 years after paving)
Black Creek Rd. 8+925 to 9+060	12	400 to 550	300 to 360	5.5	100 to 200	40 to 80
Black Creek Rd. 9+240 to 9+650	14	400 to 550	190 to 260	7.5	200 to 300	50 to 80

* Preload time to reach sufficient over-consolidation for reduction of secondary consolidation settlement.

** Calculated 95% primary settlement under full surcharge loads. Surcharge to be removed once monitoring indicates required consolidation is achieved.

TABLE 3

19-5161-73 Highway 11 Black Creek Road Embankment Instability

COMPARISON OF ALTERNATIVES

Mitigation Option	Advantages	Disadvantages	Cost Comparison (*)	Post Construction Settlement (mm) (**)	Risks/ Consequences
<i>Do Nothing Now and Construct Later</i>	<ul style="list-style-type: none"> Maintain some surcharge at the instability site Stability is adequate, toe berm and reinforcement not required 	<ul style="list-style-type: none"> ~1 year of waiting time is required for primary consolidation 	1	A – 60 to 100 mm B – 150 to 220 mm	Low to Moderate (***)
<i>Construct Now with No Surcharge</i>	<ul style="list-style-type: none"> Stability is adequate, toe berm and reinforcement not required 	<ul style="list-style-type: none"> ~ 15 to 22 months of waiting time is required for primary consolidation 	2	A - 100 to 150 mm B - 110 to 160 mm	Moderate (***)
<i>Surcharging and Berm Construction</i>	<ul style="list-style-type: none"> Reduced long term settlement 	<ul style="list-style-type: none"> ~5.5 to 13.5 months of waiting time is required for primary consolidation depending on surcharge effort Disposal of berm material after surcharging period Property available for berm construction may be limited 	3	A – 40 to 100 mm B – 50 to 110 mm	Low to Moderate (***)
<i>Surcharging and Geogrid Reinforcement</i>	<ul style="list-style-type: none"> Reduce long term settlement Smaller embankment footprint possible 	<ul style="list-style-type: none"> ~5.5 to 13.5 months of waiting time is required for primary consolidation depending on surcharge effort More costly than berm option 	4	A – 40 to 100 mm B – 50 to 110 mm	Low to Moderate (***)
<i>Wick Drain and Surcharging</i>	<ul style="list-style-type: none"> Reduce preload period Reduce long term settlement with surcharge 	<ul style="list-style-type: none"> Platform construction for wick drain installation is required 	5	A – 40 to 100 mm B – 50 to 110 mm	Not Significant (***)
<i>Light Weight Fill</i>	<ul style="list-style-type: none"> Reduce preload period Reduce long term settlement with no surcharge Stability is adequate, toe berm and reinforcement not required 	<ul style="list-style-type: none"> Buoyancy may be a problem for EPS installed below water-table. Additional analysis is required for design of light weight fill layout 	6	A – 20 to 50 mm B – 30 to 60 mm	Not Significant

Note (*): 1 (Least expensive) to 6 (Most expensive)

(): A - Black Creek Road Sta. 8+925 to Sta. 9+060 (Site of Instability), B - Black Creek Road Sta. 9+240 to Sta. 9+650 (East of Stirling Creek)**

(*): Monitoring is required prior to paving to evaluate magnitude of post-construction settlement.**

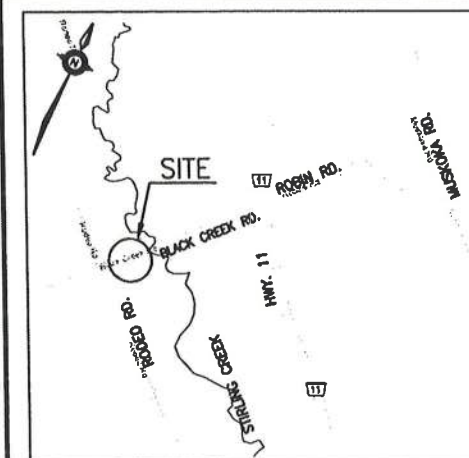
TABLE 4

Drawings

SHEET
1



THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS



LEGEND

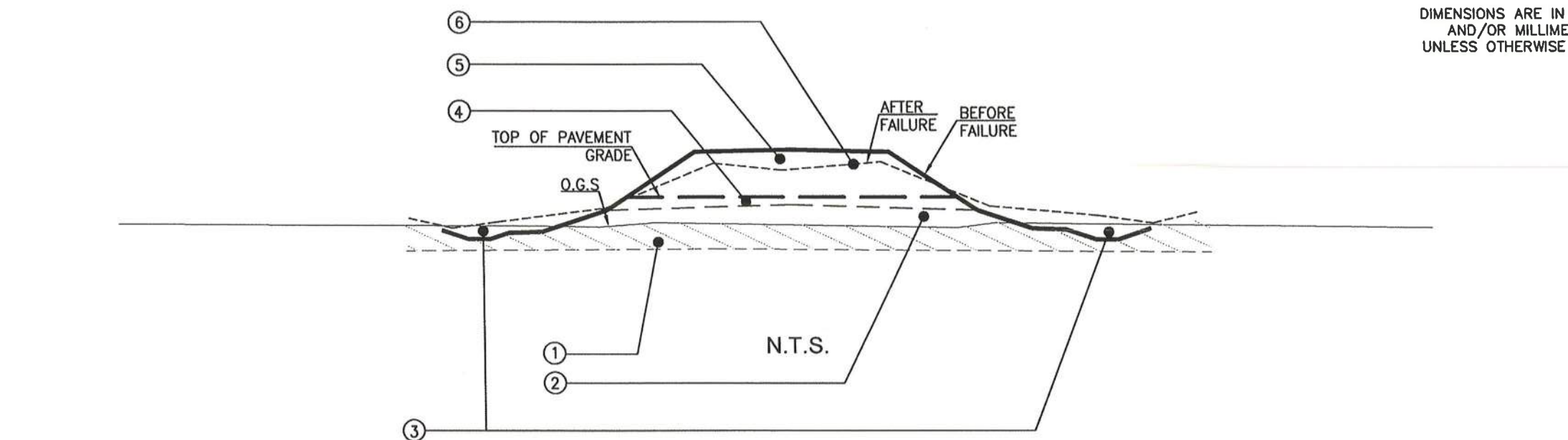


Borehole (BH)
Cone Penetration Test (CPTU)
Settlement Pin (SP)
Failure Area

NO	ELEVATION	NORTHING	EASTING
BH09-1	315.61	5 062 734.6	308 468.3
BH09-2	314.93	5 062 714.9	308 406.1
BH09-3	312.61	5 062 691.6	308 386.5
CPTU09-1	315.61	5 062 734.6	308 468.3
CPTU09-2	312.81	5 062 721.7	308 404.9
CPTU09-3	312.57	5 062 702.9	308 416.0
CPTU09-4	312.50	5 062 713.8	308 380.3
CPTU09-5	314.81	5 062 700.4	308 385.1
SP39	312.60	5 062 684.2	308 368.1
SP40	315.04	5 062 691.5	308 363.1
SP41	314.82	5 062 697.5	308 360.8
SP42	312.47	5 062 705.4	308 358.3
SP43	312.38	5 062 700.8	308 413.2
SP44	315.01	5 062 709.2	308 410.5
SP45	315.02	5 062 714.8	308 408.4

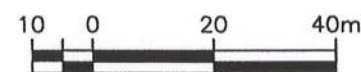
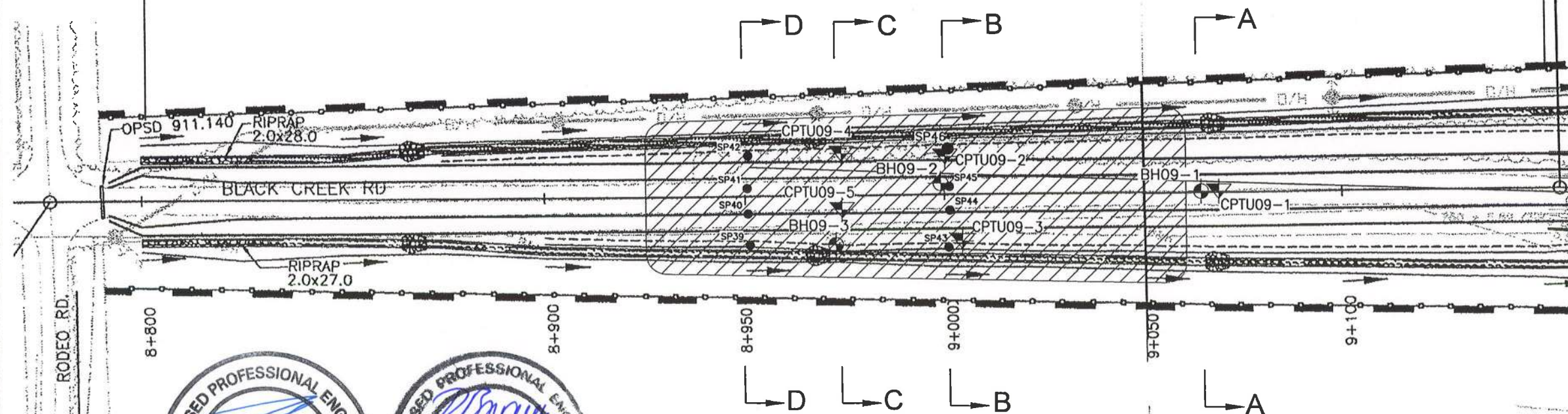
GEOCRES No.

REVISES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
---------	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--



DATE/TIME

	ACTIVITY	DATE/TIME
①	REMOVAL OF ORGANIC SOIL IMMEDIATELY FOLLOWED BY PLACEMENT OF GRAN. B TYPE 2 BACKFILL	DEC. 03, 2008 TO DEC. 17, 2008
②	SELECT SUBGRADE MATERIAL (SSM) PLACEMENT AND COMPACTION	JUNE 01, 2009 TO JUNE 8, 2009
③	DITCH EXCAVATION IN GRAN. B TYPE 2 BACKFILL	JUNE 09, 2009 TO JUNE 15, 2009
④	GRAN. B TYPE 2 PLACEMENT & COMPACTION UP TO PAVEMENT GRADE	JUNE 22, 2009 TO JUNE 25, 2009
⑤	GRAN. B TYPE 2 SURCHARGE PLACEMENT (BY END DUMPING) WAS COMPLETED ON FRIDAY JULY 03, 2009 AT APPROX. 2:30PM.	JUNE 26, 2009 TO JULY 03, 2009 (~ 2:30PM)
⑥	THE EMBANKMENT BETWEEN STA. 8+925 & STA. 9+060 FAILED ON FRIDAY, JULY 3, 2009 AT APPROX. 4:00PM	JULY 03, 2009 (~ 4:00PM)



SCALE: 1:1250

SITE PLAN



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

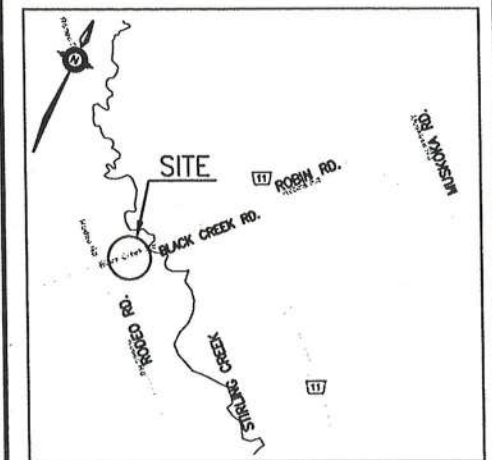
CONT No 2007-5188
GWP No 5079-06-00



SHEET
2

HWY 11, BLACK CREEK RD.
EMBANKMENT FAILURE INVESTIGATION
EMBANKMENT CROSS-SECTION &
SOIL STRATA - 1

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS



KEYPLAN

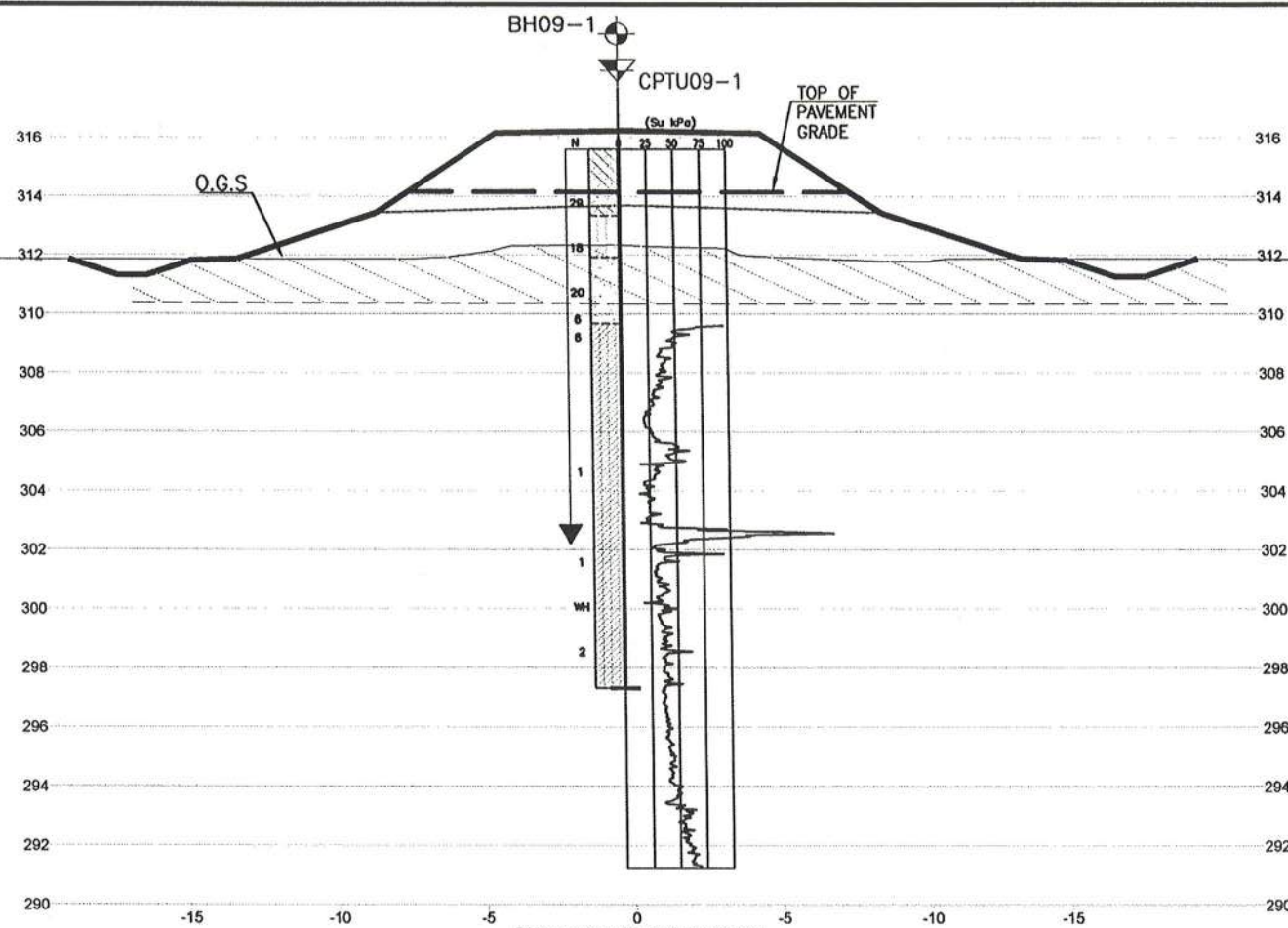
LEGEND

- Borehole (BH)
- Settlement Pin (SP)
- Cone Penetration Test (CPTU)
- Vibrating Wire Piezometer
- Original Ground Surface
- Surcharge
- Select Subgrade Material (S.S.M.)
- Backfill
- Water Level

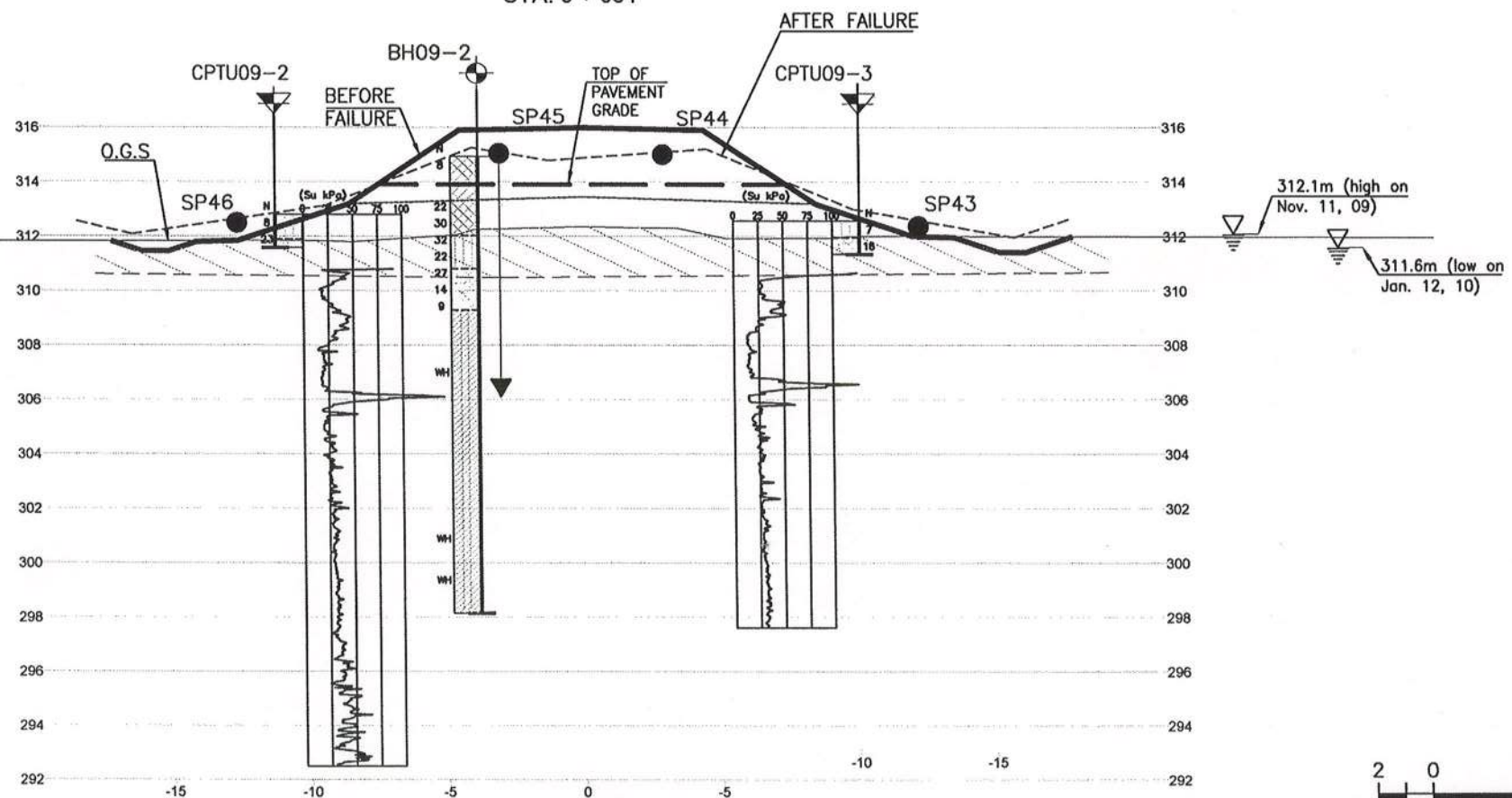
NO	ELEVATION	NORTHING	EASTING
BH09-1	315.61	5 062 734.6	308 468.3
BH09-2	314.93	5 062 714.9	308 406.1
BH09-3	312.61	5 062 691.6	308 386.5
CPTU09-1	315.61	5 062 734.6	308 468.3
CPTU09-2	312.81	5 062 721.7	308 404.9
CPTU09-3	312.57	5 062 702.9	308 416.0
CPTU09-4	312.50	5 062 713.8	308 380.3
CPTU09-5	314.81	5 062 700.4	308 385.1
SP39	312.60	5 062 684.2	308 366.1
SP40	315.04	5 062 691.5	308 363.1
SP41	314.82	5 062 697.5	308 360.8
SP42	312.47	5 062 705.4	308 358.3
SP43	312.38	5 062 700.8	308 413.2
SP44	315.01	5 062 709.2	308 410.5
SP45	315.02	5 062 714.8	308 408.4

GEOCRES No.

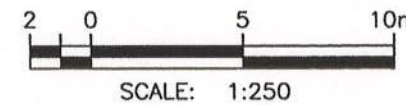
DATE	BY	DESCRIPTION
DESIGN	JPL	CHK JPL
DRAWN	AN	CHK
CODE	JPL	CODE
LOAD	JPL	LOAD
DATE	JPL	DATE
MAR. 2010	JPL	MAR. 2010
STRUCT	JPL	STRUCT
DWG 2	JPL	DWG 2



CROSS SECTION A - A
STA. 9 + 064



CROSS SECTION B - B
STA. 9+000



SCALE: 1:250

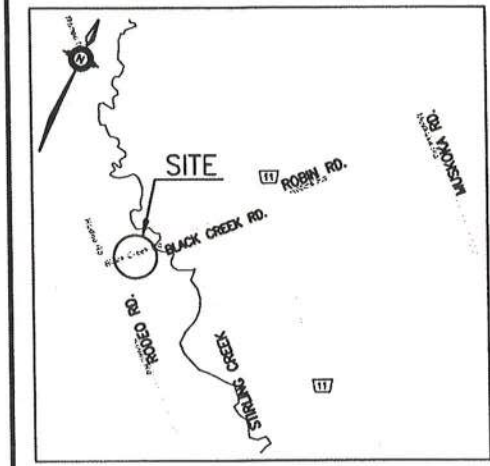
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No 2007-5188
GWP No 5079-06-00

HWY 11, BLACK CREEK RD.
EMBANKMENT FAILURE INVESTIGATION
EMBANKMENT CROSS-SECTION &
SOIL STRATA - 2

SHEET
3

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS



KEYPLAN

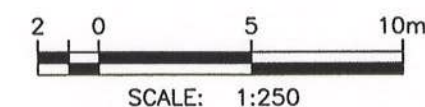
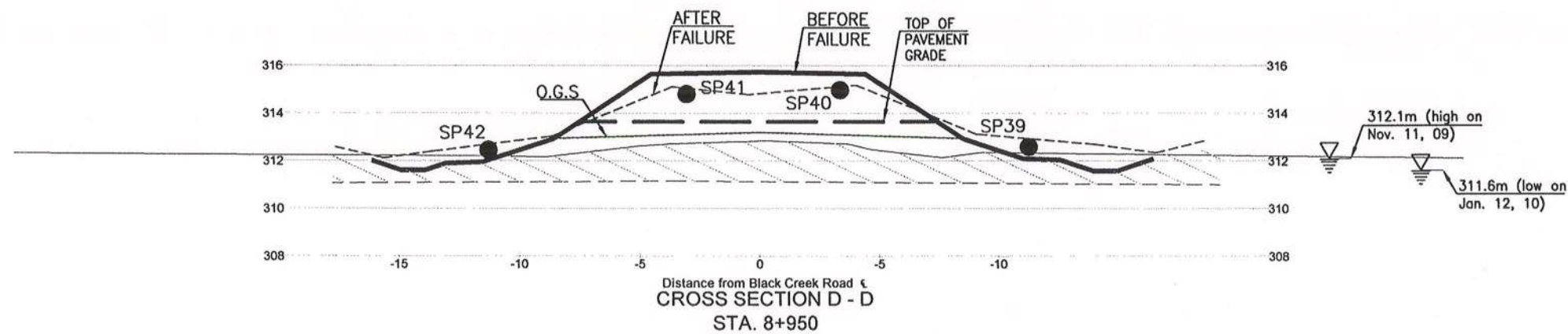
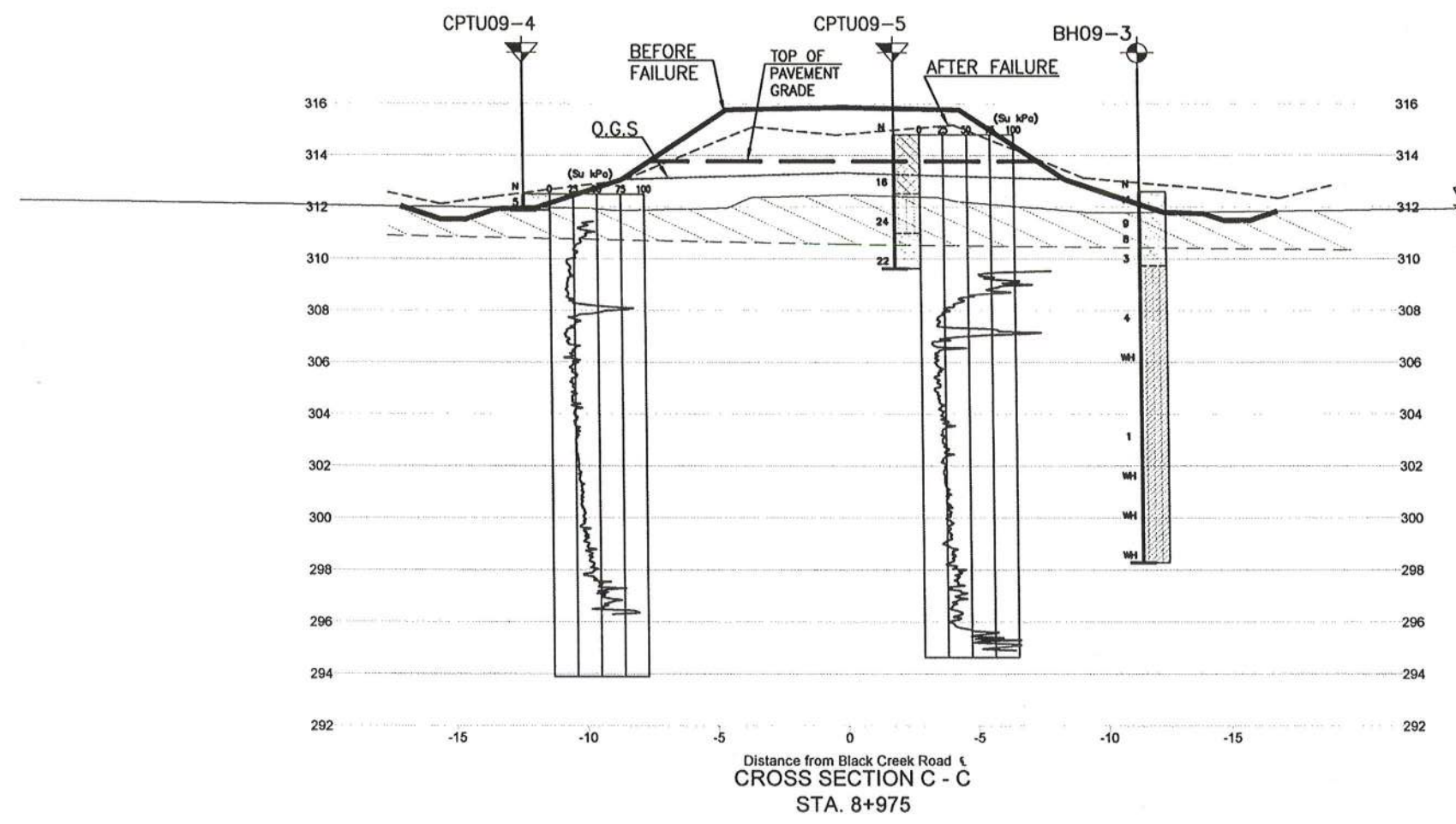
LEGEND

- Borehole (BH)
- Settlement Pin (SP)
- Cone Penetration Test (CPTU)
- ▼ Vibrating Wire Piezometer
- O.G.S. Original Ground Surface
- ▨ Surcharge
- ▨ Select Subgrade Material (S.S.M.)
- ▨ Backfill
- ▽ Water Level

NO	ELEVATION	NORTHING	EASTING
BH09-1	315.61	5 062 734.6	308 468.3
BH09-2	314.93	5 062 714.9	308 406.1
BH09-3	312.61	5 062 691.6	308 386.5
CPTU09-1	315.61	5 062 734.6	308 468.3
CPTU09-2	312.81	5 062 721.7	308 404.9
CPTU09-3	312.57	5 062 702.9	308 416.0
CPTU09-4	312.50	5 062 713.8	308 380.3
CPTU09-5	314.81	5 062 700.4	308 385.1
SP39	312.60	5 062 684.2	308 366.1
SP40	315.04	5 062 691.5	308 363.1
SP41	314.82	5 062 697.5	308 360.8
SP42	312.47	5 062 705.4	308 358.3
SP43	312.38	5 062 700.8	308 413.2
SP44	315.01	5 062 709.2	308 410.5
SP45	315.02	5 062 714.8	308 408.4

GEOCRES No.

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	JPL	CHK JPL	CODE [LOAD] DATE MAR. 2010
DRAWN	AN	CHK	SITE [STRUCT] DWG 3



Appendix A
Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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



Water Level

C_{pen}

Shear Strength Determination by Pocket Penetrometer

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

UNIFIED SOILS CLASSIFICATION







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

RECORD OF BOREHOLE No 09-01

1 OF 2

METRIC

G.W.P. 5079-06-00 LOCATION N 5 062 734.62 E 308 468.32 (Black Creek Rd. Sta. 9+063.4 O/S Left 0.9m) ORIGINATED BY TJH
 HWY 11, Black Creek Rd. BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2009-10-26 - 2009-10-29 CHECKED BY JPL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
315.6							20 40 60 80 100						
0.0	SAND and GRAVEL , trace silt Compact Grey Dry to Moist (Gran. B Type 2 Surcharge FILL)						20 40 60 80 100						
313.3			1	SS	29		20 40 60 80 100						
2.3	SAND , trace to some gravel, trace to some silt Compact Brown Moist to Wet (Select Subgrade Material SSM FILL)						20 40 60 80 100						
311.9			2	SS	18		20 40 60 80 100						
3.7	SAND and GRAVEL , trace to some silt Compact to Loose Grey Moist to Wet (Gran. B Type 2 Backfill)						20 40 60 80 100						
309.7			3	SS	20		20 40 60 80 100					49 43 8 (SI+CL)	
5.9	Silty CLAY , trace sand Firm to Stiff Grey Moist to Wet						20 40 60 80 100						
	Becoming firm		4	SS	6		20 40 60 80 100					Low Recovery	
							20 40 60 80 100						
			1	ST			20 40 60 80 100					No Recovery	
							20 40 60 80 100						
			2	ST			20 40 60 80 100					0 0 60 40	
							20 40 60 80 100						

Continued Next Page

+³, X³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 09-01

2 OF 2

METRIC

G.W.P. 5079-06-00 LOCATION N 5 062 734.62 E 308 468.32 (Black Creek Rd. Sta. 9+063.4 O/S Left 0.9m) ORIGINATED BY TJH
 HWY 11, Black Creek Rd. BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2009-10-26 - 2009-10-29 CHECKED BY JPL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	
	Continued From Previous Page							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					
	Silty CLAY, trace sand Firm Grey Moist to Wet												
			6	SS	1		305						0 2 67 31
							304						
			3	ST			303						
							302						
			7	SS	1		301						
							300						
			8	SS	WH		299						
							298						
297.3													
18.3	END OF BOREHOLE AT 18.3m. VIBRATING WIRE PIEZOMETER (VWP-1) INSTALLED AT 13.4m. BOREHOLE BACKFILLED WITH BENTONITE GROUT TO 5.9m, THEN GRANULAR CUTTINGS TO SURFACE.												

ONTMT4S 6173.GPJ 1/14/10

+³ ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 09-02

1 OF 2

METRIC

G.W.P. 5079-06-00 LOCATION N 5 062 714.87 E 308 406.12 (Black Creek Rd. Sta. 8+998.2 O/S Left 3.7m) ORIGINATED BY TJH
 HWY 11, Black Creek Rd. BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2009-10-28 - 2009-10-28 CHECKED BY JPL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
314.9							20 40 60 80 100								
0.0	SAND and GRAVEL , trace silt Loose to Compact Grey Dry to Moist (Gran. B Type 2 Surcharge FILL)		1	SS	8										
				2	SS	22									
				3	SS	30									
312.0															
2.9	SAND , trace to some gravel, some silt Compact to Dense Brown Moist to Wet (Select Subgrade Material SSM FILL)		4	SS	32									20 70 10 (SI+CL)	
				5	SS	22									3 77 20 (SI+CL)
310.8															
4.1	SAND and GRAVEL , trace to some silt Compact to Loose Grey Moist to Wet (Gran. B Type 2 Backfill)		6	SS	27									38 48 14 (SI+CL)	
				7	SS	14									59 33 8 (SI+CL)
				8	SS	9									62 33 5 (SI+CL)
309.3														0 1 63 36	
5.6	Silty CLAY , trace sand Firm to Stiff Grey Moist to Wet														
				1	ST										0 1 65 34
															OED: e _s =0.840 P' _v =136 kPa C _c =0.166 C _u =0.026 G _s =2.70 OCR=2.3
	Becoming firm			9	SS	WH									0 2 58 40

Continued Next Page

+³, x³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

METRIC

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

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 09-03

2 OF 2

METRIC

G.W.P. 5079-06-00 LOCATION N 5 062 691.57 E 308 386.51 (Black Creek Rd. Sta. 8+971.8 O/S Right 11.4m) ORIGINATED BY TJH
HWY 11, Black Creek Rd. BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2009-10-28 - 2009-10-28 CHECKED BY JPL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
	Continued From Previous Page													
	Silty CLAY, trace sand Soft to Firm Grey Moist													
			8	SS	WH		302							
							301							
			9	SS	WH		300							
							299							
298.3			10	SS	WH									
14.3	END OF BOREHOLE AT 14.3m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Oct. 30, 09 0.8 311.8 Nov. 11, 09 0.5 312.1 Nov. 20, 09 0.8 311.8 Dec. 08, 09 0.7 311.9 Dec. 17, 09 0.8 311.8 Dec. 23, 09 0.9 311.7 Jan. 12, 10 1.0 311.6													

+³, X³: Numbers refer to
Sensitivity




20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No CPTU09-02

1 OF 1

METRIC

G.W.P. 5079-06-00 LOCATION N 5 062 721.65 E 308 404.96 (Black Creek Rd. Sta. 8+999.5 O/S Left 10.5m) ORIGINATED BY TJH
HWY 11, Black Creek Rd. BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2009-10-27 - 2009-10-27 CHECKED BY JPL


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 γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE									
312.8								20	40	60	80	100					
0.0	SAND , trace to some gravel, trace to some silt Loose to Compact Brown Moist to Wet (Select Subgrade Material SSM FILL)		1	SS	8												
311.9			2	SS	23		312										
0.9	SAND and GRAVEL , trace to some silt																
311.6																	
1.2	Compact Grey Moist to Wet (Gran. B Type 2 Backfill) BEGIN CONE PENETRATION TESTING AT 1.2m (SEE CPTU LOGS).																

RECORD OF BOREHOLE No CPTU09-03

1 OF 1

METRIC

G.W.P. 5079-06-00 LOCATION N 5 062 702.94 E 308 416.01 (Black Creek Rd. Sta. 9+003.4 O/S Right 10.9m) ORIGINATED BY TJH
HWY 11, Black Creek Rd. BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2009-10-28 - 2009-10-28 CHECKED BY JPL

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 γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
312.6								20	40	60	80	100					
0.0	SAND , trace to some gravel, trace to some silt Loose Brown		1	SS	7	312											
312.0	Moist to Wet (Select Subgrade Material SSM FILL)		2	SS	16												
0.6	SAND and GRAVEL , trace to some silt																Low Recovery
311.4	Compact Grey Moist to Wet (Gran. B Type 2 Backfill)																
1.2	BEGIN CONE PENETRATION TESTING AT 1.2m (SEE CPTU LOGS).																

RECORD OF BOREHOLE No CPTU09-04

1 OF 1

METRIC

G.W.P. 5079-06-00 LOCATION N 5 062 713.79 E 308 380.29 (Black Creek Rd. Sta. 8+973.6 O/S Left 11.6m) ORIGINATED BY TJH
HWY 11, Black Creek Rd. BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2009-10-27 - 2009-10-27 CHECKED BY JPL

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
						20	40	60	80	100	W _p	W	W _L			
312.5																
0.0	SAND and GRAVEL, trace to some silt Loose Grey		1	SS	5											
311.9	Moist to Wet (Gran. B Type 2 Backfill)															
0.6	BEGIN CONE PENETRATION TESTING AT 0.6m (SEE CPTU LOGS).															

ONTMT4S 6173.GPJ 1/12/10

+³, X³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

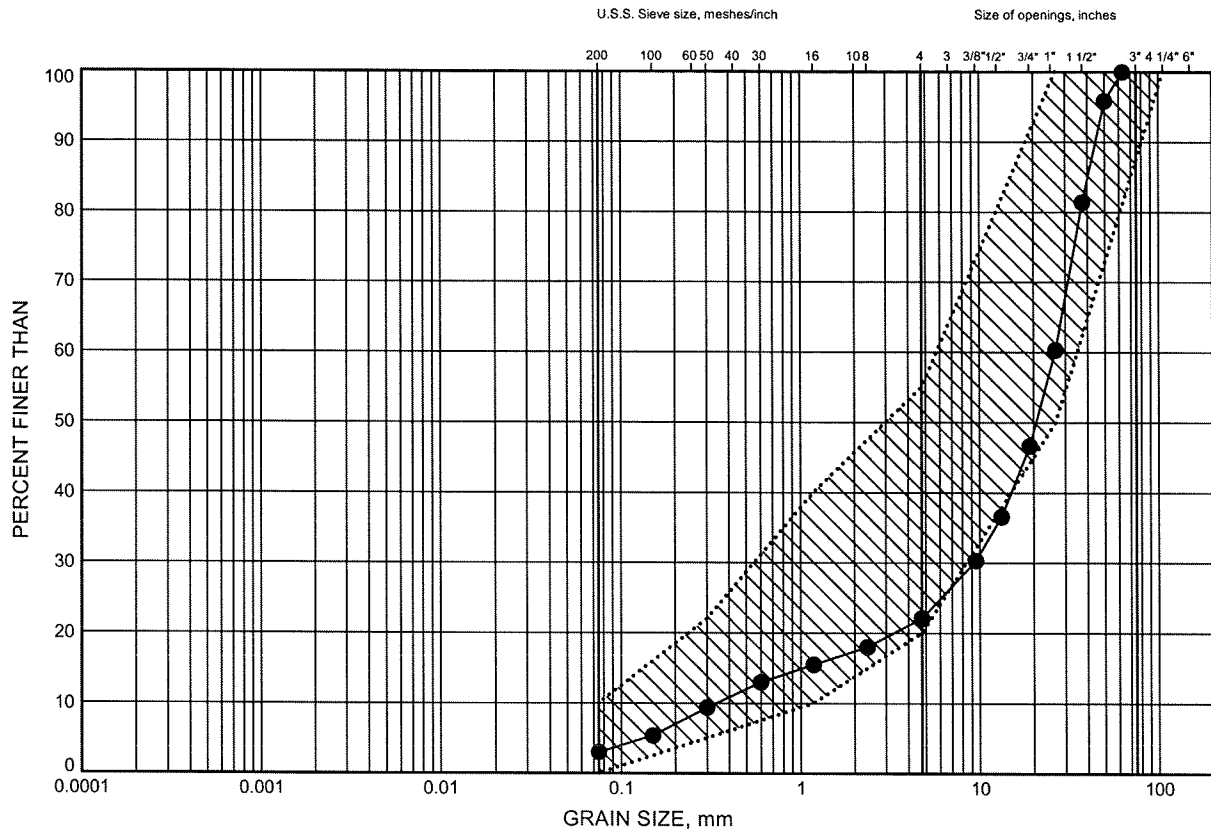
Appendix B
Laboratory Test Results

Investigation of Embankment Failure GRAIN SIZE DISTRIBUTION

FIGURE B1

SURCHARGE FILL

..... GRAN. B, TYPE II



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

LEGEND

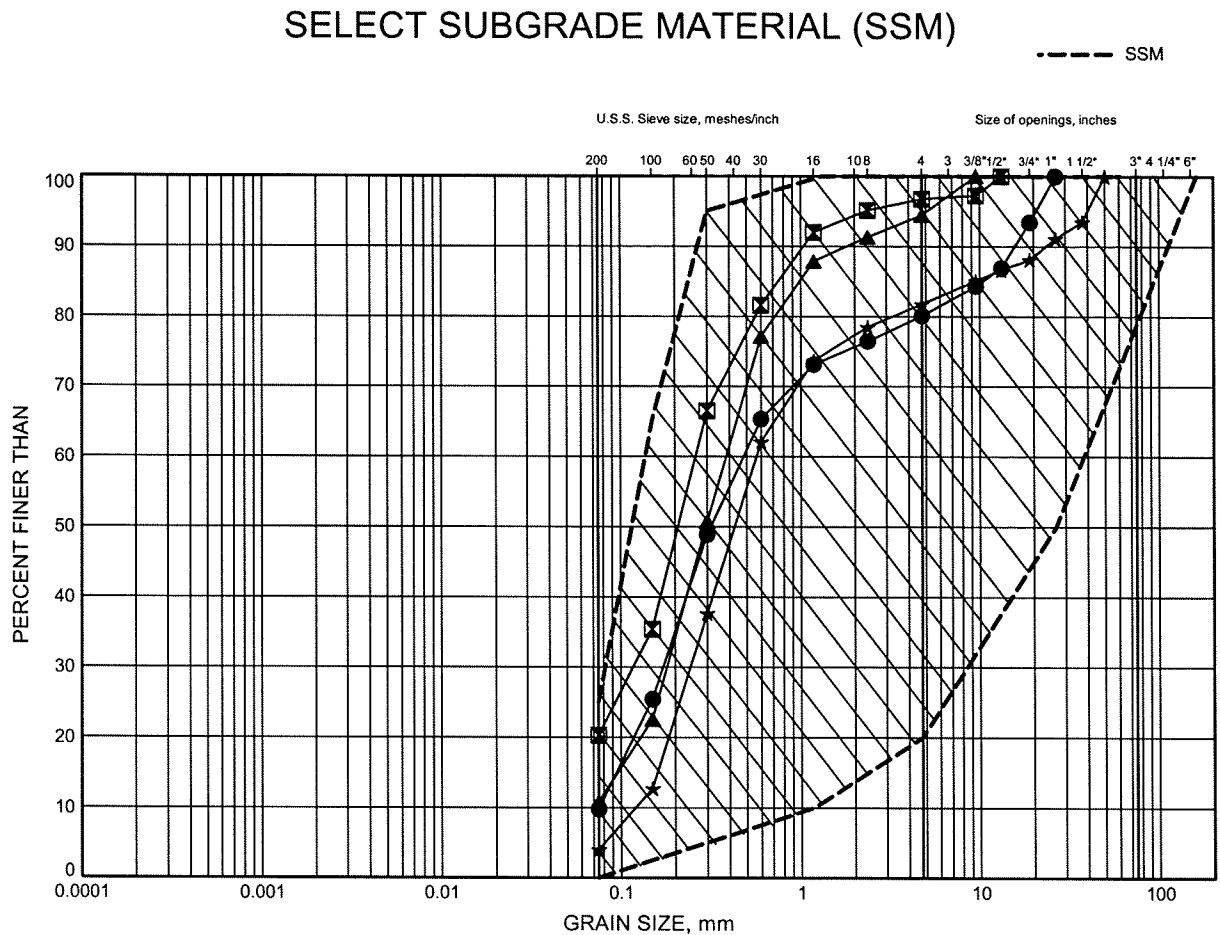
SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	GS#2	0.00	314.80



W.P.# 5079-06-00.....
Prepared By .AN.....
Checked By .JPL.....

Investigation of Embankment Failure GRAIN SIZE DISTRIBUTION

FIGURE B2



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-02	3.05	311.88
⊠	09-02	3.66	311.27
▲	CPTU09-05	3.35	311.46
★	GS#3	0.00	311.20



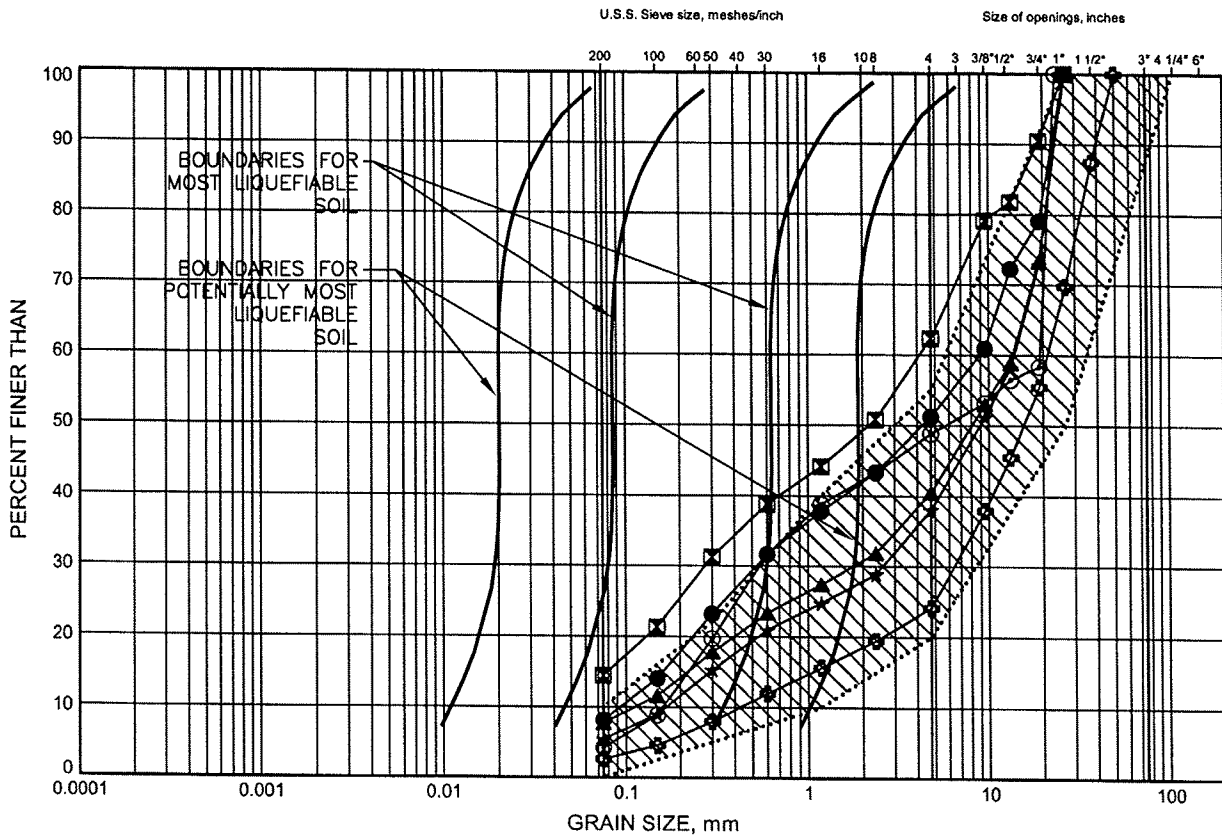
W.P.# 5079-06:00
Prepared By AN
Checked By JPL

Investigation of Embankment Failure GRAIN SIZE DISTRIBUTION

FIGURE B3

BACKFILL

..... GRAN. B, TYPE II



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-01	4.88	310.73
⊠	09-02	4.27	310.66
▲	09-02	4.88	310.05
★	09-02	5.41	309.52
⊙	CPTU09-05	4.88	309.93
⊛	GS#4	0.00	310.90

GRAIN SIZE DISTRIBUTION - THURBER 6173.GPJ 2/2/10

W.P.# .5079-06-00.....
Prepared By .AN.....
Checked By .JPL.....

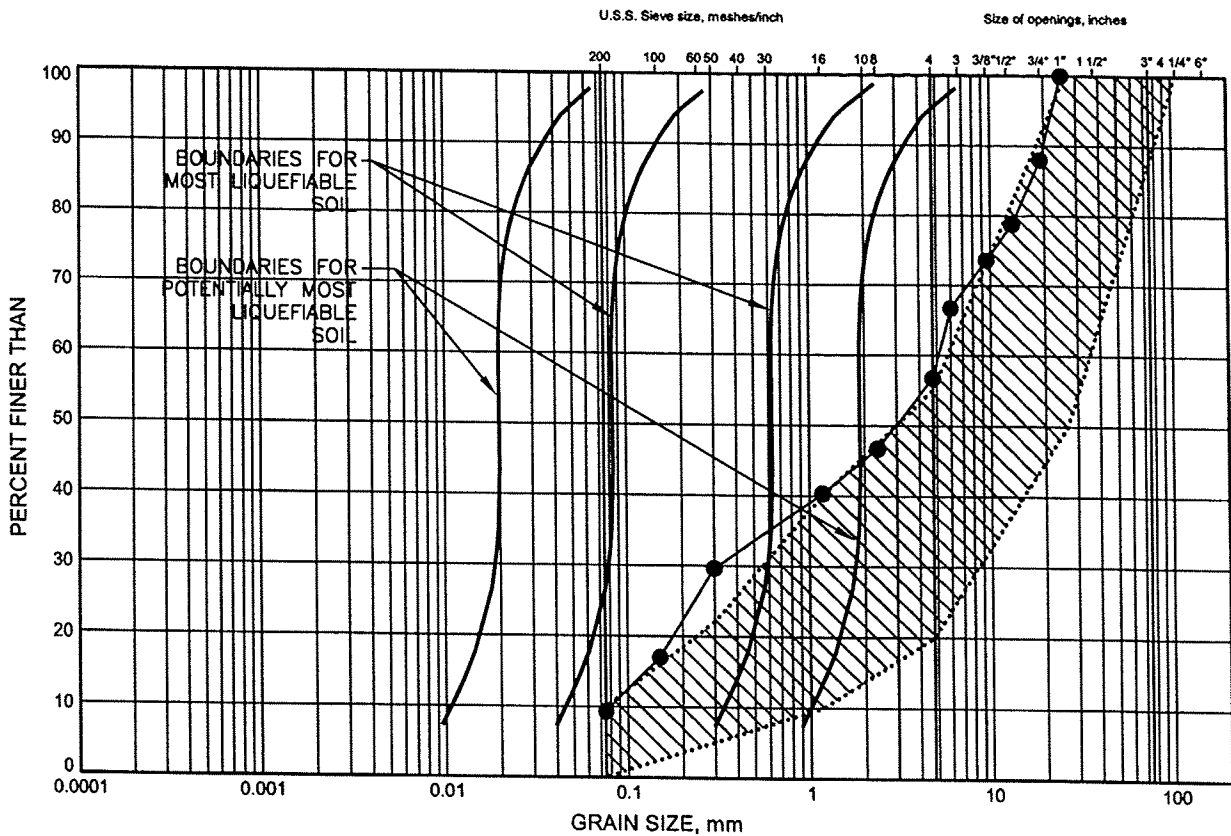


Investigation of Embankment Failure GRAIN SIZE DISTRIBUTION

FIGURE B4

BACKFILL (SAND BOILS)

..... GRAN. B, TYPE II



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	GS#1	0.00	312.30

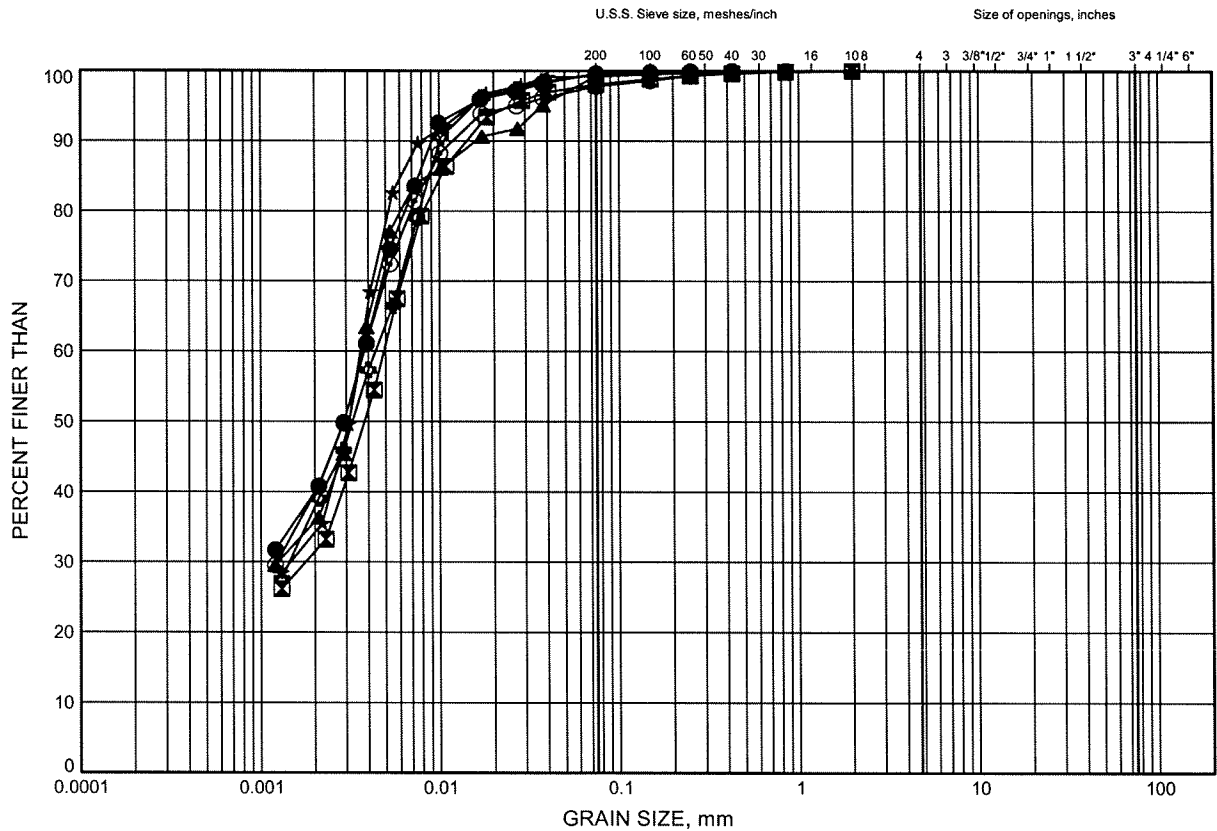


W.P.# .5079-06-00.....
Prepared By .AN.....
Checked By .JPL.....

Investigation of Embankment Failure GRAIN SIZE DISTRIBUTION

FIGURE B5

SILTY CLAY (CL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-01	9.45	306.16
⊠	09-01	10.97	304.64
▲	09-02	5.72	309.21
★	09-02	6.40	308.53
⊙	09-02	7.92	307.00
⊕	09-02	9.45	305.48

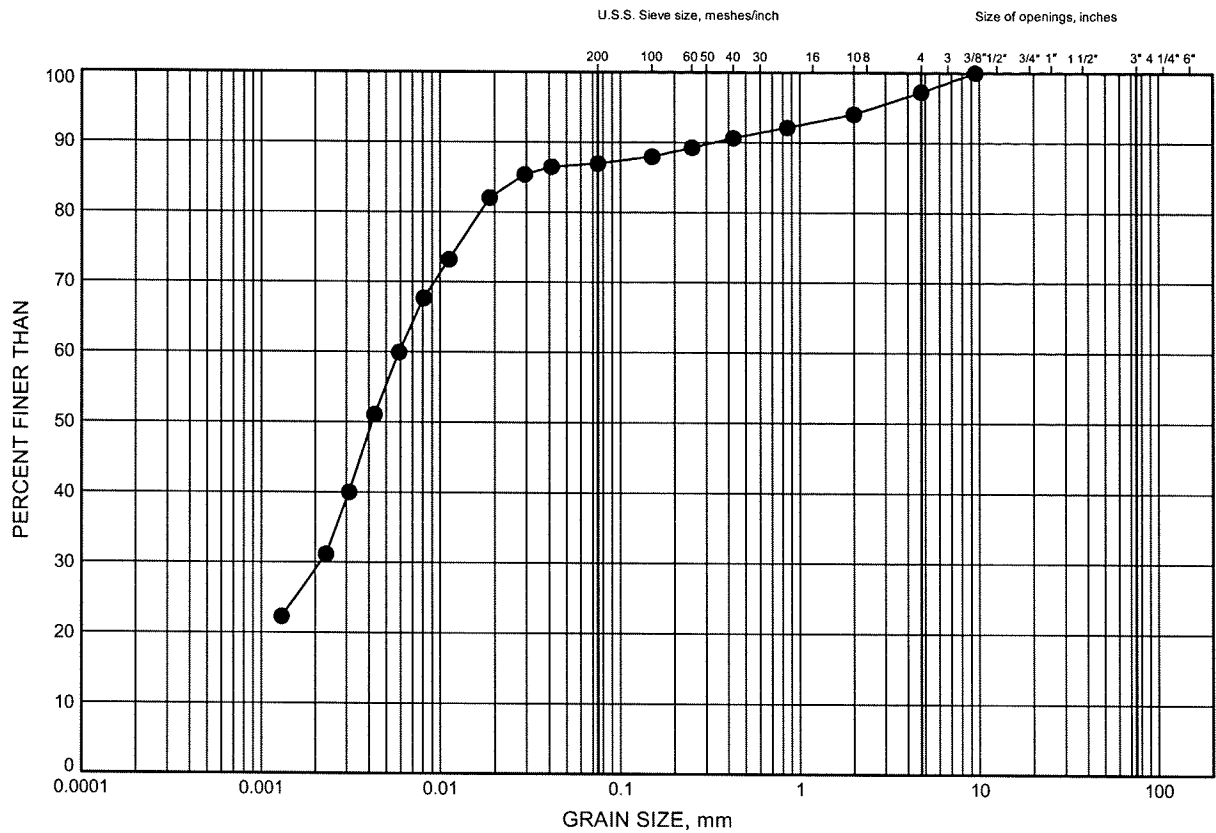


W.P.# 5079-06-00.....
Prepared By .AN.....
Checked By .JPL.....

Investigation of Embankment Failure GRAIN SIZE DISTRIBUTION

FIGURE B6

SILTY CLAY (CL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-03	4.88	307.73

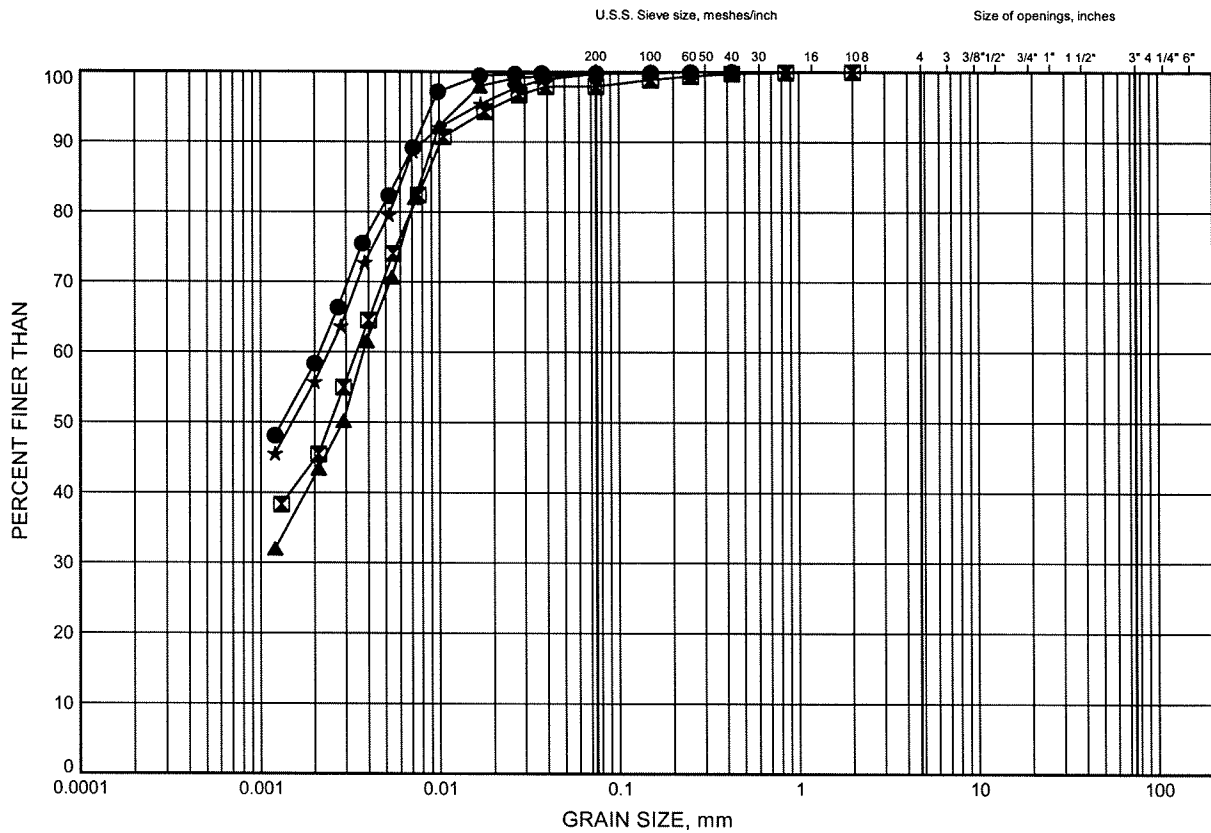


W.P.# .5079-06-00.....
Prepared By .AN.....
Checked By .JPL.....

Investigation of Embankment Failure GRAIN SIZE DISTRIBUTION

FIGURE B7

SILTY CLAY (CI)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-02	14.02	300.91
⊠	09-03	6.40	306.21
▲	09-03	7.92	304.69
★	09-03	9.45	303.16

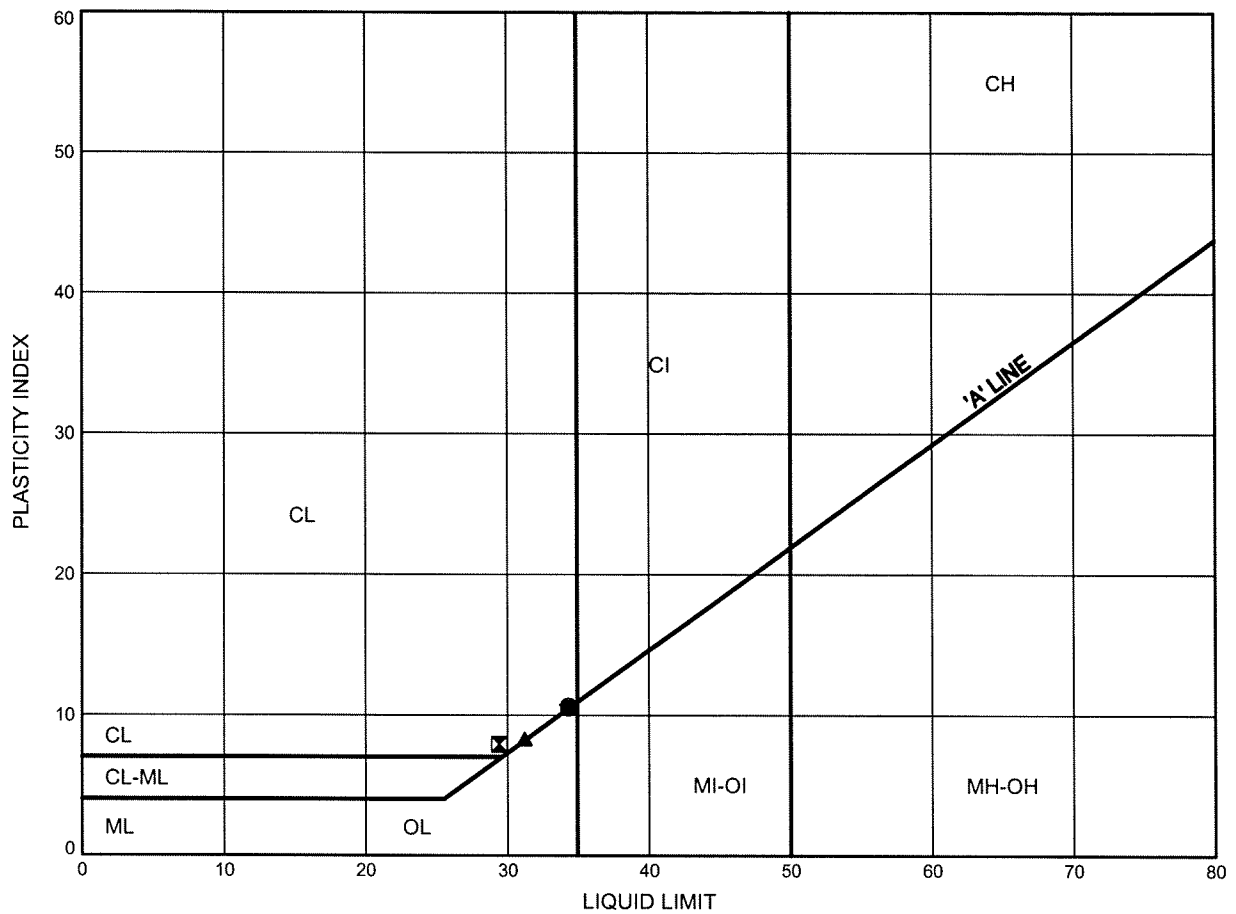


W.P.# 5079-06-00
Prepared By AN
Checked By JPL

Investigation of Embankment Failure
ATTERBERG LIMITS TEST RESULTS

FIGURE B8

SILTY CLAY (CL)



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	09-01	9.45	306.16
⊠	09-01	10.97	304.64
▲	09-02	6.40	308.53
★	09-02	7.92	307.00
⊙	09-02	9.45	305.48

Date February 2010
 Project 5079-06-00

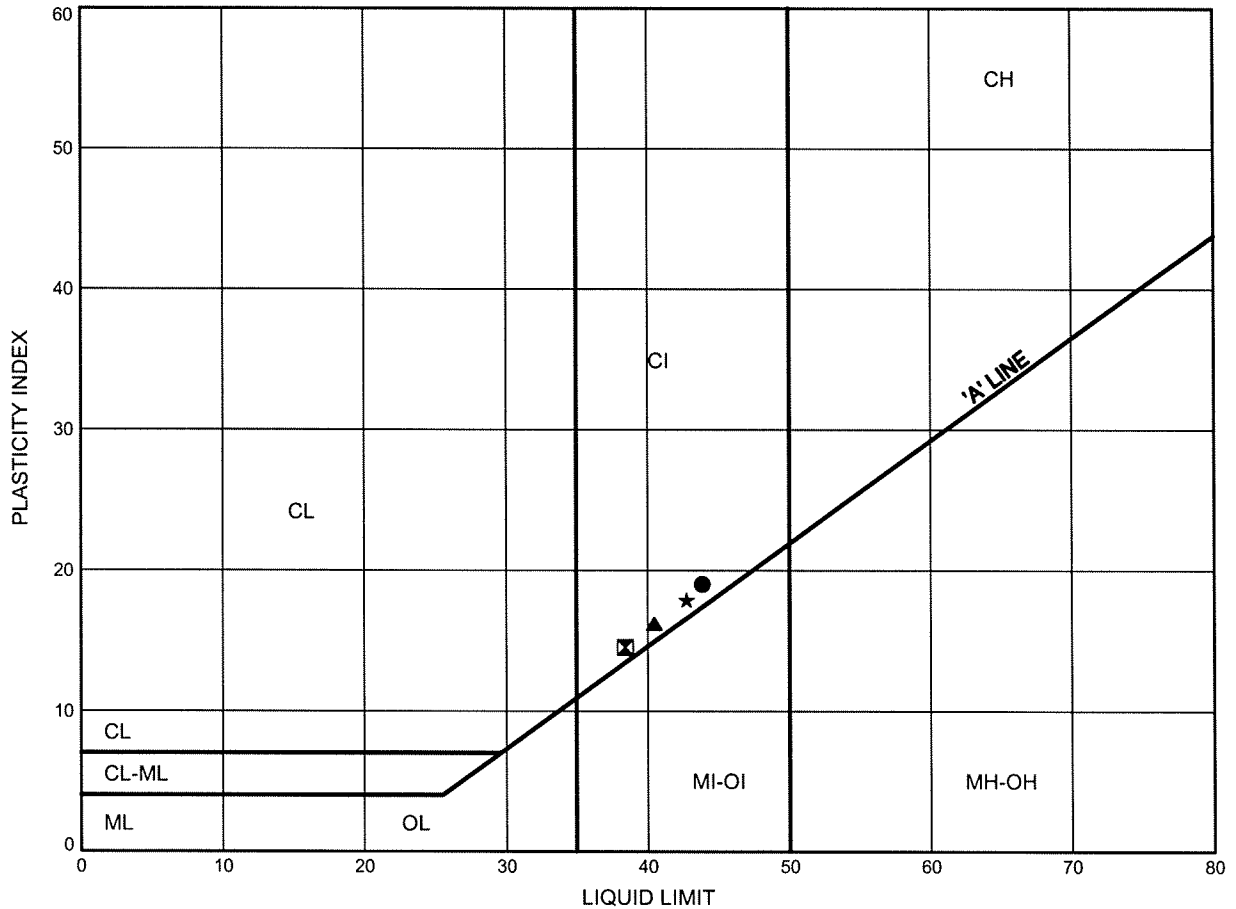


Prep'd AN
 Chkd. JPL

Investigation of Embankment Failure
ATTERBERG LIMITS TEST RESULTS

FIGURE B9

SILTY CLAY (CI)



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	09-02	14.02	300.91
⊠	09-03	6.40	306.21
▲	09-03	7.92	304.69
★	09-03	9.45	303.16

Date February 2010
 Project 5079-06-00



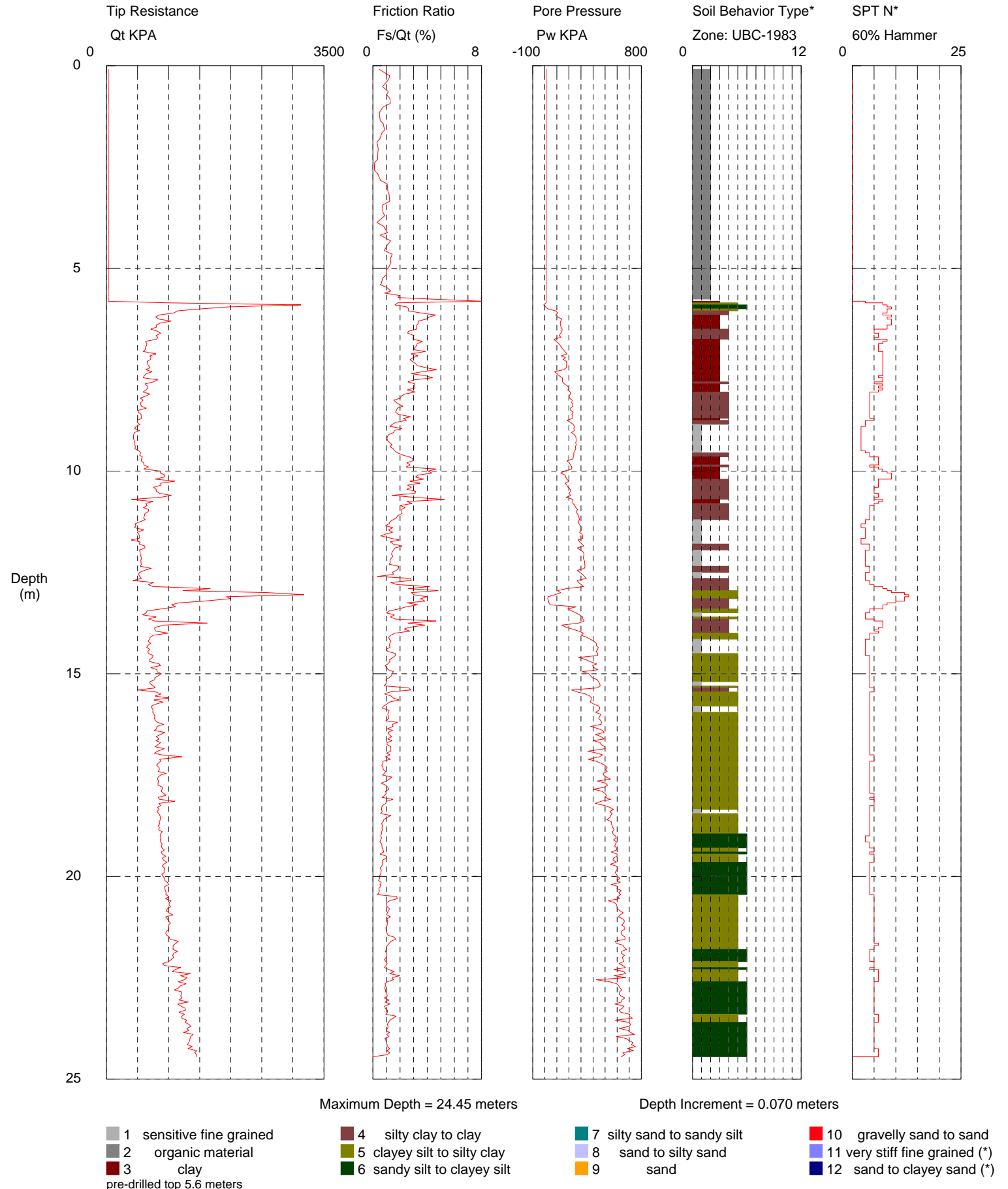
Prep'd AN
 Chkd. JPL

Appendix C
Cone Penetration Test – CPTU Results

Thurber Engineering

Operator: Brown
Sounding: CPT-01
Cone Used: DSG1029

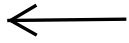
CPT Date/Time: 10/26/2009 1:24:55 PM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



*Soil behavior type and SPT based on data from UBC-1983

In Situ Engineering

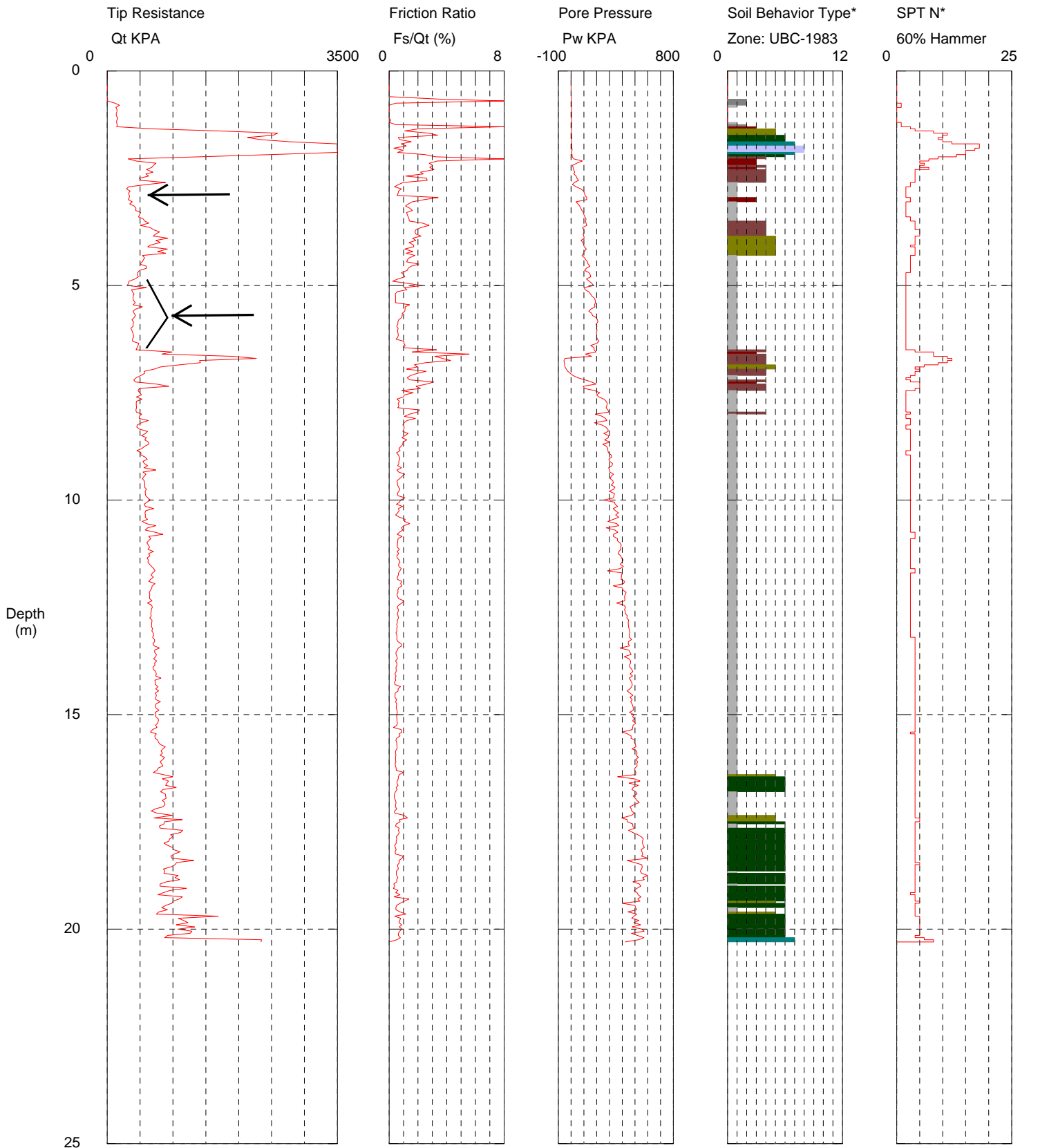
Thurber Engineering



Potential
Shear Zone

Operator: Brown
Sounding: CPT-02
Cone Used: DSG1029

CPT Date/Time: 10/27/2009 11:12:28 AM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Maximum Depth = 20.30 meters

Depth Increment = 0.040 meters

- | | | | |
|--------------------------|-----------------------------|----------------------------|--------------------------------|
| 1 sensitive fine grained | 4 silty clay to clay | 7 silty sand to sandy silt | 10 gravelly sand to sand |
| 2 organic material | 5 clayey silt to silty clay | 8 sand to silty sand | 11 very stiff fine grained (*) |
| 3 clay | 6 sandy silt to clayey silt | 9 sand | 12 sand to clayey sand (*) |

Prepunched top 4 feet with SPT

In Situ Engineering

*Soil behavior type and SPT based on data from UBC-1983

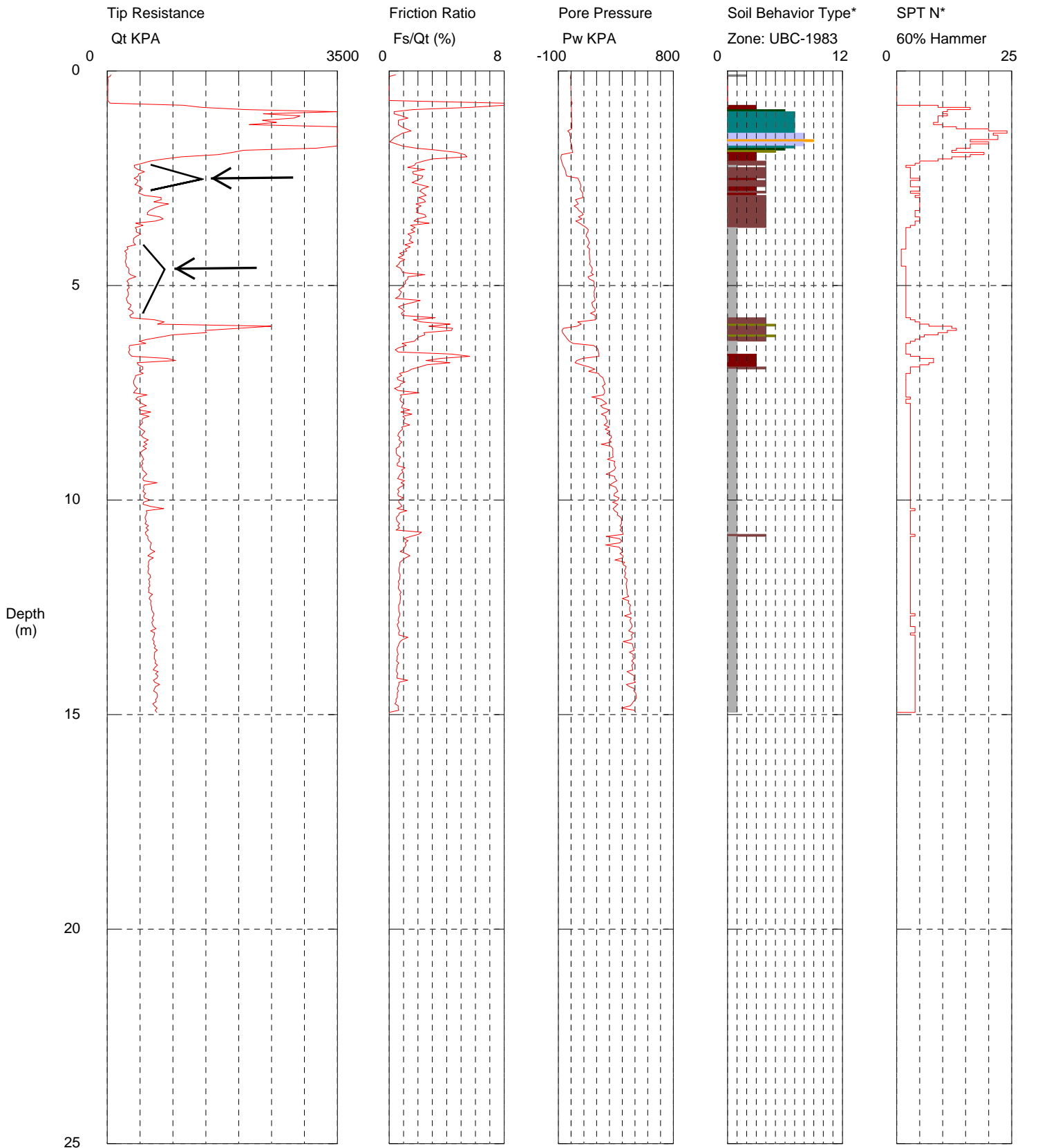
Thurber Engineering



Potential
Shear Zone

Operator: Brown
Sounding: CPT-03
Cone Used: DSG1029

CPT Date/Time: 10/28/2009 5:01:54 AM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Maximum Depth = 14.95 meters

Depth Increment = 0.040 meters

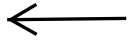
- | | | | |
|--------------------------|-----------------------------|----------------------------|--------------------------------|
| 1 sensitive fine grained | 4 silty clay to clay | 7 silty sand to sandy silt | 10 gravelly sand to sand |
| 2 organic material | 5 clayey silt to silty clay | 8 sand to silty sand | 11 very stiff fine grained (*) |
| 3 clay | 6 sandy silt to clayey silt | 9 sand | 12 sand to clayey sand (*) |

Prepunched top 4 feet with SPT

In Situ Engineering

*Soil behavior type and SPT based on data from UBC-1983

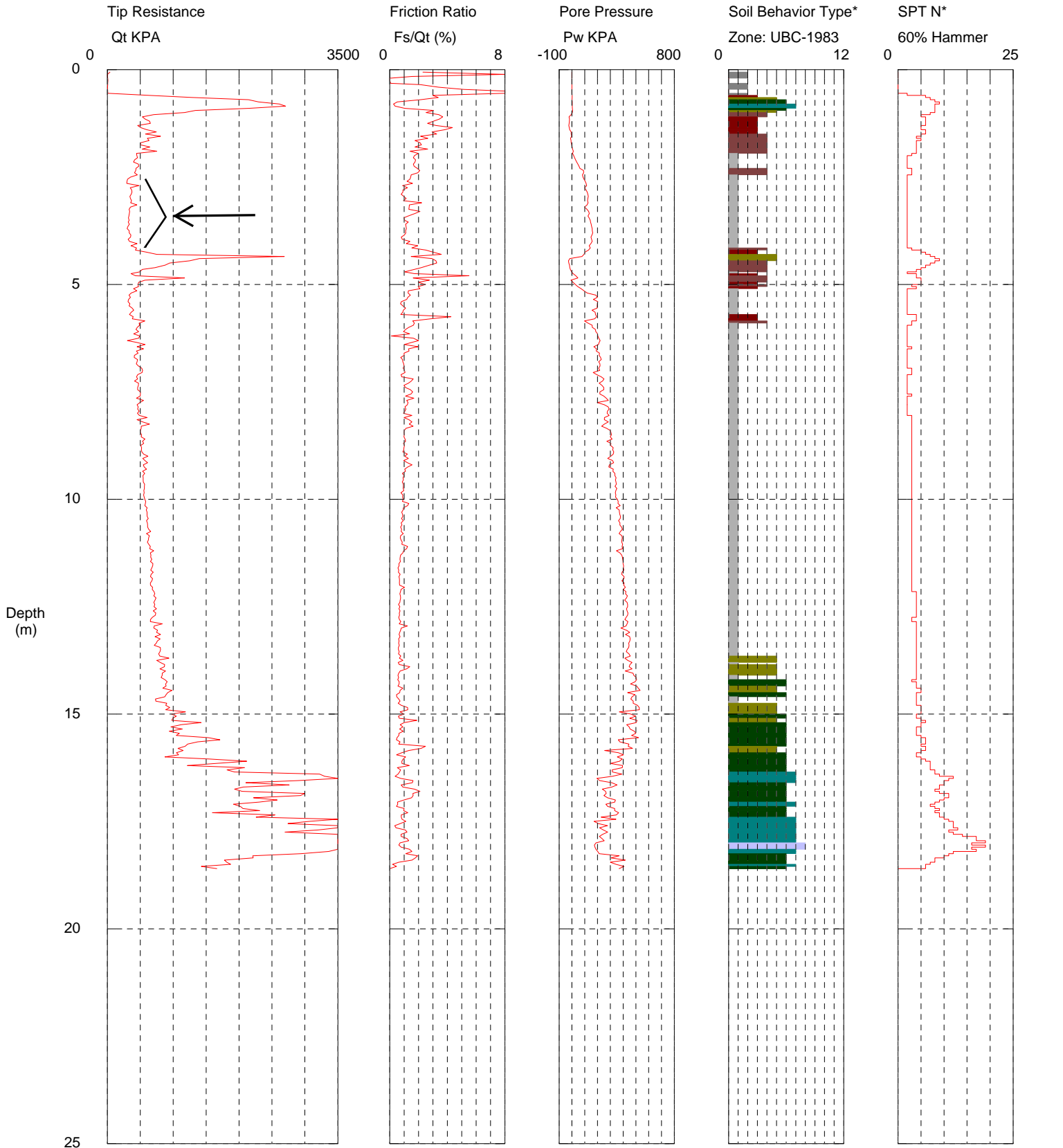
Thurber Engineering



Potential
Shear Zone

Operator: Brown
Sounding: CPT-04
Cone Used: DSG1029

CPT Date/Time: 10/27/2009 8:01:03 AM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Maximum Depth = 18.60 meters

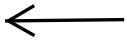
Depth Increment = 0.050 meters

- | | | | |
|--------------------------|-----------------------------|----------------------------|--------------------------------|
| 1 sensitive fine grained | 4 silty clay to clay | 7 silty sand to sandy silt | 10 gravelly sand to sand |
| 2 organic material | 5 clayey silt to silty clay | 8 sand to silty sand | 11 very stiff fine grained (*) |
| 3 clay | 6 sandy silt to clayey silt | 9 sand | 12 sand to clayey sand (*) |
- prepunched top 2 feet with SPT

*Soil behavior type and SPT based on data from UBC-1983

In Situ Engineering

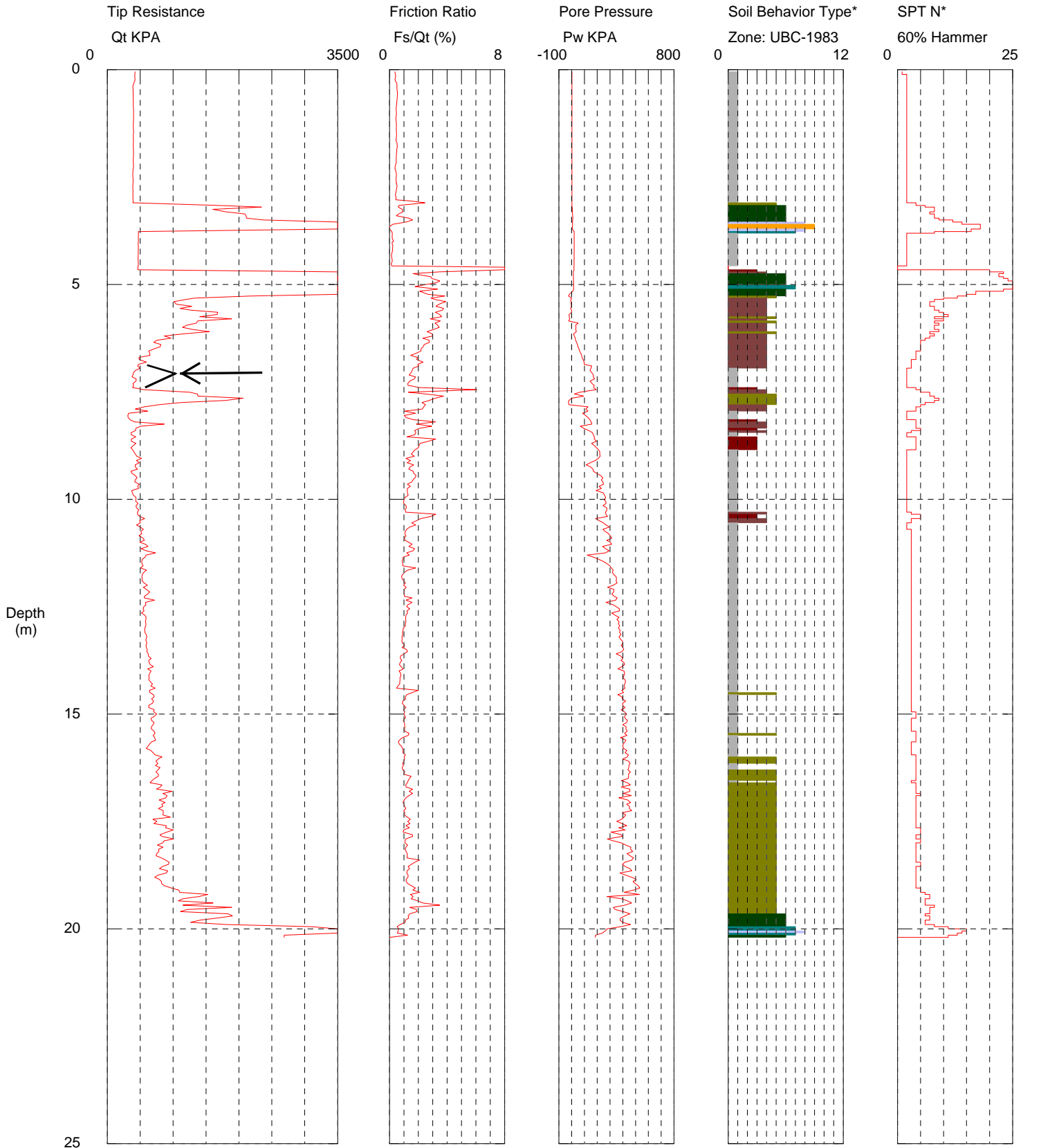
Thurber Engineering



Potential
Shear Zone

Operator: Brown
Sounding: CPT-05
Cone Used: DSG1029

CPT Date/Time: 10/27/2009 5:21:18 AM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Maximum Depth = 20.20 meters

Depth Increment = 0.060 meters

- | | | | |
|--------------------------|-----------------------------|----------------------------|--------------------------------|
| 1 sensitive fine grained | 4 silty clay to clay | 7 silty sand to sandy silt | 10 gravelly sand to sand |
| 2 organic material | 5 clayey silt to silty clay | 8 sand to silty sand | 11 very stiff fine grained (*) |
| 3 clay | 6 sandy silt to clayey silt | 9 sand | 12 sand to clayey sand (*) |
- predrilled top 4.5 meters

In Situ Engineering

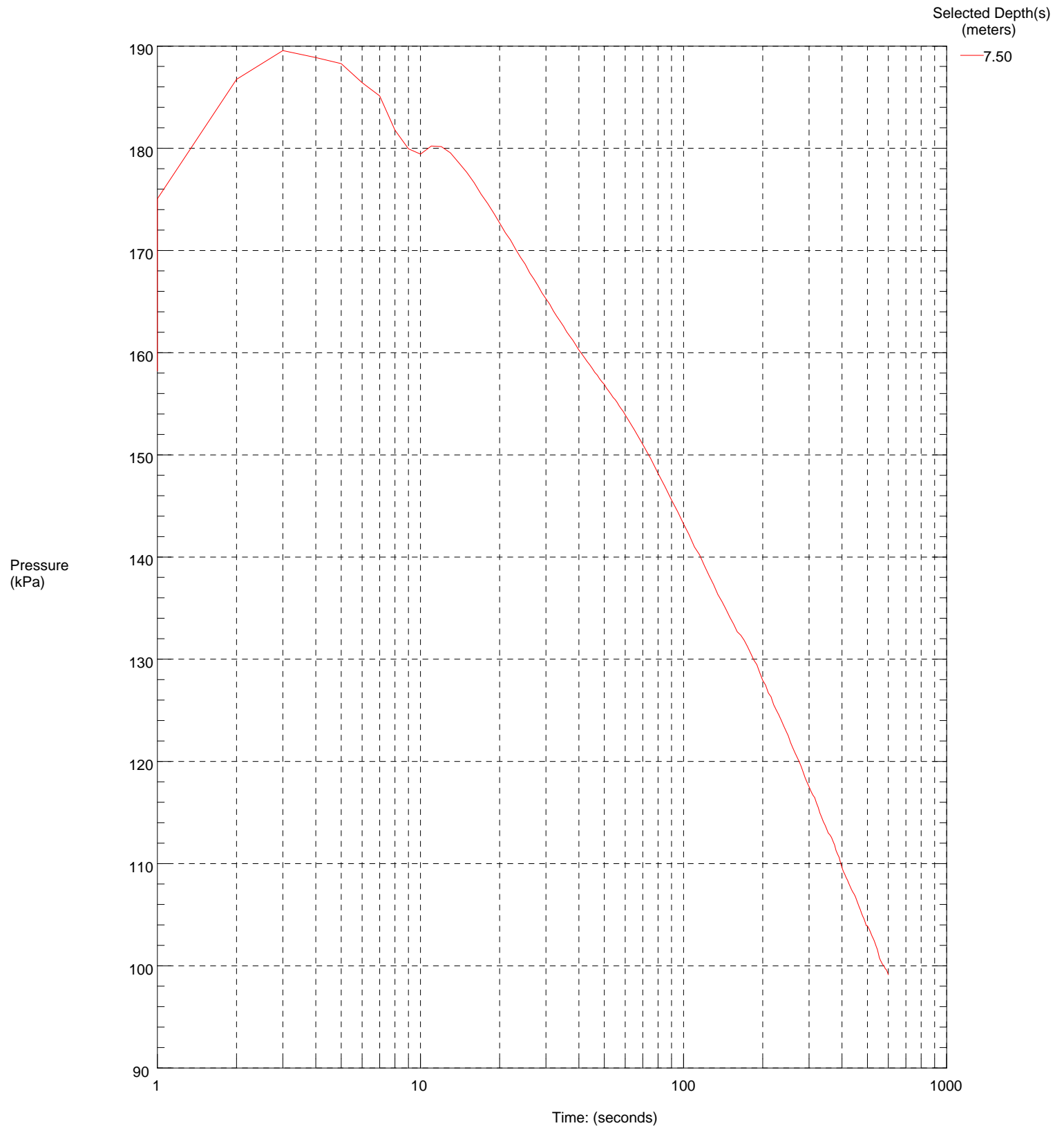
*Soil behavior type and SPT based on data from UBC-1983

Appendix D
Cone Penetration Test – Dissipation Results

Thurber Engineering

Operator Brown
Sounding: CPT-01
Cone Used: DSG1029

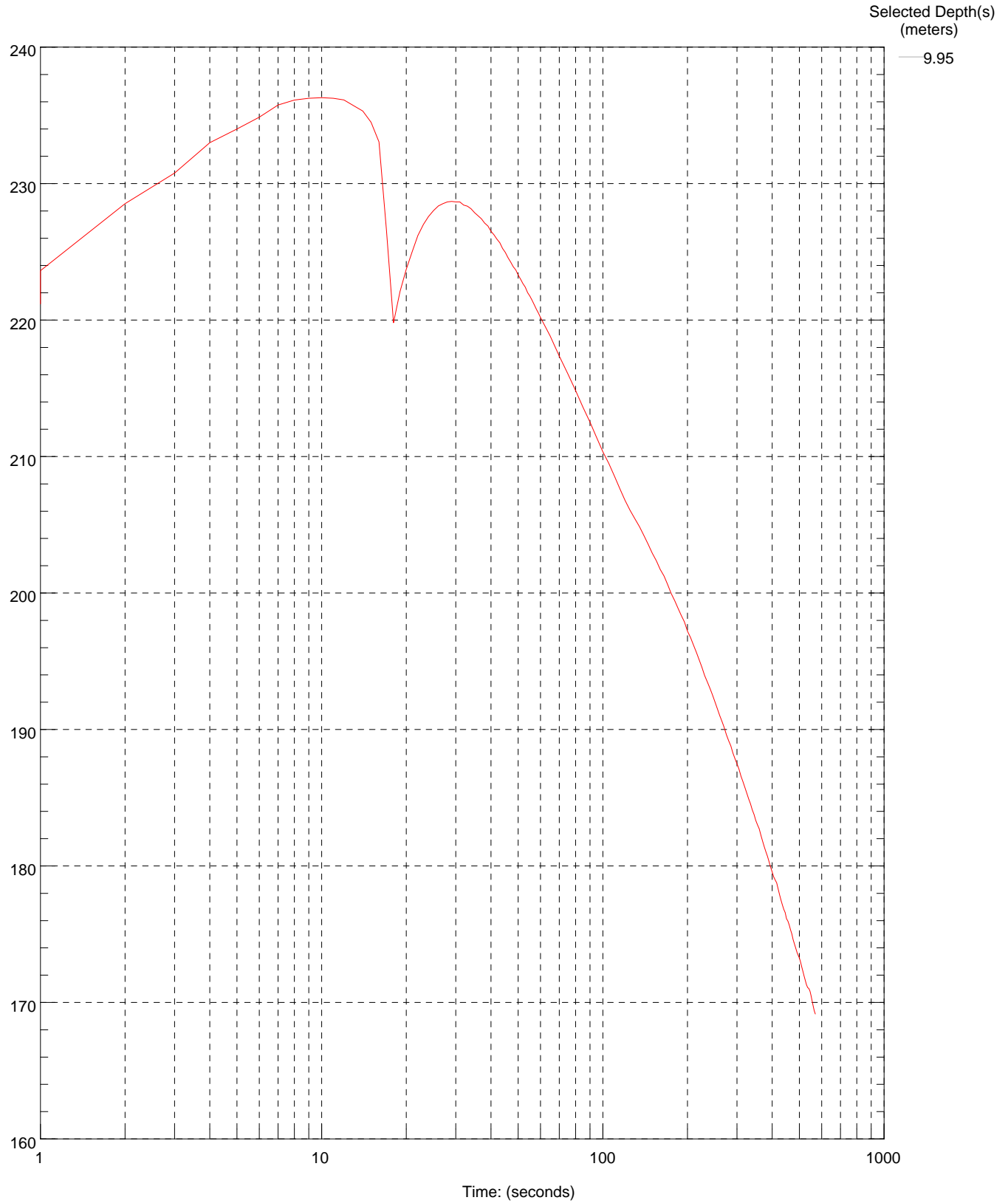
CPT Date/Time: 10/26/2009 1:24:55 PM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Thurber Engineering

Operator Brown
Sounding: CPT-01
Cone Used: DSG1029

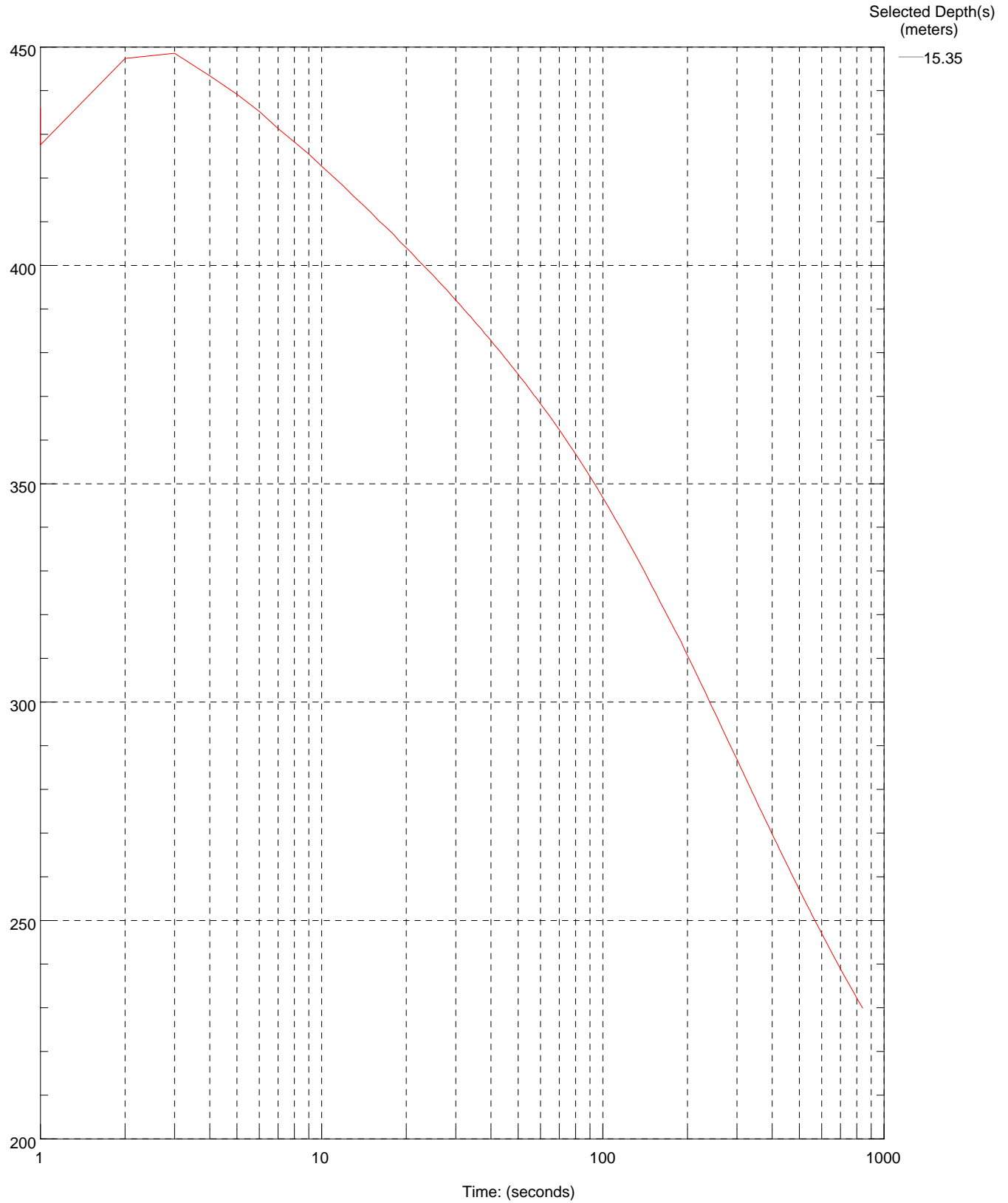
CPT Date/Time: 10/26/2009 1:24:55 PM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Thurber Engineering

Operator Brown
Sounding: CPT-01
Cone Used: DSG1029

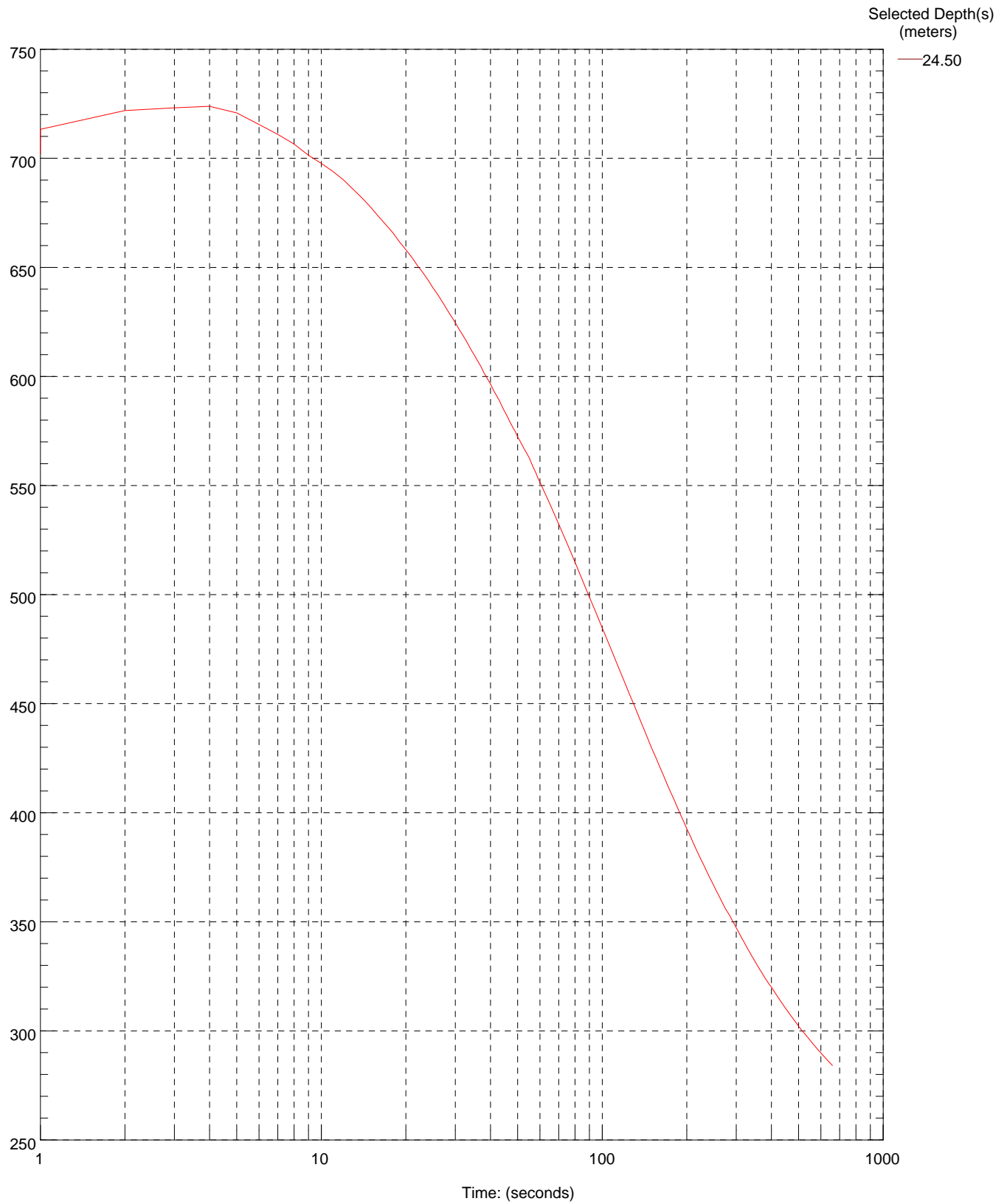
CPT Date/Time: 10/26/2009 1:24:55 PM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Thurber Engineering

Operator Brown
Sounding: CPT-01
Cone Used: DSG1029

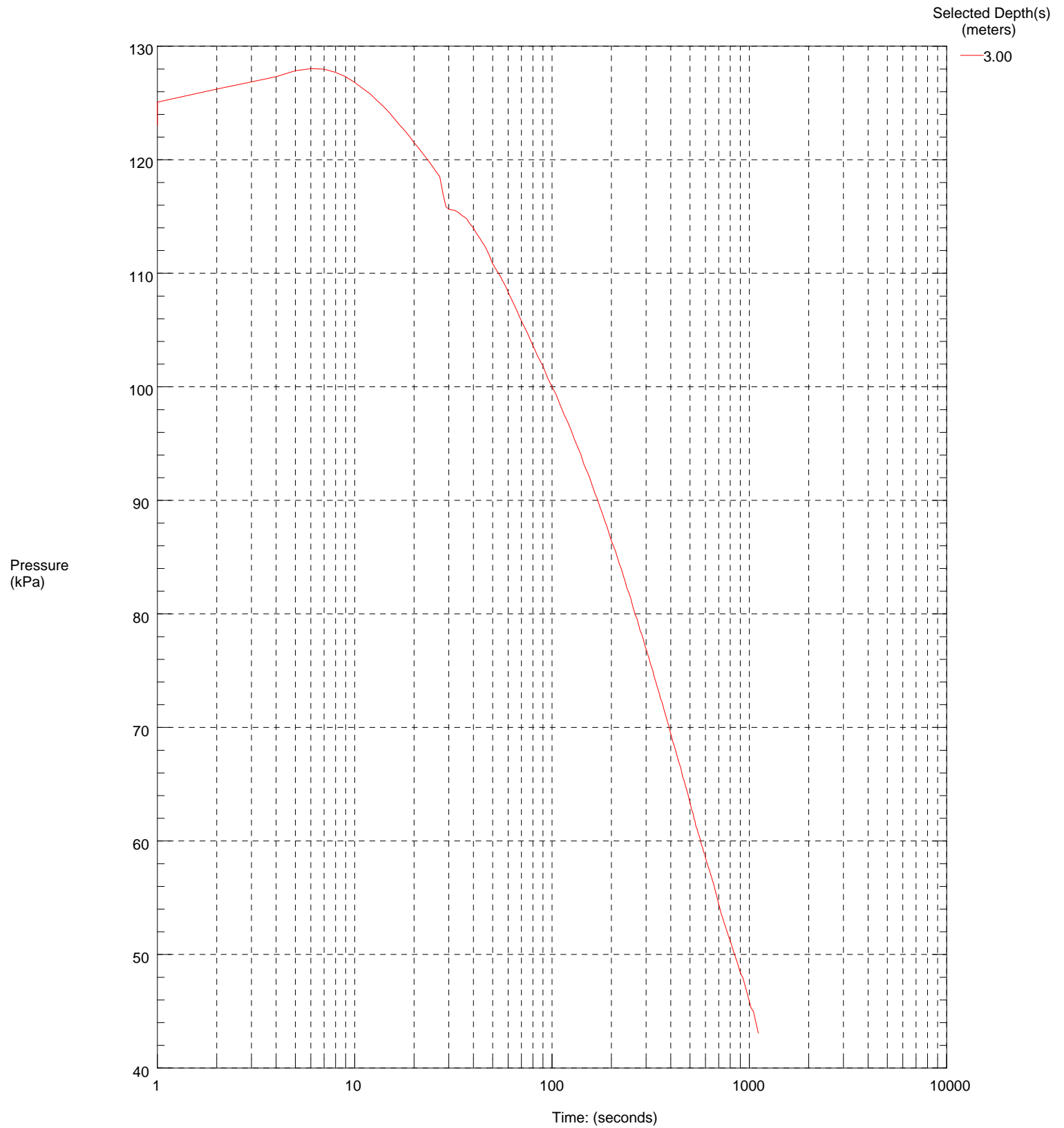
CPT Date/Time: 10/26/2009 1:24:55 PM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Thurber Engineering

Operator Brown
Sounding: CPT-02
Cone Used: DSG1029

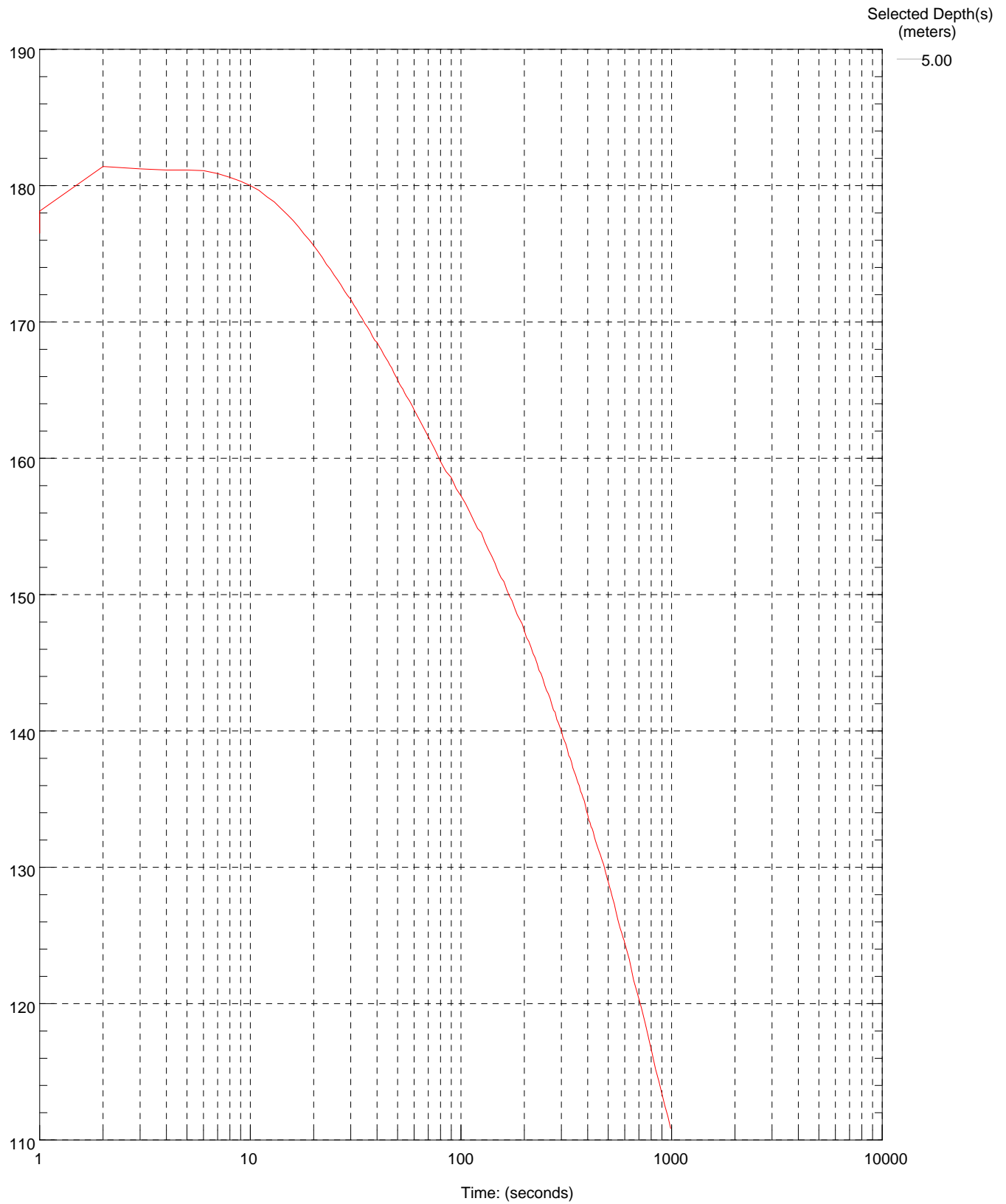
CPT Date/Time: 10/27/2009 11:12:28 AM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Thurber Engineering

Operator Brown
Sounding: CPT-02
Cone Used: DSG1029

CPT Date/Time: 10/27/2009 11:12:28 AM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24

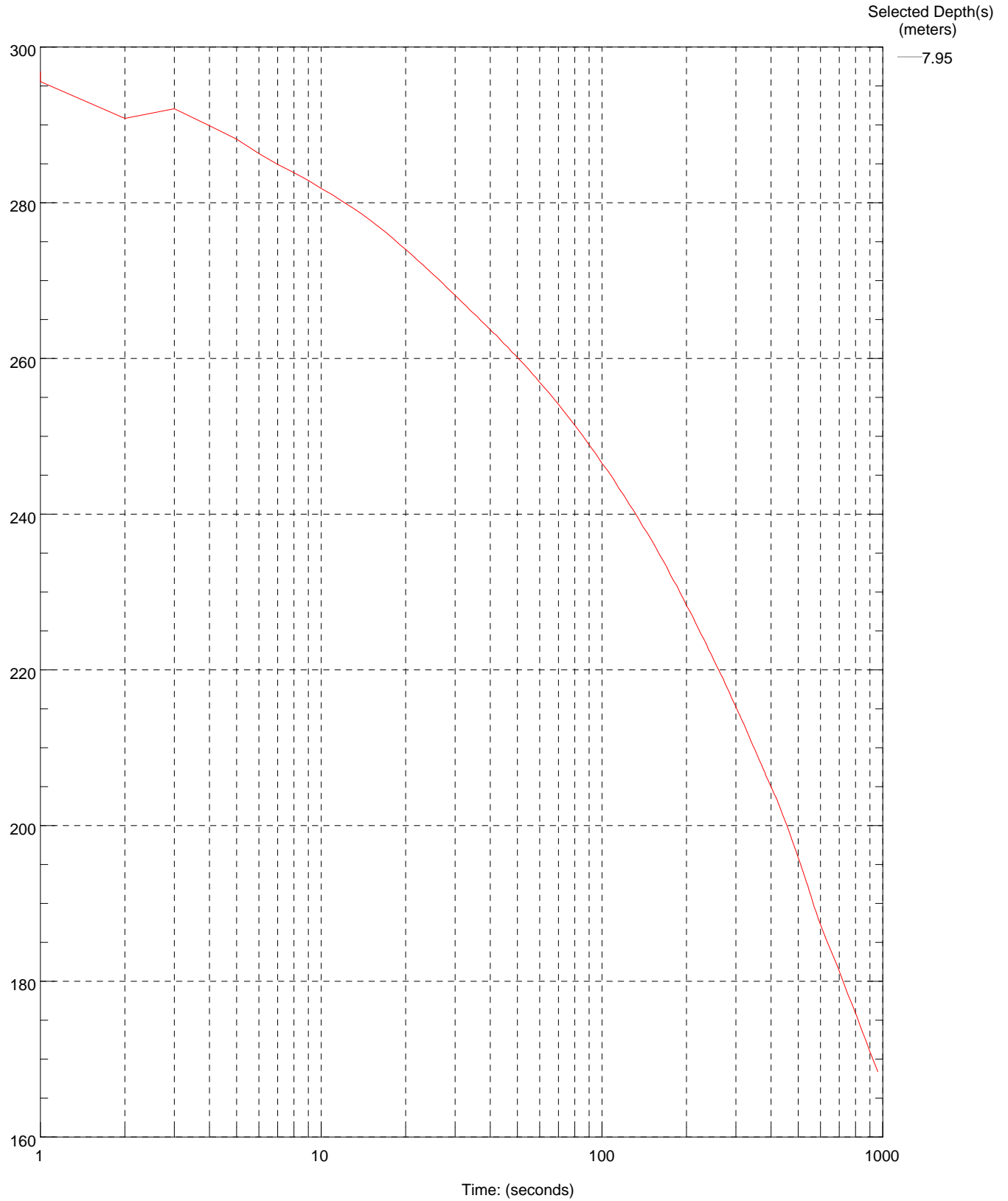


Maximum Pressure = 181.401 kPa
Hydrostatic Pressure = 49.088 kPa

Thurber Engineering

Operator Brown
Sounding: CPT-02
Cone Used: DSG1029

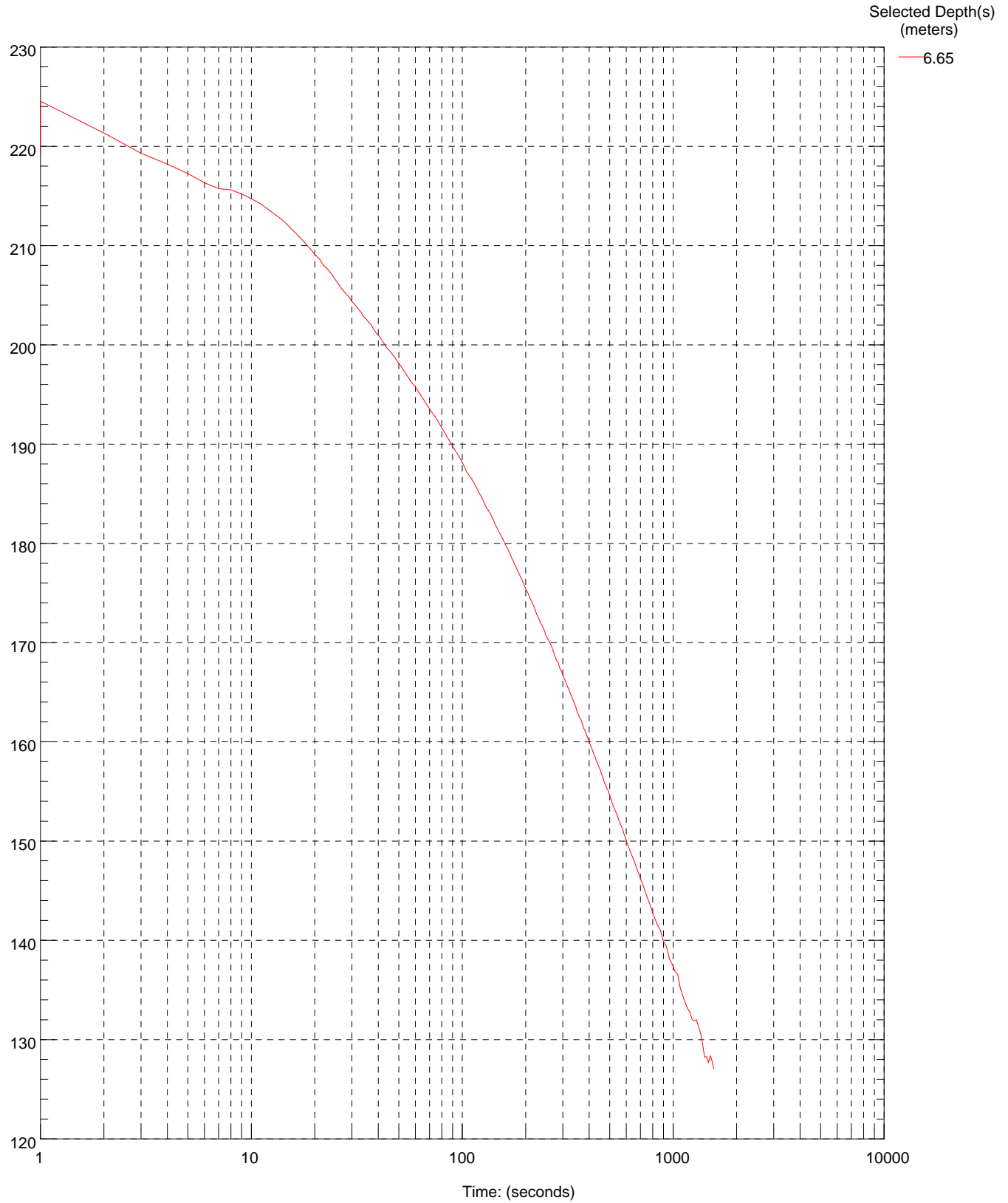
CPT Date/Time: 10/27/2009 11:12:28 AM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Thurber Engineering

Operator Brown
Sounding: CPT-03
Cone Used: DSG1029

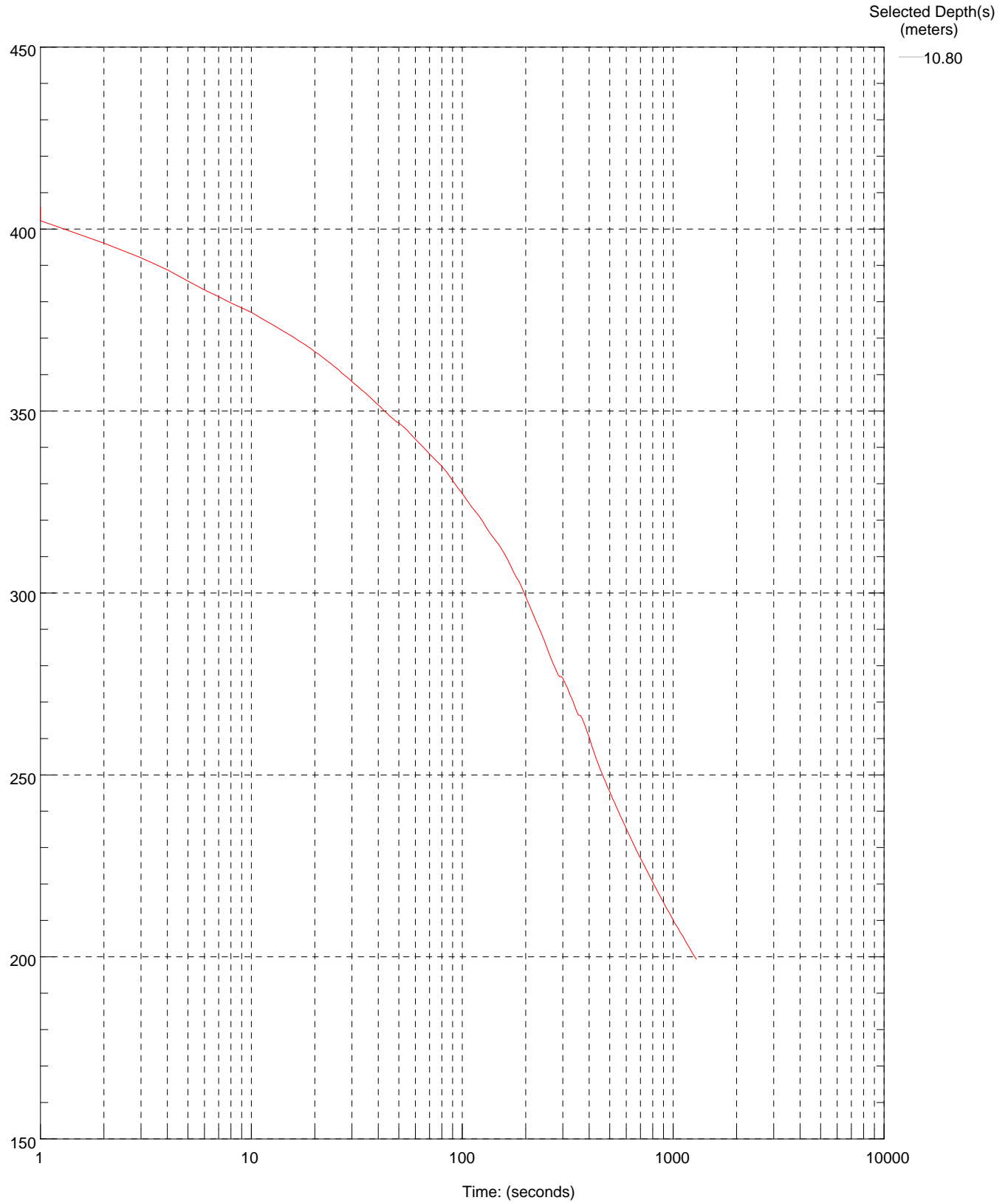
CPT Date/Time: 10/28/2009 5:01:54 AM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Thurber Engineering

Operator Brown
Sounding: CPT-03
Cone Used: DSG1029

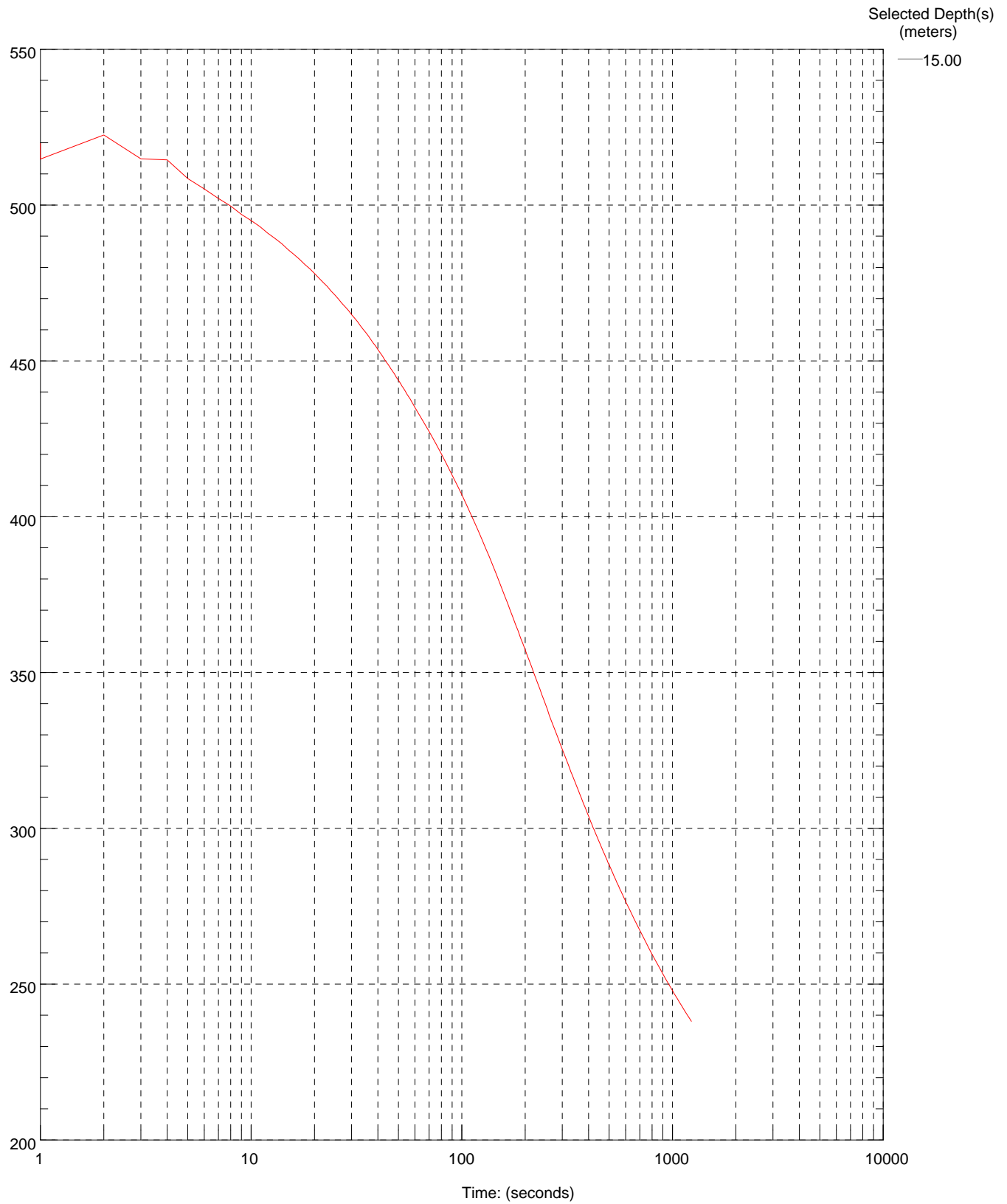
CPT Date/Time: 10/28/2009 5:01:54 AM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Thurber Engineering

Operator Brown
Sounding: CPT-03
Cone Used: DSG1029

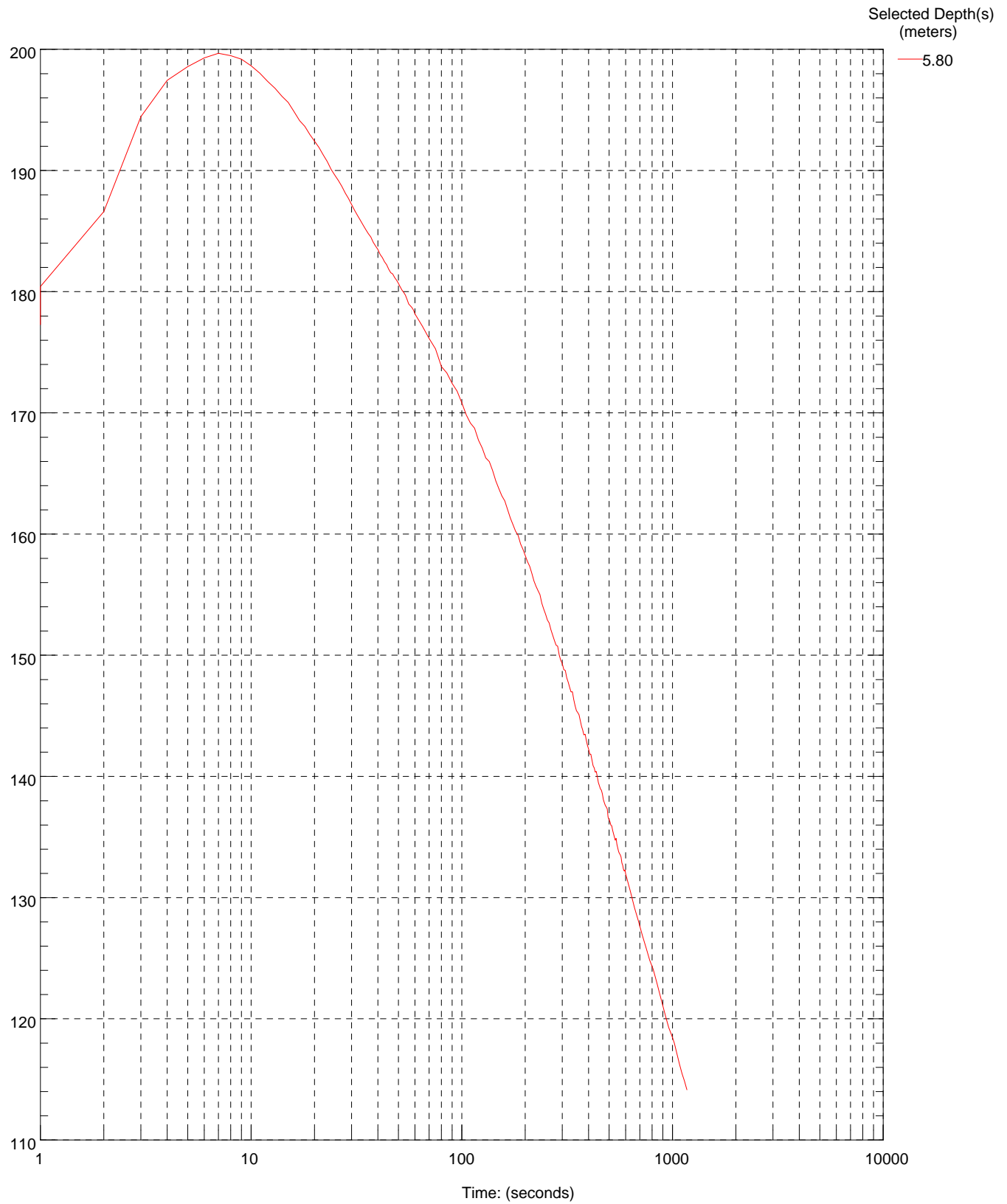
CPT Date/Time: 10/28/2009 5:01:54 AM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Thurber Engineering

Operator Brown
Sounding: CPT-04
Cone Used: DSG1029

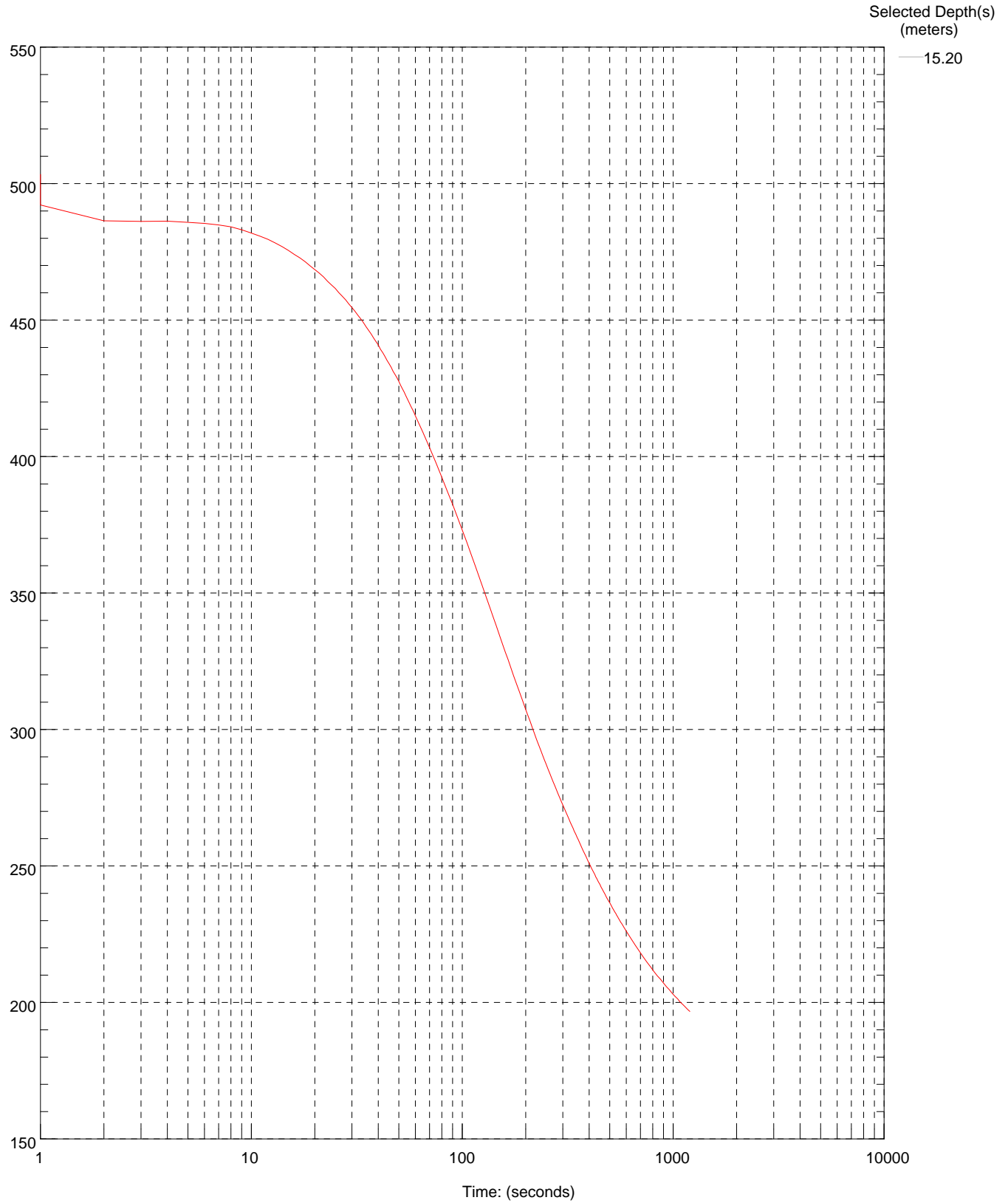
CPT Date/Time: 10/27/2009 8:01:03 AM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Thurber Engineering

Operator Brown
Sounding: CPT-04
Cone Used: DSG1029

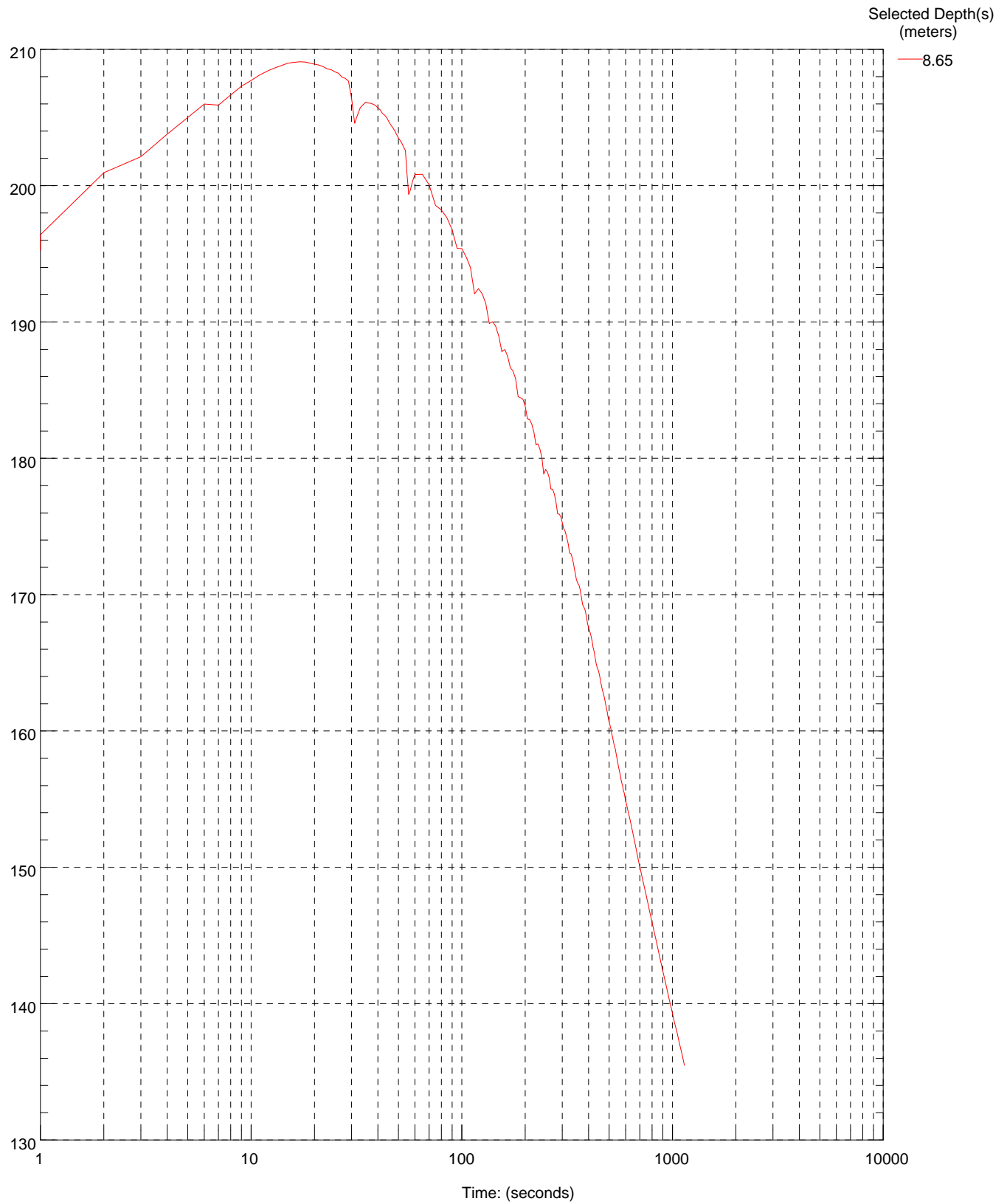
CPT Date/Time: 10/27/2009 8:01:03 AM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Thurber Engineering

Operator Brown
Sounding: CPT-05
Cone Used: DSG1029

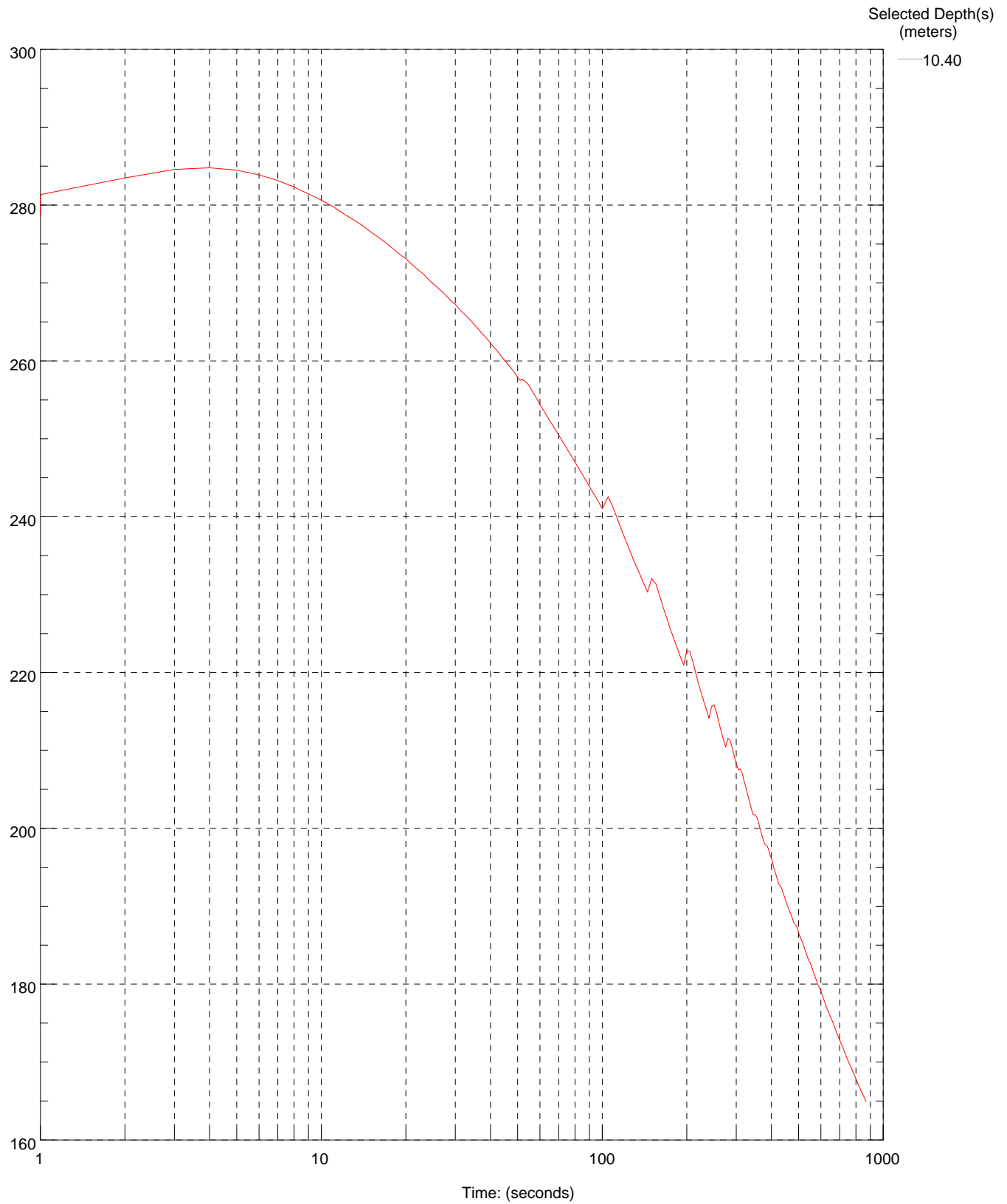
CPT Date/Time: 10/27/2009 5:21:18 AM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Thurber Engineering

Operator Brown
Sounding: CPT-05
Cone Used: DSG1029

CPT Date/Time: 10/27/2009 5:21:18 AM
Location: Hwy 11 Black Creek Road
Job Number: 19-1423-24



Appendix E
Site Photos



Figure E1 – Black Creek Road Embankment Instability on July 3, 2009 (looking west)



Figure E2 – Black Creek Road Embankment Instability (looking east)



Figure E3 – Tension Cracks at the top of embankment (looking west)



Figure E4 – Tension Cracks on the south SSM slope (looking west)



Figure E5 –South toe bulge (looking south west)



Figure E6 – north toe bulge (looking north west)



Figure E7 – Sand boil on south SSM slope (looking west)



Figure E8 – More sand boils on north SSM slope (looking west)



Figure E9 – Sand boils on south Granular B Type 2 Surcharge slope



Figure E10 – More sand boils on south Granular B Type 2 Surcharge slope



Figure E11 – Sand boil at the top of backfill in the north ditch



Figure E12 – More sand boils at the top of backfill in the north ditch



Figure E13 – Drilling of BH09-3 (looking east)

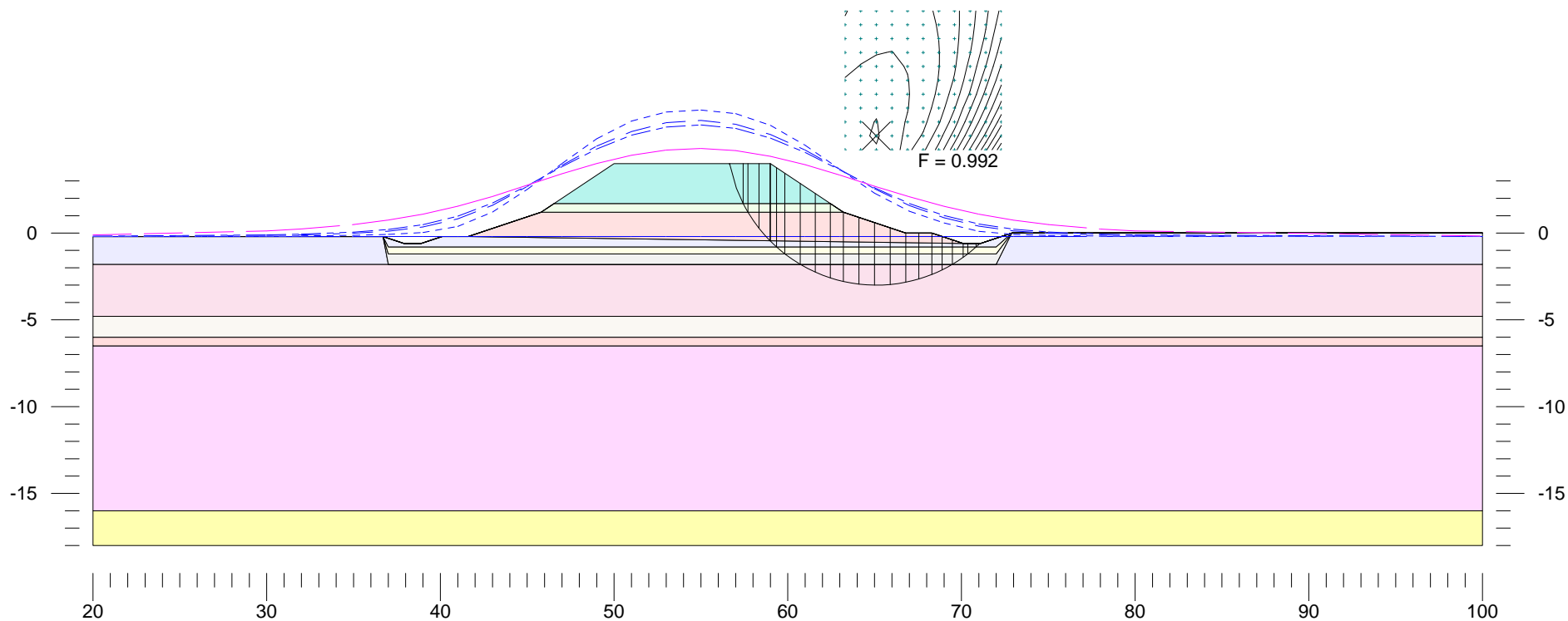


Figure E14 – Drilling of BH09-2 (looking east)

Appendix F
Slope Stability Analysis Results - Selected Runs

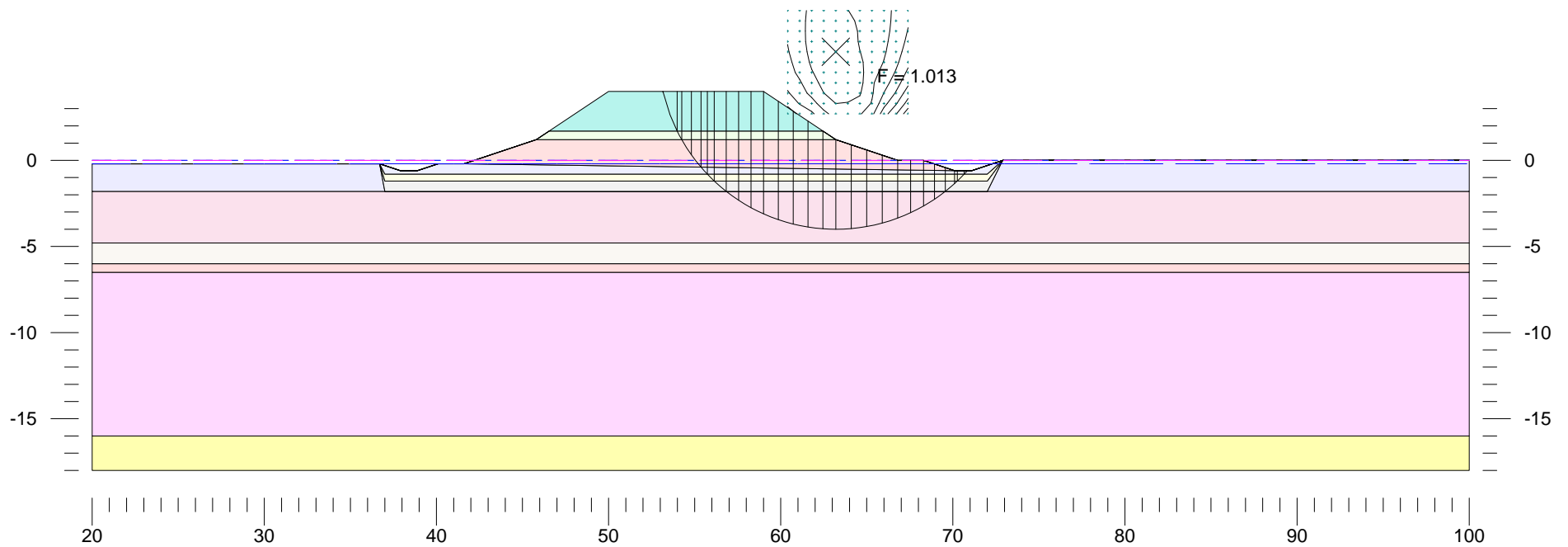
	Gamma	C	Phi	Min	Piezo
	kN/m ³	kPa	deg	c/p	Surf.
Schg-B Type II	21.5	0	30	0	1
Compacted B2	22.8	0	33	0	1
SSM Fill	22	0	32	0	1
Backfill (N=20)	21.5	0	31	0	1
Backfill (N=10)	21	0	29.5	0	1
Backfill (N=3)	21	0	28	0	1
Peat	12.5	0	25	0	1
Silty Clay-Soft	18.5	0	25	0	2
Silty Clay-Firm	18	0	26	0	3
Silty Clay-Stiff	18.5	0	29	0	4
Silty Clay-Firm	17	0	27	0	5
Hard Bottom	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
 19-5161-73
 Black Creek Road Embankment Failure
 January 25, 2010
 Black Creek Road Sta.9+000 - 4m high Embankment (Surcharge at 1.5H:1V, SSM Fill at 3H:1V)
 Effective Stress Analysis



	Gamma	C	Phi	Min	Piezo
	kN/m ³	kPa	deg	c/p	Surf.
Schg-B Type II	21.5	0	30	0	1
Compacted B2	22.8	0	33	0	1
SSM Fill	22	0	32	0	1
Backfill (N=20)	21.5	0	31	0	1
Backfill (N=10)	21	0	29.5	0	1
Backfill (N=3)	21	0	28	0	1
Peat	12.5	0	25	0	1
Silty Clay-Soft	18.5	15	0	0	1
Silty Clay-Firm	18	25	0	0	1
Silty Clay-Stiff	18.5	80	0	0	1
Silty Clay-Firm	17	35	0	0	1
Hard Bottom	(Infinitely Strong)				

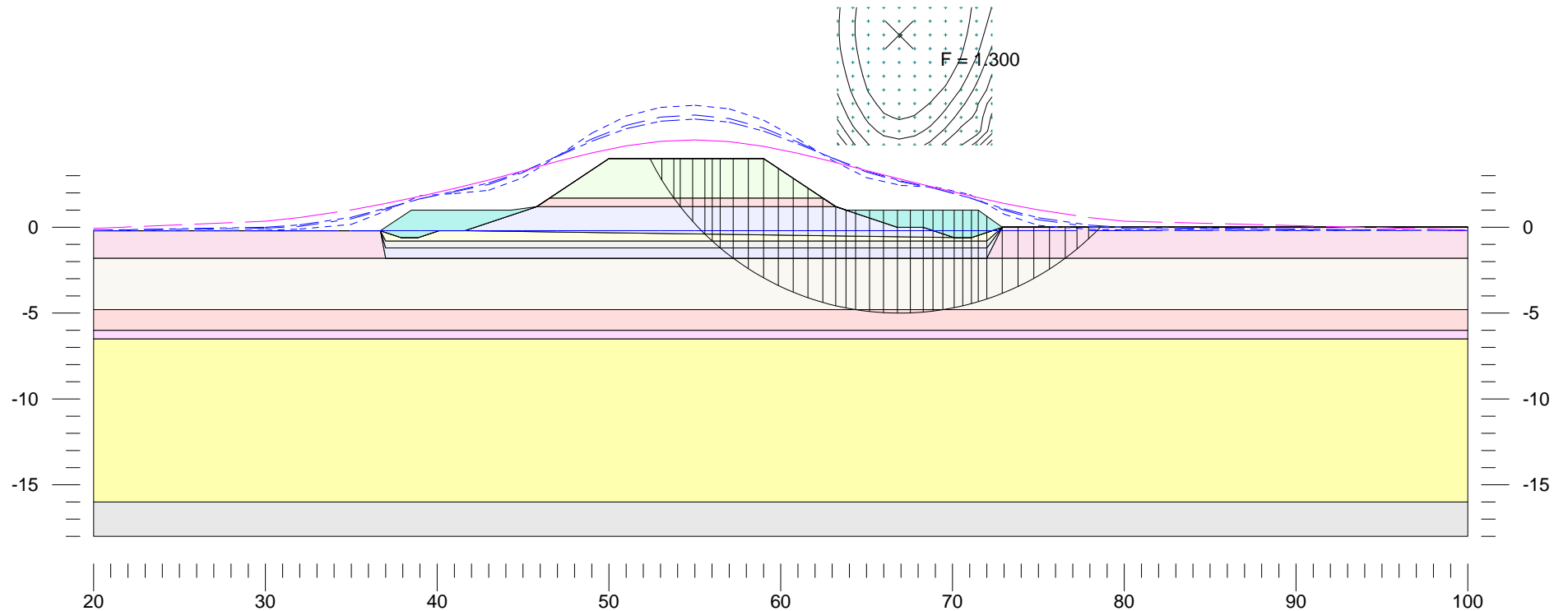
Thurber Engineering Ltd. - Toronto
19-5161-73
Black Creek Road Embankment Failure
January 25, 2010
Black Creek Road Sta.9+000 - 4m high Embankment (Surcharge at 1.5H:1V, SSM Fill at 3H:1V)
Total Stress Analysis



Thurber Engineering Ltd. - Toronto
 19-5161-73
 Black Creek Road Embankment Failure
 January 25, 2010

Black Creek Road Sta.9+000 - 4m high Embankment (Surcharge at 1.5H:1V, SSM Fill at 3H:1V)
 Effective Stress Analysis - with Stabilizing Berm (7.5 m wide and 1 m tall)

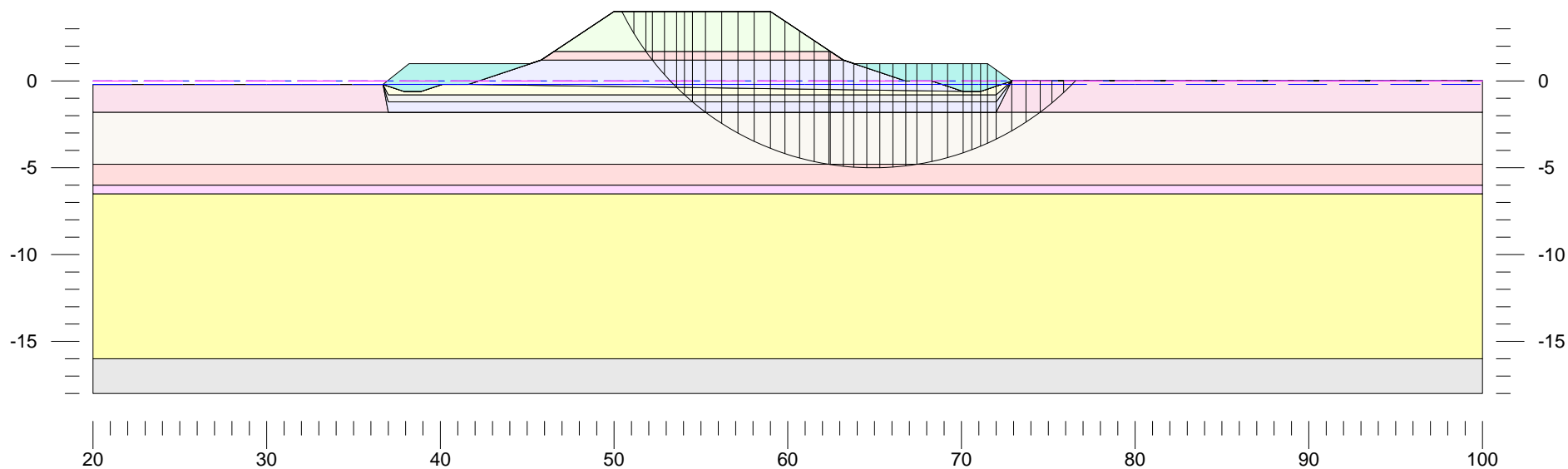
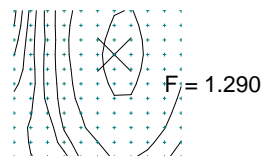
	Gamma	C	Phi	Min	Piezo
	kN/m ³	kPa	deg	c/p	Surf.
Stabilizing Berm	21.5	0	30	0	1
Schg-B Type II	21.5	0	30	0	1
Compacted B2	22.8	0	33	0	1
SSM Fill	22	0	32	0	1
Backfill (N=20)	21.5	0	31	0	1
Backfill (N=10)	21	0	29.5	0	1
Backfill (N=3)	21	0	28	0	1
Peat	12.5	0	25	0	1
Silty Clay-Soft	18.5	0	25	0	2
Silty Clay-Firm	18	0	26	0	3
Silty Clay-Stiff	18.5	0	29	0	4
Silty Clay-Firm	17	0	27	0	5
Hard Bottom	(Infinitely Strong)				



	Gamma	C	Phi	Min	Piezo
	kN/m ³	kPa	deg	c/p	Surf.
Stabilizing Berm	21.5	0	30	0	1
Schg-B Type II	21.5	0	30	0	1
Compacted B2	22.8	0	33	0	1
SSM Fill	22	0	32	0	1
Backfill (N=20)	21.5	0	31	0	1
Backfill (N=10)	21	0	29.5	0	1
Backfill (N=3)	21	0	28	0	1
Peat	12.5	0	25	0	1
Silty Clay-Soft	18.5	15	0	0	1
Silty Clay-Firm	18	25	0	0	1
Silty Clay-Stiff	18.5	80	0	0	1
Silty Clay-Firm	17	35	0	0	1
Hard Bottom	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-73
Black Creek Road Embankment Failure
January 25, 2010

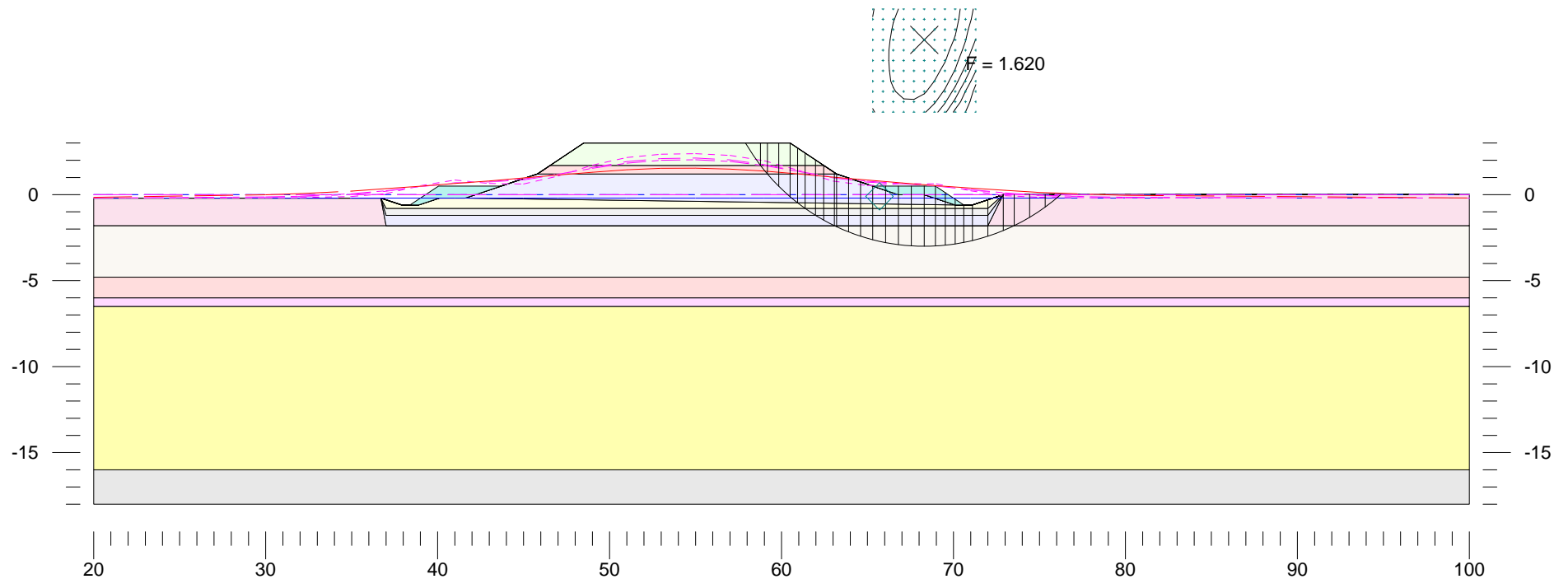
Black Creek Road Sta.9+000 - 4m high Embankment (Surcharge at 1.5H:1V, SSM Fill at 3H:1V)
Total Stress Analysis - with Stabilizing Berm (7.5 m wide and 1 m tall)



	Gamma	C	Phi	Min	Piezo
	kN/m3	kPa	deg	c/p	Surf.
Stabilizing Berm	21.5	0	30	0	1
Schg-B Type II	21.5	0	30	0	1
Compacted B2	22.8	0	33	0	1
SSM Fill	22	0	32	0	1
Backfill (N=20)	21.5	0	31	0	1
Backfill (N=10)	21	0	29.5	0	1
Backfill (N=3)	21	0	28	0	1
Peat	12.5	0	25	0	1
Silty Clay-Soft	18.5	0	25	0	6
Silty Clay-Firm	18	0	26	0	7
Silty Clay-Stiff	18.5	0	29	0	8
Silty Clay-Firm	17	0	27	0	9
Hard Bottom	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-73
Black Creek Road Embankment Failure
January 25, 2010

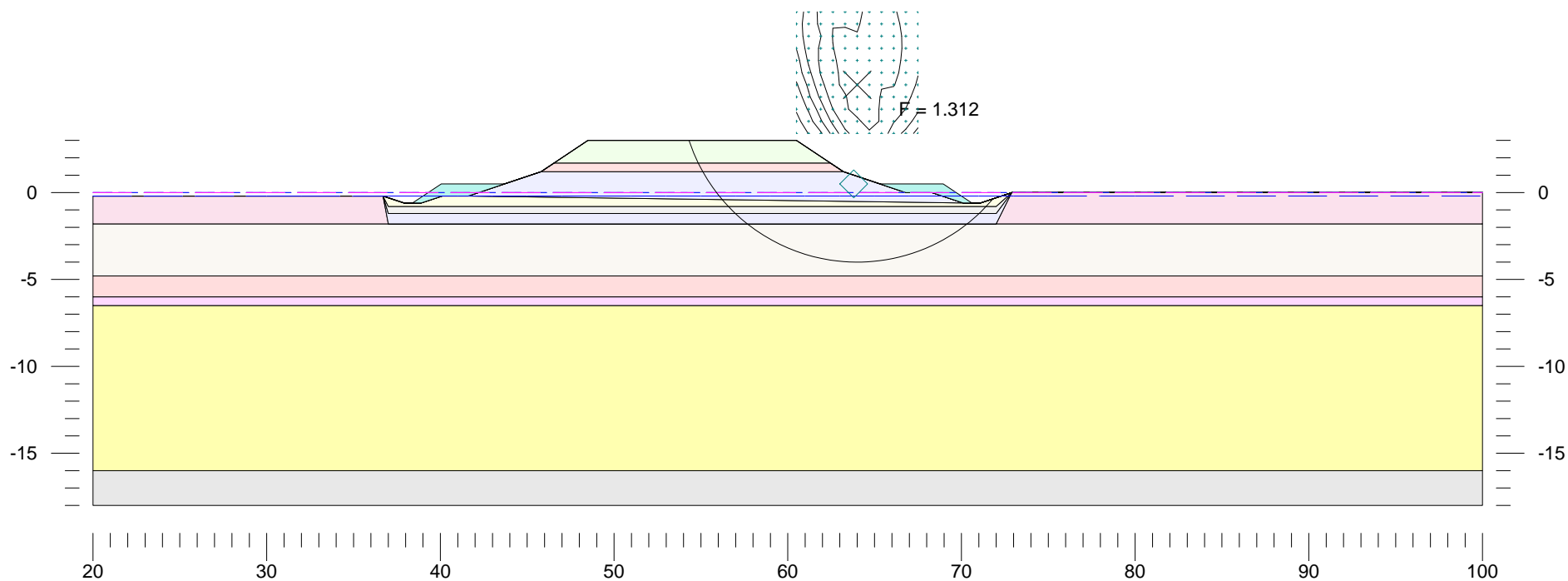
Black Creek Road Sta.9+000 - 3m high Embankment (1 m Surcharge at 1.5H:1V, SSM Fill at 3H:1V)
Effective Stress Analysis - with Stabilizing Berm (3.7 m wide and 0.5 m tall)



	Gamma	C	Phi	Min	Piezo
	kN/m ³	kPa	deg	c/p	Surf.
Stabilizing Berm	21.5	0	30	0	1
Schg-B Type II	21.5	0	30	0	1
Compacted B2	22.8	0	33	0	1
SSM Fill	22	0	32	0	1
Backfill (N=20)	21.5	0	31	0	1
Backfill (N=10)	21	0	29.5	0	1
Backfill (N=3)	21	0	28	0	1
Peat	12.5	0	25	0	1
Silty Clay-Soft	18.5	15	0	0	1
Silty Clay-Firm	18	25	0	0	1
Silty Clay-Stiff	18.5	80	0	0	1
Silty Clay-Firm	17	35	0	0	1
Hard Bottom	(Infinitely Strong)				

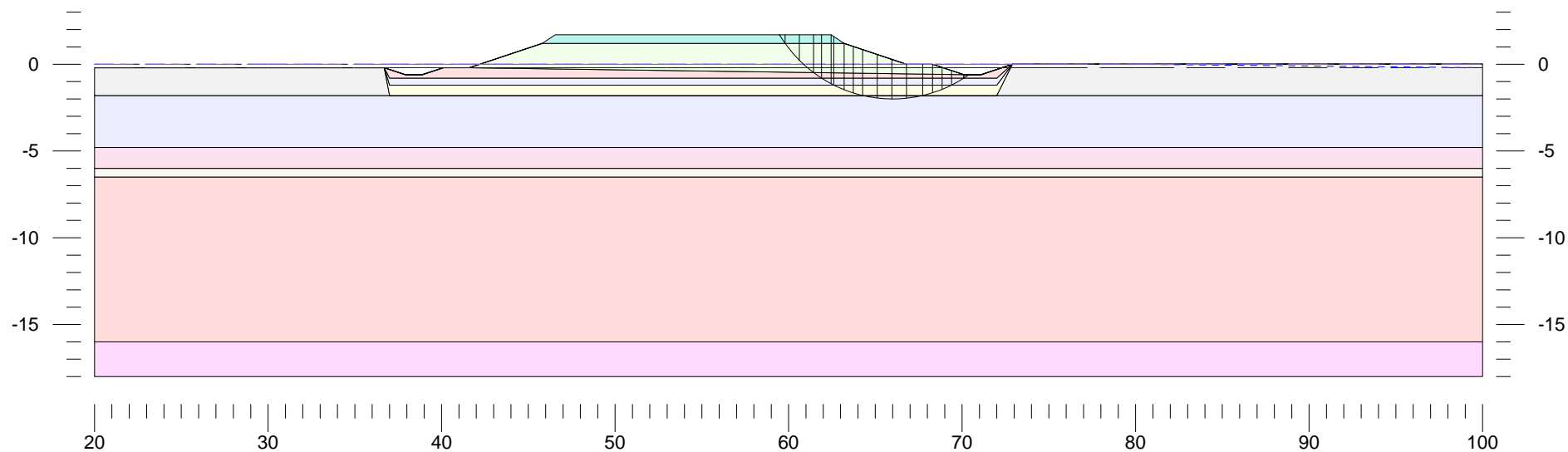
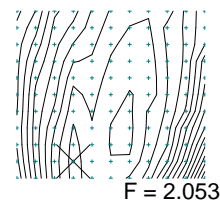
Thurber Engineering Ltd. - Toronto
19-5161-73
Black Creek Road Embankment Failure
January 25, 2010

Black Creek Road Sta.9+000 - 3m high Embankment (1 m Surcharge at 1.5H:1V, SSM Fill at 3H:1V)
Total Stress Analysis - with Stabilizing Berm (3.7 m wide and 0.5 m tall)



	Gamma	C	Phi	Min	Piezo
	kN/m ³	kPa	deg	c/p	Surf.
Compacted B2	22.8	0	33	0	1
SSM Fill	22	0	32	0	1
Backfill (N=20)	21.5	0	31	0	1
Backfill (N=10)	21	0	29.5	0	1
Backfill (N=3)	21	0	28	0	1
Peat	12.5	0	25	0	1
Silty Clay-Soft	18.5	0	25	0	1
Silty Clay-Firm	18	0	26	0	1
Silty Clay-Stiff	18.5	0	29	0	1
Silty Clay-Firm	17	0	27	0	1
Hard Bottom	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-73
Black Creek Road Embankment Failure
January 25, 2010
Black Creek Road Sta.9+000 - 2m high Embankment (SSM Fill at 3H:1V)
Effective Stress Analysis - Long Term Condition



	Gamma	C	Phi	Min	Piezo
	kN/m3	kPa	deg	c/p	Surf.
Schg-B Type II	21.5	0	30	0	1
Compacted B2	22.8	0	33	0	1
SSM Fill	22	0	32	0	1
Backfill (N=20)	21.5	0	31	0	1
Backfill (N=10)	21	0	29.5	0	1
Backfill (N=3)	21	0	28	0	1
Peat	12.5	0	25	0	1
Silty Clay-Soft	18.5	0	25	0	2
Silty Clay-Firm	18	0	26	0	3
Silty Clay-Stiff	18.5	0	29	0	4
Silty Clay-Firm	17	0	27	0	5
Hard Bottom	(Infinitely Strong)				

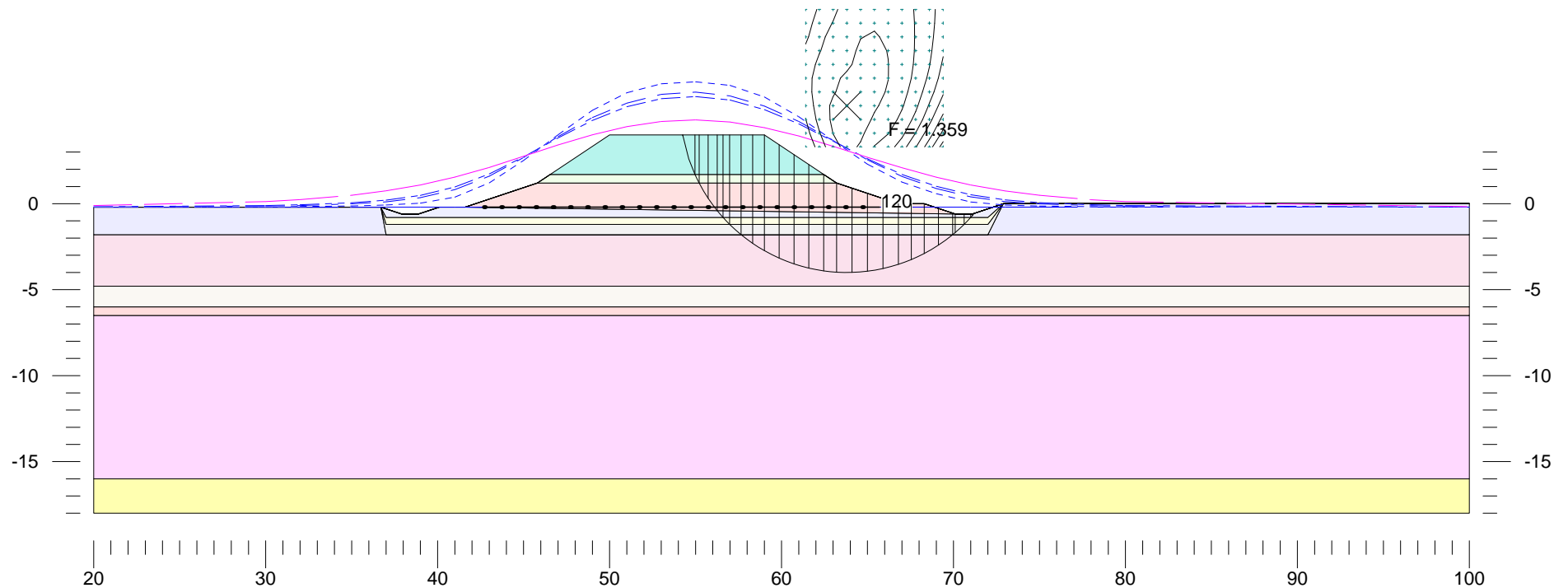
Thurber Engineering Ltd. - Toronto
19-5161-73

Black Creek Road Embankment Failure

January 25, 2010

Black Creek Road Sta.9+000 - 4m high Embankment (Surcharge at 1.5H:1V, SSM Fill at 3H:1V)

Effective Stress Analysis - Geogrid Reinforcement



	Gamma	C	Phi	Min	Piezo
	kN/m ³	kPa	deg	c/p	Surf.
Schg-B Type II	21.5	0	30	0	1
Compacted B2	22.8	0	33	0	1
SSM Fill	22	0	32	0	1
Backfill (N=20)	21.5	0	31	0	1
Backfill (N=10)	21	0	29.5	0	1
Backfill (N=3)	21	0	28	0	1
Peat	12.5	0	25	0	1
Silty Clay-Soft	18.5	15	0	0	1
Silty Clay-Firm	18	25	0	0	1
Silty Clay-Stiff	18.5	80	0	0	1
Silty Clay-Firm	17	35	0	0	1
Hard Bottom	(Infinitely Strong)				

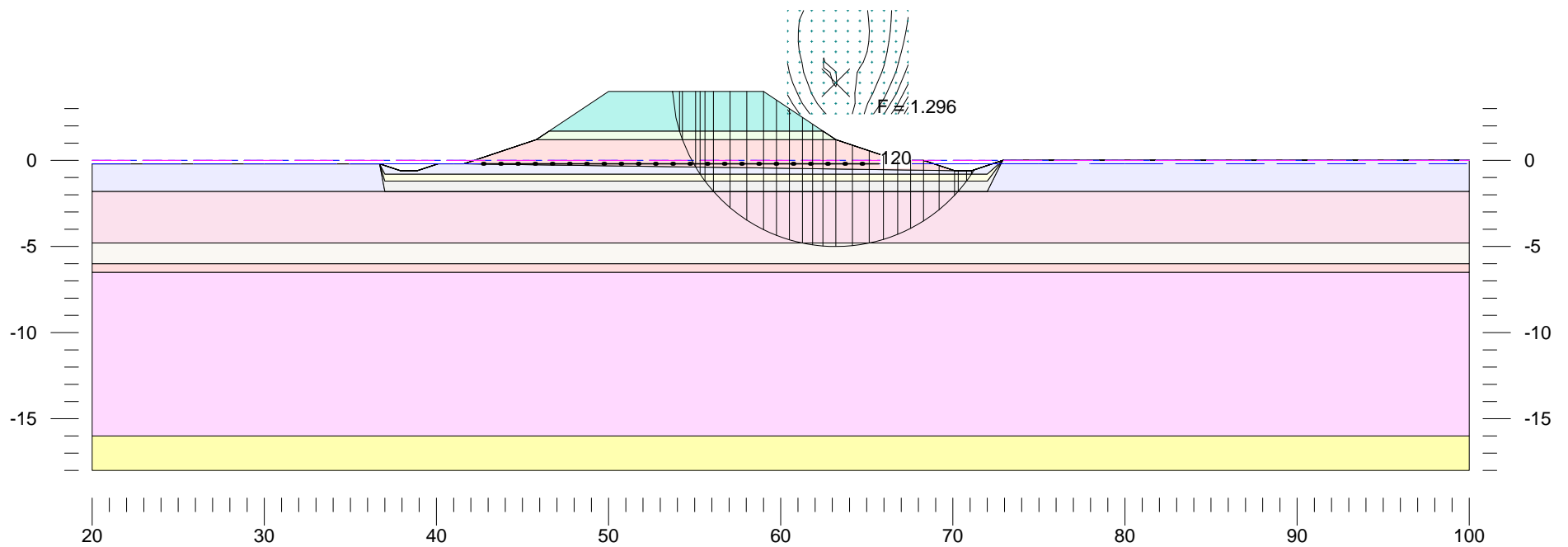
Thurber Engineering Ltd. - Toronto
19-5161-73

Black Creek Road Embankment Failure

January 25, 2010

Black Creek Road Sta.9+000 - 4m high Embankment (Surcharge at 1.5H:1V, SSM Fill at 3H:1V)

Total Stress Analysis - Geogrid Reinforcement



	Gamma	C	Phi	Min	Piezo
	kN/m3	kPa	deg	c/p	Surf.
Schg-B Type II	21.5	0	30	0	1
Compacted B2	22.8	0	33	0	1
SSM Fill	22	0	32	0	1
Backfill (N=20)	21.5	0	31	0	1
Backfill (N=10)	21	0	29.5	0	1
Backfill (N=3)	21	0	28	0	1
Peat	12.5	0	25	0	1
Silty Clay-Soft	18.5	0	25	0	8
Silty Clay-Firm	18	0	26	0	9
Silty Clay-Stiff	18.5	0	29	0	2
Silty Clay-Firm	17	0	27	0	2
Hard Bottom	(Infinitely Strong)				

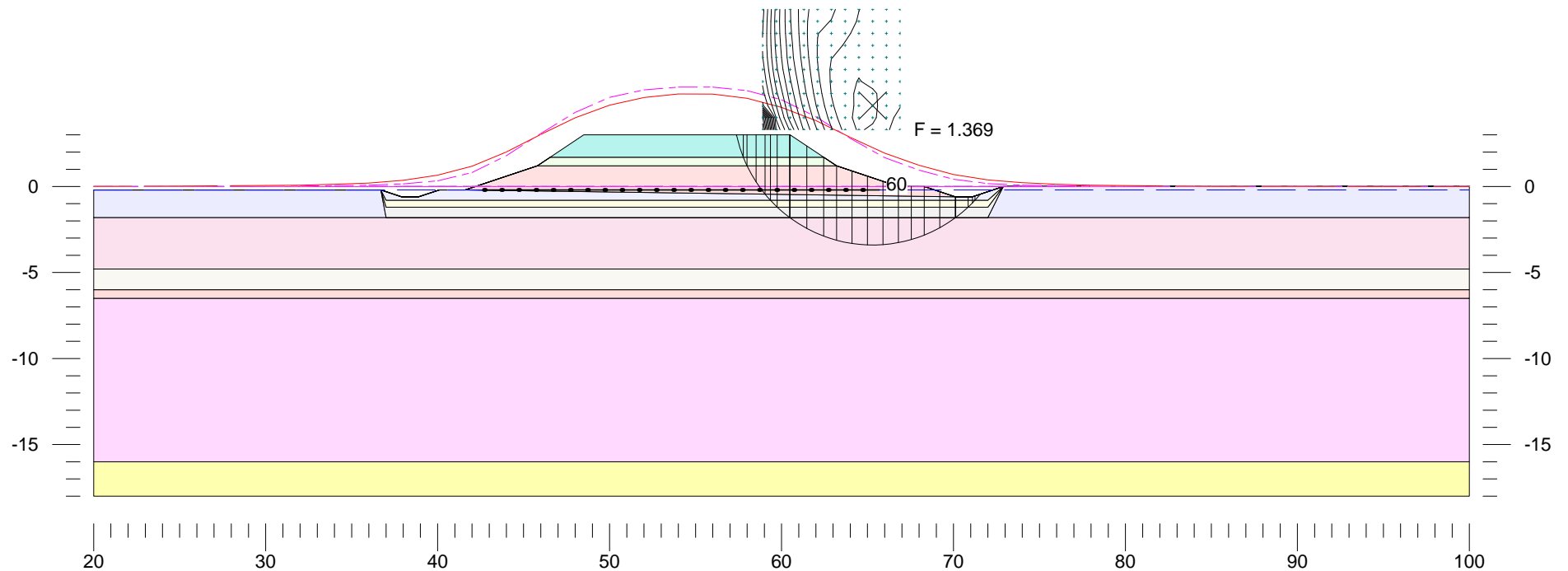
Thurber Engineering Ltd. - Toronto
19-5161-73

Black Creek Road Embankment Failure

January 25, 2010

Black Creek Road Sta.9+000 - 3m high Embankment (1m Surcharge at 1.5H:1V, SSM Fill at 3H:1V)

Effective Stress Analysis - Geogrid Reinforcement



	Gamma	C	Phi	Min	Piezo
	kN/m3	kPa	deg	c/p	Surf.
Schg-B Type II	21.5	0	30	0	1
Compacted B2	22.8	0	33	0	1
SSM Fill	22	0	32	0	1
Backfill (N=20)	21.5	0	31	0	1
Backfill (N=10)	21	0	29.5	0	1
Backfill (N=3)	21	0	28	0	1
Peat	12.5	0	25	0	1
Silty Clay-Soft	18.5	15	0	0	1
Silty Clay-Firm	18	25	0	0	1
Silty Clay-Stiff	18.5	80	0	0	1
Silty Clay-Firm	17	35	0	0	1
Hard Bottom	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-73

Black Creek Road Embankment Failure

January 25, 2010

Black Creek Road Sta.9+000 - 3m high Embankment (1m Surcharge at 1.5H:1V, SSM Fill at 3H:1V)

Total Stress Analysis - Geogrid Reinforcement

