



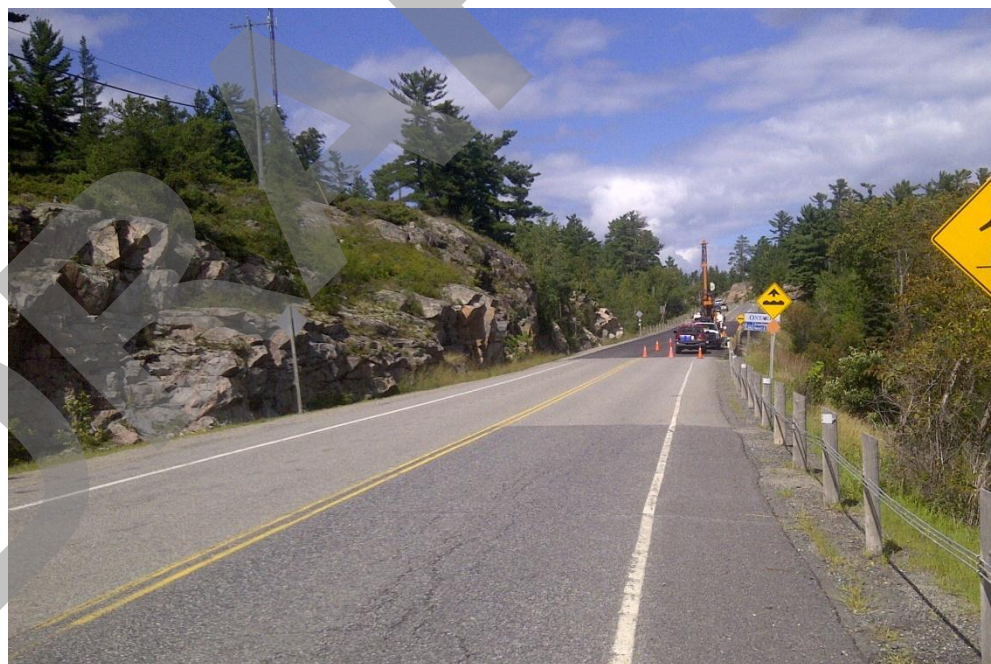
May \_\_, 2013

## FOUNDATION INVESTIGATION AND DESIGN REPORT

**STABILIZATION OF HIGHWAY 71 EMBANKMENT AT  
KAKABIKITCIWAN LAKE, NESTOR FALLS, ONTARIO  
ASSIGNMENT 6011-E-0039  
WORK ORDER NUMBER 2013-11005**

**Submitted to:**

Ministry of Transportation Ontario  
Pavements and Foundations Section  
Foundations Group  
Building 'C', Room 223  
1201 Wilson Avenue  
Downsview ON M3M 1J8



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REPORT





## Table of Contents

### PART A –FOUNDATION INVESTIGATION REPORT

<b>1.0 INTRODUCTION.....</b>	<b>1</b>
<b>2.0 SITE DESCRIPTION.....</b>	<b>1</b>
<b>3.0 INVESTIGATION PROCEDURES .....</b>	<b>2</b>
<b>4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS .....</b>	<b>3</b>
4.1 Regional Geology .....	3
4.2 Subsurface Conditions.....	3
4.2.1 Asphalt.....	4
4.2.2 Fill .....	4
4.2.3 Clay.....	5
4.2.4 Silt, Sandy Silt, Gravelly Sand .....	6
4.2.5 Refusal/Bedrock.....	6
4.2.6 Groundwater Conditions .....	7
<b>5.0 CLOSURE.....</b>	<b>7</b>

### PART B - FOUNDATION DESIGN REPORT

<b>6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS.....</b>	<b>9</b>
6.1 Potential Embankment Failure Mechanisms.....	9
6.2 Stability and Settlement Analysis.....	10
6.2.1 Parameter Selection.....	10
6.2.2 Model Geometry.....	13
6.2.3 Results of Stability Analysis .....	13
6.2.4 Settlement.....	14
6.2.5 Drainage .....	15
6.3 Embankment Stabilization Measures.....	16
6.3.1 Recommended Alternative.....	16
6.3.2 Lowering the Grade.....	17
6.3.3 Incorporating Lightweight Fill into Embankment Mass .....	18
6.3.4 Re-Shaping the Slope .....	19
6.3.5 Stabilizing Toe Berms .....	19



6.3.6	Dry Soil Mixing or Jet Grouting .....	19
6.3.7	Re-aligning Highway 71 to the West .....	21
<b>7.0</b>	<b>CLOSURE.....</b>	<b>21</b>

## REFERENCES

### TABLES

Table 1	Evaluation of Embankment Stabilization/Remedial Measures
---------	--

### DRAWINGS

Drawing 1	Borehole Locations and Soil Strata
Drawing 2	Soil Strata

### FIGURES

Figure 1	Summary Plot of Engineering Parameters for Clay Deposit
Figure 2A	Stability Analysis – STA 11+257, Undrained – LWL, Current/Existing Embankment
Figure 2B	Stability Analysis – STA 11+257, Drained – LWL, Current/Existing Embankment
Figure 3A	Stability Analysis – STA 11+280, Undrained – LWL, Current/Existing Embankment
Figure 3B	Stability Analysis – STA 11+280, Drained – LWL, Current/Existing Embankment
Figure 4A	Recommended Option, Stability Analysis – Option 1A, STA 11+257, Undrained – LWL, 1 m Grade Lowering, 3 m Shift, 1.25H:1V Slope
Figure 4B	Recommended Option, Stability Analysis – Option 1A, STA 11+257, Drained - LWL, 1 m Grade Lowering, 3 m Shift, 1.25H:1V Slope
Figure 5A	Recommended Option, Stability Analysis – Option 1A, STA 11+280, Undrained - LWL, 1 m Grade Lowering, 3 m Shift, 1.25H:1V Slope
Figure 5B	Recommended Option, Stability Analysis – Option 1A, STA 11+280, Drained - LWL, 1 m Grade Lowering, 3 m Shift, 1.25H:1V Slope

## APPENDICES

### Appendix A Photographic Record

### Appendix B Record of Boreholes and Drillholes

List of Symbols and Abbreviations
Lithological and Geotechnical Rock Description Terminology
Record of Borehole Sheets (NF12-1 to NF12-6)
Record of Drillhole Sheets (NF12-1, NF-3 to NF12-6)

### Appendix C Laboratory Test Results

Figure C1	Grain Size Distribution – Gravelly Sand to Sand and Gravel (Fill)
Figure C2	Plasticity Chart – Silty Clay to Clay
Figure C3	Grain Size Distribution –Silt
Figure C4	Grain Size Distribution –Gravelly Sand to Gravelly Sandy Silt
Figure C5	Bedrock Core Photographs (Boreholes NF12-1, NF12-3 and NF12-4)
Figure C6	Bedrock Core Photographs (Boreholes NF12-5 and NF12-6)



**Appendix D      Foundation Design Report, GEOCRETS NO. 52-F-26, DST Consulting Engineers**

Borehole Logs 1, 2 and 3

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# PART A

FOUNDATION INVESTIGATION REPORT  
STABILIZATION OF HIGHWAY 71 EMBANKMENT  
AT KAKABIKITCHIWAN LAKE, NESTOR FALLS, ONTARIO  
MINISTRY OF TRANSPORTATION, ONTARIO  
WO 2013-11005, ASSIGNMENT 6011-E-0039



## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by the Ministry of Transportation, Ontario (MTO), Northwestern Region Geotechnical Section, to provide foundation engineering services for stabilization of an approximately 100 m long section of embankment along Highway 71. The site is located approximately 200 m north of Nestor Falls and is adjacent to Kakabikitchiwan Lake.

The Terms of Reference and Scope of Work for the foundation investigation are outlined in MTO's Request for Proposal (RFP) dated July 18, 2012, and subsequent Work Item Order by Golder, number 12-1191-0005/2100 dated July 20, 2012. This work item is part of retainer Assignment 6011-E-0039. A site visit was carried out on June 19, 2012, with representatives of MTO Northwestern Region, Kenora District and MTO Foundations Section to discuss the past history of the site and potential stabilization alternatives. Available information for this site was provided to Golder by MTO and includes:

- Foundation Design Report, 200 m North of Nestor Falls Bridge, W.P. 306-85-00, Highway 71, District of Thunder Bay, Godson Township, dated March 2000, by DST Consulting Engineers Inc. (DST), GEOCRE No. 52-F-26;
- 2012 Re-Grading Contract Drawings; and
- site photographs.

The purpose of this investigation is to establish the subsurface conditions underlying the existing roadway embankment by borehole drilling, in situ testing and laboratory testing on selected samples to supplement the boreholes drilled in the lake at the toe of slope by DST.

## 2.0 SITE DESCRIPTION

The site is located on Highway 71 in Godson Township approximately 200 m north of the bridge at Nestor Falls, adjacent to Kakabikitchiwan Lake, southeast of Kenora, Ontario. We understand that the highway was constructed in the 1960s by blasting the exposed bedrock on the west side of the highway [Southbound Lane (SBL)] and using the blast rock fill to construct the lower portion of the embankment on the west and east sides of the highway [Northbound Lane (NBL)]. During and since completion of construction, an approximately 100 m long section of the roadway embankment has been noted to be "sliding" down towards the lake. Frequent patching and maintenance has occurred since construction and we understand these remedial measures typically last less than a few months before cracking and pavement distortion recurs. Based on site photographs and anecdotal information, the pavement distress includes longitudinal and crescent shaped cracks, sinkholes and differential settlement extending across both the NBL and SBL, between about STA 11+205 and STA 11+315, with the furthest lateral extent of cracking across the roadway at about STA 11+280. During re-grading work in June 2012, immediately prior to this current subsurface investigation, built-up asphalt pavement up to about 430 mm thick was encountered at some locations. Selected photographs showing the site and the noted cracking and distortion are provided in Appendix A.

The water level in Lake Kakabikitchiwan was measured at Elevation 328.9 m (ice surface) in February 1999 during the field investigation by DST, and between Elevation 328.66 m and Elevation 328.77 m in August 2012 by Tulloch Surveying Ltd (Tulloch) during the current subsurface investigation. The existing highway



embankment is between approximately 5 m and 8 m above the lake water level in the distress area. The water level in the lake is controlled by the Nestor Falls dam located approximately 200 m south of the site and typically fluctuates between about Elevation 328.64 m and 328.79 m although the water level may be higher during extreme precipitation events.

An approximately 5 m high rock cut extends along the west side of the roadway embankment, from the south portion of the impacted section at STA 11+205 northerly to about STA 11+280, where a valley is present on the west side of the highway. Large size rock fill is present on the east slope of the embankment to the lake. The lake extends parallel to the highway along the east toe of slope to about STA 11+280, where it is bounded by natural terrain. Drawing 1 shows the location of the embankment and the presence of the lake. The highway grade rises from south to north, from about Elevation 333 m to 337 m along the impacted embankment area.

### 3.0 INVESTIGATION PROCEDURES

The fieldwork along Highway 71 at Nestor Falls was carried out between August 7 and 21, 2012, at which time a total of six (6) boreholes (NF12-1 to NF12-6) were advanced: four boreholes (NF12-1 to NF12-4) along the east shoulder of Highway 71 (NBL); and two boreholes (NF12-5 and NF12-6) along the west should of Highway 71 (SBL). The locations of the boreholes are shown on Drawing 1.

All boreholes were drilled using an Acker MP8 truck-mounted drill rig supplied and operated by Paddock Drilling Ltd. (Paddock) of Brandon, Manitoba. The boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers, HW casing with wash boring and HQ size rock coring techniques. Soil samples were obtained at intervals of depth of about 0.75 m, using a 50 mm outer diameter (O.D.) split-spoon sampler operated by an automatic hammer on the drill rig, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Selected samples of cohesive soils were obtained using 76 mm O.D. thin-walled 'Shelby' tubes (ASTM D1587, Standard Practice for Thin-Walled Tube Sampling) for relatively undisturbed samples. Field vane shear tests were carried out in the cohesive soil stratum for determination of undrained shear strengths (ASTM D2573, Standard Test Method for Field Vane Strength Shear Test) using an MTO Standard 'N' size vane. All boreholes were backfilled upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The boreholes were advanced to depths ranging between 7.3 m and 12.7 m below ground roadway surface, including between 2.4 m and 3.2 m of bedrock coring. The groundwater conditions and water levels in open boreholes were observed during the drilling operations, where possible, and are described on the Record of Borehole sheets in Appendix B.

Traffic protection was implemented for the boreholes drilled within the roadway in accordance with the Traffic Protection Plan for this project and MTO Book 7 "Temporary Conditions Manual of the Ontario Traffic Manual" (2001).

The fieldwork was supervised throughout by a member of our technical staff, who located the boreholes, arranged for the clearance of underground services at the borehole locations, supervised the drilling and sampling operations, logged the boreholes, and examined and cared for the soil and rock core samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury or Mississauga geotechnical laboratories where the samples underwent further visual examination and laboratory





testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples. A laboratory vane was used to measure the undrained shear strength of cohesive material obtained on a Shelby tube from one borehole (ASTM D4686, Standard Test Method for Laboratory Miniature Vane Shear Test for Saturate Fine-grained Clayey Soil).

The location and ground surface elevation of the as-drilled boreholes were surveyed by Tulloch under sub-contract to Golder. The MTM NAD 83 northing and easting coordinates, ground surface elevations referenced to Geodetic datum and depth of each borehole are presented on the Record of Borehole sheets in Appendix B and are summarized below.

Borehole	Borehole Location		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
NF12-1	5442487.0	237241.2	333.9	9.8
NF12-2	5442513.3	237248.7	334.8	10.8
NF12-3	5442535.4	237256.5	335.8	11.2
NF12-4	5442554.6	237261.8	336.6	12.7
NF12-5	5442540.7	237249.5	336.1	9.4
NF12-6	5442511.9	237239.8	334.8	7.3

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

Based on NOEGTS<sup>1</sup> mapping, the surficial geology in the vicinity of Nestor Falls generally consists of bedrock knobs. Published literature indicates that the site is located in the Wabigoon Subprovince of the Superior Province (OGS, 1991)<sup>2</sup>. The Nestor Falls site lies within the Central Wabigoon Region, which consists primarily of large bedrock areas of granitoid and gneissic rocks.

### 4.2 Subsurface Conditions

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are presented on the Record of Borehole and Drillhole sheets in Appendix B. The detailed laboratory test results of grain size distribution and Atterberg limits are presented in Appendix C. The Log of Borehole 1, 2 and 3 from the DST subsurface investigation in 1999 are presented in Appendix D. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations of drilling progress and cuttings. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations.

<sup>1</sup> Northern Ontario Engineering Geology Terrain Study, Ontario Geological Society Map Reference Number 52FSW.

<sup>2</sup> Ontario Geological Survey, Geology of Ontario, 1991. Special Volume 4, Part 1. Eds P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott, Ministry of Northern Development and Mines, Ontario.





The inferred soil stratigraphy based on the results of the boreholes is shown in profile on Drawing 1 and in cross-section on Drawing 2.

The existing ground surface at the boreholes along Highway 71 ranges from Elevation 333.9 m to 336.6 m sloping upwards from south to north.

In general, the embankment materials consist of granular fill underlain by rock fill, which is in turn underlain by gneiss or granite gneiss bedrock. Three of the six boreholes encountered relatively thin deposits of clay and/or silt, gravelly sand or gravelly sandy silt between the rock fill and bedrock. At the toe of the east embankment slope, the DST boreholes indicate clay from the lake bed extending to refusal or overlying a silty sand deposit to the borehole termination depth. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

#### 4.2.1 Asphalt

A 50 mm to 100 mm thick layer of asphalt was encountered from the roadway surface in five boreholes (NF12-1 to NF12-5). A layer of geogrid was encountered in the boreholes at depths between 0.1 m and 0.4 m below the roadway surface, reportedly placed during the recent re-grading activities.

#### 4.2.2 Fill

Granular fill and rock fill were encountered underlying the asphalt or from ground surface in all the boreholes. The total thickness of the fill is between 4.7 m and 8.4 m.

#### *Granular Fill*

Granular fill consisting of brown gravelly sand to sand and gravel trace to some silt was encountered in Boreholes NF12-1 to NF12-6. The granular fill contains some clay in Borehole NF12-4. The granular fill deposit is between 1.4 m and 4.0 m thick.

SPT 'N'-values measured in the granular fill deposit are between 2 blows and 31 blows per 0.3 m of penetration indicating a very loose to dense relative density. The split spoon sampler was noted to be bouncing on rock fill at two locations.

Grain size distribution tests were carried out on seven samples of the granular fill and the results are shown on Figure C1 in Appendix C. The natural water content measured on seven samples of the granular fill ranges from 3 per cent to 14 per cent.

#### *Rock Fill*

Rock fill was encountered underlying the granular fill in all boreholes. The surface of the rock fill was encountered between Elevations 334.6 m and 329.8 m, being lower in the boreholes advanced through the NBL. The thickness of the rock fill deposit is between 2.8 m and 5.7 m.



Two SPT 'N'-values measured in the rock fill are 2 blows and 17 blows per 0.3 m of penetration indicating a very loose to compact relative density of the soil matrix. Two split-spoon sample drives did not penetrate the full sample depth indicative of the presence of rock fill. For the majority of the thickness of the rock fill deposit, HQ coring was required to advance the boreholes and the total core recovery is between 0 per cent and 100 per cent, typically between about 50 per cent and 90 per cent.

### **DST Boreholes**

It should be noted that four shallow probe holes were advanced by DST from the roadway and encountered refusal on rock fill at depths between 0.8 m and 2.4 m below pavement surface.

#### **4.2.3 Clay**

A deposit of wet, grey clay containing trace sand was encountered below the rock fill in Boreholes NF12-2 and NF12-4. The clay contains trace sand in Borehole NF12-2 and trace wood in Borehole NF12-4. In Borehole NF12-2, the clay deposit was sampled using HQ coring techniques as the core barrel penetrated the rock fill and the limited thickness of clay. The surface of the clay deposit was encountered at Elevations 326.3 m and 328.8 m and the deposit is 0.4 m and 1.3 m thick at the respective boreholes.

In situ field vane test carried out within the clay deposit in Borehole NF12-4 measured an undrained shear strength greater than 100 kPa. A laboratory vane test on a Shelby tube from this deposit measured an undrained shear strength of 116 kPa. The field and laboratory vane test results indicate the clay has a very stiff consistency.

Atterberg limits tests were carried out on three samples of the clay deposit and the results are presented on Figure C2 in Appendix C. The liquid limits range between about 65 per cent and 79 per cent, the plastic limits range between about 20 per cent and 25 per cent and the plasticity indices range between about 46 per cent and 55 per cent. The results indicate the deposit is classified as clay of high plasticity.

The natural moisture content measured on three samples of the clay deposit is between 24 per cent and 41 per cent.

### **DST Boreholes**

In Boreholes 1 to 3, advanced from the lake surface by DST in 1999, the clay deposit was encountered from the lake bed between about Elevation 327.4 m and 326.3 m and is between about 4.3 m and 7.1 m thick. The clay deposit is noted to contain trace organics and trace sand, as well as to be layered, and has zones of sand seams and silt lenses. In situ vane tests measured undrained shear strengths ranging between 12 kPa and 60 kPa and SPT 'N'-values ranging between 0 blows and 2 blows per 0.3 m of penetration. The field vane test results combined with the SPT 'N'-values indicate that the deposit has a soft to stiff consistency. In Boreholes 1 and 2, the field vane measurements were typically greater than about 24 kPa, while in Borehole BH-3 the field vane measurements were typically less than 24 kPa.



Atterberg limits tests carried out on three samples of the clay deposit by DST indicate that liquid limits range between about 46 per cent and 112 per cent, the plastic limits range between about 13 per cent and 33 per cent and the plasticity indices range between about 33 per cent and 79 per cent, classifying the soil as silty clay of intermediate plasticity to clay of high plasticity. The natural moisture content ranges between 30 per cent and 88 per cent.

#### 4.2.4 Silt, Sandy Silt, Gravelly Sand

A deposit of cohesionless soils comprised of wet, grey silt, gravelly sandy silt or gravelly sand was encountered underlying the clay deposit in Boreholes NF12-2 and NF12-4 and below the rock fill in Borehole NF12-5. The deposit is between 0.3 m to 1.8 m thick and the surface of the deposit was encountered between Elevations 329.9 m and 325.9 m.

One SPT 'N'-value measured in the silt portion of the deposit is 21 blows per 0.3 m of penetration indicating a compact relative density, while two values are 5 blows per 0.15 m of penetration and 8 blows per 0.1 m of penetration on or near the underlying bedrock.

Grain size distribution tests were carried out on three samples of this deposit (silt, gravelly sandy silt and gravelly sand) and the results are shown on Figures C3 and C4 in Appendix C.

The natural moisture content measured on three samples of this deposit is between 17 per cent and 30 per cent.

#### DST Boreholes

The DST Boreholes 1 and 3 encountered a 0.1 m and 2.3 m thick deposit of gravel and silty sand, respectively, below the clay stratum. SPT 'N'-values in these deposits are greater than 99 blows and 11 and 31 blows per 0.3 m of penetration indicating a compact to very dense relative density. Borehole 3 was terminated in the silty sand deposit.

#### 4.2.5 Refusal/Bedrock

Refusal to further penetration was encountered in DST Boreholes 1 and 2 at about Elevation 320.0 m and 323.1 m, respectively, corresponding to depths of about 8.9 m and 5.8 m below ice surface. In Boreholes NF12-1 to NF12-6 refusal or the bedrock surface was encountered between Elevation 330.1 m and 324.1 m, corresponding to depths of 4.7 m and 10.7 m below existing ground surface)

Bedrock was cored for lengths ranging from 2.4 m and 3.2 m in Boreholes NF12-1 and NF12-3 to NF12-6. In Borehole NF12-2, 0.1 m of rock was recovered in the casing. The retrieved bedrock core is described as weakly to moderately foliated, medium to coarse grained, fresh, grey, granite to granite gneiss, as presented in the Record of Drillhole sheets in Appendix B. Photographs of the retrieved bedrock cores are shown on Figures C5 and C6 in Appendix C.

The Total Core Recovery (TCR) ranges from 72 per cent to 100 per cent. The Rock Quality Designation (RQD) measured on the core samples ranges from 35 percent to 97 per cent, indicating a rock mass of poor to



excellent quality as per Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006). Typically, the RQD is greater than 65 per cent, except in Borehole NF12-6 where the bedrock is highly fractured.

#### 4.2.6 Groundwater Conditions

The groundwater conditions and water levels in open Boreholes NF12-2 to NF12-5 were observed during the drilling operations and are presented below.

Borehole	Time and/or Date	Depth to Groundwater Below Ground Surface (m)	Groundwater Elevation (m)
NF12-2	Upon completion of drilling	6.2	328.6
NF12-3	Upon completion of drilling	7.9	327.9
NF 12-4	Upon completion of soil drilling	Dry to 9.6	-
NF12-5	Upon completion of drilling	5.8	330.3

Groundwater levels encountered in the boreholes during and upon completion of drilling may not be representative of static levels since the groundwater levels in the boreholes may not have stabilized on completion of drilling.

The water level in Lake Kakabikitchiwan was measured at about Elevation 328.9 m (ice surface) in February 1999 during the field investigation by DST and between Elevation 328.66 m and Elevation 328.77 m in August 2012 by Tulloch Surveying Ltd (Tulloch). We understand from information obtained from the Ministry of Natural Resources (MNR), who owns the dam at Nestor Falls, that the normal water level is at Elevation 328.73 m and that the water level typically fluctuates by approximately 0.15 m (i.e. from Elevation 328.64 m to 328.79 m). We understand that a water level at Elevation 330.16 m was measured in 2002 after a period of extreme precipitation.

## 5.0 CLOSURE

The field drilling program was supervised by Ms. Justyne Biy and Mr. Ed Savard. This report was prepared by Mr. Matthew Thibeault, EIT, and the technical aspects were reviewed by Ms. Sarah E.M. Coyne, P.Eng., Associate. A quality control review of the report was provided by Mr. Jorge M.A. Costa, P.Eng., Principal and Golder's Designated MTO Contact for this project.



## Report Signature Page

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## PART B

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STABILIZATION OF HIGHWAY 71 EMBANKMENT  
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## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides an assessment of potential remedial alternatives and recommendations for remediation/stabilization of a 110 m long section of highway embankment on Highway 71 approximately 200 m north of the Nestor Falls Bridge. The recommendations for remedial measures are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at the site as well as previous boreholes advanced by others.

The interpretation and recommendations presented are intended only to provide the MTO with sufficient information to assess the feasible remediation alternatives. As such, where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the remediation/stabilization measures. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

### 6.1 Potential Embankment Failure Mechanisms

Based on the results of the subsurface investigations, the embankment is constructed of granular fill underlain by large size rock fill with potentially large voids. The rock fill zone of the embankment appears to have been constructed directly on the bedrock surface, although up to 2.2 m thick deposits of clay and/or silt/gravelly sand/gravelly sandy silt were encountered in three of the boreholes: two advanced through the NBL portion of the embankment; and one advance through the SBL portion of the embankment. The rock fill at the east toe of the embankment (toe of the NBL) was likely end-dumped directly through the water and onto the soft clay present at the surface of the lakebed. Site observations indicate that surface water runoff from the roadway over the east slope is causing some erosion of the upper granular portion of the embankment.

As discussed in Section 2.0, there is a long history of embankment distress at this site. However, it is not obvious from the site observations and results of the investigation programs what the immediate cause(s) and contributing factors are to this movement. The following potential embankment failure mechanisms, or combinations thereof, are considered plausible:

- Lateral/vertical displacements of the embankment manifested as longitudinal and crescent-shaped cracks at the roadway surface as a consequence of:
  - instability of the embankment due to the presence of a clay wedge at the toe of slope;
  - lateral spreading of the embankment due to lack of confinement of the fill at the east toe; and/or
  - settlement/compression of the clay at/near the toe of the embankment and resultant change in the geometry of the embankment.

For the three mechanisms noted above, as fill is added to the geometry of the embankment to resurface the road with every repair, the loading on the clay stratum under the embankment east slope and at the toe of the slope increases, leading to further instability or lateral spreading/settlement.

- Drainage from a low-lying area to the northwest of the slip area and surface water runoff from the roadway surface onto the unpaved shoulders could be infiltrating into the rock fill mass portion of the embankment thereby washing out the fines from the rock fill and promoting the overlying granular fill to penetrate into the voids. Movement of infiltrated surface water, washing out of fines from the rock fill and carrying of fines





from the overlying granular material into the voids within the rock fill mass promote instability of the embankment through loss of material and rearrangement of the embankment fill mass.

- The lake current (towards Nestor Falls, located about 200 m downstream), daily/seasonal changes in lake water level and/or ice action at the east toe of slope, could be infiltrating the rock fill and washing out the fines allowing additional granular fill to penetrate into the voids or rearranging the rock fill mass.
- Rock fill sliding along the bedrock surface (potentially aided by water seepage).

Considering the above-noted potential causes of instability/distress of this section of the roadway embankment, stability and settlement analyses were carried out to assess whether the causes of the distress may be due to sliding and/or compression of the underlying clay stratum or continuous compression/consolidation of the rock fill mass. Section 6.2 presents the assumptions made, the methodology used and parameter selection used in the analyses, as well as the results of stability and settlement analysis. Options for remedial measures are discussed in Section 6.3.

## 6.2 Stability and Settlement Analysis

### 6.2.1 Parameter Selection

Details of the soil and groundwater conditions at the site are given in Section 4.2. Summarized below are the simplified stratigraphy and the associated unit weights, undrained shear strengths and deformation properties employed in the subsurface model for the different soil types encountered at the site. The design lines for the magnitude of undrained shear strength, pre-consolidation pressure and index properties (water content and Atterberg limits) used for correlation purposes versus elevation, are presented on Figure 1 and summarized below.

Deposit	Thickness (m) at STA 11+257	Unit Weight	Undrained Shear Strength*	Angle of Internal Friction	Deformation Properties
Granular Fill	3.4	20 kN/m <sup>3</sup>	-	30°	-
Rock Fill	5.0	19 kN/m <sup>3</sup>	-	42°	-
Clay	7.1 (DST Boreholes)	16 kN/m <sup>3</sup>	Upper zone 20 kPa	Residual Friction Angle: 22°	$\sigma_{vo}' = 0$ to 45 kPa $\sigma_p' = 90$ to 180 kPa $e_o = 0.6 - 3.0$ $C_c = 3 - 1.3$ $C_r = 0.03 - 0.13$ (see Figure 1) $C_{\alpha\epsilon} = 0.22 - 0.88$
			Lower zone 20 kPa to 40 kPa (see Figure 1)	Friction Angle: 22°	
Silt/Gravelly Sand/Sandy Silt	2.3	20 kN/m <sup>3</sup>		30°	E = 20 MPa
Bedrock	-	-	-	-	-
EPS Fill**	-	0.5 kN/m <sup>3</sup>	Cohesion = 15 kPa	-	-
Cellular Concrete**	-	5 kN/m <sup>3</sup>	Cohesion = 4 kPa	37°	-

\*Corrected for plasticity as per Bjerrum (1973)

\*\* Used for mitigation alternatives.



## Cohesionless Deposits

For the cohesionless deposits including the granular fill, effective stress parameters were employed in the stability analysis assuming drained conditions and the parameters were estimated from empirical correlations using the results of the in situ SPT 'N'-values. The correlations proposed by Peck et al. (1974), Schmertmann (1975) and NAVFAC (1982) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils. The angle of internal friction the existing rock fill is estimated to be 42 degrees based on Figure 14 from MTC (1982), for the estimated stress level at the base of the rock fill embankment.

The immediate compression of the native cohesionless deposits was assessed by estimating an elastic modulus of deformation based on the SPT 'N'-values and empirical correlations found in literature by Bowles (1984) and Kulhawy and Mayne (1990).

## Clay Deposits

For the clay deposit, total stress parameters were employed in the stability analysis. The total stress parameters (i.e. average mobilized undrained shear strength –  $s_u$ ) for the cohesive soils were assessed based on the results of the in situ field vane tests and estimated from correlations with the SPT 'N'-values and other laboratory test data. Where appropriate, Bjerrum's (1973) correction factor as a function of the plasticity index of the soil was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests. The assumed remoulded strength of the clay was based on the DST report which notes that the clay is "sensitive to extra-sensitive" suggesting a sensitivity between 4 and 16.

In order to develop the design line of the undrained shear strength of the clay deposit shown in Figure 1 for stability analysis of the existing embankment slope and future remedial conditions, the range of values of the field (uncorrected) in situ vane undrained shear strengths were used to estimate the FoS of the existing embankment slope. The undrained shear strength of the clay deposit was then "adjusted" until a FoS close to unity was achieved. The "adjusted" undrained shear strength was then compared to the undrained shear strength corrected for plasticity as per Bjerrum (1973). This analytical process was repeated until the undrained shear strength design line was considered "reasonable", taking into account the fact that the clay deposit is somewhat sensitive as per the DST (1999) investigation. This undrained shear strength design line was then used in the assessment of embankment stability for alternative remedial measures. It is reasonable to assume that the upper zone of the clay deposit has undergone partial remoulding due to placement of the rock fill or remoulding as a result of the ongoing movement. The undrained shear strength of the upper portion of the clay deposit above Elevation 324.5 m is lower than the undrained shear strength of the clay deposit below this elevation under the slope of the embankment and of that beyond the toe of the embankment slope.

The effective stress parameters for the clay deposit (effective friction angle and cohesion) for evaluating long-term drained conditions were estimated using empirical correlations with plasticity index (PI) proposed by Mitchell (1993), Ladd et al. (1977) and Kulhawy and Mayne (1990).

The residual friction angle was estimated from Lo (1995) and Bjerrum (1968). Similar to the undrained shear strength, the upper portion of the clay deposit has likely been partially remoulded due to disturbance as noted



above and therefore, some reduction in the friction angle is appropriate in this portion of the clay deposit above Elevation 324.5 m.

The consolidation settlement of the clay deposit was assessed using the results of the in situ field vane tests carried out by DST to estimate the deformation parameters for these soils (consolidation tests were not carried out). In addition, the results of the laboratory index testing were also employed to estimate deformation parameters using empirical correlations proposed in literature by Terzaghi and Peck (1967), Kulhawy and Mayne (1990), Azzouz et al. (1976) and Koppula (1986).

The degree of over-consolidation in the cohesive strata, required in the analyses, was estimated from the results of the in situ field vane tests and the following correlations relating mobilized undrained shear strength to preconsolidation pressure:

$$S_{u(mob)} = 0.22\sigma_p' \text{ (after Mesri, 1975)}$$

where:  $S_{u(mob)}$  = average mobilized undrained shear strength (kPa)  
 $\sigma_p'$  = pre-consolidation stress (kPa) (see Figure 1)

and

$$S_{u(mob)} = \mu S_{u(FV)} \text{ (after Bjerrum, 1973)}$$

where:  $S_{u(mob)}$  = average mobilized undrained shear strength (kPa)  
 $S_{u(FV)}$  = undrained shear strength from field vane test (kPa)  
 $\mu$  = Bjerrum's correction factor based on Plasticity Index (i.e. ranging from 0.61 to 0.88 for this site as Plasticity Indices range from about 33 per cent to about 79 per cent).

It is known that some secondary consolidation settlement of clay stratum occurs following the completion of primary settlement. This secondary settlement, or creep settlement, occurs over the long term (i.e. decades) for the normally consolidated clays at this site. The magnitude of secondary (creep) settlement (Mesri, 1973 as quoted in Holtz and Kovacs, 1981) was estimated as the follows:

$$S_c = C_{\alpha\epsilon} \times L_o \times (\Delta \log t)$$

Based on Mesri (1973), the following empirical correlation was utilized to estimate  $C_{\alpha\epsilon}$  from water content:

$$C_{\alpha\epsilon} = w_n/100$$

where:  $S_c$  = secondary (creep) settlement (mm)  
 $C_{\alpha\epsilon}$  = modified secondary compression index (%)  
 $L_o$  = initial thickness of compressible deposit (mm) in the normally consolidated portion of the deposit  
 $w_n$  = water content (decimal)  
 $t$  = time period of interest



Based on the results of empirical correlations with laboratory data (NAVFAC, 1982), a coefficient of consolidation,  $c_{v(n/c)}$ , equal to  $7.1 \times 10^{-4} \text{ cm}^2/\text{s}$  is considered appropriate for the normally consolidated range; and a  $c_{v(o/c)}$ , equal to  $2.8 \times 10^{-3} \text{ cm}^2/\text{s}$  is considered appropriate for the recompression range of the clay stratum at this site. The modified secondary compression index,  $C_{\alpha\epsilon}$ , used in the analysis to calculate creep is 0.32 above Elevation 324.0 m and 0.87 below this elevation.

### Water Level

The piezometric surface used in the stability analysis model is estimated to follow the bedrock surface until the lake water level is encountered (i.e. within the rock fill mass). The lake water level used in the model is the normal lake water level at Elevation 328.73 m, based on information from MNR. A water level at Elevation 328.64 m was also used in the analysis as a typically low water level, which governs stability in open water situations, then increasing to the bedrock surface elevation where the surface is higher than the lake low water level.

## 6.2.2 Model Geometry

The existing east slope of the embankment is formed at about 1.6 horizontal to 1 vertical (1.6H:1V) as determined by the contours and profiles provided by Tulloch. Analyses were performed on two critical sections where the furthest lateral extent of cracking/slipping has been historically observed – STA 11+257 and STA 11+280, as well as based on observation of details shown on photographs, anecdotal evidence, all correlated to stations on the drawings provided by MTO. Assumptions regarding the soil stratigraphy have been made between the boreholes drilled through the NBL and the DST boreholes drilled in the lake. The slope stability analysis model geometry and stratigraphy are shown on Figures 2 and 3 for the critical sections identified above. Additional boreholes drilled through the embankment slope would be required to refine the subsurface geometry any further (i.e. to confirm the presence and thickness of clay, if any, is below the embankment slope); however, due to the presence of large rock fill on the slope, drilling such boreholes is likely not readily possible.

## 6.2.3 Results of Stability Analysis

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.19), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. Based on the subsurface model geometry for this site, it is considered that circular or “rotational” failure surfaces are not the most realistic failure mode, rather, block or “wedge” failure surfaces are likely the most representative. In addition, the slip surfaces were chosen to correspond to the approximate location of the furthest lateral extent of cracking evident on the site photographs and site observations. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static conditions at the end of construction (undrained) and for long-term effective stress (drained) conditions.



A summary of the calculated FoS for the current/existing embankment is summarized below for the low water level (LWL) condition.

Location	Analysis Case	FoS*	Figure
STA 11+257	Undrained	1.12	2A
	Drained	1.15	2B
STA 11+280	Undrained	1.07	3A
	Drained	1.16	3B

\*In some cases, the computer program generates an "optimized" slip surface which is typically the lowest factor of safety; however in most cases, the "optimized" slip surface is not considered representative of a realistic slip surface and is therefore not shown on the figures.

Since the movement of the embankment has been ongoing over approximately the last 60 years, but a catastrophic failure has not occurred, the calculated FoS of slightly above 1.0 for the chosen parameters, model stratigraphy and slope geometry "makes sense" and it is considered realistic since the failure mechanism is likely a slow creeping movement.

As the embankment was constructed many years ago, it is likely in a drained (effective stress) state. However, as there has been observed movement of the embankment consistently over this 60-year time period, the undrained shear strength is likely being mobilized and excess pore pressure build up. Therefore, both the drained and undrained conditions were considered in the stability analysis. Since the resultant FoS for both cases is close to (i.e. slightly above) 1 and numerically about the same for both conditions, or reasonably close to each other, the results of the analysis for the current embankment state and selected parameters are considered appropriate. However, since the friction angle for the clay deposit is estimated from empirical correlations with the limited data and was not directly measured by triaxial testing, the actual residual friction angle could be lower than that estimated, thereby resulting in a lower Factor of Safety for stability. Further, the re-molded shear strength and residual friction angle of the underlying clay deposit may continue to decrease over time, and eventually, the embankment may fail.

## 6.2.4 Settlement

To estimate the magnitude of anticipated settlements, analyses were carried out using hand/spreadsheet calculations. The calculations were carried out at the critical section at STA 11+257 where the thickest clay (7.1 m in DST Borehole 3) and greatest thickness of fill (total 8.4 m comprised of 3.4 m granular material over 5.0 m of rock fill) was encountered, and assuming that the full weight of the embankment is acting on the clay stratum. The rate of settlement of the cohesive foundation soils was assessed using Terzaghi's one-dimensional consolidation theory. The model geometry and stratigraphy are shown on Figure 2, as used for the stability analyses for the same critical section.

As it is postulated that the clay stratum present under the eastern portion of the existing embankment is wedge-shaped from 0 m to about 3.7 m thick (i.e. not present at the centreline of the roadway and 2.5 m to 3.7 m thick at the toe of the embankment) it is unlikely that settlement of the embankment is the primary mechanism causing the observed distress. However, settlement of the clay stratum under the toe of the rock fill embankment, where the stratum is thickest, may be occurring.



Based on the plot of preconsolidation pressure and calculated initial vertical effective stress versus elevation shown on Figure 1, the clay deposit beyond the toe of the embankment is considered to be in an over consolidated state since there has been no loading from the adjacent embankments. However, based on the calculated initial vertical effective stress on the clay deposit below the midpoint of the embankment slope, the clay, if present (as per Drawing 2), could be in the normally consolidated state and could experience additional consolidation settlement.

The primary settlement of the clay under the embankment slope is considered to have already occurred, however, some secondary (creep) settlement may still be occurring. It is estimated that approximately 45 mm of creep settlement may have occurred since completion of primary settlement; but, literature suggests that estimates of creep using the Mesri (1973) method may be low. It is possible that some additional primary settlement has been occurring over the years as a result of embankment re-grading and the placement of additional fill. Although a layer of asphalt was reportedly encountered at a depth of 0.43 m below the roadway surface during the last rehabilitation works, the addition of a 0.43 m thick layer of fill would result in only approximately 35 mm of additional primary settlement. The creep settlement combined with the possible additional primary settlement is not considered of sufficient magnitude to account for the ongoing movement noted at the roadway surface, thereby supporting the rationale that slope instability is the primary cause of the observed embankment distress.

### 6.2.5 Drainage

The presence of a topographic low area (i.e. “valley” or “ditch”) to the northwest of the roadway section exhibiting distress may be a contributing condition to the roadway instability albeit it is not considered to be the major contributing factor to the instability based on the observed deformation at the top of the embankment. Since the adjacent lake water level does not fluctuate appreciably, it is possible that during periods of precipitation, the water level in the “valley/ditch” rises and drains through the embankment, possibly along the base of rock fill and interface with the clay deposit, where it is present. We understand that there are no recorded measurements of the magnitude of vertical or lateral movement of this roadway section nor is there any information on the rate of movement, or if the rate of movement varies during lake water level fluctuations or rainfall events. Observations and actual measurements over time coupled with anecdotal observations of how the northwest “valley” and ditch system is functioning during storm events, could help in narrowing down the cause(s) of the local distress.

Another potential contributing factor to distress, albeit less likely than other factors, is the observation of the evidence of erosion of the east side slope in the upper granular fill material, as well as the presence of large rock fill fragments observed on the surface of the slope and encountered during the borehole drilling, suggesting that the portion of the embankment comprised of rock fill contains large sizes of rock and voids between the rock pieces. Further, the core recovery within the rock fill, ranging from 0 per cent to 50 per cent in places, suggests that there are voids within the rock fill mass that may or may not be filled with granular fill material. Therefore, it is possible that a portion of the ongoing movement could be attributed to infiltration or migration of fines from the overlying granular fill into the rock fill voids. However, the location of the cracking, as manifested at the top of the embankment/pavement, and the ongoing nature of the cracking (over many years) are not consistent with loss of fines as a contributing factor to the instability.





## 6.3 Embankment Stabilization Measures

Based on the assessment of the subsurface conditions, the results of the stability and settlement analyses and the historical and anecdotal (photographic) observations at the site, it is considered that the embankment distress is most likely due to the presence of the clay stratum under the east slope of the embankment and likely extending some distance under the embankment as a wedge between the rock fill and the underlying silt/silty sand deposit and possibly the bedrock surface. The noted instability could also be aided by flow/drainage of infiltrated surface water through/below the embankment. The remediation alternatives considered pertinent to mitigating the instability of the embankment include:

- lowering the grade;
- incorporating lightweight fill into embankment mass;
- re-shaping the slope;
- constructing stabilizing toe berms;
- densifying the clay stratum using dry soil mixing (i.e. “shear key”), or jet grouting through the clay stratum below the embankment slope;
- re-aligning the highway to the west; and/or
- a combination of the above remedial options.

The proposed remediation alternatives presented above have been evaluated and ranked on the basis of advantages, disadvantages, relative costs and risk/consequences and are summarized in Table 1. We recommend that a combination of lowering the grade, re-aligning the roadway to the west and re-shaping the slope be implemented to improve short- and long-term stability of the embankment.

These alternatives are presented for consideration by MTO which, together with other impacts of each alternative not considered herein such as property acquisition requirements, construction scheduling, and the like, should also be included in the MTO's choosing of a preferred alternative.

### 6.3.1 Recommended Alternative

Based on a comparison of the remedial alternatives presented in Table 1, the recommended alternative is to lower the grade by about 1 m, shift the highway to the west by 3 m, and re-grade (i.e. steepen) the rock fill slope to 1.25 Horizontal to 1 Vertical (1.25H:1V) to Elevation 328.9 m along the section of roadway from STA 11+205 to STA 11+315. The three components of this option are presented as stand-alone items in the following sections of the report but are not considered to be either technically or practically feasible on their own.

Grade lowering alone will not improve the FoS to greater than 1.3, therefore shifting the highway as far to the west as is practical is also required. However, since the maximum grade lowering and highway shift are limited due to the overall site constraints, this combination of grade lowering and highway shift will still not enhance the FoS to greater than 1.3. Therefore, re-grading/steepening the rock fill embankment east slope to 1.25H:1V to just above the lake water level (Elevation 328.9 m) is still required to improve the FoS to 1.3 for drained





conditions. A summary of the calculated FoS is summarized below for the low water level condition for the recommended option.

Location	Analysis Case	FoS*	Figure
STA 11+257	Undrained	1.46	4A
	Drained	1.30	4B
STA 11+280	Undrained	1.37	5A
	Drained	1.30	5B

\* In some cases, the computer program generates an "optimized" slip surface which is typically the lowest factor of safety; however in most cases, the optimized" slip surface is not considered representative of a realistic slip surface and is therefore not shown on the figures.

Since the existing rock fill slope will be re-graded/steepened to just above the lake water level (i.e. to Elevation 328.9 m), the portion of the rock fill embankment below the water should be left in place. This steepening results in a partial offloading of the clay soils under the east slope as well as maintain an effective "toe berm" extending about 6 m beyond the new toe of slope. It should be noted that re-grading the slope may be difficult due to the large size of the rock fill observed on the slope (see Photo 8 in Appendix A). In this regard, additional but smaller size, well-graded rock fill may be required to fill in any "gaps" and surface voids present on the excavated slope surface, and provide a "chinked" embankment mass. Some localized benching or "keying in" may be required and should be in accordance with OPD 208.010 (Benching of Earth Slopes).

This option also assumes that a suitable ditch is incorporated/maintained along the west side of the highway to promote drainage. Some controlled blasting may be required to extend or deepen the ditch as we understand that it is not desirable to relocate the utilities that are present at the top of the existing rock cut. The use of line drilling, pre-shearing or cushion blasting is recommended in order to minimize shattering and over-break and to minimize face instability resulting from blast damage to the rock mass. Further, rock face stabilization measures such as catch fencing, rock bolts and/or shotcrete may also be required to prevent rocks from falling into the ditch and/or onto the highway. A rock fall hazard assessment should also be carried out and any such rock blasting or stabilization measures as required, depending on the final highway alignment/grading, should be designed by others.

### 6.3.2 Lowering the Grade

Lowering the grade along the impacted section of embankment would reduce the load on the clay stratum under the east toe of the embankment. Similar to the lightweight fill option, the reduction of load within the roadway limits will still result in a FoS of less than 1.3 if used as a stand-alone option, particularly in the drained case.

Given the proximity of the Nestor Falls dam to the south, the presence of a rock outcrop to the west, the increasing highway grade and curve (and private driveway) to the north and the presence of a lake along the east toe of slope, it may not be possible to lower the grade significantly. Even with a 1.5 m grade lowering across the highway for the 110 m long embankment section, the FoS will be less than 1.3. Therefore, this option would only be suitable in conjunction with other partial options, such as the use of lightweight fill and construction of toe berms (discussed in Section 6.3.1), as well as drainage improvements (Section 6.3.7).



### 6.3.3 Incorporating Lightweight Fill into Embankment Mass

In order to improve the stability of the embankment at this site, the use of lightweight expanded polystyrene (EPS) fill or cellular concrete could be used to reduce the embankment fill load on the clay stratum underlying the east toe of the embankment. Even incorporating a thickness of 3 m of EPS or cellular concrete into the embankment mass, the reduction of load within the roadway limits would still result in stability with a FoS less than 1.3, particularly in the drained case. Also, given the thickness of lightweight fill required, the need to excavate into the rock fill and the limited roadway width within which to work, this option is not considered technically feasible as a stand-alone option. Therefore, this option should be considered in conjunction with other options such as grade lowering and/or toe berms.

In the case of using EPS, the EPS blocks would be required to extend along the full 110 m length of the impacted area and extend the full width of the embankment. The top of the EPS blocks should be covered with a 6 mm thick polyethylene sheet, a 125 mm thick reinforced concrete slab, and provided with a minimum 1 m of conventional granular cover (pavement structure) for ballast and for protection against differential icing at the roadway structure. The subgrade should be properly graded to promote drainage and a 300 mm thick layer of Granular 'A' or Granular 'B' Type II (or combination thereof) would be required under the EPS blocks as levelling and protection bed. The surface of the existing rock fill should be chinked and a geotextile and/or geogrid should be placed between the rock fill and the granular material to prevent particle migration of the overlying granular bedding into the underlying coarser more open structure of the rock fill mass.

The EPS blocks would need to be placed as a continuous unit and therefore, consideration would have to be given to closing the road for a limited period of time, although it may be possible to incorporate the EPS into the embankment mass and re-construct the roadway embankment in stages. Further, the lateral extent of the EPS towards the lake is limited as guide rails cannot be installed through the EPS mass.

In the case of the use of cellular concrete, a minimum of 0.5 m of soil cover is required as the insulative properties of cellular concrete are different than EPS; as well, the cellular concrete has better load spreading capabilities than EPS due to its strength. Migration of the fresh cellular concrete slurry into the existing rock fill would need to be prevented such as by the use of a membrane or granular layer placed between the rock fill and concrete.

Cellular concrete will likely have to be placed in stages using some kind of formwork to contain the slurry until it sets. In either case, an excavation up to about 3 m below the existing grade would be required and shoring may be required depending on the actual staging/detours that may be possible. Unlike EPS, the lateral extent of the cellular concrete towards the lake is not limited as guide rails can be installed through the cellular concrete. This constructability issue would need to be explored further and a method developed if this alternative is considered to be the preferred alternative by MTO.

Given the long section of embankment requiring lightweight fill, the cost of supply and installation, including excavation and subgrade preparation, may be an order of magnitude higher than natural (earth/granular) fill materials. It is estimated that the cost of EPS and cellular concrete are about of the same magnitude.



### 6.3.4 Re-Shaping the Slope

Since the embankment is constructed mainly of rock fill, consideration could be given to re-grading/steepening the slope to the stable rock fill slope angle of approximately 1.25H:1V. As discussed above, steepening the slope would result in a net unloading of the clay subsoils under and beyond the east slope of the embankment, which will improve stability. However, re-grading/steepening the slope as a stand-alone option would not result in a FoS greater than 1.3 and therefore, it must be combined with other options to be most effective (see Section 6.3.1).

### 6.3.5 Stabilizing Toe Berms

To improve stability and reduce lateral spreading of the embankment, consideration could be given to constructing a stabilizing berm along the east toe of the slope. The stabilizing berm could consist of rock fill or other granular fill material, however, rock fill is typically best suited for sub-aqueous placement as will be the case at this site. Unlike grade lowering or use of lightweight fill, a toe berm can be used as a stand-alone option since the toe berm results in an increase in passive resistance and hence an increase in the FoS to 1.3 for both the drained and undrained cases. Given the length over which the berm is required and the large width of toe berm required, the resultant quantity of fill required would be substantial.

The disadvantage of this option is that the filling would take place in the water and environmental approvals would be required from the regulatory bodies – it is possible that approvals may be denied depending on the environmental and cultural sensitivity of the lake and area. Further, in-water restrictions, if any, would apply and could impact construction schedule. Sediment control measures would also be required. The cost of the fill material and the in-water work will be high for this option if used as a stand-alone option.

In order to achieve a FoS greater than 1.3 for stability in both the drained and undrained conditions, an 11 m wide berm extending the entire 110 m length of the distress area, with a top at Elevation 330 m, would be required (i.e. for an approximate thickness of 3 m relative to the lakebed average Elevation 327 m). An additional 1 m thickness of fill should be allowed for in estimating the required quantity of rock fill to account for penetration into the clay stratum. Due to the compressibility characteristics of the soft clay, the loading from the fill material on the clay at the lake bed will result in settlement of the clay with time, and the berm will settle as the clay stratum consolidates, potentially losing the buttressing ability on the roadway embankment. It is estimated that up to between about 200 mm and 315 mm of primary settlement of the 4.3 m to 7.1 m thick clay stratum could occur under a 3.5 m high toe berm (3 m high plus an assumed 0.5 m penetration of rock fill into the clay). A portion of this settlement could translate into movement (i.e. settlement or lateral movement) at the ground surface, particularly in the NBL.

However, if a toe berm is used in conjunction with other options such as grade lowering and lightweight fill, then the width of the toe berm can be reduced, thus reducing supply/placement costs and reducing the footprint of environmental impact.

### 6.3.6 Dry Soil Mixing or Jet Grouting

Consideration could be given to enhancing the engineering properties (i.e. shear strength) of soft/weak clay soils in situ to create a “shear key” or a zone of increased shear strength within the clay stratum. This option can be considered a stand-alone option.



This treatment would be most effective under the toe of the embankment slope, from about the water line at the embankment slope to about 5 m into the embankment, for the entire length of the distressed section of embankment. Improving the engineering properties of the clay would reduce lateral spreading of the embankment and improve stability, thereby reducing the cracking of the pavement/embankment. Improving the engineering properties of the clay beyond the toe of the slope (i.e. from the water line towards the lake) would not be as effective in increasing the FoS.

Two methods may be possible to enhance the engineering properties of the clay stratum – Dry Soil Mixing (DSM) and Jet Grouting. The DSM method would encounter the constructability problem of installing the ‘columns’ through the rock fill embankment. Some details are provided below and more design details can be provided should this be the preferred option, although it is likely not as technically and economically feasible an alternative for remediation compared to other options discussed above. Further, both soil improvement methods may have some impact on the water or would have to be installed from the water (as opposed to from the top of the embankment/roadway) and therefore environmental approvals, in-water restrictions and sediment control measures may be required. The cost of specialized equipment, design, additional investigation and in-water work would be significant for this option.

The feasibility of each type of ground improvement discussed below would have to be confirmed by a firm that specializes in the specific technique of soil improvement.

### ***Dry Soil Mixing (DSM)***

Enhancing the shear strength of clayey soils by the addition of cement and/or lime (cement-stabilization or lime-stabilization) has been utilized in the past, mostly to improve the subgrade characteristics of roadways prior to pavement structure construction. However, such an approach could be considered for this site to improve the strength of the clay stratum under the east toe of the slope. The details of the actual method of construction/in situ mixing would require additional design and perhaps even field trials to assess its effectiveness. However, the mixing could be carried out in strips of limited width across the slope near the toe.

Based on literature review along with previous experience on cement-stabilization projects, it is understood that mixing between about 5 per cent and 15 per cent of cement, by mass, into a clayey soil is a typical approach used to improve the geotechnical characteristics of the clay stratum. DSM is typically used in high groundwater conditions and can be carried out in sub-aqueous conditions, such as is the case at Nestor Falls. The DSM process involves mixing individual “soilcrete” columns or rows of columns and does not produce any cuttings/spoil. As noted above, the largest barrier to DSM is the presence of the rock fill and the proximity of the lake and limited working area.

### ***Jet Grouting***

Jet grouting involves injecting high velocity grout into the soil matrix to produce “soilcrete” panels or buttresses. Typically, up to 2 m diameter jet grout columns are constructed in an overlap in order to create a series of buttresses. The buttress columns initially would be placed at approximately 7.5 m spacing along the 100 m length of the impacted section, to replace approximately 25 per cent of the clay stratum to a specified strength. The need for closer spacing or overlap of buttresses would need to be confirmed. The equipment used to inject



the grout should be able to penetrate the rock fill. The details of the actual method of construction/grouting would require additional design and perhaps even field trials to assess the effectiveness.

### 6.3.7 Re-aligning Highway 71 to the West

Consideration could be given to re-aligning the highway to the west such that a portion of, or the entire width of the roadway lies over the bedrock outcrop. The advantage of re-aligning the highway is that the new embankment would be constructed over bedrock, including the east toe of the slope, and thus eliminate potential instability associated with the clay stratum under the toe and east slope of the embankment. Other advantages of this option include not impacting the lake (in terms of in-water work) and the ability for maintaining traffic on the existing embankment during construction. This option is a stand-alone option and does not need to be combined with other mitigation options.

Disadvantages of this option include the proximity of the Nestor Falls dam (located approximately 200 m to the south) which may restrict the space available to allow for adequate re-alignment to the south, the need to acquire additional property, and the potential need to relocate utilities. These impacts would have to be explored by MTO to determine the ultimate feasibility of the re-alignment option along with the cost implications.

In order to achieve a FoS of greater than 1.3, the highway centreline would have to be moved approximately 8.5 m to the west. It may be possible to optimize/reduce the distance/width of realignment to less than an 8.5 m offset, depending on constructability issues that may be identified by MTO.

Consideration should be given to a partial highway shift to the west which, combined with other options as noted in Section 6.3.1, results in a FoS greater than 1.3 (i.e. this option of partial shift to the west is a component of the overall technically preferred/recommended option).

## 7.0 CLOSURE

This report was prepared by Ms. Sarah Coyne, P.Eng., with technical input provided by Paul Dittrich, PhD., P. Eng., Principal. A quality control review of the report was provided by Mr. Jorge M.A. Costa, P.Eng., Principal and Golder's Designated MTO Contact for this project.



## Report Signature Page

**GOLDER ASSOCIATES LTD.**

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[http://capws.golder.com/sites/p211910005mtonwretainerthunderbay/2100 nestor falls/5reporting/3final report/12-1191-0005-2100 rpt 13may\\_\\_\\_ final fidr nestor falls.docx](http://capws.golder.com/sites/p211910005mtonwretainerthunderbay/2100%20nestor%20falls/5reporting/3final%20report/12-1191-0005-2100%20rpt%2013may___final%20fidr%20nestor%20falls.docx)





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Schmertmann, J.H., 1975. Measurement of In-Situ Shear Strength. In Proceedings, ASCE Specialty Conference on In-Situ Measurement of Soil Properties, Vol. 2, Raleigh, pp. 57-138.

Terzaghi, K. and Peck, R.B., 1967. Soil Mechanics in Engineering Practice, 2<sup>nd</sup> Edition, John Wiley and Sons, New York.

ASTM International

- ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
- ASTM D1587 Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes
- ASTM D2573 Standard Test Method for Field Vane Shear Test in Cohesive Soil
- ASTM D4648 Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-grained Clayey Soil

Commercial Software

GeoStudio (Version 7.19) by Geo-Slope International Ltd.

Ontario Provincial Standard Drawings

OPSD 208.010 Benching of Earth Slopes

Ontario Water Resources Act

- Ontario Regulation 468/10 Amendment to Ontario Regulation 903
- Ontario Regulation 903/90 Wells



**FOUNDATION INVESTIGATION AND DESIGN REPORT**  
**HIGHWAY 71 EMBANKMENT AT NESTOR FALLS, WO 2013-11005**

**Table 1: Evaluation of Embankment Stabilization Remediation Measures**

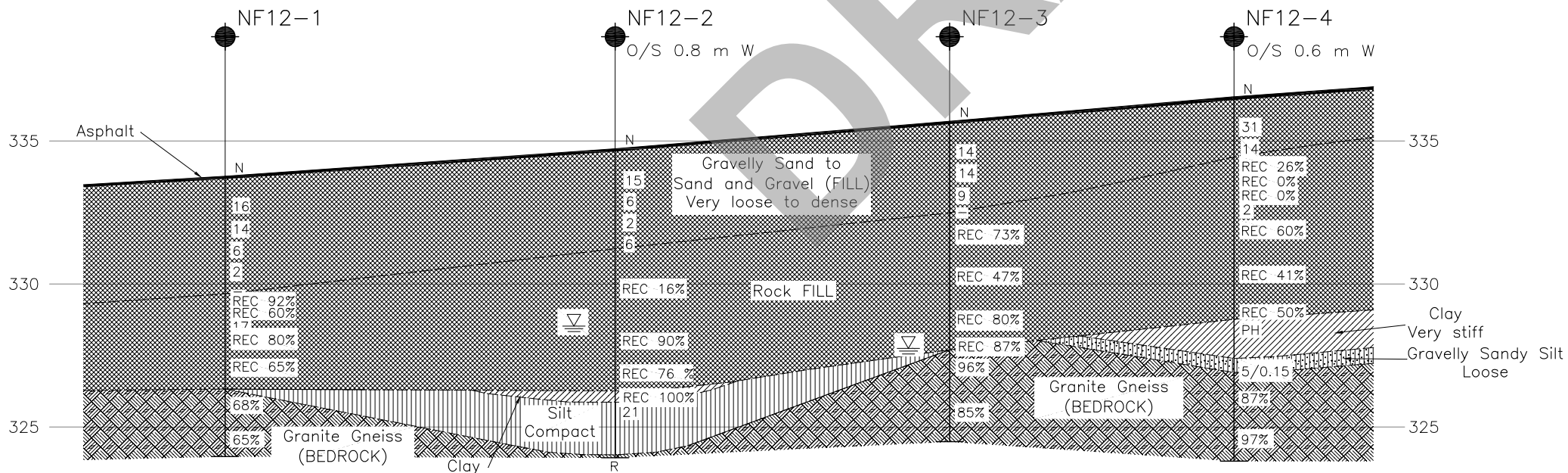
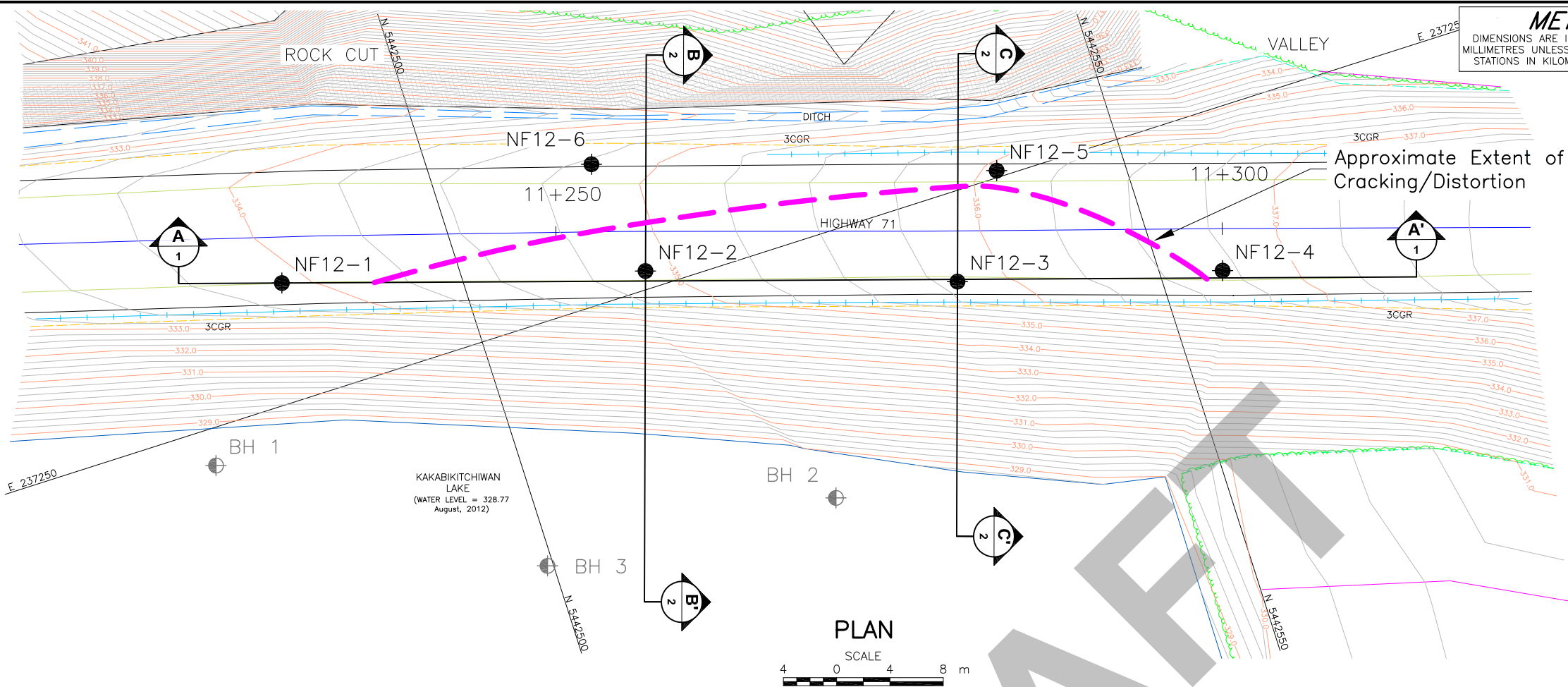
Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Combination of: <ul style="list-style-type: none"> <li>■ 1 m grade lowering</li> <li>■ 3 m highway shift to west</li> <li>■ Re-grade east slope to 1.25H:1V (to Elev. 328.9 m)</li> </ul>	1	<ul style="list-style-type: none"> <li>■ Minimal excavation required.</li> <li>■ Utility relocation on top of rock cut not required.</li> <li>■ Filling in lake not required.</li> <li>■ Standard construction operations – readily constructible.</li> </ul>	<ul style="list-style-type: none"> <li>■ Re-grading of rock fill slope may be difficult due to large sizes of existing rock fill. Importing a limited amount of smaller size rock fill may be required to fill in “gaps” or to “key in” to the rest of the embankment.</li> <li>■ The use of controlled blasting techniques and rock face stabilization measures may be required to maintain an adequate ditch along the west slope after shifting the highway.</li> </ul>	<ul style="list-style-type: none"> <li>■ Likely lowest cost option as no specialized equipment or materials are required.</li> <li>■ Potential cost for localized rock blasting.</li> </ul>	<ul style="list-style-type: none"> <li>■ Potential impact to traffic during construction.</li> <li>■ Lowers risk of further instability by offloading clay subsoils under and at toe of east slope.</li> </ul>
Combination of: <ul style="list-style-type: none"> <li>■ 1 m grade lowering</li> <li>■ 1 m EPS (with 1 m cover) or 1.5 m of cellular concrete (with 0.5 m cover)</li> <li>■ 7 m wide toe berm</li> </ul>	2	<ul style="list-style-type: none"> <li>■ Shifting of highway not required.</li> </ul>	<ul style="list-style-type: none"> <li>■ Toe berm required for stability to satisfy long-term drained conditions. Requires fill to be placed in the water in a culturally and environmentally sensitive area; in-water time restrictions may also apply.</li> <li>■ Toe berm will apply additional loading to the clay subsoils at the toe of slope which will result in settlement, particularly under the NBL and could result in need for future maintenance.</li> <li>■ Requires 0.5 m (cellular concrete) to 1 m (EPS) of conventional soil cover and concrete slab (EPS) to mitigate potential for differential icing.</li> </ul>	<ul style="list-style-type: none"> <li>■ Cost of EPS or cellular concrete typically about \$240/m<sup>3</sup>; estimated quantity between about 560 m<sup>3</sup> and 1,000 m<sup>3</sup> for a total cost between about \$160,000 and \$240,000.</li> <li>■ Cost of rock fill placement for toe berm about \$10/m<sup>3</sup> or about \$27,000.</li> </ul>	<ul style="list-style-type: none"> <li>■ Will be difficult to maintain traffic flow during construction.</li> <li>■ Future settlement of the toe berms and lateral spreading of the clay stratum may lead to re-initiation of distress in the existing embankment.</li> </ul>
Stabilizing Toe Berms (11 m wide x 3 m high x 110 m long)	3	<ul style="list-style-type: none"> <li>■ Minimal impact on traffic flow during construction.</li> <li>■ Shifting of highway not required.</li> </ul>	<ul style="list-style-type: none"> <li>■ Toe berm required for stability to satisfy long-term drained conditions. Requires fill to be placed in the water in a culturally and environmentally sensitive area; in-water time restrictions may also apply.</li> <li>■ Toe berm will apply additional loading to the clay subsoils at the toe of slope which will result in settlement and/or lateral spreading, particularly under the NBL and could result in future</li> </ul>	<ul style="list-style-type: none"> <li>■ Cost of rock fill placement for toe berm about \$10/m<sup>3</sup> for about 4200 m<sup>3</sup> (11 m wide x 3.5 m high x 110 m long) of material or about \$43,000.</li> <li>■ Cost of carrying out in-water work.</li> </ul>	<ul style="list-style-type: none"> <li>■ Future settlement of the toe berms and lateral spreading of the clay stratum may lead to re-initiation of distress in the existing embankment.</li> </ul>



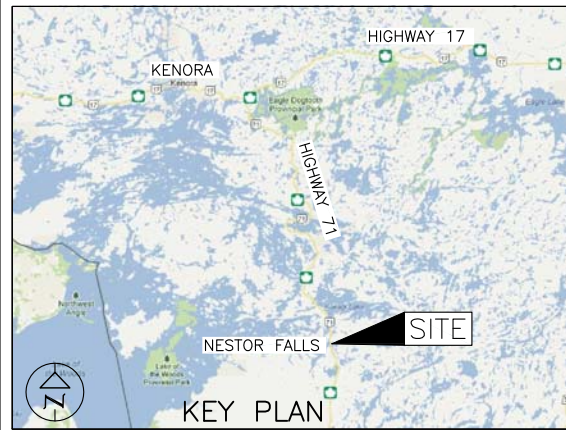
**FOUNDATION INVESTIGATION AND DESIGN REPORT  
HIGHWAY 71 EMBANKMENT AT NESTOR FALLS, WO 2013-11005**

**Table 1: Evaluation of Embankment Stabilization Remediation Measures**

Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
			<p>maintenance.</p> <ul style="list-style-type: none"> <li>Large quantity of fill required for a 110 m long berm for the required width/height of berm.</li> <li>May need to obtain additional right-of-way.</li> </ul>		
Dry Soil Mixing or Jet Grouting	4	<ul style="list-style-type: none"> <li>Can use cement rather than more expensive lime to enhance the shear strength of the clay stratum, with which improves stability and reduces lateral spreading of clay stratum.</li> <li>Material mixed in situ with minimal cuttings/spoil – no off-site disposal.</li> <li>Minimal impact on traffic.</li> <li>Shifting of highway not required.</li> <li>Jet grouting may be easier to construct compared to DSM.</li> </ul>	<ul style="list-style-type: none"> <li>Requires fill to be placed and mixed from the water surface in potentially a culturally and environmentally sensitive area; in-water time restrictions may also apply.</li> <li>Subsurface information and laboratory testing will be required to permit optimum mix design.</li> <li>Specialized construction techniques; would likely require trial section on site.</li> <li>Would require penetrating through rock fill embankment to be able to improve clay stability under the embankment slope.</li> </ul>	<ul style="list-style-type: none"> <li>Specialized equipment required, possibly from the U.S.A.</li> <li>Cost of additional foundation investigation.</li> <li>Cost of lime is likely much greater than that of cement.</li> <li>Approximate cost on the order of about \$160,000 (includes \$75,000 mobilization) based on \$180/linear m of columns for 30 columns at 15 m/column.</li> </ul>	<ul style="list-style-type: none"> <li>Lowers risk of further instability especially if combined with other options.</li> </ul>
Re-align Highway to the West (fully within the rock cut)	5	<ul style="list-style-type: none"> <li>Road moved away from area of instability.</li> <li>East toe will be constructed fully within a rock cut and/or over bedrock.</li> <li>No in-water work required.</li> <li>Traffic could be maintained on existing embankment.</li> </ul>	<ul style="list-style-type: none"> <li>Given the short distance to the Nestor Falls dam to the south, there may be insufficient distance to permit sufficient re-alignment and transitioning of surface grade and curve.</li> <li>Utility relocation likely required.</li> <li>Property acquisition may be required.</li> <li>Bedrock blasting required.</li> </ul>	<ul style="list-style-type: none"> <li>Most expensive compared to other options due to property and utility issues and bedrock excavation/new roadway construction.</li> </ul>	<ul style="list-style-type: none"> <li>Eliminates potential for further instability.</li> <li>May not be feasible to re-align far enough to the west due to highway grading issues.</li> </ul>

CONT No.  
WO No. 2013-11005EMBANKMENT STABILIZATION  
HIGHWAY 71 (NESTOR FALLS)  
BOREHOLE LOCATIONS  
AND SOIL STRATA

SHEET

Golder Associates Ltd.  
SUDBURY, ONTARIO, CANADA

## LEGEND

- Borehole - Current Investigation
- Approximate Borehole - 1999 (DST)
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- REC Recovery (%)
- WL upon completion of drilling
- R Refusal

## BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
NF12-1	333.9	5442487.0	237241.2
NF12-2	334.8	5442513.3	237248.7
NF12-3	335.8	5442535.4	237256.5
NF12-4	336.6	5442554.6	237261.8
NF12-5	336.1	5442540.7	237249.5
NF12-6	334.8	5442511.9	237239.8
BH 1	328.9	Approximate Borehole locations estimated from DST Report.	
BH 2	328.9		
BH 3	328.9		

## NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.




## REFERENCE

Base plans provided in digital format by TULLOCH Engineering, drawing file nos. 12-4550-Nestor Falls.dwg, received Aug 22, 2012. Previous boreholes from "Foundation Design Report, 200 m North of Nestor Falls Bridge W.P. 306-85-00", by DST Consulting Engineers, March 2000, GEOCRE 52F-26.

NO.	DATE	BY	REVISION
Geocres No. 52F-40			
HWY. 71	PROJECT NO. 12-1191-0005		DIST.
SUBM'D.	CHKD.	DATE: MAY 2013	SITE:
DRAWN: TB	CHKD. SEMC	APPD. JMAC	DWG. 1





LEGEND	
	Borehole – Current Investigation
	Approximate Borehole – 1999 (DST)
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
REC	Recovery (%)
	WL upon completion of drilling
R	Refusal

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
NF12-2	334.8	5442513.3	237248.7
NF12-3	335.8	5442535.4	237256.5
NF12-5	336.1	5442540.7	237249.5
NF12-6	334.8	5442511.9	237239.8
BH 2	328.9	Approximate Borehole locations estimated from DST Report.	
BH 3	328.9		

---

NOTES

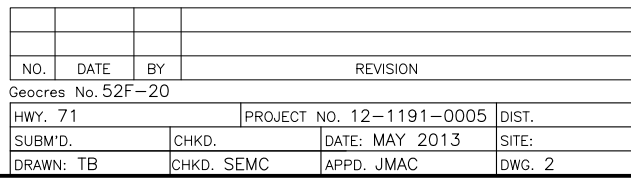
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

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## REFERENCE

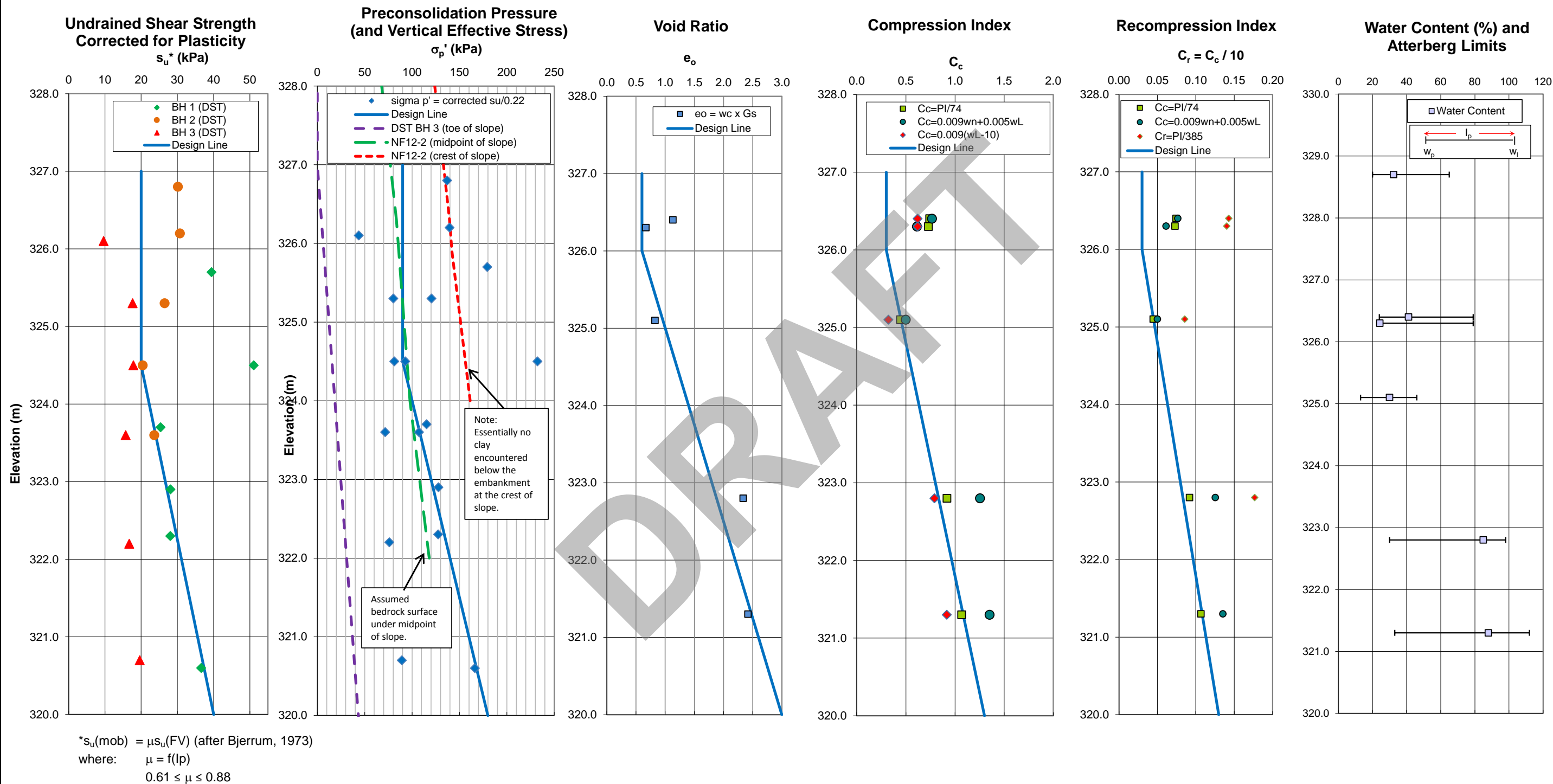
Base plans provided in digital format by TULLOCH Engineering, drawing file nos. 12-4550-Nestor Falls.dwg, received Aug 22, 2012. Previous boreholes from "Foundation Design Report, 200 m North of Nestor Falls Bridge W.P. 306-85-00", by DST Consulting Engineers, March 2000, GEOCRES 52F-26.



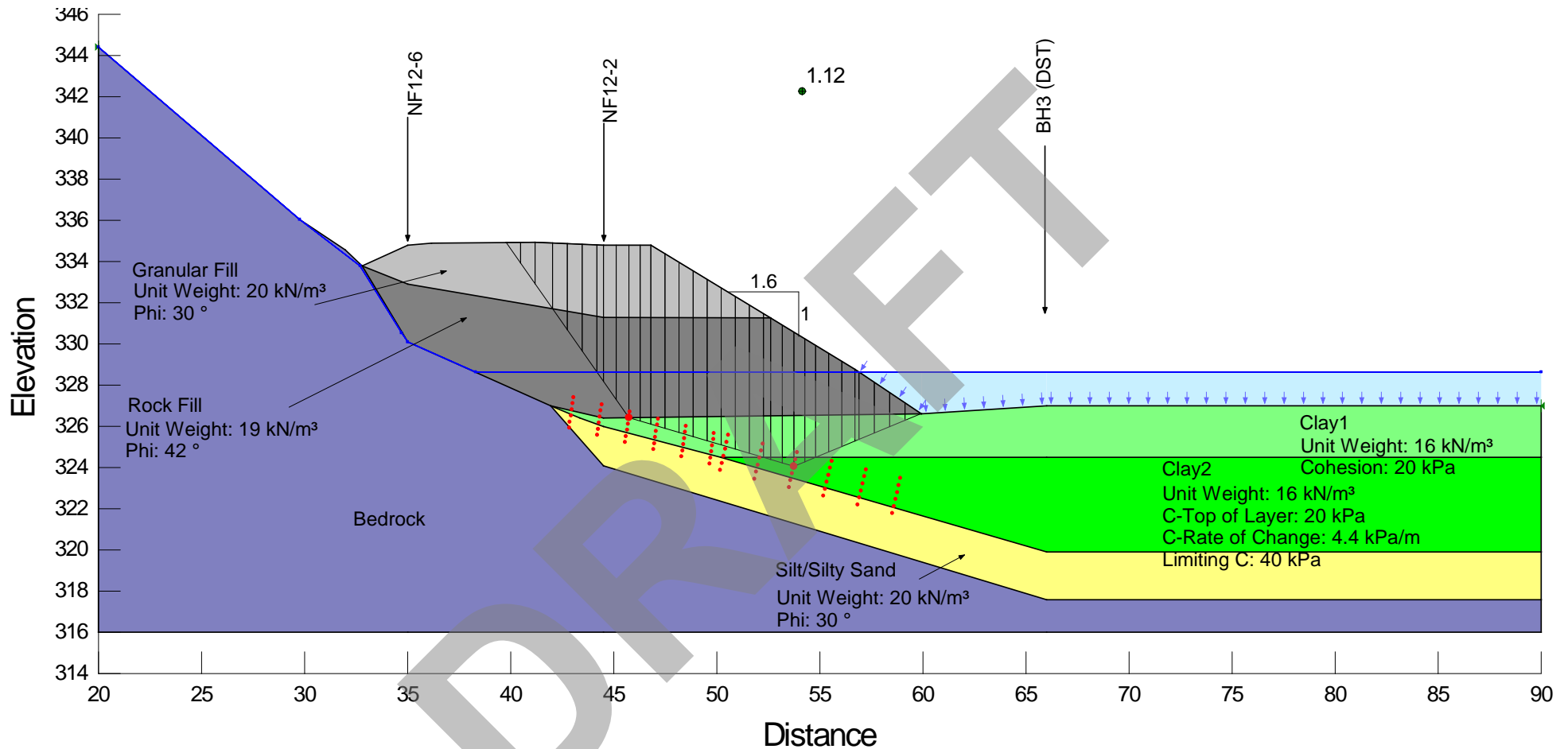
https://capws.golder.com/sites/P211910005/mtoNwRetainerThunderBay/2100 Nestor Falls/5Reporting/Final Report/Figures (pdf only)/12-1191-0005-2100 Nestor Falls Parameter Summary.xlsx|Report Plots

SUMMARY PLOT OF ENGINEERING PARAMETERS  
FOR CLAY DEPOSIT  
NESTOR FALLS

FIGURE 1



FoS = 1.12



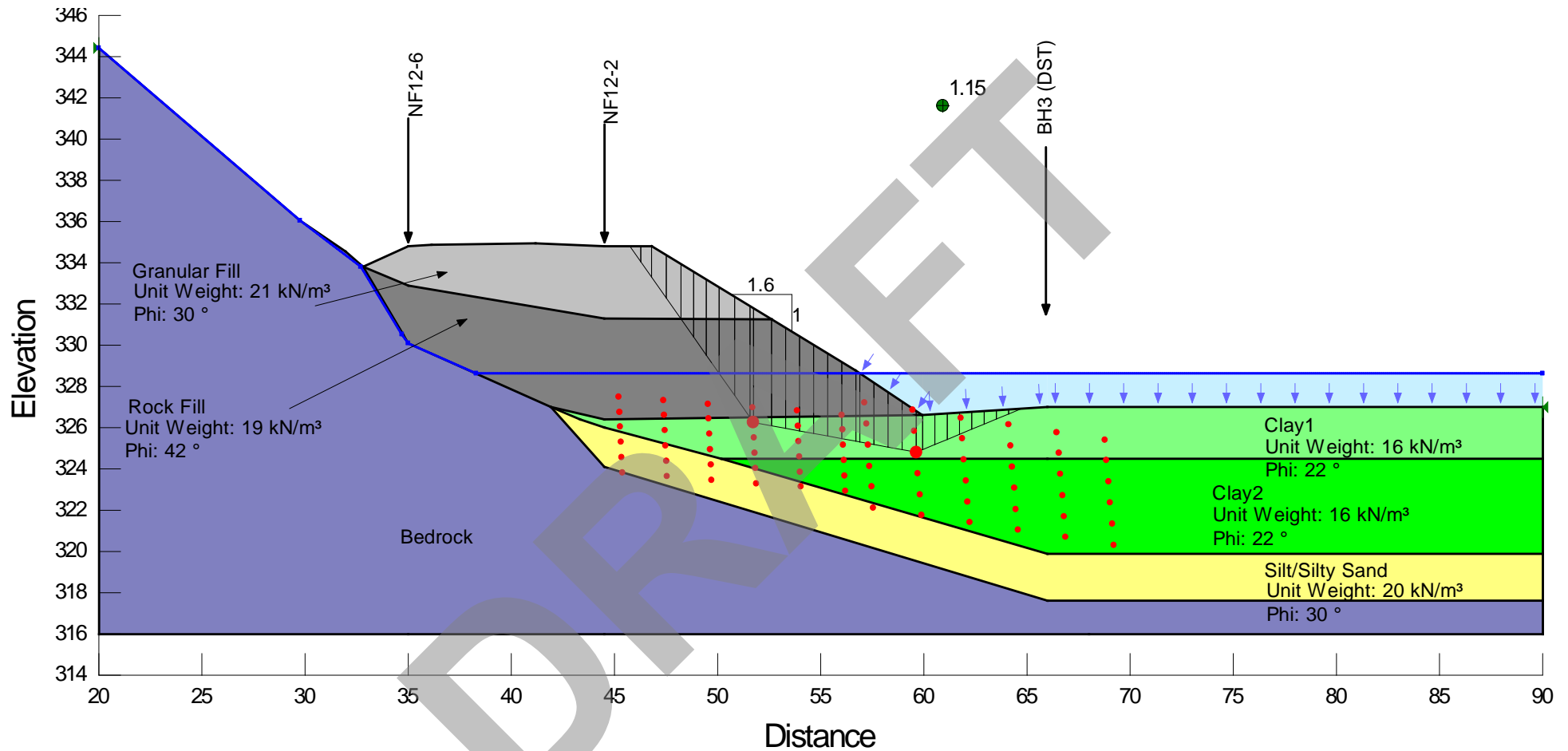
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TITLE		STABILITY ANALYSIS STA 11+257 – UNDRAINED – LWL Current/Existing Embankment			
		PROJECT No. 12-1191-0005-2100	FILE No.	----	
DESIGN	SEMC	MAY 2013	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	JPD	MAY 2013			
REVIEW	JMAC	MAY 2013			



Figure 2A



FoS = 1.15

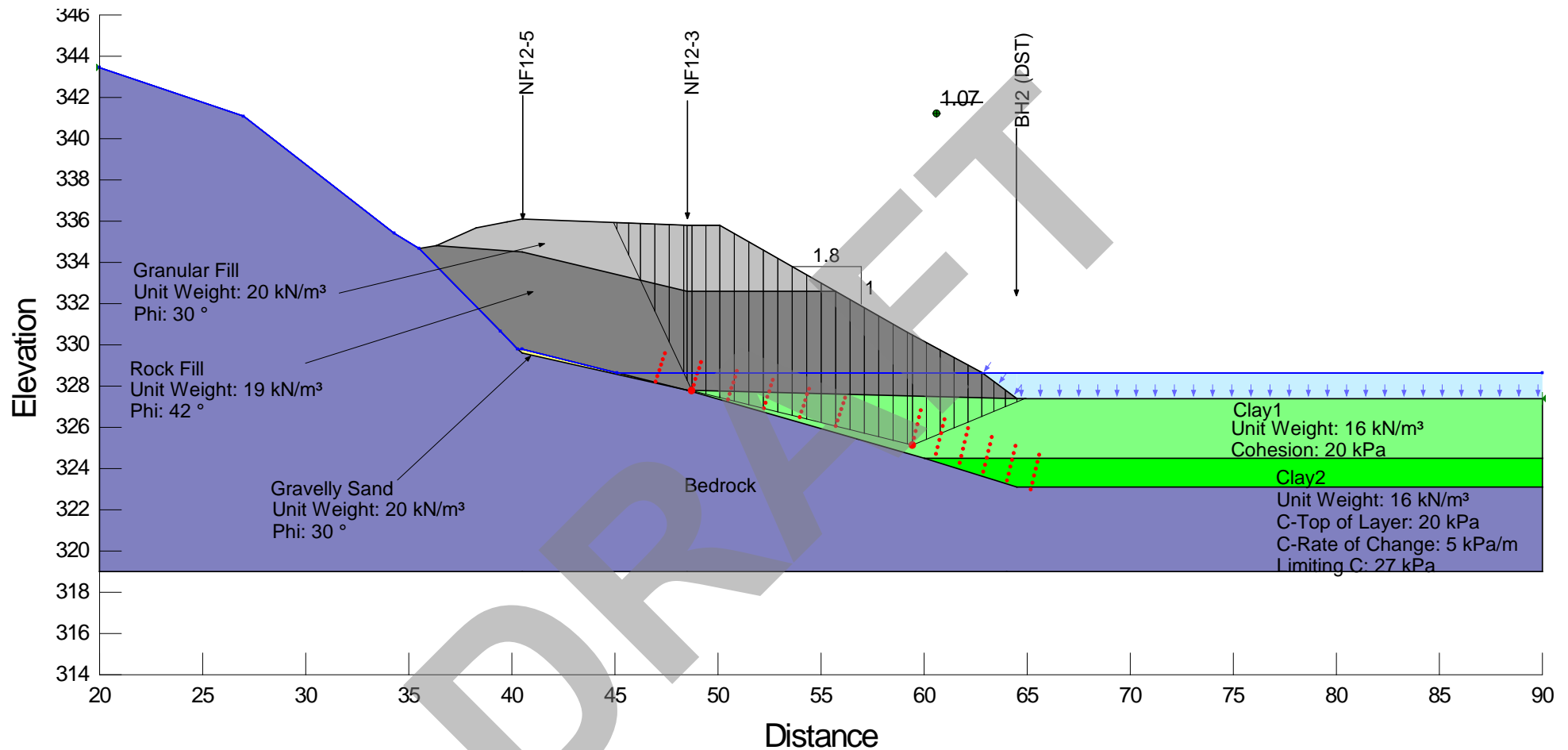


PROJECT		NESTOR FALLS EMBANKMENT HIGHWAY 71			
TITLE		STABILITY ANALYSIS STA 11+257 – DRAINED – LWL Current/Existing Embankment			
		PROJECT No. 12-1191-0005-2100	FILE No. ----		
DESIGN	SEMC	MAY 2013	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	JPD	MAY 2013			
REVIEW	JMAC	MAY 2013			



Figure 2B

FoS = 1.07

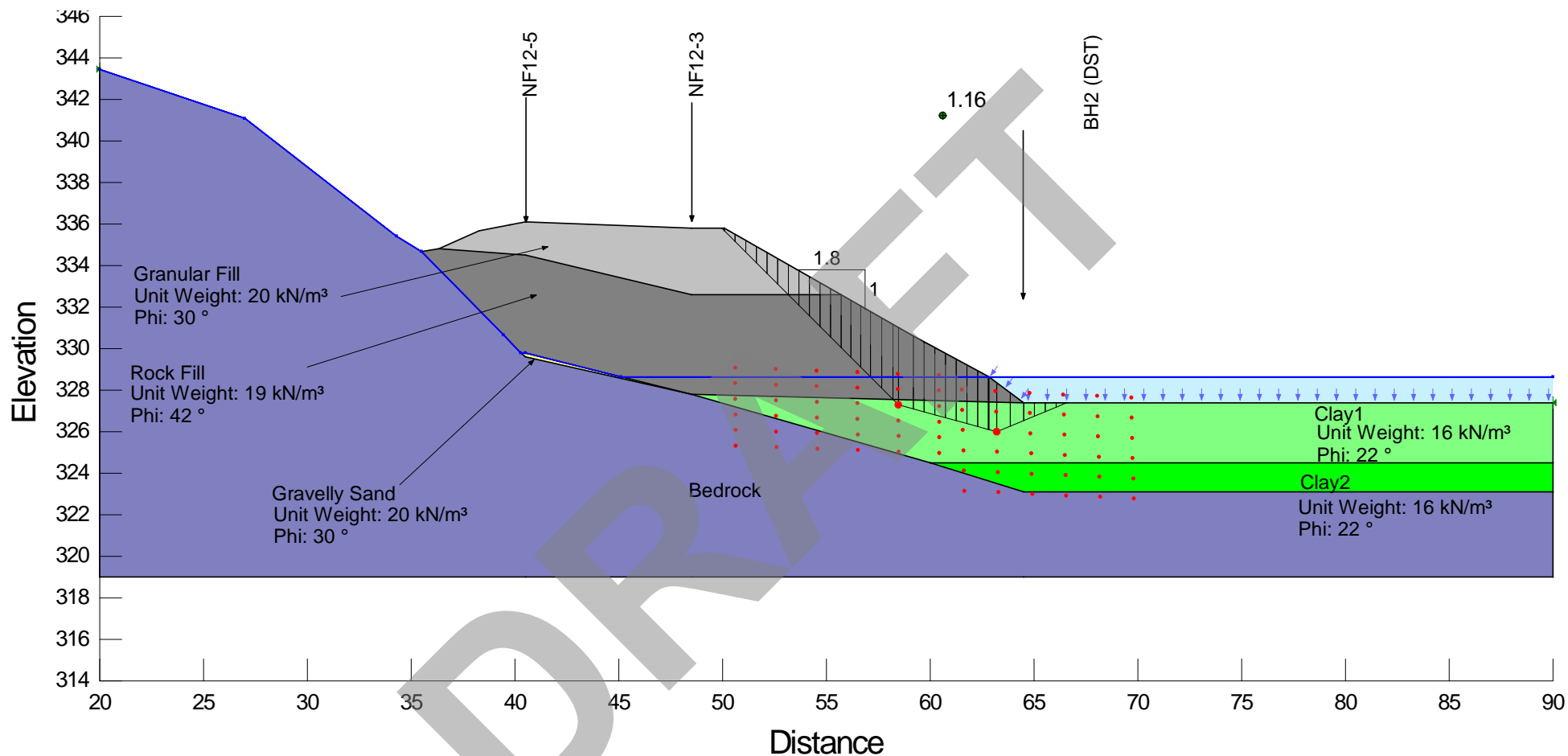


PROJECT		NESTOR FALLS EMBANKMENT HIGHWAY 71			
TITLE		STABILITY ANALYSIS STA 11+280 – UNDRAINED – LWL Current/Existing Embankment			
		PROJECT No. 12-1191-0005-2100	FILE No. ----		
DESIGN	SEMC	MAY 2013	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	JPD	MAY 2013			
REVIEW	JMAC	MAY 2013			



Figure 3A

FoS = 1.16

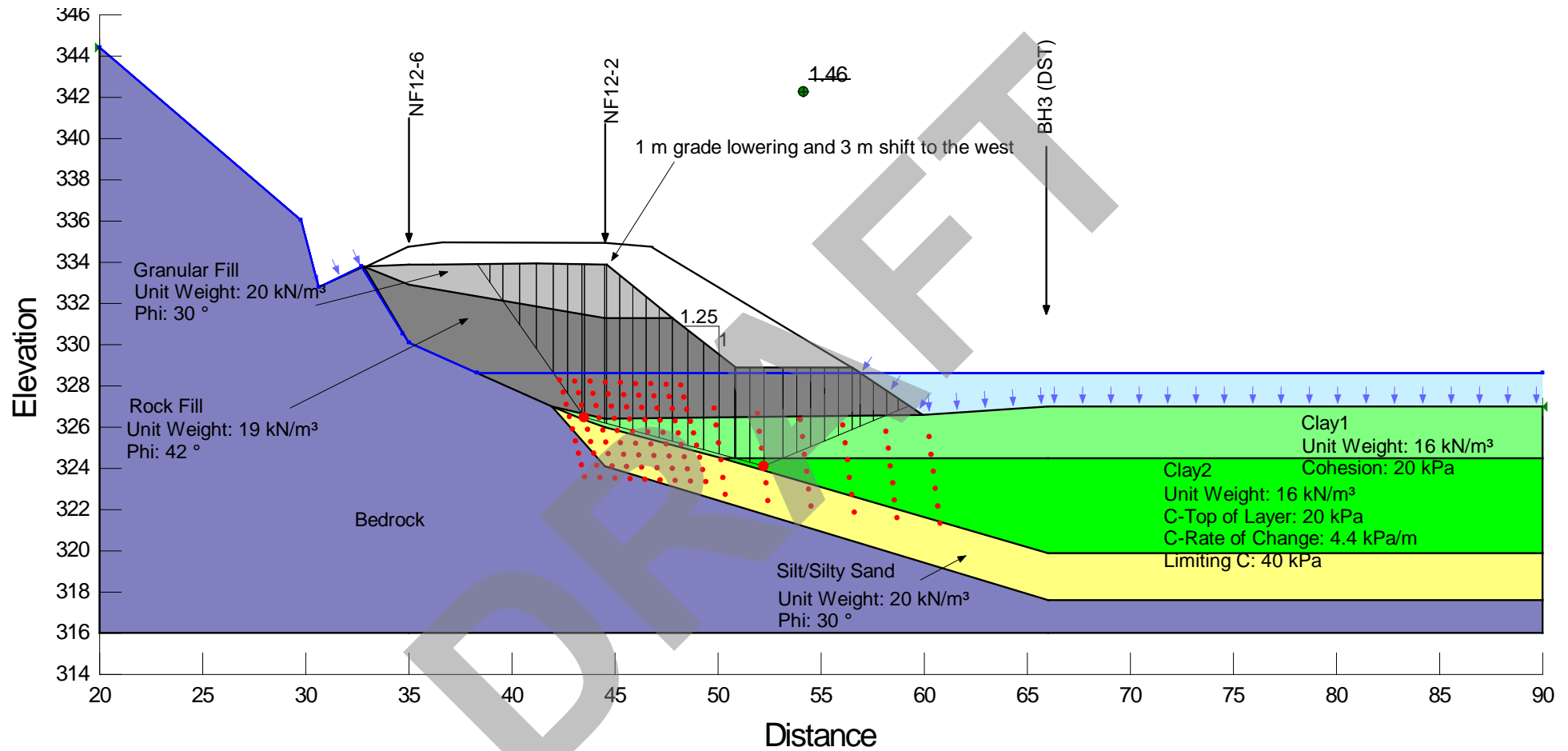


PROJECT				
NESTOR FALLS EMBANKMENT HIGHWAY 71				
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STABILITY ANALYSIS STA 11+280 – DRAINED – LWL Current/Existing Embankment				
PROJECT No. 12-1191-0005-2100		FILE No. ----		
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CADD	--			
CHECK	JPD	MAY 2013		
REVIEW	JMAC	MAY 2013		



Figure 3B

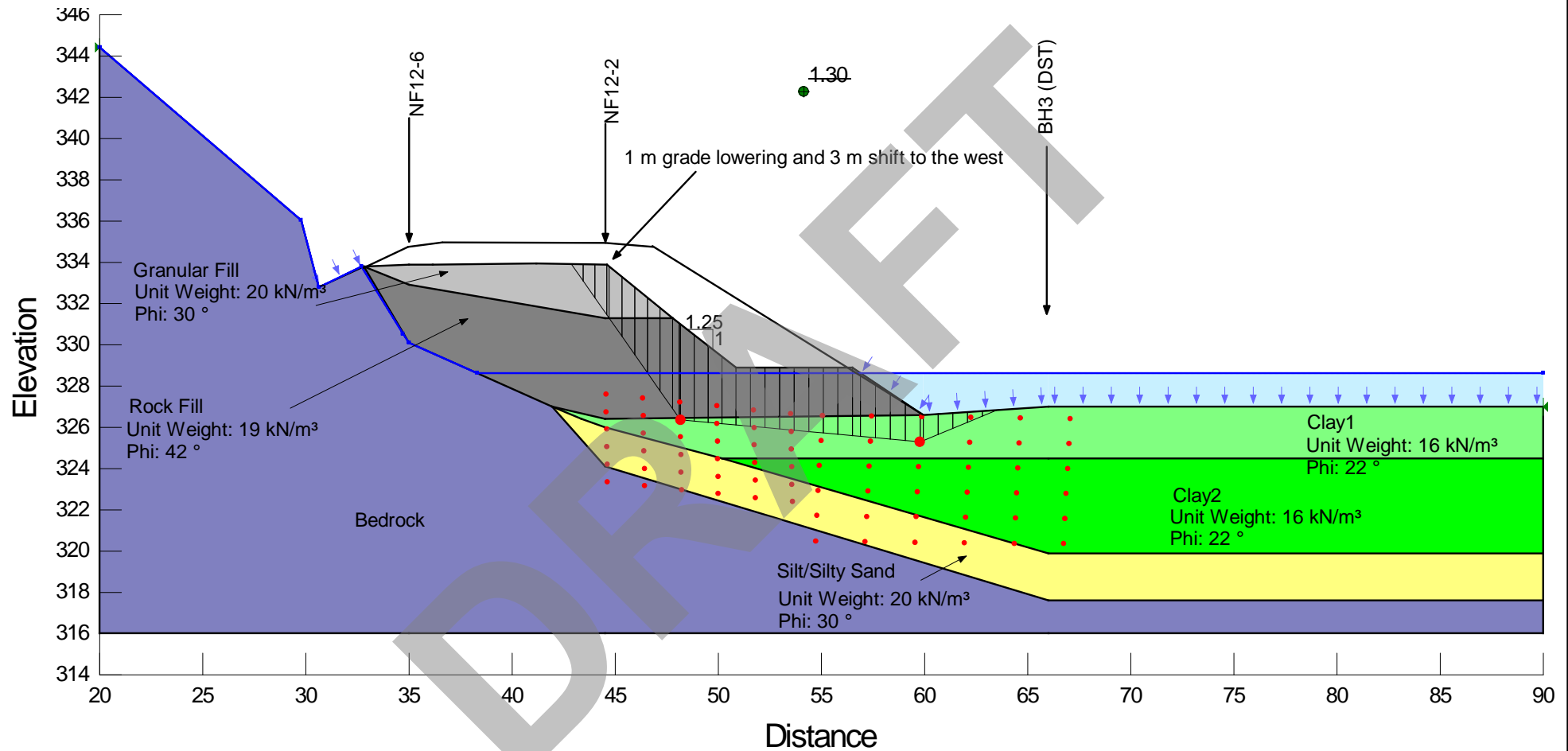
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PROJECT				
NESTOR FALLS EMBANKMENT HIGHWAY 71				
TITLE				
Recommended Option: STABILITY ANALYSIS STA 11+257 – UNDRAINED – LWL 1 m Grade Lowering, 3 m Shift, 1.25H:1V Slope				
PROJECT No. 12-1191-0005-2100		FILE No. ----		
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CADD	--		REV.	
CHECK	JPD	MAY 2013	Figure 4A	
REVIEW	JMAC	MAY 2013		

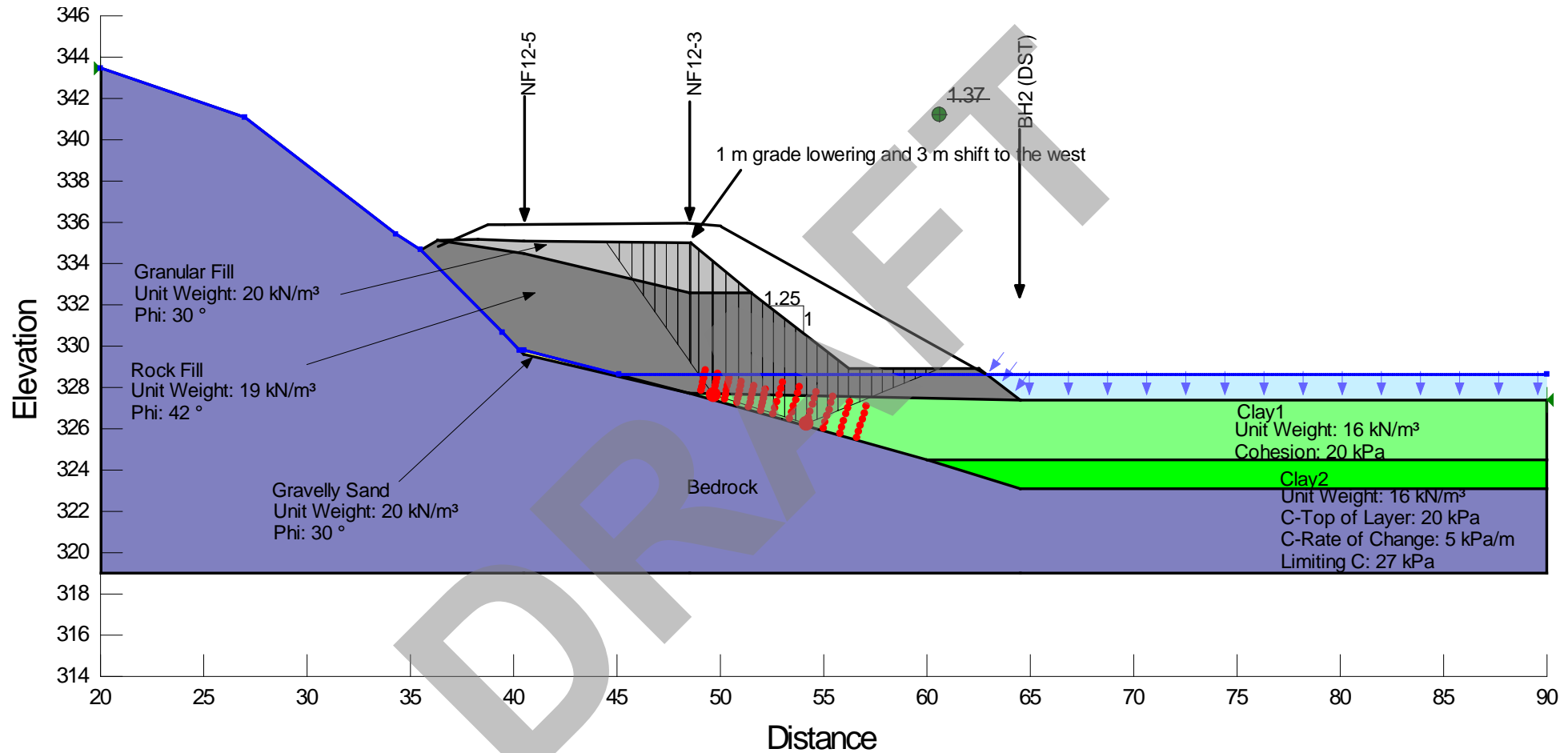


FoS = 1.30



PROJECT		NESTOR FALLS EMBANKMENT HIGHWAY 71			
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PROJECT No. 12-1191-0005-2100		FILE No. ----			
DESIGN	SEMC	MAY 2013	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	JPD	MAY 2013			
REVIEW	JMAC	MAY 2013			
Golder Associates		Figure 4B			

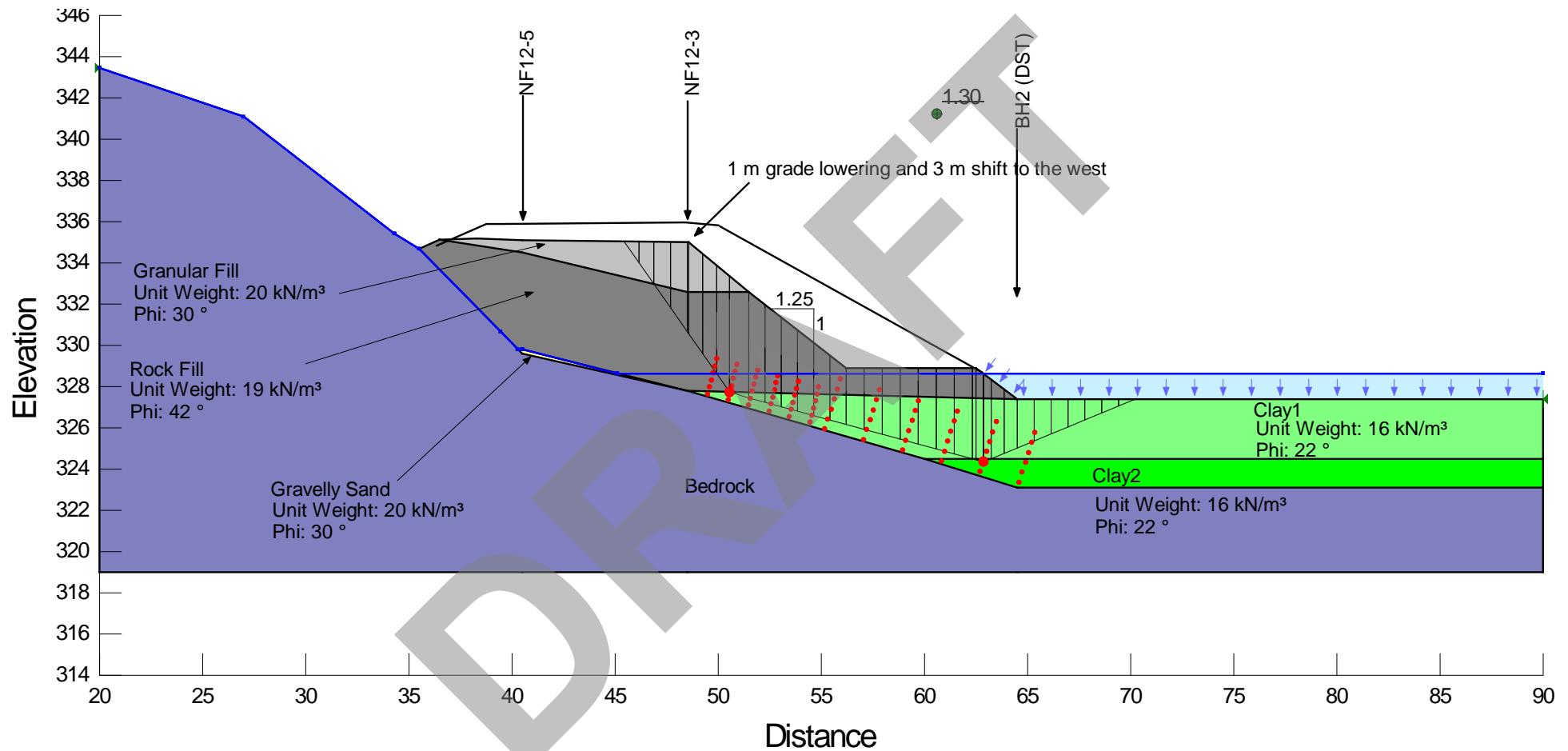
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PROJECT				
NESTOR FALLS EMBANKMENT HIGHWAY 71				
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Recommended Option: STABILITY ANALYSIS STA 11+280 – UNDRAINED – LWL 1 m Grade Lowering, 3 m Shift, 1.25H:1V Slope				
PROJECT No. 12-1191-0005-2100		FILE No. ----		
DESIGN	SEMC	MAY 2013	SCALE	AS SHOWN
CADD	--		REV.	
CHECK	JPD	MAY 2013	Figure 5A	
REVIEW	JMAC	MAY 2013		



FoS = 1.30



PROJECT				
NESTOR FALLS EMBANKMENT HIGHWAY 71				
TITLE				
Recommended Option: STABILITY ANALYSIS STA 11+280 – DRAINED – LWL 1 m Grade Lowering, 3 m Shift, 1.25H:1V Slope				
PROJECT No. 12-1191-0005-2100		FILE No. ----		
DESIGN	SEMC	MAY 2013	SCALE	AS SHOWN
CADD	--		REV.	
CHECK	JPD	MAY 2013	Figure 5B	
REVIEW	JMAC	MAY 2013		







# APPENDIX A

## Photographic Record

DRAFT



## APPENDIX A



Photo 1: View of site looking west (August 2012).

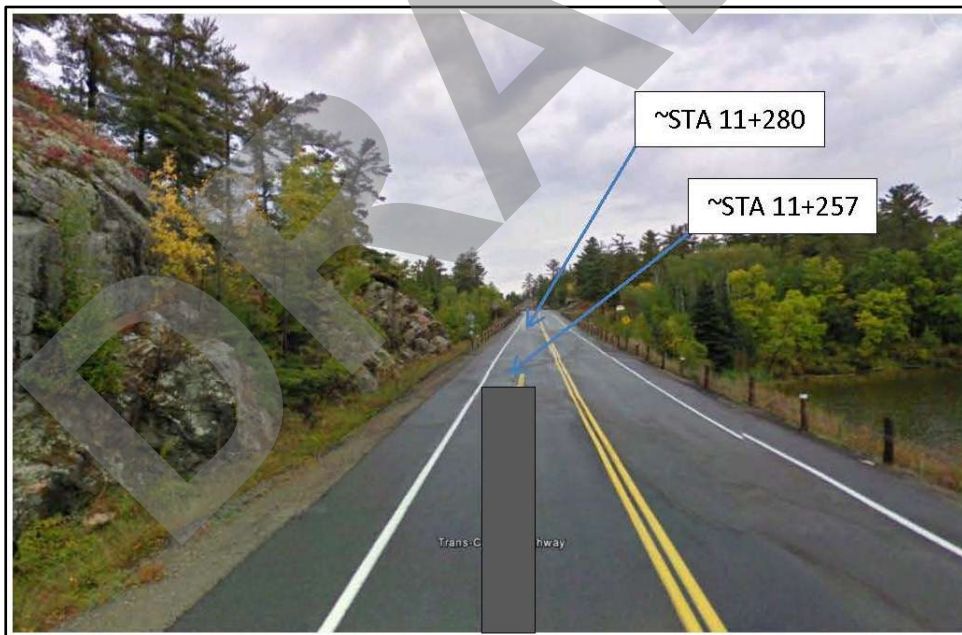


Photo 2: Embankment distress section looking north (courtesy of Google - 2009).



## APPENDIX A

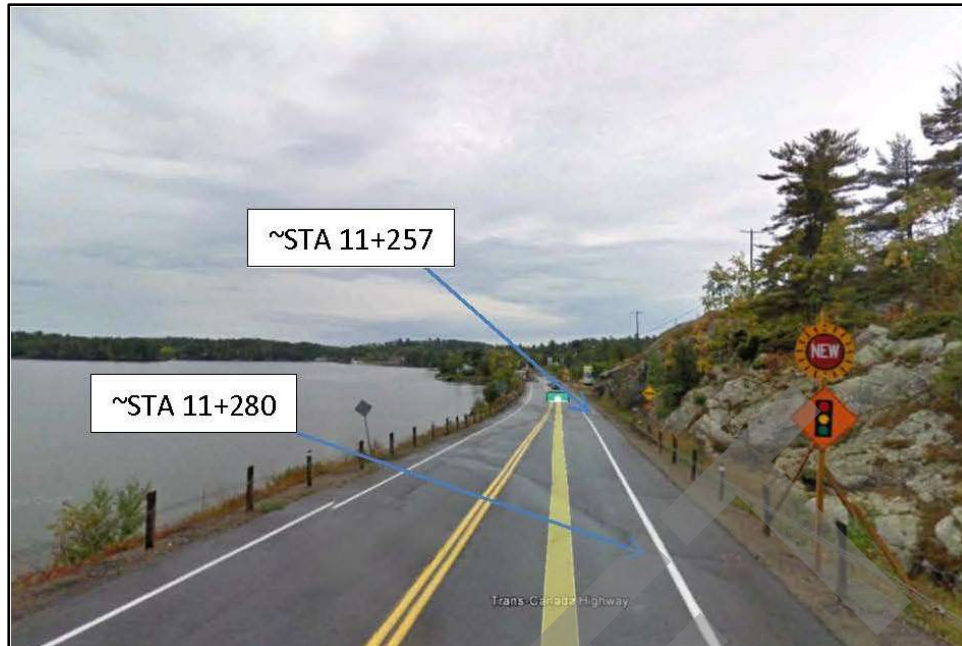


Photo 3: Embankment distress section looking south (courtesy of Google – 2009)



Photo 4: Cracking April 2012 (provided by MTO).



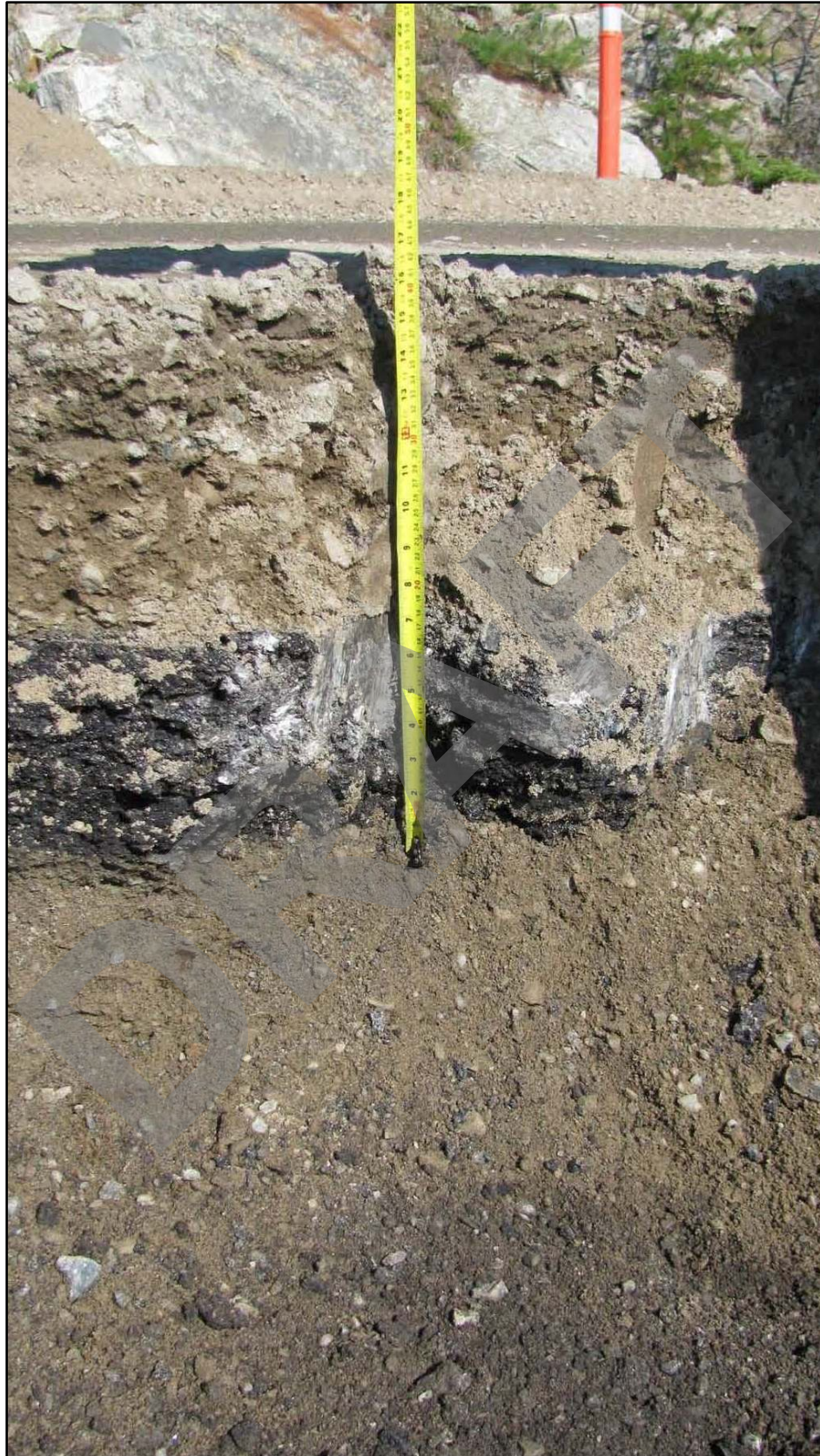


Photo 5: Pavement repair – note eight inches (200 mm) of fill over previous asphalt pavement (June 2012 by MTO).





Photo 6: East slope looking north (August 2012).



Photo 7: Erosion of East slope by surface water runoff (August 2012).





Photo 8: Large rock fill along toe of east slope (August 2012).



Photo 9: Valley adjacent to west toe of embankment looking north.





## APPENDIX A

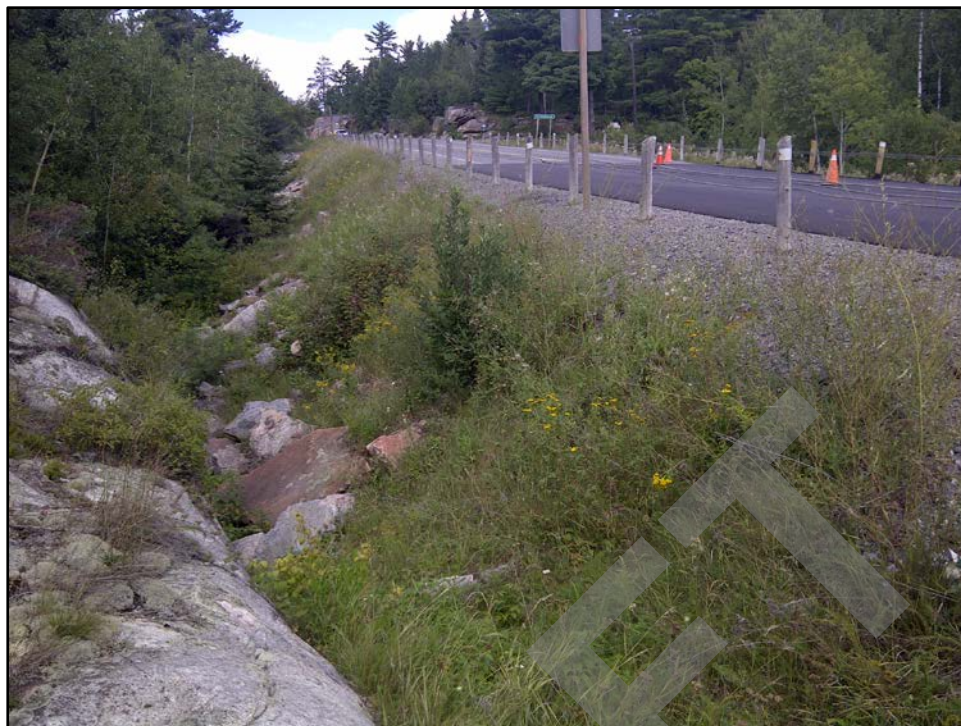


Photo 10: West slope of valley looking north.



Photo 11: Valley looking south.





# APPENDIX B

## Record of Boreholes and Drillholes

DRAFT



## LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

### I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

#### Dynamic Cone Penetration Resistance; $N_d$ :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

<b>PH:</b>	Sampler advanced by hydraulic pressure
<b>PM:</b>	Sampler advanced by manual pressure
<b>WH:</b>	Sampler advanced by static weight of hammer
<b>WR:</b>	Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ( $Q_t$ ), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

Density Index	N
Relative Density	Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

#### (b) Cohesive Soils Consistency

	$C_u, S_u$	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_r$	relative density (specific gravity, $G_s$ )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

**Note:** 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

### V. MINOR SOIL CONSTITUENTS

Percent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (cohesionless) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



## LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

### I. GENERAL

$\pi$	3.1416
$\ln x$ ,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

#### (a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) (formerly $G_s$ )
e	void ratio
n	porosity
S	degree of saturation

#### (a) Index Properties (continued)

w	water content
$w_l$ or LL	liquid limit
$w_p$ or PL	plastic limit
$I_p$ or PI	plasticity index $= (w_l - w_p)$
$w_s$	shrinkage limit
$I_L$	liquidity index $= (w - w_p) / I_p$
$I_C$	consistency index $= (w_l - w) / I_p$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$I_D$	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

#### (b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

#### (c) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (over-consolidated range)
$C_s$	swelling index
$C_\alpha$	secondary compression index
$m_v$	coefficient of volume change
$C_v$	coefficient of consolidation (vertical direction)
$C_h$	coefficient of consolidation (horizontal direction)
$T_v$	time factor (vertical direction)
U	degree of consolidation
$\sigma'_p$	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	coefficient of friction $= \tan \delta$
$c'$	effective cohesion
$c_u, s_u$	undrained shear strength ( $\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
$p'$	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
$q_u$	compressive strength $(\sigma_1 - \sigma_3)$
$S_t$	sensitivity

\* Density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1  
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



# LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

## WEATHERINGS STATE

**Fresh:** no visible sign of weathering

**Faintly weathered:** weathering limited to the surface of major discontinuities.

**Slightly weathered:** penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

**Moderately weathered:** weathering extends throughout the rock mass but the rock material is not friable.

**Highly weathered:** weathering extends throughout rock mass and the rock material is partly friable.

**Completely weathered:** rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

## BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

## JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

## GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: \* Grains greater than 60 microns diameter are visible to the naked eye.

## CORE CONDITION

### Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

### Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

### Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

## DISCONTINUITY DATA

### Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

### Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

### Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

### Abbreviations

JN Joint	PL Planar
FLT Fault	CU Curved
SH Shear	UN Undulating
VN Vein	IR Irregular
FR Fracture	K Slickensided
SY Stylolite	PO Polished
BD Bedding	SM Smooth
FO Foliation	SR Slightly Rough
CO Contact	RO Rough
AXJ Axial Joint	VR Very Rough
KV Karstic Void	
MB Mechanical Break	

PROJECT		12-1191-0005		<b>RECORD OF BOREHOLE No NF12-1</b>				1 OF 1 <b>METRIC</b>					
W.P.				LOCATION		N 5442487.0; E 237241.2		ORIGINATED BY		JB			
DIST		HWY 71		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers, HW Casing, HQ Coring		COMPILED BY		EC			
DATUM		GEODETIC		DATE		August 7 and 8, 2012		CHECKED BY		SEMC			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT		UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>p</sub> W W <sub>L</sub>			
333.9	GROUND SURFACE							20 40 60 80 100					
0.1	ASPHALT (100 mm)							20 40 60 80 100					
	Gravelly sand to sand and gravel, trace to some silt (FILL) Very loose to compact Brown with oxidation staining Moist  Geogrid encountered at 0.4 m depth.		1	SS	16		333						
			2	SS	14		332						29 67 (4)
			3	SS	6		331						
			4	SS	2		330						
	Auger refusal at 4.0 m depth. Spoon bouncing at 4.1 m depth.		5	SS	-		329						32 60 (8)
329.8	ROCK FILL		1	RC	REC 92%		329						
4.1			2	RC	REC 60%		328						
	No recovery in Sample 6.		6	SS	17		327						
			3	RC	REC 80%		326						
			4	RC	REC 65%		325						
326.5	GRANITE (BEDROCK)		5	RC	REC 100%		326						RQD = 68%
7.4	Bedrock cored from 7.4 m to 9.8 m depth. For coring details see Record of Drillhole NF12-1.		6	RC	REC 72%		325						RQD = 65%
324.1	END OF BOREHOLE												
9.8	Note:  1. Water level not recorded due to coring operations. Borehole dry prior to start of coring.												

SUD-MTO 001 12-1191-0005 NF.GPJ CAL-MISS.GDT 07/11/12 DATA INPUT:

PROJECT: 12-1191-0005

**RECORD OF DRILLHOLE: NF12-1**

SHEET 1 OF 1

LOCATION: NESTOR FALLS

DRILLING DATE: August 7 and 8, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Acker MP8

DRILLING CONTRACTOR: PADDOCK DRILLING LTD.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock  NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
				DEPTH (m)	FLUSH									RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)	RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
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DEPTH SCALE

1 : 50



LOGGED: JB

CHECKED: SEMC

MTO-RCK 001 12-1191-0005 NF GPJ GAL-MISS.GDT 07/11/12 DATA INPUT:

PROJECT		12-1191-0005		<b>RECORD OF BOREHOLE No NF12-2</b>		1 OF 1 <b>METRIC</b>							
W.P.				LOCATION		N 5442513.3; E 237248.7							
DIST		HWY 71		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers, HW Casing, HQ Coring							
DATUM		GEODETIC		DATE		August 9, 2012							
						ORIGINATED BY JB							
						COMPILED BY EC							
						CHECKED BY SEMC							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>p</sub>	W		
334.8	GROUND SURFACE							20 40 60 80 100					
0.0	ASPHALT (50 mm)							20 40 60 80 100					
	Gravelly sand to sand and gravel, trace to some silt (FILL) Very loose to compact Brown Moist		1	SS	15								
	Geogrid encountered at 0.15 m depth.												
			2	SS	6								26 66 (8)
			3	SS	2								
	Augers grinding below 2.7 m depth. HQ Coring below 3.0 m depth.		4	SS	6								56 40 (4)
331.3	ROCK FILL												
3.5	Possible void between 4.1 m and 4.9 m depth.		1	RC	REC 16%								
			2	RC	REC 90%								
	Void from 7.7 m to 8.0 m depth.		3	RC	REC 76%								
326.3	CLAY, trace sand												
325.9	Wet		4	RC	REC 100%								
8.9	SILT, some sand, trace gravel, trace clay		5	SS	21								9 18 64 9
	Compact Grey Wet												
324.1	BEDROCK												
10.8	END OF BOREHOLE CASING REFUSAL												
	Note: 1. Water level at a depth of 6.2 m below ground surface (Elev. 328.6 m) upon completion of drilling.												

SUD-MTO 001 12-1191-0005 NF.GPJ CAL-MISS.GDT 07/02/13 DATA INPUT:



PROJECT 12-1191-0005		RECORD OF BOREHOLE No NF12-3				1 OF 1 METRIC							
W.P. _____		LOCATION N 5442535.4; E 237256.5		ORIGINATED BY JB									
DIST _____ HWY 71		BOREHOLE TYPE HQ Casing, HQ Coring		COMPILED BY EC									
DATUM GEODETIC		DATE August 8, 2012		CHECKED BY SEMC									
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
335.8	GROUND SURFACE						20 40 60 80 100	20 40 60 80 100	25 50 75				
0.1	ASPHALT (50 mm)												
	Gravelly sand to sand and gravel, trace to some silt (FILL) Loose to compact Brown Moist		1	SS	14								25 66 (9)
	Geogrid encountered at 0.2 m depth.		2	SS	14								
			3	SS	9								
332.6	Spoon bouncing at 3.2 m depth.		4	SS	-								
3.2	ROCK FILL		1	RC	REC 73%								
			2	RC	REC 47%								
			3	RC	REC 80%								
			4	RC	REC 87%								
327.8	GRANITE GNEISS (BEDROCK)		5	RC	REC 100%								RQD = 96%
8.0	Bedrock cored from 8.0 m to 11.2 m depth. For coring details see Record of Drillhole NF12-3.		6	RC	REC 77%								RQD = 85%
324.6	END OF BOREHOLE												
11.2	Note: 1. Water level at a depth of 7.9 m below ground surface (Elev. 327.9 m) upon completion of drilling.												

PROJECT: 12-1191-0005

**RECORD OF DRILLHOLE: NF12-3**

SHEET 1 OF 1

LOCATION: NESTOR FALLS

DRILLING DATE: August 8, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Acker MP8

DRILLING CONTRACTOR: PADDOCK DRILLING LTD.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	COLOUR % RETURN	FLUSH	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock  NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
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DEPTH SCALE

1 : 50



LOGGED: JB

CHECKED: SEMC

MTO-RCK 001 12-1191-0005 NF GPJ GAL-MISS.GDT 07/11/12 DATA INPUT:

PROJECT		12-1191-0005		<b>RECORD OF BOREHOLE No NF12-4</b>				1 OF 1 <b>METRIC</b>					
W.P.				LOCATION		N 5442554.6; E 237261.8		ORIGINATED BY		JB			
DIST		HWY 71		BOREHOLE TYPE		HW Casing, HQ Coring		COMPILED BY		EC			
DATUM		GEODETIC		DATE		August 15 and 16, 2012		CHECKED BY		SEMC			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
336.6	GROUND SURFACE												
0.9	ASPHALT (50 mm)												
	Gravelly sand, some silt, some clay (FILL)												
	Compact to dense												
	Brown												
	Moist												
	Geogrid encountered at 0.2 m depth.												
334.5													
2.1	ROCK FILL												
			1	SS	31								
			2	SS	14								
			1	RC	REC 26%								
			2	RC	REC 0%								
			3	RC	REC 0%								
	No recovery in split spoon Sample 3.		3	SS	2								
			4	RC	REC 60%								
			5	RC	REC 41%								
			6	RC	REC 50%								
328.8													
7.8	CLAY, trace wood												
	Very stiff												
	Grey												
	Wet												
			4	TO	PH								
327.5													
9.1	Gravelly Sandy SILT, trace clay												
	Loose												
	Grey												
	Wet												
	Spoon bouncing at 9.6 m depth.												
	GRANITE GNEISS (BEDROCK)												
	Bedrock cored from 9.6 m to 12.7 m depth.												
	For details see Record of Drillhole NF12-4.												
			7	RC	REC 87%								
			8	RC	REC 100%								
323.9													
12.7	END OF BOREHOLE												
	Note:												
	1. Water level not recorded due to coring operations. Borehole dry prior to start of coring.												
	2. Laboratory vane taken in Shelby tube in Sample 4.												

PROJECT: 12-1191-0005

**RECORD OF DRILLHOLE: NF12-4**

SHEET 1 OF 1

LOCATION: NESTOR FALLS

DRILLING DATE: August 15 and 16, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Acker MP8

DRILLING CONTRACTOR: PADDOCK DRILLING LTD.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break BR - Broken Rock										NOTE: For additional abbreviations refer to list of abbreviations & symbols.										NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
				DEPTH (m)			RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA						HYDRAULIC CONDUCTIVITY				Diameter Point Load Index (MPa)	RMC -Q <sup>2</sup> AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
							TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	k <sub>1</sub> cm/s	k <sub>2</sub> cm/s	k <sub>3</sub> cm/s																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
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DEPTH SCALE

1 : 50



LOGGED: JB

CHECKED: SEMC

MTO-RCK 001 12-1191-0005 NF GPJ GAL-MISS.GDT 07/11/12 DATA INPUT:

PROJECT 12-1191-0005		RECORD OF BOREHOLE No NF12-5				1 OF 1 METRIC						
W.P. _____		LOCATION N 5442540.7; E 237249.5		ORIGINATED BY ES								
DIST _____ HWY 71		BOREHOLE TYPE HQ Casing with tricone, HQ Coring		COMPILED BY EC								
DATUM GEODETIC		DATE August 21, 2012		CHECKED BY SEMC								
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>			25 50 75
336.1	GROUND SURFACE											
0.0	ASPHALT (50 mm)											
	Gravelly sand to sand and gravel, trace to some silt (FILL)											
	Compact Brown Moist		1	SS	22							
334.6	Geogrid encountered at 0.1 m depth.											
1.5	ROCK FILL		2	SS	19/0.28							
			1	RC	REC 50%							
			2	RC	REC 87%							
			3	RC	REC 5%							
			4	RC	REC 56%							
	Approximately 50 mm of clay in bottom of core barrel at 5.5 m depth.		5	RC	REC 100%							
329.9	Shelby tube attempted at 6.2 m depth.		4	SS	8/0.1							
329.6	Gravelly SAND, some silt, trace to some clay											24 49 20 7
6.5	Loose Brown Wet		6	RC	REC 100%							RQD = 90%
	Spoon bouncing at 6.3 m depth.											
	GNEISS (BEDROCK)											
	Bedrock cored from 6.5 m to 9.4 m depth.		7	RC	REC 100%							RQD = 86%
	For details see Record of Drillhole NF12-5.											
326.7	END OF BOREHOLE											
9.4	Note: 1. Water level at a depth of 5.8 m below ground surface (Elev. 330.3 m) upon completion of drilling.											

SUD-MTO 001 12-1191-0005 NF.GPJ CAL-MISS.GDT 07/02/13 DATA INPUT:

PROJECT: 12-1191-0005

**RECORD OF DRILLHOLE: NF12-5**

SHEET 1 OF 1

LOCATION: NESTOR FALLS

DRILLING DATE: August 21, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Acker MP8

DRILLING CONTRACTOR: PADDOCK DRILLING LTD.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate												BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage												PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular												PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break												BR - Broken Rock	NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
							RECOVERY				R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA												HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)	RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
							FLUSH	TOTAL CORE %	SOLID CORE %	B Angle			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION												k, cm/s	Ja	Jn																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
7	HQ Coring August 21, 2012	Refer to Previous Page		329.6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															

DEPTH SCALE

1 : 50




LOGGED: ES

CHECKED: SEMC

MTO-RCK 001 12-1191-0005 NF GPJ GAL-MISS.GDT 07/11/12 DATA INPUT:



PROJECT		12-1191-0005		<b>RECORD OF BOREHOLE No NF12-6</b>				1 OF 1 <b>METRIC</b>									
W.P.				LOCATION		N 5442511.9; E 237239.8		ORIGINATED BY		ES							
DIST		HWY 71		BOREHOLE TYPE		HQ Casing with tricone, HQ Coring		COMPILED BY		EC							
DATUM		GEODETIC		DATE		August 20, 2012		CHECKED BY		SEMC							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
334.8 0.0	GROUND SURFACE Gravelly sand to sand and gravel, trace to some silt (FILL) Compact Brown Moist		1	AS	-												24 69 (7)
			2	SS	15												
332.9 1.9	Spoon refusal at 1.9 m depth. ROCK FILL		3	SS	84/0.28												
			1	RC	REC 50%												
			2	RC	REC 63%												
330.1 4.7	GNEISS (BEDROCK) Bedrock cored from 4.7 m to 7.3 m depth. For details see Record of Drillhole NF12-6.		3	RC	REC 100%												RQD = 48%
			4	RC	REC 100%												RQD = 35%
327.5 7.3	END OF BOREHOLE  Note: 1. Water level not recorded due to coring operations. Borehole dry prior to start of coring.																

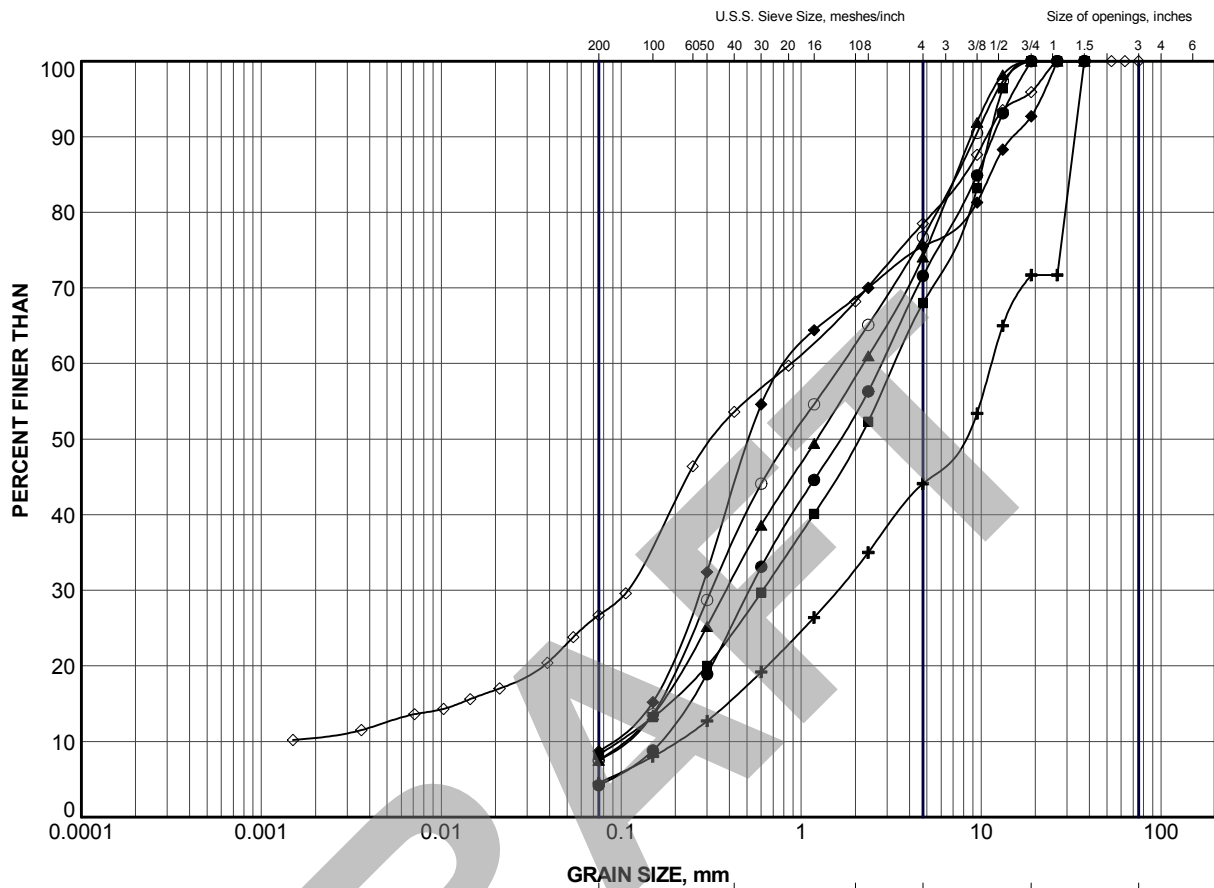




# APPENDIX C

## Laboratory Test Results

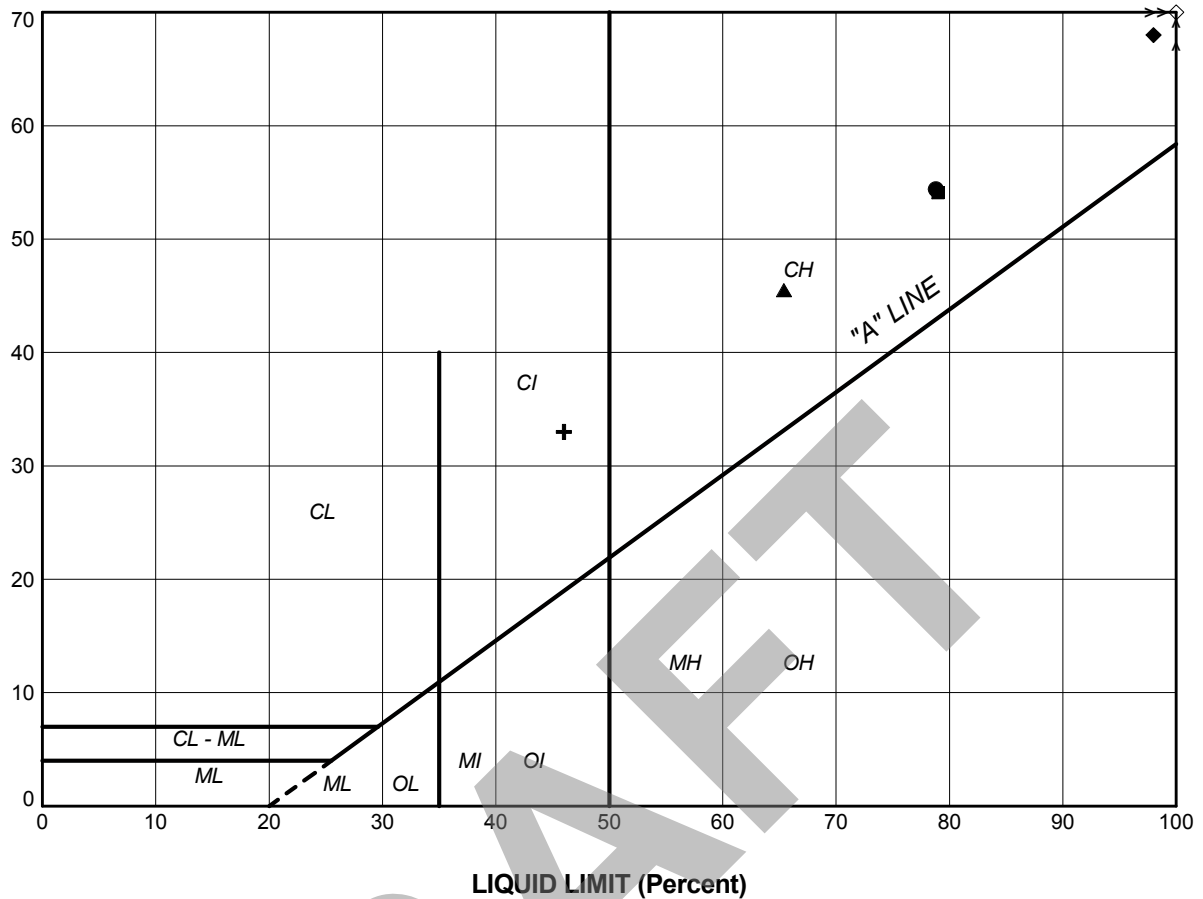
DRAFT



PROJECT					HIGHWAY 71 NESTOR FALLS				
TITLE					<b>GRAIN SIZE DISTRIBUTION</b> Gravelly Sand to Sand and Gravel (Fill)				
PROJECT No.		12-1191-0005		FILE No.		12-1191-0005 NF.GPJ			
DRAWN	TB	Oct 2012		SCALE	N/A		REV.		
CHECK	SEMC	Oct 2012		<b>FIGURE C1</b>					
APPR	JMAC	Oct 2012							



PLASTICITY INDEX (Percent)




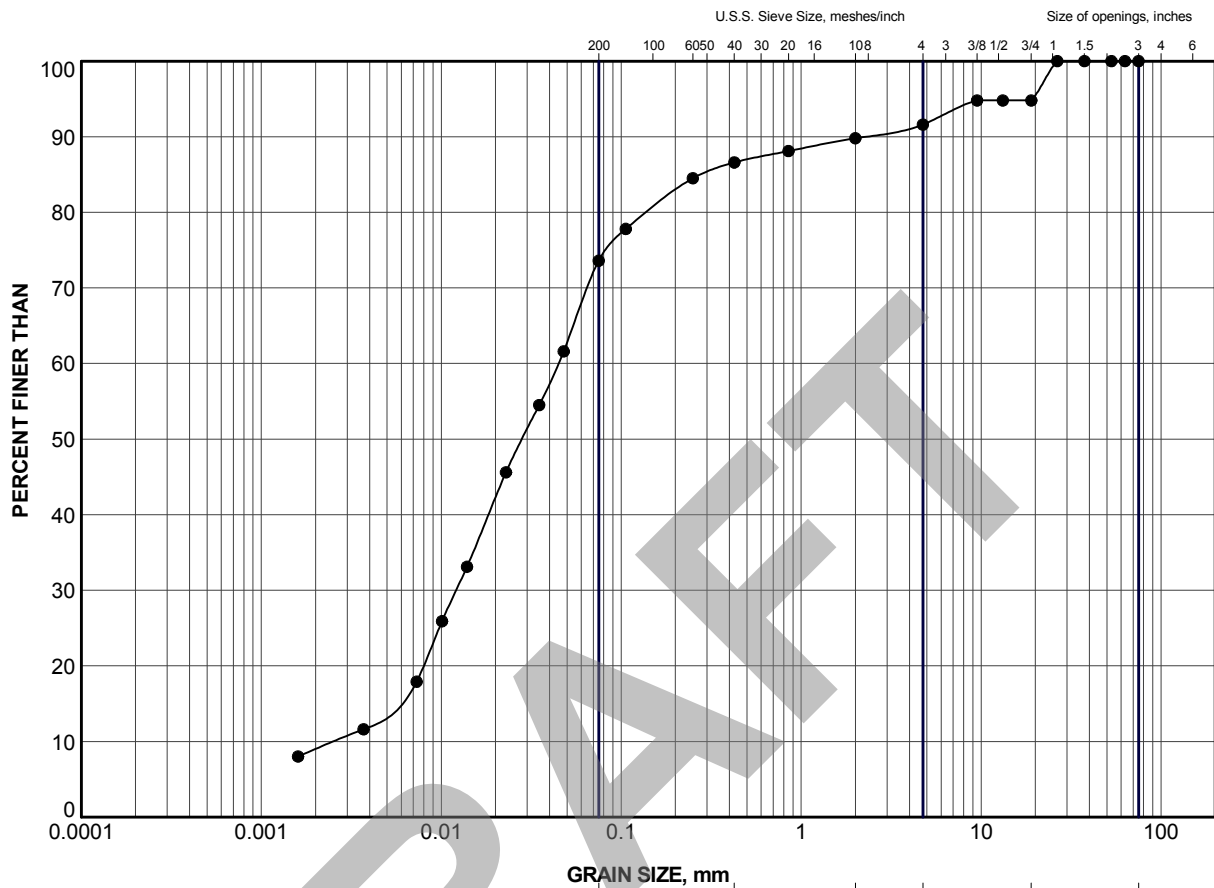
**SOIL TYPE**  
C = Clay  
M = Silt  
O = Organic

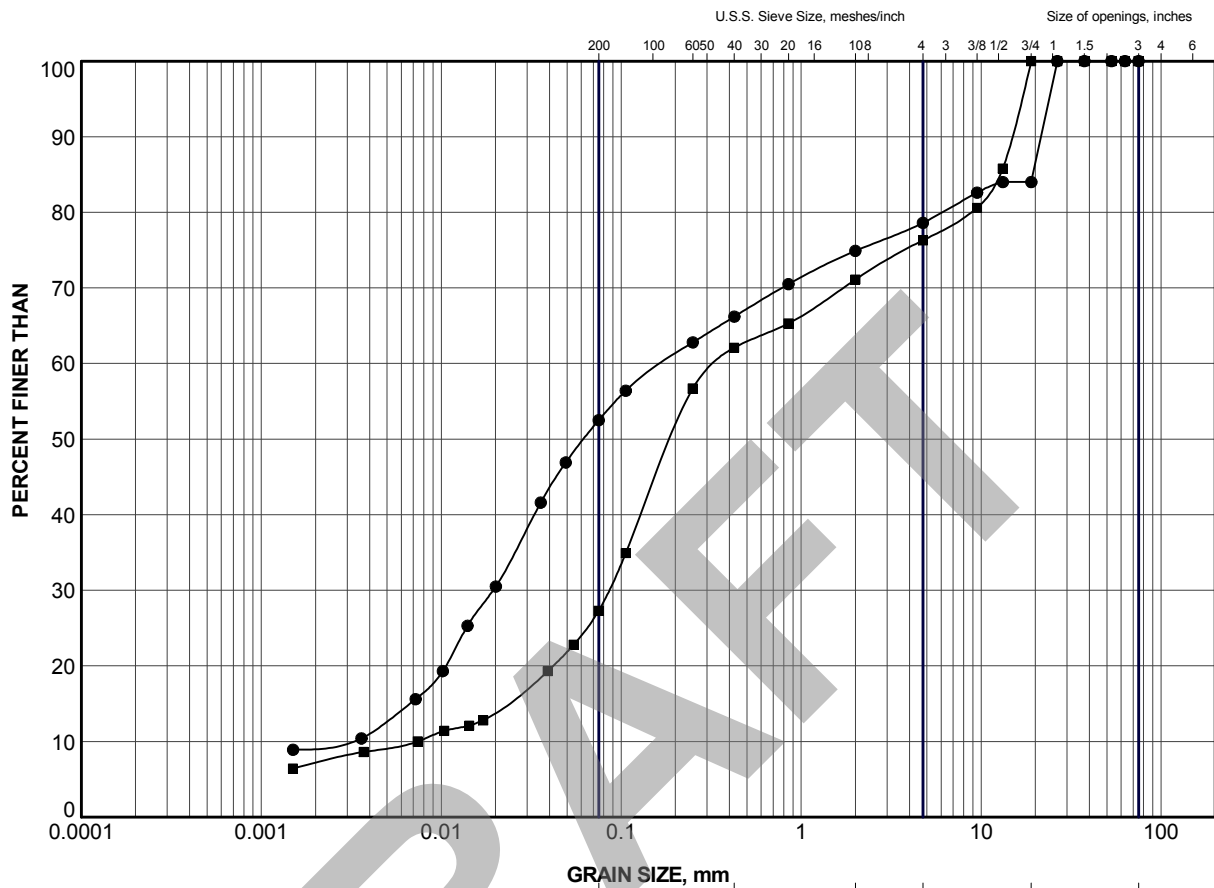
**PLASTICITY**  
L = Low  
I = Intermediate  
H = High

### LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	NF12-2	RC4	78.8	24.4	54.4
■	NF12-2	RC4	79.0	24.9	54.1
▲	NF12-4	4	65.4	19.9	45.5
+	BH 1 (DST)	2	46.0	13.0	33.0
◆	BH 1 (DST)	5	98.0	30.0	68.0
◇	BH 1 (DST)	6	112.0	33.0	79.0

PROJECT					HIGHWAY 71 NESTOR FALLS				
TITLE					<b>PLASTICITY CHART</b> Silty Clay to Clay				
PROJECT No. 12-1191-0005			FILE No. 12-1191-0005 NF.GPJ						
DRAWN	TB	Feb 2013	SCALE	N/A	REV.				
CHECK	SEMC	Feb 2013							
APPR	JMAC	Feb 2013							
 <b>Golder Associates</b> SUDBURY, ONTARIO			<b>FIGURE C2</b>						






GRAIN SIZE, mm							Cobble Size
CLAY AND SILT	fine	medium	coarse	fine	coarse		
		SAND SIZE			GRAVEL SIZE		

#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NF12-4	5	327.1
■	NF12-5	4	329.8

PROJECT					HIGHWAY 71 NESTOR FALLS				
TITLE					<b>GRAIN SIZE DISTRIBUTION</b> Gravelly Sand and Gravelly Sandy Silt				
PROJECT No.		12-1191-0005		FILE No.		12-1191-0005 NF.GPJ			
DRAWN	TB	Feb 2013		SCALE	N/A	REV.			
CHECK	SEMC	Feb 2013		<b>FIGURE C4</b>					
APPR	JMAC	Feb 2013							
 <b>Golder Associates</b> SUDBURY, ONTARIO									

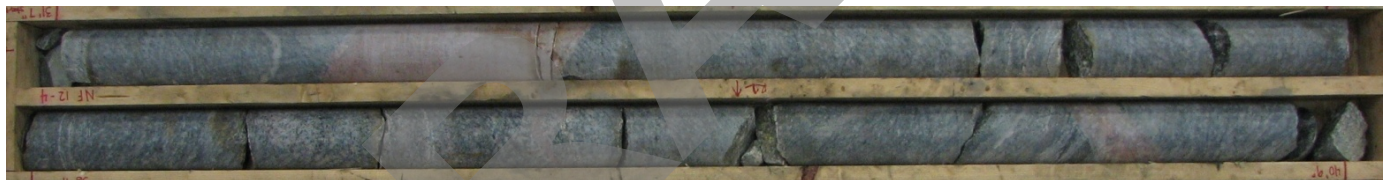




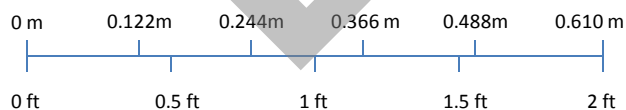
Borehole NF12-1  
Elevation 326.5 m to 324.1 m




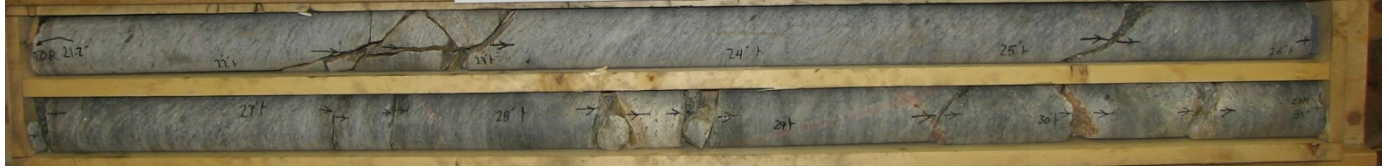
Borehole NF12-3  
Elevation 327.8 m to 324.6 m



Borehole NF12-4  
Elevation 327.0 m to 323.9 m



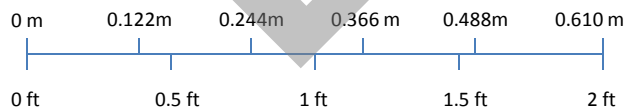
PROJECT		HIGHWAY 71 NESTOR FALLS			
TITLE		BEDROCK CORE PHOTOGRAPHS (Boreholes NF12-1, NF12-3, NF12-4)			
	PROJECT No. 12-1191-0005			FILE No. ----	
	DESIGN	MH	MAY 2013	SCALE	AS SHOWN
	CADD	--	MAY 2013	REV.	
	CHECK	SEMC		FIGURE C5	
	REVIEW	JMAC	MAY 2013		




Borehole NF12-5  
Elevation 329.6 m to 326.7 m



Borehole NF12-6  
Elevation 330.1 m to 327.5 m



PROJECT		HIGHWAY 71 NESTOR FALLS			
TITLE		BEDROCK CORE PHOTOGRAPHS (Boreholes NF12-5, NF12-6)			
	PROJECT No. 12-1191-0005			FILE No. ----	
	DESIGN	MH	MAY 2013	SCALE	AS SHOWN
	CADD	--			REV.
	CHECK	SEMC	MAY 2013	FIGURE C6	
	REVIEW	JMAC	MAY 2013		



## APPENDIX D

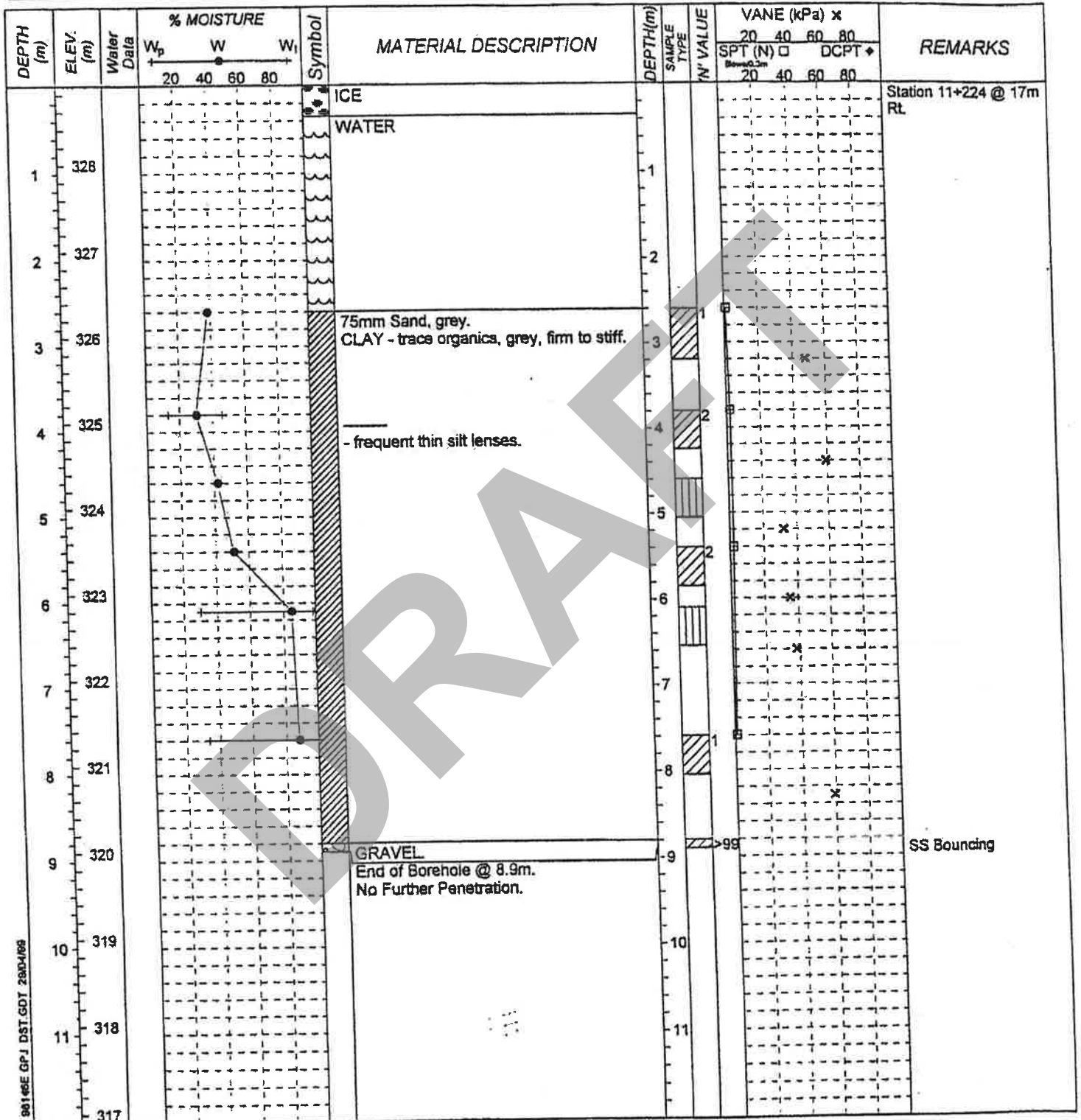
Foundation Design Report, GEOCRES NO. 52-F-26,  
DST Consulting Engineers

DRAFT

# LOG OF BOREHOLE 1

DST REF. No.: TG98146F  
 CLIENT: Cook Engineering  
 PROJECT: Foundation Investigation  
 LOCATION: Nestor Falls, Ontario  
 SURFACE ELEV.: 328.9 metres

Drilling Data  
 METHOD: Washbore  
 DIAMETER: 80mm ID  
 DATE: February 15 1999

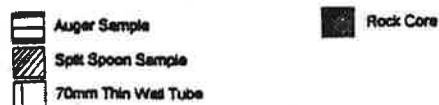


BOREHOLE (STANDARD) 98146E GPJ DST-GDT 2804/99



DST Consulting Engineers Inc.  
 605 Hewitson Street  
 Thunder Bay, Ontario P7B 2M8  
 PH: (807) 623-2929  
 FX: (807) 623-1792  
 Email: dst@d-st-engineers.on.ca  
 Web: www.d-st-engineers.on.ca

## SAMPLE TYPE LEGEND



ENCLOSURE 3

# LOG OF BOREHOLE 2

DST REF. No.: TG98146F  
 CLIENT: Cook Engineering  
 PROJECT: Foundation Investigation  
 LOCATION: Nestor Falls, Ontario  
 SURFACE ELEV.: 328.9 metres

Drilling Data  
 METHOD: Washbore  
 DIAMETER: 80mm ID  
 DATE: February 15 1999

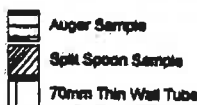
DEPTH (m)	ELEV. (m)	Water Data	% MOISTURE			Symbol	MATERIAL DESCRIPTION	DEPTH(m)	SAMPLE TYPE	N VALUE	VANE (kPa) x				REMARKS
			W <sub>p</sub>	W	W <sub>i</sub>						SPT (N) □ DCPT ♦				
											20	40	60	80	
			20	40	60	80					20	40	60	80	
							ICE								Station 11+271 @ 20m Rt.
							WATER								
1	328														
2	327						CLAY - Trace organics, grey, soft to firm.								
3	326														
4	325						- layered.								
5	324						- thin seams sand.								
6	323						End of Borehole @ 5.8m. No Further Penetration.								
7	322														
8	321														
9	320														
10	319														
11	318														
	317														

BOREHOLE (STANDARD) 98146E GPJ DST.GOT 29/04/99



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 Web: www.d-st-engineers.on.ca

## SAMPLE TYPE LEGEND



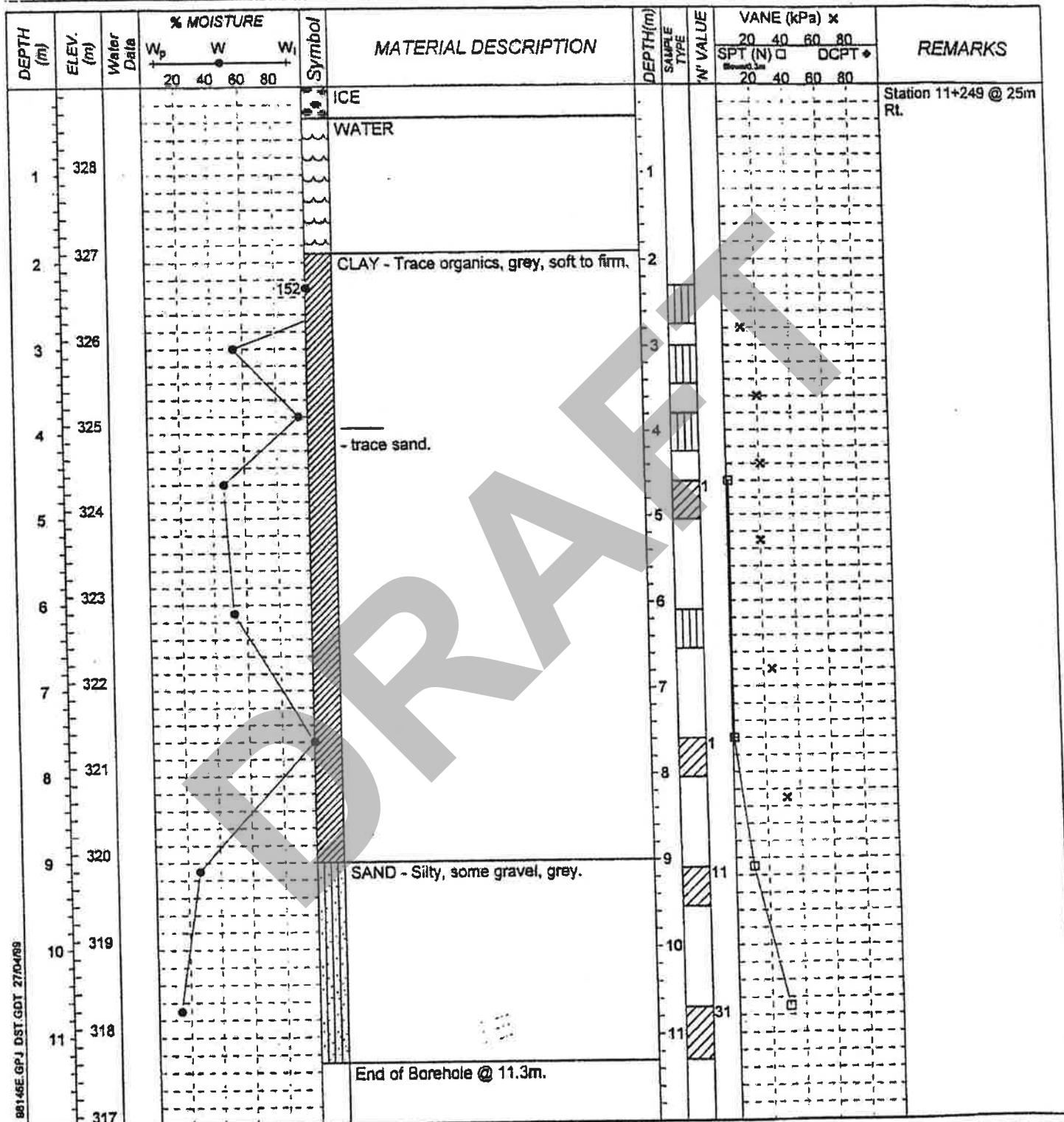
ENCLOSURE 4



# LOG OF BOREHOLE 3

DST REF. No.: TG98146F  
 CLIENT: Cook Engineering  
 PROJECT: Foundation Investigation  
 LOCATION: Nestor Falls, Ontario  
 SURFACE ELEV.: 328.9 metres

Drilling Data  
 METHOD: Washbore  
 DIAMETER: 80mm ID  
 DATE: February 15 1999

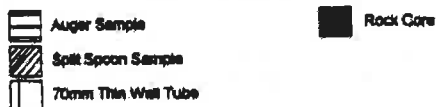


BOREHOLE (STANDARD) 98146E.GPJ DST GDT 27/04/99



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 805 Hewitson Street  
 Thunder Bay, Ontario P7B 2M8  
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 FX: (807) 623-1792  
 Email: dst@d-st-engineers.on.ca  
 Web: www.dst-engineers.on.ca

## SAMPLE TYPE LEGEND



ENCLOSURE 5

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

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