



## Foundation Investigation and Design Report

*Embankment Instability on Highway 658, Township of Redditt*

Agreement No. 6021-E-0019

Assignment No. 5

Geocres No.: 52E-75

(Latitude: 49.975717°; Longitude: -94.393332°)

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## Table of Contents

<b>1</b>	<b>FOUNDATION INVESTIGATION REPORT.....</b>	<b>1</b>
1.1	Introduction.....	1
1.2	Site Description and Geological Setting .....	1
1.2.1	Site Description .....	1
1.2.2	Geological Setting.....	2
1.3	Previous Investigations.....	2
1.4	Investigation Procedures.....	2
1.4.1	Site Investigation and Field Testing.....	2
1.4.2	Laboratory Testing .....	4
1.5	Subsurface Conditions.....	5
1.5.1	Soil Stratigraphy .....	5
1.5.1.1	Asphalt.....	5
1.5.1.2	Topsoil .....	5
1.5.1.3	Fill: Gravel / Sand and Gravel / Gravelly Sand / Sand .....	6
1.5.1.4	Clay / Silty Clay / Clayey Silt .....	6
1.5.1.5	Silt / Sandy Silt / Silty Sand / Sand and Silt .....	8
1.5.1.6	Sandy Gravel / Gravelly Sand.....	9
1.5.1.7	Bedrock.....	10
1.6	Groundwater and Surface Water Conditions .....	11
1.7	Chemical Analysis .....	11
<b>2</b>	<b>ENGINEERING DISCUSSION &amp; RECOMMENDATIONS .....</b>	<b>13</b>
2.1	General .....	13
2.2	Assessment of Existing Conditions .....	13
2.2.1	Background.....	13
2.2.2	Site Observations .....	14
2.2.3	Assessment of Causes of Slope Instability.....	15
2.2.4	Slope Stability Analyses .....	16
2.2.4.1	General .....	16
2.2.4.2	Analysis Methodology .....	16
2.2.4.3	Soil Properties and Groundwater Conditions .....	17
2.2.4.4	Seismic Properties .....	18
2.2.4.5	Stability Results of Original Slope Before the 2022 Failure .....	19
2.2.4.6	Stability Results of Existing Slope After the 2022 Failure .....	20
2.3	Remediation Alternatives.....	21
2.4	Recommended Remediation Measure.....	26
2.5	Risk Assessment and Immediate Measures .....	28
2.6	Monitoring Plan.....	29
<b>3</b>	<b>CLOSURE .....</b>	<b>30</b>

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Date: December 20, 2022*

<b>REFERENCES.....</b>	<b>31</b>
<b>LIMITATIONS AND USE OF REPORT .....</b>	<b>32</b>

## **Appendices**

APPENDIX A: SITE PHOTOGRAPHS
APPENDIX B: DRAWINGS
APPENDIX C: BOREHOLE LOGS
APPENDIX D: LABORATORY DATA
APPENDIX E: BEDROCK CORE PHOTOGRAPHS
APPENDIX F: SUMMARY OF SOIL PROPERTIES AND SEISMIC HAZARD VALUES
APPENDIX G: SLOPE STABILITY ANALYSES
APPENDIX H: REMEDIATION MEASURES - SKETCHES
APPENDIX I: TECHNICAL MEMORANDUM – SITE REVIEW
APPENDIX J: NSSP
APPENDIX K: SETTLEMENT MONITORING PLAN
APPENDIX L: SCHEMATIC DRAWING FOR RIGID EPS EMBANKMENT



# 1 FOUNDATION INVESTIGATION REPORT

## 1.1 Introduction

This report presents the results of a geotechnical investigation completed by EXP Services Inc. to investigate the slope instability along Highway 658 that occurred in late June of 2022 in Redditt, Ontario. The slope instability occurred along the west side of Highway 658 (Latitude: 49.975717°; Longitude: - 94.393332°), just south of the Black River bridge and immediately north of a past slope instability in the area that occurred in 2014. The work was undertaken under Agreement No. 6021-E-0019, Assignment No. 5, as a continuation of a site review visit conducted by EXP on July 7, 2022. The terms of reference (TOR) for Assignment No. 5 were provided by MTO, in a request dated July 14, 2022.

The purpose of the investigation was to geotechnically characterize the subsurface soil and groundwater conditions and permit slope stabilization improvement strategies associated with the embankment/slope south of the Black River bridge on Highway 658. The site-specific geotechnical investigation consisted of a field investigation including visual inspections, drilling of boreholes, in-situ soil testing (i.e., SPT and shear vane tests), soil sampling, and laboratory testing. Based on this data, borehole location plans, cross sectional subsurface profiles, record of boreholes, laboratory test results and a written description of subsurface conditions will be provided herein.

This foundation investigation report has been prepared specifically and solely for the project described herein. It contains the factual results of the investigation and the laboratory testing completed for this project, inclusive of the initial instrumentation installation details.

## 1.2 Site Description and Geological Setting

### 1.2.1 Site Description

The slope instability that occurred in 2022 is located along the west shoulder of Highway 658, south of the Black River bridge and directly north of a previously documented slope instability which occurred in 2014 in Redditt, Ontario (Latitude: 49.975717°; Longitude: - 94.393332°). The project site subject to investigation is approximately 60 m in length covering both the new instability area and the previous failure area from 2014. Appendix A in this report includes the photographs of this site.

At the site, Highway 658 is a two-lane paved roadway that runs northeast/southwest (55°/235° from the horizontal). The width of the roadway ranges from 5 m to 8 m from edge of pavement to edge of pavement, becoming narrower at the Black River bridge. Gravel shoulders are present on both sides of the roadway with guardrails along the bridge section and the slope instability area of the roadway. Power poles are present along the west side of the highway. Based on the AutoCAD drawings provided by MTO, the elevation of the highway centre line at the site ranges from approximately 336.5 m to 326.8 m where the elevation decreases from southwest to northeast. The general topography of Highway 658 has a steep downward gradient along the roadway surface from south to north along the section of roadway that has experienced slope instabilities in 2014 and 2022. The existing west roadway embankment side has a steepness of 1H:1V at the leading edge of the slope instability. Photographs 1 and 2 in Appendix A show Highway 658 at the failure site.

The general soil and site conditions were visually assessed during the site visit on July 7, 2022, in which shallow manual hand digging was used to evaluate surficial soil conditions near the head scarp of the slope instability along the roadway embankment. The soils encountered consisted of a gravelly sand layer directly underneath the asphalt

followed by a very soft, medium plastic clay layer. The surficial soils near the toe of the slope instability were visually observed to consist of a wet to saturated organic layer over a silt/clay mix. Mature vegetation growth was observed on both sides of the roadway and in the vicinity of the slope instability. Various trees and shrubs were observed to exhibit a downslope lean, consistent with the down slope movement of the slope instability (Photograph 3 in Appendix A). It was also observed that the west side of the roadway embankment was inundated with water at the top of the roadway embankment from surficial drainage along the roadway (e.g., heavy spring precipitation, snow melt, etc.), coupled with flood water rise along the Black River floodplain area at the slope toe. The east gravel shoulder of the roadway across from the slope instability exhibited ditch erosion from surficial drainage flow downward from south to north towards the Black River floodplain (Photograph 4 in Appendix A). At the north portion of the site adjacent to the location of the bridge, the area was observed to be soft swampy marshlands with various vegetative growth during the investigation in October 2022 (Photograph 6 in Appendix A).

### 1.2.2 Geological Setting

According to the Ministry of Northern Development and Mines, Map 2554 (Quaternary Geology of Ontario, West-Central Sheet, 1991) the surface conditions in the vicinity of the project area consists of Precambrian bedrock, undifferentiated igneous and metamorphic rock, exposed at surface or rock covered by a discontinuous, thin layer of drift and according to Map 2542 (Bedrock Geology of Ontario, West-Central Sheet, 1991), the bedrock geology of the site is of massive granodiorite to granite – potassium feldspar megacrystic units. According to the Northern Ontario Engineering Geology Terrain Study, Data Base Map, Rat Potage Bay, Ontario Geological Survey, Map 5055, the site landform is of organic terrain.

## 1.3 Previous Investigations

There is no previous investigation at this site. However, there is the available report of the previous investigation performed in the vicinity of the site in the MTO GEOCRETS library:

- *Geocres No. 52E-22: "Soils and Foundation Report, Proposed Realignment of Black River Bridge, Redditt, Ontario", prepared by Morton, Dodds & Partners Limited Consulting Engineers & Specialists, October 1978.*

## 1.4 Investigation Procedures

### 1.4.1 Site Investigation and Field Testing

The site investigation was performed on October 3 – 7, 2022. The proposed field program prepared based on the MTO Guideline for Foundation Engineering Services (April 2022) and approved by MTO consisted of drilling nine (9) sampled boreholes, numbered BH22-01 to BH22-09. Three (3) boreholes, BH22-01 to BH22-03, were located on the southbound lane of the roadway along Highway 658 adjacent to the instability areas, spaced approximately 15 m apart. Three (3) boreholes, BH22-04 to BH22-06, were proposed to be drilled at the toe of the existing instability area spaced approximately 14 m to 15 m apart, while three (3) boreholes, BH22-07 to BH22-09 were located approximately 5 m to 10 m further away from the toe (to provide stability analyses at three sections), spaced about 10 m to 12 m apart. The exploration locations are shown on the drawings in Appendix B. Table 1.1 summarizes the locations of the boreholes completed during this investigation.

Table 1.1. Summary of boreholes completed during this investigation

Borehole /Test Pit No.	Location	Location (MTM NAD 83 Zone ON-16)		Latitude	Longitude	Ground Surface Elevation <sup>1</sup> (m)	Borehole Depth <sup>2</sup> (m)
		Northing	Easting				
22-01	Roadway	5538523.4	204865.7	49.97572	-94.39332	329.1	6.7
22-02		5538509.8	204857.2	49.97560	-94.39344	331.2	5.2
22-03		5538497.9	204847.6	49.97549	-94.39357	333.5	3.2
22-04	Slope Toe	5538531.8	204856.9	49.97580	-94.39345	325.1	1.1 <sup>3</sup>
22-05		5538521.5	204846.1	49.97570	-94.39360	326.5	3.6
22-06		5538508.8	204839.7	49.97559	-94.39369	328.8	1.8 <sup>3</sup>
22-07	5 m to 10 m away from the slope toe	5538531.5	204849.8	49.97579	-94.39355	325.0	2.3 <sup>3</sup>
22-08		5538524.7	204843.0	49.97573	-94.39364	325.3	2.2 <sup>3</sup>
22-09		5538514.4	204837.0	49.97564	-94.39373	327.7	2.8 <sup>3</sup>

## Notes:

1. The ground surface elevations referenced are Geodetic.
2. Depths are relative to ground surface.
3. Terminated at shallow depth due to auger/SPT refusal on assumed bedrock surface.

Boreholes were advanced using either a rubber track mounted CME 55/300 drill rig or a portable skid-mounted B20L drill rig, depending on the location and accessibility of the borehole. BH22-01 to BH22-03, which were located on the roadway, were completed using a rubber track mounted CME 55/300 drill rig equipped with continuous flight hollow stem augers, HQ casing, NQ core barrels, wash boring equipment, and standard soil sampling equipment (Photograph 7 in Appendix A). BH22-04 to BH22-09, which were drilled off the roadway near the toe of the slope instability, were completed using a portable skid-mounted B20L drill rig equipped with solid stem augers and a tripod (Photograph 8 in Appendix A). To position the portable rig at borehole locations a backhoe was used. The boreholes were advanced to a depth between 1.1 m to 6.7 m below ground surface. The drill rigs and backhoe were owned

and operated by a specialist drilling contractor, Maple Leaf Drilling from Sunnyside, Manitoba. The traffic control was also provided by Maple Leaf Drilling.

The borehole locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were surveyed by EXP personnel using a GPS (Garmin Montana 680) and a basic level and survey rod, respectively, having an accuracy of  $\pm 1$  m in the horizontal directions and 0.1 m in the vertical direction. The relative distances between the location of boreholes and geographical and structural features on the site were also measured by a field measuring tape. A temporary benchmark (TBM) at the western corner of the southern bridge abutment was used having an elevation of Elev. 326.8 m.

For the drilling program, majority of soil samples were obtained using a 51 mm outside diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586) at intervals ranging from 0.75 m to 1.5 m in depth as shown on the attached borehole logs (Appendix C). The original field (uncorrected) SPT “N” values were recorded on the borehole logs as recommended in the Canadian Foundation Engineering Manual (CFEM, pg. 40) and used to provide an assessment of in-situ relative density of cohesionless soils and consistency of cohesive soils. When a hard stratum or refusal was encountered, sampling of the stratigraphic layer was performed by diamond core drilling, using a 1.5 m long NQ double tube wireline core barrel, when possible. Shear vane testing was completed in the cohesive soils encountered within the boreholes during drilling, using a MTO vane in accordance with ASTM-D2573. A stabilizing vane collar rigidly attached to the borehole casing to maintain the shear vane in a central position at a consistent elevation during the torquing process was used. In-situ shear vane results are shown on the borehole logs as an indication of shear strength, corrected for plasticity index using Bjerrum method. Shelby tube sampling was also conducted at select borehole locations for additional laboratory testing on relatively undisturbed soil samples.

Upon completion of the boreholes, groundwater level measurements were carried out in boreholes in accordance with MTO guidelines. Twenty-five (25) mm inside diameter PVC piezometers were installed in all boreholes for short-term and potential long-term groundwater monitoring. The slotted sections of the piezometers were backfilled with filter sand, inclusive of a bentonite plug constructed above (to existing ground surface) and below (to bottom of borehole) the slotted zone to isolate the targeted depth of groundwater measurement. The recorded groundwater levels after completion of drilling boreholes and subsequent to the completion of drilling on various dates were presented in the borehole log sheets in Appendix C. The boreholes were registered as monitoring wells in accordance with the Ministry of the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the Ontario Water Resources Act).

The fieldwork was supervised by an EXP geotechnical representative who directed the drilling and sampling operation, logged borehole data in accordance with MTO and/or ASTM Standards for Soils Classification and retrieved soil samples for subsequent laboratory testing and identification.

All recovered soil samples were placed in labelled moisture-proof bags and returned to EXP’s Thunder Bay laboratory for additional visual, textual, olfactory examination and selective testing. However, all Shelby tube samples were returned to EXP’s Brampton laboratory for testing.

#### **1.4.2 Laboratory Testing**

All samples returned to the laboratory were subjected to visual examination and classification. The laboratory testing program included the determination of natural moisture content on all soil samples, grain size distribution

testing for approximately 25% of the collected soil samples, and Atterberg tests on about 25% of cohesive samples collected. In addition, soil chemical package tests were performed on one (1) soil sample. All laboratory tests were carried out according to MTO and/or ASTM Standards as appropriate.

The results of laboratory tests on soil samples are provided on the attached borehole log sheets in Appendix C. The results of the grain size analyses and Atterberg limits tests are presented graphically in Appendix D. The results of chemical tests are also included in Appendix D.

## **1.5 Subsurface Conditions**

The detailed subsurface conditions encountered in the boreholes advanced during the investigation are presented on the borehole log sheets in Appendix C. Laboratory test results of grain size distribution and Atterberg limit tests are provided in Appendix D. The “Explanation of Terms Used in Report” preceding the borehole in Appendix C forms an integral part of and should be read in conjunction with this report.

A borehole location plan and cross section subsurface profiles are provided in the drawings attached in Appendix B. It should be noted that the stratigraphic boundaries indicated on the borehole logs and cross section stratigraphic profiles are inferred from semi-continuous sampling, observations of drilling progress and results of Standard Penetration Tests (SPT). These boundaries typically represent transitions from one soil type to another and should not be regarded as exact planes of geological change. Furthermore, subsurface conditions may vary between and beyond the borehole locations.

In general, the subsurface profile at the location of the roadway (BH22-01 to BH22-03) consists of sand and gravel fill, sand fill, and or gravel fill below the asphalt, or at the ground surface for BH22-03, followed by native clay then a layer of sandy silt to silty sand which is followed by bedrock in BH22-01 and BH22-02 and a layer of silty clay in BH22-03. The subsurface profile within the location of the slope toe of the slope instability (BH22-04 to BH22-06) consists of clay or silty clay in BH22-04 and BH22-05 and silt in BH22-06 below topsoil followed by sandy gravel in BH22-04, bedrock in BH22-05, and sand in BH22-06. At the locations 5 m to 10 m away from the slope toe (BH22-07 to BH22-08), the subsurface profile consists of clay or silty clay below topsoil followed by silt to silty sand in BH22-07 and BH22-08 and gravelly sand in BH22-09. A detailed description of the subsurface conditions encountered is discussed further in subsequent sections.

### **1.5.1 Soil Stratigraphy**

#### **1.5.1.1 Asphalt**

Asphalt, approximately 0.04 m to 0.05 m thick, was encountered at the surface of boreholes BH22-01 and BH22-02.

#### **1.5.1.2 Topsoil**

Topsoil, approximately 0.025 m to 0.1 m thick, was encountered at the surface of boreholes BH22-04 to BH22-09.

### 1.5.1.3 Fill: Gravel / Sand and Gravel / Gravelly Sand / Sand

Gravel / sand and gravel / gravelly sand / sand fill was encountered below the asphalt in the boreholes advanced through the roadway, BH22-01 and BH22-02, and at the ground surface in BH22-03. The depths and elevations of the fill layer encountered at these borehole locations are listed in Table 1.2.

Table 1.2. Summary of gravel / sand and gravel / sand fill layer

Borehole No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
BH22-01	329.0	327.9	0.1	1.1
BH22-02	331.1	330.2	0.0	0.9
BH22-03	333.5	331.9	0.0	1.5

Note:

1. Depths are relative to ground surface.

The composition of this fill material generally consisted of sand and gravel to gravelly sand with trace asphalt, trace silt, trace clay with occasional cobbles. The fill was generally brown to black in colour, and moist. The SPT “N” values obtained within this layer ranged from about 10 to 18 blows per 0.3 m penetration, suggesting that this fill layer was compact in compactness.

Laboratory testing performed on selected samples consisted of eight (8) moisture content tests and four (4) grain size distribution tests. The test results are as follows:

Moisture Content:

- 2% to 8%

Grain Size Distribution:

- 19% to 82% gravel
- 15% to 72% sand
- 2% to 7% silt and clay

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 1 in Appendix D.

### 1.5.1.4 Clay / Silty Clay / Clayey Silt

Native clay /silty clay / clayey silt was encountered below the fill soils in boreholes BH22-01, BH22-02, and BH22-03, and below the topsoil in boreholes BH22-04, BH22-05, BH22-07, BH22-08, and BH22-09. This soil was also encountered below the silty sand layer and just above the bedrock in borehole BH22-03. The depths and elevations of this layer encountered at these borehole locations are listed in Table 1.3.

Table 1.3. Summary of clay / silty clay / clayey silt layer

Borehole No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
BH22-01	327.9	326.8	1.2	1.1
BH22-02	330.2	327.9	0.9	2.3
BH22-03	331.9	331.0	1.5	0.9
	330.4	330.3	3.1	0.1
BH22-04	325.1	324.2	0.1	0.8
BH22-05	326.5	324.4	0.1	2.0
BH22-07	325.0	324.4	0.0	0.6
BH22-08	325.2	324.4	0.0	0.9
BH22-09	327.6	325.4	0.0	2.3

Notes:

1. Depths are relative to ground surface.

The composition of this native material generally consisted of clay with some silt, trace to some sand, trace gravel, and trace to some organic material. This layer was brown to grey in BH22-01, BH22-03, BH22-04, BH22-05 and dark brown to black in BH22-02, BH22-07, BH22-08, and BH22-09, and moist to wet. The SPT “N” values obtained within this layer ranged from about 1 to 18 blows per 0.3 m penetration, suggesting that this layer was very soft to very stiff in consistency.

In-situ shear vane tests performed in this soil measured the undrained shear strength is between 27 kPa and 100 kPa indicating firm to very stiff state of consistency of this layer, but predominantly stiff to very stiff. Sensitivity of this layer soil was measured to be between 2.0 and 7.0. All results of vane tests are plotted on Figure F1 in Appendix F.

Laboratory testing performed on selected samples consisted of twenty (20) moisture content tests, eleven (11) grain size distribution tests, and eleven (11) Atterberg limits tests. The test results are as follows:

Moisture Content:

- 11% to 83%

Grain Size Distribution:

- 0% to 25% gravel
- 1% to 28% sand
- 16% to 80% silt
- 17% to 83% clay
- 50% to 95% silt and clay

#### Atterberg Limits:

- Liquid Limit: 25% to 86%
- Plastic Limit: 15% to 27%
- Plasticity Index: 8% to 62%

The results of the moisture content, grain size distribution, and Atterberg limits tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests and Atterberg limits tests are also provided on Figures 2, 3 and Figures 7, 8 respectively in Appendix D.

#### 1.5.1.5 Silt / Sandy Silt / Silty Sand / Sand and Silt

A layer of silt / sandy silt / silty sand / sand and silt was encountered beneath the native clay layer in boreholes BH22-01 to BH22-03 and BH22-07 to BH22-08 and below the topsoil in BH22-06. The depths and elevations of this layer encountered at these borehole locations are listed in Table 1.4.

Table 1.4 Summary of silt /sandy silt / silty sand / sand and silt layer

Borehole No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
BH22-01	326.8	325.6	2.3	1.3
BH22-02	327.9	327.6	3.2	0.4
BH22-03	331.0	330.4	2.4	0.7
BH22-06	328.7	327.1	0.1	1.7 <sup>2</sup>
BH22-07	324.4	322.7	0.6	1.7 <sup>2</sup>
BH22-08	324.4	323.1	0.9	1.3 <sup>2</sup>

#### Notes:

1. Depths are relative to ground surface.
2. End of borehole/terminated in this layer.

The composition of this native material generally consisted of silt, trace sand, trace to some clay, trace gravel, and trace organic material. The layer was generally dark brown to grey in colour and moist to wet. The SPT “N” values obtained within this layer ranged from about 1 to 24 blows per 0.3 m penetration, suggesting that this layer was very loose to compact in compactness.

Laboratory testing performed on selected samples consisted of twelve (12) moisture content tests, twelve (12) grain size distribution tests, and three (3) Atterberg limits tests. The test results are as follows:

#### Moisture Content:

- 8% to 106%



#### Grain Size Distribution:

- 0% to 20% gravel
- 5% to 71% sand
- 32% to 82% silt
- 4% to 13% clay
- 21% to 84% silt and clay

#### Atterberg Limits:

- Liquid Limit: 18% to 32%,
- Plastic Limit: 17% to 27%,
- Plasticity Index: 1 to 5

The results of the moisture content, grain size distribution, and Atterberg limits tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests and Atterberg limits tests are also provided on Figures 4, 5 and Figure 9 respectively in Appendix D.

#### 1.5.1.6 Sandy Gravel / Gravelly Sand

A layer of sandy gravel / gravelly sand was encountered beneath the clay and silty clay layers in BH22-04 and BH22-09. The depths and elevations of this layer encountered at these borehole locations are listed in Table 1.5.

Table 1.5. Summary of sandy gravel / gravelly sand layer

Borehole No.	Elevation (m)		Layer Surface Depth <sup>1</sup> (m)	Layer Thickness (m)
	Top	Bottom		
BH22-04	324.2	324.1	0.9	0.2 <sup>2</sup>
BH22-09	325.4	324.8	2.3	0.5 <sup>2</sup>

#### Notes:

1. Depths are relative to ground surface.
2. End of borehole/terminated in this layer.

The composition of this native material generally consisted of sandy gravel or gravelly sand with trace to some silt, trace clay, and trace to some organic material. The layer was generally dark brown/brown to grey in colour and wet. The SPT “N” values obtained within this layer ranged from about 16 to 35 blows per 0.3 m penetration to 10 blows per 0.08 m penetration, suggesting that this layer was compact to dense in compactness.

Laboratory testing performed on selected samples consisted of two (2) moisture content tests and three (3) grain size distribution tests. The test results are as follows:

#### Moisture Content:

- 11% to 18%

#### Grain Size Distribution:

- 22% to 63% gravel
- 29% to 60% sand
- 6% to 18% silt and clay

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 6 in Appendix D.

#### 1.5.1.7 Bedrock

Bedrock was encountered below the native silt / sandy silt / silty sand soils in boreholes BH22-01 to BH22-03 and below the clay layer in BH22-05, as proved by coring. The depth and elevation of bedrock encountered at these boreholes as well as the length of bedrock cores are listed in Table 1.6. Photographs of rock cores are included in Appendix E. The depth and elevation of bedrock in other boreholes are inferred at the auger/SPT refusal.

Table 1.6. Depth and elevation of bedrock surface

Borehole	Depth Below Ground Surface (m)	Elevation (m)	Length of Bedrock Cores (m)
Based on Bedrock Coring			
BH22-01	3.6	325.6	3.1
BH22-02	3.6	327.6	1.6
BH22-03	3.2	330.3	1.7
BH22-05	2.1	324.4	1.5
Based on Refusal			
BH22-04	1.1	324.1	N/A
BH22-06	1.8	327.1	N/A
BH22-07	2.3	322.7	N/A
BH22-08	2.2	323.1	N/A
BH22-09	2.8	324.8	N/A

Based on the bedrock NQ cores (~ core diameter 47 mm) recovered, the bedrock at the site consists of granodiorite composition. In general, the rock samples are described as grey to pink/white in colour, fine to medium grained, and severely fractured to very sound. The Rock Quality Designation (RQD) measured on the core samples typically ranged from approximately 45% to 100%, indicating a rock mass of poor to excellent quality, but mostly good to excellent

quality. The total core recovery (TCR) ranges from 73% to 100%. It should be noted that sloping bedrock was observed at the site, decreasing in elevation from east to west and south to north.

## 1.6 Groundwater and Surface Water Conditions

The groundwater level was observed in the boreholes during and upon completion of their drilling. The groundwater levels measured in piezometers installed in boreholes BH22-04, and BH22-06 to BH22-09 are shown on the borehole logs and are presented below in Table 1.7. Groundwater levels were not observed in BH22-1 to BH22-3 and BH22-5 in open hole prior to rock coring and in installed piezometers since the screen was installed in bedrock. However, based on the moisture content of recovered soil samples in these boreholes the groundwater level in the existing embankment was estimated to be at the top of clay layer (i.e., observed saturated clay).

Table 1.7. Groundwater levels encountered at the site

Borehole No.	Date Measured	Ground Surface Elevation (m)	Groundwater Depth <sup>1</sup> /Elevation (m)
Groundwater Measured in Piezometer			
BH22-04	October 6, 2022	325.1	0.1/325.0
BH22-06	October 6, 2022	328.8	0.9/327.9
BH22-07	October 6, 2022	325.0	0.1/324.9
BH22-08	October 6, 2022	325.3	0.4/324.9
BH22-09	October 6, 2022	327.7	0.02/327.7
Groundwater Estimated Based on Measured Moisture Content			
BH22-01	October 4, 2022	329.1	1.2/327.9
BH22-02	October 3, 2022	331.2	0.9/330.2
BH22-03	October 4, 2022	333.5	1.5/331.9
BH22-05	October 7, 2022	326.5	0.1/326.4

Note:

1. Depths are relative to ground surface.

The water level in the Black River was measured to have an elevation of 324.4m at the time of the site investigation in October 2022. Within the swampy area at the north portion of the site near the Black River, water was observed ponding on the ground surface. Seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year and lower levels during drier periods

## 1.7 Chemical Analysis

One soil sample was selected for a chemical analysis, and it was sent via courier in a secure cooler under chain of custody, to BV Labs (formerly Maxxam Analytics Inc.), a CALA-certified and accredited laboratory in Mississauga, Ontario. The analytical laboratory results are presented in Appendix F and are summarized in Table 1.8 below.

*Foundation Investigation and Design Report  
Embankment Instability on Highway 658  
Agreement No. 6021-E-0019, Assignment No. 5  
Date: December 20, 2022*

Table 1.8. Corrosivity chemical analysis

Sample Identification	pH (unitless)	Soluble Chloride (ppm)	Soluble Sulphate (ppm)	Resistivity (ohm-cm)	Conductivity (mS/cm)
BH22-05 (SS3A)	7.26	190	<400	2100	0.48

## 2 ENGINEERING DISCUSSION & RECOMMENDATIONS

### 2.1 General

This section of the report provides an interpretation of the factual data from Part 1 of this report to determine the mechanisms for slope instability along Highway 658 in Redditt (Latitude: 49.975717°; Longitude: - 94.393332°), Ontario, the Ministry of Transportation (MTO) Northwestern Region. It provides also geotechnical design recommendations for potential remedial work. The geotechnical assessment and recommendations are based on the information presented by MTO and the factual data obtained from the boreholes advanced during the current investigation at the site performed by EXP. The compiled factual data for this project site is presented in Part I- Foundation Investigation Report of this report. Previous investigations by others as noted in this report were used to aid in assessments.

The interpretation and recommendations provided are intended solely to permit stability analyses and design remedial measures. Comments on construction are only provided to highlight issues that could affect the design. Contractors bidding on the works should make their own assessments of the factual data and how it might affect construction means and methods, scheduling and the like.

It was reported by MTO that slope instability occurred along the west side of Highway 658 in Redditt just south of the Black River bridge (Latitude: 49.975933°; Longitude: - 94.393022°) in late June of 2022. The recent slope instability occurred immediately north of a past slope instability in the area that occurred in 2014. EXP conducted a site review in July 2022 as a requested emergency response from the MTO for this project site. The factual and photographic documentation are provided in the site review memorandum which is the part of this report. To determine subsurface conditions, examine the causes of instability and recommend the necessary remedial measures for the slope on Highway 658, an investigation was commenced in October 2022.

This part of the report provides geotechnical design parameters that may be required in accordance with the latest edition of the *Canadian Highway Bridge Design Code (CHBDC) (CAN/CSA-S6-19)*, the *Canadian Foundation Engineering Manual (CFEM) (2006)*, the *Guideline for MTO Foundation Engineering Services, Version 02 (October 2020)*, the *National Building Code of Canada (2020)*, the *MTO Terms of References* and generally accepted good practice. The following subsections present the assessment of the slope instability at Highway 658 and results of slope stability analyses. Construction strategies for remediation are discussed including guidance regarding stability and mitigation or control measures that would be required for rectification of the problem considering longer term approaches and practical constraints.

### 2.2 Assessment of Existing Conditions

#### 2.2.1 Background

The site of investigation on Highway 658 is located just south of the Black River bridge in Redditt, Ontario (Latitude: 49.975933°; Longitude: - 94.393022°). The highway is a two-lane road having a general direction of north-south. High ground and a residential area are south of the site. The general topography of Highway 658 has a steep downward gradient along the roadway surface from south to north. The area is characterized by steeply sloping bedrock overlain by soil deposits which consist of layers of loose fluvio-lacustrine silty sand and soft clays.

MTO reported that the roadway experienced slope instabilities in 2014 and 2022. The 2014 slope instability occurred immediate south of the recent 2022 slope instability. The MTO reviewed and photographed the 2022 slope

instability on June 27, 2022. This slope instability was the focal point of the current site investigation including the site reconnaissance conducted on July 7, 2022 by EXP as an emergency response measure requested by MTO.

The MTO provided the following documents related to the previous investigation in the vicinity of the project site and the 2014 slope instability record:

- “Soils and Foundation Report, Proposed Realignment of Black River Bridge, Redditt, Ontario”, GEOCREs No. 52E-22, prepared by Morton, Dodds & Partners Limited in October 1978 (Morton 1978), and
- “Highway 658, Redditt, Highway Embankment Instability at Lot 116, File Number 2014-01309”, Memorandum prepared by the Ministry of Transportation on July 15, 2014 (MTO 2014).

Based on these documents and observations at the site during the current investigation, the 2014 slope instability area appears to have been remediated and/or stabilized by rough grading/placement of fill soils, coupled with the removal of the previous residential development (i.e., existing house, landscaping and any out-building structures), since the 2014 documentation of this slope instability event (MTO 2014).

### 2.2.2 Site Observations

EXP’s senior geotechnical engineer conducted a visual site reconnaissance of the June 2022 slope instability on July 7, 2022 at the request of the MTO as an emergency response measure. The factual and photographic documentation and visual review of the 2022 slope instability by EXP has been detailed in the technical memorandum “Site Review Geotechnical Commentary for Highway 658 Slope Instability (2022), Redditt, Ontario”, submitted to MTO on July 26, 2022. The memorandum is attached in Appendix I.

The 2022 slope instability was observed along the west shoulder of Highway 658, south of the Black River bridge and directly north of a previously documented slope instability which occurred in 2014 (Photo 1 in Appendix I). The followings visual observations on the roadway, slope, embankment, vegetation, adjacent structures such as power poles and subsurface conditions are noted in the technical memorandum (Appendix I):

- Slope instability length  $\approx 26.0$  m (from Sta. 21+031 to Sta. 21+057); aligned north-south parallel to Highway 658 (Photo 1 in Appendix I);
- Slope height  $\approx 3.5$  m to  $\approx 5.0$  m (Photo 2 in Appendix I);
- Slope instability  $\approx 0.3$  m to  $\approx 1.2$  m into Highway 658 shoulder and southbound lane (Photo 1 in Appendix I);
- Head scarp depth  $\approx 0.2$  m to  $\approx 1.2$  m, increasing in depth from south to north (Photo 3 in Appendix I);
- Existing guardrail and supports bent and twisted with down slope movement (Photo 1 in Appendix I);
- Large tension cracks with variable apertures and depths. Tension cracks were observed to the west of Highway 658 within the slumped soils, approximately 5.0 m to 6.0 m west of the exposed head scarp. These tension cracks ranged from 0.3 m to upwards of 1.0 m in depth with apertures upwards of 0.5 m in width and drops of 0.1 m or greater across the apertures (Photo 4 and Photo 5 in Appendix I); and
- Existing embankment slopes to the west as steep as 1H:1V at leading edge of slope/instability (Photo 2 and Photo 6 in Appendix I).
- Various trees and shrubs were observed to exhibit a downslope lean, consistent with the down slope movement of the slope instability (Photo 7 in Appendix I).

- Two existing power poles were observed to be in close vicinity of the 2022 slope instability. These power poles are just beyond the northern and southern extents of the 2022 slope instability and exhibit a lean down slope (Photo 8 in Appendix I).
- A pre-existing gravel-treated roadway patch/repair was observed along the west edge of the southbound lane and shoulder, fully encompassing the area of instability which had slumped down slope to the west and a portion of the existing roadway edge above the exposed head scarp. The remaining width of the pre-existing treated gravel patch/repair directly above the fresh head scarp ranged from 0.6 m to 0.8 m in width (Photo 10 in Appendix I). This portion of the intact roadway and the surface of the slumped soils below the head scarp consisted of what appears to be a treated gravel surface (likely oil emulsion-based tack coat), not a paved asphalt surface, suggesting that a previous roadway repair was undertaken within the current instability area prior to the recent slope movement occurring. This may have been a result of initial slope movements preceding the recent slope instability; however, EXP has not been provided any historic information associated with this observed area of previous roadway repair.
- There were no signs of instability observed along the east shoulder or ditch line of Highway 658 across from the recent slope instability, only ditch erosion east of asphalt edge from surficial drainage flow downward from south to north towards the Black River floodplain (Photo 11 in Appendix I).
- The surficial soil conditions assessed using a shovel to expose soils near the head scarp along the roadway embankment (Photo 12 and Photo 13 in Appendix I) were: (i) Asphalt (75 mm thickness) followed by the layers of gravelly sand, trace cobbles, moist to wet, and brown in colour (below asphalt to clay subgrade) and clay, some silt, trace sand, medium plastic, moist to wet, very soft with pocket penetrometer readings of less than 0.25 kg/cm<sup>2</sup>, and grey in colour (approximately 2.0 m below roadway). No Groundwater seepage observed.
- The surficial soils near the toe of the slope instability were visually observed to consist of a wet to saturated organic layer over a silt/clay mix.

EXP's senior engineer also briefly inspected and addressed the condition of 2014 slope instability area in the memorandum. It is noted that the overall 2014 slope instability area was covered with mature vegetation, inclusive of tall grasses/weeds, shrubs and small seedling trees. The 2014 slope instability area consisted of variable grades (as steep as 1H:1V with short vertical drops of less than 0.3 m to as flat as 3H:1V near the existing slope toe), sloping down with short drops from the Highway 658 roadway surface down to what is assumed to be the toe of the slope instability or toe of previous fill soil placement (Photo 15 in Appendix I). Rockfill placement as a slope armoring mechanism was observed at the leading edge of the 2014 slope instability after remedial grading was assumed to be undertaken (Photo 16 in Appendix I). No recent signs of slope movement were observed within the overall 2014 slope instability area; however, EXP was not provided any remediation photos after the 2014 slope instability as a comparison to its existing condition as of July 7, 2022. However, pavement stress cracks were observed within an area of previous asphalt repair along Highway 658 directly adjacent to the known area of the 2014 slope instability (Photo 17 in Appendix I). These stress cracks did not appear to be fresh and are not necessarily indicative of any slope movement since 2014, as they may only be associated with the previous roadway repairs (base/subbase gravel and asphalt patching).

### 2.2.3 Assessment of Causes of Slope Instability

The general topography of Highway 658 has a steep downward gradient along the roadway surface from south to north along the section of roadway which has experienced slope instabilities in 2014 and 2022. There is very little surficial drainage control except for the crown of the roadway which directs surface water over the embankment sides and/or ditch lines. As well, the existing west roadway embankment side slopes appear to be very steep (as steep as 1H:1V in localized sections) with mature vegetation growth in the vicinity of the 2022 slope instability.

Based on EXP's visual observations of the 2022 slope instability, it initially appears that the west side of the roadway embankment was inundated with water at the top of the roadway embankment from surficial drainage along the roadway (e.g., heavy spring precipitation events, snow melt, etc.), coupled with flood water rise along the Black River floodplain area at the embankment toe.

The overall slope instability along the west side of Highway 658 was visually observed to be a wedge of soil that has slumped down upwards of approximately a meter in elevation from roadway surface and slid out westward towards the toe of the failure mass. There was no pronounced soil buckling visually observed along the overall toe of the slope instability, as it appeared the failure mass predominately bulged out mid-slope, as shown on the drawings in Appendix B. Based on the site observations and the results of current geotechnical investigation, potential contributing causations of the 2022 slope instability are as follows, as each of these factors either increase the "driving" forces or reduce the "resisting" forces of global stability:

- Poor surficial drainage resulting in saturation of the west roadway embankment sand fill and clay subgrade soils, increasing the weight of the embankment soils ("driving" force ↑);
- Saturation of the clay subgrade soils at the embankment toe from flood water rise, increasing the pore water pressures of the clay subgrade soils near the slope toe ("resisting" force ↓);
- Steep roadway embankment geometry ("driving" force ↑); and
- Weak clay subgrade soils ("resisting" force ↓).

The aforementioned factors are preliminary and are not listed in any particular order of effect, as it is difficult to determine the direct impact of each from a visual site reconnaissance. The culminating effects of these, and potentially other contributing factors (unknown), are intertwined and have resulted in the observed slope instability of the roadway embankment.

Global slope stability analyses for the west slope of the embankment were performed as a part of this assessment and the results are presented in a subsequent section.

## **2.2.4 Slope Stability Analyses**

### **2.2.4.1 General**

Based on the results of the geotechnical investigation and available site geometry, slope stability assessments of existing slope before and after the 2022 failure have been performed. A potential failure surface was identified based on the visible head scarp and zone of accumulation, and then, it was analyzed using the back analyses assuming its factor of safety (FOS) close to one (1). The objective of these analyses was to identify subsurface conditions (i.e., soil parameters and groundwater levels) associated with the subsequent slope instability. The slope stability analyses have also been performed for the current slope geometry and repaired slope using different remedial options.

### **2.2.4.2 Analysis Methodology**

Slope stability analyses were performed using the Morgenstern-Price method developed on the basis of limit equilibrium. The SLOPE/W computer program developed by GeoSlope International (Geostudio 2018 version 9.0.2.15352) was employed for computation.



First, stability assessment was performed using the slope profile in the area outside of the 2022 failure zone (i.e. South Section B-B' at Sta. 21+021) and assumed geometry within the zone failure (i.e. Central Section C-C' at Sta. 21+043 and North Section D-D' at Sta. 21+051). Section C-C' and Section D-D' are perpendicular to the road while Section B-B' was developed along the repaired 2014 failure slope. To assess the critical soil parameters and groundwater conditions which were associated with the subsequent slope instability, sensitivity analyses varying undrained and drained clay parameters and groundwater levels were performed. Both total stress (undrained conditions) and effective stress (drained conditions) analyses for a short term and long-term assessment of the slope stability, respectively, were performed.

Stability assessments were subsequently performed on the profile defined by recent topographic survey data provided on MTO's AutoCAD drawing at three sections: Central Section C-C' at Sta. 21+043, North Section D-D' at Sta. 21+051 and North Section E-E' at 21+050. For the stability assessment of the existing slope, North Section E-E' was developed along the existing failure slope which was assessed as the most critical (i.e., the steepest slope). The other two sections were developed perpendicular to the road as mentioned before. Beside the drained and undrained conditions, both static and seismic conditions were considered in the analyses.

#### **2.2.4.3 Soil Properties and Groundwater Conditions**

The stratigraphy and groundwater conditions at the site were developed based on the results of the geotechnical investigation presented in Part I - Foundation Investigation Report and available AutoCAD drawings. The soil parameters were generally estimated based on the results of field and laboratory tests. The parameters of disturbed clay were estimated based on the results of back-analyses for the west slope assuming its factor of safety of one (1) and failure within the possible shear zone.

Results of all in-situ and laboratory tests available are summarized on Figures F.1 in Appendix F. The graphs show the distributions of following soil properties with elevation: (a) undrained shear strength ( $C_u$ ) measured with a field vane; (b) water content and Atterberg Limits; and (c) measured N-values. As can be seen in Figure F.1 (a), the undrained shear strength ( $C_u$ ) of the native clayey soil was measured to be between 27 kPa and 66 kPa beyond the embankment footprint and approximately 100 kPa below the embankment. For the slope stability analyses, the undrained shear strength ( $C_u$ ) of the native clayey soil was estimated to be two values to assess its effect on the factor of safety (FOS). The lower value of between 6 to 10 kPa was adopted as a representative strength value of disturbed/remolded clay within the failure zone to evaluate the factor of safety (FOS) for the existing slope if the total stress condition is considered, while an undrained shear strength ( $C_u$ ) of 20 kPa was used for native undisturbed clay. In addition, since the sensitivity of clay is measured to be between 2 and 7, it is reasonable to assume that its disturbed/remolded undrained shear strength ( $C_{uR}$ ) is about 6-10 kPa (i.e., within the shear zone). For the effective stress analyses, the effective friction angle of  $28^\circ$  was adopted for the clay.

Table 2.1 summarizes soil parameters used in the slope stability analyses performed for this project.

Table 2.1 Soil properties used in slope stability analyses

Soil Type	Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	Undrained Shear Strength, $C_u$ (kPa)	Effective Stress Parameters	
			Friction Angle $\phi'$ (°)	Cohesion $c'$ (kPa)
Sand and Gravel Fill (Compact)	21	-	32	0
Native Clay (Firm to Very Stiff)	18	20	28*	0
Native Sand/Silt (Loose to Compact)	19	-	30	0
Shear Zone (Disturbed/Remolded Clay)	18	6 to 10	28*	0

Note:

\*-Assumed to be same.

Based on the moisture content of recovered soil samples and measurements in the installed piezometers the groundwater level in the existing embankment and beyond it was estimated to be at the top of clay layer (i.e., saturated clay), and that water level was used in the slope stability analyses. However, to assess the effect of groundwater levels on the stability of the slope, a sensitivity analysis was performed for two water levels: (a) at the top of clay layer (i.e., High GWL) and (b) at the bottom of clay layer (i.e., Low GWL).

#### 2.2.4.4 Seismic Properties

Seismic characterization of the site must be compliant with the Canadian Highway Bridge Design Code (CHBDC - CAN/CSA-S6-19). The potential for seismic loading must be considered for design in accordance with Section 4.4 of the CHBDC with respect to soil conditions encountered at the site. Table 4.1 in CHBDC (see Clause 4.4.3.2) shows site classification for seismic site response based on soil average properties in top 30 m. The borehole information shows the presence of native clay/silty clay and silt/sandy silt/silty sand to sand below the fill, followed by bedrock at the approximate depth of 3 to 4 m below the ground surface. Based on these soil characteristics, the site class for this site is estimated to be Class "C" according to Table 4.1.

From the Natural Resources Canada website, 2020 NBC seismic hazard values obtained using the site location coordinates (49.976°N, -94.393°W) and the damped reference spectral accelerations for the project site are  $S_a(0.2)=0.0982g$ ,  $S_a(0.5)=0.0582g$ ,  $S_a(1.0)=0.0271g$ ,  $S_a(2.0)=0.0106g$  and the reference peak ground acceleration (PGA) is  $0.0485g$  ( $g$  = acceleration due to gravity  $-9.81 \text{ m/s}^2$ ). These values are associated with an earthquake having 2% probability of exceedance in a 50-year period for Site Class C as shown on the GSC seismic hazard calculation data sheet for this site attached in Appendix F.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class and the reference peak ground acceleration ( $PGA_{ref}$ ). For Seismic Site Class C, a  $PGA_{ref}$  with a 2% probability of exceedance in 50 years of  $0.8 * 0.0485g$  and a  $F(PGA)$  of 1.00 as per Table 4.8 of the CHBDC (S6-19) can be considered in the seismic analyses.

#### 2.2.4.5 Stability Results of Original Slope Before the 2022 Failure

The slope stability analyses were performed for the original slope of Highway 658 before the 2022 failure which geometry was defined based on in the available documentations including the currently provided MTO's survey data. The analyses were performed for three sections (South Section at 21+021, Central Section at Sta. 21+043 and North Section at Sta. 21+051) assuming the groundwater levels at the top of clay layer (i.e., High GWL). For two critical sections within the 2022 failure zone (Central Section at Sta. 21+043 and North Section at Sta. 21+051) the analyses were performed assuming the groundwater level at the bottom of clay layer as well (i.e., Low GWL). As noted before, the objective of these analyses was to identify soil and groundwater properties at the time of failure by back-calculating the FOS =1 along the potential failure surface between the head scarp and toe of failure. The results of these analyses were shown on Figures G1 to G10 in Appendix G for effective stress and total stress conditions. The results are summarized in Table 2.2 below.

Table 2.2. Summary of slope stability analyses of original slope before the 2022 failure

Section	Method of Analyses	GWL Condition	Soil Conditions	Min FOS	Figure
South Section B-B' @ Sta. 21+021	Static	High GWL	Drained Conditions	1.0	Figure G1
			Undrained Conditions	1.0	Figure G2
High GWL		Drained Conditions	0.9	Figure G3	
		Undrained Conditions	0.9	Figure G4	
Low GWL		Drained Conditions	1.6	Figure G5	
		Undrained Conditions	0.9	Figure G6	
Central Section C-C' @ Sta. 21+043		High GWL	Drained Conditions	0.9	Figure G7
			Undrained Conditions	1.0	Figure G8
		Low GWL	Drained Conditions	1.4	Figure G9
			Undrained Conditions	0.9	Figure G10
North Section D-D' @ Sta. 21+051	High GWL	Drained Conditions	0.9	Figure G7	
		Undrained Conditions	1.0	Figure G8	
	Low GWL	Drained Conditions	1.4	Figure G9	
		Undrained Conditions	0.9	Figure G10	

Based on these results, it is estimated that the effective strength parameters of clay along the potential circular slip surface identified based on observations on the site (i.e. head scarp and soil accumulation at the toe of failure) with FOS~1 are  $\phi' = 28^\circ$  and  $c' = 0$  kPa, while the total strength parameter of disturbed clay (i.e. undrained shear strength) is estimated to be between 6 to 9 kPa. The undrained shear strength of undisturbed clay beyond the zone of failure is conservatively estimated to be approximately 20 kPa. The results of sensitive analyses with two different groundwater levels suggest that the more critical condition for slope stability is when the groundwater level is high

causing that the existing clay is completely saturated. This condition of clay was confirmed by visual observation at the site as well as with the results of current geotechnical investigation.

#### 2.2.4.6 Stability Results of Existing Slope After the 2022 Failure

The slope stability analyses were also performed for existing slopes within the zone of 2022 failure using its geometry provided in the AutoCAD drawing (i.e., Central Section C-C' at Sta. 21+43 and North Section D-D' at Sta. 21+051 and North Section E-E' at Sta. 21+050) and soil conditions estimated in Section 2.2.4.4 (i.e., full clay saturation condition). The results of these analyses are listed in Table 2.3 below. Figures G11 to G22 in Appendix G show the results of stability analyses for effective stress and total stress analyses as well as for static and seismic conditions. The global stability of the slope under seismic conditions was performed applying a peak horizontal ground acceleration value of 0.0485 g suggested by the 2020 NBC for this location.

Table 2.3 Summary of results of slope stability analyses for the existing failed slope

Section	Method of Analyses	GWL Condition	Soil Conditions	Min FOS	Figure
Central Section C-C' @ Sta. 21+043	Static	High GWL	Drained Conditions	1.0	Figure G11
			Undrained Conditions	1.2	Figure G12
	Seismic		Drained Conditions	0.8	Figure G13
			Undrained Conditions	1.0	Figure G14
North Section D-D' @ Sta. 21+051	Static		Drained Conditions	1.0	Figure G15
			Undrained Conditions	1.0	Figure G16
	Seismic		Drained Conditions	0.9	Figure G17
			Undrained Conditions	0.9	Figure G18
North Section E-E' @ Sta. 21+050	Static		Drained Conditions	1.0	Figure G19
			Undrained Conditions	1.2	Figure G20
	Seismic		Drained Conditions	0.9	Figure G21
			Undrained Conditions	1.1	Figure G22

Based on the results of slope stability analyses for existing slopes, it appears that the existing west slope of Highway 658 within the failure zone is on the verge of instability.

## 2.3 Remediation Alternatives

When dealing with the embankment slope instability, two approaches are generally possible: (i) movement of highway away from the failure zone, in this case toward the east, and (ii) rectification of the existing embankment with slope stability enhancements. In this project, movement of the highway alignment sufficiently to the east such that it is clear of stability issues related to the slope on the west side of Highway 658, is impractical considering that this road is an approach road for the existing bridge. Therefore, some other remediation measures on the existing embankment with slope stability enhancement were considered for this area, such as:

OPTION 1: Excavation of disturbed soil up to bedrock and installation of shear key

OPTION 2: Construction of toe berm and slope flattening

OPTION 3: Replacement of disturbed soil with lightweight fill and slope flattening

OPTION 4: Installation of anchors embedded into bedrock

OPTION 5: Installation of timber piles

OPTION 6: Ground improvement techniques (e.g., Controlled Modulus Columns (CMC), Vibro Stone Columns/Aggregate Piers, Soil Mixing or Chemical Grouting)

Table 2.4 lists these remediation measures considered for this project along with their advantages and disadvantages. Sketches attached in Appendix H graphically illustrates the suggested remediation approaches.

Table 2.4. Remediation alternatives considered for the site

Remediation Alternatives	Advantages	Disadvantages	Relative cost	Ranking
<b>OPTION 1:</b> Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key	<ul style="list-style-type: none"> <li>• Improve stability of slope by eliminating disturbed soil and increasing the resistance</li> <li>• Improve drainage of slope</li> <li>• Cost effective</li> <li>• The highway does not need to be closed</li> </ul>	<ul style="list-style-type: none"> <li>• Require significant excavation of soil</li> <li>• Require material for shear key</li> <li>• Overhead utility lines must be protected or temporary replaced</li> </ul>	<ul style="list-style-type: none"> <li>• Less costly than other options</li> <li>• Less costly construction due to shorter construction time, but cost of transportation of rockfill has to be considered.</li> </ul>	1 Recommended and preferred
<b>OPTION 2:</b> Construction of Toe Berm and Slope Flattening	<ul style="list-style-type: none"> <li>• Stabilize the slope by increasing resistance forces at the toe</li> <li>• Improve erosion issues</li> <li>• Do not require disturbance of traffic flow</li> </ul>	<ul style="list-style-type: none"> <li>• Do not eliminate disturbed soil</li> <li>• Requires significant size of berm to ensure the required FOS (~ 7 m wide and 4 m high with 2H:1V slope)</li> <li>• Additional fill to be placed in the toe and flood plain</li> </ul>	<ul style="list-style-type: none"> <li>• More costly than shear key and timber pile options due to significant size of berm.</li> </ul>	3 Recommended

Remediation Alternatives	Advantages	Disadvantages	Relative cost	Ranking
<b>OPTION 3:</b> Replacement of Disturbed Soil with Lightweight Fill and Slope Flattening	<ul style="list-style-type: none"> <li>Stabilize the slope by eliminating disturbed soils and reducing the driving forces</li> <li>Local experience is available</li> </ul>	<ul style="list-style-type: none"> <li>Requires a roadway protection system to maintain traffic at the site</li> <li>Disturbance of traffic flow</li> <li>Excavation of large amount of soil</li> <li>Need expensive EPS</li> <li>Overhead utility lines must be protected or temporary replaced</li> </ul>	<ul style="list-style-type: none"> <li>More costly than other options due to expensive EPS</li> <li>Higher cost also due to TPS and French drain</li> </ul>	4 Not recommended
<b>OPTION 4:</b> Installation of Anchors Embedded into Bedrock	<ul style="list-style-type: none"> <li>Stabilize the slope by reinforcing disturbed soil</li> <li>Do not require change in slope geometry</li> <li>Minimum earthwork</li> <li>The highway does not need to be closed</li> <li>Minimum disturbance of traffic flow</li> </ul>	<ul style="list-style-type: none"> <li>Require significant number of anchors</li> <li>Installation may be difficult</li> <li>Require construction of working platform</li> <li>In-situ design test or proof test required</li> <li>Overhead utility lines must be protected or temporary replaced</li> </ul>	<ul style="list-style-type: none"> <li>More expensive than other options due to longer construction time for construction of working platform, embedment of anchors and in-situ tests,</li> <li>Higher cost due to significant number of anchors and French drain</li> </ul>	5 Not recommended
<b>OPTION 5:</b> Installation of Timber Piles	<ul style="list-style-type: none"> <li>Stabilize the slope by reinforcing disturbed soil</li> <li>The highway does not need to be closed</li> <li>Local experience is available</li> </ul>	<ul style="list-style-type: none"> <li>Require significant number of timber piles</li> <li>Requires a roadway protection system to maintain traffic at the site</li> <li>Disturbance of traffic flow</li> <li>Require excavation of soil</li> <li>Overhead utility lines must be protected or temporary replaced</li> </ul>	<ul style="list-style-type: none"> <li>More expensive than shear key option due to longer construction time (need construction of working platform and time for driving piles)</li> <li>Higher cost due to significant number of timber piles, TPS and French drain</li> </ul>	2 Recommended

Remediation Alternatives	Advantages	Disadvantages	Relative cost	Ranking
<b>OPTION 6:</b> Ground Improvement Techniques	<ul style="list-style-type: none"> <li>• Improve some soil properties</li> <li>• Minimum excavation required</li> <li>• A sustainable method</li> </ul>	<ul style="list-style-type: none"> <li>• It is not effective for lateral soil movement; requires a central steel reinforcing bar</li> <li>• Expensive mobilization of equipment</li> </ul>	<ul style="list-style-type: none"> <li>• More expensive than other options due to longer construction time and high cost of mobilization of equipment</li> </ul>	6 Not recommended

The purpose of these remedial measures and installation of reinforcing elements is to increase the safety factor to an acceptable level. For road embankments, the recommended minimum factors of safety are 1.5 and >1.1 under static and seismic conditions, respectively. The global stability of the slope with these reinforcing elements as well as the stability at the toe of slope under static and seismic conditions has been analyzed and the results are presented through Figures G19 to G46 in Appendix G and summarized in Table 2.5 below.

Table 2.5. Summary of results of slope stability analyses on slope subjected to remedial works

Remedial Work	Method of Analyses	Failure Mode	Soil Conditions	Min FOS	Figure
OPTION 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key (Central Section C-C' at Sta. 21+043)	Static	Global Failure	Drained	1.9	Figure G23
			Undrained	2.0	Figure G24
		Toe Failure	Drained	1.5	Figure G25
			Undrained	2.5	Figure G26
	Seismic	Global Failure	Drained	1.6	Figure G27
			Undrained	1.7	Figure G28
		Toe Failure	Drained	1.3	Figure G29
			Undrained	2.1	Figure G30
OPTION 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key (Critical North Section E-E' at Sta. 21+050)	Static	Global Failure	Drained	1.9	Figure G31
			Undrained	2.2	Figure G32
		Toe Failure	Drained	1.6	Figure G33
			Undrained	2.3	Figure G34
	Seismic	Global Failure	Drained	1.6	Figure G35
			Undrained	1.9	Figure G36
		Toe Failure	Drained	1.3	Figure G37
			Undrained	2.0	Figure G38

Remedial Work	Method of Analyses	Failure Mode	Soil Conditions	Min FOS	Figure
OPTION 2: Construction of Toe Berm and Slope Flattening	Static	Global Failure	Drained	1.7	Figure G39
			Undrained	1.3	Figure G40
		Toe Failure	Drained	1.5	Figure G41
			Undrained	1.7	Figure G42
	Seismic	Global Failure	Drained	1.5	Figure G43
			Undrained	1.1	Figure G44
		Toe Failure	Drained	1.3	Figure G45
			Undrained	1.4	Figure G46
OPTION 3: Replacement of Disturbed Soil with Lightweight Fill and Slope Flattening	Static	Global Failure	Drained	1.7	Figure G47
		Toe Failure	Drained	1.6	Figure G48
	Seismic	Global Failure	Drained	1.5	Figure G49
		Toe Failure	Drained	1.4	Figure G50
OPTION 4: Installation of Anchors Embedded into Bedrock	Static	Global Failure	Drained	2.0	Figure G51
		Toe Failure	Drained	1.5	Figure G52
	Seismic	Global Failure	Drained	1.6	Figure G53
		Toe Failure	Drained	1.3	Figure G54
OPTION 5: Installation of Timber Piles	Static	Global Failure	Drained	1.8	Figure G55
		Toe Failure	Drained	1.5	Figure G56
	Seismic	Global Failure	Drained	1.5	Figure G57
		Toe Failure	Drained	1.3	Figure G58

All remedial options considered in Table 2.4 are feasible from the geotechnical perspective. However, some other perspectives such as economical and/or constructability perspectives also influence the decision which option will be chosen. In Table 2.4 the options are ranked considering all three perspectives. Following paragraphs provide the brief discussion of each alternative considered.

#### **Option 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key**

Option 1 involves excavation/removal of the disturbed soil at the failed slope up to the bedrock and construction of rock-filled shear key to intercept the subsurface shear plane/ shear zone to stabilize the slope, as shown on Sketch 1 in Appendix H. The top fill material and approximately 3 m of underlying soft/disturbed clay should be excavated along the embankment from the roadway Sta. 21+025 to Sta. 21+060 (i.e. ~35 m) and then backfilled. The shear key



is recommended to be installed up to bedrock to eliminate any potential deep-seated failure. The results of slope stability shown on Figures G23 to G38 in Appendix G show that the global stability of the embankment slope could be improved by replacing the disturbed soil with the shear key. Therefore, the major advantages of this option are that it eliminates the disturbed/soft soil at the toe and improves the drainage of the slope. The option is relatively inexpensive. Therefore, this option is assessed as the most practical and economical from the geotechnical and constructability perspectives and it was ranked 1. This option will be discussed in more details in the subsequent section.

#### **Option 2: Construction of Toe Berm and Slope Flattening**

Option 2 considers construction of a rockfill berm at the toe of the existing slope and flattening of embankment slope. The berm is recommended to be located at the bottom of the existing slope as shown on Sketch 2 in Appendix H. The results of the slope stability analyses performed on the flattened slope to 4H:1V and with a berm placed at the toe (Figures G39 to G46 in Appendix G) show that the minimum factor of safety can be increased above the acceptable levels applying these remedial measures. The slope stability analyses show that the berm should be approximately 7 m wide and 4 m high with side slope of 2H:1V to be stable in drained and undrained conditions. Actually, the slope stability analyses with this berm show that the minimum FOS could be 1.3 for the global failure mode in undrained condition. Comparing this option with Option 1 this option was also assessed as economical from the constructability perspective, but from the geotechnical perspective it was assessed as less reliable since it does not include replacement of disturbed soil within the failed zone. For the same reason it is ranked 2.

#### **Option 3: Replacement of Disturbed Soil with Lightweight Fill and Slope Flattening**

Option 3 ranked 4 in Table 2.4 considers remediation of this slope failure by replacement of disturbed soil with expanded polystyrene (EPS) lightweight fill to reduce embankment loads and embankment slope flattening to minimum 3H:1V (see Sketch 3 in Appendix H). As shown on the schematic sketch, lightweight fill replacement will require excavation through the existing fill and disturbed native soil up to approximately 2 to 3 m depth. The subgrade soils at the base of the EPS fill placement should be prepared and proof rolled to a minimum of 95% of the SPMDD. Subgrade should be sloped to the west a minimum of 2% to promote drainage. A minimum of 0.3 m of Granular B leveling pad should be placed beneath the EPS. A non-woven geotextile separator (Terrafix 270R or equal) should be placed between the subgrade soils and the granular material. This option would require roadway protection installed behind the head scarp of slope failure to maintain at least a single lane of traffic flow along the current highway embankment. The results of slope stability analyses with lightweight fill (Figures G47 to G50 in Appendix G) show that the minimum factor of safety can be increased above the acceptable levels if a French drain is installed at the toe of the slope to lower/control the groundwater level within the remediated slope. If this option is selected the typical drawing showing installation of EPS in road embankment is attached in Appendix L.

#### **Option 4: Installation of Anchors Embedded into Bedrock**

Anchors and/or soil nails are usually installed in unstable material for the purpose of stabilizing a slope. Soil nails are grouted the full length of the bar allowing the entire bar to develop a bond with the surrounding soil or rock matrix. Anchors differ from soil nails in the grouting of the anchor, loading and subsequent load distribution along the tendon. They are only grouted along a portion of their length, providing for a free stressing zone. Considering the shallow bedrock, installation of anchors was assessed as more effective at this site.

Sketch 4 in Appendix H shows the suggested installation of anchors within the failure zone. In addition, placement of geogrid on the top of slope which an inclination should be minimum 3H:1V is recommended. The slope should be protected from erosion through the use of vegetation, geoweb or shotcrete. The results of stability analyses

shown on Figures G51 to G54 in Appendix G also suggest that a French drain is required to drawdown groundwater at the toe of the slope to achieve the FOS of 1.5. At this site, significant number of anchors will be required to create in-situ “gravity wall embedded into bedrock” to resist creep loading and/or sliding along the bedrock surface. Soil anchors requires triple corrosion protection. The slope stability analyses were performed assuming that the majority of resisting force is derived from the shear mobilized at the interface between the anchors and bedrock. The shear capacity of the steel bar was not included in these analyses. However, the design of anchors could be refined by including the shear capacity of a steel bar and appropriate anchor adhesion values defined by in-situ specific pullout tests. This could result in reduction of number of anchors and its length, as well as in increase in vertical and horizontal spacing between anchors. Such design and execution refinements should be conducted by qualified and experienced contractors/engineers. In addition, it should be noted that installation of anchors requires robust equipment and construction of a working platform. Considering all disadvantages of this option listed in Table 2.4, it was assessed that it is not economical from the constructability perspective; therefore, it was ranked 5.

#### **Option 5: Installation of Timber Piles**

Option 5 suggests reinforcement of the failure zone with timber piles as shown in Sketch 5 in Appendix H. As shown on the schematic sketch, the 0.3 m diameter timber piles having 1.5 m spacing are recommended to be installed at the excavated surface within the footprint of the failure zone needed as a working platform. All piles should be driven to refusal. This option would require roadway protection installed behind the head scarp of slope failure to maintain traffic flow along the current highway embankment. The results of stability analyses shown on Figures G55 to G58 in Appendix G suggest that a French drain is required at the toe of the slope to achieve the required FOS of 1.5 in static condition. This option could be costly since it requires a significant number of timber piles and a roadway protection system. However, since this option stabilize the slope by reinforcing disturbed soil and local experience is available, Option 5 was ranked 2.

#### **Option 6: Ground Improvement Technique**

Ground improvement techniques such as Controlled Modulus Columns (CMC), Vibro Stone Columns/Aggregate Piers, Soil Mixing or Chemical Grouting might be used to control and reduce settlement and increase bearing capacity in soft or loose soils. In addition, these sustainable methods require less excavation. However, these methods are not effective for lateral soil movement what is the case at this site. To prevent lateral movements, a central steel reinforcing bar is required to be installed in the column/pier, which significantly raises their price. Therefore, this option was ranked as a least viable option among the considered options.

## **2.4 Recommended Remediation Measure**

Excavation of disturbed soil within the failure zone up to bedrock and installation of shear key is the recommended remedial measure for this site. Sketch 1 attached in Appendix H graphically illustrates the recommended remedial approach for the slope. As shown on the sketch it is recommended that the top loose fill and approximately 3 m of underlying disturbed/soft clay are excavated along the toe of failed embankment from Sta. 21+025 to Sta. 21+060 (i.e. ~35 m) and then backfilled by rockfill. The shear key option is an effective method of dealing with the likely mode of failure (i.e., a deep-seated failure). It is relatively simple to construct and inexpensive.

For the stabilization work using a shear key, the procedures listed below should be applied. It is recommended that a NSSP with these procedures be included in the Contract Documents. The NSSP with suggested wording is attached in Appendix J as an example.

1. Excavation shall be done in sections parallel to the existing highway not longer than 3 m as shown on Sketch 1 in Appendix H
2. Excavation shall begin at the head scarp of the 2022 failure and be tapered down to the bottom of excavation (i.e. up to bedrock). Temporary excavation side slopes through loose fill and disturbed clay shall be in accordance to the latest edition of Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The native clay above the groundwater level would be classified as Type 3 soil, while below the groundwater it would be classified as Type 4 soil.
3. Excavation is approximately 3 m deep and 5 m wide at the bottom.
4. Excavation must be backfilled with rockfill immediately after excavation. Do not keep the excavation opened under any circumstances.
5. If the groundwater level at the site is high, it is recommended that the layer of rockfill shall be deposited at once approximately 0.5 m above the groundwater level and then the shear key using rockfill shall be constructed in accordance with OPSS.PROV 206 and/or OPSS.PROV 902. For rock-filled structure the layers shall not exceed 1.5 m thickness prior to construction. Material in each layer shall be fully compacted prior to the succeeding layer is placed. Each rockfill layer shall be compacted with a tractor bulldozer with a minimum number of complete passes of 6 and the maximum passes of 8. A complete pass shall be defined as 100% coverage of layer surface.
6. Where practical the shear key shall be periodically outleted toward the bottom of the slope or to the river to avoid accumulation of water during heavy rainfall.
7. The granular fill (Granular A or Granular B Type II as per OPSS.PROV 1010) shall be placed on the top of shear key flattening the embankment slope to minimum 3H:1V. A suitable geotextile (according to OPSS.PROV 1860) shall be placed at any contact between granular fill and rockfill. Surface of rockfill has to be chinked. Backfill shall be placed according to OPSS.PROV 206 and compacted according to OPSS.PROV 501.
8. The final lift of embankment fill prior to placing pavement sub-base shall be compacted to 100 % SPMDD. The Granular A base and Granular B sub-base courses (for pavement) shall be compacted to 100% of the material's SPMDD. Before placing any granular fill over the rockfill, proper chinking shall be applied. Alternatively, a suitably robust geotextile can be placed for separation purposes.
9. The final embankment side slopes shall be protected against erosion by surface water runoff as soon as practical after completion of slope grading using a combination of materials in accordance with OPSS.PROV 803 and/or OPSS.PROV 804.
10. Groundwater conditions shall be controlled by maintaining/addressing suitable drains and controlled outlets.
11. During construction the traffic flow shall be mentioned on the NBL of Highway 658.
12. During construction geotechnical monitoring by competent personnel shall be provided.

Slope stability analyses of the slope with implemented remedial measures mentioned above were performed to check the factor of safety under new conditions. The results are shown on Figures G23 to G38 (Appendix G) for two critical slope sections (Section C-C' and Section E-E'). As can be see, the minimum factors of safety for global stability of these slopes were calculated to be between 1.9 and 2.2 depending on soil condition (for static condition), meeting the MTO requirements. Comparing these results with those shown in Figure G11 to G22 which represent the stability condition of the current slope, there is a significant improvement in global stability of the slope. The slope stability of the lower part of the slope (i.e., at the toe of the existing slope) was also checked and the results suggest that the toe slope is also stable (i.e., min FOS is between 1.5 and 2.5 depending on soil condition (for static condition)) due to improvement of drainage of the west side of road by placing the shear key.

The existing highway drainage system including ditches/swales and culverts has to be assessed based on current watershed characteristics and drainage system conditions by hydrologic and hydraulic studies. The hydrologic analysis should show if the existing ditches/swales are adequate to convey surface runoff into the culverts, or their adjustment to provide adequate pavement drainage is required. Ideally drainage on the east side should extend to rock level or be set deep enough to cut-off infiltrated water and seepage from the east. On the west side, it is recommended that the shear key should be periodically outleted toward the bottom of the slope and/or the river to eliminate any accumulation of water during heavy rainfall.

All excavations at this site must be conducted in accordance with the Occupational Health and Safety Act (OHSA) and Regulations for Construction (O. Reg. 213/91). Sand and gravel and gravelly sand fills may be classified as a Type 3 soil above the groundwater table in conformance with the OHSA. The native clay/silty clay/silt/silty sand/sand soils above the groundwater table may be classified as a Type 3 soil and Type 4 soil below the groundwater table. The ingress of surface water must be controlled during excavation.

Since any slope movement or failure in the area could threaten the safety of the vehicles and people using Highway 658, it is recommended that until the stabilization works have been completed, the slope in the failure zone is closely monitored for further movements. The subsequent section provides the risk assessment and recommendations of interim measures.

## 2.5 Risk Assessment and Immediate Measures

The level of risk, timing of events, and the impact on road safety and convenience to the public are difficult to predict with an elevated level of certainty. These include weather conditions, water level fluctuations, traffic and driver variables, implementation of any mitigating action etc. Notwithstanding the preceding comments and based on observations on the site, the following potential immediate, short and long-term risks at the site are assessed and rated:

Table 2.6 Assessment of risk

Term	Potential Risks	Risk Rating
Immediate	<ul style="list-style-type: none"> <li>Continuous movements of the failure head scarp in Highway 648 fills.</li> <li>On-going development of cracks and differential settlement at Highway 658 resulting in safety issues for approaching traffic.</li> <li>Impact loads from heavily laden lumber trucks at embankment, with possible safety implications on two-way traffic.</li> </ul>	<p>Small</p> <p>(Assuming site inspection visit by MTO maintenance staff once weekly and any necessary interventions.)</p>
Short	<ul style="list-style-type: none"> <li>Risks as noted in immediate term above</li> <li>Heavy rainfall event with increased probability of significant and sudden slope failure</li> <li>Erosion at the toe resulting in decreased stability</li> </ul>	<p>Medium</p>

Term	Potential Risks	Risk Rating
Long	<ul style="list-style-type: none"> <li>Risks as listed in immediate and short term above</li> <li>Repetitive heavy rain – get into cracks and “greases” the shear plane causing progressive failure that could be sudden</li> <li>Failure of complete embankment with impact on the bridge approach</li> </ul>	<p>High</p> <p>(Reducing with the nature of any interim mitigation measures)</p>

Remedial options considered in above sections provide the permanent repair/rehabilitation of Highway 658 slope, and it is recommended to implement them as soon as possible. Until then, monitoring the site by maintenance staff at least once weekly for the next weeks to observe any obvious changes in the conditions. The monitoring should consist of survey targets mounted on the guiderails, and/or steel bars driven into the pavement along the crest of the slope. Targets should also set up at approximately at the toe of the failure and at the toe of the slope. These targets should be monitored for lateral movement as well settlement. The results of the monitoring should be continually reviewed, and if significant movements are detected, warning signs should be set up to reduce traffic speed in the area, and/or close the southbound lane, if necessary. Notwithstanding the above, it is possible that slope failures occur suddenly, and the potential cannot be ruled out.

## 2.6 Monitoring Plan

Immediately after repairing Highway 658 it is recommended to install settlement pins to monitor movements of the repaired slope. Settlement pins are recommended to be installed along the slope at three monitoring sections as shown in the attached drawing in Appendix K. As shown on the drawing, four (4) settlement pins are recommended to be installed along each section (i.e., total 12 settlement pins). The drawing shows selected locations of the proposed settlement pins and method of their installation. The settlement pins should be installed on the selected locations after the embankment fill placement.

After their installation, it is recommended that frequency of reading is once a month within a 12-month period. The review and alert levels for the settlement of existing Highway 658 roadway are recommended to be 20 mm and 25 mm, respectively.

### 3 CLOSURE

The recommendations made in this report are in accordance with our present understanding of the project and are provided solely for the team responsible for the design of the works described herein.

We recommend that we be retained to review our recommendations as the design nears completion to ensure that the final design is in agreement with the assumptions on which our recommendations are based and that our recommendations have been interpreted as intended. If not accorded this review, EXP will assume no responsibility for the interpretation and use of the recommendations in this report.

A subsurface investigation is a limited sampling of a site; the subsurface conditions have been established only at the test hole locations. Should conditions at the site be encountered which differ from those reported at the test locations, we require that we be notified immediately in order to assess this additional information and our recommendations, as appropriate. It may then be necessary to perform additional investigation and analysis.

Contractors bidding on or undertaking any proposed work at this site should, relative to the subsurface conditions, decide on their own investigations, if deemed necessary, as well as their own interpretations of the factual results provided herein, so they may draw their own conclusions as to how the subsurface conditions may affect them.

This Foundation Investigation and Design Report has been prepared by Daniel Mroz, M.Eng, EIT, and Silvana Micic, Ph.D., P.Eng. It was reviewed by TaeChul Kim, M.E.Sc., P.Eng. and by Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact. The field investigation was supervised by Kole Pitkanen.

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## REFERENCES

- Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.
- Canadian Standards Association (CSA), 2019. Canadian Highway Bridge Design Code and Commentary on CAN/CSA-S6-19. CSA Special Publication.
- Ministry of Northern Development and Mines, Ontario Geological Survey, 1991. Map 2554: Quaternary Geology of Ontario, West-Central Sheet
- Ministry of Northern Development and Mines, Ontario Geological Survey, 1991. Map 2542: Bedrock Geology of Ontario, West-Central Sheet
- Northern Ontario Engineering Geology Terrain Study, Data Base Map, Rat Portage Bay, 1980. Ontario Geological Survey, Map 5055.
- Ontario Ministry of Transportation, April 2022. Guideline for Foundation Engineering Services, Version 3.0.

### **ASTM International:**

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

### **Ontario Provincial Standard Specifications (OPSS):**

OPSS.PROV 206 Construction Specification for Grading

OPSS.PROV 501 Construction Specification for Compacting

OPSS.PROV 803 Construction Specification for Vegetative Cover

OPSS.PROV 804 Construction Specification for Seed and Cover

OPSS.PROV 902 Construction Specification for Excavating and Backfilling – Structures

OPSS.PROV 1010 Material Specification for Aggregates - Base, Subbase, Select Subgrade, and Backfill Material

OPSS.PROV 1860 Material Specification for Geotextiles

### **Non-standard Special Provisions (NSSP):**

Non-Standard Special Provision for Construction of Shear Key

### **Ontario Water Resources Act:**

R.R.O 1990, Regulation 903 Wells, under Ontario Water Resources Act, R.S.O. 1990, c. O.40

### **Ontario Occupational Health and Safety Act (OHSA):**

Ontario Regulation 213/91 Construction Projects

## LIMITATIONS AND USE OF REPORT

### **BASIS OF REPORT**

This report ("Report") is based on site conditions known or inferred by the geotechnical investigation undertaken as of the date of the Report. Should changes occur which potentially impact the geotechnical condition of the site, or if construction is implemented more than one year following the date of the Report, the recommendations of EXP may require re-evaluation.

The Report is provided solely for the guidance of design engineers and on the assumption that the design will be in accordance with applicable codes and standards. Any changes in the design features which potentially impact the geotechnical analyses or issues concerning the geotechnical aspects of applicable codes and standards will necessitate a review of the design by EXP. Additional field work and reporting may also be required.

Where applicable, recommended field services are the minimum necessary to ascertain that construction is being carried out in general conformity with building code guidelines, generally accepted practices and EXP's recommendations. Any reduction in the level of services recommended will result in EXP providing qualified opinions regarding the adequacy of the work. EXP can assist design professionals or contractors retained by the Client to review applicable plans, drawings, and specifications as they relate to the Report or to conduct field reviews during construction.

Contractors contemplating work on the site are responsible for conducting an independent investigation and interpretation of the borehole results contained in the Report. The number of boreholes necessary to determine the localized underground conditions as they impact construction costs, techniques, sequencing, equipment and scheduling may be greater than those carried out for the purpose of the Report.

Classification and identification of soils, rocks, geological units, contaminant materials, building envelopment assessments, and engineering estimates are based on investigations performed in accordance with the standard of care set out below and require the exercise of judgment. As a result, even comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations or building envelope descriptions involve an inherent risk that some conditions will not be detected. All documents or records summarizing investigations are based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated. Some conditions are subject to change over time. The Report presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, these should be disclosed to EXP to allow for additional or special investigations to be undertaken not otherwise within the scope of investigation conducted for the purpose of the Report.

### **RELIANCE ON INFORMATION PROVIDED**

The evaluation and conclusions contained in the Report are based on conditions in evidence at the time of site inspections and information provided to EXP by the Client and others. The Report has been prepared for the specific site, development, building, design or building assessment objectives and purpose as communicated by the Client. EXP has relied in good faith upon such representations, information and instructions and accepts no responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of any misstatements, omissions,



*Foundation Investigation and Design Report  
Embankment Instability on Highway 658  
Agreement No. 6021-E-0019, Assignment No. 5  
Date: December 20, 2022*

misrepresentation or fraudulent acts of persons providing information. Unless specifically stated otherwise, the applicability and reliability of the findings, recommendations, suggestions or opinions expressed in the Report are only valid to the extent that there has been no material alteration to or variation from any of the information provided to EXP.

#### **STANDARD OF CARE**

The Report has been prepared in a manner consistent with the degree of care and skill exercised by engineering consultants currently practicing under similar circumstances and locale. No other warranty, expressed or implied, is made. Unless specifically stated otherwise, the Report does not contain environmental consulting advice.

#### **COMPLETE REPORT**

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment form part of the Report. This material includes, but is not limited to, the terms of reference given to EXP by its client ("Client"), communications between EXP and the Client, other reports, proposals or documents prepared by EXP for the Client in connection with the site described in the Report. In order to properly understand the suggestions, recommendations and opinions expressed in the Report, reference must be made to the Report in its entirety. EXP is not responsible for use by any party of portions of the Report.

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The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. No other party may use or rely upon the Report in whole or in part without the written consent of EXP. Any use of the Report, or any portion of the Report, by a third party are the sole responsibility of such third party. EXP is not responsible for damages suffered by any third party resulting from unauthorized use of the Report.

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Where EXP has submitted both electronic file and a hard copy of the Report, or any document forming part of the Report, only the signed and sealed hard copy shall be the original documents for record and working purposes. In the event of a dispute or discrepancy, the hard copy shall govern. Electronic files transmitted by EXP have utilized specific software and hardware systems. EXP makes no representation about the compatibility of these files with the Client's current or future software and hardware systems. Regardless of format, the documents described herein are EXP's instruments of professional service and shall not be altered without the written consent of EXP.

## Appendix A – Site Photographs



Photograph 1. 2022 Slope instability on Highway 658, facing north, July 7, 2022



Photograph 2. 2022 Slope instability on Highway 658, facing south, July 7, 2022





Photograph 3. Highway 658 leaning trees at slope instability, facing southeast, July 7, 2022



Photograph 4. Highway 658 east shoulder surficial drainage erosion, facing north, July 7, 2022





Photograph 5. 2022 Slope instability on Highway 658, facing east, October 2022



Photograph 6. General conditions of north portion of site, facing northeast, October 2022





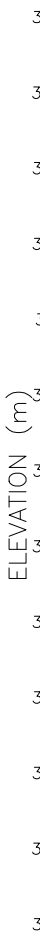
Photograph 7. Drilling BH22-02, October 3, 2022



Photograph 8. Drilling BH22-05, October 6, 2022

## Appendix B – Drawings





LEGEND

- 

	TOPSOIL		SAND		SILTY SAND
	ASPHALT		SILT		GRAVELLY SAND/ SANDY GRAVEL
	FILL		CLAYEY SILT		BEDROCK
	CLAY		SILTY CLAY		

BH No.	ELEV.	NORTHING	EASTING
BH22-01	329.1	5538523.407	204865.727
BH22-02	331.2	5538509.836	204857.182
BH22-03	333.5	5538497.879	204847.554
BH22-04	329.1	5538531.751	204856.898
BH22-05	326.5	5538521.465	204846.100
BH22-06	328.8	5538521.465	204839.744
BH22-07	325.0	5538521.465	204849.832
BH22-08	325.3	5538521.465	204842.991
BH22-09	327.7	5538521.465	204836.969
EXP-BM	326.8	5538521.465	204881.288

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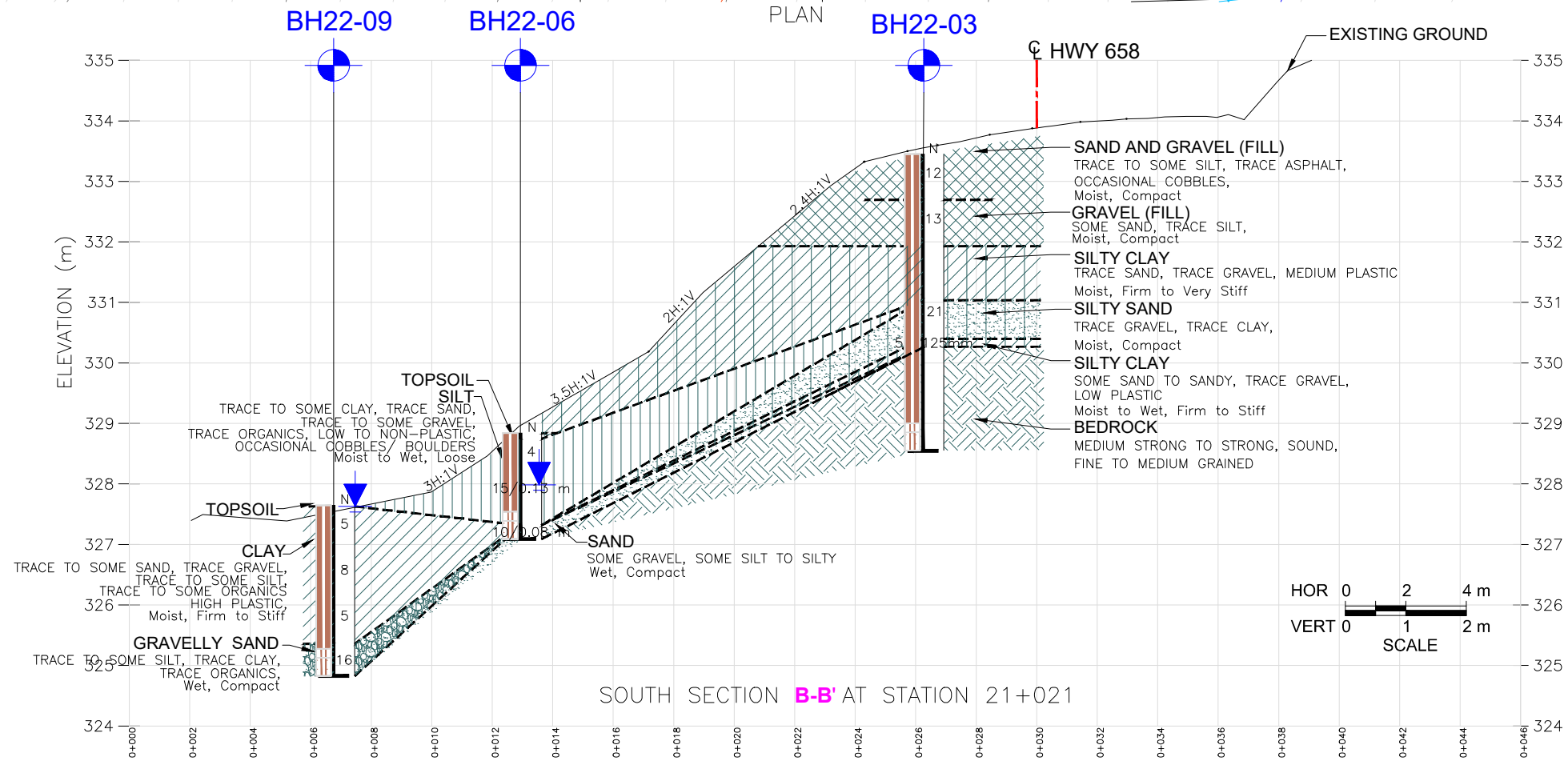
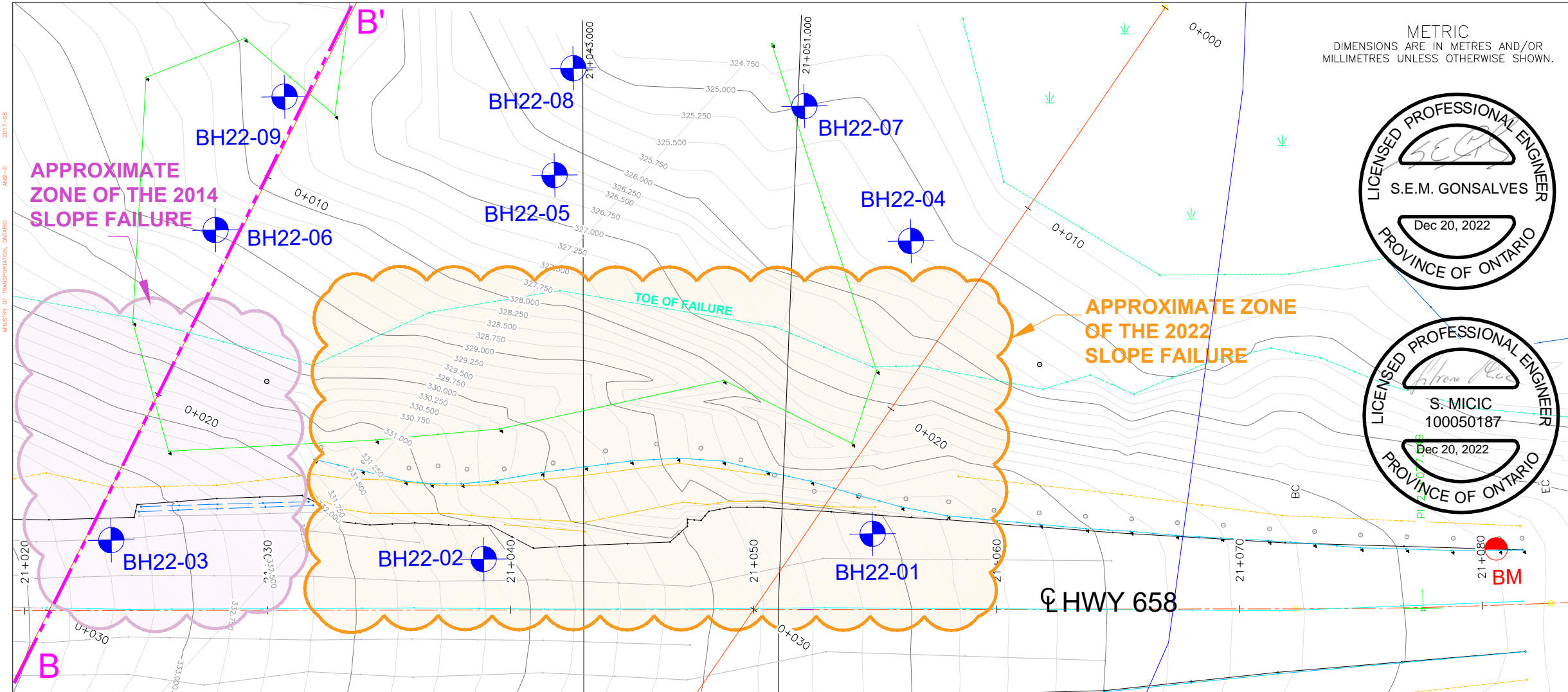
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	NO	DATE	BY	DESCRIPTION			
PROJECT No.		ADM-21019842-E0		GEOCES No. 52E-75			
SUBM'D	SH	CHKD.	SM	DATE	DEC 20, 2022	SITE	
DRAWN	SH	CHKD.	TC	APPRD	SG	DWG	01

SUBMISSION FOR MTO REVIEW

REVISIONS				SUBMISSION FOR MTO REVIEW			
	NO	DATE	BY	DESCRIPTION			
	PROJECT No. ADM-21019842-E0			GEOCRETS No. 52E-75			
	SUBM'D SH	CHKD. SM		DATE	DEC 20, 2022	SITE	
	DRAWN SH	CHKD. TC		APPRD	SG	DWG	01



FILE NAME: I:\2003-Brampton\Proposals\Projects\International\WTO Projects\Retainer NWR\6021-E-0019\A 5 - Hwy 658\AutoCAD Drawings\working drawings\A 5 - Hwy 658\Borehole location plan & soil strata.dwg  
MODIFIED: 2022-11-25 10:46



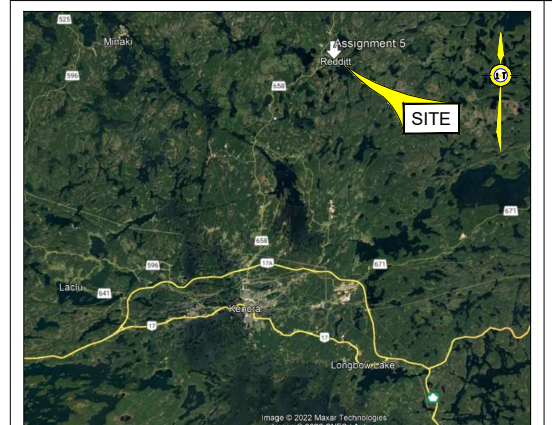
CONT No. 6021-E-0019  
ASSIG No. 5  
GWP No.



Foundation Investigation and Design for an Embankment  
Instability on Highway 658, Township of Redditt  
Latitude: 49.975717°, Longitude: -94.393332°  
BOREHOLE LOCATION PLAN & SOIL STRATA

SHEET  
2

exp. EXP SERVICES INC.



KEY PLAN  
N.T.S.

LEGEND

- Proposed Borehole Location
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level in Piezometer (most recent)  
( W. L. STABILIZED)
- Piezometer
- Bench Mark Location

SOIL STRATA SYMBOLS

- TOPSOIL
- ASPHALT
- FILL
- CLAY
- SAND
- SILT
- CLAYEY SILT
- SILTY CLAY
- SILTY SAND
- GRAVELLY SAND/  
SANDY GRAVEL
- BEDROCK

BOREHOLE CO-ORDINATES/ NAD 83/ MTM ON-16

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NOTES

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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 the report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

SUBMISSION FOR MTO REVIEW			
NO	DATE	BY	DESCRIPTION
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SUBM'D SH	CHKD. SM	DATE	DEC 20, 2022 SITE
DRAWN SH	CHKD. TC	APPRD SG	DWG 02

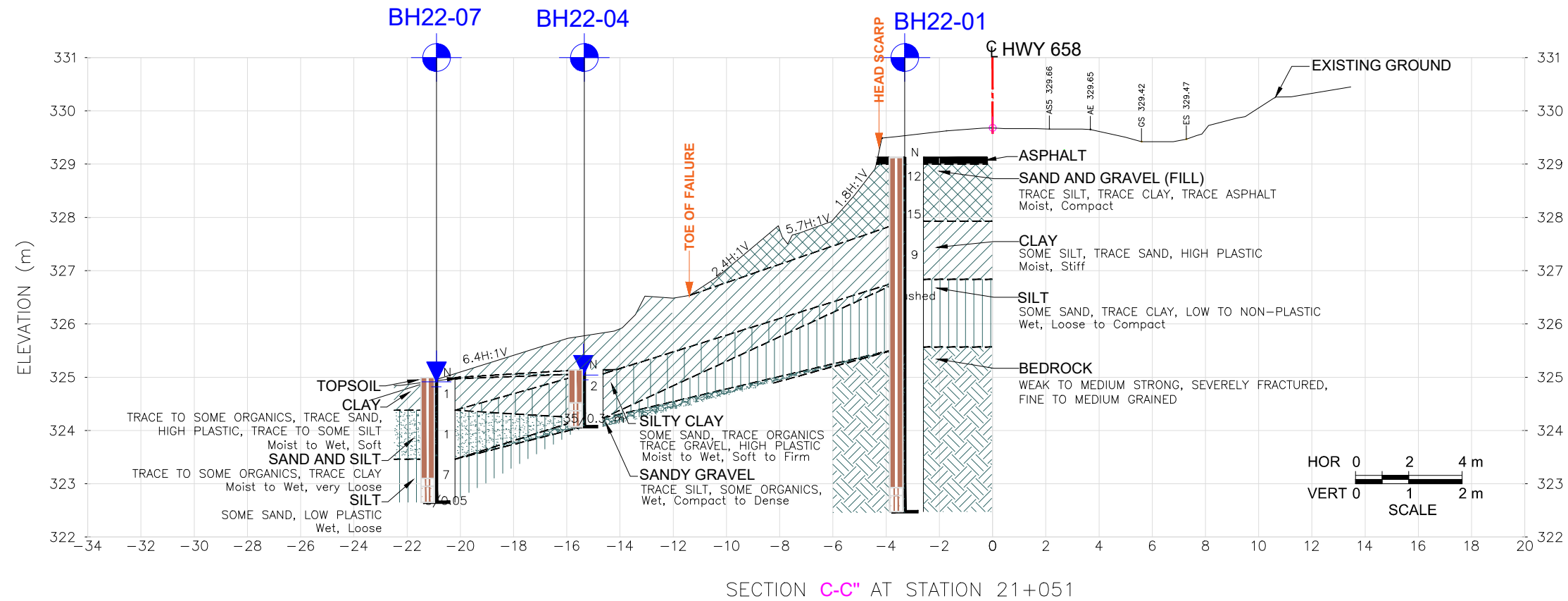
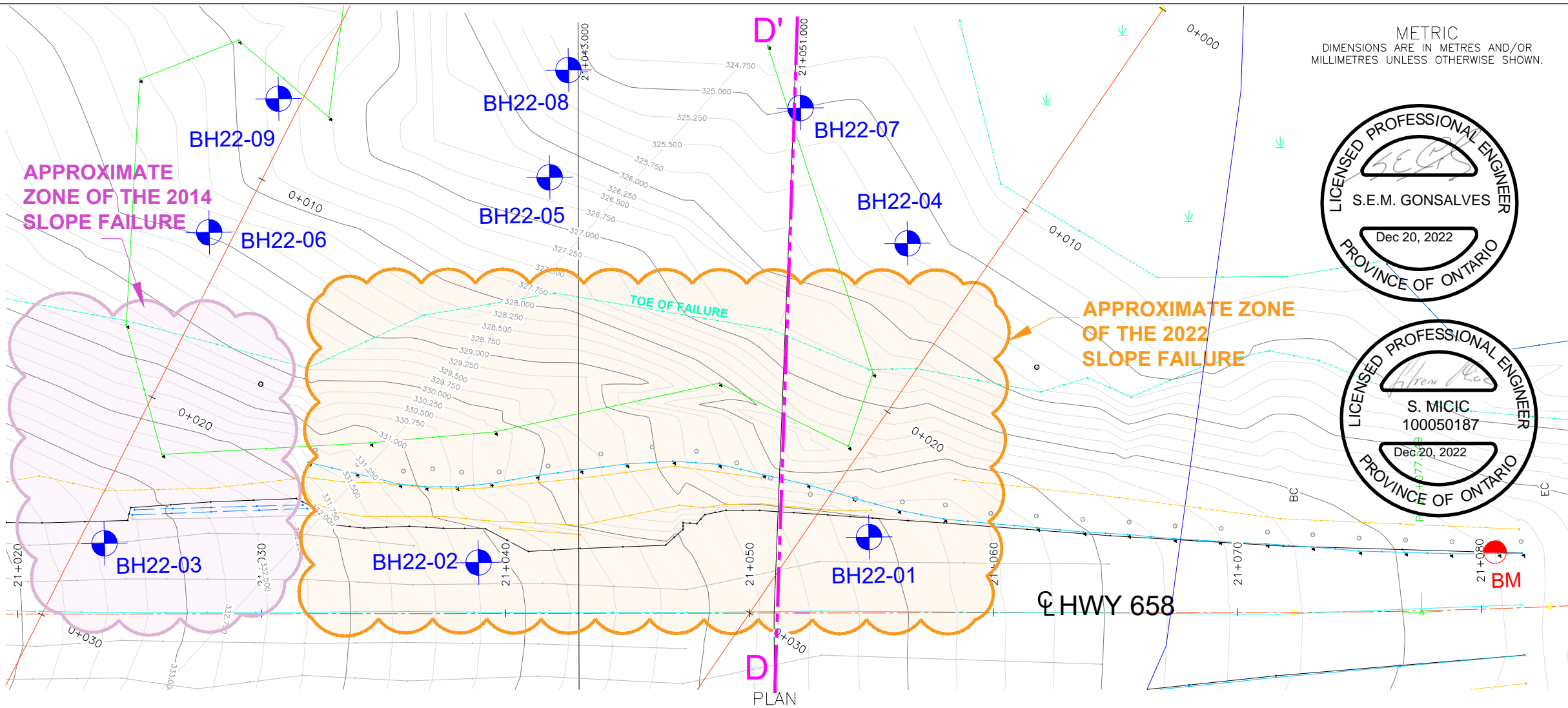




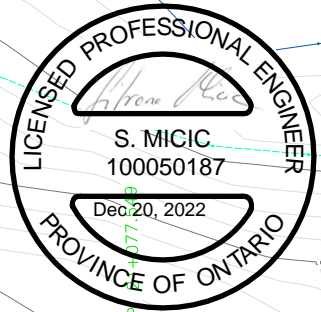


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MODIFIED: 2022-11-25 10:46

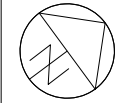
MINISTRY OF TRANSPORTATION, ONTARIO  
AUG-0 2017-08



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



CONT No. 6021-E-0019  
ASSIG No. 5  
GWP No.

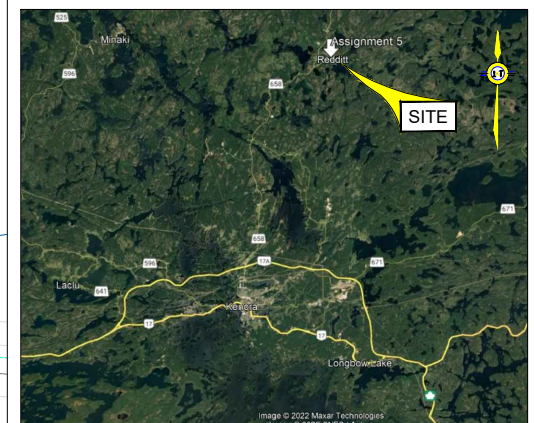


Foundation Investigation and Design for an Embankment  
Instability on Highway 658, Township of Redditt  
Latitude: 49.975717°, Longitude: -94.393332°  
BOREHOLE LOCATION PLAN & SOIL STRATA

SHEET  
4



EXP SERVICES INC.



KEY PLAN  
N.T.S.

LEGEND

- Borehole Location
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level in Piezometer (most recent)  
(W. L. STABILIZED)
- Piezometer
- Bench Mark Location

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BOREHOLE CO-ORDINATES/ NAD 83/ MTM ON-16

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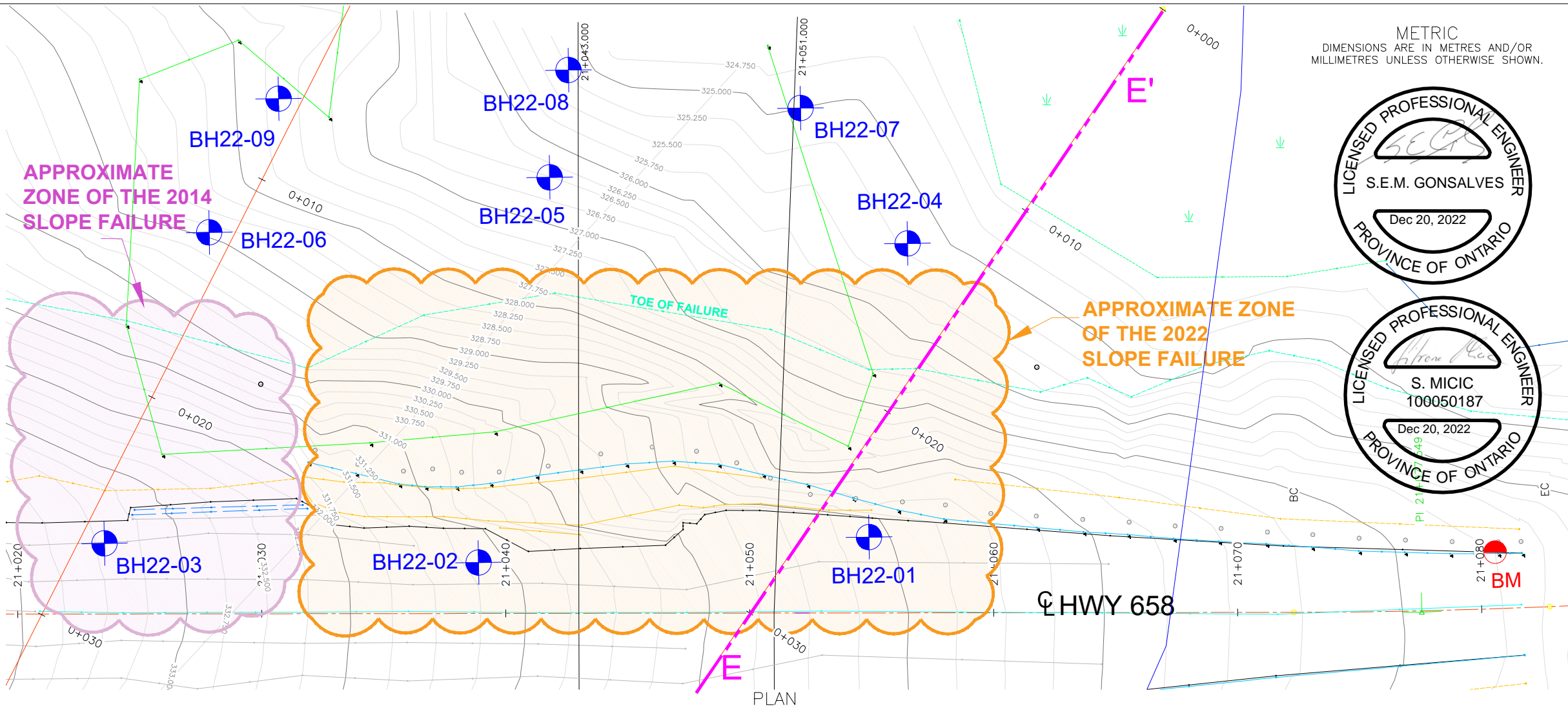
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SUBM'D SH	CHKD. SM	DATE	DEC 20, 2022 SITE
DRAWN SH	CHKD. TC	APPRD SG	DWG 04



FILE NAME: I:\2003-Brampton\Proposals\Projects\International\WTO Projects\Retainer NWR\6021-E-0019\A 5 - Hwy 658\AutoCAD Drawings\working drawings\A 5 - Hwy 658\Borehole location plan & soil strata.dwg  
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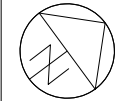
MINISTRY OF TRANSPORTATION, ONTARIO  
AUG-0 2017-08



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ASSIG No. 5  
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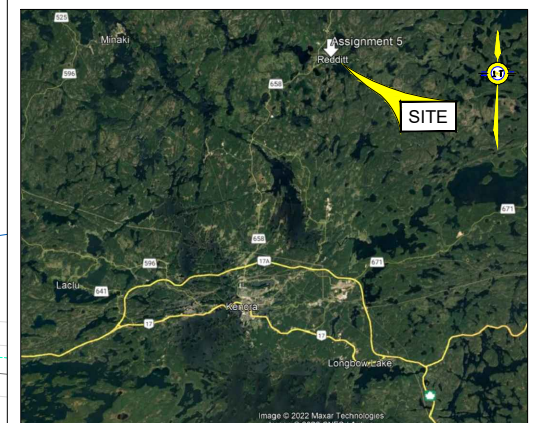


Foundation Investigation and Design for an Embankment  
Instability on Highway 658, Township of Redditt  
Latitude: 49.975717°, Longitude: -94.393332°  
BOREHOLE LOCATION PLAN & SOIL STRATA

SHEET  
5



EXP SERVICES INC.



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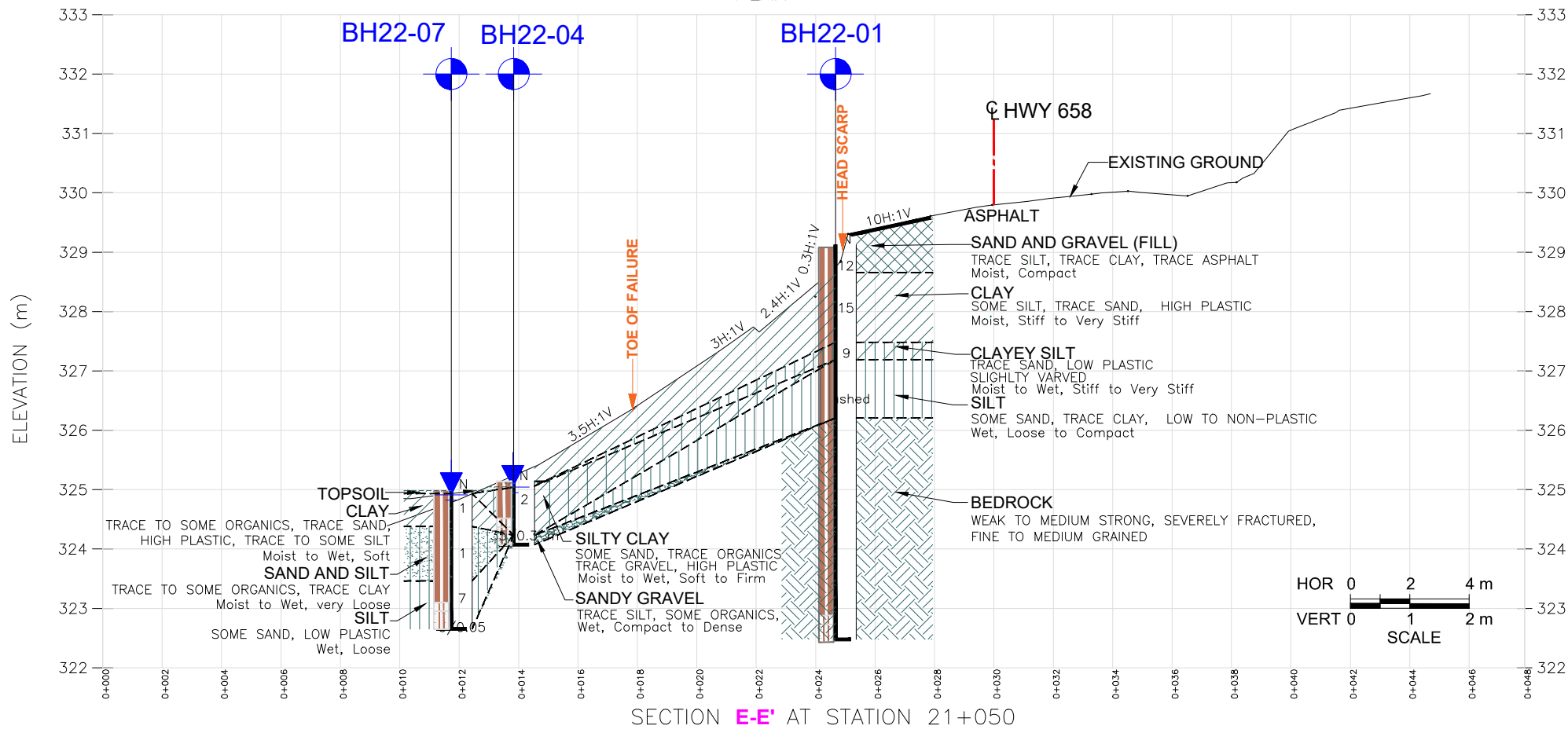
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DRAWN SH	CHKD. TC	APPRD SG	DWG 05



## Appendix C – Borehole Logs

# Explanation of Terms Used on Borehole Records

## SOIL DESCRIPTION

Terminology describing common soil genesis:

*Topsoil:* mixture of soil and humus capable of supporting good vegetative growth.

*Peat:* fibrous fragments of visible and invisible decayed organic matter.

*Fill:* where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc.; none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

*Till:* the term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Terminology describing soil structure:

*Desiccated:* having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.

*Stratified:* alternating layers of varying material or color with the layers greater than 6 mm thick.

*Laminated:* alternating layers of varying material or color with the layers less than 6 mm thick.

*Fissured:* material breaks along plane of fracture.

*Varved:* composed of regular alternating layers of silt and clay.

*Slickensided:* fracture planes appear polished or glossy, sometimes striated.

*Blocky:* cohesive soil that can be broken down into small angular lumps which resist further breakdown.

*Lensed:* inclusion of small pockets of different soil, such as small lenses of sand scattered through a mass of clay; not thickness.

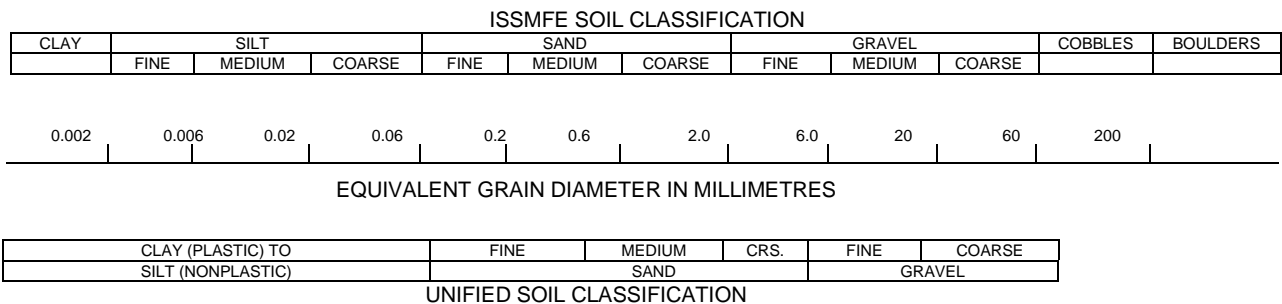
*Seam:* a thin, confined layer of soil having different particle size, texture, or color from materials above and below.

*Homogeneous:* same color and appearance throughout.

*Well Graded:* having wide range in grain sized and substantial amounts of all predominantly on grain size.

*Uniformly Graded:* predominantly on grain size.

All soil sample descriptions included in this report follow generally the ASTM D2487-11 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) with some modification to reflect current MTO practices. The system divides soils into three major categories: (1) coarse grained, (2) fine-grained, and (3) highly organic. The soil is then subdivided based on either gradation or plasticity characteristics. The system provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification. The classification excludes particles larger than 76 mm. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually in accordance with ASTM D2488-09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems. Others may use different classification systems; one such system is the ISSMFE Soil Classification.



Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present and as described below in accordance with Canadian Foundation Engineering Manual (CFEM):

Table a: Percent or Proportion of Soil

Term	Description	Criteria
"trace"	trace gravel, trace sand, etc.	1% - 10%
"some"	some gravel, some sand, etc.	10% - 20%
Adjective	gravelly, sandy, silty and clayey	20% - 35%
"and"	and gravel, and sand, etc.	>35%
Noun	gravel, sand, silt, clay	>35% and main fraction

The standard terminology to describe cohesionless soils includes the compactness as determined by the Standard Penetration Test 'N' value:

Table b: Apparent Density of Cohesionless Soil

	'N' Value (blows/0.3 m)
Very Loose	N<5
Loose	5≤N<10
Compact	10≤N<30
Dense	30≤N<50
Very Dense	50≤N



The standard terminology to describe cohesive soils includes consistency, which is based on undrained shear strength as measured by insitu vane tests, penetrometer tests, unconfined compression tests or similar field and laboratory analysis, Standard Penetration Test 'N' values can also be used to provide an approximate indication of the consistency and shear strength of fine grained, cohesive soils:

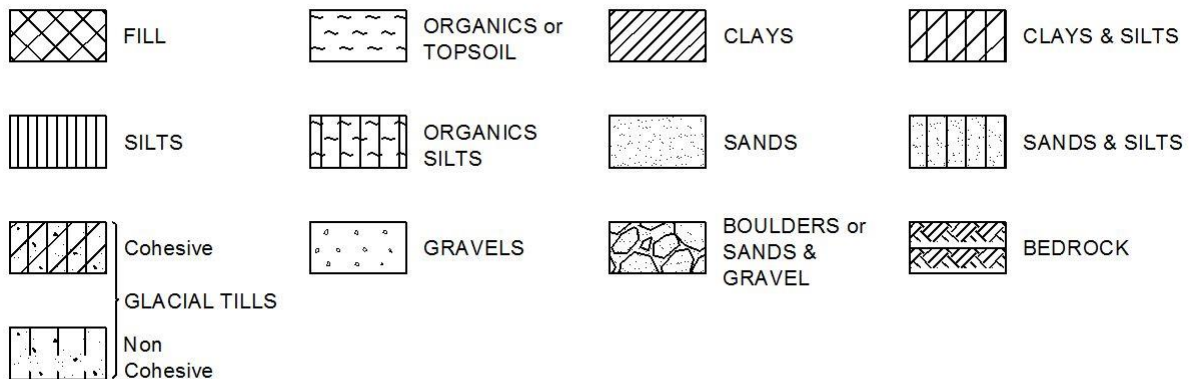
Table c: Consistency of Cohesive Soil

Consistency	Vane Shear Measurement (kPa)	'N' Value
Very Soft	<12.5	<2
Soft	12.5-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

Note: 'N' Value - The Standard Penetration Test records the number of blows of a 140 pound (64kg) hammer falling 30 inches (760mm), required to drive a 2 inch (50.8mm) O.D. split spoon sampler 1 foot (305mm). For split spoon samples where full penetration is not achieved, the number of blows is reported over the sampler penetration in meters (e.g. 50/0.15).

## STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols:



## WATER LEVEL MEASUREMENT



Open Borehole or Test Pit



Monitoring Well, Piezometer or Standpipe



## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

SS	Split spoon sample (obtained from the Standard Penetration Test)
WS	Wash sample
BS	Bulk sample
TW	Thin wall sample or Shelby tube
PS	Piston sample
AS	Auger sample
VT	Vane test
GS	Grab sample
HQ, NQ, etc.	Rock core samples obtained with the use of standard size diamond drilling bits

### STRESS AND STRAIN

$u_w$	kPa	Pore water pressure
$r_u$	1	Pore pressure ratio
$\sigma$	kPa	Total normal stress
$\sigma'$	kPa	Effective normal stress
$\tau$	kPa	Shear stress
$\sigma_1, \sigma_2, \sigma_3$	kPa	Principal stresses
$\varepsilon$	%	Linear strain
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	Principal strains
E	kPa	Modulus of linear deformation
G	kPa	Modulus of shear deformation
$\mu$	1	Coefficient of friction

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	Coefficient of volume change
$c_c$	1	Compression index
$c_s$	1	Swelling index
$c_r$	1	Recompression index
$c_v$	m <sup>2</sup> /s	Coefficient of consolidation
H	m	Drainage path
$T_v$	1	Time factor
U	%	Degree of consolidation
$\sigma'_{v0}$	kPa	Effective overburden pressure
$\sigma'_p$	kPa	Preconsolidation pressure
$\tau_f$	kPa	Shear strength
$c'$	kPa	Effective cohesion intercept
$\phi'$	—°	Effective angle of internal friction
$c_u$	kPa	Apparent cohesion intercept
$\phi_u$	—°	Apparent angle of internal friction
$\tau_R$	kPa	Residual shear strength
$\tau_r$	kPa	Remoulded shear strength
$S_t$	1	Sensitivity = $c_u/\tau_r$

### PHYSICAL PROPERTIES OF SOIL

$P_s$	kg/m <sup>3</sup>	Density of solid particles
$\gamma_s$	kN/m <sup>3</sup>	Unit weight of solid particles
$\rho_w$	kg/m <sup>3</sup>	Density of water
$\gamma_w$	kN/m <sup>3</sup>	Unit weight of water
$\rho$	kg/m <sup>3</sup>	Density of soil
$\gamma$	kN/m <sup>3</sup>	Unit weight of soil
$\rho_d$	kg/m <sup>3</sup>	Density of dry soil
$\gamma_d$	kN/m <sup>3</sup>	Unit weight of dry soil
$\rho_{sat}$	kg/m <sup>3</sup>	Density of saturated soil
$\gamma_{sat}$	kN/m <sup>3</sup>	Unit weight of saturated soil
$\rho'$	kg/m <sup>3</sup>	Density of submerged soil
$\gamma'$	kN/m <sup>3</sup>	Unit weight of submerged soil
$e$	1, %	Void ratio
$n$	1, %	Porosity
$w$	1, %	Water content
$S_r$	%	Degree of saturation
$W_L$	%	Liquid limit
$W_P$	%	Plastic limit
$W_s$	%	Shrinkage limit
$I_p$	%	Plasticity index = $(W_L - W_P)$
$I_L$	%	Liquidity index = $(W - W_P)/I_p$
$I_C$	%	Consistency index = $(W_L - W)/I_p$
$e_{max}$	1, %	Void ratio in loosest state
$e_{min}$	1, %	Void ratio in densest state
$I_D$	1	Density index = $(e_{max} - e)/(e_{max} - e_{min})$
D	mm	Grain diameter
$D_n$	mm	N percent - diameter
$C_u$	1	Uniformity coefficient
h	m	Hydraulic head or potential
q	m <sup>3</sup> /s	Rate of discharge
v	m/s	Discharge velocity
i	1	Hydraulic gradient
k	m/s	Hydraulic conductivity
j	kN/m <sup>3</sup>	Seepage force

METRIC

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-02

1 OF 1

METRIC

W.P. Agreement No. 6021-E-0019, WO No. 5 LOCATION N5538509.8, E204857.2, NAD83 MTM Zone 16 ORIGINATED BY KP  
 DIST NWR HWY 658 BOREHOLE TYPE Continuous Flight HSA, HQ Casing, and NQ Core Barrel COMPILED BY DM  
 DATUM Local DATE 2022.10.03 - 2022.10.03 LATITUDE 49.975599 LONGITUDE -94.393443 CHECKED BY SM

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						
								20 40 60 80 100						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER						
331.2								20 40 60 80 100	20 40 60			kN/m <sup>3</sup>	GR SA SI CL	
331.0	ASPHALT, ~40 mm thick		SS1A + SS1B	SS	10		331						27 66 (7)	
330.8	GRAVELLY SAND (FILL), trace silt, trace asphalt, dark brown to black, moist, compact												19 72 6 2	
330.2	SAND (FILL), some gravel, trace silt, trace clay, trace organics, brown, moist, compact													
330.2	- some silt, cobble encountered		SS2A + SS2B	SS	18		330							
0.9	CLAY, some silt, trace to some organics, peat/wood debris, trace sand, dark brown to black, high plastic, moist, stiff to very stiff													
	- no organics, brown to grey, stiff		SS3	SS	7							86	0 1 17 83	
328.9	- measured vane shear strength greater than 100 kPa at 2.1 m			VANE			329							
2.3	CLAYEY SILT, trace sand, grey, low plastic, moist to wet, slightly varved, stiff to very stiff		SS4A + SS4B	SS	14								0 3 80 17	
	- firm to stiff, wet												0 5 (95)	
327.9			SS5A + SS5B	SS	19		328						0 13 (87)	
3.2	SILT, some sand, trace clay, grey, wet, compact												0 12 81 7	
327.7													20 54 (26)	
3.4	SILTY SAND, some gravel to gravelly, grey, wet, compact		SS5C											
327.6	BEDROCK, strong, very sound, grey to white/pink, fine to medium grained													
3.6	Run 1: Start/End 3.58 m to 4.47 m TCR = 0.89 m, 100% RQD = 93%		Run 1	NQ			327							
	Run 2: Start/End 4.47 m to 5.21 m TCR = 0.74 m, 100% RQD = 100%		Run 2	NQ										
325.9	BOREHOLE TERMINATED AT ~ 5.2 m DEPTH DUE TO REFUSAL						326							
5.2	Notes: 1. No groundwater measured in open hole prior to rock coring. 2. 25 mm inside diameter PVC piezometer installed upon completion. Screened from approximately 4.9 m to 5.2 m below ground surface. 3. No groundwater observed in piezometer upon installation.													

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

ONTARIO MTO ADM-21019842-EO - NWR - ASSIGNMENT 5 - INSTABILITY ON HWY 658.GPJ ONTARIO MTO.GDT 11/14/22

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-03

1 OF 1

METRIC

W.P. Agreement No. 6021-E-0019, WO No. 5 LOCATION N5538497.9, E204847.6, NAD83 MTM Zone 16 ORIGINATED BY KP  
 DIST NWR HWY 658 BOREHOLE TYPE Continuous Flight HSA, HQ Casing, and NQ Core Barrel COMPILED BY DM  
 DATUM Local DATE 2022.10.04 - 2022.10.04 LATITUDE 49.97549 LONGITUDE -94.393574 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER		W <sub>P</sub> W                      W <sub>L</sub> WATER CONTENT (%)				
333.5 0.0	SAND AND GRAVEL (FILL), trace to some silt, trace asphalt, occasional cobbles, dark brown to black, moist, compact		SS1	SS	12		333							
332.7 0.8	GRAVEL (FILL), some sand, trace silt, grey to brown, moist, compact		SS2	SS	13									
331.9 1.5	SILTY CLAY, trace gravel, trace sand, grey to brown, medium plastic, moist, firm to stiff		ST3	SH			332							
331.0 2.4	- very stiff, measured vane shear strength greater than 100 kPa at 2.1 m SILTY SAND, trace gravel, trace clay, grey, moist, compact			VANE										
330.4 3.2	SILTY SAND, trace gravel, trace clay, grey, moist, compact		SS4A + SS4B	SS	21		331							6 60 (34)
330.3 3.2	SILTY CLAY, some sand to sandy, trace gravel, grey, low plastic, moist to wet, firm to stiff - spoon bouncing BEDROCK, medium strong to strong, sound, grey to white/pink, fine to medium grained		SS5	SS	59/ 0.125 m									
328.6 4.9	Run 1: Start/End 3.18 m to 4.50 m TCR = 1.32 m, 100% RQD = 86%  Run 2: Start/End 4.50 m to 4.88 m TCR = 0.38 m, 100% RQD = 100%		Run 1	NQ			330							
	- very sound		Run 2	NQ										
328.6 4.9	BOREHOLE TERMINATED AT ~ 4.9 m DEPTH DUE TO REFUSAL  Notes: 1. No groundwater measured in open hole prior to rock coring. 2. 25 mm inside diameter PVC piezometer installed upon completion. Screened from approximately 4.6 m to 4.9 m below ground surface. 3. No groundwater observed in piezometer upon installation.													

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

ONTARIO MTO ADM-21019842.E0 - NWR - ASSIGNMENT 5 - INSTABILITY ON HWY 658.GPJ ONTARIO MTO.GDT 11/14/22





Brampton, Ontario

## RECORD OF BOREHOLE No BH22-04

1 OF 1

METRIC

W.P. Agreement No. 6021-E-0019, WO No. 5 LOCATION N5538531.8, E204856.9, NAD83 MTM Zone 16 ORIGINATED BY KP  
 DIST NWR HWY 658 BOREHOLE TYPE Continuous Flight SSA COMPILED BY DM  
 DATUM Local DATE 2022.10.05 - 2022.10.05 LATITUDE 49.975796 LONGITUDE -94.393453 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER		W <sub>P</sub> W                      W <sub>L</sub> WATER CONTENT (%)				
325.1 325.0 0.1	<b>TOPSOIL</b> , ~75 mm thick  <b>SILTY CLAY</b> , some sand, trace gravel, trace organics, grey, high plastic, moist to wet, soft to firm  - firm		SS1A + SS1B	SS	2		325							Shelby Tube sample in adjacent hole (0 m to 0.6 m) 1   11   42   46
324.2 0.9 324.1 1.1	<b>SANDY GRAVEL</b> , trace silt, some organics, dark brown to grey, wet, compact to dense - spoon bouncing <b>BOREHOLE TERMINATED AT ~ 1.1 m DEPTH DUE TO REFUSAL</b>  Notes: 1. 25 mm inside diameter PVC piezometer installed upon completion. Screened from approximately 0.8 m to 1.1 m below ground surface. 2. Water level measured in piezometer at 0.1 m depth below ground surface on October 6, 2022.		SS2A + SS2B	SS	35/ 0.3 m									63   29   (8)

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

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-05

1 OF 1

METRIC

W.P. Agreement No. 6021-E-0019, WO No. 5 LOCATION N5538521.5, E204846.1, NAD83 MTM Zone 16 ORIGINATED BY KP  
 DIST NWR HWY 658 BOREHOLE TYPE Continuous Flight SSA, HQ Casing, and NQ Core Barrel COMPILED BY DM  
 DATUM Local DATE 2022.10.06 - 2022.10.07 LATITUDE 49.975701 LONGITUDE -94.393601 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER									
326.5								20	40	60	80	100					
326.0	TOPSOIL ~50 mm thick		SS1A + SS1B	SS	5		326										
	SILTY CLAY, trace gravel, trace sand, trace to some organics, grey, high plastic, moist to wet, firm to stiff			VANE					+								Shelby Tube sample in adjacent hole (0.3 m to 0.9 m)
	- stiff		SS2	SS	8												0 3 26 70
							325										
	- some sand to sandy		SS3A + SS3B	SS	10												
324.4	- spoon bouncing																
2.1	BEDROCK, medium strong, fractured, grey to pink/white, fine to medium grained						324										
	Run 1: Start/End 2.1 m to 3.6 m TCR = 1.52 m, 100% RQD = 55%		Run 1	NQ													
322.9							323										
3.6	BOREHOLE TERMINATED AT ~ 3.6 m DEPTH DUE TO REFUSAL																
	Notes: 1. No groundwater measured in open hole prior to rock coring. 2. 25 mm inside diameter PVC piezometer installed upon completion. Screened from approximately 3.3 m to 3.6 m below ground surface. 3. No groundwater observed in piezometer upon installation.																

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

ONTARIO MTO ADM-21019842.E0 - NWR - ASSIGNMENT 5 - INSTABILITY ON HWY 658.GPJ ONTARIO MTO.GDT 11/14/22

Brampton, Ontario

# RECORD OF BOREHOLE No BH22-06

1 OF 1

METRIC

W.P. Agreement No. 6021-E-0019, WO No. 5 LOCATION N5538508.8, E204839.7, NAD83 MTM Zone 16 ORIGINATED BY KP  
 DIST NWR HWY 658 BOREHOLE TYPE Continuous Flight SSA COMPILED BY DM  
 DATUM Local DATE 2022.10.05 - 2022.10.05 LATITUDE 49.975587 LONGITUDE -94.393686 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER		W <sub>P</sub>	W	W <sub>L</sub>	WATER CONTENT (%)				
328.8								20	40	60	80	100	20	40	60	kN/m <sup>3</sup>	GR SA SI CL
328.0	TOPSOIL, ~100 mm thick		SS1A + SS1B	SS	4												Shelby Tube sample in adjacent hole (0.2 m to 0.8m)
0.1	SILT, trace to some clay, trace sand, trace gravel, trace organics, brown to grey, low to non-plastic, moist to wet, loose																0 5 82 13
	- some gravel, trace clay, non-plastic, occasional cobble/boulder, spoon bouncing		SS2	SS	15/ 0.13 m		328										19 7 70 5
327.3																	
1.5	SAND, some silt to silty, some gravel, grey to brown, wet, compact - spoon bouncing		SS3	SS	10/ 0.08 m												13 66 (21)
327.1																	
1.8	BOREHOLE TERMINATED AT ~ 1.8 m DEPTH DUE TO REFUSAL																
	Notes: 1. 25 mm inside diameter PVC piezometer installed upon completion. Screened from approximately 1.5 m to 1.8 m below ground surface. 2. Water level measured in piezometer at 0.9 m depth below ground surface on October 6, 2022.																

Brampton, Ontario

# RECORD OF BOREHOLE No BH22-07

1 OF 1

METRIC

W.P. Agreement No. 6021-E-0019, WO No. 5 LOCATION N5538531.5, E204849.8, NAD83 MTM Zone 16 ORIGINATED BY KP  
 DIST NWR HWY 658 BOREHOLE TYPE Continuous Flight SSA COMPILED BY DM  
 DATUM Local DATE 2022.10.05 - 2022.10.05 LATITUDE 49.975792 LONGITUDE -94.393551 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
325.0								20	40	60	80	100		
324.9	TOPSOIL, ~25 mm thick													
324.4	CLAY, trace to some organics, trace to some silt, trace sand, dark brown to black, high plastic, moist to wet, soft		SS1	SS	1									Shelby Tube sample in adjacent hole (0 m to 0.6 m)
324.4														
0.6	SAND AND SILT, trace to some organics, trace clay, dark brown, moist to wet, very loose			VANE										
			SS2A + SS2B	SS	1		324							0 54 41 5
	- silty sand													0 71 (29)
323.5														
1.5	SILT, some sand, dark brown, low plastic, wet, loose		SS3A + SS3B	SS	7		323						106.3	0 16 (84)
322.7	- trace organics, grey - spoon bouncing		SS4	SS	53/ 0.05 m									
2.3	BOREHOLE TERMINATED AT ~ 2.3 m DEPTH DUE TO REFUSAL													
	Notes: 1. 25 mm inside diameter PVC piezometer installed upon completion. Screened from approximately 2.0 m to 2.3 m below ground surface. 2. Water level measured in piezometer at 0.1 m depth below ground surface on October 6, 2022.													

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

ONTARIO MTO ADM-21019842.E0 - NWR - ASSIGNMENT 5 - INSTABILITY ON HWY 658.GPJ ONTARIO MTO.GDT 11/14/22






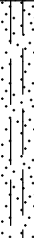

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-08

1 OF 1

METRIC

W.P. Agreement No. 6021-E-0019, WO No. 5 LOCATION N5538524.7, E204843.0, NAD83 MTM Zone 16 ORIGINATED BY KP  
 DIST NWR HWY 658 BOREHOLE TYPE Continuous Flight SSA COMPILED BY DM  
 DATUM Local DATE 2022.10.05 - 2022.10.05 LATITUDE 49.97573 LONGITUDE -94.393645 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER									
325.3								20	40	60	80	100					
325.0	TOPSOIL ~25 mm thick CLAYEY SILT TO SILTY CLAY, some sand to sandy, trace gravel, some organics, dark brown, low to medium plastic, moist, firm		SS1	SS	4		325										4 28 44 25
324.4	- vane refusal on cobble/boulder - trace organics, grey																Shelby Tube sample in adjacent hole (0 m to 0.6 m)
0.9	SILTY SAND , trace gravel, trace to some organics, trace clay, moist to wet, dark brown, loose to compact		SS2A + SS2B	SS	5		324										
	- compact																
			SS3A + SS3B	SS	24												1 63 32 4
323.1	- some gravel																13 64 (23)
2.2	BOREHOLE TERMINATED AT ~ 2.2 m DEPTH DUE TO REFUSAL  Notes: 1. 25 mm inside diameter PVC piezometer installed upon completion. Screened from approximately 1.9 m to 2.2 m below ground surface. 2. Water level measured in piezometer at 0.4 m depth below ground surface on October 6, 2022.																

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MTO ADM-21019842.E0 - NWR - ASSIGNMENT 5 - INSTABILITY ON HWY 658.GPJ ONTARIO MTO.GDT 11/14/22

Brampton, Ontario

## RECORD OF BOREHOLE No BH22-09

1 OF 1

METRIC

W.P. Agreement No. 6021-E-0019, WO No. 5 LOCATION N5538514.4, E204837.0, NAD83 MTM Zone 16 ORIGINATED BY KP  
 DIST NWR HWY 658 BOREHOLE TYPE Continuous Flight SSA COMPILED BY DM  
 DATUM Local DATE 2022.10.05 - 2022.10.05 LATITUDE 49.975636 LONGITUDE -94.393726 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL P. PENETROMETER		W <sub>P</sub> W                      W <sub>L</sub> WATER CONTENT (%)					
327.7								20	40	60	80	100			
327.6	TOPSOIL, ~25 mm thick														
	CLAY, trace to some sand, trace to some silt, trace gravel, trace to some organics, dark brown, high plastic, moist, firm		SS1	SS	5										
	- stiff			VANE			327								
			SS2	SS	8										
	- grey, firm						326								
			SS3	SS	5										
325.4															
2.3	GRAVELLY SAND, trace to some silt, trace clay, trace organics, brown to grey, wet, compact		SS4A + SS4B	SS	16		325								30   54   10   6
324.8	- spoon bouncing														22   60   (18)
2.8	BOREHOLE TERMINATED AT ~2.8 m DEPTH DUE TO REFUSAL														
	Notes: 1. 25 mm inside diameter PVC piezometer installed upon completion. Screened from approximately 2.5 m to 2.8 m below ground surface. 2. Water level measured in piezometer at 0.02 m depth below ground surface on October 6, 2022.														

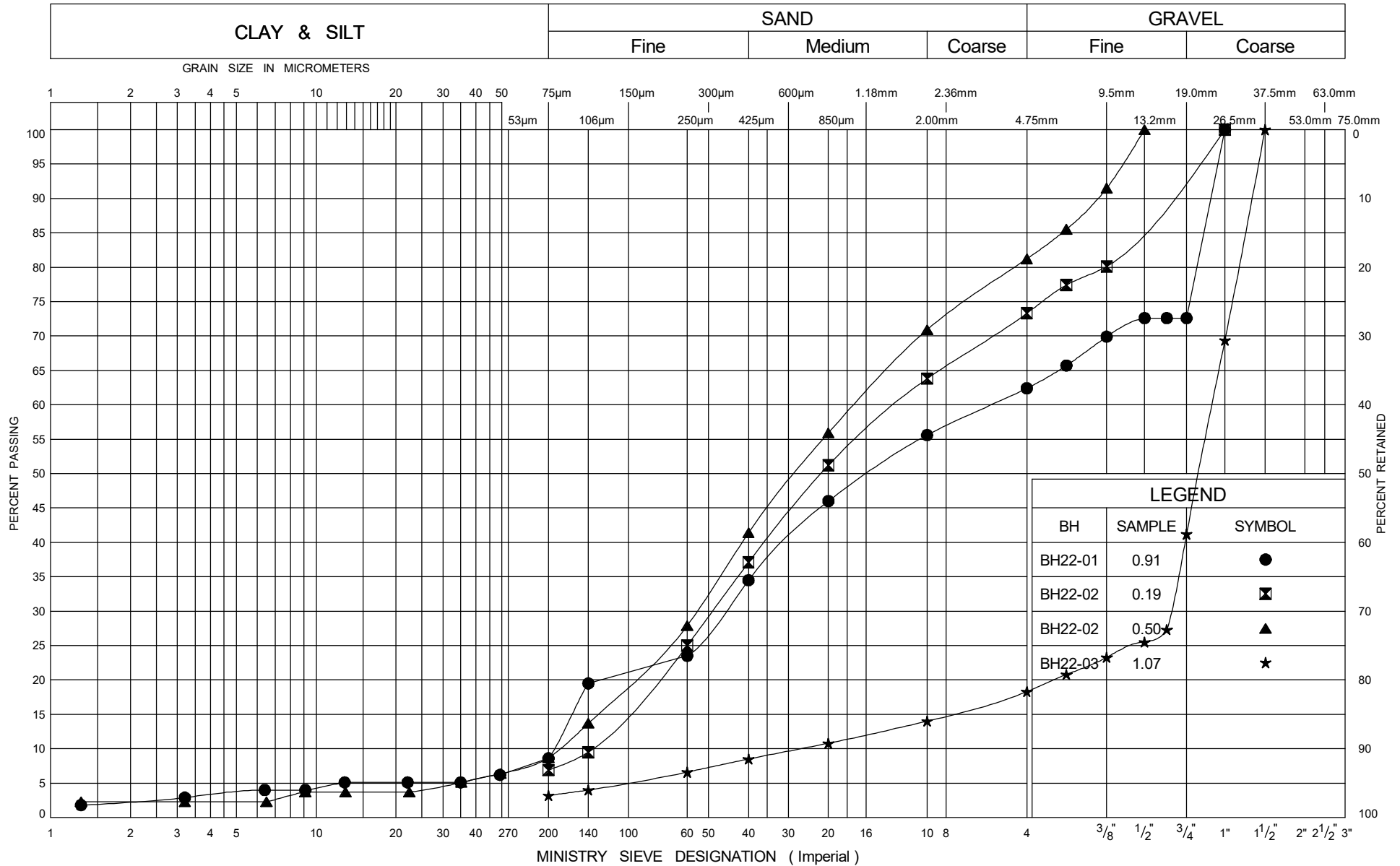
Shelby Tube  
sample in  
adjacent hole  
(0.6 m to 1.2 m)  
0 13 29 59

30 54 10 6  
22 60 (18)

## Appendix D – Laboratory Data

## Results of Grain Size Analyses

# UNIFIED SOIL CLASSIFICATION SYSTEM



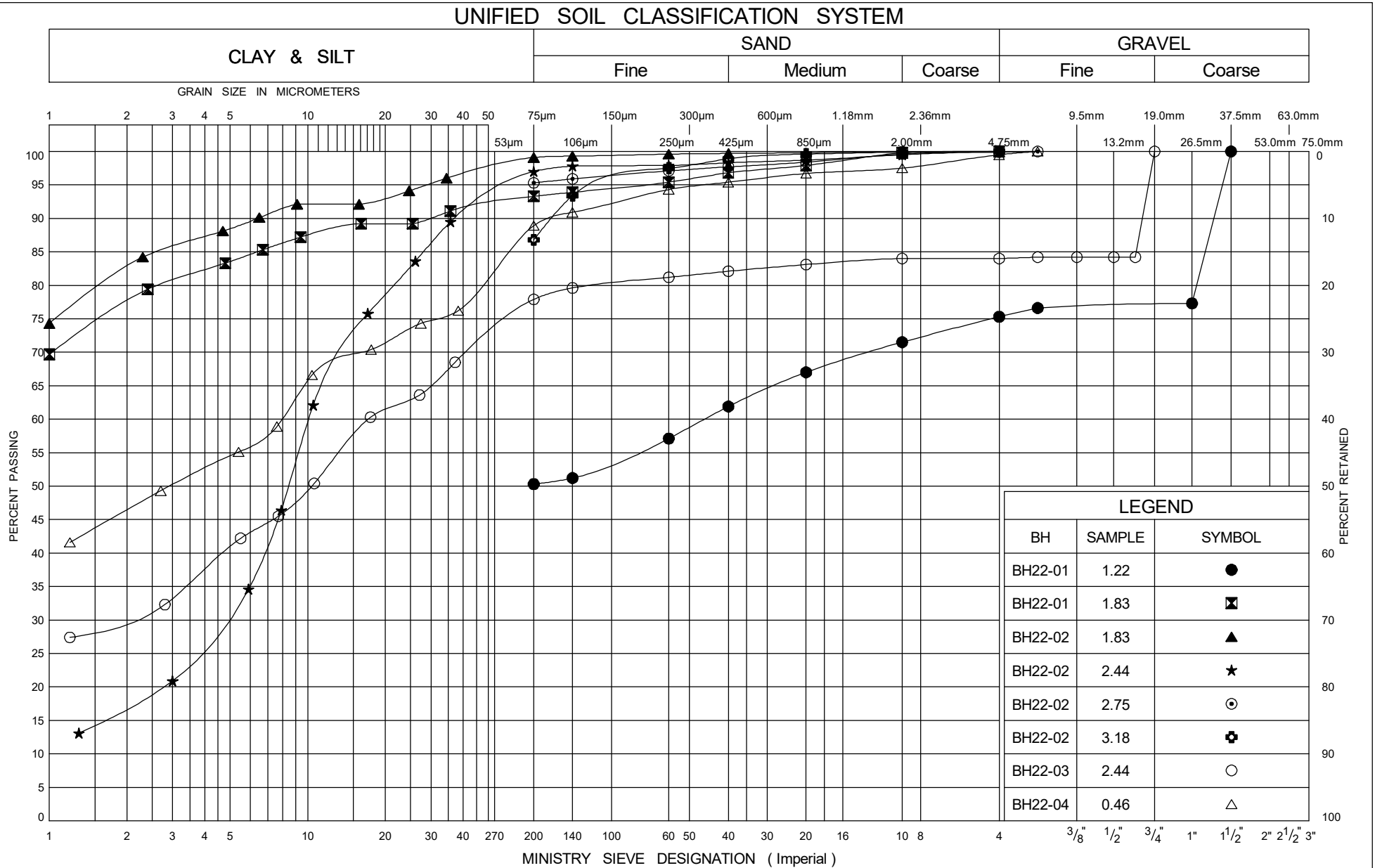
**GRAIN SIZE DISTRIBUTION**

Gravel / Sand & Gravel / Gravelly Sand / Sand (FILL)

**FIG No 1**

W P Assignment No. 6021-E-0019

Instability on Hwy 658



Ministry of  
Transportation

## GRAIN SIZE DISTRIBUTION

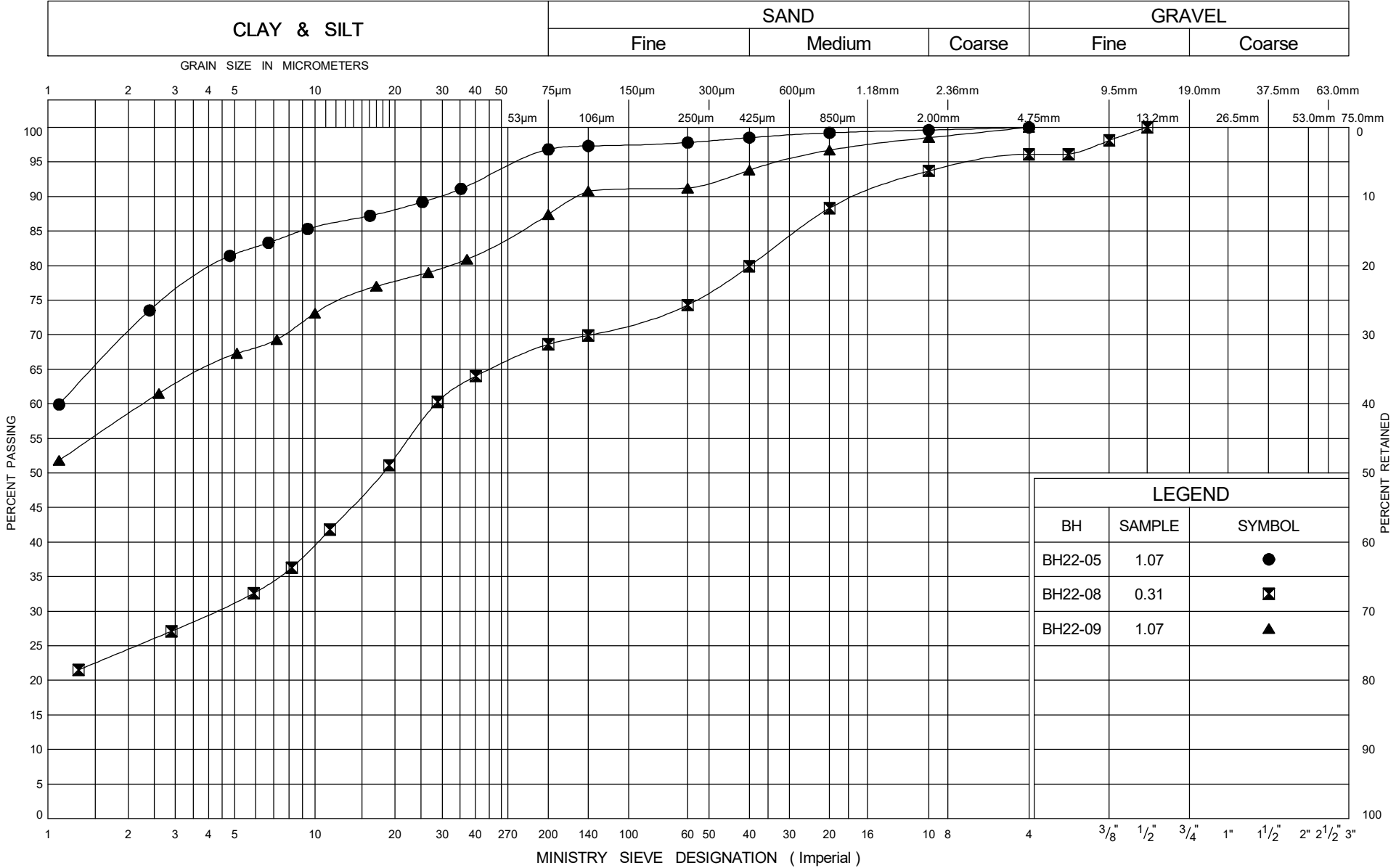
Clay / Silty Clay / Clayey Silt

FIG No 2

W P Assignment No. 6021-E-0019

Instability on Hwy 658

UNIFIED SOIL CLASSIFICATION SYSTEM



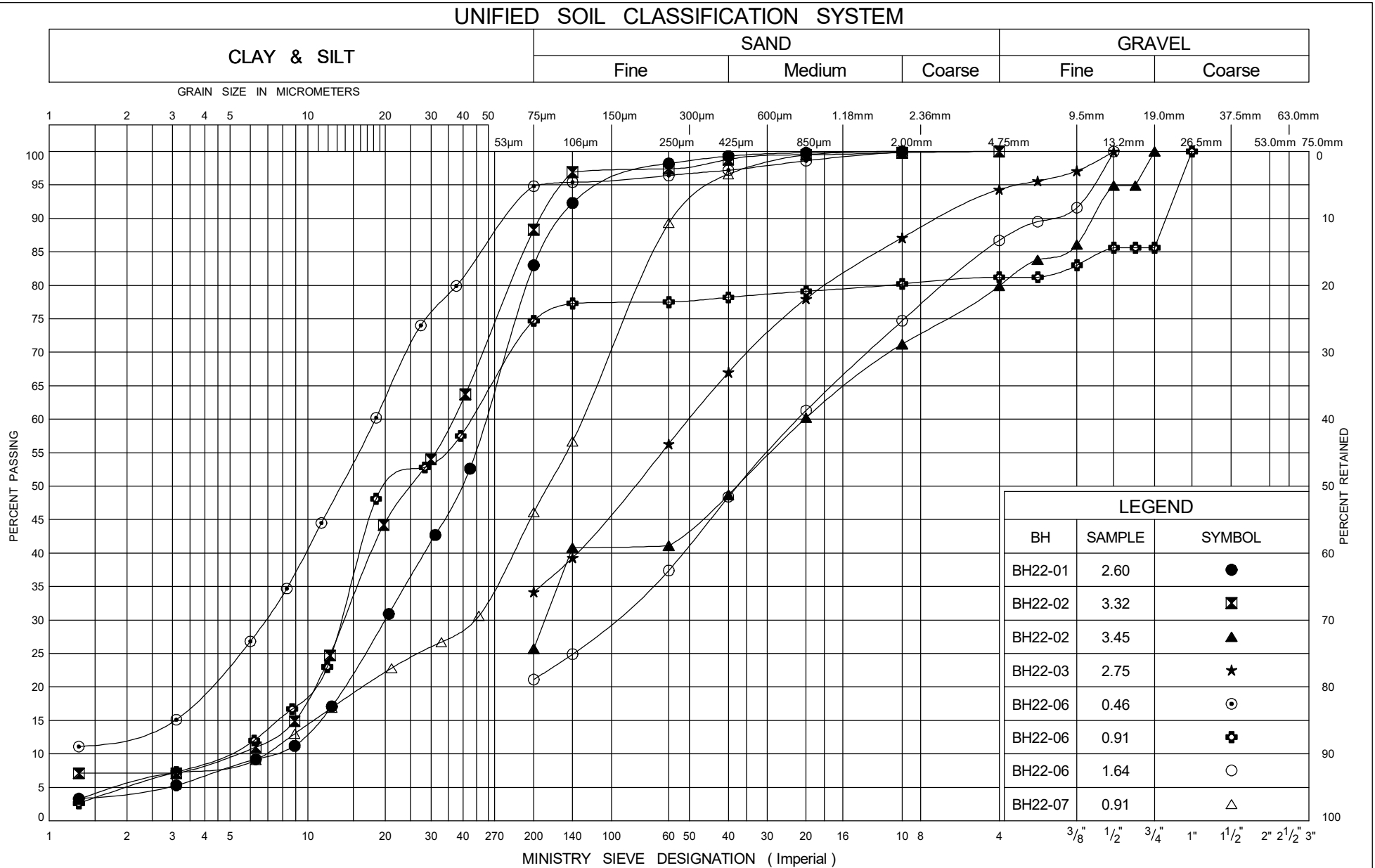
GRAIN SIZE DISTRIBUTION

Clay / Silty Clay / Clayey Silt

FIG No 3

W P Assignment No. 6021-E-0019

Instability on Hwy 658



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Transportation

## GRAIN SIZE DISTRIBUTION

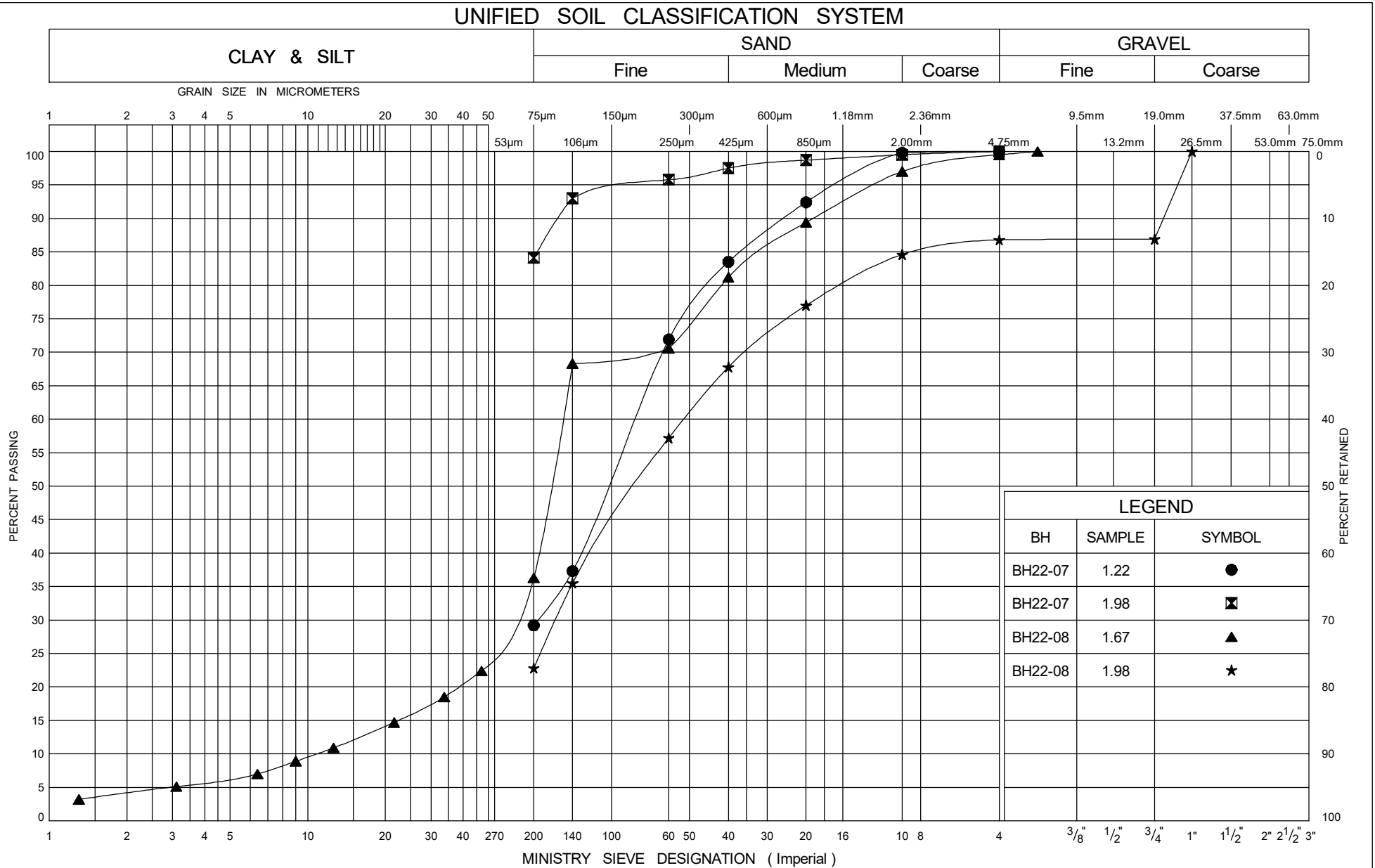
Silt / Sandy Silt / Silty Sand / Sand and Silt

FIG No 4

W P Assignment No. 6021-E-0019

Instability on Hwy 658





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## GRAIN SIZE DISTRIBUTION

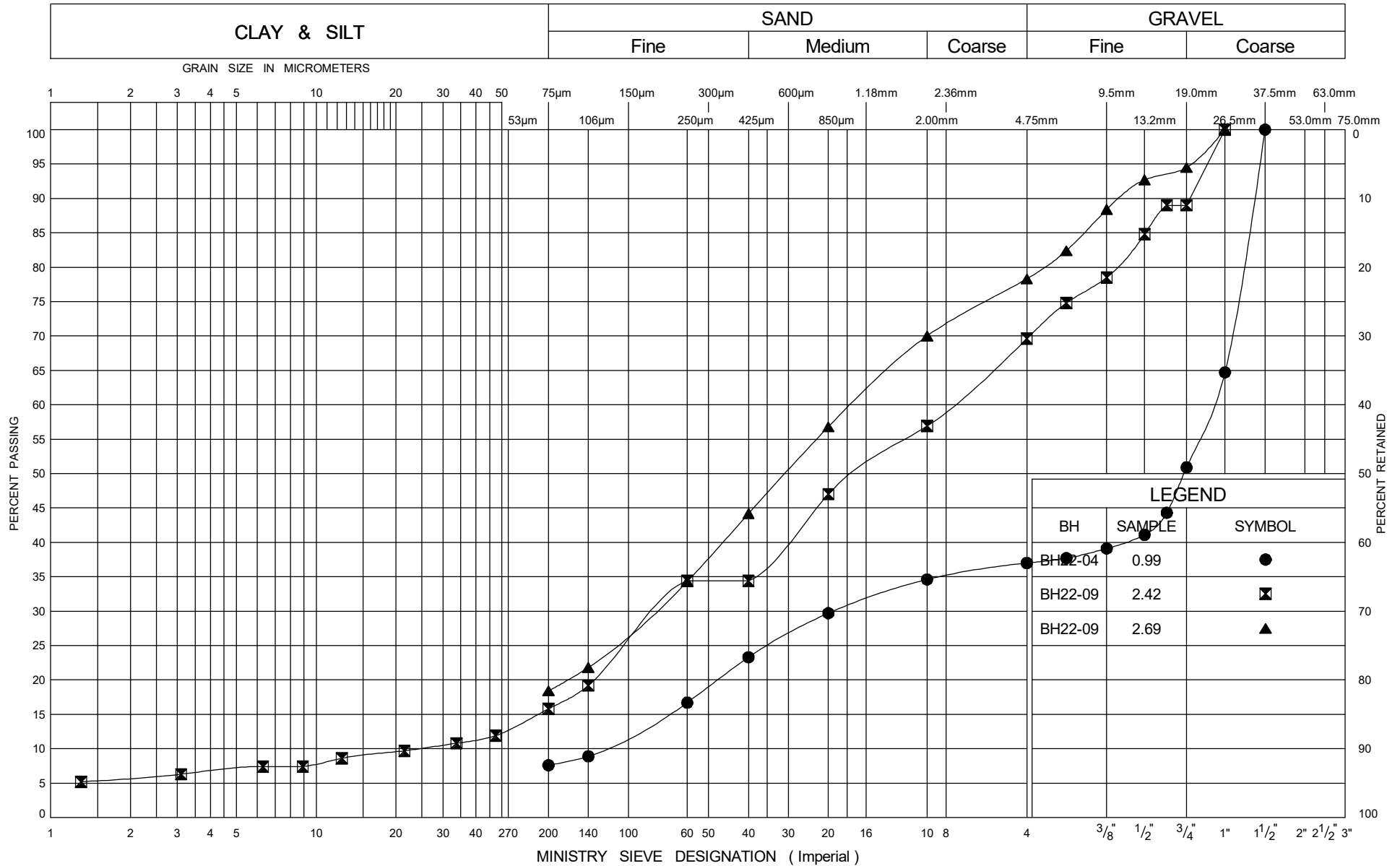
Silt / Sandy Silt / Silty Sand / Sand and Silt

FIG No 5

W P Assignment No. 6021-E-0019

Instability on Hwy 658

# UNIFIED SOIL CLASSIFICATION SYSTEM



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## GRAIN SIZE DISTRIBUTION

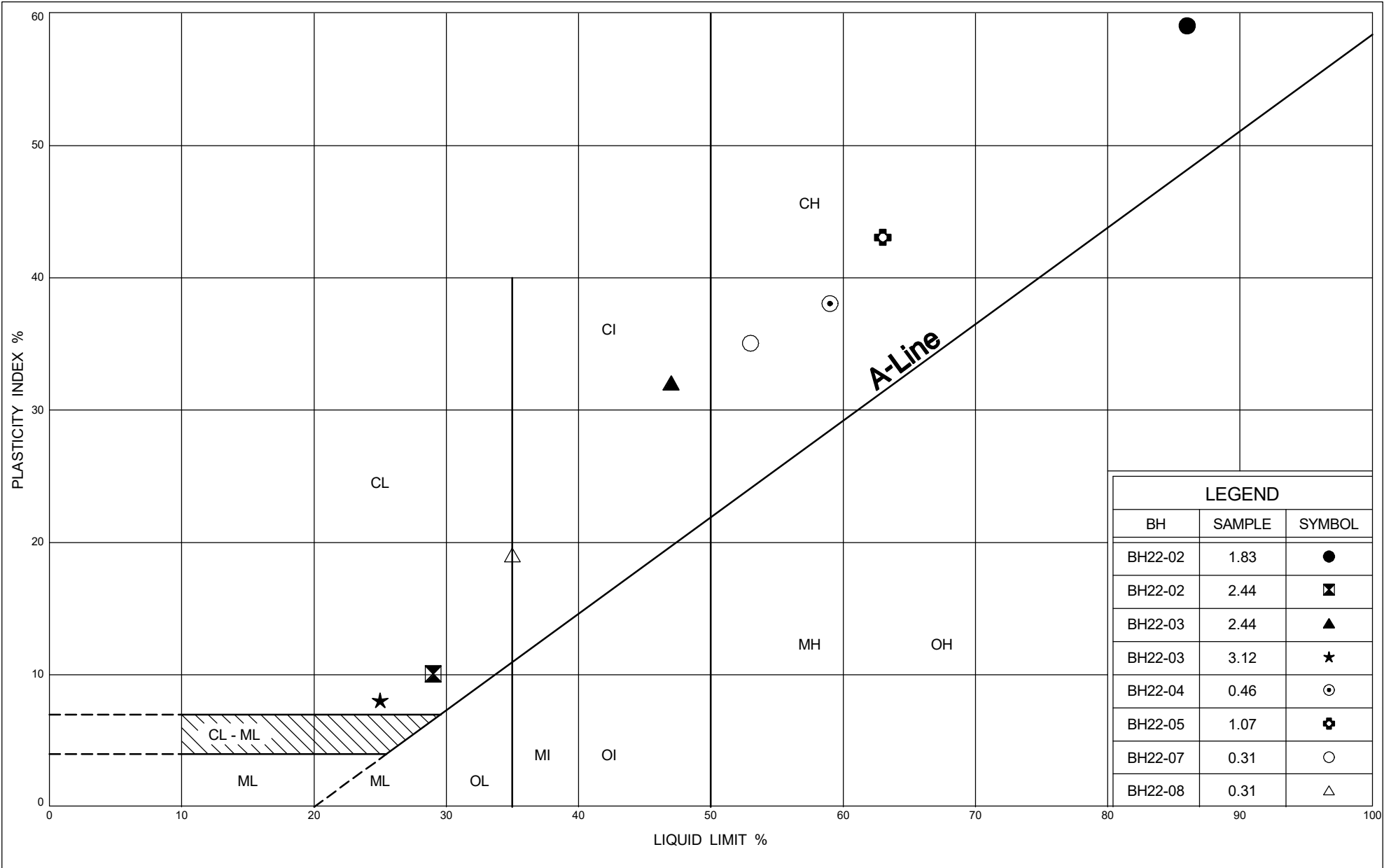
Sandy Gravel / Gravelly Sand

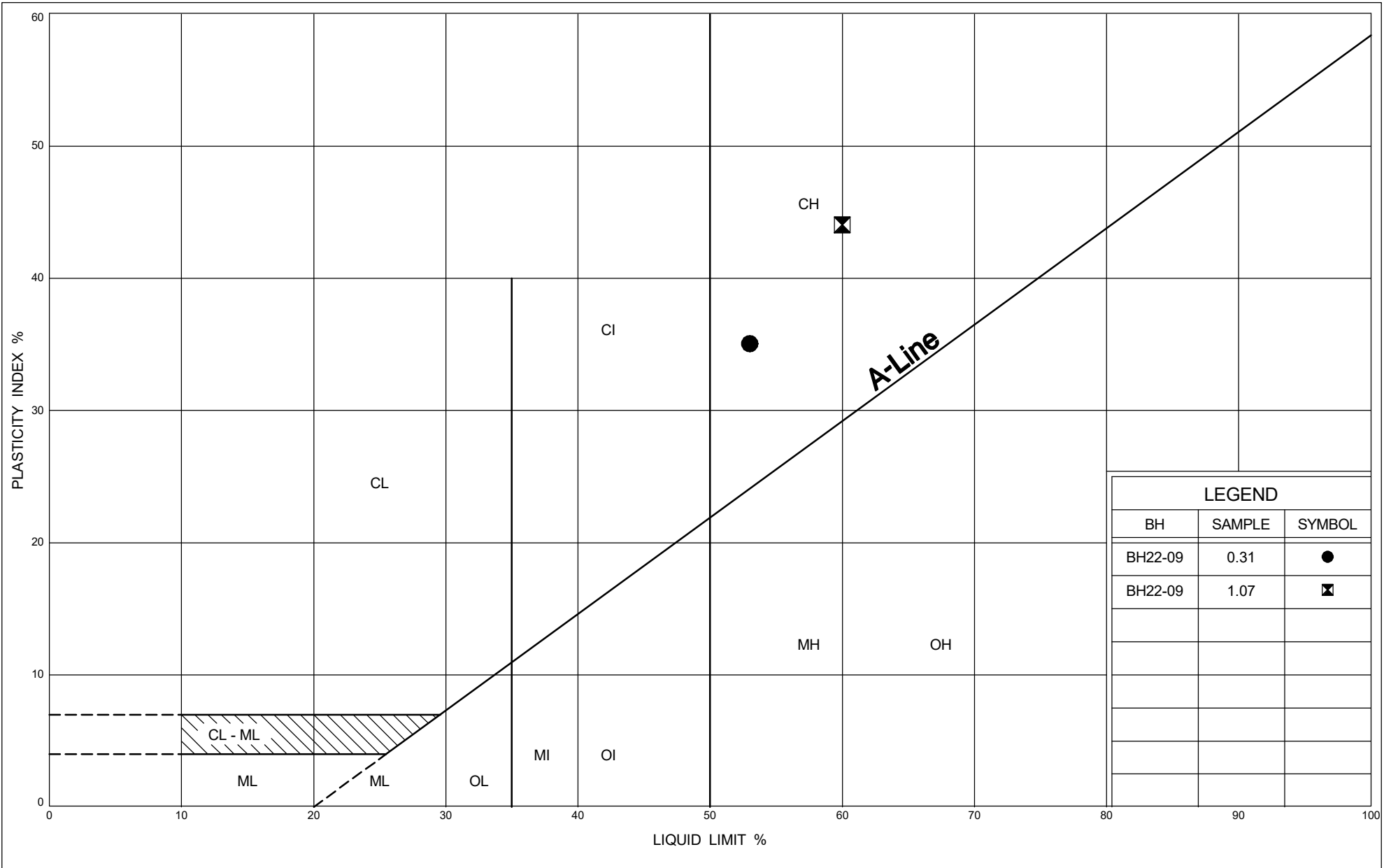
FIG No 6

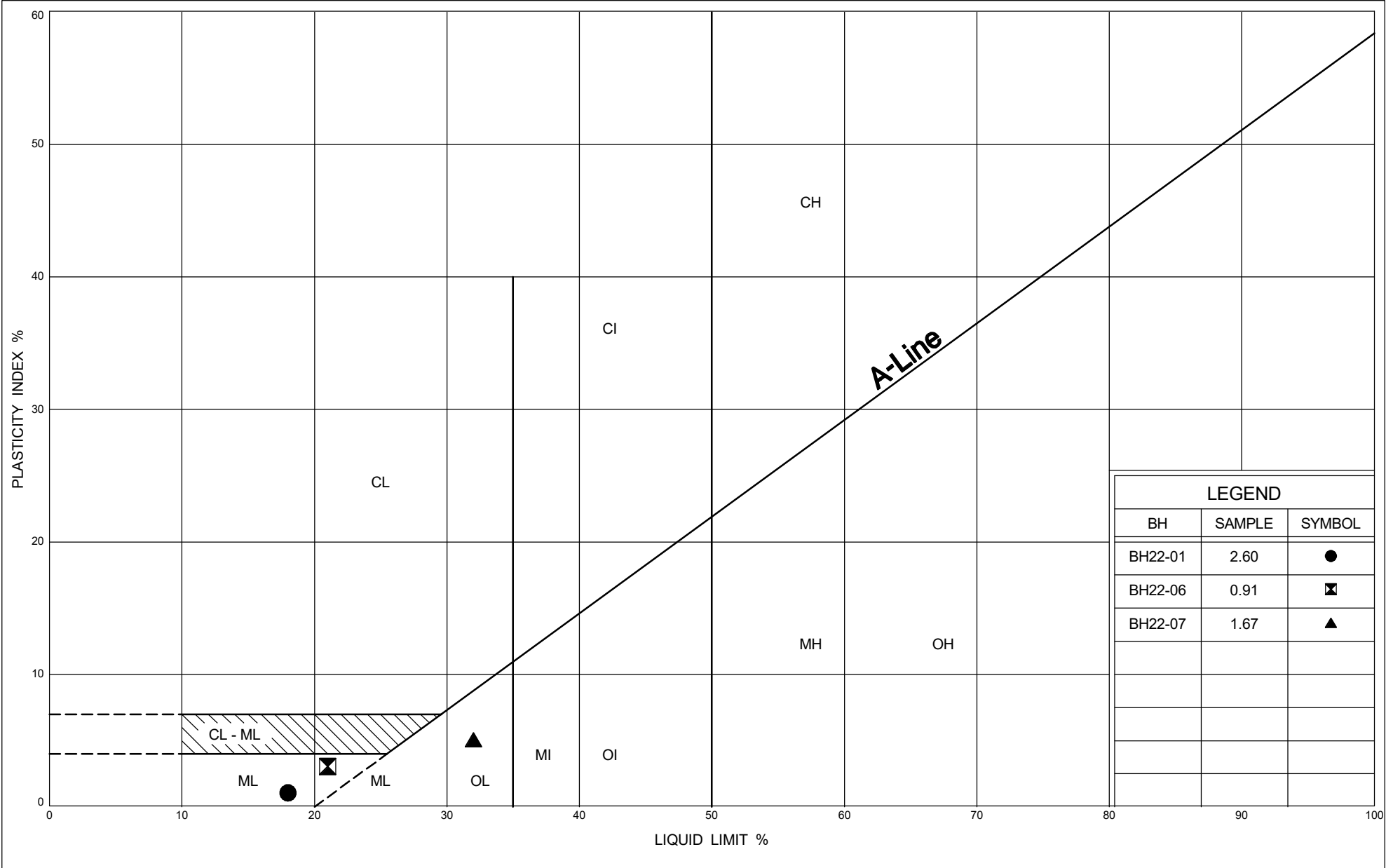
W P Assignment No. 6021-E-0019

Instability on Hwy 658

## Results of Atterberg Limits Tests







## Results of Chemical Tests



Your Project #: ADM-22019842-E0  
Site Location: REDDIT, ON  
Your C.O.C. #: N/A

**Attention: Ahileas Mitsopoulos**

exp Services Inc  
Thunder Bay Branch  
1142 Roland St  
Thunder Bay, ON  
CANADA P7B 5M4

**Report Date: 2022/10/24**  
Report #: R7355315  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BUREAU VERITAS JOB #: C2U3910**

**Received: 2022/10/18, 13:31**

Sample Matrix: Soil  
# Samples Received: 1

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	1	2022/10/21	2022/10/21	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	1	2022/10/21	2022/10/21	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	1	2022/10/21	2022/10/21	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	1	2022/10/19	2022/10/21	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	1	2022/10/21	2022/10/24	CAM SOP-00464	EPA 375.4 m

**Remarks:**

Bureau Veritas is accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Bureau Veritas are based upon recognized Provincial, Federal or US method compendia such as CCME, MELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Bureau Veritas' profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Bureau Veritas in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

Bureau Veritas liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Bureau Veritas has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Bureau Veritas, unless otherwise agreed in writing. Bureau Veritas is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.

Results relate to samples tested. When sampling is not conducted by Bureau Veritas, results relate to the supplied samples tested.

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Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.





Your Project #: ADM-22019842-E0  
Site Location: REDDIT, ON  
Your C.O.C. #: N/A

**Attention: Ahileas Mitsopoulos**

exp Services Inc  
Thunder Bay Branch  
1142 Roland St  
Thunder Bay, ON  
CANADA P7B 5M4

**Report Date: 2022/10/24**  
Report #: R7355315  
Version: 1 - Final

**CERTIFICATE OF ANALYSIS**

**BUREAU VERITAS JOB #: C2U3910**

**Received: 2022/10/18, 13:31**

Encryption Key



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RAPPORT AUTORISÉ**

Bureau Veritas

24 Oct 2022 15:25:44

Please direct all questions regarding this Certificate of Analysis to:

Julie Clement, Technical Account Manager

Email: Julie.CLEMENT@bureauveritas.com

Phone# (613)868-6079

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Total Cover Pages : 2

Page 2 of 8

Bureau Veritas 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.bvna.com

Microbiology testing is conducted at 6660 Campobello Rd. Chemistry testing is conducted at 6740 Campobello Rd.



BUREAU  
VERITAS

Bureau Veritas Job #: C2U3910

Report Date: 2022/10/24

exp Services Inc

Client Project #: ADM-22019842-E0

Site Location: REDDIT, ON

Sampler Initials: KM

### RESULTS OF ANALYSES OF SOIL

Bureau Veritas ID		UAY031			UAY031		
Sampling Date		2022/10/06 12:30			2022/10/06 12:30		
COC Number		N/A			N/A		
	UNITS	BH22-05-SS3A	RDL	QC Batch	BH22-05-SS3A Lab-Dup	RDL	QC Batch
<b>Calculated Parameters</b>							
Resistivity	ohm-cm	2100		8292439			
<b>Inorganics</b>							
Soluble (20:1) Chloride (Cl <sup>-</sup> )	ug/g	190	20	8297712	180	20	8297712
Conductivity	mS/cm	0.48	0.002	8297274			
Available (CaCl <sub>2</sub> ) pH	pH	7.26		8297661			
Soluble (20:1) Sulphate (SO <sub>4</sub> )	ug/g	<400 (1)	400	8297720			
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate (1) Due to colour interferences, sample required dilution. Detection limit was adjusted accordingly.							



BUREAU  
VERITAS

Bureau Veritas Job #: C2U3910  
Report Date: 2022/10/24

exp Services Inc  
Client Project #: ADM-22019842-E0  
Site Location: REDDIT, ON  
Sampler Initials: KM

## TEST SUMMARY

**Bureau Veritas ID:** UAY031  
**Sample ID:** BH22-05-SS3A  
**Matrix:** Soil

**Collected:** 2022/10/06  
**Shipped:**  
**Received:** 2022/10/18

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8297712	2022/10/21	2022/10/21	Samuel Law
Conductivity	AT	8297274	2022/10/21	2022/10/21	Surinder Rai
pH CaCl2 EXTRACT	AT	8297661	2022/10/21	2022/10/21	Taslima Aktar
Resistivity of Soil		8292439	2022/10/21	2022/10/21	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8297720	2022/10/21	2022/10/24	Alina Dobreanu

**Bureau Veritas ID:** UAY031 Dup  
**Sample ID:** BH22-05-SS3A  
**Matrix:** Soil

**Collected:** 2022/10/06  
**Shipped:**  
**Received:** 2022/10/18

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8297712	2022/10/21	2022/10/21	Samuel Law



### GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	9.0°C
-----------	-------

Results relate only to the items tested.



QUALITY ASSURANCE REPORT

exp Services Inc  
Client Project #: ADM-22019842-E0  
Site Location: REDDIT, ON  
Sampler Initials: KM

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
8297274	Conductivity	2022/10/21			106	90 - 110	<0.002	mS/cm	0.20	10
8297661	Available (CaCl2) pH	2022/10/21			100	97 - 103			0.39	N/A
8297712	Soluble (20:1) Chloride (Cl-)	2022/10/21	NC	70 - 130	104	70 - 130	<20	ug/g	0.86	35
8297720	Soluble (20:1) Sulphate (SO4)	2022/10/24	NC	70 - 130	106	70 - 130	<20	ug/g	NC	35

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).



BUREAU  
VERITAS

Bureau Veritas Job #: C2U3910  
Report Date: 2022/10/24

exp Services Inc  
Client Project #: ADM-22019842-E0  
Site Location: REDDIT, ON  
Sampler Initials: KM

## VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by:

---

Cristina Carriere, Senior Scientific Specialist

---

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### CHAIN OF CUSTODY RECORD

## Page 1 of 1

Rec'd In Thunder Bay

## Appendix E – Bedrock Core Photographs





Photograph E1. Bedrock core samples, BH22-01, Run 1 (top) and Run 2 (middle and bottom), October 2022



Photograph E2. Bedrock core samples, BH22-02, Run 1 (top) and Run 2 (bottom), October 2022



Photograph E3. Bedrock core samples, BH22-03, Run 1 (top and middle) and Run 2 (bottom), October 2022

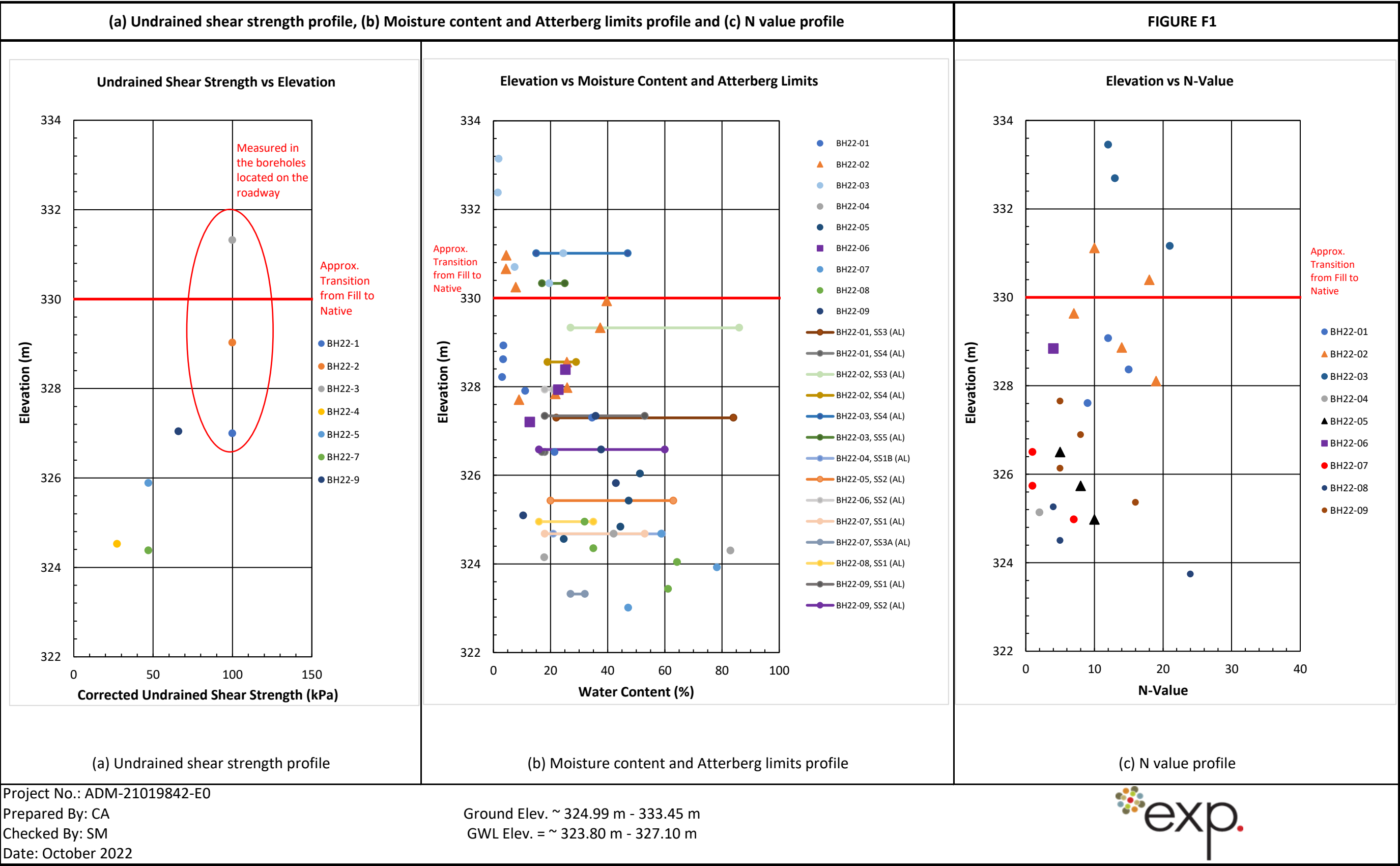


Photograph E4. Bedrock core samples, BH22-05, Run 1, October 2022

## Appendix F – Summary of Soil Properties and Seismic Hazard Values

## Summary of Soil Properties

Summary of Field Work and Laboratory Results



## Seismic Hazard Values



Government  
of Canada

Gouvernement  
du Canada

[Canada.ca](#) > [Natural Resources Canada](#) > [Earthquakes Canada](#)

# 2020 National Building Code of Canada Seismic Hazard Tool

**i** This application provides seismic values for the design of buildings in Canada under Part 4 of the National Building Code of Canada (NBC) 2020 as prescribed in Article 1.1.3.1. of Division B of the NBC 2020.

## Seismic Hazard Values

### User requested values

Code edition	NBC 2020
Site designation $X_S$	$X_C$
Latitude (°)	49.976
Longitude (°)	-94.393

Please select one of the tabs below.

NBC 2020

Additional Values

Plots

API

Background Information

The 5%-damped spectral acceleration ( $S_a(T, X)$ , where  $T$  is the period, in s, and  $X$  is the site designation) and peak ground acceleration ( $PGA(X)$ ) values are given in units of acceleration due to gravity ( $g$ ,  $9.81 \text{ m/s}^2$ ). Peak

ground velocity, (PGV(X)) values are given in m/s. Probability is expressed in terms of percent exceedance in 50 years. Further information on the calculation of seismic hazard is provided under the *Background Information* tab.

The 2%-in-50-year seismic hazard values are provided in accordance with Article 4.1.8.4. of the NBC 2020. The 5%- and 10%-in-50-year values are provided for additional performance checks in accordance with Article 4.1.8.23. of the NBC 2020.

See the *Additional Values* tab for additional seismic hazard values, including values for other site designations, periods, and probabilities not defined in the NBC 2020.

**NBC 2020 - 2%/50 years (0.000404 per annum) probability**

$S_a(0.2, X_C)$	$S_a(0.5, X_C)$	$S_a(1.0, X_C)$	$S_a(2.0, X_C)$	$S_a(5.0, X_C)$	$S_a(10.0, X_C)$	PGA( $X_C$ )	PGV( $X_C$ )
0.0982	0.0582	0.0271	0.0106	0.00214	0.000698	0.0485	0.0308

The log-log interpolated 2%/50 year  $S_a(4.0, X_C)$  value is : **0.0032**

▼ Tables for 5% and 10% in 50 year values

**NBC 2020 - 5%/50 years (0.001 per annum) probability**

$S_a(0.2, X_C)$	$S_a(0.5, X_C)$	$S_a(1.0, X_C)$	$S_a(2.0, X_C)$	$S_a(5.0, X_C)$	$S_a(10.0, X_C)$	PGA( $X_C$ )	PGV( $X_C$ )
0.0512	0.0306	0.0137	0.00505	0.000943	0.000303	0.0242	0.0152

The log-log interpolated 5%/50 year  $S_a(4.0, X_C)$  value is : **0.0014**

**NBC 2020 - 10%/50 years (0.0021 per annum) probability**

$S_a(0.2, X_C)$	$S_a(0.5, X_C)$	$S_a(1.0, X_C)$	$S_a(2.0, X_C)$	$S_a(5.0, X_C)$	$S_a(10.0, X_C)$	PGA( $X_C$ )	PGV( $X_C$ )
-----------------	-----------------	-----------------	-----------------	-----------------	------------------	--------------	--------------



$S_a(0.2, X_C)$	$S_a(0.5, X_C)$	$S_a(1.0, X_C)$	$S_a(2.0, X_C)$	$S_a(5.0, X_C)$	$S_a(10.0, X_C)$	PGA( $X_C$ )	PGV( $X_C$ )
0.0288	0.0171	0.00727	0.0025	0.000429	0.000132	0.0131	0.00794

The log-log interpolated 10%/50 year  $S_a(4.0, X_C)$  value is : **0.0007**

Download CSV

← Go back to the [seismic hazard calculator form](#)

**Date modified:** 2021-04-06

## Appendix G – Slope Stability Analysis

Before 2022 Failure

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 South Section - Sta. 21+021  
 Existing Conditions - Drained - Static  
 High Groundwater Level/Saturated Clay

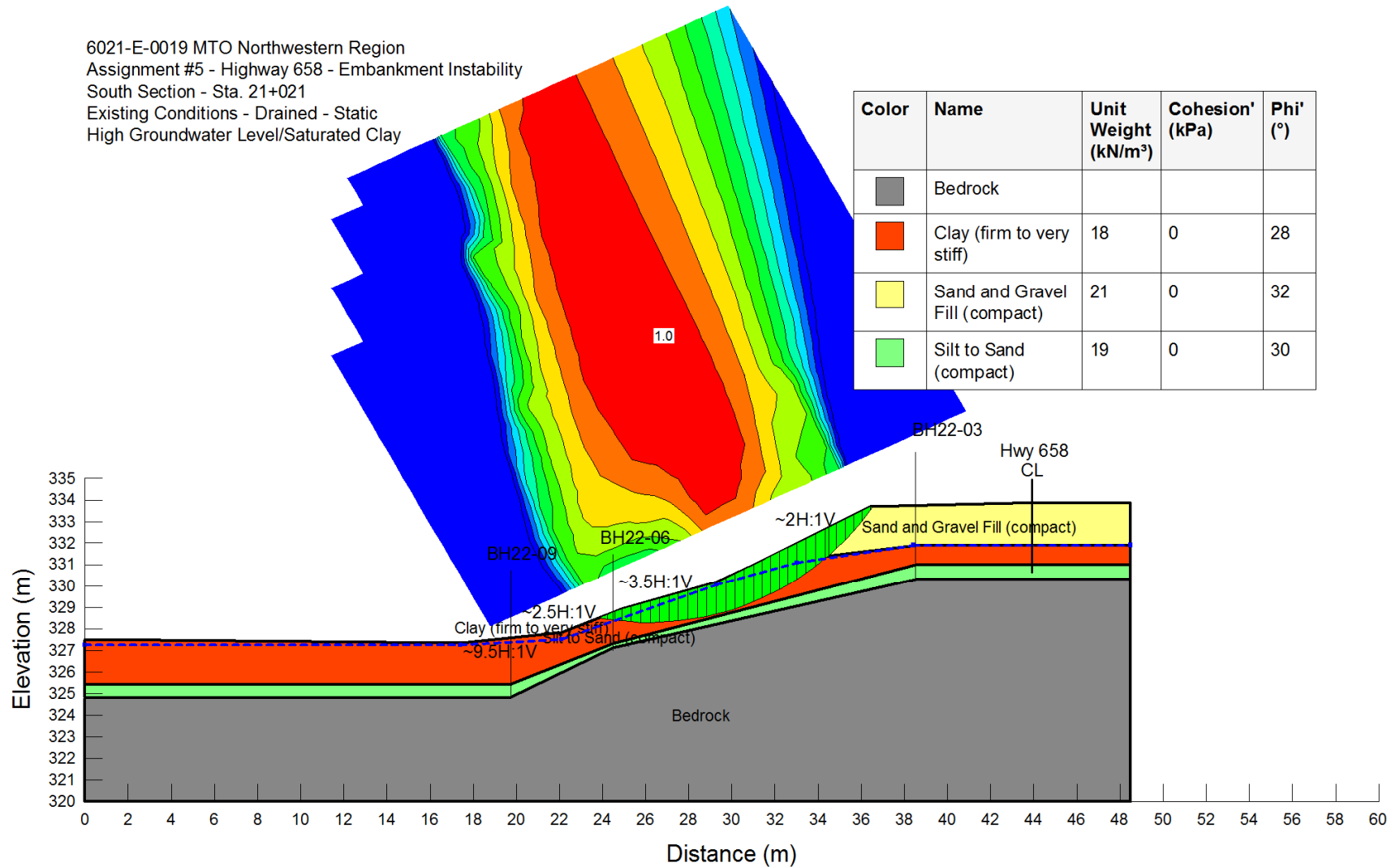






Figure G1. South Section B-B' (Sta. 21+021) – Back Analysis for Clay Friction Angle, High GWL – Drained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 South Section - Sta. 21+021  
 Existing Conditions - Undrained - Static  
 High Groundwater Level/Saturated Clay

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Cohesion (kPa)
	Bedrock				
	Clay (firm to very stiff)	18			10
	Sand and Gravel Fill (compact)	21	0	32	
	Silt to Sand (compact)	19	0	30	

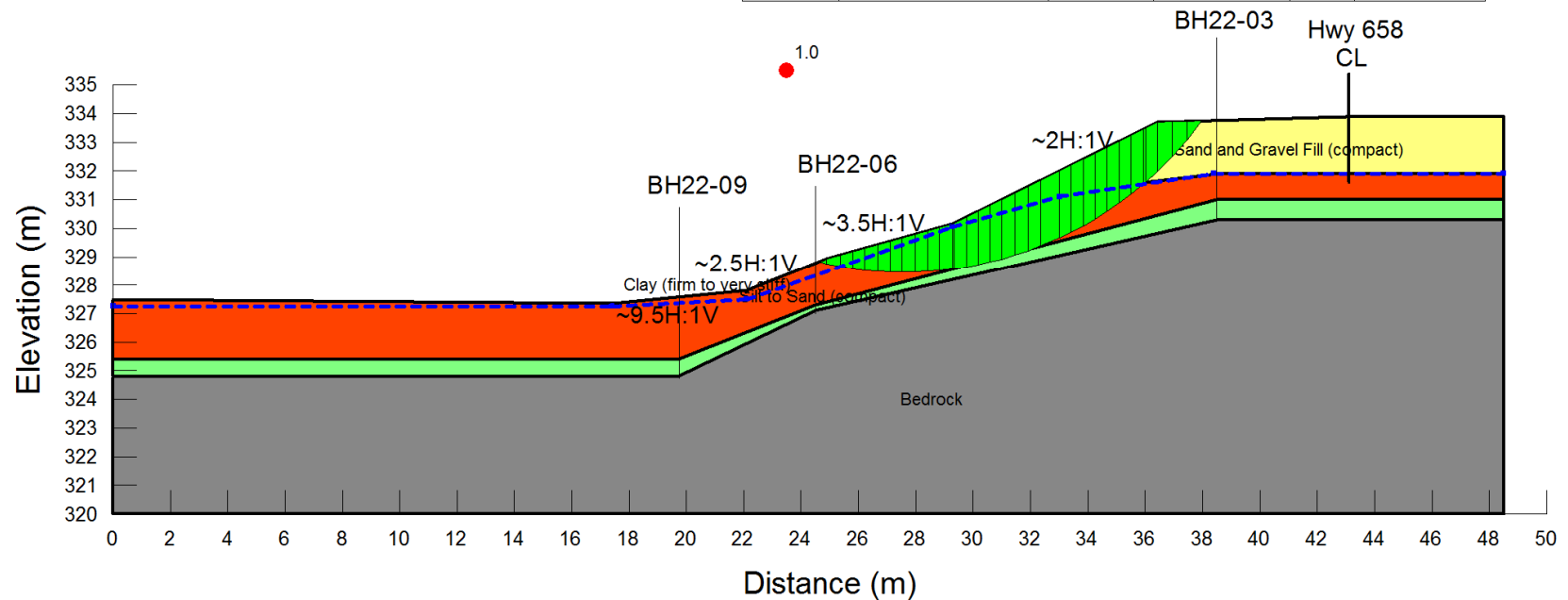






Figure G2. South Section B-B' (Sta. 21+021) – Back Analysis for Clay Undrained Shear Strength, High GWL – Undrained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained - Static  
 High Groundwater Level/Saturated Clay  
 Assumed Original Slope Profile

Color	Name	Unit Weight (kN/m <sup>3</sup> )	Cohesion' (kPa)	Phi' (°)
	Bedrock			
	Clay (firm to very stiff)	18	0	28
	Sand and Gravel Fill (compact)	21	0	32
	Silt to Sand (loose to compact)	19	0	30

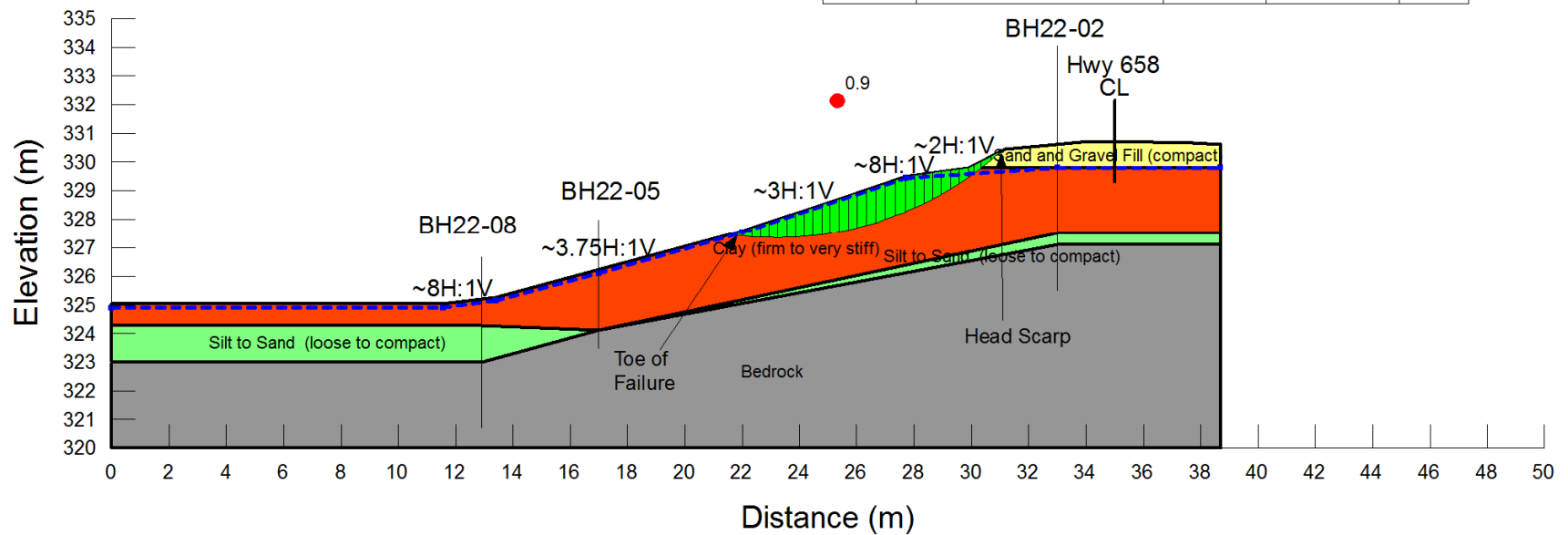






Figure G3. Central Section C-C' (Sta. 21+043) – Back Analysis for Clay Friction Angle Using Assumed Original Slope Profile, High GWL – Drained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Undrained - Static  
 High Groundwater Level/Saturated Clay  
 Assumed Original Slope Profile

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Cohesion (kPa)
	Bedrock				
	Clay (firm to very stiff)	18			6
	Sand and Gravel Fill (compact)	21	0	32	
	Silt to Sand (loose to compact)	19	0	30	

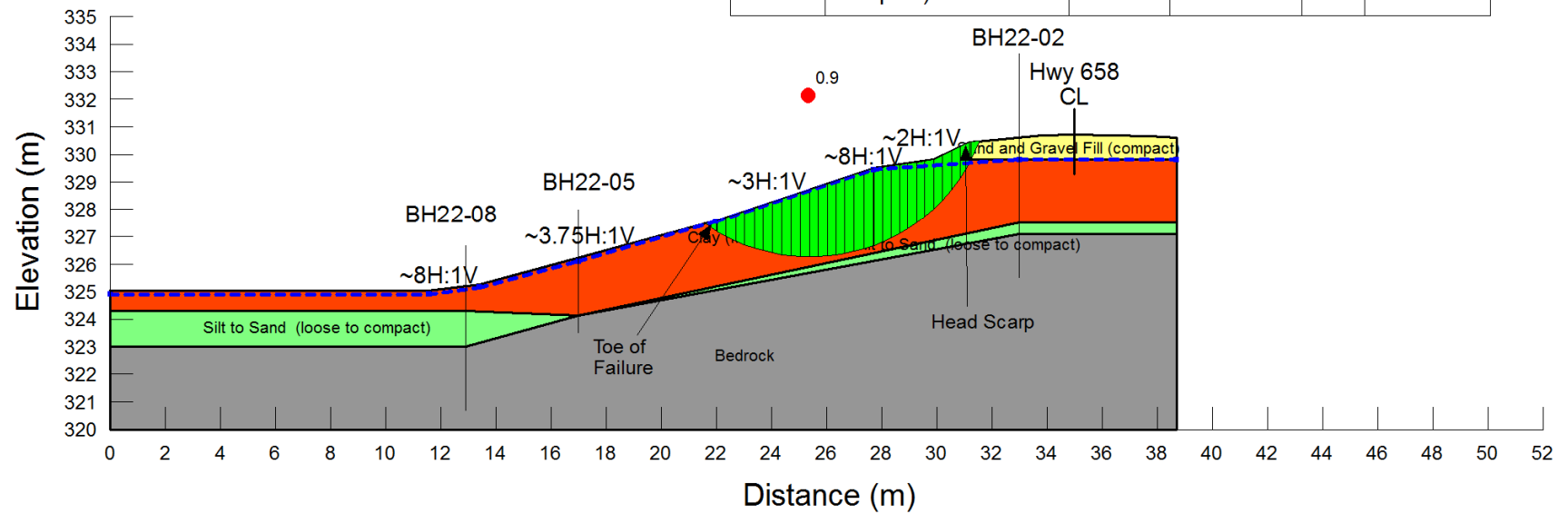


Figure G4. Central Section C-C' (Sta. 21+043) – Back Analysis for Clay Undrained Shear Strength Using Assumed Original Slope Profile, High GWL – Undrained Static Conditions



6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained - Static  
 Low Groundwater Level  
 Assumed Original Slope Profile

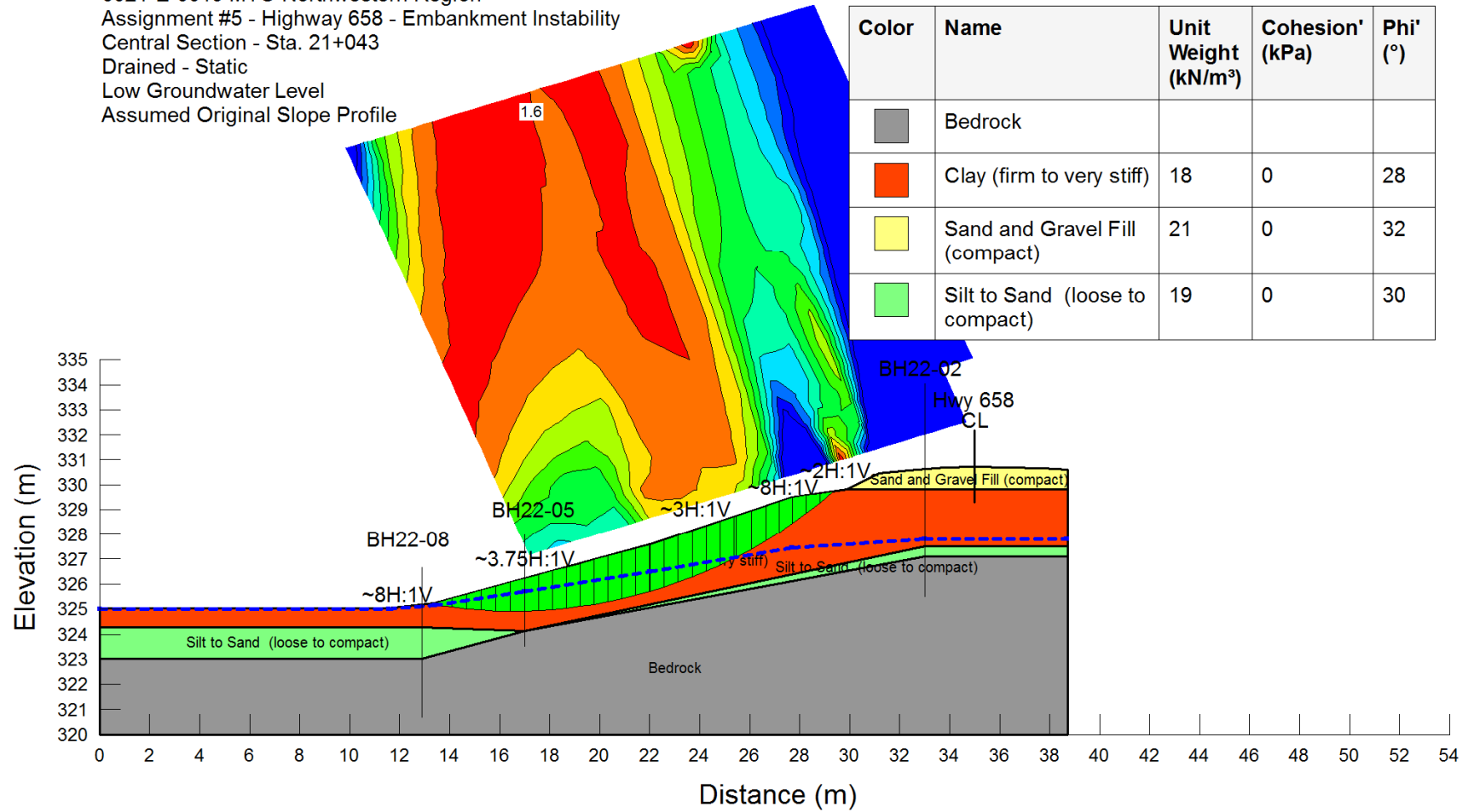






Figure G5. Central Section C-C' (Sta. 21+043) – Slope Prior to 2022 Failure, Assumed Original Slope Profile, Low GWL – Drained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Undrained - Static  
 Low Groundwater Level  
 Assumed Original Slope Profile

Color	Name	Unit Weight (kN/m <sup>3</sup> )	Cohesion' (kPa)	Phi' (°)	Cohesion (kPa)
	Bedrock				
	Clay (firm to very stiff)	18			6
	Sand and Gravel Fill (compact)	21	0	32	
	Silt to Sand (loose to compact)	19	0	30	

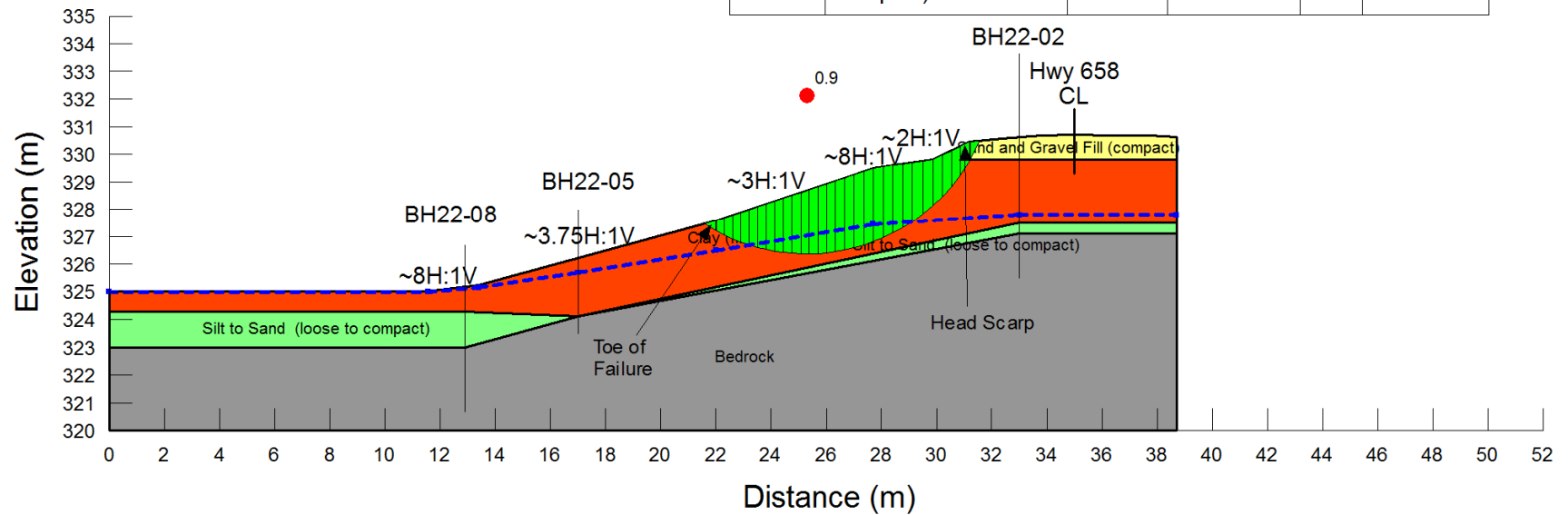


Figure G6. Central Section C-C' (Sta. 21+043) – Slope Prior to 2022 Failure, Assumed Original Slope Profile, Low GWL – Undrained Static Conditions

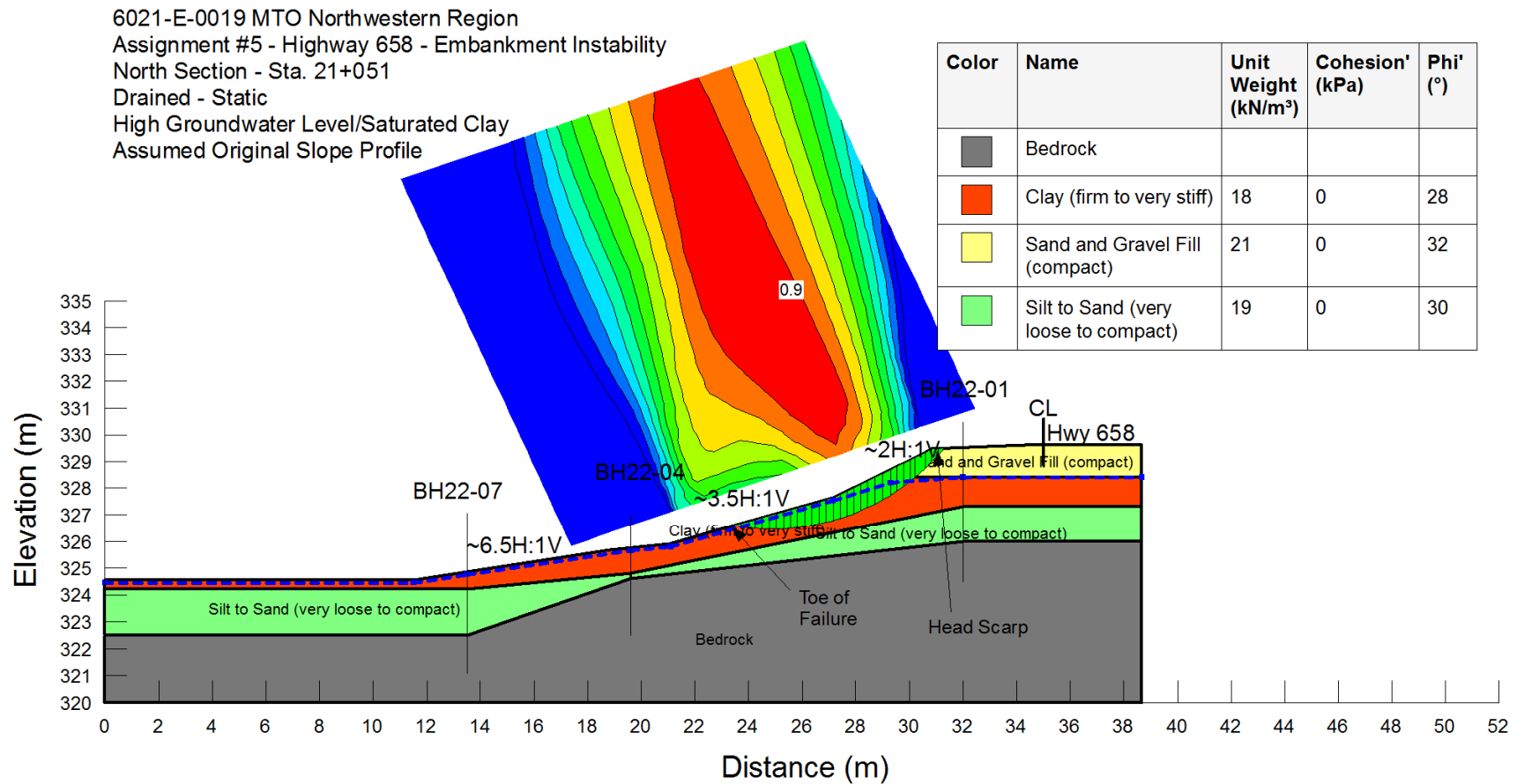


Figure G7. North Section D-D' (Sta. 21+051) – Back Analysis for Clay Friction Angle Using Assumed Original Slope Profile, High GWL – Drained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 North Section - Sta. 21+051  
 Undrained - Static  
 High Groundwater Level/Saturated Clay  
 Assumed Original Slope Profile

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Cohesion (kPa)
Gray	Bedrock				
Red	Clay (firm to very stiff)	18			6
Yellow	Sand and Gravel Fill (compact)	21	0	32	
Green	Silt to Sand (very loose to compact)	19	0	30	

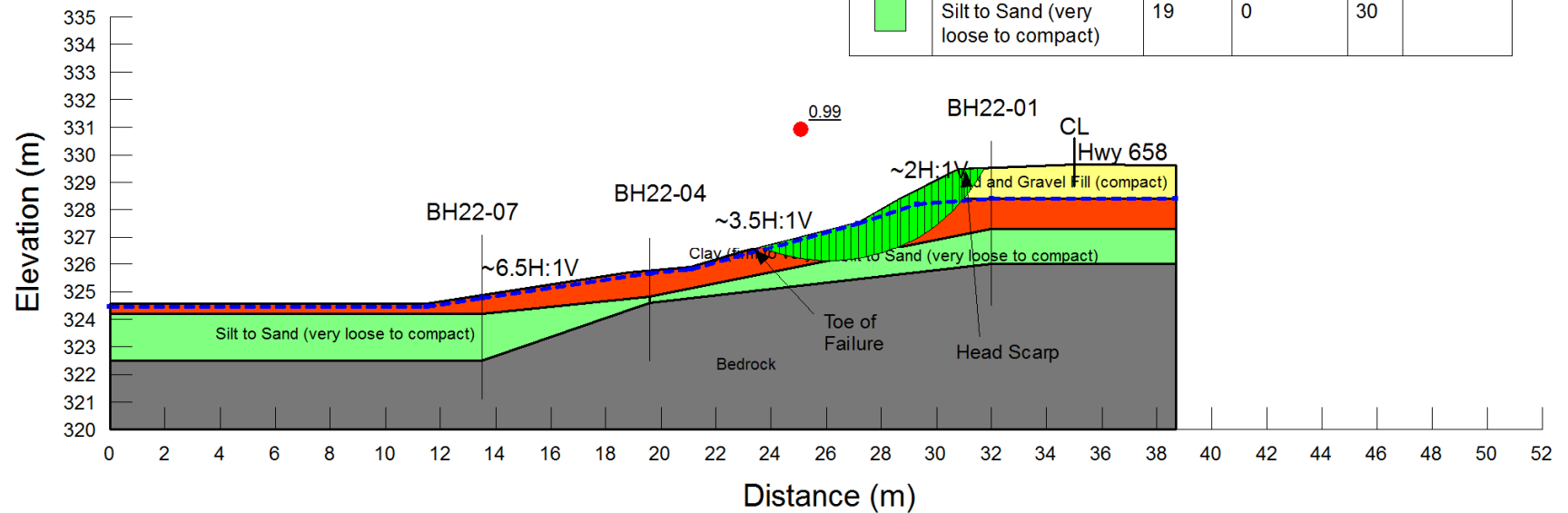


Figure G8. North Section D-D' (Sta. 21+051) – Back Analysis for Clay Undrained Shear Strength Using Assumed Original Slope Profile, High GWL – Undrained Static Conditions

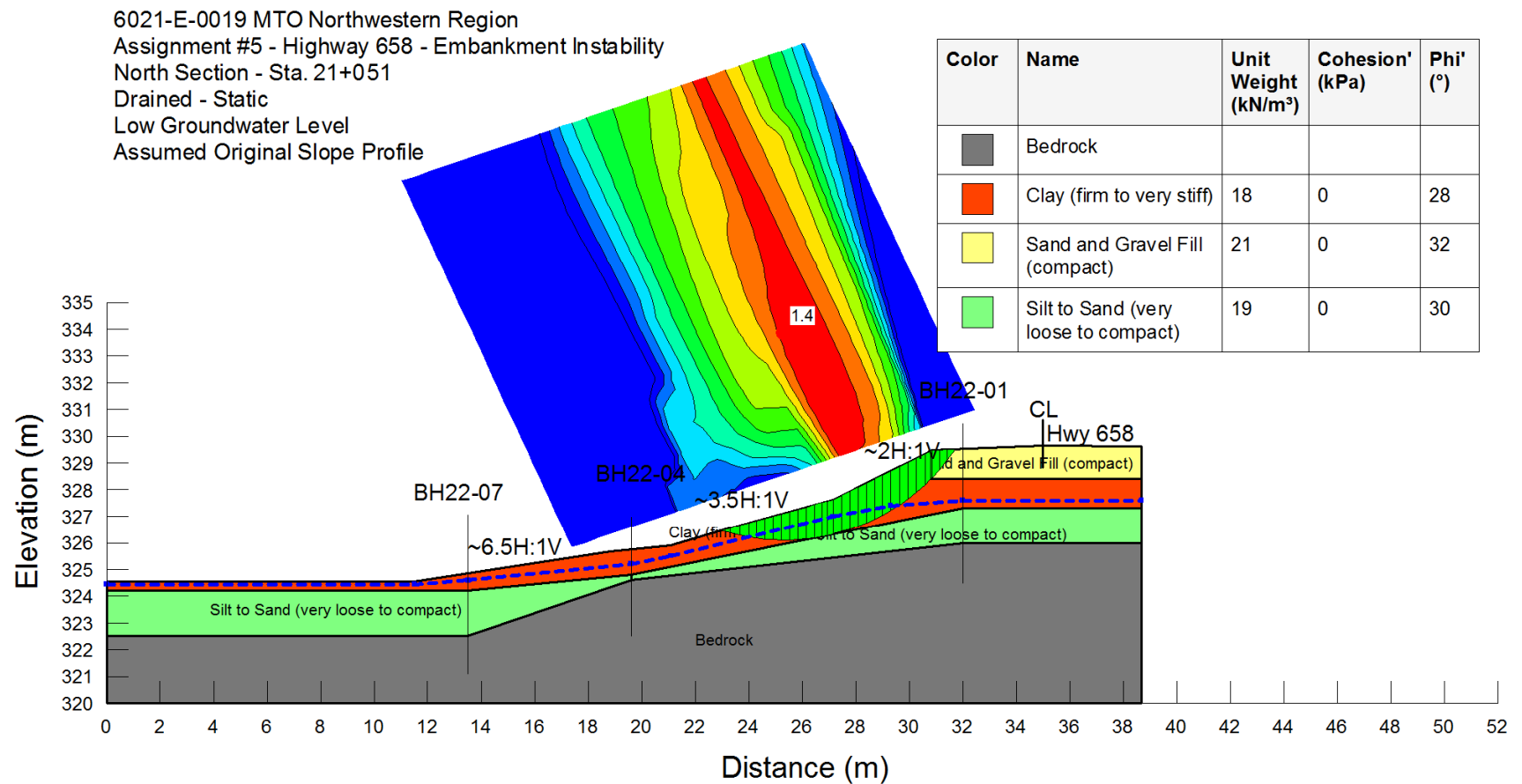


Figure G9. North Section D-D' (Sta. 21+051) – Slope Prior to 2022 Failure, Assumed Original Slope Profile, Low GWL – Drained Static Conditions

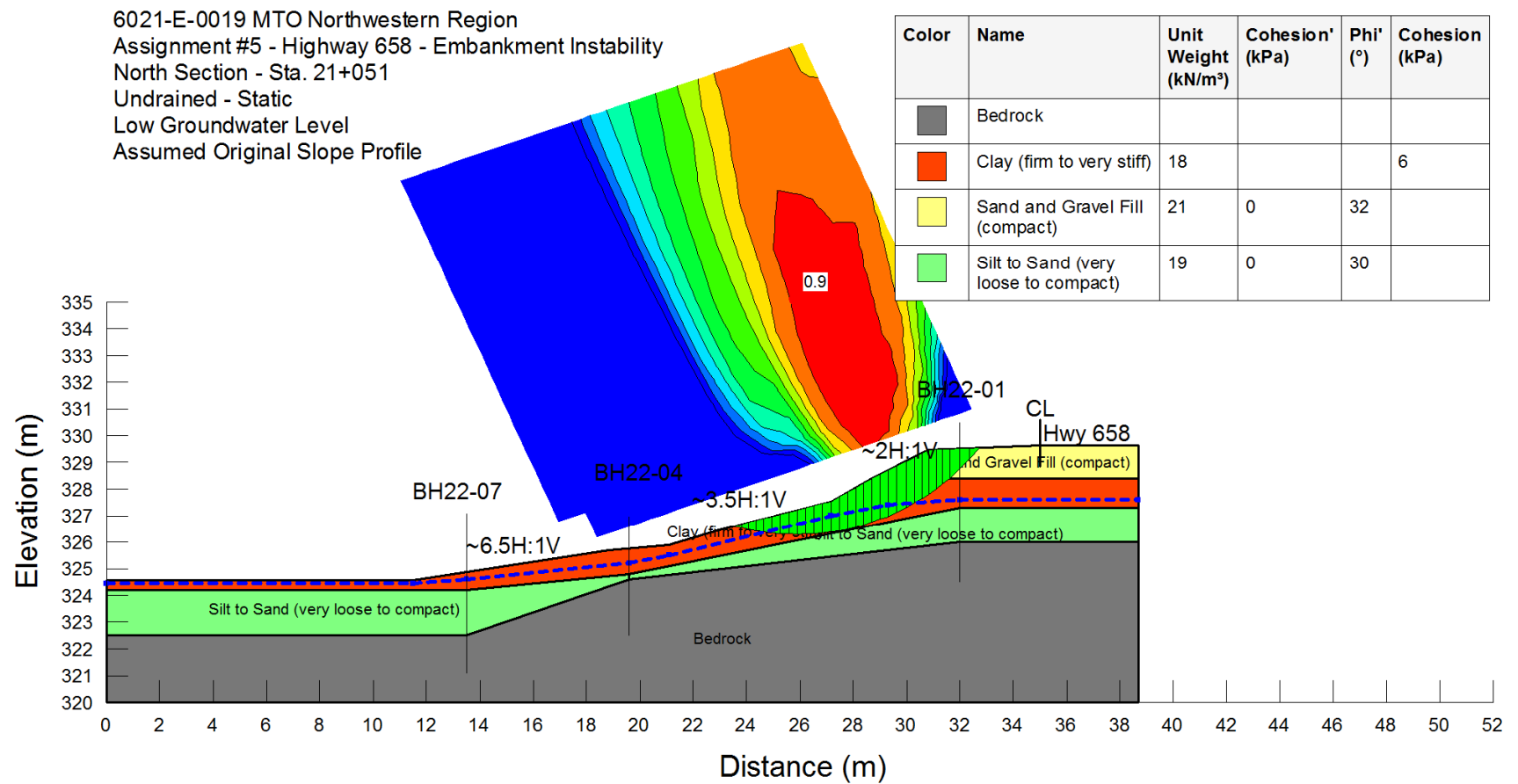


Figure G10. North Section D-D' (Sta. 21+051) – Slope Prior to 2022 Failure, Assumed Original Slope Profile, Low GWL – Undrained Static Conditions



After 2022 Failure

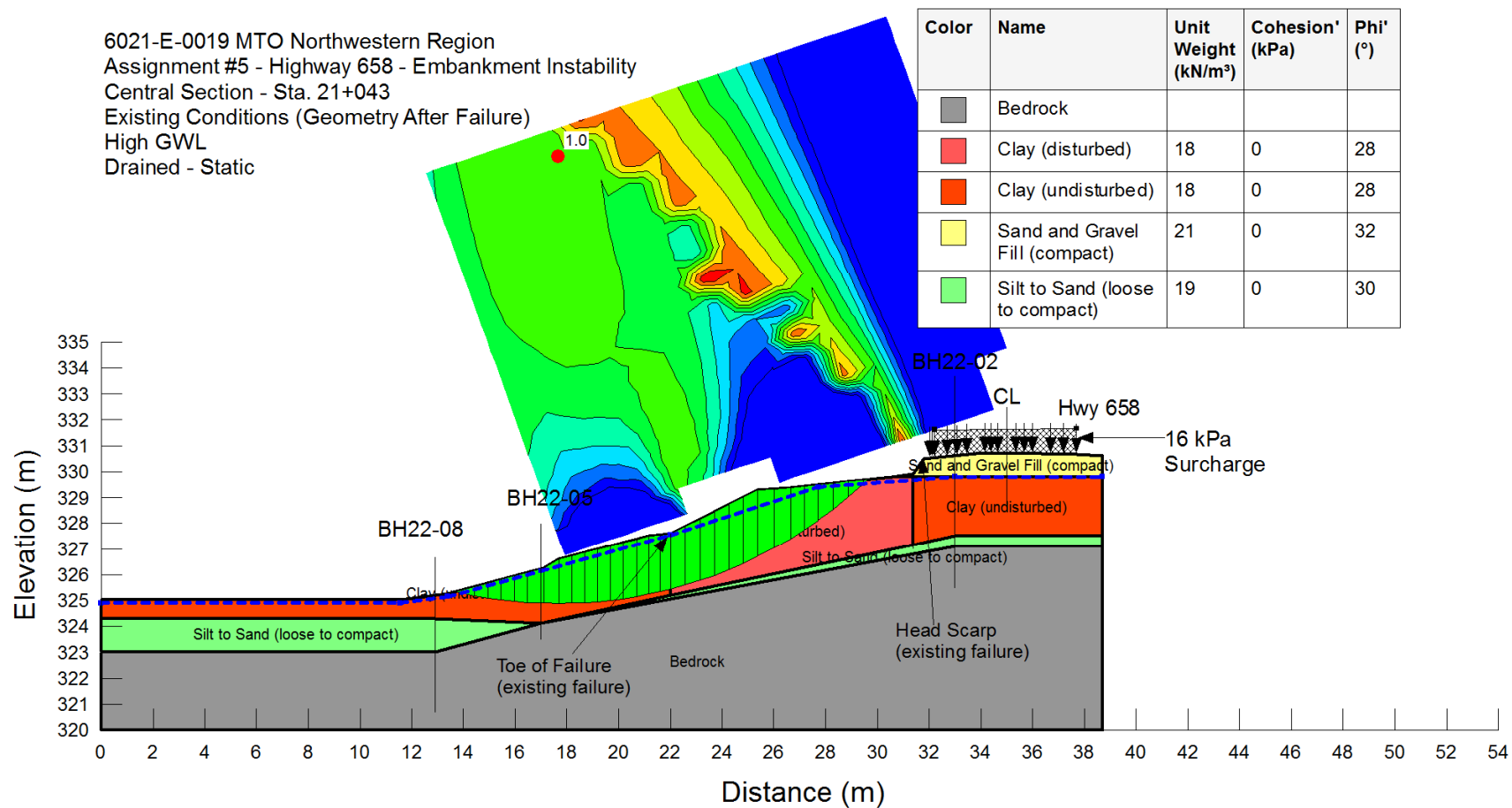


Figure G11. Central Section C-C' (Sta. 21+043) – Stability of Slope After 2022 Failure (Current Slope Geometry), High GWL – Drained Static Conditions

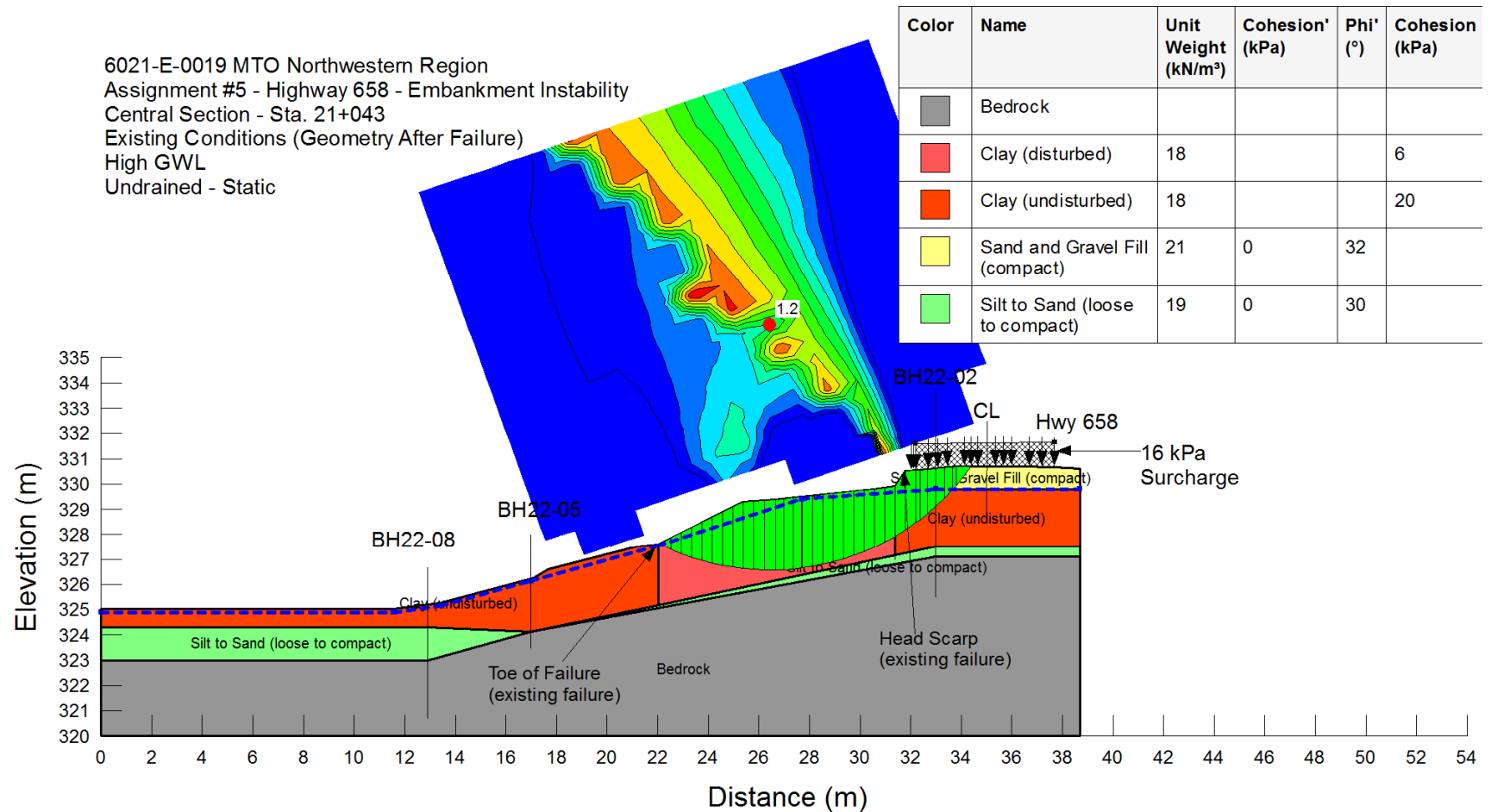


Figure G12. Central Section C-C' (Sta. 21+043) – Stability of Slope After 2022 Failure (Current Slope Geometry), High GWL – Undrained Static Conditions

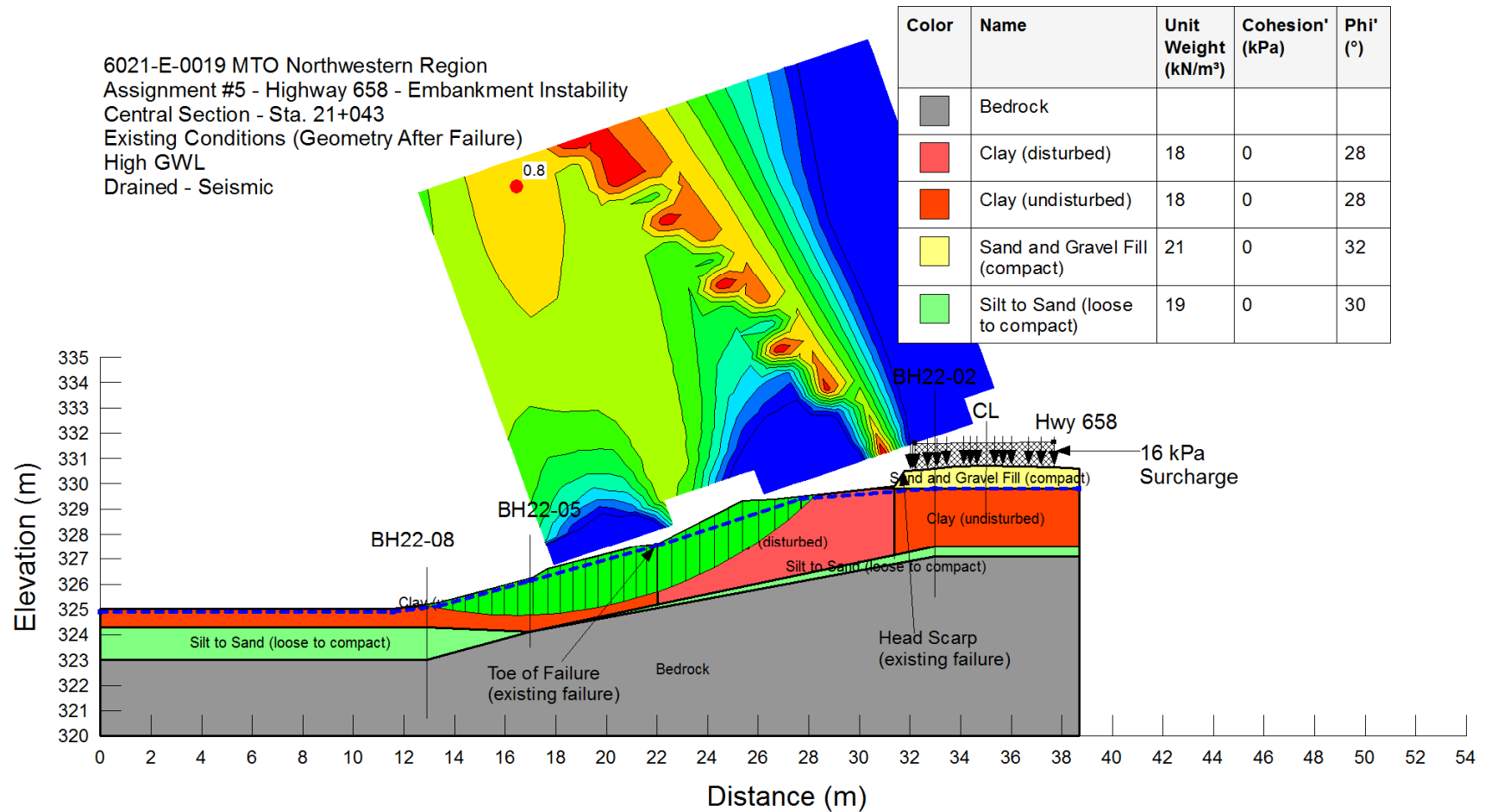


Figure G13. Central Section C-C' (Sta. 21+043) – Stability of Slope After 2022 Failure (Current Slope Geometry), High GWL – Drained Seismic Conditions

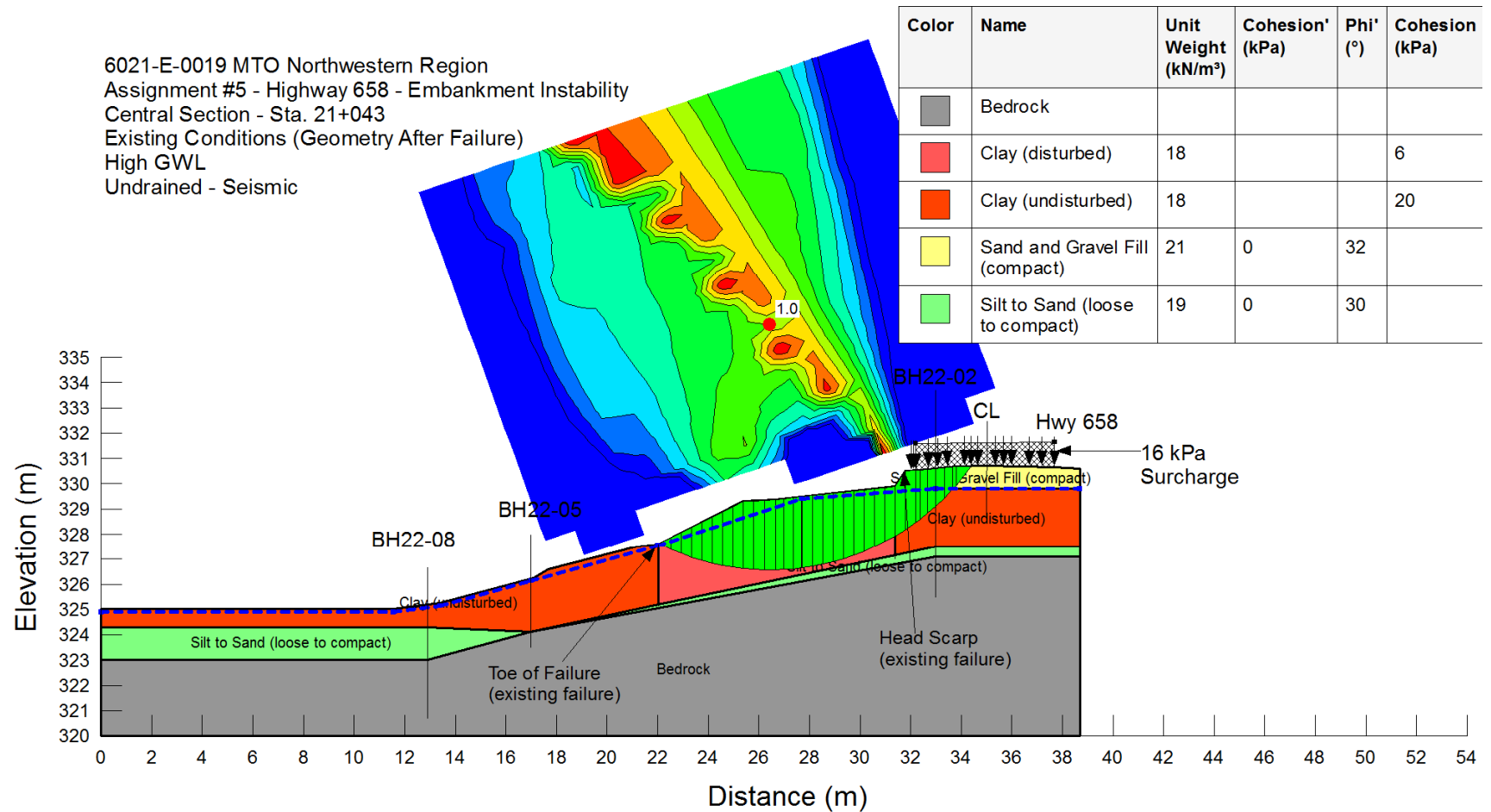


Figure G14. Central Section C-C' (Sta. 21+043) – Stability of Slope After 2022 Failure (Current Slope Geometry), High GWL – Undrained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 North Section - Sta. 21+051  
 Existing Conditions (Geometry After Failure)  
 Drained - Static

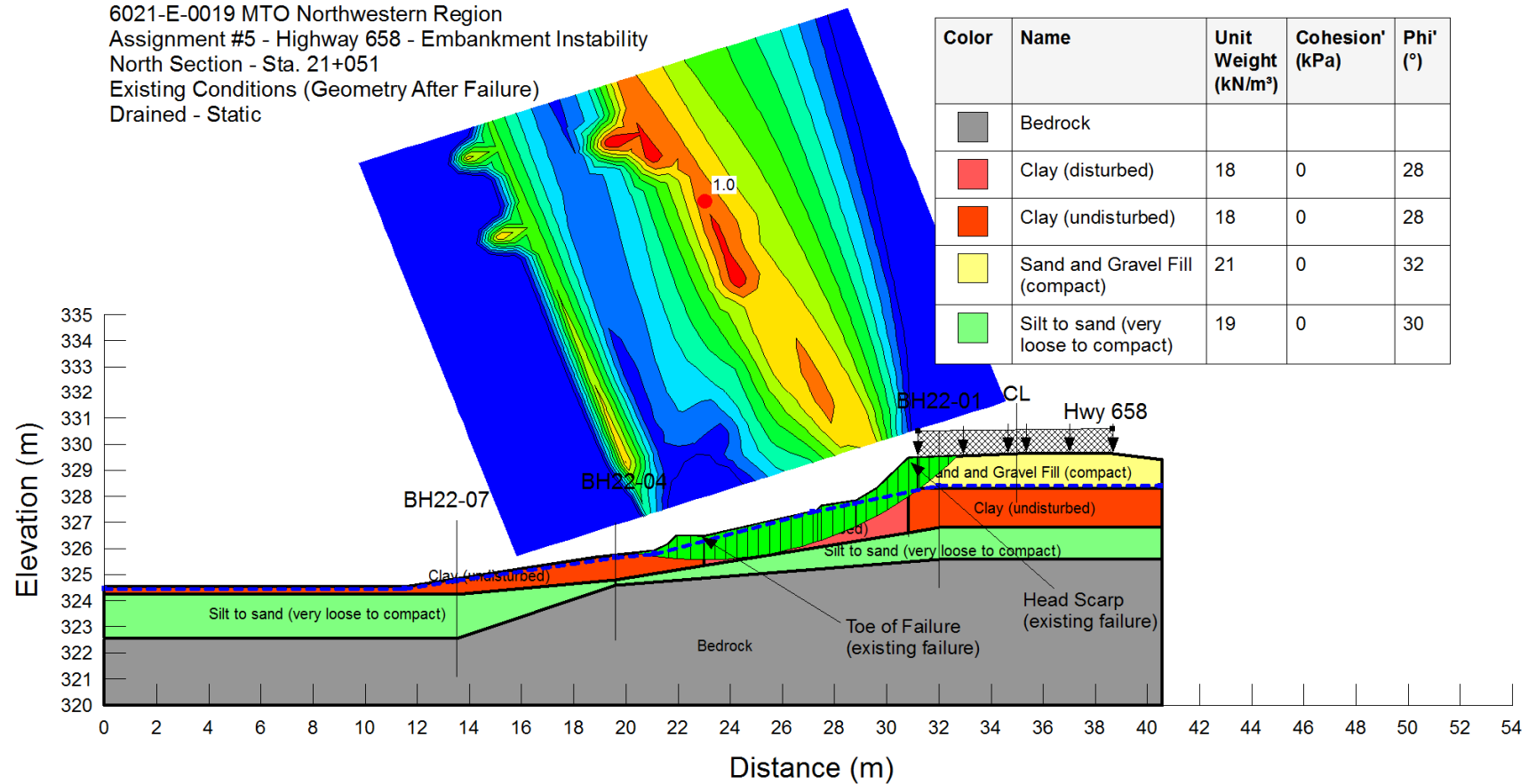


Figure G15. North Section D-D' (Sta. 21+051) – Stability of Slope After 2022 Failure (Current Slope Geometry), High GWL – Drained Static Conditions



6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 North Section - Sta. 21+051  
 Existing Conditions (Geometry After Failure)  
 Undrained - Static

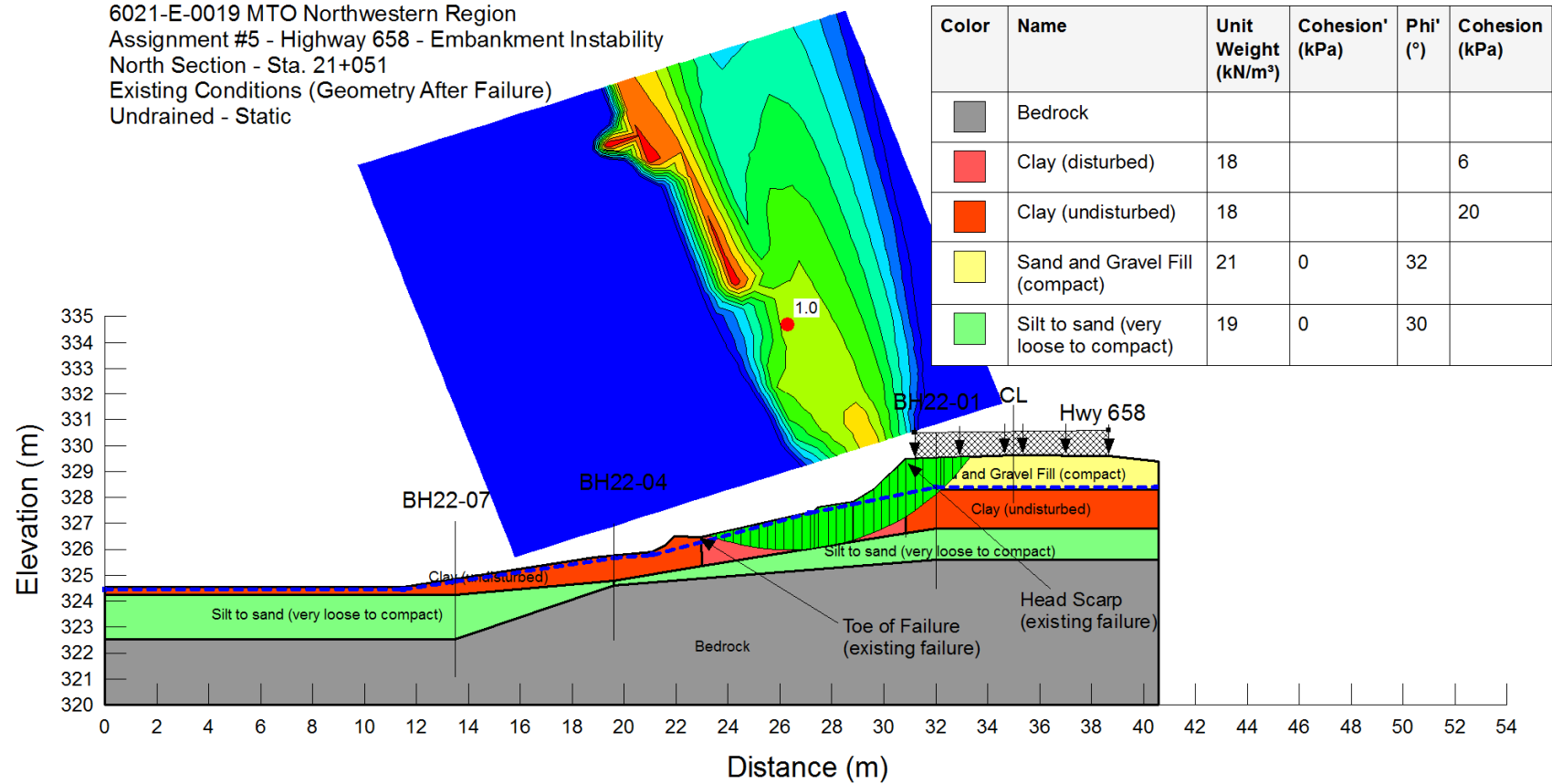


Figure G16. North Section D-D' (Sta. 21+051) – Stability of Slope After 2022 Failure (Current Slope Geometry), High GWL – Undrained Static Conditions

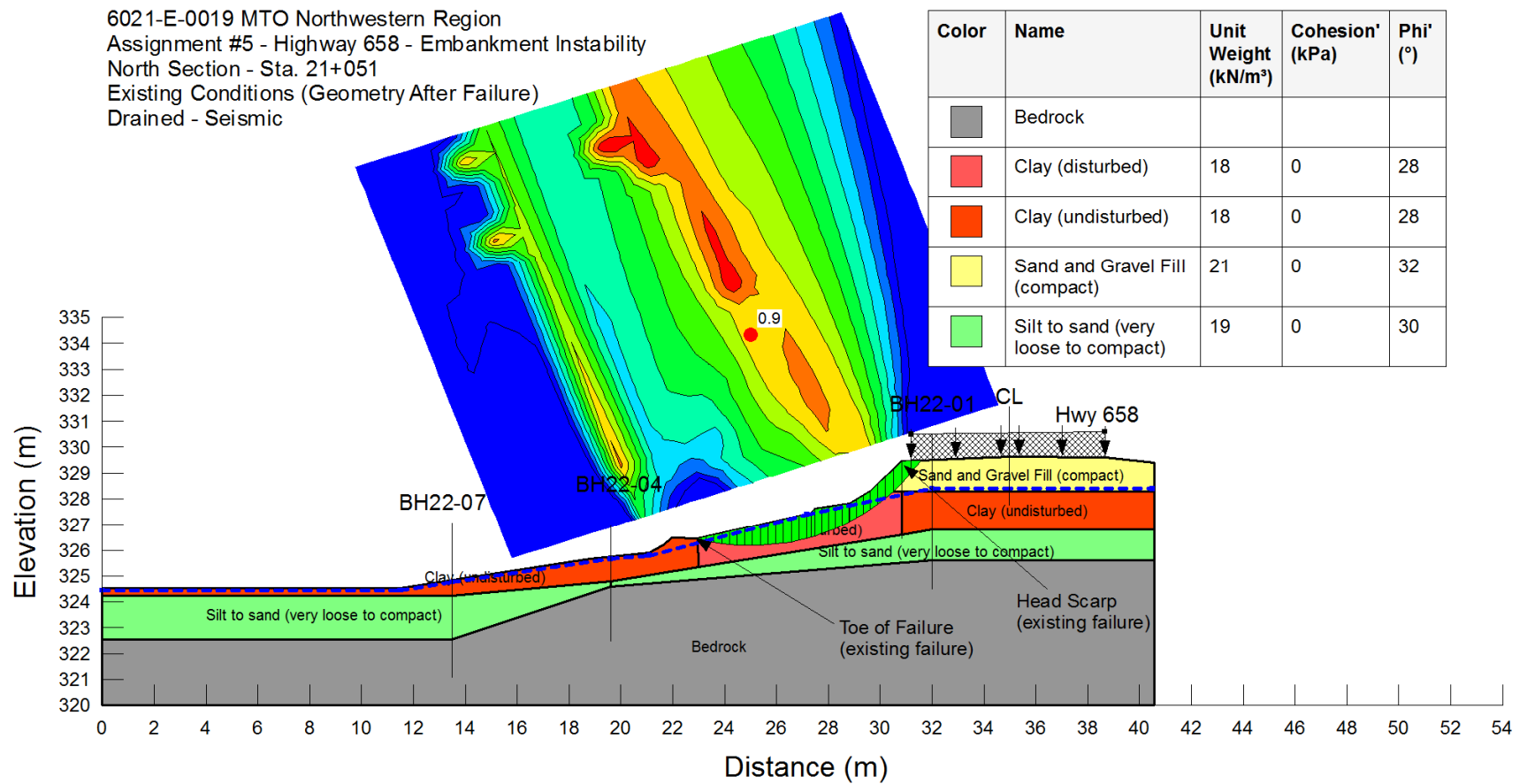


Figure G17. North Section D-D' (Sta. 21+051) – Stability of Slope After 2022 Failure (Current Slope Geometry), High GWL – Drained Seismic Conditions

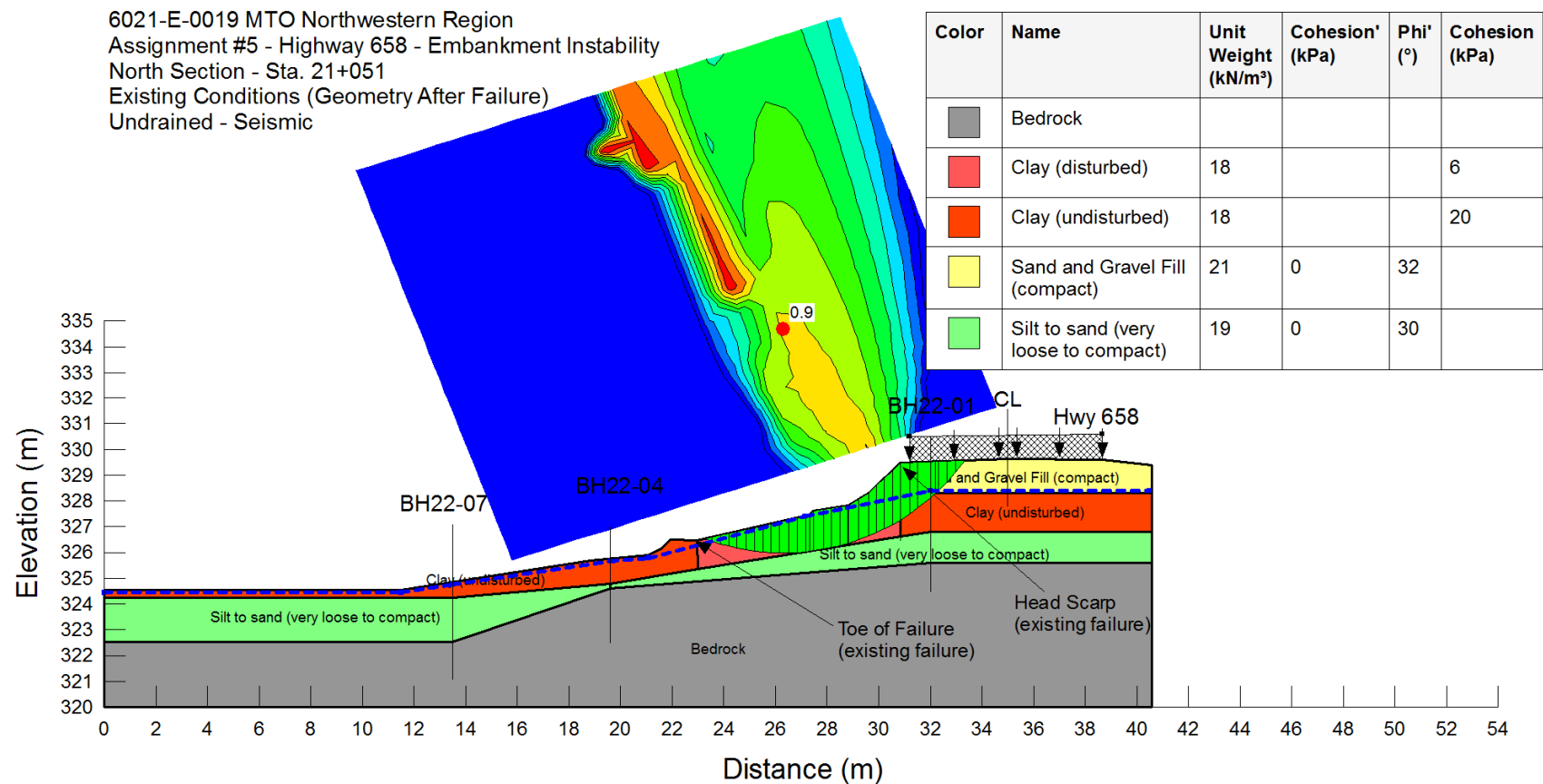


Figure G18. North Section D-D' (Sta. 21+051) – Stability of Slope After 2022 Failure (Current Slope Geometry), High GWL – Undrained Seismic Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Section EE - Sta. 21+050  
 Existing Conditions (Geometry After Failure)  
 Drained - Static

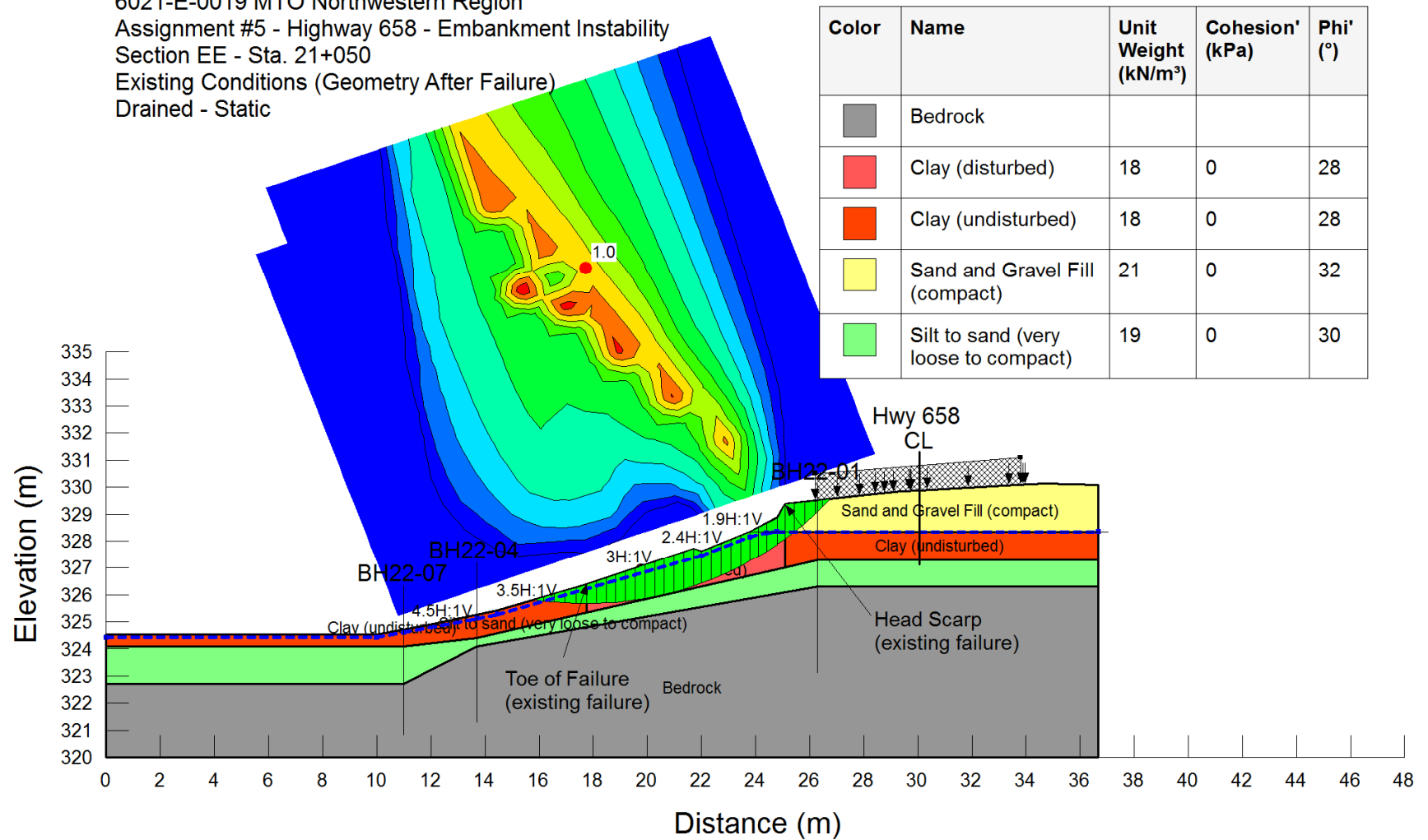


Figure G19. North Section E-E' (Sta. 21+050) – Stability of Slope After 2022 Failure (Current Slope Geometry), High GWL – Drained Static Conditions

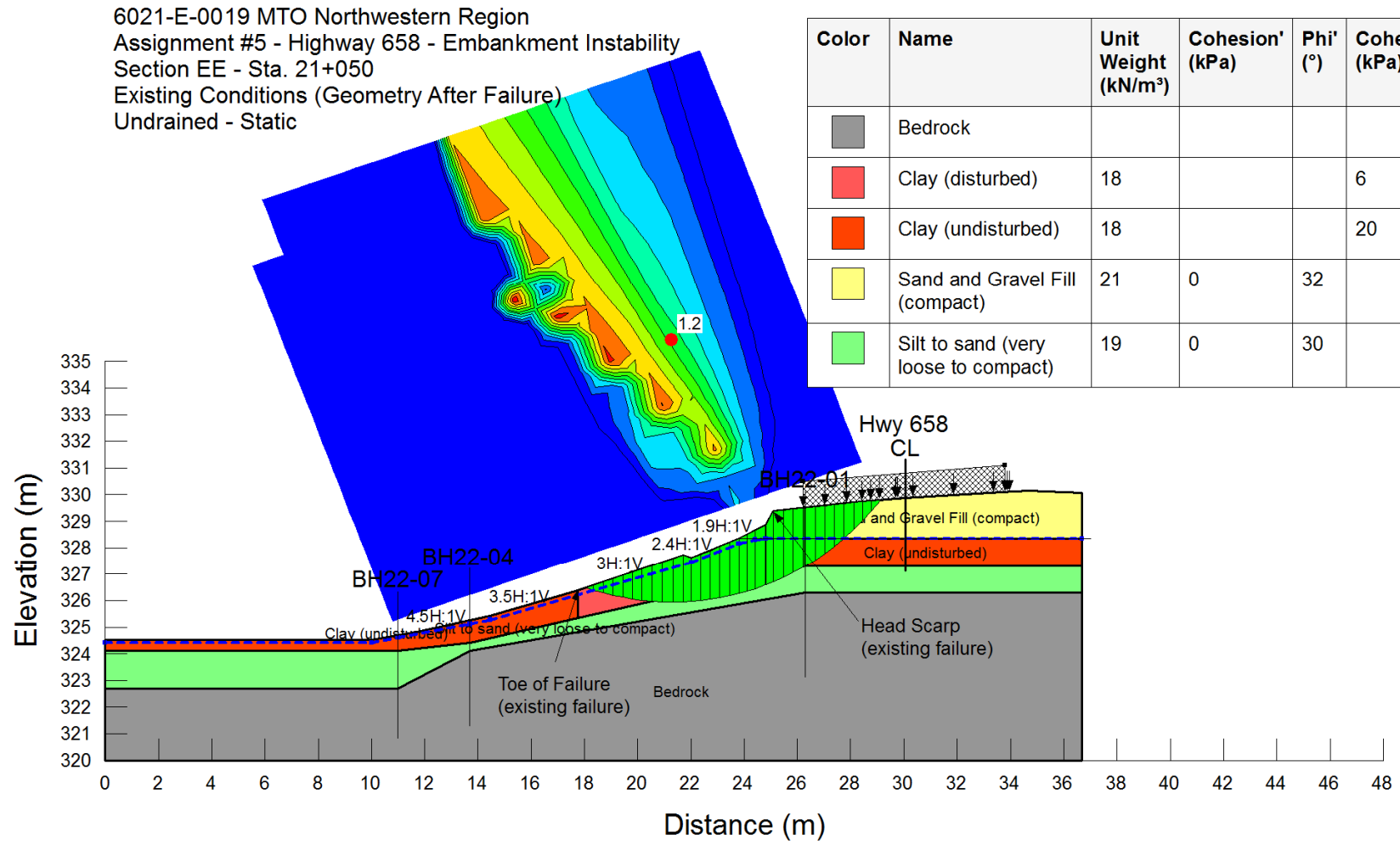


Figure G20. North Section E-E' (Sta. 21+050) – Stability of Slope After 2022 Failure (Current Slope Geometry), High GWL – Undrained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Section EE - Sta. 21+050  
 Existing Conditions (Geometry After Failure)  
 Drained - Seismic

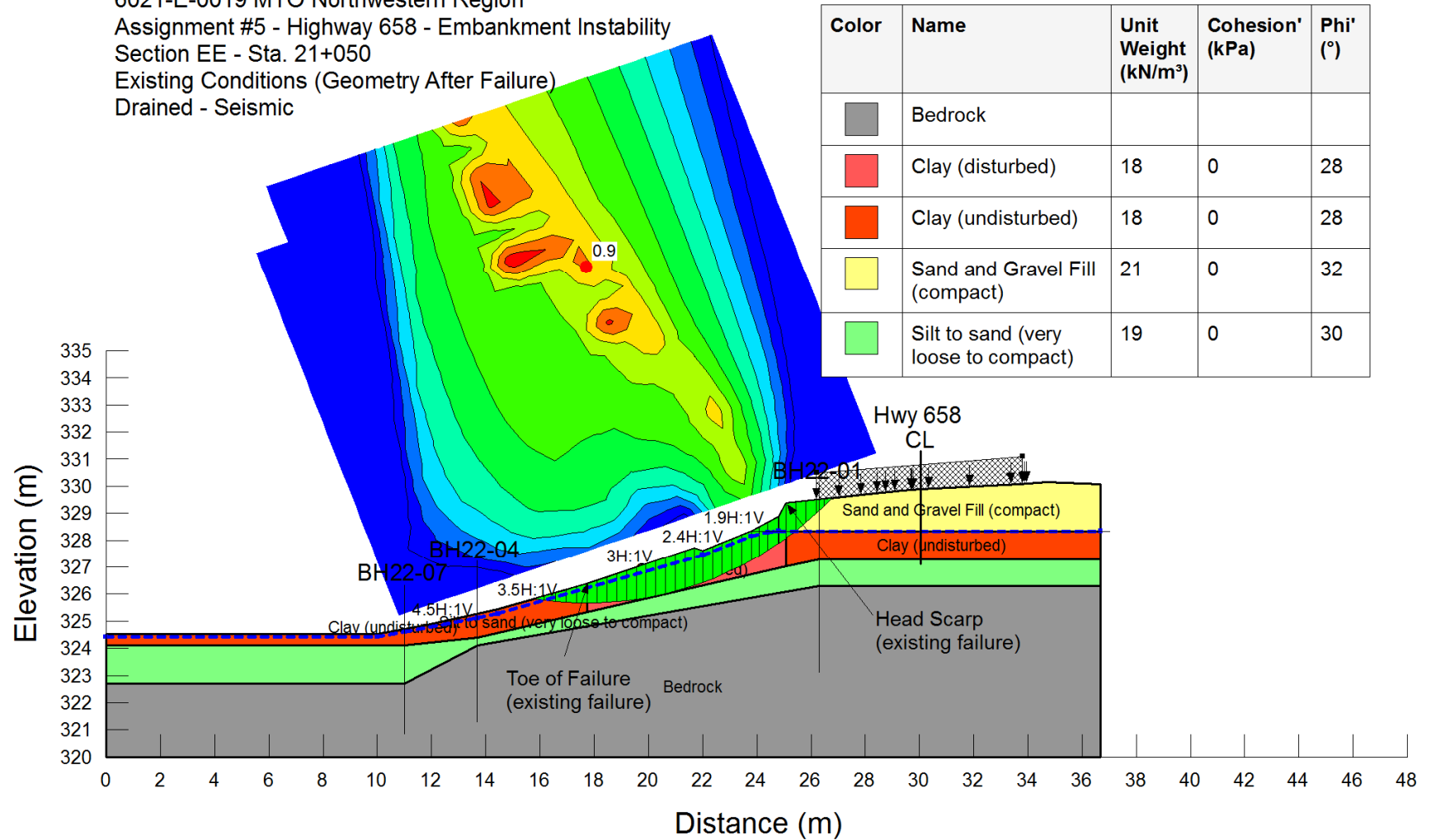


Figure G21. North Section E-E' (Sta. 21+050) – Stability of Slope After 2022 Failure (Current Slope Geometry), High GWL – Drained Seismic Conditions





## Remediation Measures

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Static Conditions  
 OPTION 1: Excavation of Disturbed Soil up to Bedrock  
 and Installation of Shear Key

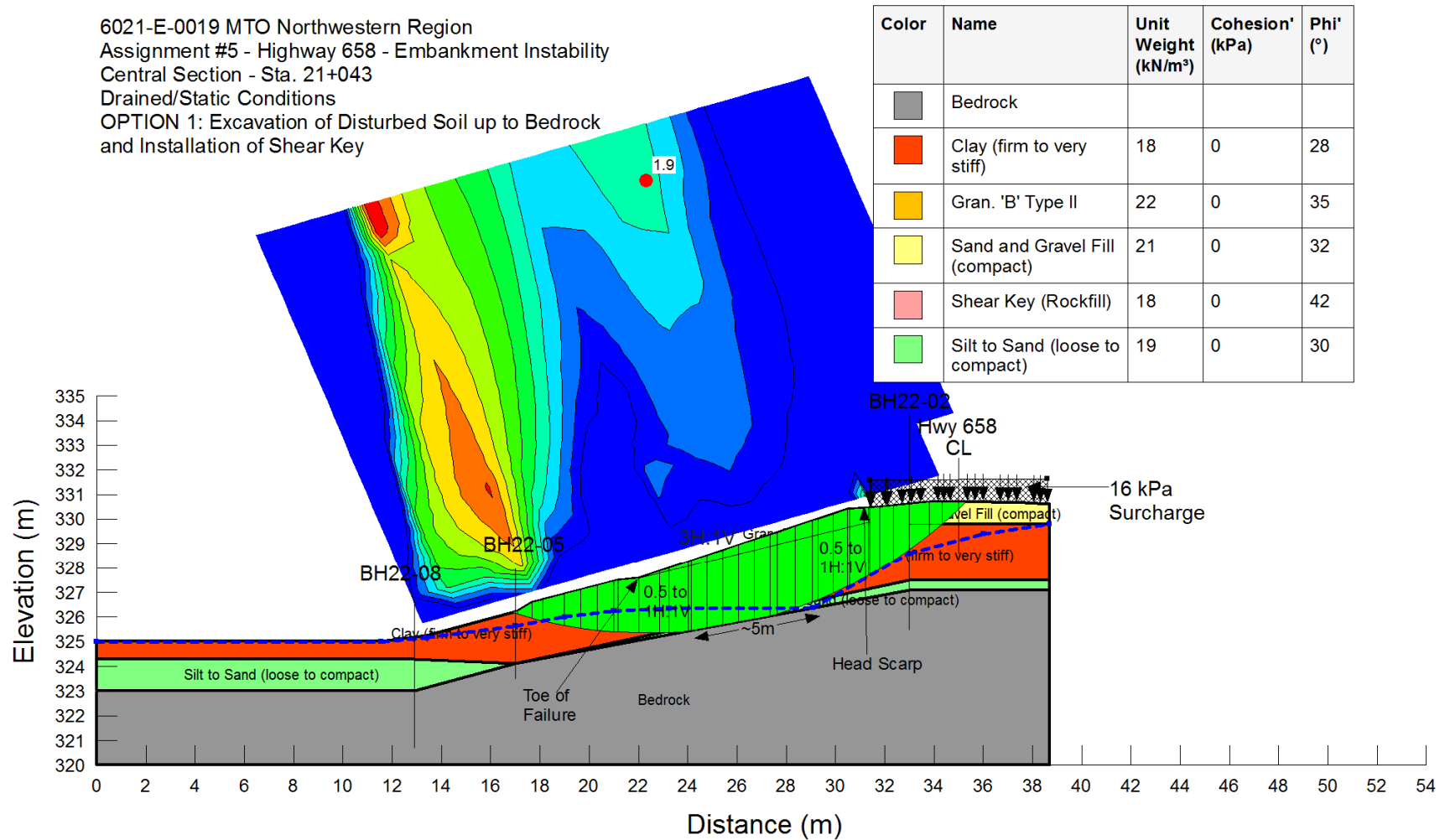


Figure G23. Central Section (Sta. 21+043) – Remedial Option 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key, Global Failure – Drained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Undrained/Static Conditions  
 OPTION 1: Excavation of Disturbed Soil up to Bedrock  
 and Installation of Shear Key

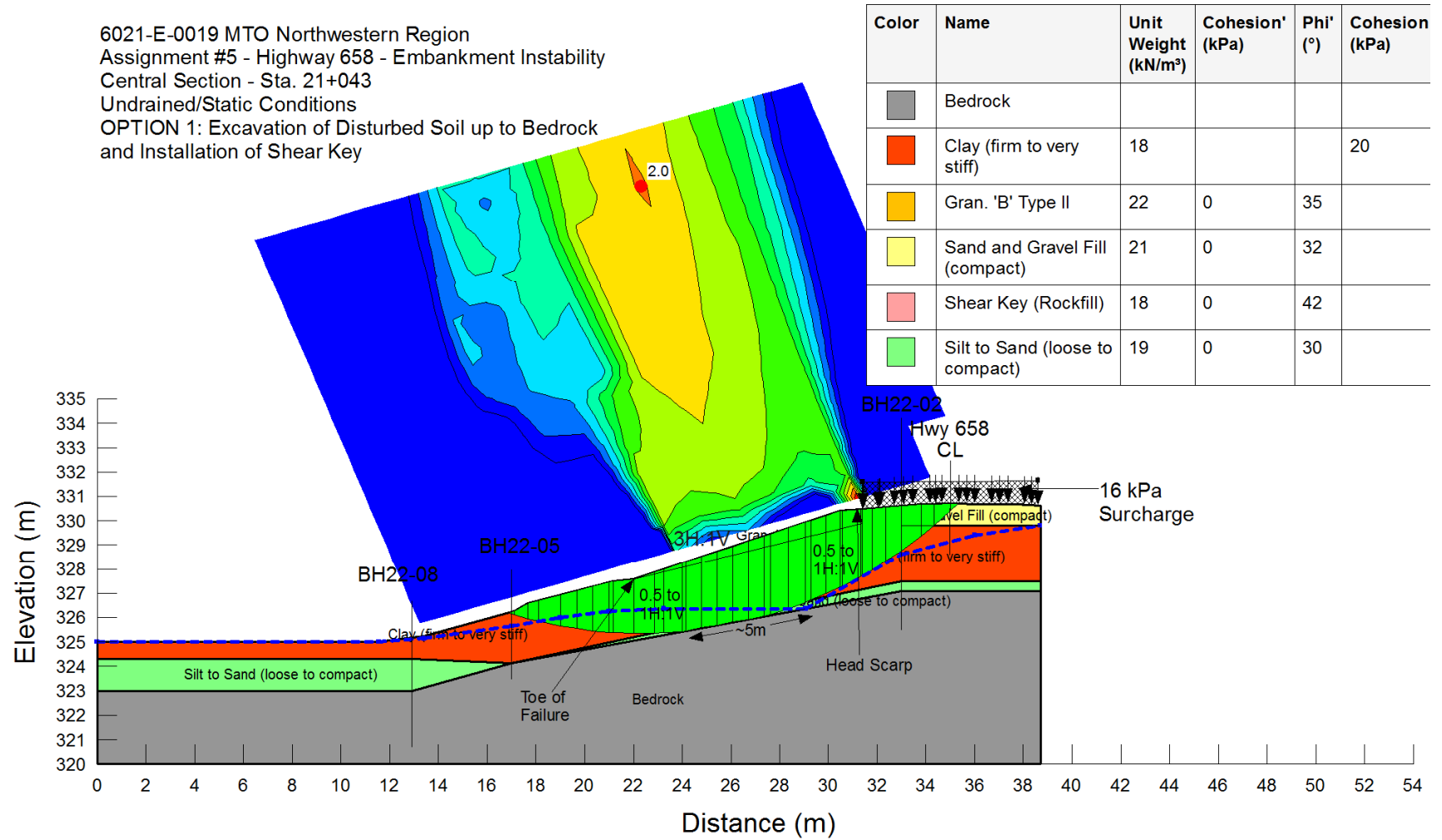


Figure G24. Central Section (Sta. 21+043) – Remedial Option 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key, Global Failure – Undrained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Static Conditions  
 OPTION 1: Excavation of Disturbed Soil up to Bedrock  
 and Installation of Shear Key

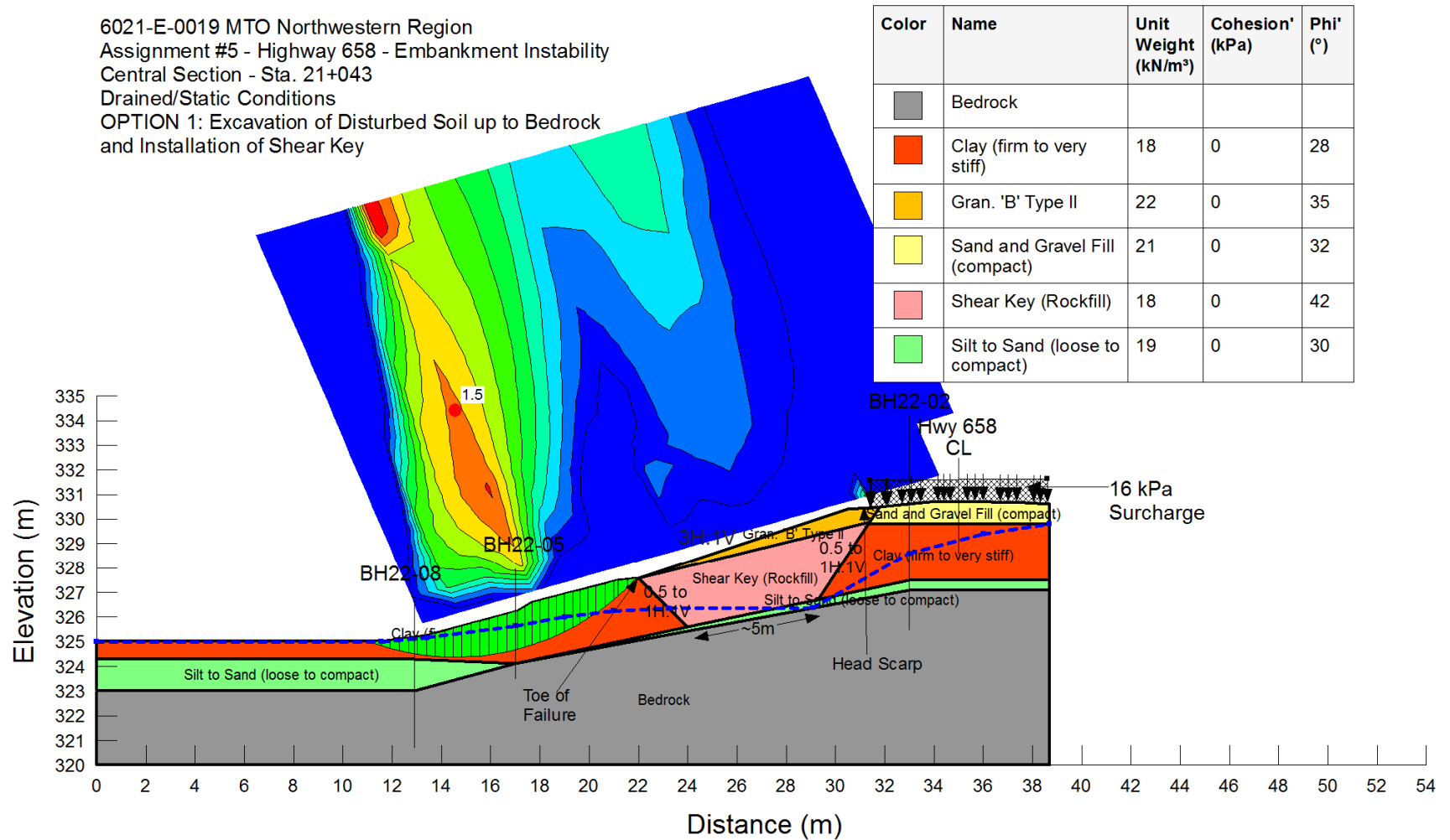


Figure G25. Central Section (Sta. 21+043) – Remedial Option 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key, Toe Failure – Drained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Undrained/Static Conditions  
 OPTION 1: Excavation of Disturbed Soil up to Bedrock  
 and Installation of Shear Key

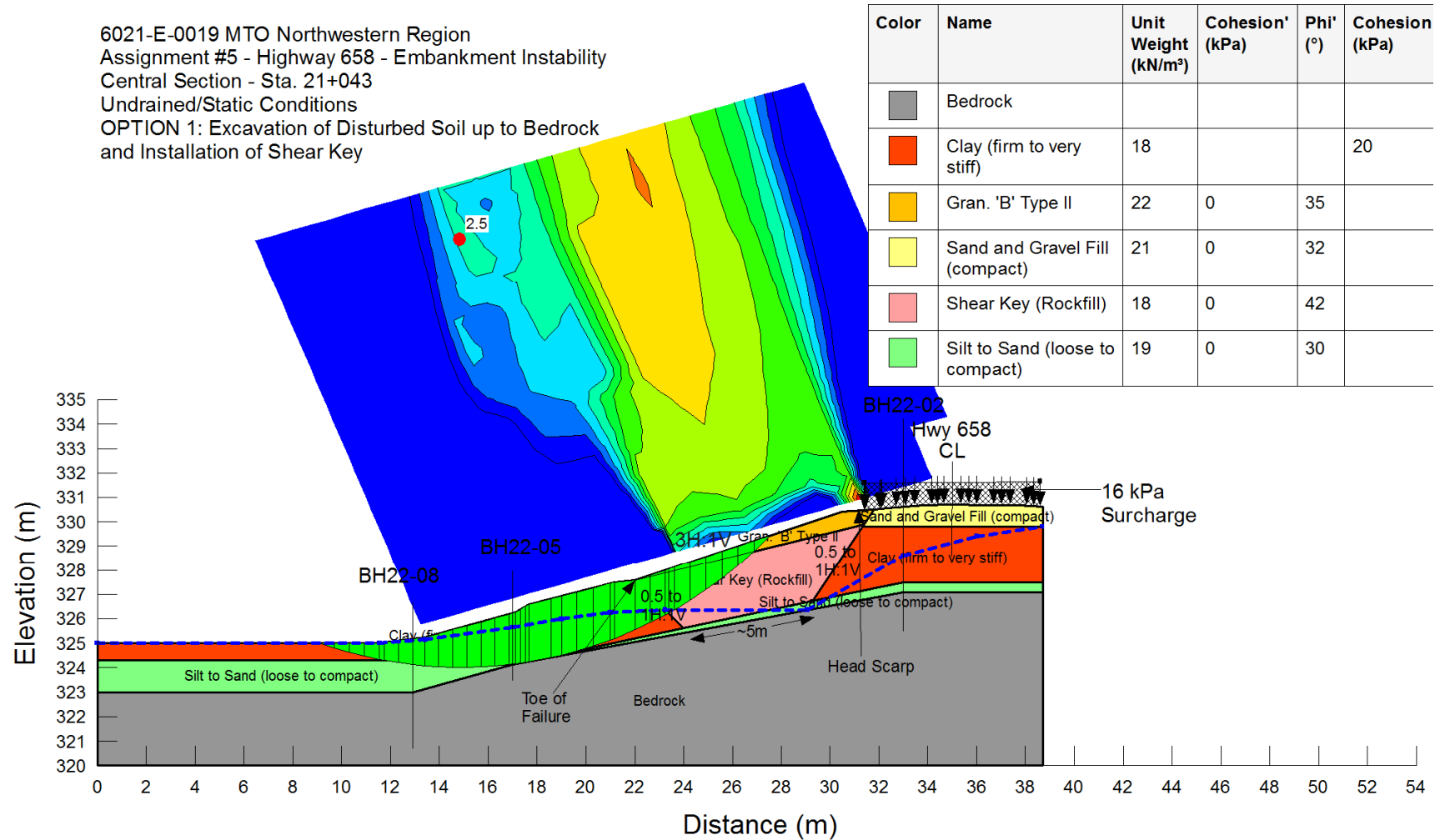


Figure G26. Central Section (Sta. 21+043) – Remedial Option 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key, Toe Failure – Undrained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Seismic Conditions  
 OPTION 1: Excavation of Disturbed Soil up to Bedrock  
 and Installation of Shear Key

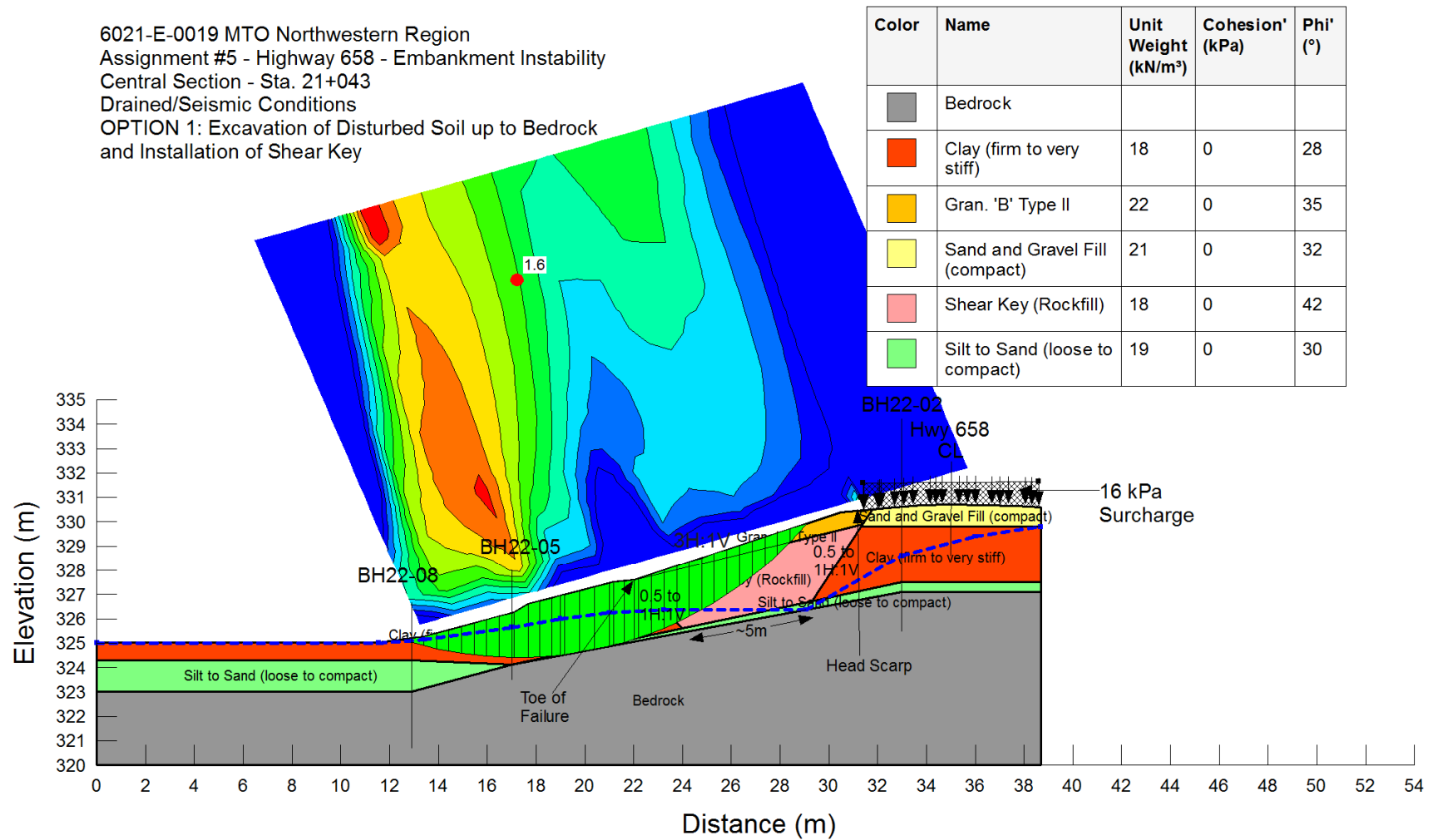


Figure G27. Central Section (Sta. 21+043) – Remedial Option 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key, Global Failure – Drained Seismic Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Undrained/Seismic Conditions  
 OPTION 1: Excavation of Disturbed Soil up to Bedrock  
 and Installation of Shear Key

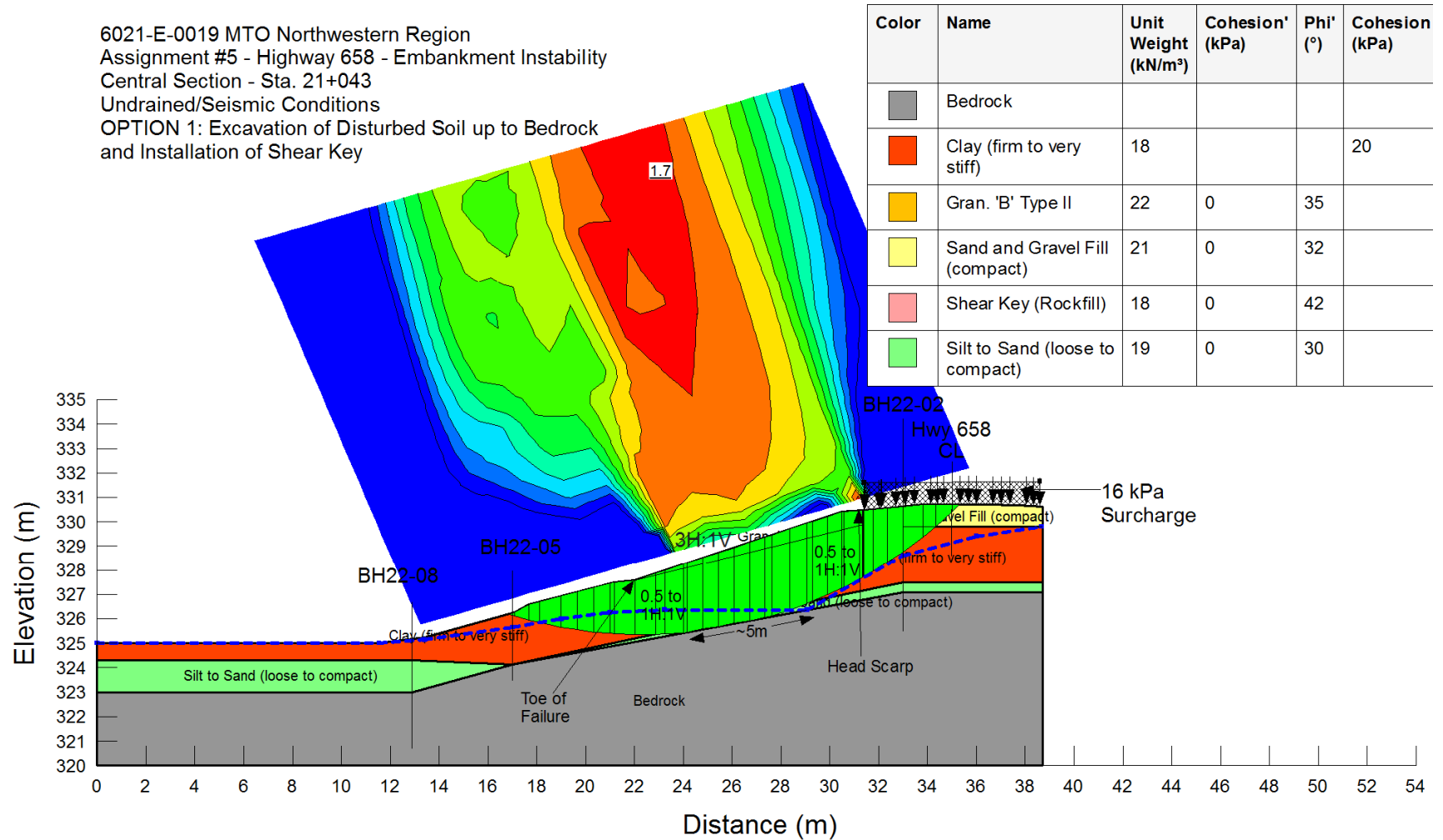


Figure G28. Central Section (Sta. 21+043) – Remedial Option 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key, Global Failure – Undrained Seismic Conditions



6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Seismic Conditions  
 OPTION 1: Excavation of Disturbed Soil up to Bedrock  
 and Installation of Shear Key

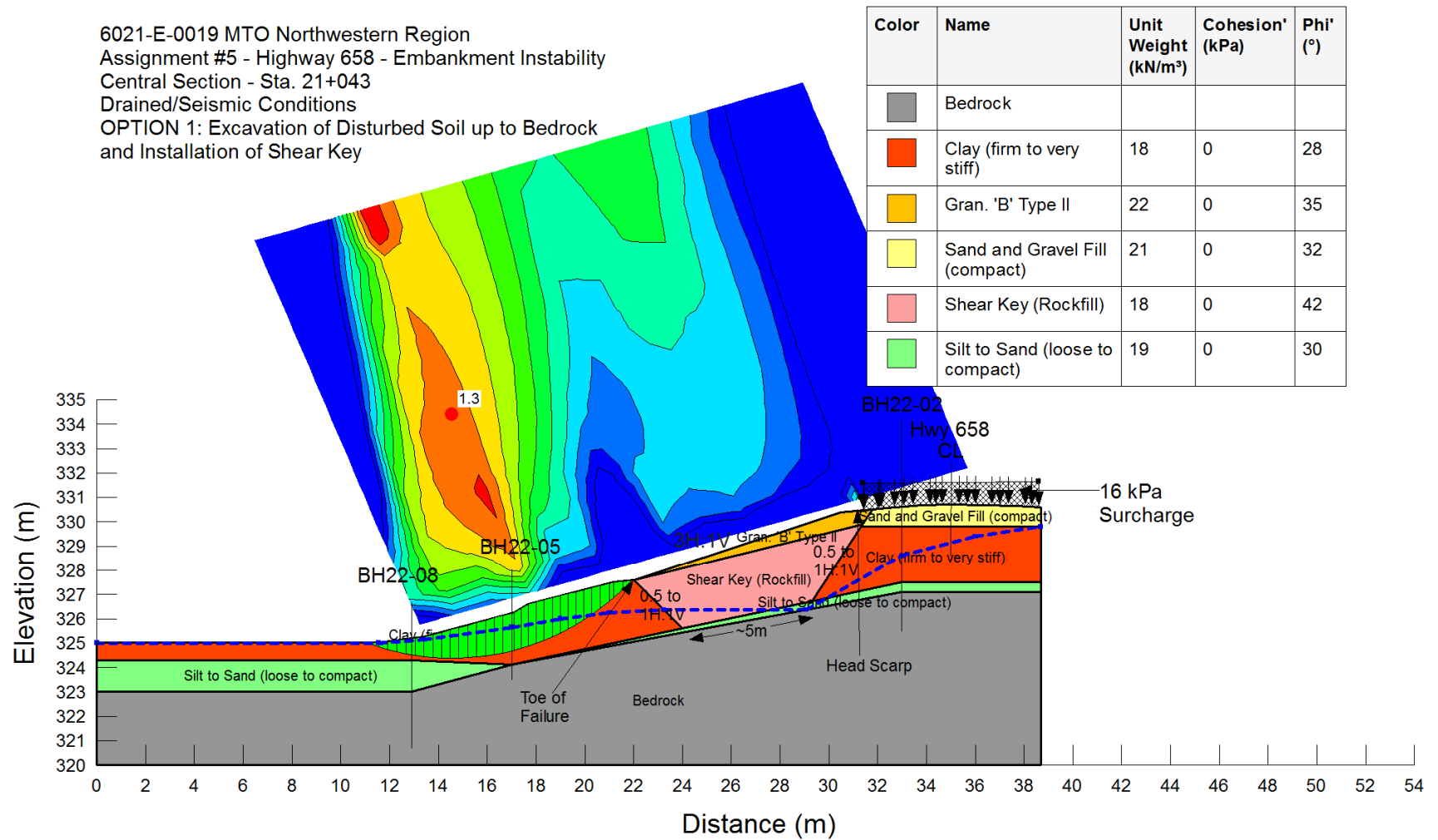


Figure G29. Central Section (Sta. 21+043) – Remedial Option 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key, Toe Failure – Drained Seismic Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Undrained/Seismic Conditions  
 OPTION 1: Excavation of Disturbed Soil up to Bedrock  
 and Installation of Shear Key

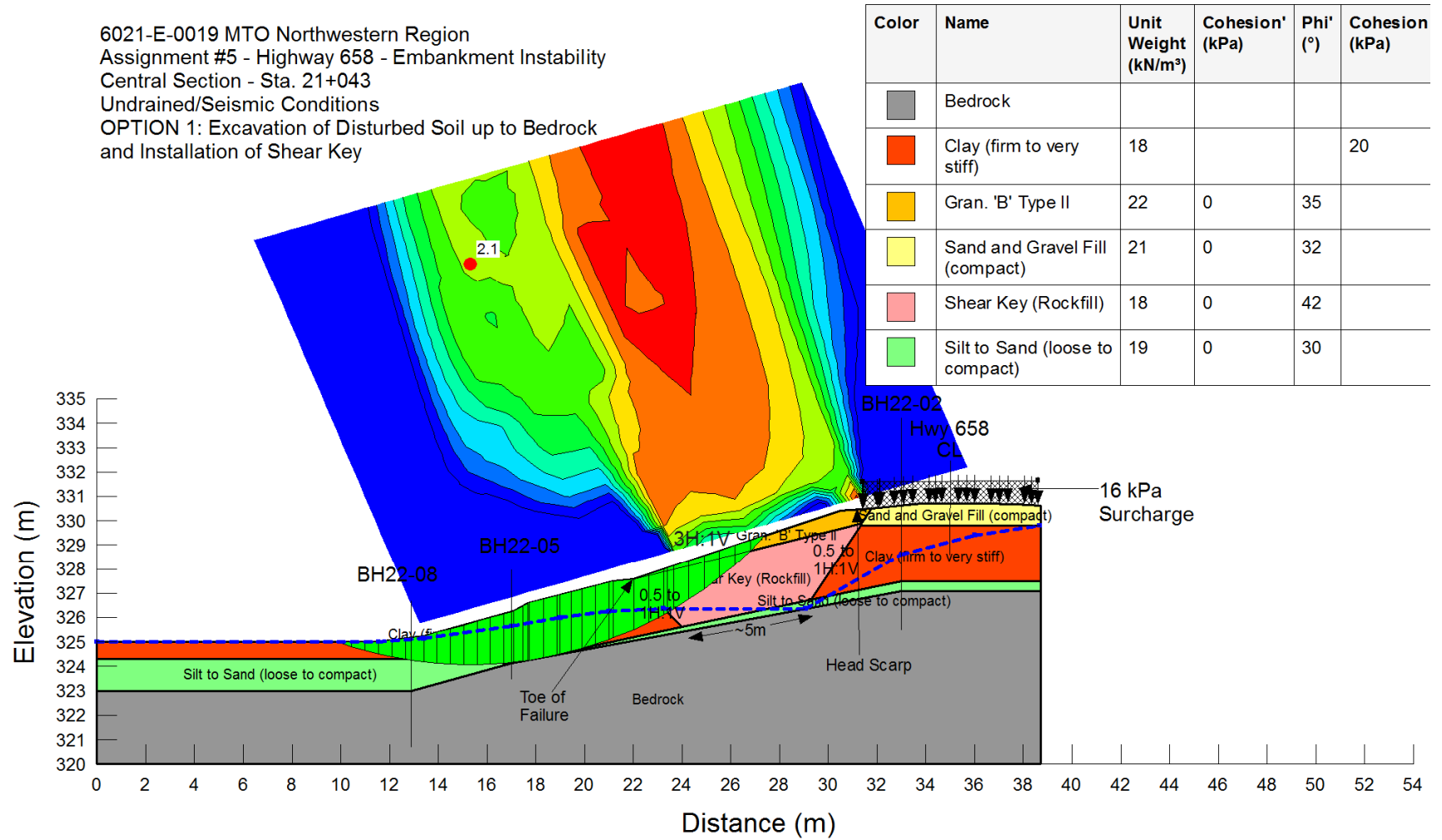


Figure G30. Central Section (Sta. 21+043) – Remedial Option 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key, Toe Failure – Undrained Seismic Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Section EE - Sta. 21+050  
 Drained/Static Conditions  
 OPTION 1: Excavation of Disturbed Soil up to  
 Bedrock and Installation of Shear Key

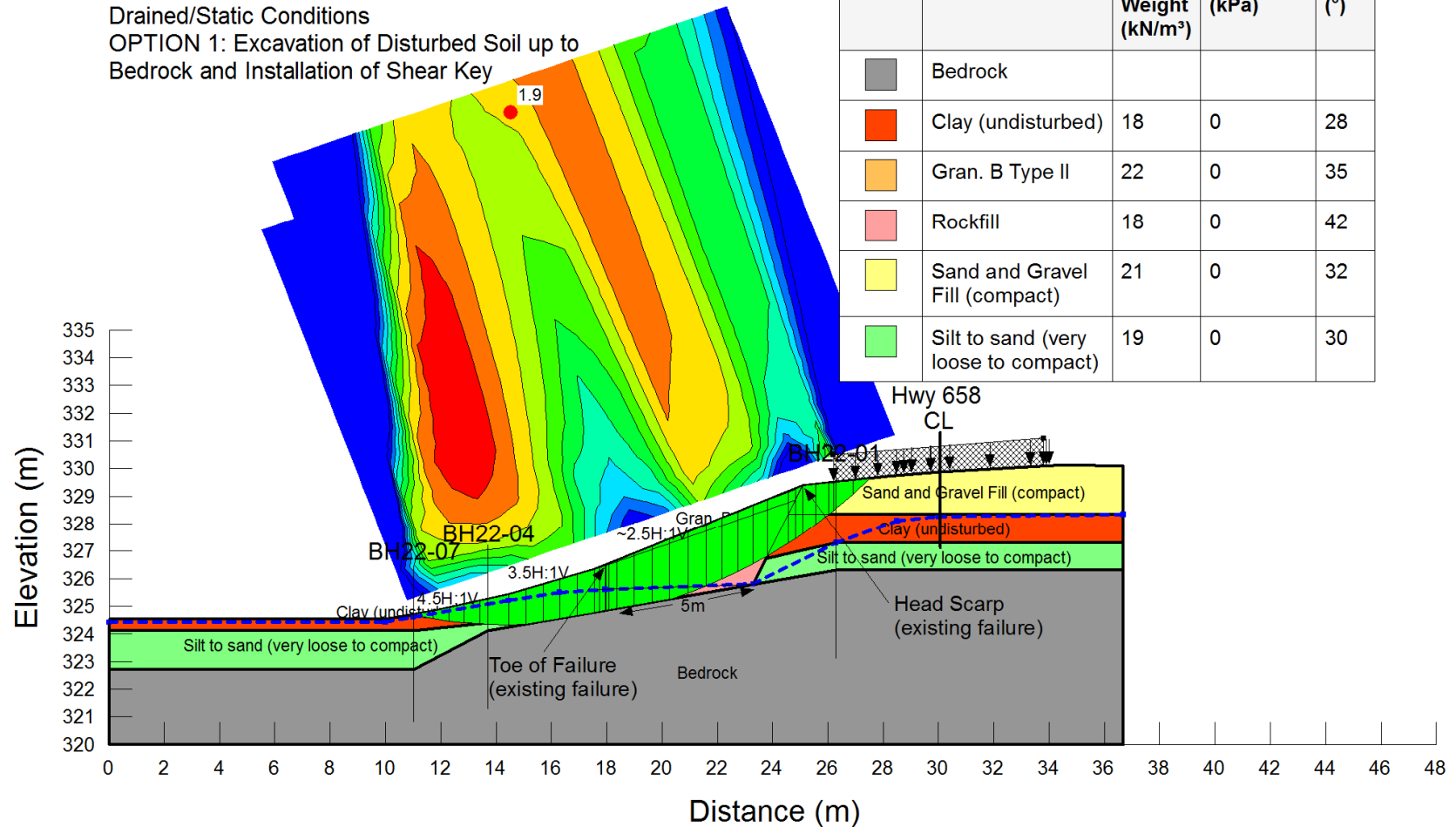


Figure G31. Critical North Section E-E' (Sta. 21+050) – Remedial Option 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key, Global Failure – Drained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Section EE - Sta. 21+050  
 Undrained/Static Conditions  
 OPTION 1: Excavation of Disturbed Soil up to  
 Bedrock and Installation of Shear Key

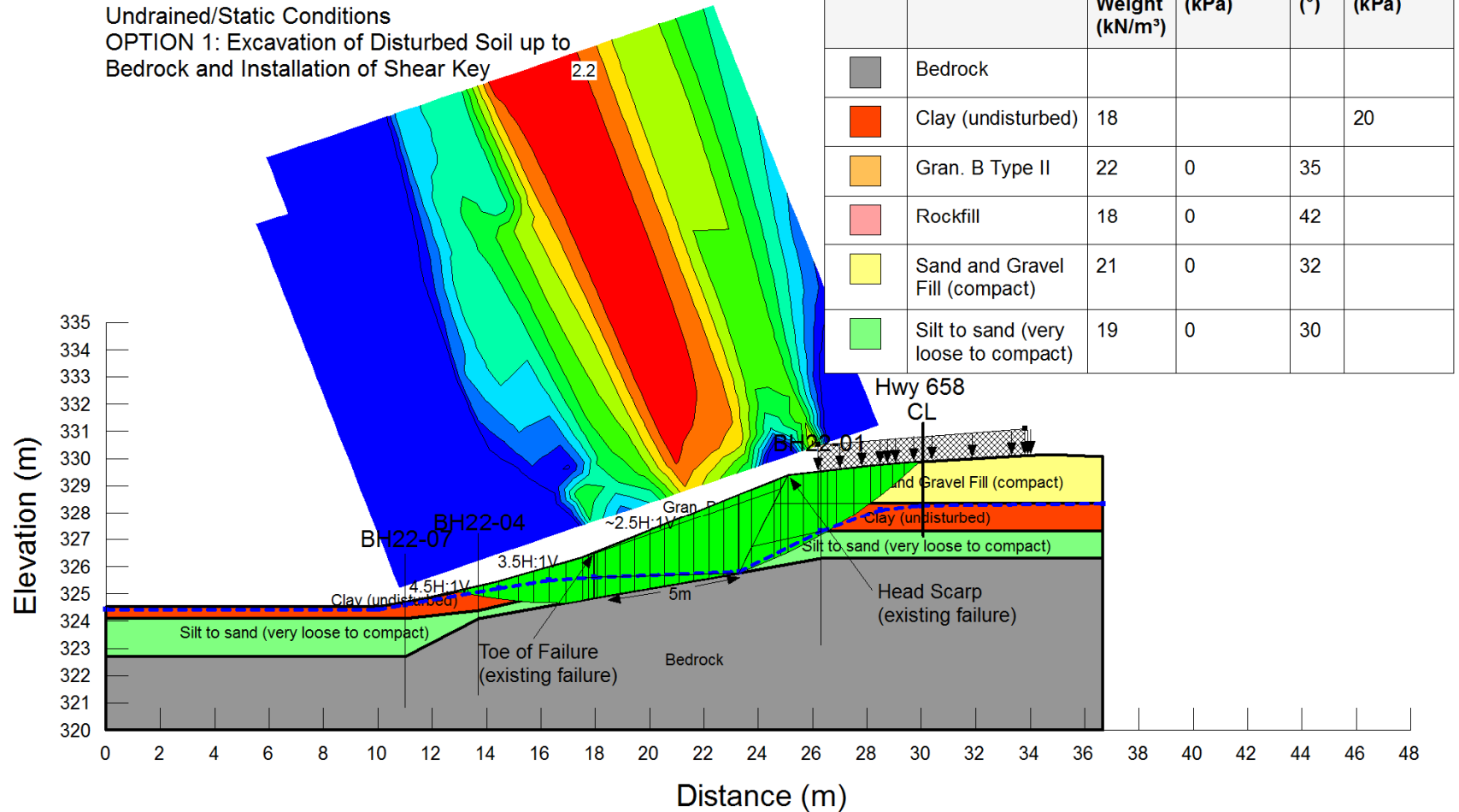


Figure G32. Critical North Section E-E' (Sta. 21+050) – Remedial Option 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key, Global Failure – Undrained Static Conditions



6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Section EE - Sta. 21+050  
 Undrained/Static Conditions  
 OPTION 1: Excavation of Disturbed Soil up to  
 Bedrock and Installation of Shear Key

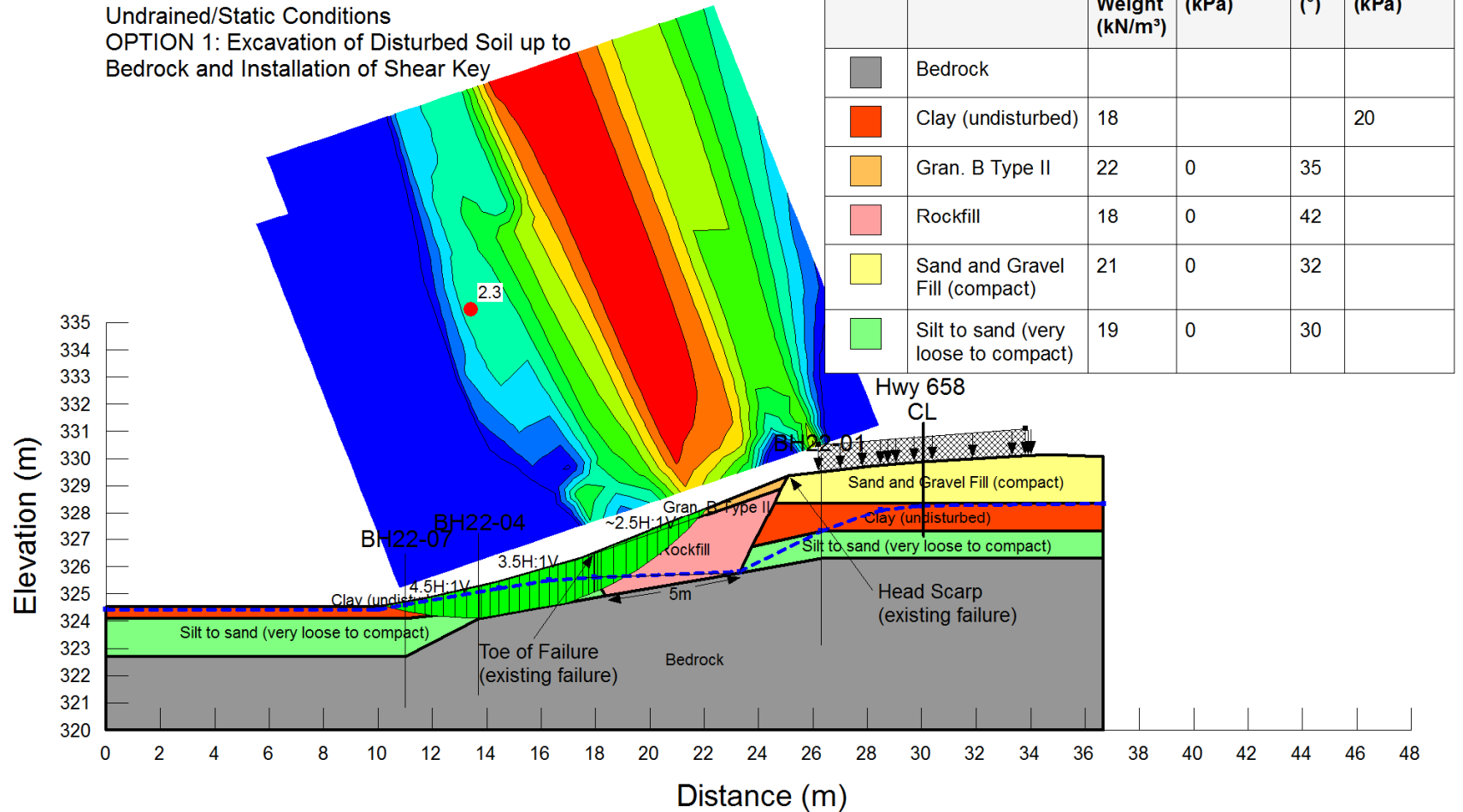


Figure G34. Critical North Section E-E' (Sta. 21+050) – Remedial Option 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key, Toe Failure – Undrained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Section EE - Sta. 21+050  
 Drained/Seismic Conditions  
 OPTION 1: Excavation of Disturbed Soil up to  
 Bedrock and Installation of Shear Key

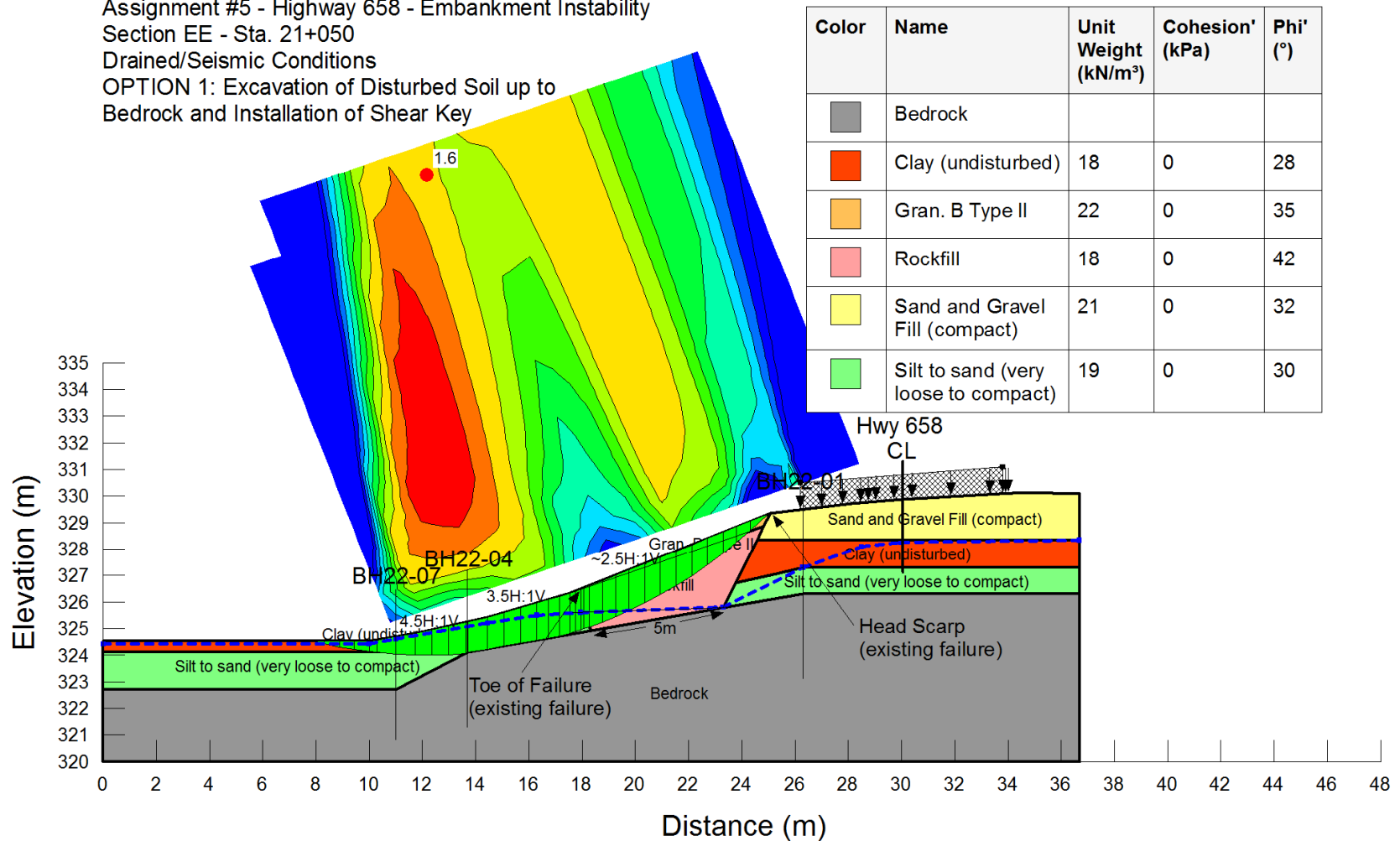


Figure G35. Critical North Section E-E' (Sta. 21+050) – Remedial Option 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key, Global Failure – Drained Seismic Conditions



6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Section EE - Sta. 21+050  
 Undrained/Seismic Conditions  
 OPTION 1: Excavation of Disturbed Soil up to  
 Bedrock and Installation of Shear Key

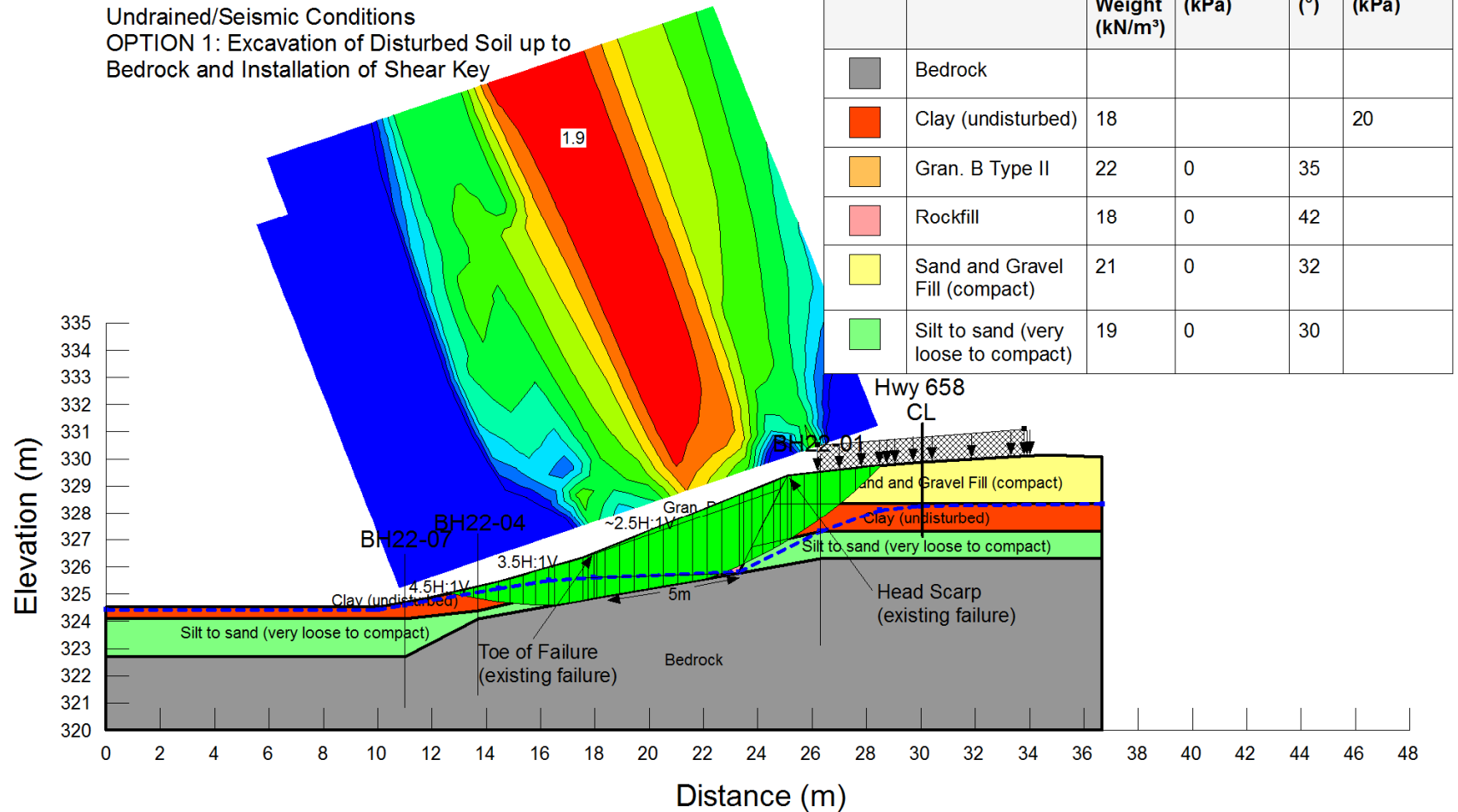


Figure G36. Critical North Section E-E' (Sta. 21+050) – Remedial Option 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key, Global Failure – Undrained Seismic Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Section EE - Sta. 21+050  
 Drained/Seismic Conditions  
 OPTION 1: Excavation of Disturbed Soil up to  
 Bedrock and Installation of Shear Key

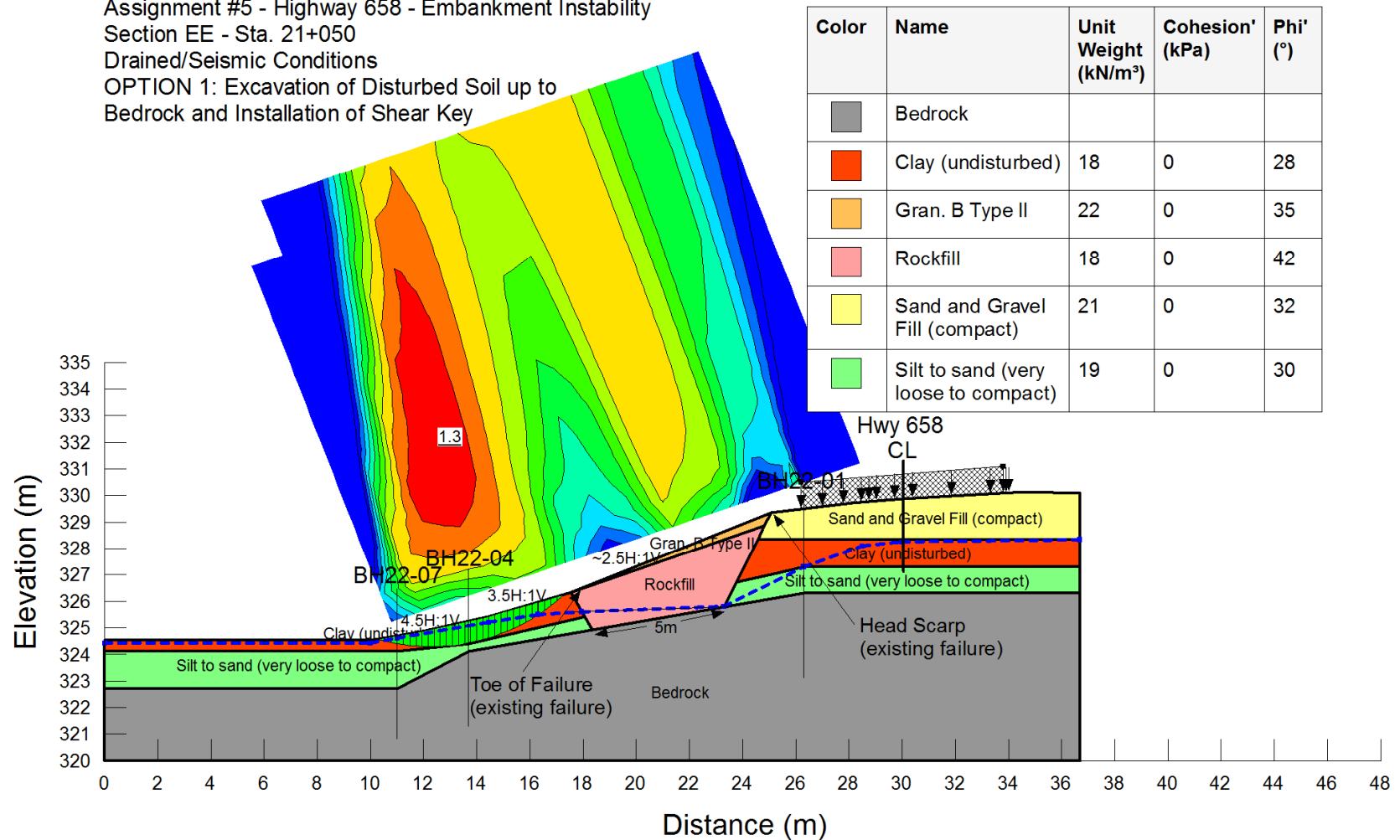


Figure G37. Critical North Section E-E' (Sta. 21+050) – Remedial Option 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key, Toe Failure – Drained Seismic Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Section EE - Sta. 21+050  
 Undrained/Seismic Conditions  
 OPTION 1: Excavation of Disturbed Soil up to  
 Bedrock and Installation of Shear Key

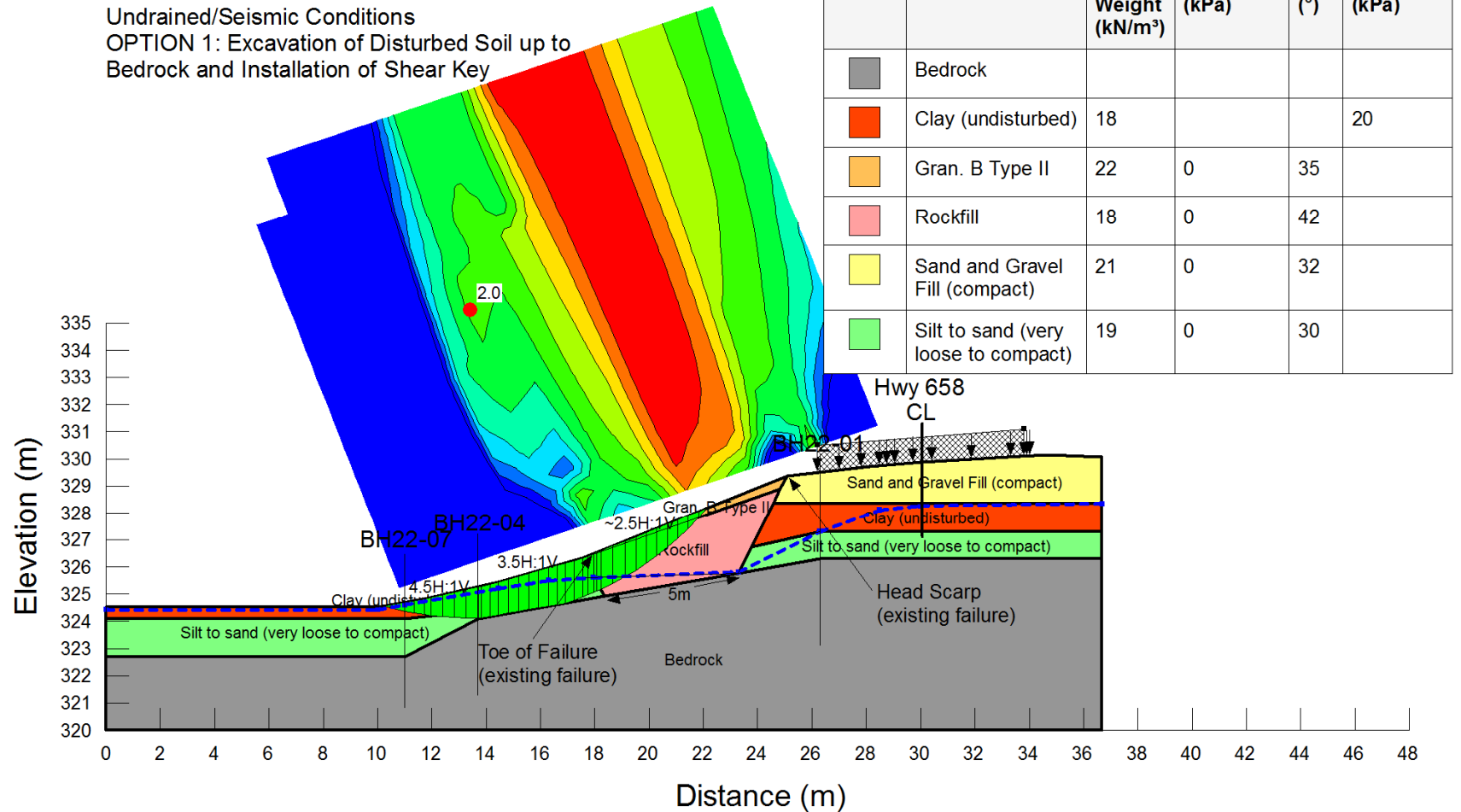


Figure G38. Critical North Section E-E' (Sta. 21+050) – Remedial Option 1: Excavation of Disturbed Soil up to Bedrock and Installation of Shear Key, Toe Failure – Undrained Seismic Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Static Conditions  
 OPTION 2: Construction of Toe Berm and  
 Slope Flattening

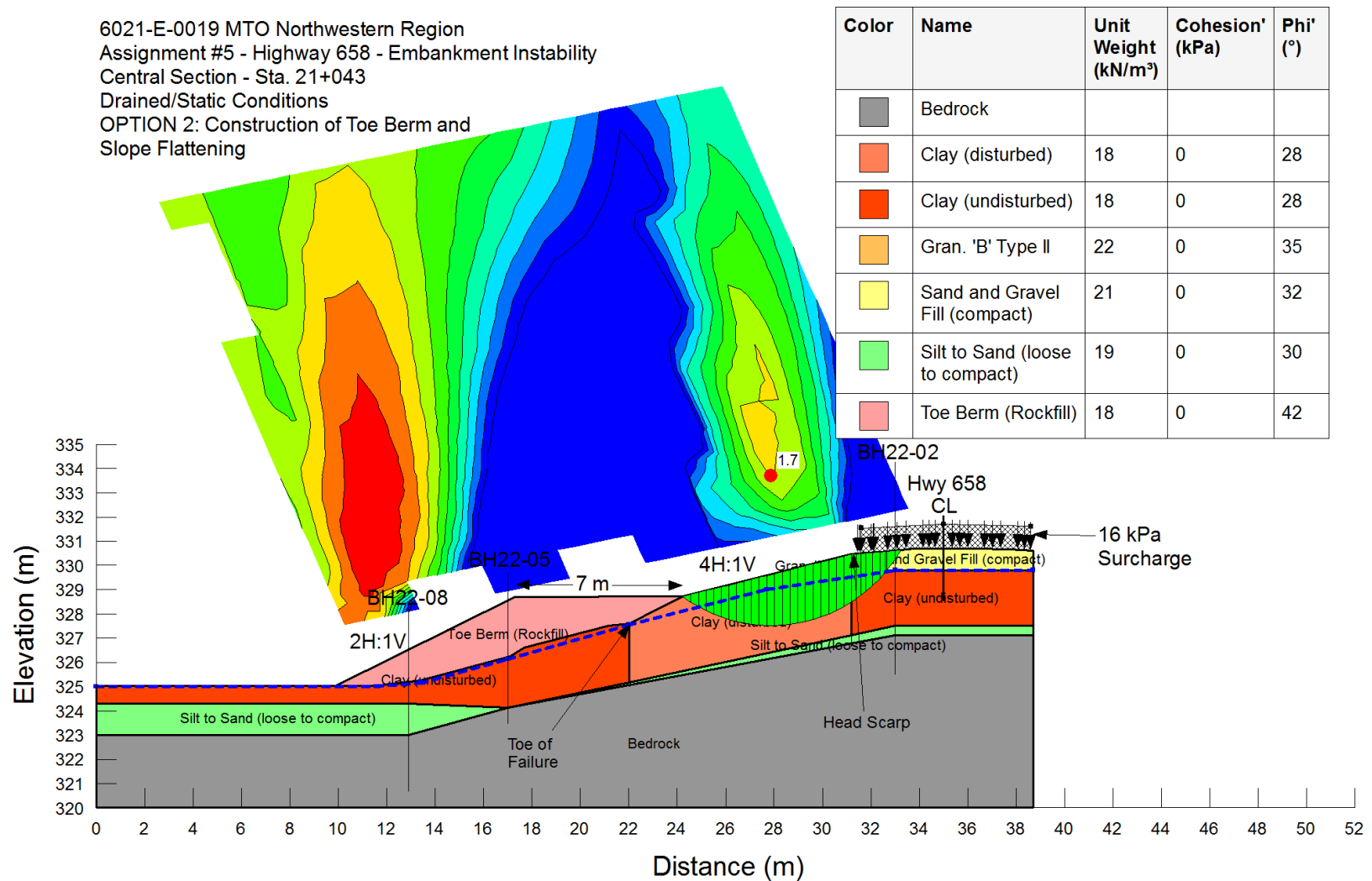


Figure G39. Central Section (Sta. 21+043) – Remedial Option 2: Construction of Toe Berm and Slope Flattening, Global Failure – Drained Static Conditions



6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Static Conditions  
 OPTION 2: Construction of Toe Berm and  
 Slope Flattening

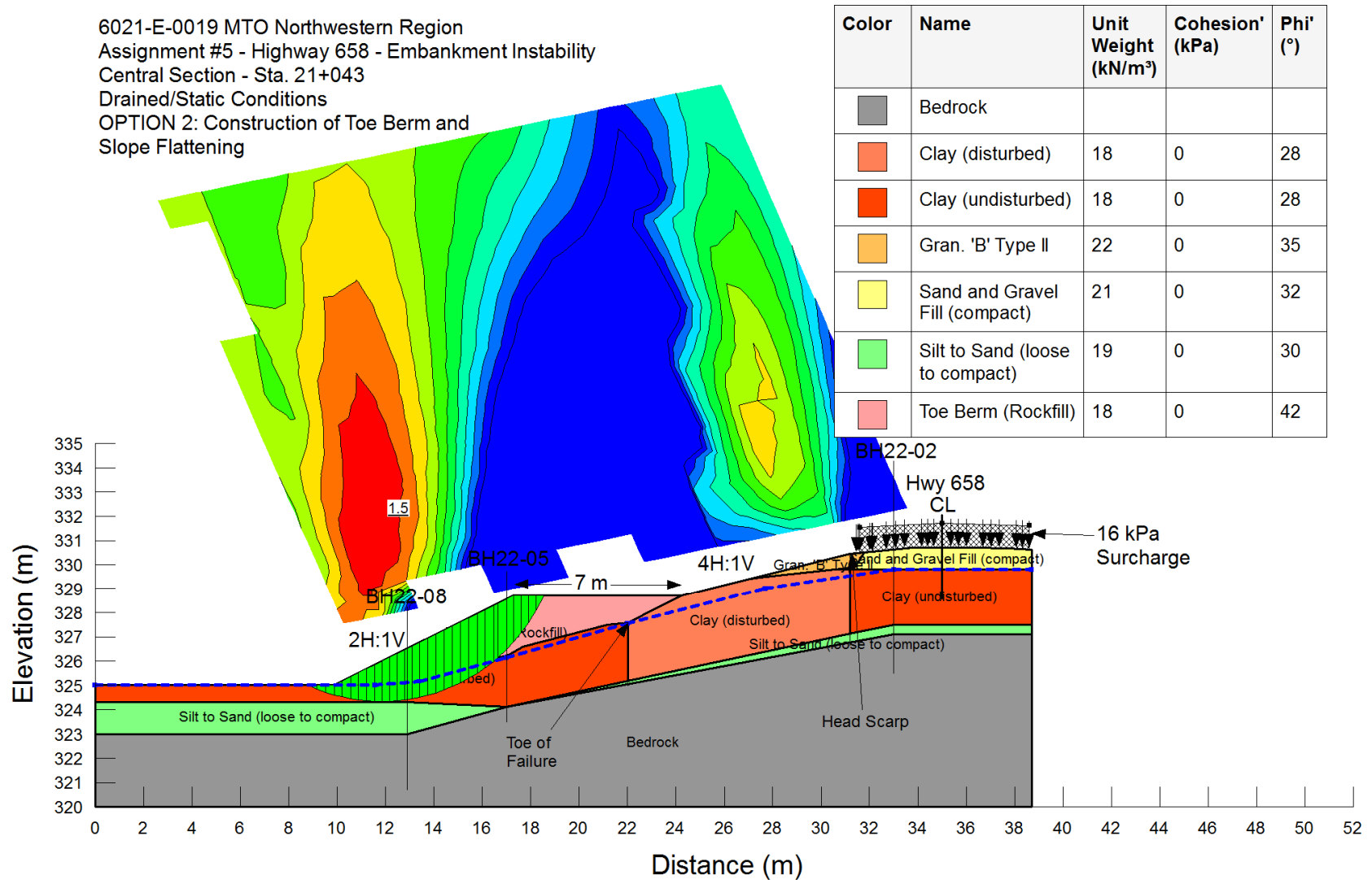
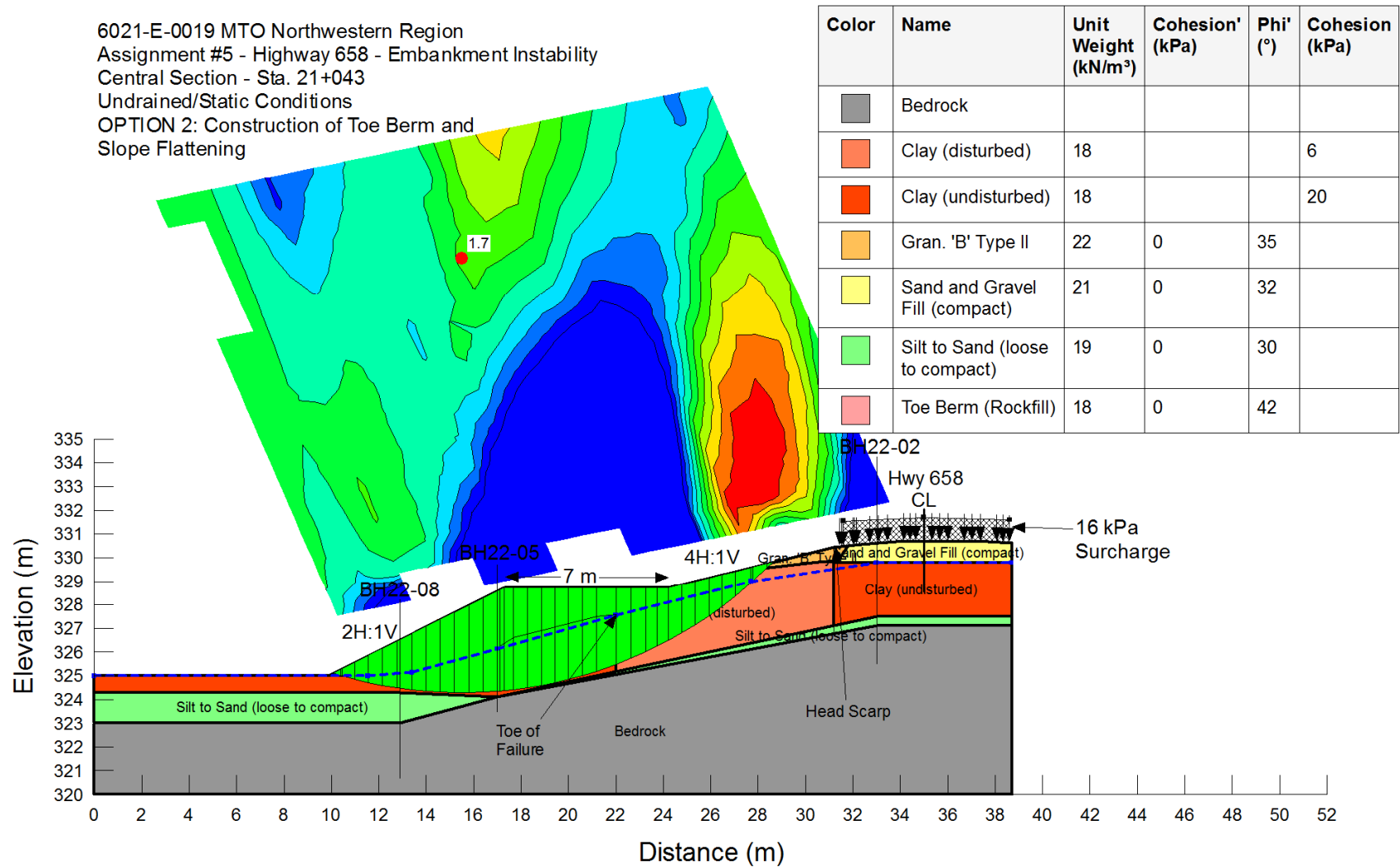


Figure G41. Central Section (Sta. 21+043) – Remedial Option 2: Construction of Toe Berm and Slope Flattening, Toe Failure – Drained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Undrained/Static Conditions  
 OPTION 2: Construction of Toe Berm and  
 Slope Flattening





6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Seismic Conditions  
 OPTION 2: Construction of Toe Berm and  
 Slope Flattening

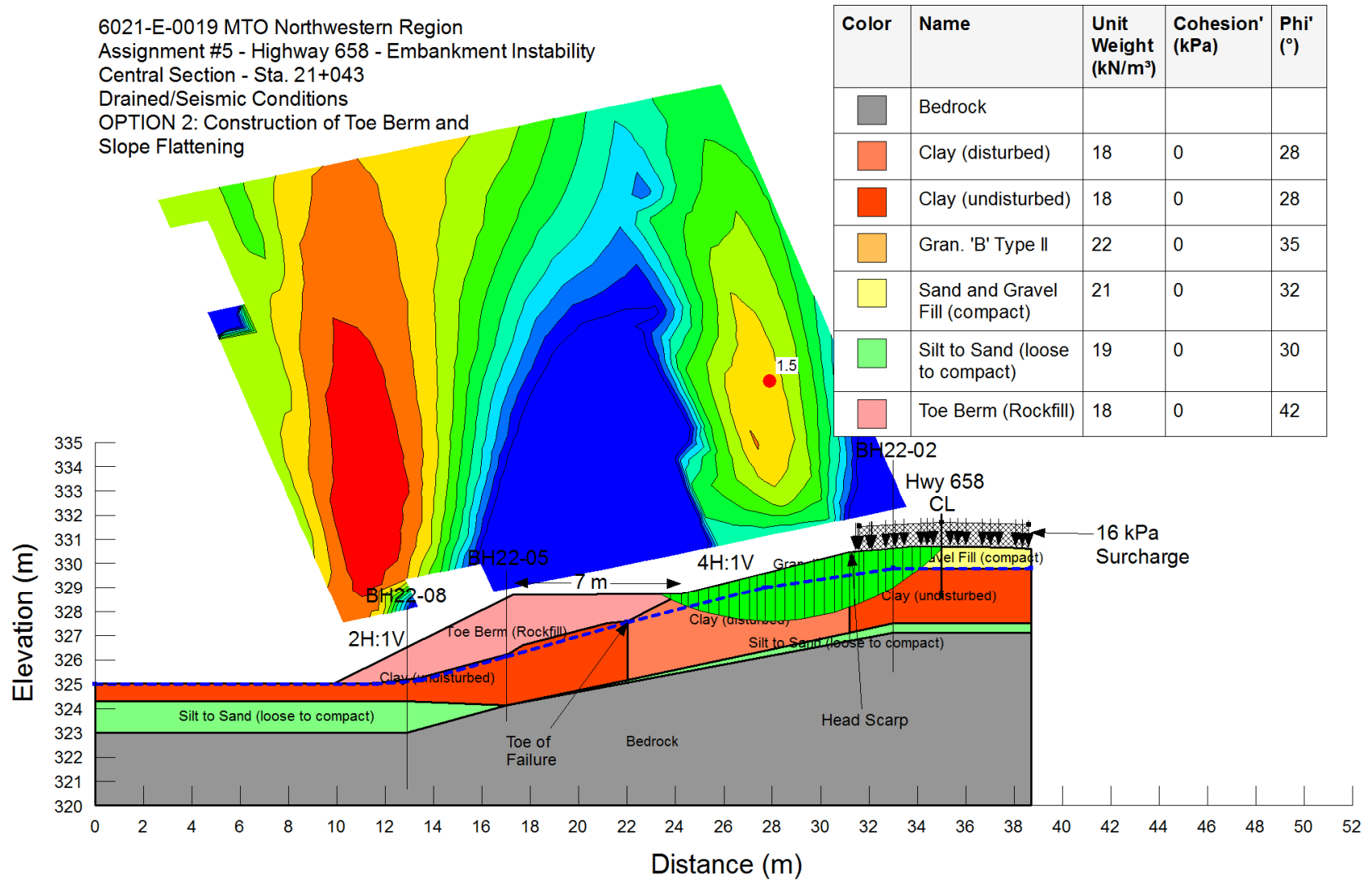


Figure G43. Central Section (Sta. 21+043) – Remedial Option 2: Construction of Toe Berm and Slope Flattening, Global Failure – Drained Seismic Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Undrained/Seismic Conditions  
 OPTION 2: Construction of Toe Berm and  
 Slope Flattening

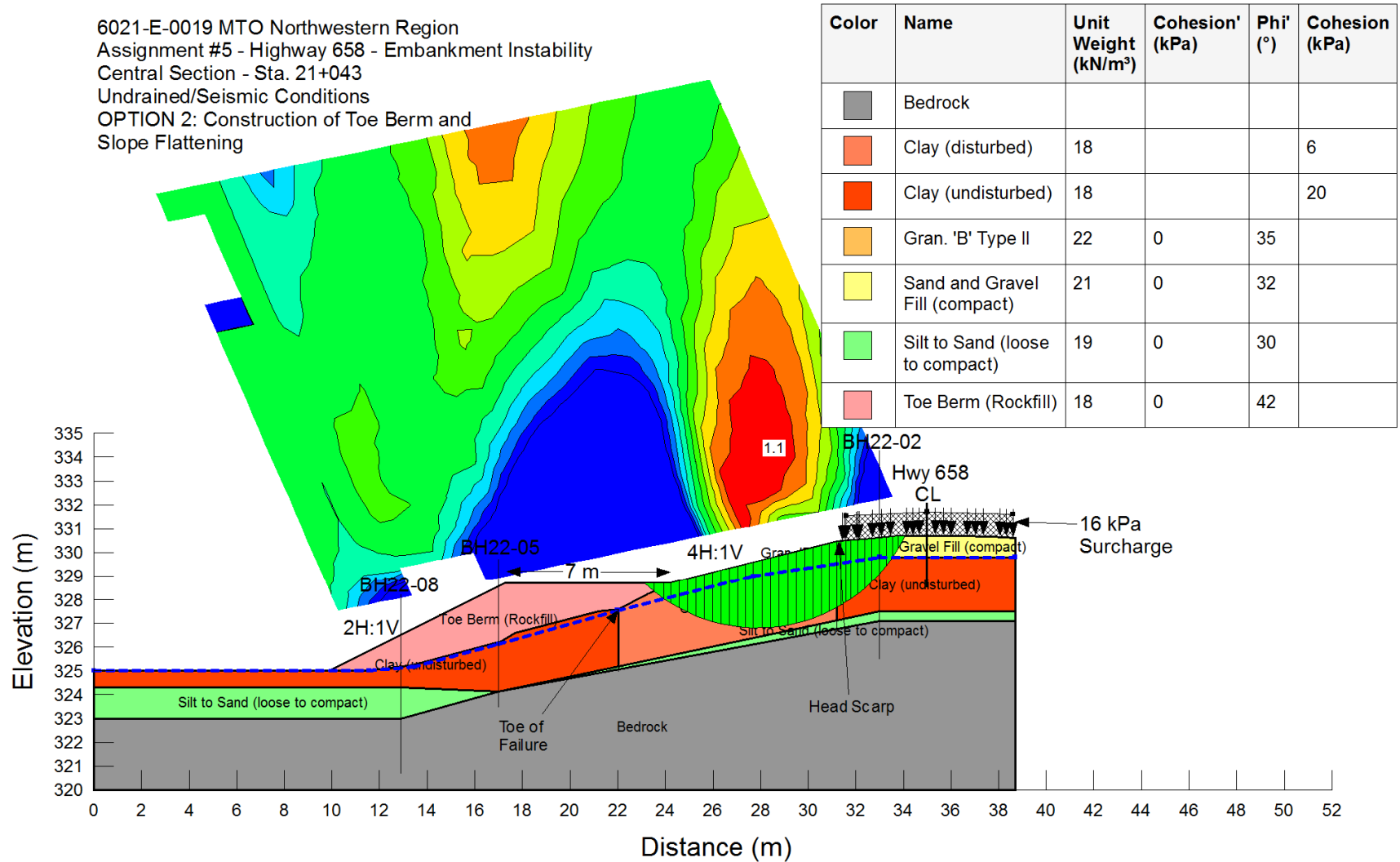


Figure G44. Central Section (Sta. 21+043) – Remedial Option 2: Construction of Toe Berm and Slope Flattening, Global Failure – Undrained Seismic Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Seismic Conditions  
 OPTION 2: Construction of Toe Berm and  
 Slope Flattening

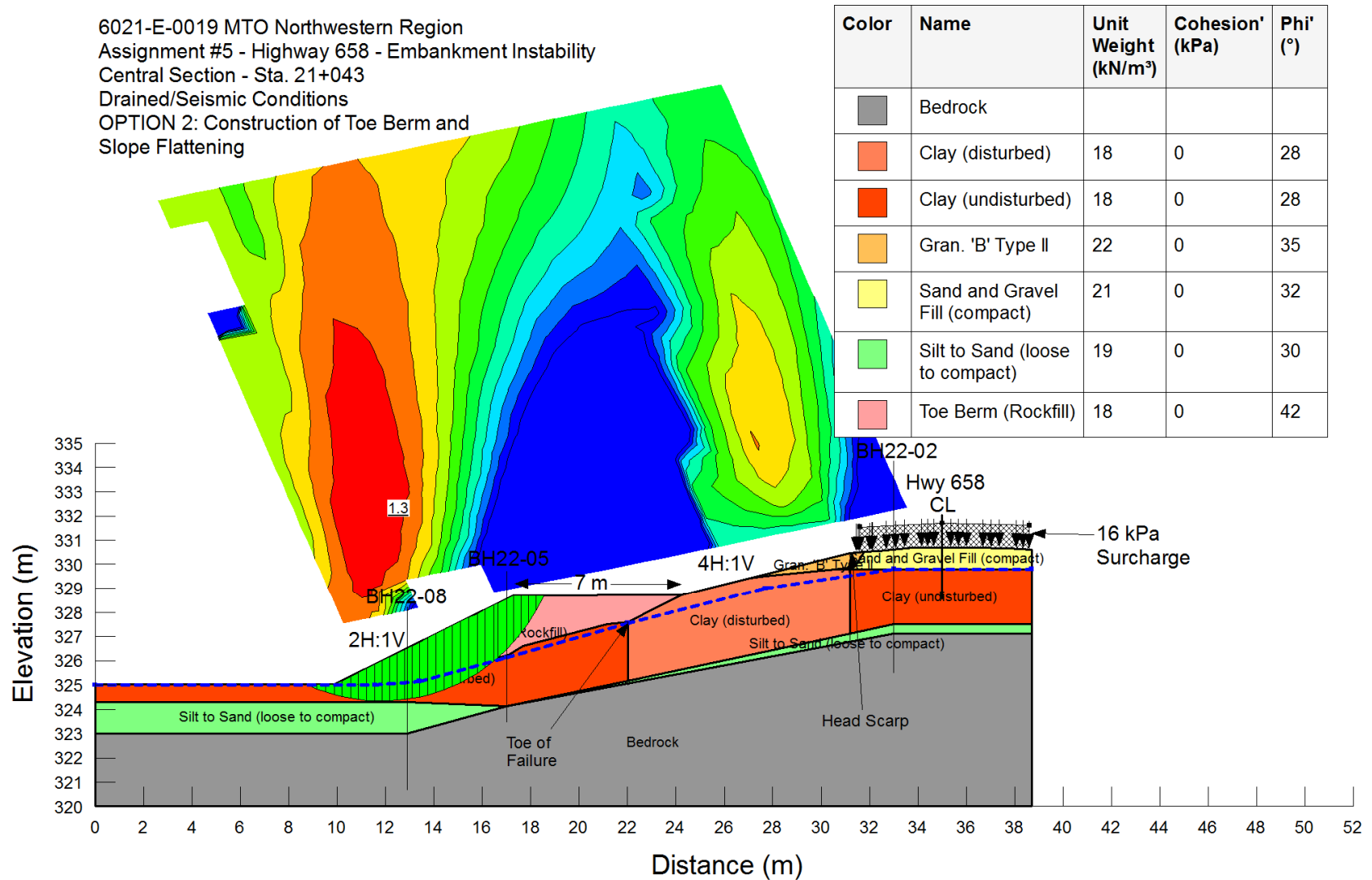


Figure G45. Central Section (Sta. 21+043) – Remedial Option 2: Construction of Toe Berm and Slope Flattening, Toe Failure – Drained Seismic Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Undrained/Seismic Conditions  
 OPTION 2: Construction of Toe Berm and  
 Slope Flattening

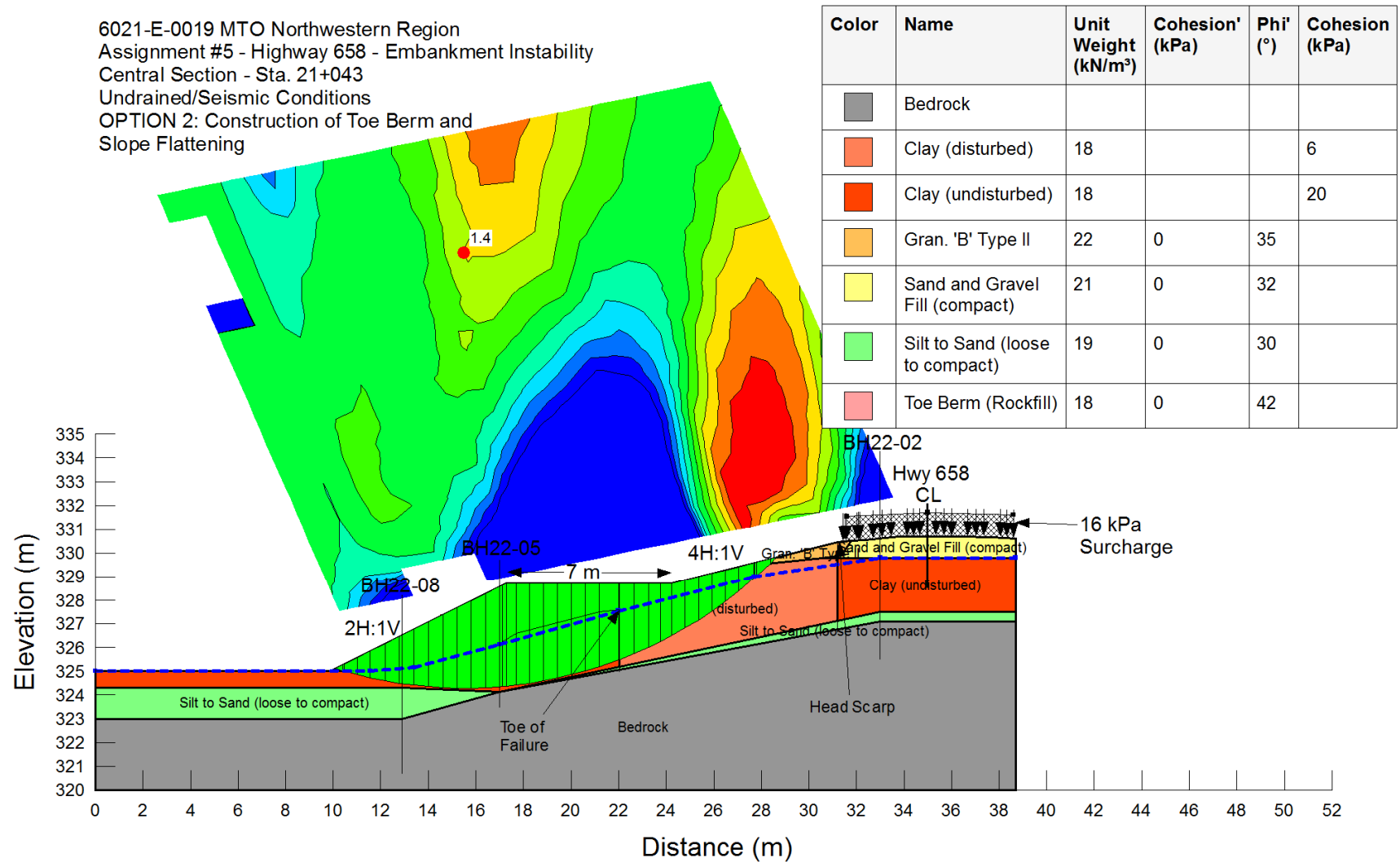


Figure G46. Central Section (Sta. 21+043) – Remedial Option 2: Construction of Toe Berm and Slope Flattening, Toe Failure – Undrained Seismic Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Static Conditions  
 OPTION 3: Replacement of Disturbed Soil with  
 Lightweight Fill and Slope Flattening

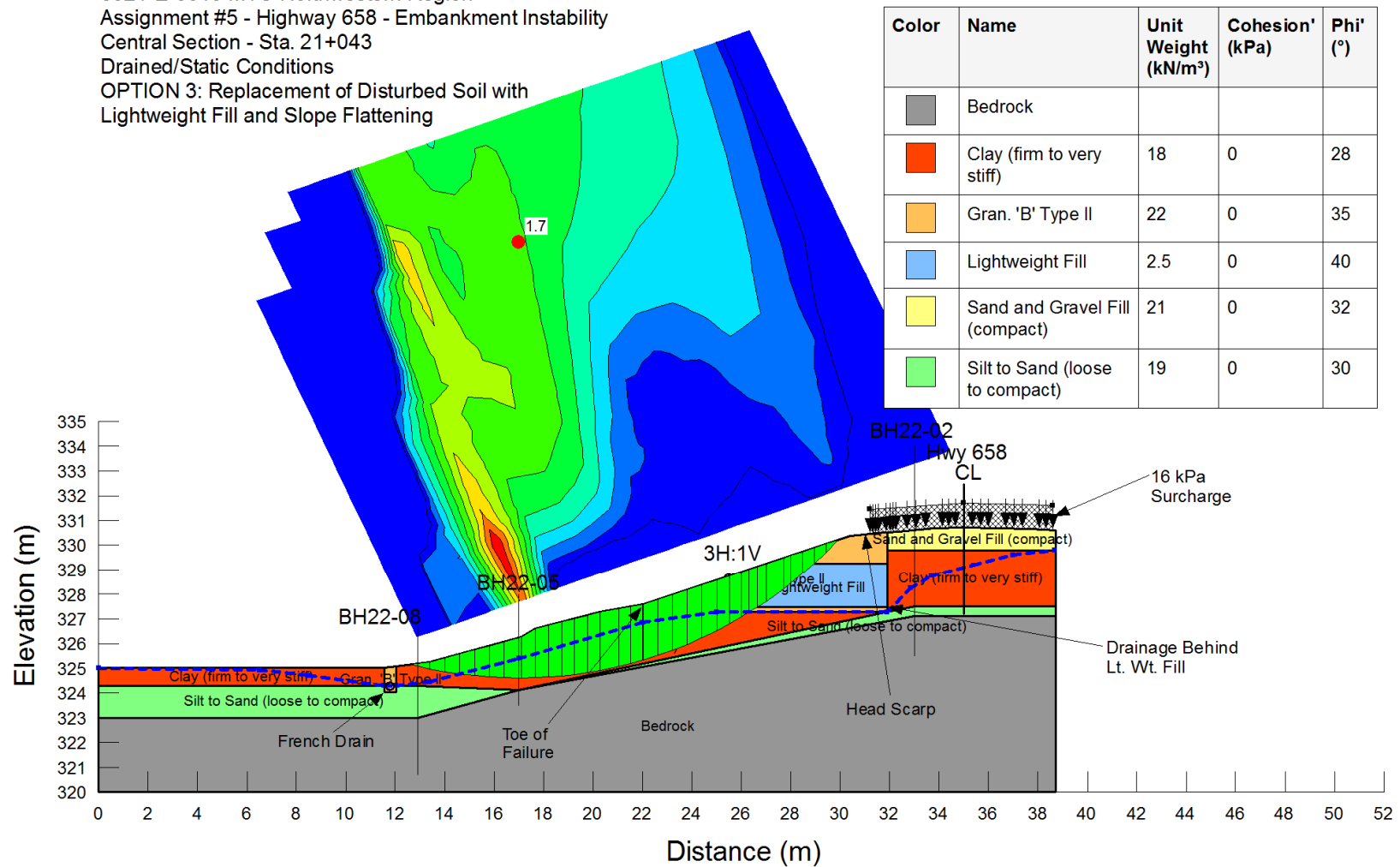


Figure G47. Central Section (Sta. 21+043) – Remedial Option 3: Replacement of Disturbed Soil with Lightweight Fill and Slope Flattening, Global Failure – Drained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Static Conditions  
 OPTION 3: Replacement of Disturbed Soil with  
 Lightweight Fill and Slope Flattening

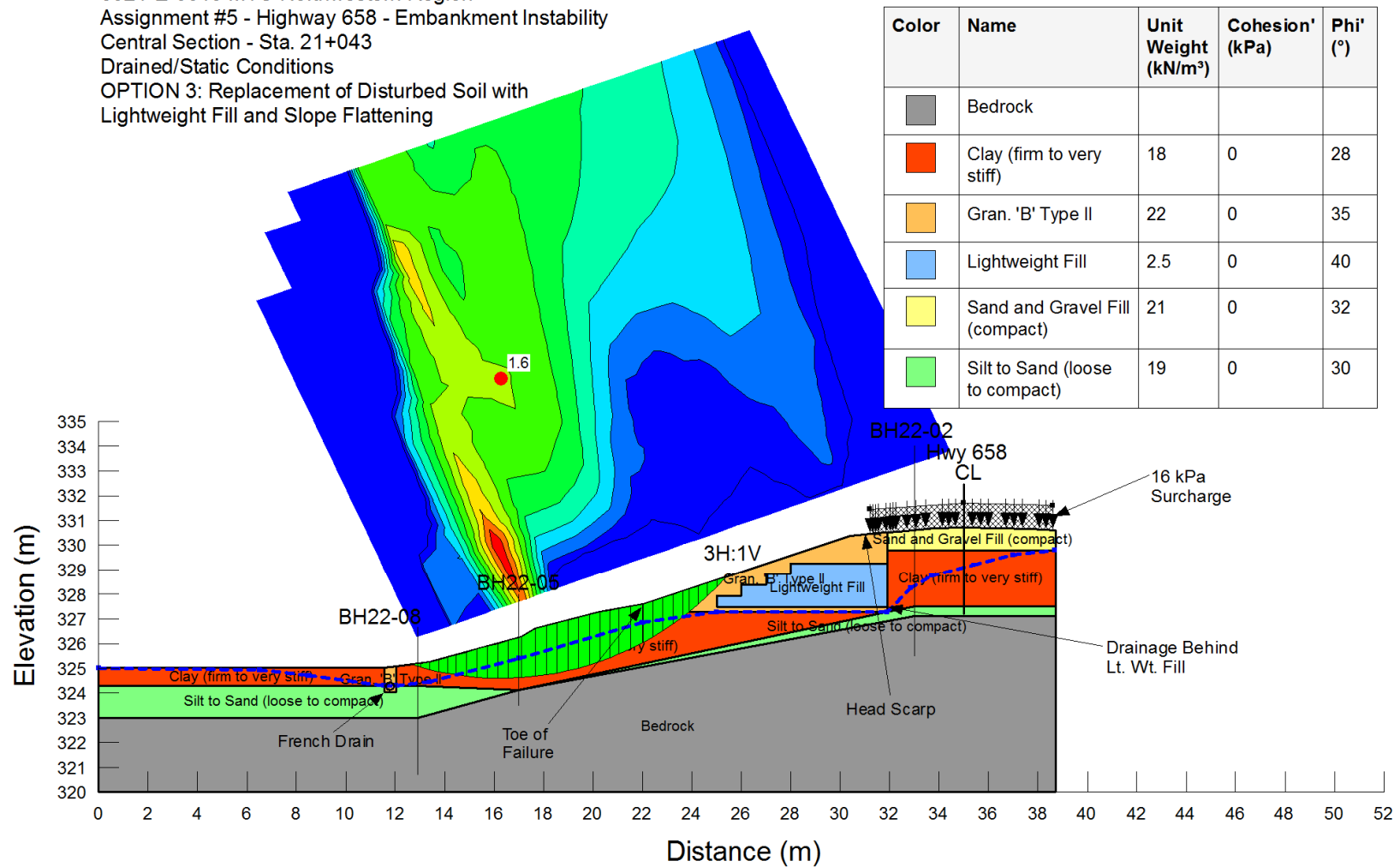


Figure G48. Central Section (Sta. 21+043) – Remedial Option 3: Replacement of Disturbed Soil with Lightweight Fill and Slope Flattening, Toe Failure – Drained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Seismic Conditions  
 OPTION 3: Replacement of Disturbed Soil with  
 Lightweight Fill and Slope Flattening

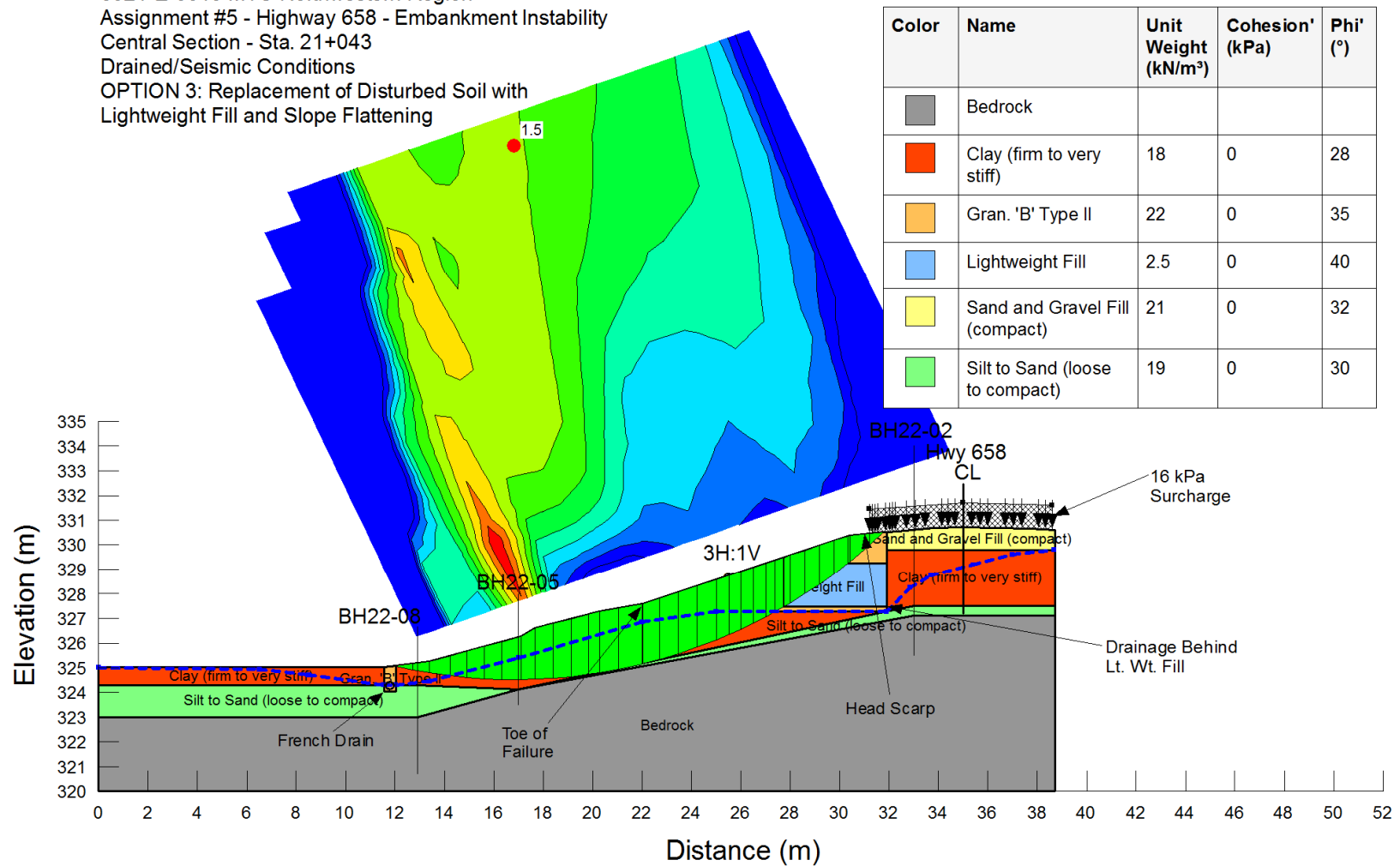


Figure G49. Central Section (Sta. 21+043) – Remedial Option 3: Replacement of Disturbed Soil with Lightweight Fill and Slope Flattening, Global Failure – Drained Seismic Conditions



6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Seismic Conditions  
 OPTION 3: Replacement of Disturbed Soil with  
 Lightweight Fill and Slope Flattening

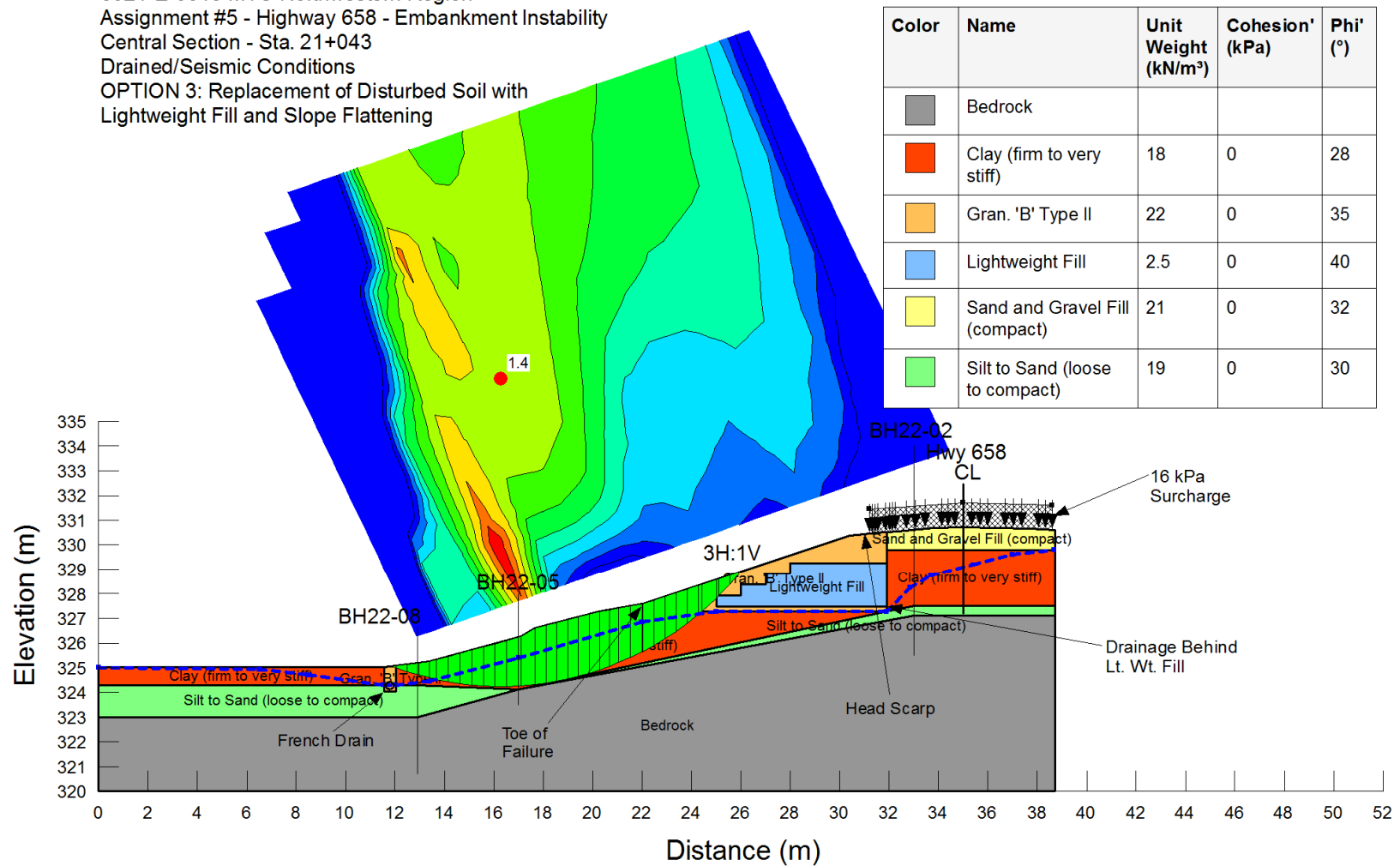


Figure G50. Central Section (Sta. 21+043) – Remedial Option 3: Replacement of Disturbed Soil with Lightweight Fill and Slope Flattening, Toe Failure – Drained Seismic Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Static Conditions  
 OPTION 4: Installation of Anchors Embedded  
 into Bedrock

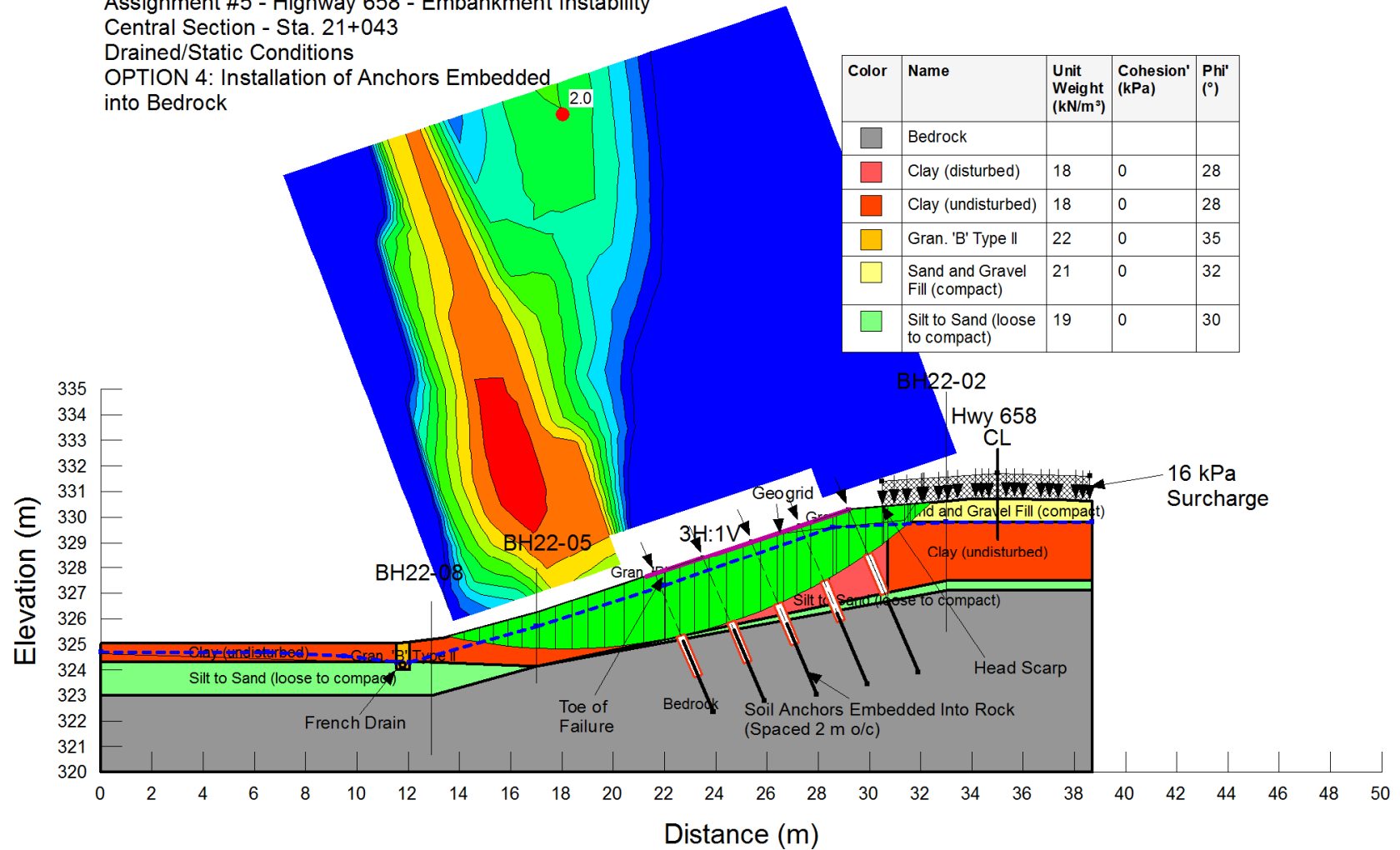


Figure G51. Central Section (Sta. 21+043) – Remedial Option 4: Installation of Anchors Embedded into Bedrock, Global Failure – Drained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Static Conditions  
 OPTION 4: Installation of Anchors Embedded  
 into Bedrock

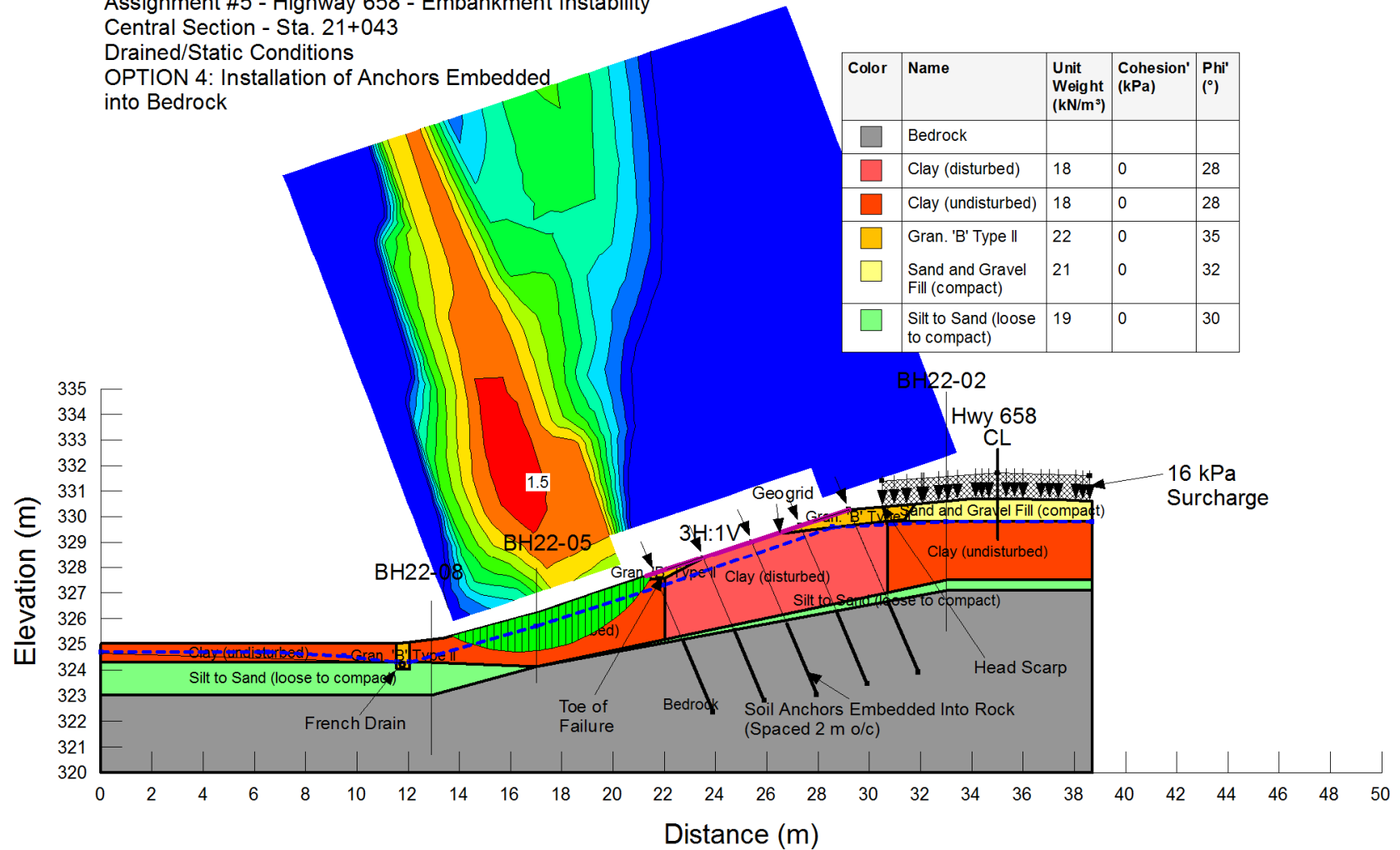


Figure G52. Central Section (Sta. 21+043) – Remedial Option 4: Installation of Anchors Embedded into Bedrock, Toe Failure – Drained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Seismic Conditions  
 OPTION 4: Installation of Anchors Embedded  
 into Bedrock

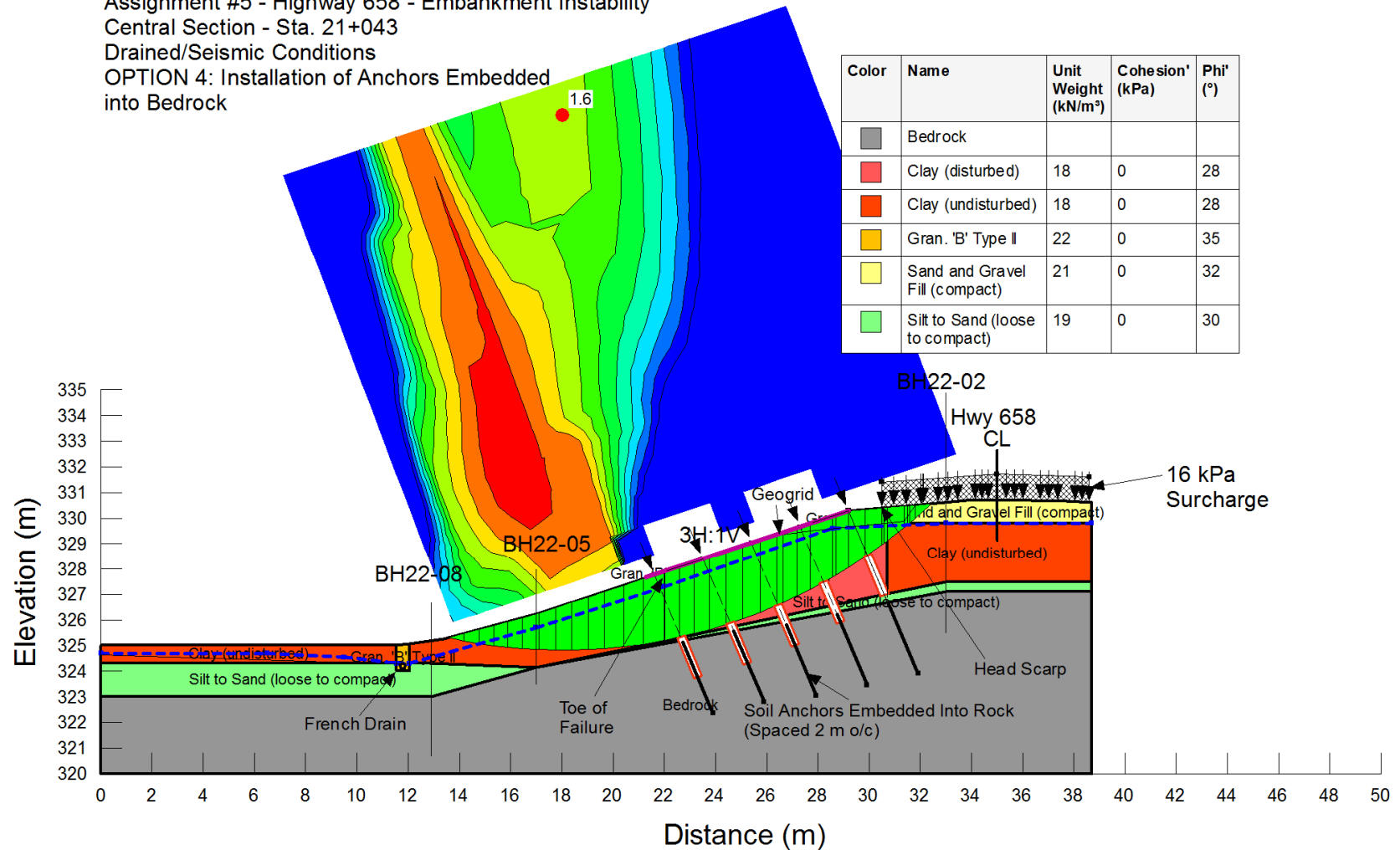


Figure G53. Central Section (Sta. 21+043) – Remedial Option 4: Installation of Anchors Embedded into Bedrock, Global Failure – Drained Seismic Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Seismic Conditions  
 OPTION 4: Installation of Anchors Embedded  
 into Bedrock

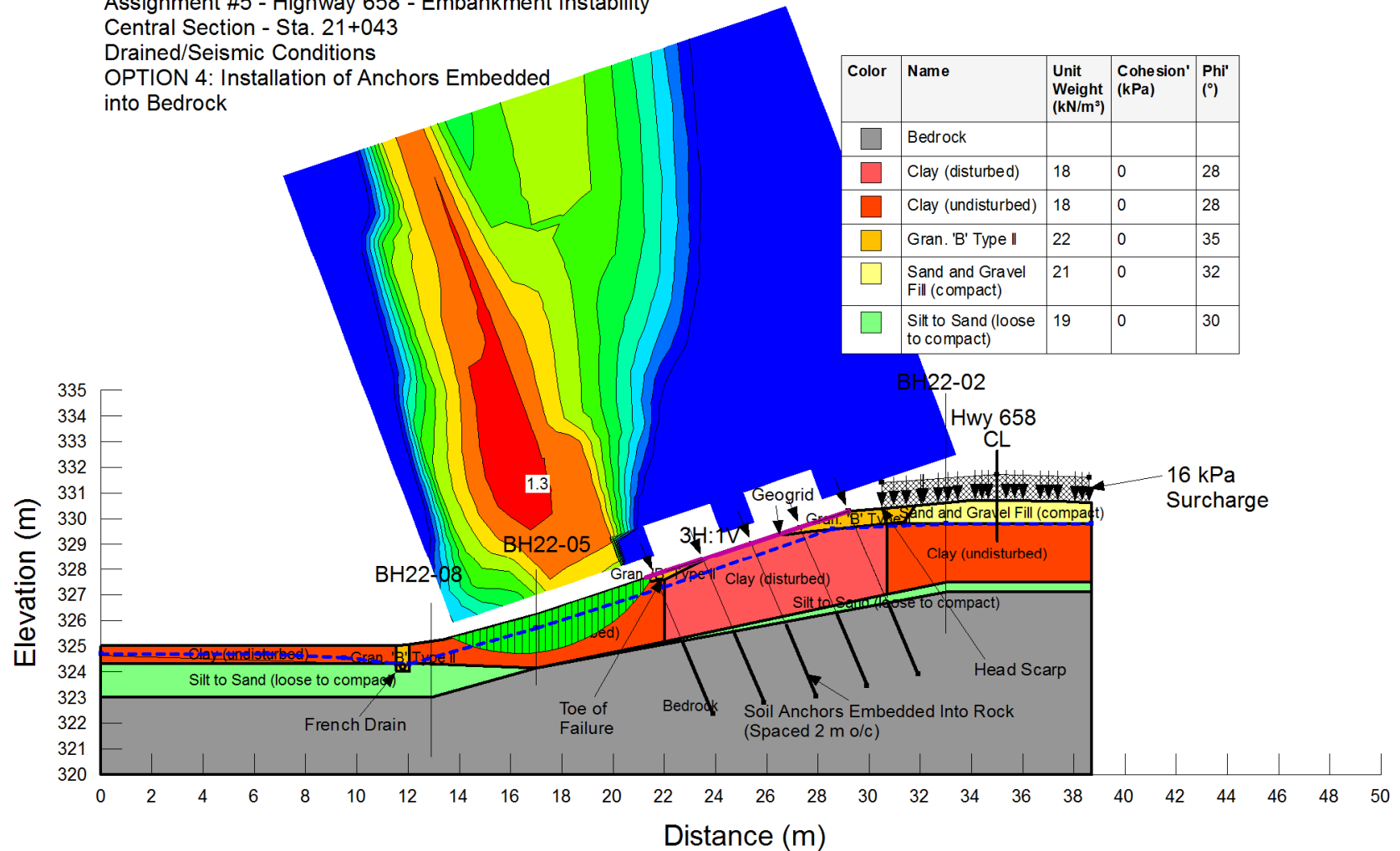


Figure G54. Central Section (Sta. 21+043) – Remedial Option 4: Installation of Anchors Embedded into Bedrock, Toe Failure – Drained Seismic Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Static Conditions  
 OPTION 5: Installation of Timber Piles

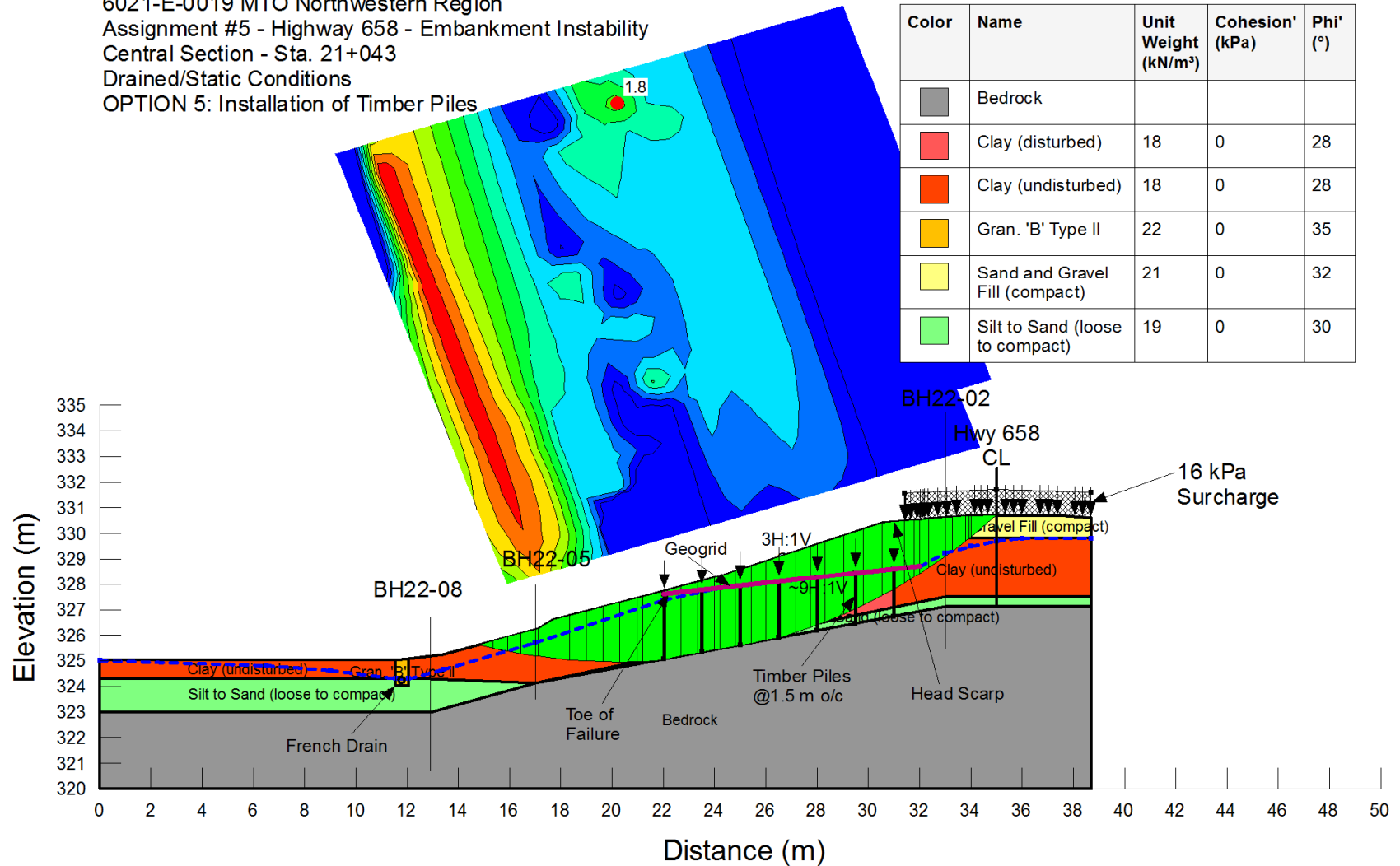


Figure G55. Central Section (Sta. 21+043) – Remedial Option 5: Installation of Timber Piles, Global Failure – Drained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Static Conditions  
 OPTION 5: Installation of Timber Piles

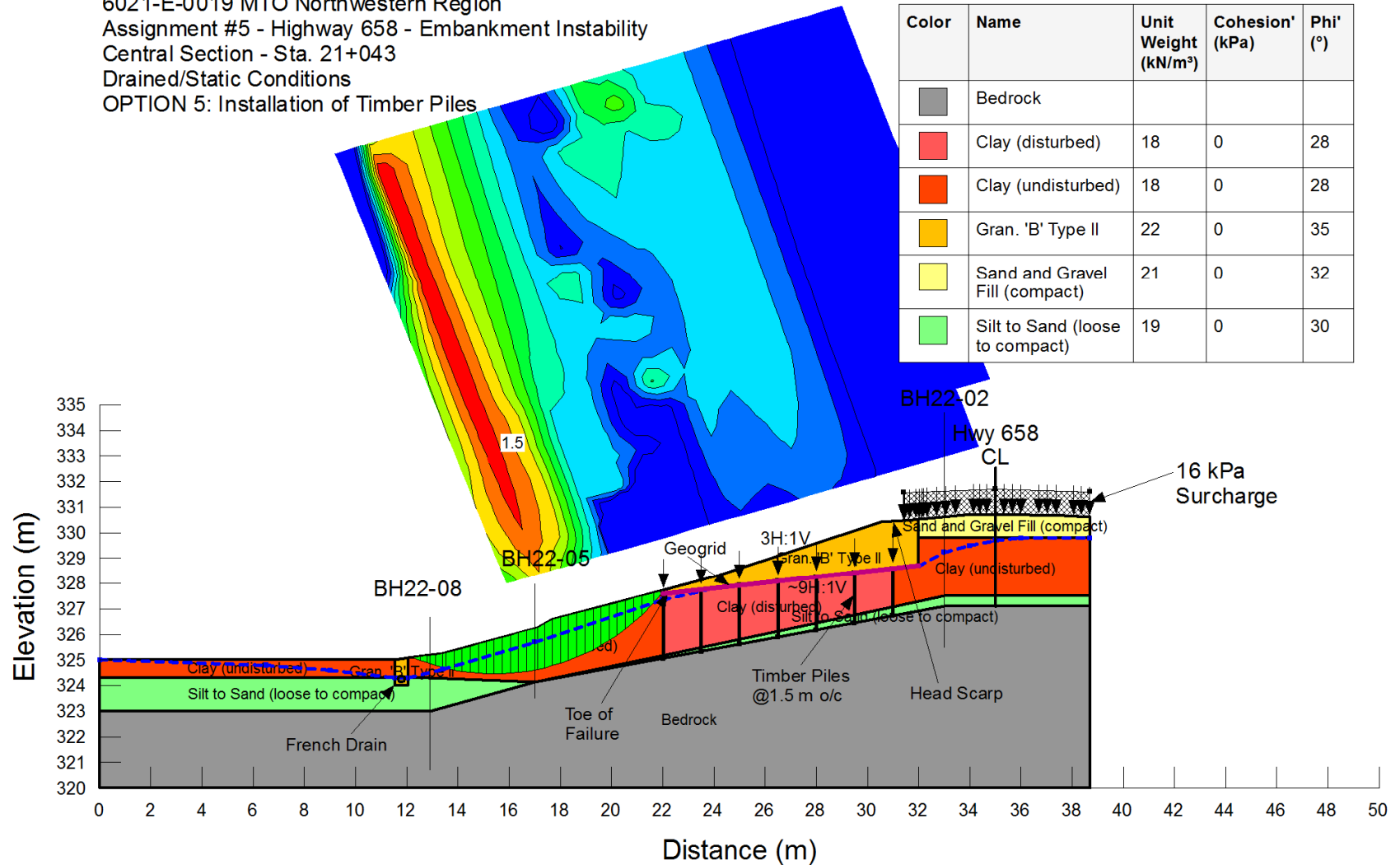


Figure G56. Central Section (Sta. 21+043) – Remedial Option 5: Installation of Timber Piles, Toe Failure – Drained Static Conditions

6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Seismic Conditions  
 OPTION 5: Installation of Timber Piles

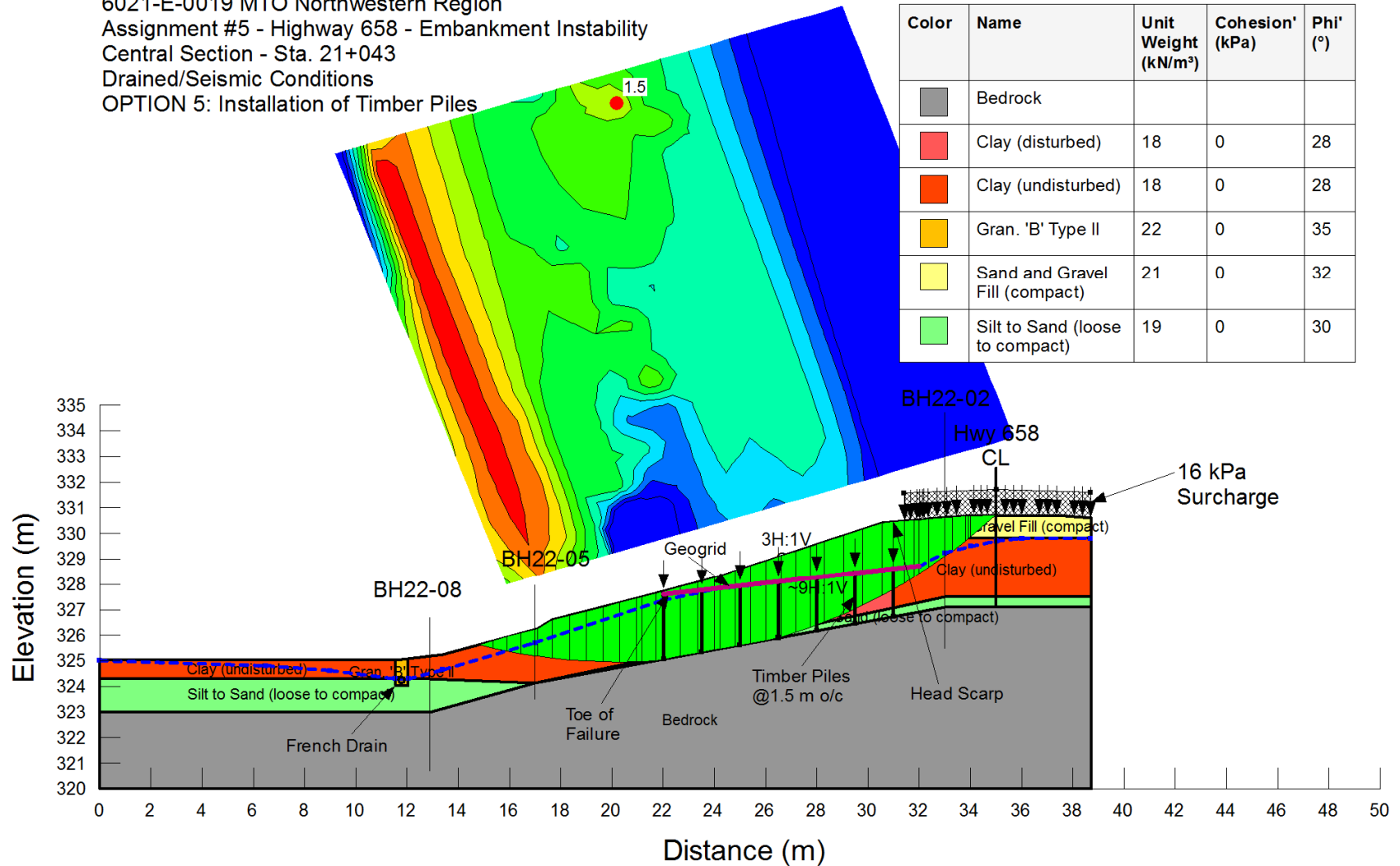


Figure G57. Central Section (Sta. 21+043) – Remedial Option 5: Installation of Timber Piles, Global Failure – Drained Seismic Conditions



6021-E-0019 MTO Northwestern Region  
 Assignment #5 - Highway 658 - Embankment Instability  
 Central Section - Sta. 21+043  
 Drained/Seismic Conditions  
 OPTION 5: Installation of Timber Piles

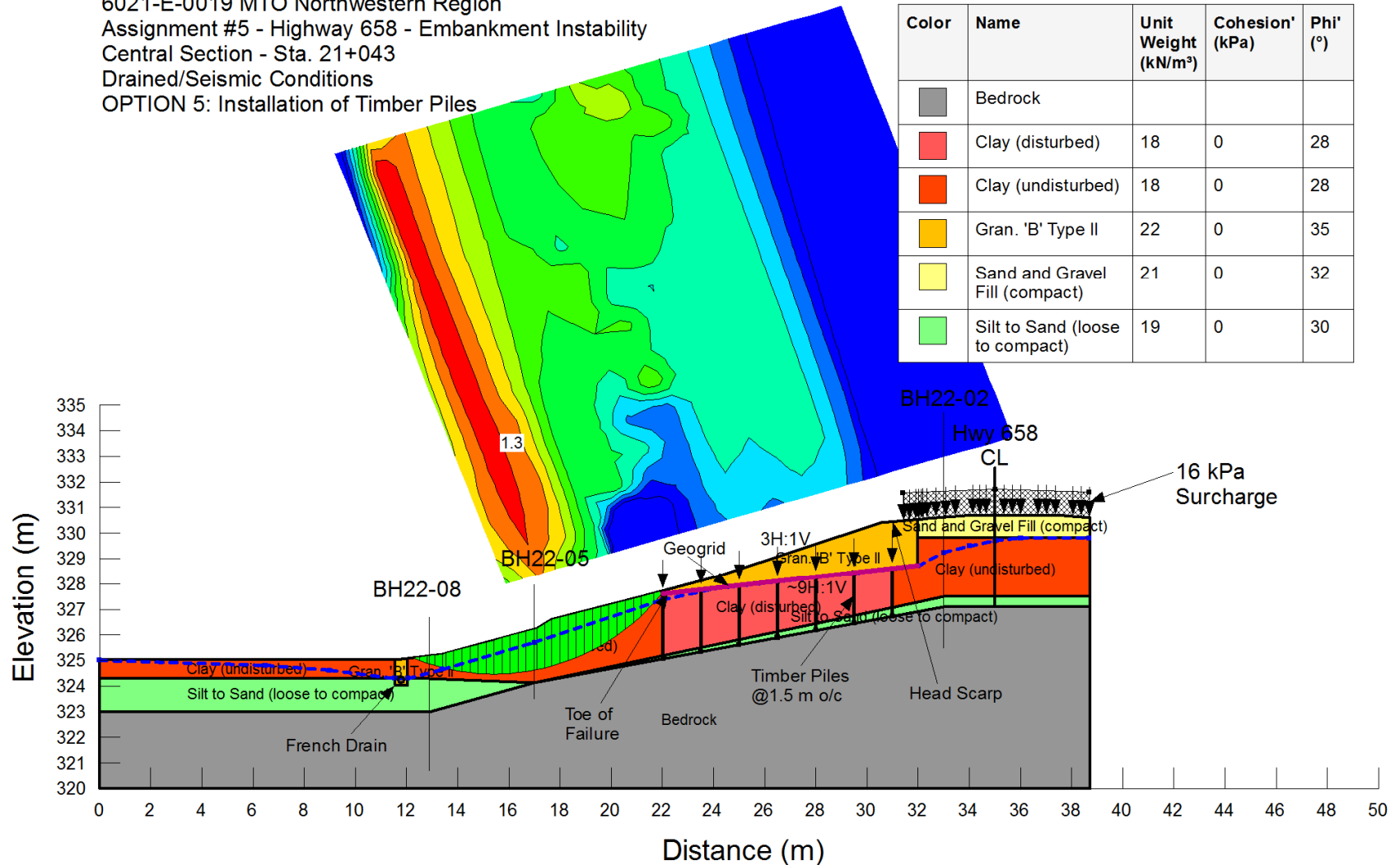
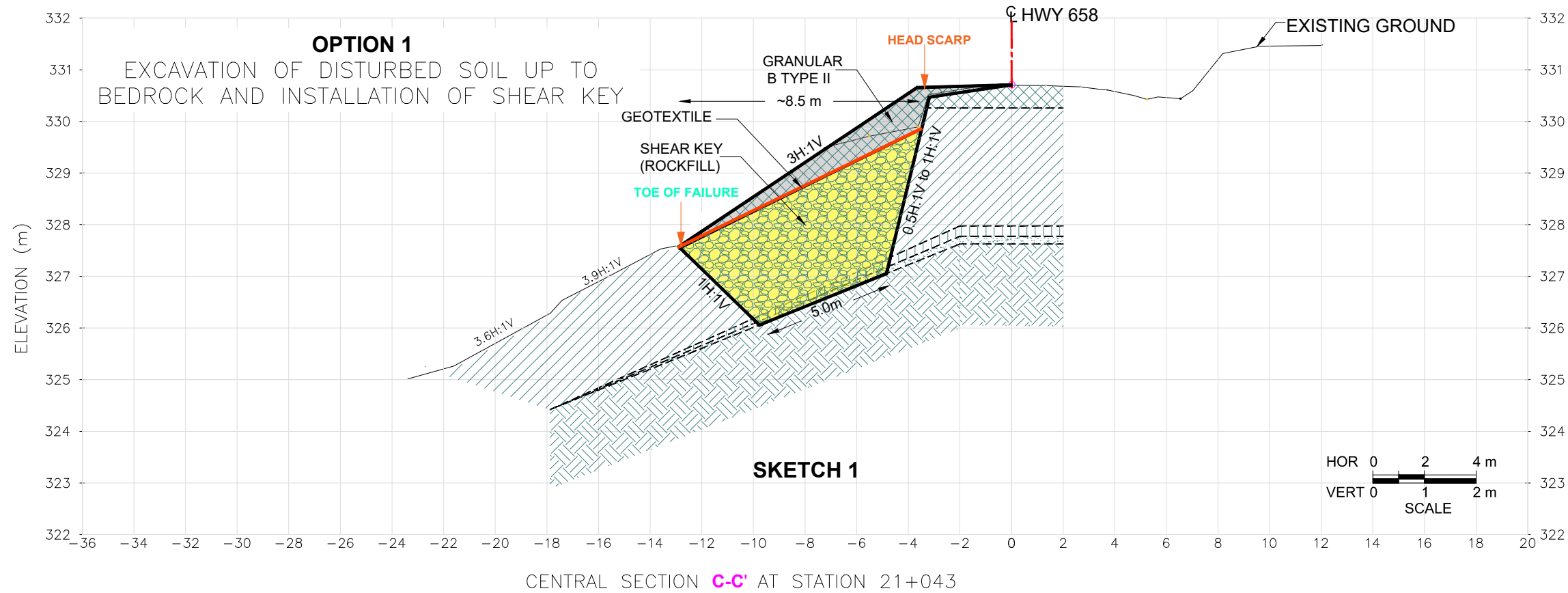
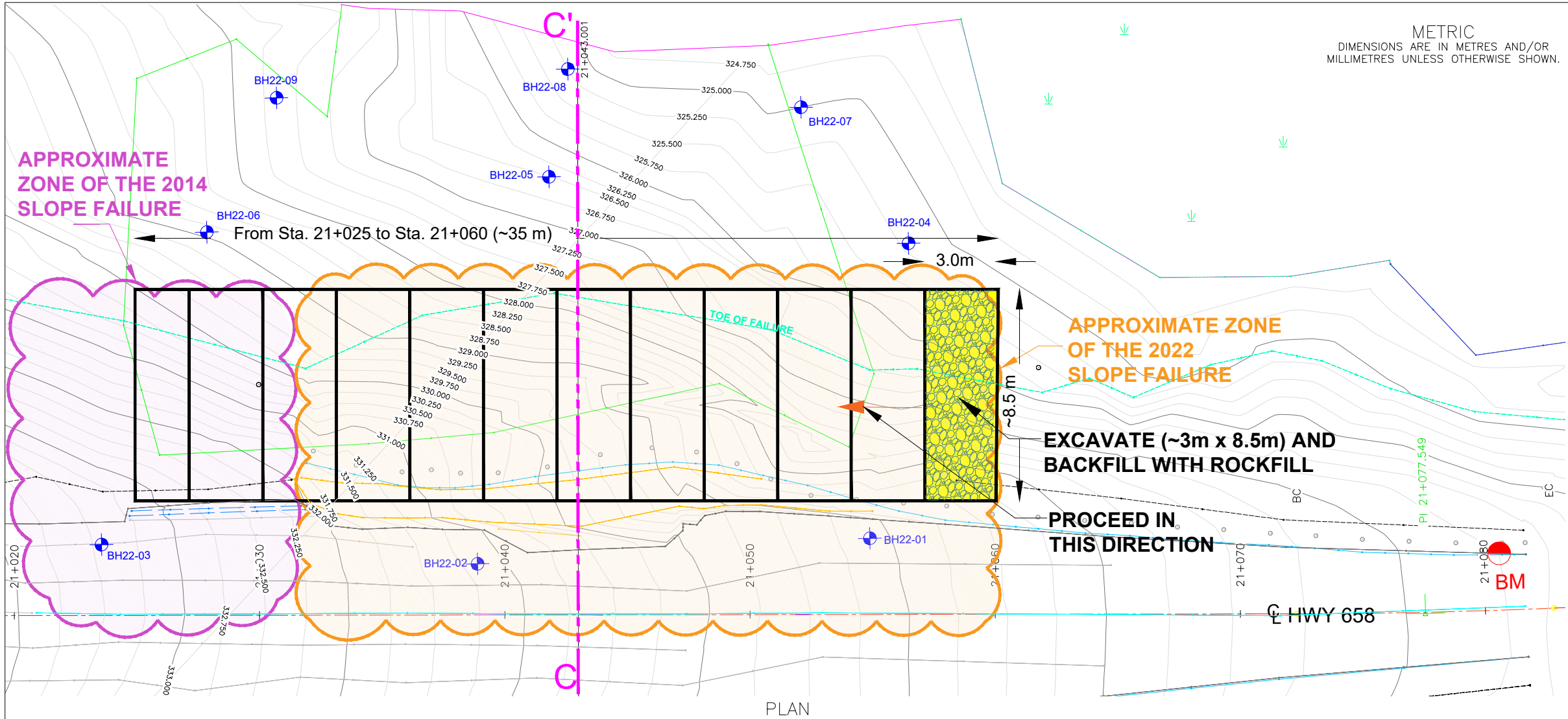


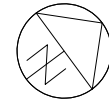
Figure G58. Central Section (Sta. 21+043) – Remedial Option 5: Installation of Timber Piles, Toe Failure – Drained Seismic Conditions

## Appendix H – Remediation Measures - Sketches

FILE NAME: I:\2003-Brampton\Proposals\Projects\International\WTO Projects\Retainer NWR\6021-E-0019\A 5 - Hwy 658 - Remedial Options.dwg  
MODIFIED: 2022-11-25 15:06



CONT No. 6021-E-0019  
ASSIG No. 5  
GWP No.

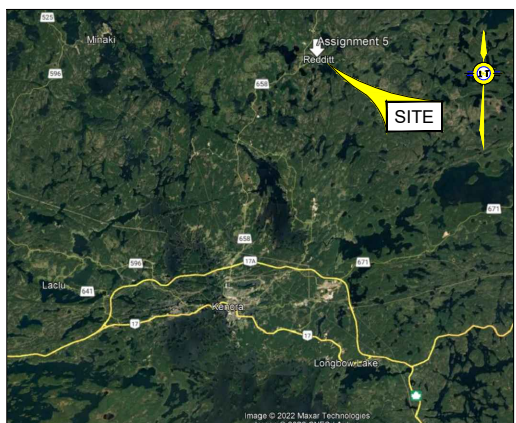


Foundation Investigation and Design for an Embankment  
Instability on Highway 658, Township of Redditt  
Latitude: 49.975717°, Longitude: -94.393332°  
Remediation Options

SHEET  
1



EXP SERVICES INC.



KEY PLAN  
N.T.S.

LEGEND

- Proposed Borehole Location
- Bench Mark Location

SUBMISSION FOR MTO REVIEW				
NO	DATE	BY	DESCRIPTION	
PROJECT No.	ADM-21019842-E0		GEOCRE No.	
SUBM'D SH	CHKD. SM	DATE	NOV. 18, 2022	SITE
DRAWN SH	CHKD. TC	APPRD SG	DWG	01



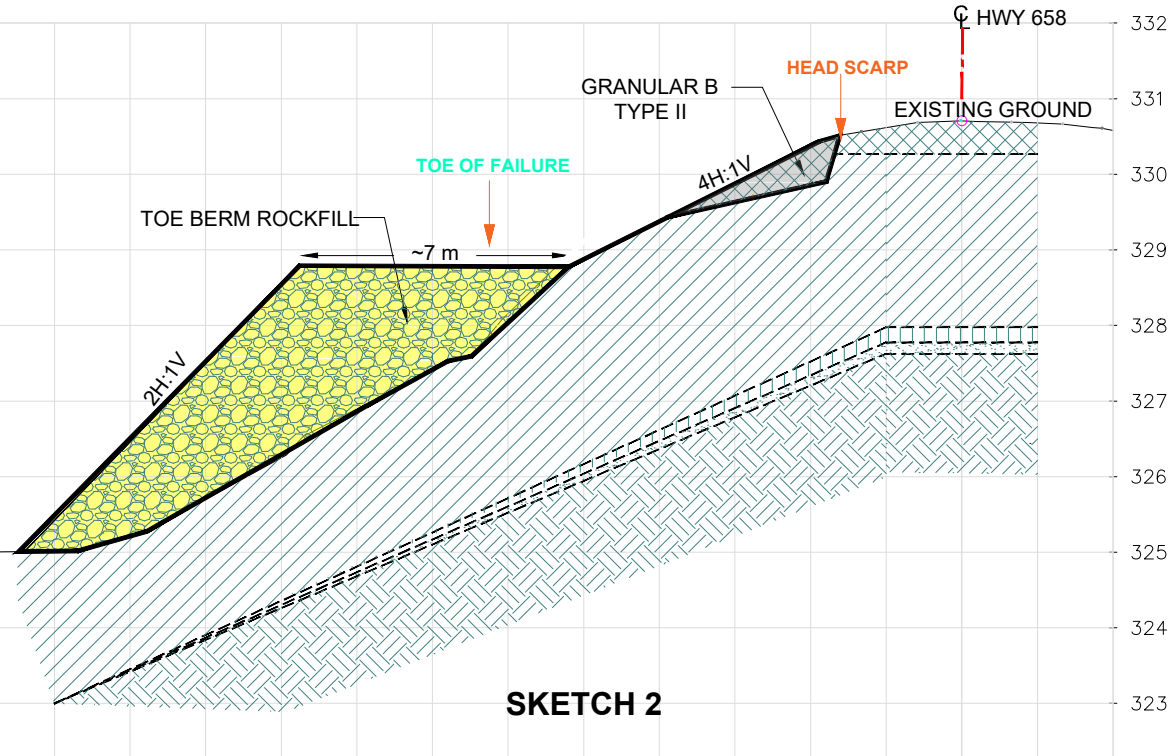


EXP SERVICES INC.



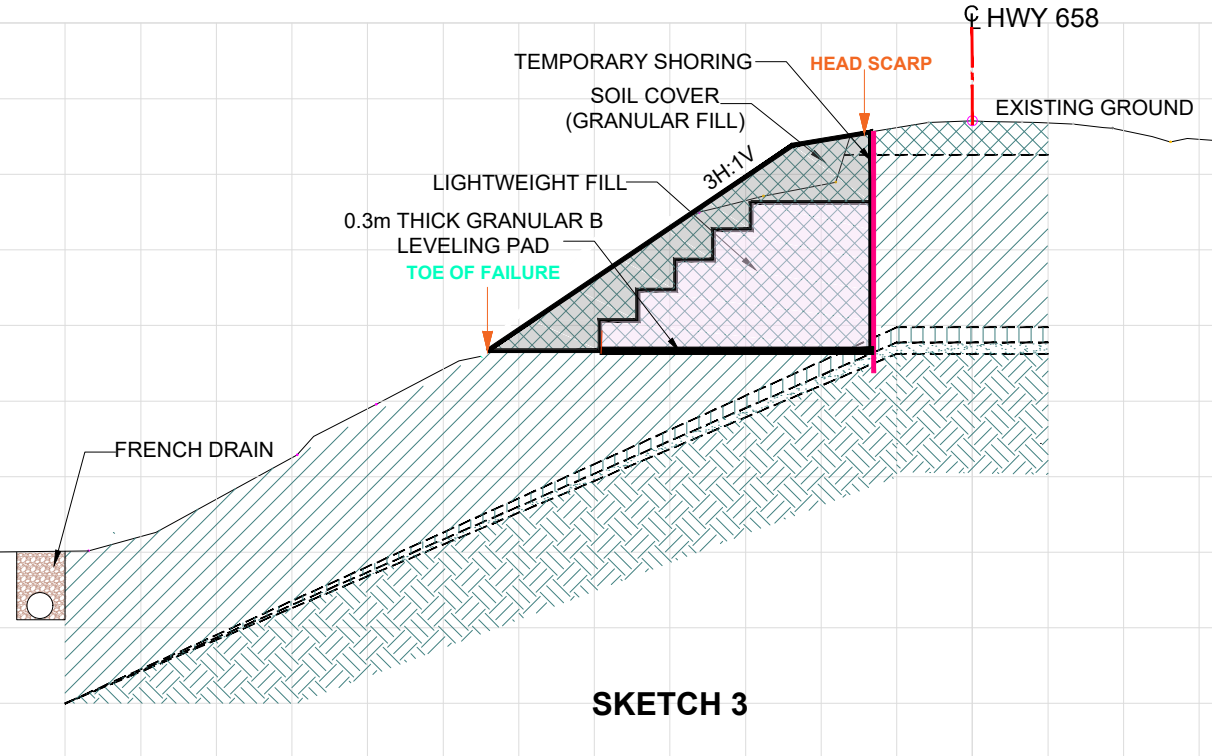
KEY PLAN  
N.T.S.

**OPTION 2**  
CONSTRUCTION OF TOE BERM AND  
SLOPE FLATTENING



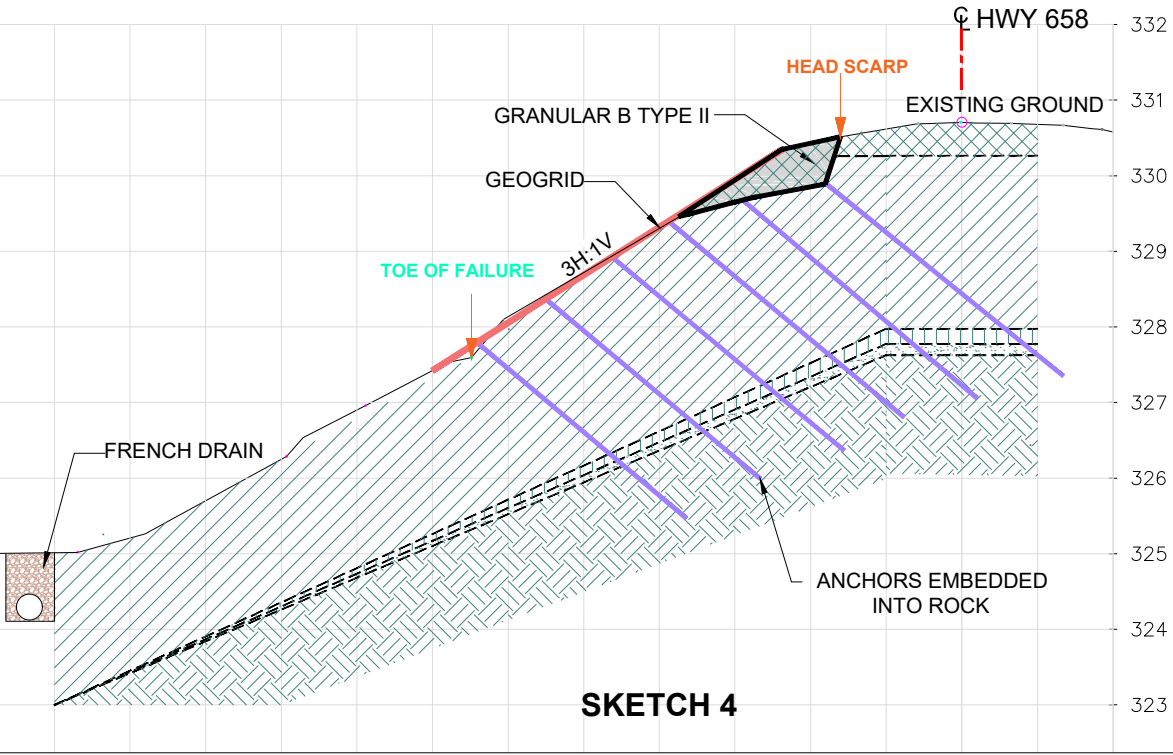
SKETCH 2

**OPTION 3**  
REPLACEMENT OF DISTURBED SOIL WITH LIGHTWEIGHT  
FILL AND SLOPE FLATTENING



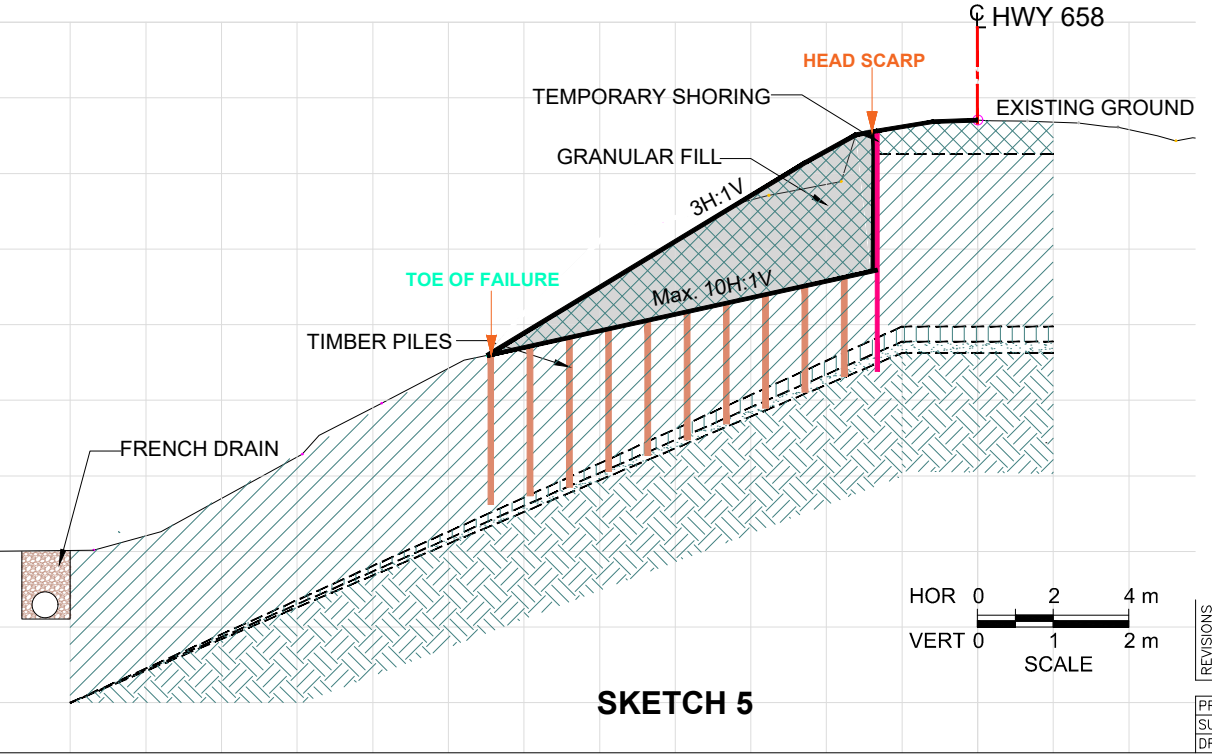
SKETCH 3

**OPTION 4**  
INSTALLATION OF ANCHORS EMBEDDED  
INTO BEDROCK

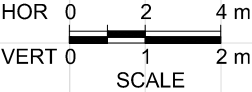


SKETCH 4

**OPTION 5**  
INSTALLATION OF TIMBER PILES



SKETCH 5



SUBMISSION FOR MTO REVIEW					
NO	DATE	BY	DESCRIPTION		
PROJECT No. ADM-21019842-E0			GEOCRETS No.		
SUBM'D SH	CHKD. SM	DATE NOV. 18, 2022	SITE		
DRAWN SH	CHKD. TC	APPRD SG	DWG	02	

Appendix I –  
Technical Memorandum – Site Review



July 26, 2022

Mr. Matthew Leavitt, P.Eng.  
Ministry of Transportation  
Geotechnical Section, Northwestern Region  
615 James Street South  
Thunder Bay, ON P7E 6P6

via email: [Matthew.Leavitt@ontario.ca](mailto:Matthew.Leavitt@ontario.ca)

Mr. Brady Lin, P. Eng.  
Ministry of Transportation  
Foundation Section  
Sir William Hearst Avenue  
Downsview, ON M3M 1J8

Agreement No.: 6021-E-0019, Assignment 5  
Subject: Site Review Geotechnical Commentary for Highway 658 Slope Instability (2022)  
(Latitude: 49.975717°; Longitude: - 94.393332°), Redditt, Ontario

Dear Mr. Leavitt:

## 1. INTRODUCTION

EXP was on site to geotechnically review the slope instability along Highway 658 that occurred in late June of 2022 in Redditt, Ontario. The recent slope instability occurred along the west side of Highway 658 just south of the Black River bridge and immediately north of a past slope instability in the area that occurred in 2014.

EXP conducted a site review under our current Ontario Ministry of Transportation (MTO) Northwestern Ontario geotechnical foundation retainer as a requested emergency response from the MTO (6021-E-0019, Assignment No. 5) for this project site. The factual and photographic documentation, coupled with geotechnical commentary of various project facets associated with the recent 2022 slope instability along Highway 658, are provided in the following sections of this site review report.

EXP has also relied on the following reports provided by the MTO, with respect to project site relevance:

- “Highway 658, Redditt, Highway Embankment Instability at Lot 116, File Number 2014-01309”, Memorandum prepared by the Ministry of Transportation on July 15, 2014 (MTO 2014); and

- “Soils and Foundation Report, Proposed Realignment of Black River Bridge, Redditt, Ontario”, GEOCREs No. 52E-22, prepared by Morton, Dodds & Partners Limited in October 1978 (Morton 1978).

## 2. VISUAL SITE REVIEW (2022 SLOPE INSTABILITY - JULY 7, 2022)

The undersigned, Joel Kliner, P.Eng., conducted a visual site reconnaissance of the June 2022 slope instability on July 7, 2022 at the request of the MTO as an emergency response measure. The MTO initially reviewed/photographed the slope instability on June 27, 2022. The factual and photographic documentation and visual review of the 2022 slope instability by EXP has been detailed within this site review report.

### 2.1 2022 Slope Instability Layout/Geometry along Highway 658

The 2022 slope instability was observed along the west shoulder of Highway 658, south of the Black River bridge and directly north of a previously documented slope instability which occurred in 2014 (Photo 1).



*Photo 1 (Looking North): July 7, 2022 – Highway 658 Slope Instability*

The recent slope instability was reviewed and documented with the following particulars measured or visually approximated by EXP while on site:

- Slope Instability Length  $\approx 26.0$  m, Aligned North-South Parallel to HWY 658 (Photo 1);
- Slope Height  $\approx 3.5$  m to  $\approx 5.0$  m (Photo 2);
- Slope Instability  $\approx 0.3$  m to  $\approx 1.2$  m into Highway 658 Shoulder and Southbound Lane (Photo 1);
- Head Scarp Depth  $\approx 0.2$  m to  $\approx 1.2$  m, Increasing in Depth from South to North (Photo 3);
- Existing Guardrail and Supports Bent and Twisted with Down Slope Movement (Photo 1);



- Large Tension Cracks with Variable Apertures and Depths. Tension cracks were observed to the west of Highway 658 within the slumped soils, approximately 5.0 m to 6.0 m west of the exposed head scarp. These tension cracks ranged from 0.3 m to upwards of 1.0 m in depth with apertures upwards of 0.5 m in width and drops of 0.1 m or greater across the apertures (Photo 4 and Photo 5); and
- Existing Embankment Slopes to the West as Steep as 1H:1V at Leading Edge of Slope/Instability (Photo 2 and Photo 6).



*Photo 2 (Looking East): July 7, 2022 – Highway 658 Slope Instability Toe to Roadway*



*Photo 3 (Looking North): July 7, 2022 – Slope Instability Head Scarp Parallel to Highway 658*





*Photo 4 and Photo 5: July 7, 2022 – Tension Cracks Directly West of Highway 658 within Slope Instability*



*Photo 6 (Looking South): July 7, 2022 – Highway 658 West Embankment Slope Instability*



## 2.2 Visual Signs of Structure and/or Vegetation Disturbance

Various trees and shrubs were observed to exhibit a downslope lean, consistent with the down slope movement of the slope instability (Photo 7).



*Photo 7 (Looking Southeast): July 7, 2022 – Leaning Trees 2022 Slope Instability*

Two existing power poles were observed to be in close vicinity of the 2022 slope instability. These power poles are just beyond the northern and southern extents of the 2022 slope instability and exhibit a lean down slope (Photo 8). Comparative photos of these power poles from the MTO 2014 report (2014 slope instability) also show these power poles with similar down slope leans (Photo 9); thus, it is inconclusive if the 2022 slope instability had any further effect on these power poles.



*Photo 8 (Looking North): July 7, 2022 – Leaning Power Poles 2022 Slope Instability*



*Photo 9 (Looking North): July 3, 2014 – Leaning Power Poles 2014 Slope Instability*

### 2.3 Surficial Conditions of Roadway and/or Shoulder

A pre-existing gravel-treated roadway patch/repair was observed along the west edge of the southbound lane and shoulder, fully encompassing the area of instability which had slumped down slope to the west and a portion of the existing roadway edge above the exposed head scarp. The remaining width of the pre-existing treated gravel patch/repair directly above the fresh head scarp ranged from 0.6 m to 0.8 m in width (Photo 10). This portion of the intact roadway and the surface of the slumped soils below the head scarp consisted of what appears to be a treated gravel surface (likely oil emulsion-based tack coat), not a paved asphalt surface, suggesting that a previous roadway repair was undertaken within the current instability area prior to the recent slope movement occurring. This may have been a result of initial slope movements preceding the recent slope instability; however, EXP has not been provided any historic information associated with this observed area of previous roadway repair.





*Photo 10 (Looking North): July 7, 2022 – Highway 658 West Shoulder Gravel-Treated Roadway Repair*

There were no signs of instability observed along the east shoulder or ditch line of Highway 658 across from the recent slope instability, only ditch erosion east of asphalt edge from surficial drainage flow downward from south to north towards the Black River floodplain (Photo 11).



*Photo 11 (Looking North): July 7, 2022 – Highway 658 East Shoulder Surficial Drainage Erosion*

## 2.4 Visually Observed Soil Conditions

EXP also visually assessed the surficial soil conditions using a shovel to expose soils near the head scarp along the roadway embankment (Photo 12 and Photo 13). The soils encountered were noted as follows:

- Asphalt (75 mm thickness)

- Gravelly sand, trace cobbles, moist to wet, and brown in colour (below asphalt to clay subgrade)
- Clay, some silt, trace sand, medium plastic, moist to wet, very soft with pocket penetrometer readings of less than  $0.25 \text{ kg/cm}^2$ , and grey in colour (approximately 2.0 m below roadway)
- No Groundwater seepage observed



*Photo 12 and Photo 13: July 7, 2022 – Soils Adjacent to the Exposed Head Scarp Adjacent to Highway 658*

The surficial soils near the toe of the slope instability were visually observed to consist of a wet to saturated organic layer over a silt/clay mix.

### 3. PRELIMINARY GEOTECHNICAL COMMENTARY ON THE 2022 SLOPE INSTABILITY

The general topography of Highway 658 has a steep downward gradient along the roadway surface from south to north along the section of roadway which has experienced slope instabilities in 2014 and 2022. There is very little surficial drainage control except for the crown of the roadway which directs surface water over the embankment sides and/or ditch lines. As well, the existing west roadway embankment



side slopes appear to be very steep (as steep as 1H:1V in localized sections) with mature vegetation growth in the vicinity of the 2022 slope instability.

Based on EXP's visual observations of the 2022 slope instability, it initially appears that the west side of the roadway embankment was inundated with water at the top of the roadway embankment from surficial drainage along the roadway (e.g., heavy spring precipitation events, snow melt, etc.), coupled with flood water rise along the Black River floodplain area at the embankment toe.

The overall slope instability along the west side of Highway 658 was visually observed to be a wedge of soil that has slumped down upwards of approximately a meter in elevation from roadway surface and slid out westward towards the toe of the failure mass. There was no pronounced soil buckling visually observed along the overall toe of the slope instability, as it appeared the failure mass predominately bulged out mid-slope. Potential contributing causations of the 2022 slope instability are as follows, as each of these factors either increase the "driving" forces or reduce the "resisting" forces of global stability:

- Poor surficial drainage resulting in saturation of the west roadway embankment sand fill and clay subgrade soils, increasing the weight of the embankment soils ("driving" force ↑);
- Saturation of the clay subgrade soils at the embankment toe from flood water rise, increasing the pore water pressures of the clay subgrade soils near the slope toe ("resisting" force ↓);
- Steep roadway embankment geometry ("driving" force ↑); and
- Weak clay subgrade soils ("resisting" force ↓).

The aforementioned factors are preliminary and are not listed in any particular order of effect, as it is difficult to determine the direct impact of each from a visual site reconnaissance. The culminating effects of these, and potentially other contributing factors (unknown), are intertwined and have resulted in the observed slope instability of the roadway embankment.

A detailed geotechnical investigation program, coupled with stability analysis, are recommended to further to assess the slope instabilities along Highway 658 and permanent mitigation/remediation measures.

## 4. 2014 SLOPE INSTABILITY

The previous slope instability immediate south of the recent 2022 slope instability was not the focal point of EXP's site reconnaissance. However, EXP did briefly look at the existing condition of the previous 2014 slope instability area, as it was directly south of the 2022 instability area. The 2014 slope instability area appears to have been remediated and/or stabilized by rough grading/placement of fill soils, coupled with the removal of the previous residential development (i.e., existing house, landscaping and any out-building structures – Photo 14), since the 2014 documentation of this slope instability event (MTO 2014).

The overall 2014 slope instability area was covered with mature vegetation, inclusive of tall grasses/weeds, shrubs and small seedling trees. The 2014 slope instability area consisted of variable

grades (as steep as 1H:1V with short vertical drops of less than 0.3 m to as flat as 3H:1V near the existing slope toe), sloping down with short drops from the Highway 658 roadway surface down to what is assumed to be the toe of the slope instability or toe of previous fill soil placement (Photo 15). Rock fill placement as a slope armoring mechanism was observed at the leading edge of the 2014 slope instability after remedial grading was assumed to be undertaken (Photo 16). No recent signs of slope movement were observed within the overall 2014 slope instability area; however, EXP was not provided any remediation photos after the 2014 slope instability as a comparison to its existing condition as of July 7, 2022.



*Photo 14 (Looking South): July 3, 2014 – Existing House within 2014 Instability Area*



*Photo 15 (Looking West Down Slope): July 7, 2022 – 2014 Instability Area*





*Photo 16 (Looking West Down Slope): July 7, 2022 – 2014 Instability Area with Rock Fill Slope Armoring*

Pavement stress cracks were observed within an area of previous asphalt repair along Highway 658 directly adjacent to the known area of the 2014 slope instability (Photo 17). These stress cracks did not appear to be fresh and are not necessarily indicative of any slope movement since 2014, as they may only be associated with the previous roadway repairs (base/subbase gravel and asphalt patching).



*Photo 17 (Looking North): July 7, 2022 – Asphalt Repair Patch Adjacent to 2014 Instability Area*

## 5. IMMEDIATE TEMPORARY CONTROL MEASURES

As there is a potential for continued retrogressive/progressive slope instability of the exposed head scarp and further down slope movement of the failed mass of slumped soils adjacent to Highway 658, it is recommended to reduce Highway 658 to one lane adjacent to the slope instability until permanent repairs can be implemented. A minimum lane restriction of 2.0 m from the leading edge of the exposed head scarp near the existing pavement edge is recommended to be implemented at this time. Temporary load limits should also be considered for this section of Highway 658 to prevent unnecessary traffic loading adjacent to the current slope instability. As well, EXP should be informed of any further signs of continued slope movement in the area by MTO personnel during their highway maintenance routines.

## 6. RECOMMENDED GEOTECHNICAL INVESTIGATION PROGRAM

The MTO also requested EXP to provide a recommendation for an initial geotechnical investigation program to assess the following for the continuation of this assignment:

- “Foundation investigation and design for an embankment instability that has occurred on Hwy 658 in the township of Redditt at the north of Hwy 658 and just south of Black River (approx. GPS coordinates: 49.975717, -94.393332)”. The movement is adjacent to a similar embankment instability that occurred back in 2014 where temporary remediation work was done to hold the slope in place. The scope of this investigation shall cover both new instability area and previous instability area for a total of ~60 m. A technical memo prepared by MTO Foundation in 2014 is available for review.”
- “The service provider shall conduct site reconnaissance, propose and complete borehole investigation, determine instability mechanism, and provide recommendations. Deliverables shall include a FIR and FIDR.”

EXP recommends the following initial geotechnical investigation program with a total of 9 boreholes (see attached Proposed Initial Geotechnical Investigation Borehole Layout) to assess the slope instability areas (2014 and 2022) along Highway 658 in Redditt, Ontario:

- **Roadway along Highway 658 Adjacent to Instability Areas** → 3 boreholes to 15 m (solid stem or hollow stem may be required) or refusal with one cored 1.5 m to prove bedrock (if/when encountered);
- **Slope Toe of Instability Areas** → 6 boreholes to 10 m (solid stem or hollow stem may be required) or refusal with one cored 1.5 m to prove bedrock (if/when encountered), 3 at the existing instability toe and 3 approximately 5m to 10 m further away from the toe (for stability analyses at three sections);
- 25 mm piezometers installed in each borehole for short-term and long-term groundwater monitoring; and

- Typical SPT sampling to be conducted (every 0.75 m in upper 3 m, every 1.5 m thereafter. Shear vanes in cohesive soils, if encountered. As well, Shelby Tube sampling for relatively undisturbed soil samples in each borehole.

NOTE: It may be possible to reduce the required number of boreholes beyond the slope instability toe, if observed soil conditions appear to be consistent on the flatter area at/near the toe of the instability surfaces.

Due to the lack of access to the slope toe of the instability areas (i.e, slopes too steep for drill rig traversing, soft/marshy soils at the flatter toe areas, overhead power lines, etc.), it is anticipated that a crane will be required to drop a portable drill rig at the slope toe. EXP is currently looking into available drill rig options such as a portable skid-mounted B20L rig or a tripod rig to facilitate fieldwork logistics for this project site.

The proposed fieldwork program above, or a variation of similar scope agreed upon by the MTO, will be provided with fee estimates under separate cover.

## 7. CLOSURE

The recent slope instability along the west side of the Highway 658 roadway embankment which occurred in late June 2022 has been visually review by EXP on July 7, 2022. This report is limited to the visual observations available at the time of EXP's site review. Detailed geotechnical investigation is recommended to assess the cause(s) of slope instabilities that have occurred in this general area of Highway 658 (2014 and 2022 slope instability areas) and aid in permanent mitigation/remediation measures to be implemented to repair the roadway embankment and provide an acceptable long-term factor of safety for global stability.

This report was prepared for the exclusive use of the Ontario Ministry of Transportation and their designated consultants and agents, and may not be used by other parties without the written consent of EXP Services Inc. The attached "Interpretation & Use of Study and Report" forms an integral part of this report and must be included with any copies of this report.

We trust that this letter report meets your present requirements. Should you have any questions or comments, please contact the undersigned at your convenience.

Sincerely,

EXP Services Inc.

Prepared by:

Reviewed by:



Joel Kliner, M.Sc., P.Eng.  
Senior Geotechnical Engineer

A handwritten signature in blue ink, appearing to read "D. Georgiou".

Demetri N. Georgiou, M.A.Sc., P.Eng.  
Principal Engineer/Branch Manager

A handwritten signature in blue ink, appearing to read "Silvana Micic".

Silvana Micic, Ph.D., P.Eng.  
Senior Geotechnical Engineer  
Project Manager

A handwritten signature in blue ink, appearing to read "Stan E. Gonsalves".

Stan E. Gonsalves, M.Eng., P.Eng.  
Principal Engineer  
Designated MTO Foundation Contact

Enclosures:      Proposed Initial Geotechnical Investigation Borehole Layout  
                         Interpretation & Use of Study and Report





## **INTERPRETATION & USE OF STUDY AND REPORT**

### **1. STANDARD OF CARE**

This study and Report have been prepared in accordance with generally accepted engineering consulting practices in this area. No other warranty, expressed or implied, is made. Engineering studies and reports do not include environmental consulting unless specifically stated in the engineering report.

### **2. COMPLETE REPORT**

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. WE CANNOT BE RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

### **3. BASIS OF THE REPORT**

The Report has been prepared for the specific site, development, building, design or building assessment objectives and purpose that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document are only valid to the extent that there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation.

### **4. USE OF THE REPORT**

The information and opinions expressed in the Report, or any document forming the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT. WE WILL CONSENT TO ANY REASONABLE REQUEST BY THE CLIENT TO APPROVE THE USE OF THIS REPORT BY OTHER PARTIES AS "APPROVED USERS". The contents of the Report remain our copyright property and we authorize only the Client and Approved Users to make copies of the Report only in such quantities as are reasonably necessary for the use of the Report by those parties. The Client and Approved Users may not give, lend, sell or otherwise make the Report, or any portion thereof, available to any party without our written permission. Any use which a third party makes of the Report, or any portion of the Report, are the sole responsibility of such third parties. We accept no responsibility for damages suffered by any third party resulting from unauthorized use of the Report.

### **5. INTERPRETATION OF THE REPORT**

- a. Nature and Exactness of Descriptions: Classification and identification of soils, rocks, geological units, contaminant materials, building envelopment assessments, and engineering estimates have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature and even comprehensive sampling and testing programs, implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations, or building envelope descriptions, utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of, and accept, this risk. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.
- b. Reliance on Provided information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the report as a result of misstatements, omissions, misrepresentations or fraudulent acts of persons providing information.
- c. To avoid misunderstandings, EXP Services Inc. (EXP) should be retained to work with the other design professionals to explain relevant engineering findings and to review their plans, drawings, and specifications relative to engineering issues pertaining to consulting services provided by EXP. Further, EXP should be retained to provide field reviews during the construction, consistent with building codes guidelines and generally accepted practices. Where applicable, the field services recommended for the project are the minimum necessary to ascertain that the Contractor's work is being carried out in general conformity with EXP's recommendations. Any reduction from the level of services normally recommended will result in EXP providing qualified opinions regarding adequacy of the work.

### **6.0 ALTERNATE REPORT FORMAT**

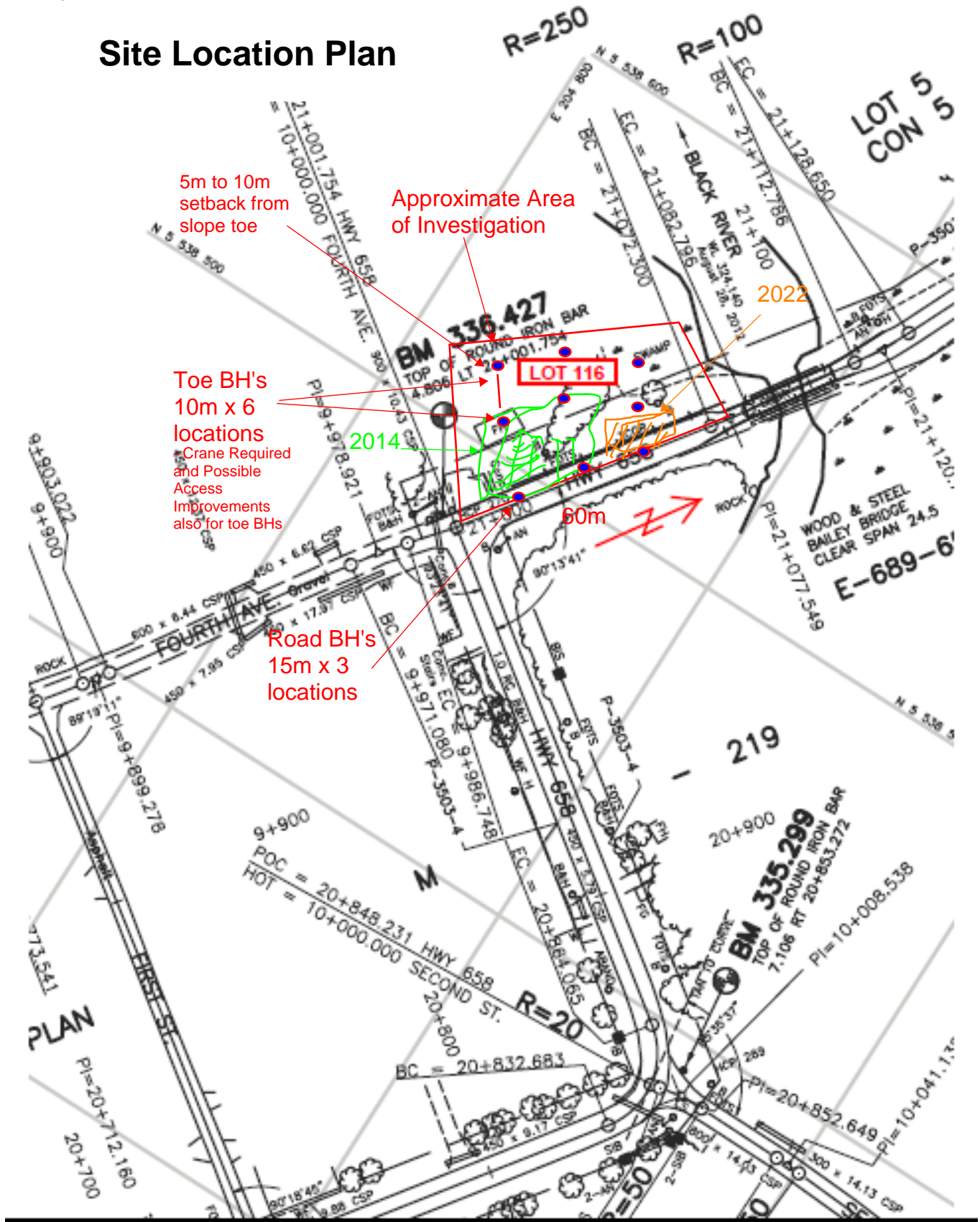
When EXP submits both electronic file and hard copies of reports, drawings and other documents and deliverables (EXP's instruments of professional service), the Client agrees that only the signed and sealed hard copy versions shall be considered final and legally binding. The hard copy versions submitted by EXP shall be the original documents for record and working purposes, and, in the event of a dispute or discrepancy, the hard copy versions shall govern over the electronic versions. Furthermore, the Client agrees and waives all future right of dispute that the original hard copy signed version archived by EXP shall be deemed to be the overall original for the Project.

The Client agrees that both electronic file and hard copy versions of EXP's instruments of professional service shall not, under any circumstances, no matter who owns or uses them, be altered by any party except EXP. The Client warrants that EXP's instruments of professional service will be used only and exactly as submitted by EXP.

The Client recognizes and agrees that electronic files submitted by EXP have been prepared and submitted using specific software and hardware systems. EXP makes no representation about the compatibility of these files with the Client's current or future software and hardware systems.

# Proposed Initial Geotechnical Investigation Borehole Layout - Highway 658 (Redditt, Ontario)

## Site Location Plan



## Appendix J – NSSP



## **EXCAVATION OF DISTURBED SOIL WITHIN FAILURE ZONE AND CONSTRUCTION OF SHEAR KEY**

### **Non-Standard Special Provision for Construction of Shear Key**

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#### **Scope of Work**

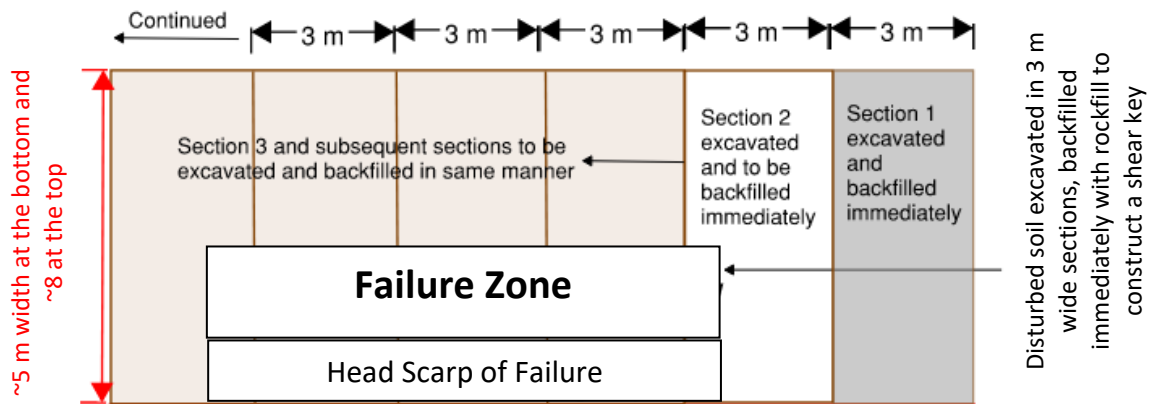
Excavation of top layers of loose fill and undelaying disturbed clay within the failure zone at Highway 658 between Sta.21+025 and Sta. 21+060, and backfill of excavation with rockfill to construct a shear key.

#### **Construction**

For excavation of top layers of loose fill and undelaying disturbed clay within the failure zone at Highway 658 between Sta.21+025 and Sta. 21+060 and construction of a shear key, the following procedures should be applied:

1. Excavation shall be done in sections parallel to the existing highway not longer than 3 m as shown on the sketch below.
2. Excavation should begin at the head scarp of the 2022 failure and be tapered down to the bottom of excavation (i.e. up to bedrock). Temporary excavation side slopes through loose fill and disturbed clay shall be in accordance to the latest edition of Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The native clay above the groundwater level would be classified as Type 3 soil, while below the groundwater it would be classified as Type 4 soil.
3. Excavation is approximately 3 m deep and 5 m wide at the bottom.
4. Excavation must be backfilled with rockfill immediately after excavation. Do not keep the excavation opened under any circumstances.
5. If the groundwater level at the site is high, it is recommended that the layer of rockfill shall be deposited at once approximately 0.5 m above the groundwater level and then the shear key using rockfill shall be constructed in accordance with OPSS.PROV 206 and/or OPSS.PROV 902. For rock-filled structure the layers shall not exceed 1.5 m thickness prior to construction. Material in each layer shall be fully compacted prior to the succeeding layer is placed. Each rockfill layer shall be compacted with a tractor bulldozer with a minimum number of complete passes of 6 and the maximum passes of 8. A complete pass shall be defined as 100% coverage of layer surface.
6. Where practical the shear key shall be periodically outleted toward the bottom of the slope or to the river to avoid accumulation of water during heavy rainfall.
7. The granular fill (Granular A or Granular B Type II as per OPSS.PROV 1010) shall be placed on the top of shear key flattening the embankment slope to minimum 3H:1V. A suitable geotextile (according to OPSS.PROV 1860) shall be placed at any contact between granular fill and rockfill. Surface of rockfill has to be chinked. Backfill shall be placed according to OPSS.PROV 206 and compacted according to OPSS.PROV 501.
8. The final lift of embankment fill prior to placing pavement sub-base shall be compacted to 100 % SPMDD. The Granular A base and Granular B sub-base courses (for pavement) shall be compacted to 100% of the material's SPMDD. Before placing any granular fill over the rockfill, proper chinking shall be applied. Alternatively, a suitably robust geotextile can be placed for separation purposes.

9. The final embankment side slopes shall be protected against erosion by surface water runoff as soon as practical after completion of slope grading using a combination of materials in accordance with OPSS.PROV 803 and/or OPSS.PROV 804.
10. The groundwater conditions shall be controlled by maintaining/addressing suitable drains and controlled outlets.
11. During construction geotechnical monitoring by competent personnel shall be provided.



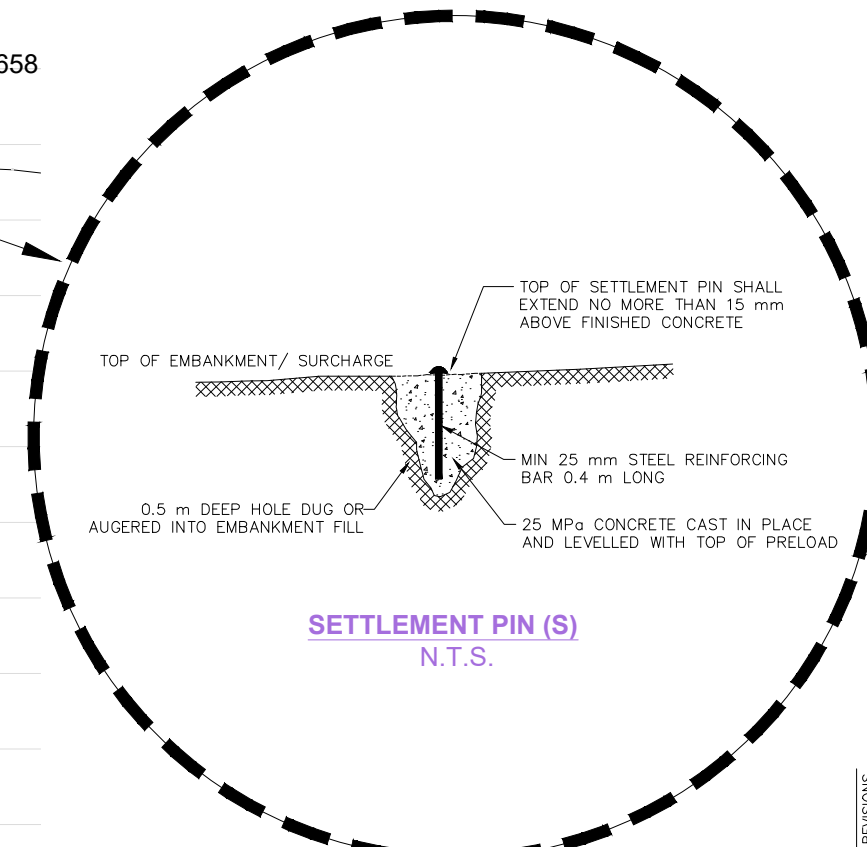
## Appendix K – Settlement Monitoring Plan

SHEET

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LEGEND

- ## PLAN



REVISIONS				SUBMISSION FOR MTO REVIEW			
	NO	DATE	BY	DESCRIPTION			
	PROJECT No. ADM-21019842-E0			GEOCRE'S No.			
	SUBM'D SH	CHKD. SM	DATE	NOV. 25, 2022		SITE	
	DRAWN SH	CHKD. TC	APPRD	SG	DWG	01	

Appendix L –  
Schematic Drawing for Rigid EPS Embankment

