

FINAL REPORT

**Foundation Investigation and Design Report
Englehart Project
HWY 11, Township of Pacaud
Culvert # 60 Extension Replacement and Slope Failure
Station 11+684
Englehart, Ontario
WO 2009-11030
MTO GEOCRES No. 31M-81**

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Part 1 Foundation Investigation

1.1 Introduction

This report presents the results of a geotechnical investigation carried out by Trow Associates Inc. (Trow) at the outlet extension of Culvert # 60, located on the east side of Highway 11 at Station 11+684 within Township of Pacaud. Trow was retained by D. F. Elliot on behalf of MTO to undertake this assignment. Terms of Reference and scope of work are as outlined in the Trow proposal dated June 2, 2009, pertaining to this project. Culvert # 60 consists of a 1200 diameter by 76.2 m long new liner pipe placed in a 1670 mm wide by 1830 mm high old concrete box and two new extensions, a 1200 mm diameter by 30 m long Corrugated Steel Pipe (CSP) extension at the inlet (west) side and a 1200 mm diameter by 13 m long CSP extension at the outlet (east) side. The new liner pipe and extensions were installed in the fall of 2008. However, a failure occurred in November, 2008 on the east (outlet) side which caused a failure of the end section of the CSP extension. It is Trow's understanding that the north side slope, the slope perpendicular to the culvert extension, had failed and resulted in and damage to the pipe. The end section of the failed CSP culvert was observed to have an elliptical shape with the vertical dimension being greater. The new CSP at the outlet side was installed in two 6 to 7 m sections beyond the termination of the concrete box. Subsequent examination indicated that the full section of the CSP extension (13m) was damaged and, therefore, must be replaced. Photographs of the site and damaged pipe are included in Appendix A.

The site specific geotechnical investigation consisted of test borings, borehole logging, and field and laboratory testing. Three boreholes were strategically located aiming to identify the location of its surface of rupture and extent of displaced materials (i.e. zones of depletion and accumulation). The first boring was placed in the area above the slope failure, but beyond its perimeter, to provide comparative data on the stable and unstable portions of the slope. The second boring was located near centre of the main body of the slope failure, while the third borehole was drilled in the area below the toe of the slope failure. These two boreholes were located in the area of the landslide main body and its toe to explore for the displaced material and underlying stable material.

The purposes of this subsurface investigation was to obtain sufficient geotechnical data to analyze the likely reasons for failure, including the impact of construction approaches, and to assess current geotechnical conditions and their influence on repair proposal. The results of the site specific geotechnical investigation are presented in Part 1 Foundation Investigation of this report. Part 2 Design Report includes a background of the site, back-analyses of failure and guidance regarding stability and any mitigation or control measures that would be required during repair/replacement of section(s) of the CSP extension.

1.2 Site Description and Geological Setting

1.2.1 Site Description

Culvert # 60 is located at Station 11+684 on Highway 11, approximately 17 km north of the Town of Englehart, and 4.3 km south of the junction of Highways 11 and 112, in the District of Timiskaming. The site plan is shown on Drawing No. 1 in Appendix B.

Culvert # 60 crosses the highway embankment and conveys collected surface water from the valley at the west side of the embankment. The culvert is skewed to the highway embankment, having a SW-NE direction. As mentioned above, the existing culvert consists of a 1200 diameter by 76.2 m long new liner pipe placed in a concrete box 1670 mm wide by 1830 mm high, and its obvert is at a depth of approximately 8 m below the profile grade at the centerline of Highway 11. The liner pipe culvert is extended at both, inlet and outlet, sides. The extension at the inlet (west) side is a 1220 mm diameter by 30 m long Corrugated Steel Pipe (CSP), while a 1200 mm diameter by 13 m long CSP extension is placed at the outlet (east) side. The elevations of the top of the culvert at the inlet and outlet ends are about 243.65 m and 239.28 m, respectively.

The failed section of CSP culvert is located at the outlet (east) side of Culvert # 60. The inlet of the culvert is in place and functioning properly, including manholes and catch basins. The side slopes at the inlet area are similar to, or steeper, than the east outlet side.

During the field investigation it was observed that the failed CSP culvert area had been backfilled/graded with final slopes from the highway embankment and with gentler north and south slopes to the outlet ravine. Some rock fill was apparent on the slopes, and in the outlet areas, as can be seen on the photographs in Appendix A. Sections of the slope had been filled/excavated to current conditions. Within the right of way, the site was clear of bushes and trees.

The drainage in the area generally consists of roadside ditches which drain into nearby streams. The drainage from the roadside ditch located south of the outlet CSP extension is conveyed from the ditch to the south down to the area of the culvert by a gravel/cobble lined ditch, as shown on Photographs 3 and 5 in Appendix A. A small gully partway up the south slope also drained into the area, but it was not connected to the gravel-lined downslope ditch. The roadside ditch to the north drained, unchannelized, into and through the north slope material, as shown on Photographs 4 and 6 in Appendix A.

1.2.2 Geological Setting

According to Ontario Geological Survey (OGS) Map 5021, as well as Ontario Geological Survey Map 2555 (Quaternary Geology, New Liskeard) and the Ministry of Northern Development and Mines Map 2543 (Bedrock Geology of Ontario, East-Central Sheet), the site is located in the Neoarchean Group comprised of mainly igneous origin rock.

The Englehart area is situated in a physiographic division of the Canadian Shield known as the Cobalt Plain. The overburden soils have been mapped as glaciolacustrine deposits consisting of upper massive to lower laminated, rhythmically bedded (also referred to as varved) silts and clays with occasional rock knobs.

1.3 Investigation Procedures

1.3.1 Field Program

The fieldwork for this investigation was performed between June 15, 2009 and June 22, 2009. The fieldwork consisted of drilling three (3) sampled boreholes (BH60-1, BH60-2 and BH60-3) and installing of two (2) piezometers in BH60-1 and BH60-3. The boreholes were strategically located to permit assessment of slopes. The 20.4 m deep Borehole BH60-2 was drilled at the east shoulder of the existing highway embankment in the area beyond the estimated slope failure perimeter. Borehole BH60-1 was drilled while Borehole BH60-3 was drilled near the outlet of the culvert. Boreholes BH60-1 and BH60-3 were 10.2 m and 6.7 m deep, respectively.

Boreholes BH60-1 and BH60-2 were advanced using a Bombardier mounted CME-55 drill rig, equipped with a hollow stem auger (4-1/4" HAS) and standard soil sampling equipment. They were drilled by Marathon Drilling. Due to uncertainty in slope stability and difficulties to mobilize the drill rig down the slope, Borehole BH60-3 drilled near the culvert outlet was advanced by hand drilling/sampling equipment operated by a specialist drilling contractor, Sonic Soil Sampling.

From the drilling program, soil samples were obtained using a 51 mm OD split-spoon sampler in conjunction with Standard Penetration Tests at 0.75 m intervals within the estimated zone of surface rapture (estimated Elevations between 241 m and 235 m) and 1.5 m intervals in other zones. Sampling and testing procedures were in general accordance with ASTM D1586. This consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm diameter OD split-spoon sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as SPT 'N' value of the soil and this gives an indication of the consistency or the relative density of the soil deposit. However, the manual hammer used for hand testing was 31.7 kg, one half of the standard hammer weight. As a result, the corresponding blow counts were factored by 0.5.

In addition, selected, undisturbed, 50 mm diameter "Shelby" tube samples were obtained in cohesive deposits. Field vane testing was also completed in the boreholes throughout the cohesive soils to measure the *in-situ* undrained shear strength of the soils. The field vane testing was conducted in accordance with ASTM D2573-08.

All fieldwork was supervised by a member of Trow's engineering staff who directed the drilling and sampling operations, logged the factual borehole data in accordance with the MTO Soils Classification System for foundation report, and retrieved soil samples for subsequent laboratory testing and identification. All of the recovered soil samples were placed in moisture-proof bags and returned to Trow's Brampton laboratory for additional visual, textual and olfactory examination and for selected laboratory testing.

Following completion of the boreholes, water level measurements were obtained from the boreholes in accordance with Ministry of Transportation guidelines. Standpipe piezometers were installed in Boreholes BH60-1 and BH60-3 to permit long term monitoring of groundwater levels at the site. Borehole BH60-2 was backfilled with auger cuttings and sealed with bentonite pellets.

Details of the soil strata encountered in the boreholes are included in attached logs in Appendix C, and plotted on the profile included in Drawing No. 1 in Appendix B. The locations of the boreholes were determined in the field using Garmin Global Positioning Systems (GPS) equipment. The final geodetic locations and elevations shown on Drawing No. 1, Appendix B, are established based on the site survey map provided by D. F. Elliott Consulting Engineers Ltd.

1.3.2 Laboratory Testing

All samples returned to the laboratory were subjected to detailed visual examination and classification. The laboratory testing program included moisture content determination of all samples (LS-701) and routine classification testing of approximately 25% of the selected soil samples. The routine testing included grain size distribution (LS-702) and Atterberg limits (LS-703/704). In addition, two undisturbed, "Shelby" tube cohesive samples were subjected to laboratory unconfined compression tests (ASTM D2166).

The laboratory test results are provided on the attached borehole logs in Appendix C. The results of the grain size analyses and Atterberg limits tests are presented in Appendix D.

1.4 Subsurface Conditions

The subsurface conditions encountered during the field investigation are summarized on the attached borehole logs in Appendix C. The "Explanation of Terms Used in Report" preceding the borehole logs (Appendix C) forms an integral part of and should be read in conjunction with this report.

A borehole location plan and a strata plot of the soils encountered in the boreholes are provided on Drawing No.1 in Appendix B. In general, the stratigraphy along the centerline of the slope failure between Boreholes BH60-1 and BH60-2 consisted of sand and gravel fill, silty clay fill, silty clay, clayey silt and silty sand.

A summary of the soil and groundwater conditions encountered in the boreholes is provided below.

1.4.1 Sand and Gravel Fill

In BH60-1 and BH60-2 sand and gravel fill was encountered at the ground surface. The thickness of the sand fill was 1.1 m in BH60-2 and 2.3 m in BH60-1. The composition of this layer was sand and gravel matrix with a 0.12 m thick layer of topsoil over (BH60-2). This layer was brown in colour, very moist to damp. Based on the “N” value from the Standard Penetration Tests, the compactness of the sand and gravel fill was assessed as loose to compact.

Laboratory testing performed on selected samples consisted of moisture content tests and grain size distribution tests. The test results are as follows:

Moisture Content:

- 4.5% to 12.6%

Grain Size Distribution:

- 20% gravel;
- 66% sand;
- 14% silt and clay

The results of the moisture content tests are provided on the Record of Borehole sheets in Appendix C. The results of the grain size distribution test on sand and gravel fill are provided on Figure 1 in Appendix D.

1.4.2 Silty Clay and Clayey Silt Fill

Beneath sand and gravel fill materials, there is a silty clay and clayey silt fill as indicated on Boreholes BH60-2 and BH60-1. The thickness of this fill was 6.5 m at BH60-2. At that location, the layer extended to depth up to 7.6 m, which corresponds to approximate Elevation of 238.4 m. A similar layer was encountered in BH60-1 below the gravel and sand fill at elevation of 240.0 m, but it was found to be only 0.8 m thick. The silty clay fill was encountered in BH60-3 at the ground surface. The fill predominately consists of silt and clay with a trace of gravel. The surficial material in BH60-3 contains decayed wood fragments and rootlets as well. The silty clay and clayey silt fill is grey and very moist to wet. Measured SPT “N” values varied from 1 to 5 indicating very soft to firm consistency. Field vane measurements indicated that the undrained shear strength of this silty clay fill was about 20 kPa at the elevation of around 242 m, and then it increased with depth to a maximum value of about 50 kPa at Elevation 239 m. Sensitivity, the measure of peak shear strength and remolded shear strength, ranged from 1.5 to 3, indicating the silty clay fill is low to medium sensitive.

Laboratory testing performed on selected samples consisted of moisture content tests. The test results are as follows:

Moisture Content:

- 24.8% to 32.9%

The results of the moisture content are provided on the Record of Borehole sheets in Appendix C.

1.4.3 Silty Clay

Beneath the fill materials, a stratum of silty clay was encountered as the principal native soil unit in all boreholes (BH60-1, BH60-2 and BH60-3). The silty clay was encountered at a depth of approximately 7.6 m below existing grade (approximate Elevation 238.4 m) in BH60-2. In Boreholes BH60-1 and BH60-3 the silty clay was encountered at a depth of approximately 3.1 m (approximate Elevation 239.2 m) and 1.8 m (approximate Elevation 237.0 m), respectively. In BH60-2 the layer was about 9.4 m thick extending to a depth of approximately 17 m below ground (approximate Elevation 229 m). Boreholes BH60-1 and BH60-3 were terminated in the silty clay stratum at depths of approximately 10.2 m (approximate Elevation 232.1 m) and 6.7 m (approximate Elevation 232.1 m), respectively.

Generally, the silty clay was thinly laminated with clayey silt (varved). The individual layers or laminations varied in thickness from a few millimeters to a few centimeters, but in general were about one centimeter thick. The portions of silty clay and clayey silt varied also, but in general the clay portion dominated. The silty clay is grey in colour and saturated.

SPT “N” values within the silty clay encountered in BH60-1 and BH60-2 ranged from 0 to 9 indicating very soft to stiff material. SPT “N” values measured in Borehole BH60-3 were somewhat higher (up to 13), probably due to use of hand equipment for drilling and testing. The standard penetration resistance values should not be considered an accurate assessment of the consistency of the soil given the varying composition and method of drilling.

Field vane tests and laboratory unconfined compression tests were performed to examine undrained shear strengths of silty clay. All results of the *in-situ* field vane tests measured in all boreholes and unconfined compression tests are plotted on the records of boreholes, Appendix C. In addition, the summary of the results is shown on Figure 5, attached in Appendix D. As it can be seen, the measured values of the undrained shear strength ranged from 27 kPa to 55 kPa. Even though the data scattered, it suggests that the undrained shear strength profile of native silty clay changes with depth. Undrained shear strengths of around 30 kPa were measured at Elevations between 238 m and 235 m. Subsequently the strength increased to about 45 kPa at Elevation 230 m. The undrained shear strengths of the two samples measured in the unconfined compression tests were

34.6 kPa (from BH60-2 at Elevation 236.5 m) and 30 kPa (from BH60-3 at Elevation 233.2 m). Sensitivity ranged from 1.8 to 2.75, indicating the silty clay is low to medium sensitive.

Laboratory testing performed on selected samples consisted of moisture content, grain size distribution, Atterberg Limits and unconfined compression tests. The test results are as follows:

Moisture Content:

- 25.4% to 61.3%

Grain Size Distribution:

- 0% gravel;
- 1% to 2% sand;
- 37% to 42% silt; and
- 57% to 62% clay.

Atterberg Limit:

- Liquid Limits: 34% to 53%
- Plastic Limits: 17% to 22%.

Unconfined Compression:

- Undrained Shear Strength 30 kPa and 34.6 kPa

The results of the moisture content, grain size distribution and Atterberg Limits and unconfined compression tests are provided on the Record of Borehole sheets in Appendix C. The results of the grain size distribution tests on the silty clay are provided on Figure 2 in Appendix D. The results of the Atterberg Limits tests are provided on Figure 4 in Appendix D. The results of two unconfined compression tests are shown on Figure 7, Appendix D.

1.4.4 Clayey Silt

Clayey silt was encountered underlying the silty clay in Borehole BH60-2. The clayey silt was encountered at a depth of approximately 17 m below ground, corresponding to Elevation of approximately 229 m. The layer was 1.8 m thick and extended to the depth of 18.6 m (approximate Elevation 227.4 m).

The clayey silt is grey and generally wet. Based on the “N” values obtained from the SPT, the consistency of the clayey silt was considered soft.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture Content:

- 26%

Grain Size Distribution:

- 0% gravel;
- 9% sand;
- 66% silt; and
- 25% clay.

The result of the moisture content test are provided on the Record of Borehole sheet in Appendix C. The results of the grain size distribution tests on the clayey silt are provided on Figure 3 in Appendix D.

1.4.5 Silty Sand

Silty sand was encountered underlying the clayey silt in Borehole BH60-2. The silty sand was encountered at a depth of approximately 18.6 m below ground, corresponding to elevation of approximately 227.4 m. Borehole BH60-2 was terminated in this layer at the depth of 20.4 m (approximate Elevation 225.6 m).

The silty sand generally contained silt and sand, trace gravel, and was generally wet. Based on the “N” values obtained from the SPT, the compactness of the silty sand was very loose to loose.

Laboratory testing performed on selected samples consisted of moisture content tests. The test results are as follows:

Moisture Content:

- 19.7% to 30%

The results of the moisture content tests are provided on the Record of Borehole sheets in Appendix C.

1.4.6 Groundwater

Information regarding the groundwater levels at the site was obtained by measuring the water levels in the open boreholes after completion of drilling and in the piezometers installed in Boreholes BH60-1 and BH60-3. The measured groundwater levels are shown on the borehole logs. The groundwater levels encountered in the boreholes are also shown in table below.

The difference in groundwater level between boreholes could be due to disturbance in the holes at the time of drilling and that the boreholes had not stabilized prior to backfilling.

Seasonal variations in the water table should be anticipated, with higher levels occurring during wetter periods of the year (such as spring thaw and late fall) and lower levels during drier periods.

Table 1.1. Groundwater levels recorded at Culvert # 60

Borehole Number [Top Elevation (m)]	Date of Drilling	Groundwater Level Depth Below Existing Grade (m) [Elevation (m)]			
		After Completion	06/24/2009	06/25/2009	06/27/2009
BH60-1* [242.3]	06/16/2009	dry	1.8 [240.5]	1.8 [240.5]	1.8 [240.5]
BH60-2 [246.0]	06/15/2009	7.6 [238.4]	-	-	-
BH60-3* [238.8]	06/22/2009	4.5 [234.3]	2.4 [236.4]	1.5 [237.3]	1.2 [237.6]

* - piezometer

1.5 Closure

A soil investigation is a limited sampling of a site. The information is collected at specific borehole locations and can be extrapolated to an approximate limited area around the borehole. The extent of the limited area depends on the variability of the soil and groundwater conditions as influenced by geological processes and the construction activities. Should any 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 allow reassessment of our recommendations. It may then be necessary to carry out additional field work and analyses.

Part 2 Engineering Discussion and Recommendations

2.1 Introduction

The following subsections address assessment of the slope failure that occurred during replacement of the outlet extension of Culvert # 60, located on the east side of Highway 11 at Station 11+684 within Township of Pacaud, and construction strategies for remediation. Included is a background of the site, slope stability assessment and discussion related to the likely cause of the failure. Guidance regarding stability and mitigation or control measures that would be required during repair/replacement of section(s) of the CSP extension are also provided in this report. In undertaking the forensic review and preparing for assessments, a wide variety of excavation scenarios, geometry, surcharge and combination thereof have been evaluated. Photographs and related records from this initial construction period were used to and in the assessment. The slope stability analyses were subjected to sensitivity analyses that permitted assessment of the implication of strength parameters differences from those measured and/or influenced by disturbance and remoulding.

2.2 Background

Culvert # 60 is located at Station 11+684 on Highway 11, approximately 17 km north of the Town of Englehart, and 4.3 km south of the junction of Highways 11 and 112, in the District of Timiskaming. The culvert is skewed to the highway embankment having direction from the south-west toward the north-east. The site plan and soil strata profile along the estimated centerline of the slope failure are as shown on Drawing No. 1 in Appendix B.

Culvert # 60 consists of a 1200 diameter by 76.2 m long liner pipe placed in a concrete box 1670 mm wide by 1830 mm high, and two extensions, a 1200 mm diameter by 30 m long Corrugated Steel Pipe (CSP) extension at the inlet (west) side and a 1200 mm diameter by 13 m long CSP extension at the outlet (east) side. Elevations of inlet and outlet inverts are approximately 242.5 m and 237.85 m, respectively. The new CSP extension at the outlet side was installed in two 6 to 7 m sections beyond the termination of the concrete box. The new liner pipe and extensions were installed in the fall of 2008.

It is understood that excavation for removal of the existing concrete extension and installation of the new CSP extension at the outlet side (the east side of Highway 11) were carried out on November 4th and November 5th, 2008 by open cut method without a protection system. It is Trow's understanding that there is some evidence that the original north side slope (the slope perpendicular to the culvert alignment) was loaded with heavy equipment (excavators), access road/construction pad materials and excavated

earth materials during the excavation. The actual placement of the new 1200 diameter CSP extension into the excavated trench occurred on November 11th. Between November 5th and November 11th the excavation was left open without any support. In the morning of November 12th, the concrete appurtenance for connection of the existing concrete box and new CSP extension was poured, and the final lift of granulars for cover material on the CSP extension was placed. Subsequently, the contractor proceeded with excavation for new channel alignment for the off-take ditch. It is reported that the first sign of the failure was noticed around the noon of November 12th. A crack in the native material parallel with the CSP installation and the new ditch direction was reported and the north side slope started to slide around the mid-point of the new installed CSP extension causing its squeezing laterally. The end section of the failed CSP culvert was observed to have an elliptical shape with the vertical dimension being greater. This evidence suggested that the movement of the failed earth was perpendicular to the CSP installation. Photographs of the site and damaged pipe are included in Appendix A. Subsequent examination indicated that the full section of the extension (13m) was damaged and must be replaced.

Trow Associates visited the site in May 2009 following the failure and recommended that an investigative program of 3 to 4 strategically located borings be carried out to assess current geotechnical conditions and the influence on repair proposals. The specific site geotechnical investigation was performed in late June 2009, and the results are presented in this report.

During the site visit and the field investigation it was observed that the failed CSP culvert area had been backfilled/graded with final slopes from the highway embankment and with gentler north and south slopes to the outlet ravine. Some rock fill was apparent on the slope, and in the outlet area, as can be seen on the photographs in Appendix A. Sections of the slope had been filled/excavated to current conditions. A gravel/cobble lined ditch was placed south of the outlet CSP extension to convey drainage from the roadside ditch to the south down to the area of the culvert, as shown on Photographs 3 and 5 in Appendix A. A small gully partway up the south slope also drained into the area, but it was not connected to the gravel-lined downslope ditch. The roadside ditch to the north drained, unchannelized, into and through the north slope material, as shown on Photographs 4 and 6 in Appendix A.

It was noted that at the time of the geotechnical investigation the inlet side of the culvert was in place and functioning properly. The slopes around this side of the culvert (west side of Highway 11) were similar to or steeper than the east (outlet) side. It was understood that the inlet side installations were successfully undertaken with the use of trench boxes.

2.3 Site Stratigraphy

The stratigraphy of the failed area generally consisted of granular and silty clay highway embankment fills overlying a stratum of silty clay followed by layers of clayey silt and silty sand. Granular fill, approximately 1 to 2 m thick, was encountered at the ground surface of the upper part of the failed slope. The thickness of the underlying, cohesive silty clay fill ranged from 0.8 m at the middle of the failed slope to 6.4 m at the top of the slope. The thickness of the native silty clay in the depth explored was about 9 m, extending from approximate Elevation 238 m to 229 m. The undrained shear strength of the silty clay fill as measured by *in-situ* field vane testing, ranged from 16 kPa to 47 kPa, while the undrained shear strength of the silty clay was somewhat higher ranging from 27 kPa to 55 kPa. The location of boreholes and profile of soil strata at the slope failure area are shown on Drawing No. 1 in Appendix B, with the summary borehole logs shown in Appendix C. All laboratory data is attached in Appendix D.

The undrained shear strength profile of two cohesive soil layers at the slope failure area, developed from the vane shear strengths in all boreholes and laboratory unconfined compression tests, suggests that the strength of these layers changes with depth (Figure 5, Appendix D). In general assuming a linear relationship, the strength of the silty clay fill layer increases from 20 kPa to 30 kPa with depth, while the strength of the silty clay increases from about 30 kPa to 45 kPa in the zone examined. No obvious reduction in the strength within the depth of boring was noticed, so the location of the surface of rupture of the slope failure was not conclusive or not intercepted.

The moisture content vs. elevation profile is included on Figure 6 in Appendix D. This figure shows all measured moisture content data from all boreholes. As shown on Figure 6, there is a significant increase in the moisture content of the clayey materials near Elevation 233 m, and then a subsequent decrease with depth. The maximum moisture content value of 61% was measured within BH60-1 at Elevation 233 m.

2.4 Slope Stability Assessments

Based on the results of the geotechnical investigation and specified site characteristics including topography and external loads, slope stability assessment of failure and slope stability assessment of alternative remedial works have been performed. A series of slope stability analyses using a wide variety of excavation scenarios including geometry, surcharge, soil properties and combination thereof have been performed to further analyze the reasons of failure. In addition, alternative remediation scenarios have been evaluated to suggest optimum mitigation actions, if necessary. The slope stability analyses were performed using the conventional limit equilibrium method proposed by Morgenstern-Price. The SLOPE/W computer program developed by GeoSlope International was employed for computation. Some non circular failure modes were also examined.

Both slope stability assessments were performed on the cross-section perpendicular to the culvert alignment at the outlet side, while the slope stability assessment for remedial works was also evaluated on the cross-section perpendicular to Highway 11. The side cross-section profile was developed close to the existing connection between the liner pipe culvert and the CSP extension. The cross-section profiles were created based on a topographic map with contour lines provided by D.F. Elliott Consulting Engineering Ltd. It is Trow's understanding that this plan was created after the slope failure. In absence of the geometry of the original slope before failure, the slope stability assessment of failure in this report was performed assuming that the original slope geometry was similar to the current slope geometry and modified by excavations or local filling, needed to execute the replacement.

The stratigraphy at the site was developed based on the results of the geotechnical investigation presented in Part 1 Foundation Investigation. Total stress analyses using undrained shear strength values for cohesive materials, as would apply to rapid construction (short term stability) of a trench, was performed. Sensitivity analyses were performed using the interpreted strength profile (Figure 5, Appendix D), the assumption supported by results of back analyses as outline below, and upper values of the remoulded undrained shear strength of the silty clay. In these analyses, the undrained shear strength, C_u , of the silty clay fill varied between 20 kPa and 30 kPa, while the silty clay varied from 20 kPa to 45 kPa. The groundwater level was defined based on its measurements during the geotechnical investigation (see Section 1.4.4 in Part 1 Foundation Investigation).

The SLOPE/W graphical printouts, for various analyses performed are included in Appendix F.

2.4.1 Slope Stability Assessment of Failure

As noted above, a series of back analyses of failure was performed by analyzing the slope stability of the original slope for several different cases and scenarios: (i) before trench excavation and failure, (ii) after trench excavation without surcharge on the slope and (iii) after trench excavation with surcharge on the slope. Sensitivity analyses for soil properties are also included.

- (i) Figures 1(a) through 2 in Appendix F show the results of slope stability analyses performed on the original side slope before the excavation for culvert extension replacement and failure. The results are also summarized in Table 2.1.

The slope stability analyses indicate that the undrained shear strength of the silty clay along a critical slip surface should be more than 25 kPa to assure a factor of safety above 1.0. The results of these back analyses supported the values of soil undrained shear strength derived from the interpreted strength profile. The estimated values of 20 kPa for the undrained shear strength of the silty clay fill and 30 kPa for the undrained shear strength of the silty clay in the failure area appear realistic.

Table 2.1 Summary of results of stability analyses for the original slope before trench excavation and failure

Case Analyzed	Method of Analyses	Cu of Silty Clay Fill (kPa)	Cu of Silty Clay (kPa)	Min. Factor of Safety	Figure in Appendix F
Before Trench Excavation for Culvert Extension Replacement	Circular Slip Surface	20	20	0.9	Figure 1(a)
		20	25	1.07	Figure 1(b)
		20	30	1.23	Figure 1(c)
	Non-circular Slip Surface	20	30	1.43	Figure 2

- (ii) Figures 4 through 5(b) in Appendix F show the results of slope stability analyses performed on the original side slope after the unsupported trench for the segment of the old concrete box culvert and installation of the new CSP extension was excavated. It was assumed that the trench was approximately 2.2 m wide in the base with side slopes of 2H:1V or 1H:1V, and 2.5 m deep (approximate invert Elevation 237.85 m). In this scenario no surcharge was placed on the slope. The results are also summarized in Table 2.2.

Table 2.2 Summary of results of stability analyses for the original slope after the unsupported trench was excavated (no surcharge on the slope)

Case Analyzed	Side Slopes	Cu of Silty Clay Fill (kPa)	Cu of Silty Clay (kPa)	Min. Factor of Safety	Figure in Appendix F
After Trench Excavation for Culvert Extension Replacement	2H:1V	20	30	1.0	Figure 3
	1H:1V	20	25	0.89	Figure 4(a)
		20	30	1.03	Figure 4(b)

The results of the analyses suggest that the side slopes of the excavated trench were probably between 1H:1V and 2H:1V. Any slope flatter than this yields the factor of safety less than 1.0, since the excavation in the toe of the slope reduces resisting forces along a potential slip surface for global stability.

- (iii) To assess the influence of any surcharge on the slope stability during the construction, the slope stability analyses for the original slope were performed assuming that an excavator pad and/or excavator were placed on the slope. It was assumed that a midsize excavator such as the John Deere Excavator 120C or 330C LC with a total ground pressure around 50 kPa was used for excavation. The results of those

analyses are shown on Figures 5 through 7(c) in Appendix F and Table 2.3 below.
 Figures 6(b) and 7(c) simulate the condition after culvert backfill.

Table 2.3 Summary of results of stability analyses for the original slope after the unsupported trench with slopes of 1H:1V was excavated and surcharge was placed on the slope

Case Analyzed	Type of Surcharge	Cu of Silty Clay Fill (kPa)	Cu of Silty Clay (kPa)	Min. Factor of Safety (Slope Failure Mode)	Figure in Appendix F
After Trench Excavation for Culvert Extension Replacement	None	20	30	1.03 (Global slope failure)	Figure 4(b)
	Excavator Pad	20	30	1.04 (Global slope failure)	Figure 5
	Excavator Pad and Excavator (50 kPa)	20	30	0.94 (Trench slope failure)	Figure 6(a)
	Excavator Pad, Excavator (50 kPa) and Culvert Backfill	20	30	0.89 (Trench slope failure)	Figure 6(b)
	Excavator (50 kPa)	20	30	0.9 (Trench slope failure)	Figure 7(a)
		20	30	1.06 (Global slope failure)	Figure 7(b)
	Excavator (50 kPa) and Culvert Backfill	20	30	0.86 (Trench slope failure)	Figure 7(c)

According to the results of these slope stability analyses, it appears that the surcharge on the side slope does not contribute to global slope instability. However, the surcharge on the slope reduces stability of the trench slope.

In summary, the slope stability analyses discussed above:

- (a) support the soil strength profile interpreted from *in-situ* and laboratory strength measurements suggesting that the estimated values of 20 kPa for the undrained shear strength of the silty clay fill and 30 kPa for the undrained shear strength of the silty clay in the failure area appear realistic for assessment purpose
- (b) suggest that the unsupported trench excavation could cause local and global instability of the side slope
- (c) indicate that the surcharge at the slope could contribute to local instability of the trench slope
- (d) suggest that the most probable scenario of failure is soil movement in a toe failure caused by slope loading with external surcharge and geometry of the open unsupported trench. This is consistent with the observation of the pipe and anticipated activity during excavation.

2.4.2 Slope Stability Assessment of Alternative Remedial Works

Slope stability analyses were performed on the current slope to assess its stability and suggest optimal remedial works. For these analyses, first the slope stability of an unsupported trench excavation was assessed. Figure 8 in Appendix F shows the results of the slope stability analyses after trench excavation for the failed culvert extension replacement without a support system having the 2.5H:1V slopes. Previously presented Figures 3 and 4(b) show the results of the stability analyses if the unsupported trench slope was 2H:1V and 1H:1V. As can be seen from these analyses the factors of safety of these slopes for the unsupported trench are less than 1.0 applying the undrained shear strength of 20 kPa for the silty clay fill and 30 kPa for the silty clay. Therefore, it appears again that open trench excavation without any support is not a safe method for excavation of the failed CSP extension and installation of the new pipe.

In addition, the slope stability assessment assuming that a trench box would be used for excavation protection in the area of the failed extension and installation of the new pipe was performed

For this slope stability assessment, the following cases and scenarios were analyzed: (i) stability of the side slope (perpendicular to the existing culvert) after trench excavation using trench boxes, (ii) stability of the side slope after trench excavation using trench boxes and flattening of the current slope, and (iii) stability of the highway embankment west of the inlet before and after excavation of the trench.

- (i) The results of the slope stability analyses on the current slope assuming that repair work for replacement of the failed CSP extension would be carried out with trench

boxes, are shown on Figures 9(a) through 10, Appendix F. Table 2.4 summarizes these results as well.

Table 2.4 Summary of results of stability analyses for the current slope after the supported trench was excavated

Case Analyzed	Method of Analyses	Cu of Silty Clay Fill (kPa)	Cu of Silty Clay (kPa)	Min. Factor of Safety	Figure in Appendix F
Supported Trench Excavation for Culvert Extension Replacement	Circular Slip Surface	20	20	0.77	Figure 9(a)
		20	25	0.92	Figure 9(b)
		20	30	1.06	Figure 9(c)
	Non-circular Slip Surface	20	30	1.28	Figure 10
		20	30	1.28	

As indicated in Table 2.4, the factor of safety for the current slope would be 1.06 if the trench was excavated to replace the failed CSP pipe. The results of these analyses suggest that some additional control measure should be applied to assure safe excavation of the trench.

- (ii) Figures 11(a), (b) and (c) illustrate the results of the stability analyses of the current slope after its flattening to 5H:1V. The results are summarized in Table 2.5.

Table 2.5 Summary of results of stability analyses for the current slope after the supported trench was excavated and the slope is flatten to 5H:1V

Case Analyzed	Remedial Work	Method of Analyses	Cu of Silty Clay Fill (kPa)	Cu of Silty Clay (kPa)	Min. Factor of Safety	Reference Figure in Appendix F
Supported Trench Excavation for Culvert Extension Replacement	Without Slope Flattening	Circular Slip Surface	20	30	1.06	Figure 9(c)
	With Slope Flattening	Circular Slip Surface	20	20	0.97	Figure 11(a)
		Circular Slip Surface	20	25	1.16	Figure 11(b)
		Circular Slip Surface	20	30	1.34	Figure 11(c)
		Non-circular Slip Surface	20	30	1.43	Figure 11(d)

The results of these analyses show that flattening of the current slope to 5H:1V yields an increase in a factor of stability from 1.06 to 1.34 for the critical slip surface.

(iii) In addition, the slope stability analyses were performed for the cross section perpendicular to the highway embankment before and after excavation for CSP culvert extension replacement. Figures 12(a) through 15(b) show the results of the analyses. The results are also summarized in Table 2.6.

Table 2.6 Summary of results of stability analyses for the highway embankment slope

Case Analyzed	Method of Analyses	Cu of Silty Clay Fill (kPa)	Cu of Silty Clay (kPa)	Factor of Safety	Figure in Appendix F
Before Trench Excavation for Culvert Extension Replacement	Circular Slip Surface	20	20	0.72	Figure 12(a)
		20	30	1.01	Figure 12(b)
		20	35	1.07	Figure 12(c)
		30	35	1.22	Figure 12(d)
		30	45	1.51	Figure 12(e)
	Non-circular Slip Surface	20	35	1.16	Figure 13
After Trench Excavation for Culvert Extension Replacement	Circular Slip Surface	20	20	0.65	Figure 14(a)
		20	30	0.92	Figure 14(b)
		20	35	0.96	Figure 14(c)
		30	35	1.11	Figure 14(d)
		30	45	1.38	Figure 14(e)
	Non-circular Slip Surface	20	35	0.93	Figure 15(a)
		30	35	1.4	Figure 15(b)

The results of these analyses show that the undrained shear strengths of the silty clay fill and silty clay in that direction, different than the direction of failure, should be above 30 kPa and 35 kPa, respectively, to obtain the factor of safety above 1.0. This suggests that the undrained shear strengths of the undisturbed silty clay fill and silty clay were minimum 30 kPa and 35 kPa, respectively, which is in good agreement with *in-situ* soil strength measurements obtained at the other adjacent sites.

In summary, the presented results of slope stability analyses on the current slope suggest:

- (a) that an open trench excavation method without a support system is not safe;
- (b) that suitably designed shoring can be used for the works, and
- (c) that excavation for repair work could be performed by flattening the side slopes to 5H:1V; and by using trench boxes in accordance with good practice provided that access and surcharge issues can be suitably managed.

2.5 Conclusions and Recommendations for Remedial Works

2.5.1 General

Based on the results of geotechnical investigation, results of slope stability assessments and site observations, the most probable scenario for the observed slope failure is soil movement caused by slope loading and the geometry of the open unsupported trench adjacent to the installed culvert section as illustrated in Figure 6b. This is consistent with reported sequences and observed deformation of pipe.

2.5.2 Excavation Support Options for Remedial Works

As outlined above, based on the results of slope stability assessment two excavation support options are possible for the repair work:

- (i) *Suitably designed shoring such as, sheet piling braced as and where required, or*
- (ii) *Flattening of the existing slope to 5H:1V and using trench boxes to protect the local excavation for removal and replacement.* The slope stability analyses suggest that the north side slope can be stabilized by its flattening to 5H:1V. Then, for removal of the failed CSP extension and installation of the new extension trench boxes (shields), 7 m or longer, can be used to protect workers. The excavation behind the trench wall should be backfilled after installation of the trench box. If this excavation support option is considered any surcharge placement on the side slope should be avoided.

2.5.3 General Temporary Support Requirements

Since the proposed work for culvert extension replacement is located in the vicinity of the highway embankment, it will be the Contractors responsibility to design a suitable temporary support system for the MTO review prior to installation. The Contractor is to follow OPSS 538 and SP No. 902S01, regarding excavations for structures, and OPSS 539 and SP No. 105S19, regarding temporary protection systems (e.g. braced sheet piles, or some other form of bracing such as a soldier pile and lagging system).

The Contractor shall be responsible for the complete design, construction, monitoring and removal of the installed protection system. The protection system shall be designed to provide protection for excavations as required by the occupational Health and Safety Act, at locations specified in the contract, and at any locations where the stability, safety or function of an existing structure and/or utility may be impaired by construction work. The protection system should be designed for the Performance Level 2. The minimum requirements for monitoring shall include the survey measurements of 6 m apart scaled targets attached to the shoring wall at the elevations specified. If movement approaches the allowable limit of 25 mm, suitable measures should be taken to ensure stability of the protection system and to ensure that the movement does not exceed the performance level specified.

2.5.4 Additional Remedial Measures

In addition, the following remedial measures are also suggested:

- Surface water should be directed away from distressed zone. Final grades along top of slope should be such that water is directed away from slope and taken down as required in controlled ditches or conduits. Re-channelize any affected surface drainage;
- Work should be completed as soon as practical and during dry weather where possible;
- All required health and safety measures should be in place. Excavation must be performed in accordance with the Ontario Occupational Health and Safety Act (OHSA) and its regulations; and
- During construction qualified geotechnical personnel should attend the site to observe conditions and provide geotechnical guidance and reporting.

2.5.5 Possible Sequence of Construction Events

The contractor should devise his own means and methods to execute the works. For guidance, the following approach for the sequence of construction events that will need to be performed to allow for the failed CSP extension removal and new CSP extension installation should be accommodated:

2.5.5.1 (i) Excavation support using shoring:

- Divert surface water of the culvert watershed away from the culvert.
- Place the excavation equipment at the bottom of the slope.
- Install shoring (suitably designed sheet piles or other approved scheme) at both sides of the existing failed CSP extension (~ 13 m long). Include any surcharge in the

assessment and design. Surface removal of rock pieces may be required to accommodate driving (refer to attached photographs in Appendix A).

- Excavate around the existing failed CSP extension. The trench will be approximately 2.5 m deep and slightly more at the connection to the lined box section.
- Remove the damaged sections of the CSP culvert extension (~13 m long).
- Examine the condition of the subgrade bedding material and repair as necessary.
- Install the new CSP culvert extension and re-connect to the existing culvert.
- Backfill the excavation using the recommended procedures. Shape grades to final design levels and reinstate required lined ditches etc.
- Remove any section of temporarily shoring as necessary.

Lateral Earth Pressure

Temporary shoring should be designed to resist lateral earth pressure. The expression for calculating lateral earth pressure is given by:

$$P = K(\gamma h + q)$$

where

P = earth pressure intensity at depth h , kPa

K = earth pressure coefficient

γ = unit weight of retained soil, kN/m³

q = surcharge near wall, kPa

h = depth to point of interest, m

The above expression does not take into account hydrostatic pressure, which must be included for the groundwater level prevailing.

Table 2.7 below lists various earth pressure properties for given materials.

Table 2.7 Material types and earth pressure properties

Material	Friction Angle ϕ' (unfactored)	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (K_p)	Coefficient of Lateral Earth Pressure at Rest (K_o)	Unit Weight γ (kN/m ³)
Granular A	35°	0.27	3.7	0.43	22
Granular B	30°	0.31	3.3	0.5	21
Granular B	35°	0.27	3.7	0.43	21
Rock Fill	42°	0.2	5	0.33	21
Sand Fill	30°	0.33	3.0	0.5	21
Silty Clay Fill	16°	0.57	1.77	0.72	19
Silty Clay	16°	0.57	1.77	0.72	19

Note: Values given for horizontal earth pressures are for horizontal backfill. For sloping backfill, the design requirements outline in Sec C6.91(c) of the Canadian Highway Bridge Design Code should be used.

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation. Therefore, unless the structural element can tolerate these deflections, the at-rest earth pressure should be used in design.

The effect of compaction surcharge should be taken into account in the calculations of active and at rest earth pressures. The lateral pressure due to compaction should be taken as at least 12 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at rest) pressure is equal to 12 kPa. This pressure distribution should be added to the calculated active (or at rest) pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to walls to prevent overstressing.

2.5.5.2 (ii) Excavation approach using side slope flattening and trench box protection

- Divert surface water of the culvert watershed away from the culvert.
- Flatten the north side slope to 5H:1V. Do not surcharge slope in the areas to be excavated.
- Place the excavation equipment at the bottom of the slope.

- Excavate around the existing failed CSP extension using the 1H:1V side slope trench (section by section). The trench is expected to be approximately 2.5 m deep.
- Place a suitable trench box/boxes into the trench leaving the trench slightly above the bottom unsupported to allow appropriate backfilling in the haunching area.
- Backfill the excavation behind the trench box/boxes
- Remove the damaged sections of the CSP culvert extension (~13 m long). Sequence as practical.
- Examine condition of the subgrade bedding material and repair as necessary.
- Install the new CSP culvert extension and re-connect to the existing culvert.
- Backfill the excavation using the recommended procedures. Reshape the final slope to design configurations.

2.5.6 Design Soil Parameters

The parameters shown in Table 2.8 below may be used for the design of the culvert extension and associated works.

Table 2.8 Material types and strength parameters

Material	Effective		Undrained Shear Strength C_u (kPa)	Unit Weight γ (kN/m ³)
	Friction Angle ϕ'	Cohesion c' (kPa)		
Granular A	35°	0	-	22
Granular B Type I	30°	0	-	21
Granular B Type II	35°	0	-	21
Silty Clay Fill	16°	3	20	19
Silty Clay	16°	3	30	19

2.5.7 Culvert Bedding and Backfilling

For the new culvert installations, the following requirements apply. The new CSP culvert extension must be placed on the appropriate culvert bedding as specified in OPSS 514. The culvert bedding should consist of Granular “A” (OPSS 1010) with a minimum

thickness of 500 mm beneath the culvert and extend a minimum of 500 mm horizontally on either side of the culvert edge and slope down at 1H:1V, as specified in OPSD 802.010 attached in Appendix E. The bedding material should be placed in layers not exceeding 200 mm in thickness, loose measurement, and compacted to at least 95% of the Standard Proctor Maximum Dry Density before a subsequent layer is placed in accordance with OPSS 514. Bedding material placed in the haunches must be compacted prior to continued placement of cover material. Bedding on each side of the pipe shall be completed simultaneously. At no time shall the levels on each side differ more than the 200 mm uncompacted layer.

Prior to placing any fill material, the exposed native subgrade should be inspected according to SSP 902S01. A non-woven geotextile separator is to be placed between the approved subgrade and the compacted fill to assist in material placement and maintain the integrity of the founding soil along the entire length of the culvert extension.

The culvert backfill should consist of Granular “B”, Type I or Granular “A” (OPSS 1010) placed in layers not exceeding 300 mm in thickness for the full width of the trench and each layer shall be compacted to 95% of the Standard Proctor Maximum Dry Density before a subsequent layer is placed according to OPSS 514.

The culvert should be encased with a minimum of 300 mm of compacted material. Typical backfill diagrams are presented in Appendix E, OPSD 802.010 and 802.014. The minimum height of fill cover above the crown of the pipe before power operated tractors or rolling equipment shall be 900 mm, unless otherwise noted by the structural engineer.

For this application, it is assumed that the existing bedding material was installed in accordance with the specifications and good practices. After removal of the damaged pipe, this should be verified and any deficient areas made good. The rest of the installations can then proceed as specified.

2.5.8 Dewatering

It is the responsibility of the Contractor to propose a suitable dewatering system based on the time of construction and groundwater levels and creek flow conditions for prior approval of the MTO. The method used should not undermine the existing road. During the construction the upstream flow of Culvert # 60 (west side of Highway 11) should be directed away from the culvert. It is recommended that the work be done in good weather to minimize impacts of water and weak soil.

2.6 Assessment of Options for Remedial Works

As indicated, two approaches for the repair have been considered. These are:

- (i) Suitably designed shoring such as, sheet piling braced as and where required.
- (ii) Flattening of the existing slope to 5H:1V and using trench boxes to protect the local excavation for removal and replacement.

Costs for these systems would depend on a number of variables including mobilization. It is expected for Option 1 (shoring) that for excavations about 2.5 m to 3 m deep, cantilevered or braced systems using AZ 19-700 sheeting or similar, would be in the order of \$600/m² to \$800/m². Quantities of about 200 m² to 250 m² are anticipated.

For the trench boxes, supply rates in the range of \$5000 to \$10,000/week would be anticipated. Additional costs would be incurred for excavations, material removals etc. to accommodate slope flattening. Preliminary estimates indicate that about 600 – 800 m³ of material would need to be moved.

For ease of assessments, advantages and disadvantages of the two schemes are listed in Table 2.9.

The first option is significantly more expensive, but provides more positive support noting that the area experienced slope movement during initial installation. The stability analyses suggest that it should also be possible to execute the remedial work employing a scheme of initial slope excavation (flattening) and trench box use for local side slope protection during reinstatement. While significantly less costly and less time consuming, this latter option is less positive, more susceptible to local poor zones and assume that access for equipment can be accommodated and the work carried out without surcharging side slopes during excavations.

After assessing all related elements, it is considered that given the existing conditions and history, Option 1, shored excavation employing a suitably designed sheet pile scheme, is the preferred option.

Table 2.9 Advantages and disadvantages of the recommended excavation support options

Recommended Excavation Support Options	Advantages	Disadvantages
Trench Box and Side Slope Flattening to 5H:1V	<ul style="list-style-type: none"> -trench slope stability enhanced -global slope stability enhanced by flattening -positive protection for the people in the trench -the excavation and backfill quantities are reduced -costs 	<ul style="list-style-type: none"> -slope flattening is required for global stability -excavated material or equipment cannot be placed on the side slope. Access and practical approach to execution must be carefully planned. -excavation must be performed from the bottom of the slope -disposal of excavated material for flattening
Shoring	<ul style="list-style-type: none"> -flattening is not required -trench and global slope stability enhanced -positive protection for the people in the trench -access to the trench is maximized -help to control ground water 	<ul style="list-style-type: none"> -installation and extract can cause logistic delays -difficulties in installation if boulders and cobbles are present -installation and extraction process may cause disturbance of the adjacent highway structures -higher costs and specialists subcontractors may be required

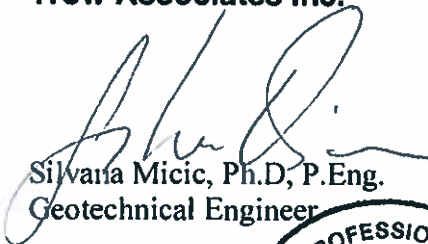
3.0 Closure

This report has been prepared by S. Micic, Ph.D., P.Eng. and reviewed by S. Gonsalves, M.Eng., P.Eng. Designated MTO Foundation Contact. The field investigation was conducted by Victor Tam and Greg Qu.

We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

Yours truly,

Trow Associates Inc.


Silvana Micic, Ph.D, P.Eng.
Geotechnical Engineer




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



APPENDIX A

Photographs



Photograph 1: Culvert # 60 at Station 11+684 (Pacaud Township).
On east side of Highway 11, looking west. Culvert outlet and failed CSP pipe.



Photograph 2: Culvert # 60 at Station 11+684 (Pacaud Township).
On east side of Highway 11, looking north-west. Failed slope.



Photograph 3: Culvert # 60 at Station 11+684 (Pacaud Township).
On east side of Highway 11, looking south. Culvert outlet and gravel lined ditch.



Photograph 4: Culvert # 60 at Station 11+684 (Pacaud Township).
On east side of Highway 11, looking south-east. Failed slope and north roadside ditch.



Photograph 5: Culvert # 60 at Station 11+684 (Pacaud Township).
On east side of Highway 11, looking south. Culvert outlet and gravel lined ditch.



Photograph 6: Culvert # 60 at Station 11+684 (Pacaud Township).
On east side of Highway 11, looking north. Roadside ditch.

APPENDIX B

Drawing

APPENDIX C

Borehole Logs

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

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

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

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

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

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINT AND BEDDING:

SPACING	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_i	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{\text{max}} - e}{e_{\text{max}} - e_{\text{min}}}$
P_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ'	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(w_L - w_p)$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(w - w_p) / I_p$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(w_L - w) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						









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1595 Clark Boulevard Ltd.
Brampton, Ontario L6T 4V1

RECORD OF BOREHOLE No BH60-1

SHEET 1 OF 1

METRIC

PROJECT NO. SD000391349A LOCATION Failed Zone, Culvert #60, N379988.2 E5311734 ORIGINATED BY VT
DIST ON HWY 11 BOREHOLE TYPE Hollow Stem Auger, 200mm COMPILED BY GQ
DATUM Geodetic DATE 06/16/2009 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	SPT TEST (N-Value) ● DYNAMIC CONE PENETRATION 		PLASTIC LIMIT	NATURAL WATER CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						PL	w	LL	WATER CONTENT (%)	GR	SA	SI	CL	
242.3 0.0	FILL - gravel and sand; compact, brown, damp		1	AS			242															
			2	SS	14		241												20	66	(14)	
240.0 2.3								240														
	FILL - clayey sand, interbedded with silty clay layers, trace gravel, brown and grey, wet		3	SS	5			239														
239.2 3.1	SILTY CLAY (CI) - varved structure, trace wet clayey silt seams, grey, saturated, firm		4	SS	3																	
			5	SS	3			238														
								237														
			6	SS	3		236											0	1	37	62	
							235															
			7	SS	1		234															
			8	SS	1		233															
232.1 10.2	END OF BOREHOLE Notes: 1. Borehole advanced by hollow-stem augers. 2. Standpipe piezometer installed to 9.14m depth; bentonite sealed between 0.0m to 0.91m depth and 6.4m to 7.3m depth. 3. This drawing is part of subject report, project number as referenced, and must only be read in conjunction with that report. 4. Interpretation assistance by Trow is required before use by others.																					

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

ONL_MOT_HWY11_ENGLEHEART_17 JULY 2009 VANEGREGMETHOD.UNMODIFIED.GPJ ON_MOT.GDT 09/08/06

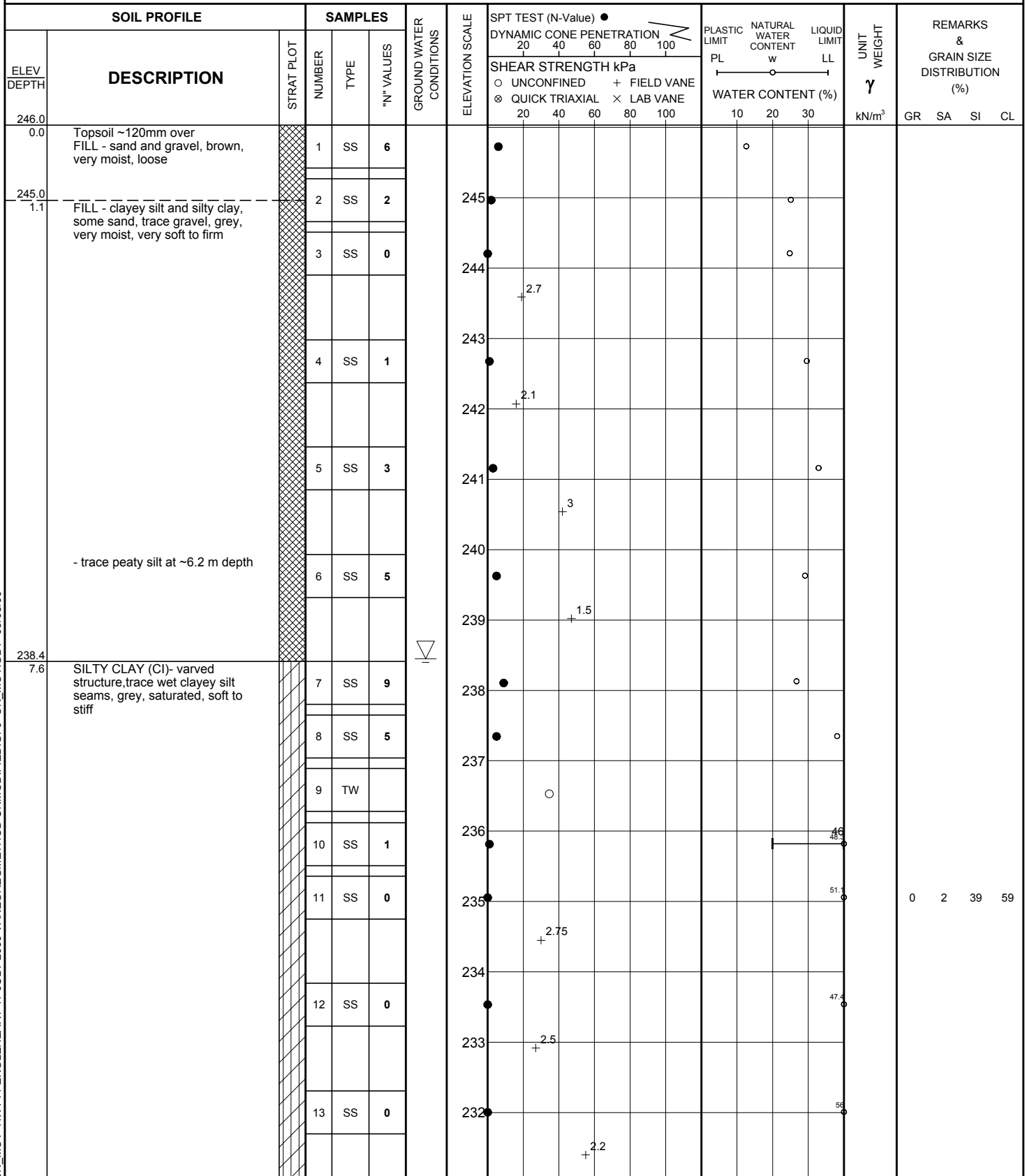


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Brampton, Ontario L6T 4V1

RECORD OF BOREHOLE No BH60-2 SHEET 1 OF 2

METRIC

PROJECT NO. SD000391349A LOCATION Embankment Crest (East), Culvert #60, N379977.4 E5311731 ORIGINATED BY VT
DIST ON HWY 11 BOREHOLE TYPE Hollow Stem Auger, 200mm COMPILED BY GQ
DATUM Geodetic DATE 06/15/2009 - 06/16/2009 CHECKED BY SM



Continued Next Page

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

ON_MOT_HWY11 ENGLEHEART 17 JULY 2009 VANEGREGMETHOD UNMODIFIED.GPJ ON_MOT_GDT 09/08/06



Trow Associates Inc.
1595 Clark Boulevard Ltd.
Brampton, Ontario L6T 4V1

RECORD OF BOREHOLE No BH60-2 SHEET 2 OF 2

METRIC

PROJECT NO. SD000391349A LOCATION Embankment Crest (East), Culvert #60, N379977.4 E5311731 ORIGINATED BY VT
DIST ON HWY 11 BOREHOLE TYPE Hollow Stem Auger, 200mm COMPILED BY GQ
DATUM Geodetic DATE 06/15/2009 - 06/16/2009 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	SPT TEST (N-Value) ●		PLASTIC LIMIT PL	NATURAL WATER CONTENT w	LIQUID LIMIT LL	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			DYNAMIC CONE PENETRATION						WATER CONTENT (%)	GR	SA	SI	CL	
								SHEAR STRENGTH kPa											
								○ UNCONFINED ⊗ QUICK TRIAXIAL	+ FIELD VANE × LAB VANE										
						20	40	60	80	100	10	20	30						
			14	SS	1														
229.0																			
17.0	CLAYEY SILT - trace sand; grey, wet, soft		15	SS	2											0	9	66	25
227.4																			
18.6	SILTY SAND - trace gravel, occasional silty clay seams, grey, wet, very loose to loose		16	SS	5														
225.6			17	SS	1														
20.4	END OF BOREHOLE																		
	Notes: 1. Borehole advanced by hollow-stem augers 2. This drawing is part of subject report, project number as referenced, and must only be read in conjunction with that report. 3. Interpretation assistance by Trow is required before use by others.																		

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






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RECORD OF BOREHOLE No BH60-3

SHEET 1 OF 1

METRIC

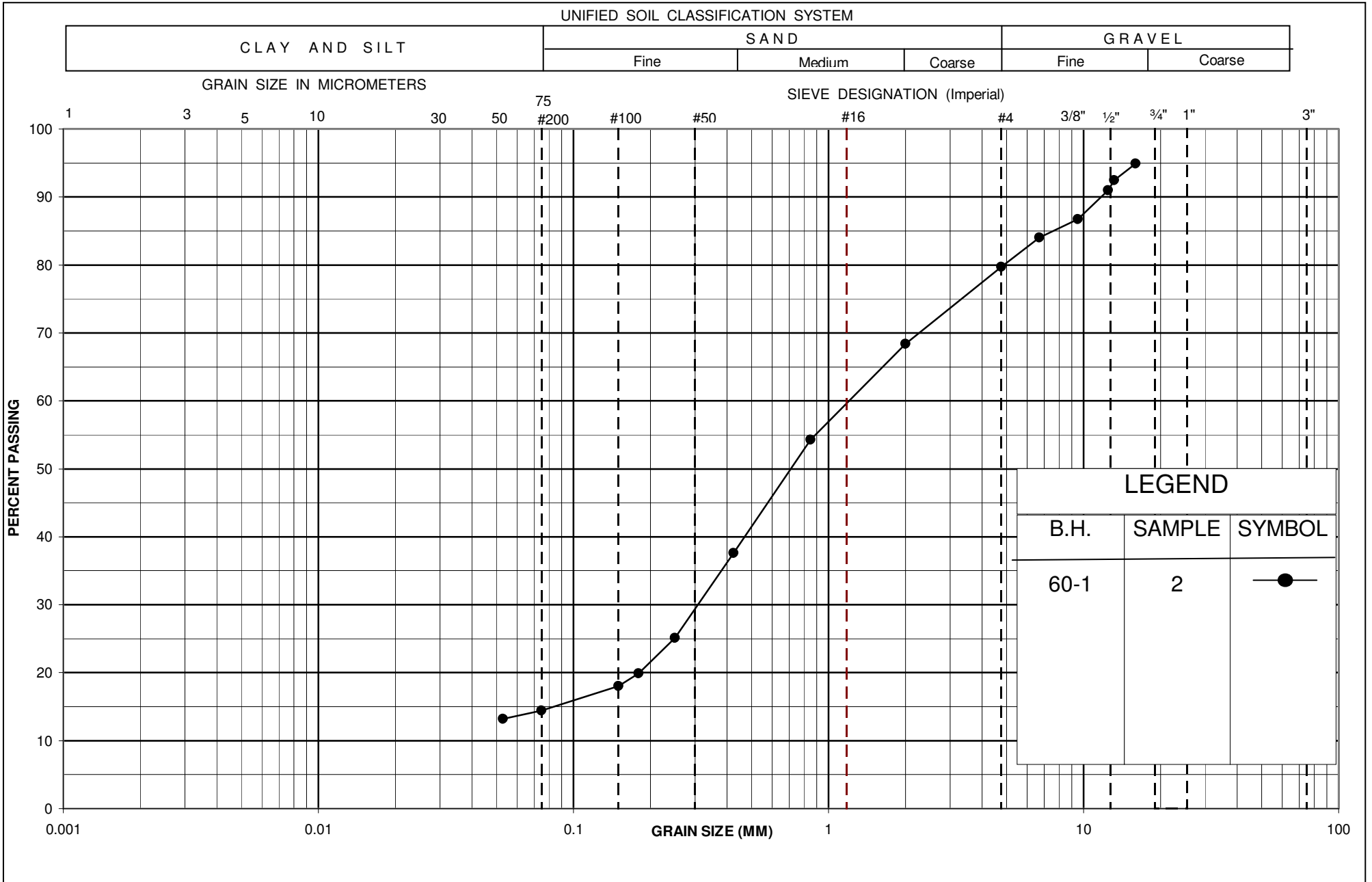
PROJECT NO. SD000391349A LOCATION Embankment Toe(East), Culvert #60, N379997.4 E5311747 ORIGINATED BY VT
DIST ON HWY 11 BOREHOLE TYPE Hand Drilling COMPILED BY GQ
DATUM Geodetic DATE 06/22/2009 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	SPT TEST (N-Value) ●		PLASTIC LIMIT PL	NATURAL WATER CONTENT w	LIQUID LIMIT LL	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			DYNAMIC CONE PENETRATION 						WATER CONTENT (%)	GR	SA	SI	CL				
								SHEAR STRENGTH kPa											+ FIELD VANE × LAB VANE			
238.8 0.0	Topsoil ~120mm over FILL -silty clay, trace gravel, occasional decayed wood fragments and rootlets; organic stains, brown, damp -becoming grey and very moist below 1.5m SILTY CLAY (CI to CH)- varved structure,trace wet clayey silt seams, grey, saturated, firm		1	SS	1		20	40	60	80	100	10	20	30	53 53.9	0	1	42	57			
			2	SS	3		238															
237.0 1.8			3	SS	7		237															
			4	SS	13		236															
			5	SS	9		235															
			6	SS	6		234															
			7	SS	4		233															
			8	TW																		
			9	SS	5																	
232.1 6.7	END OF BOREHOLE																					
	Notes: 1. Borehole advanced by hand drilling/sampling equipment. 2. Standpipe piezometer installed to 4.88m depth; bentonite sealed between 0.0m to 0.91m depth. 3. This drawing is part of subject report, project number as referenced, and must only be read in conjunction with that report. 4. Interpretation assistance by Trow is required before use by others.																					

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

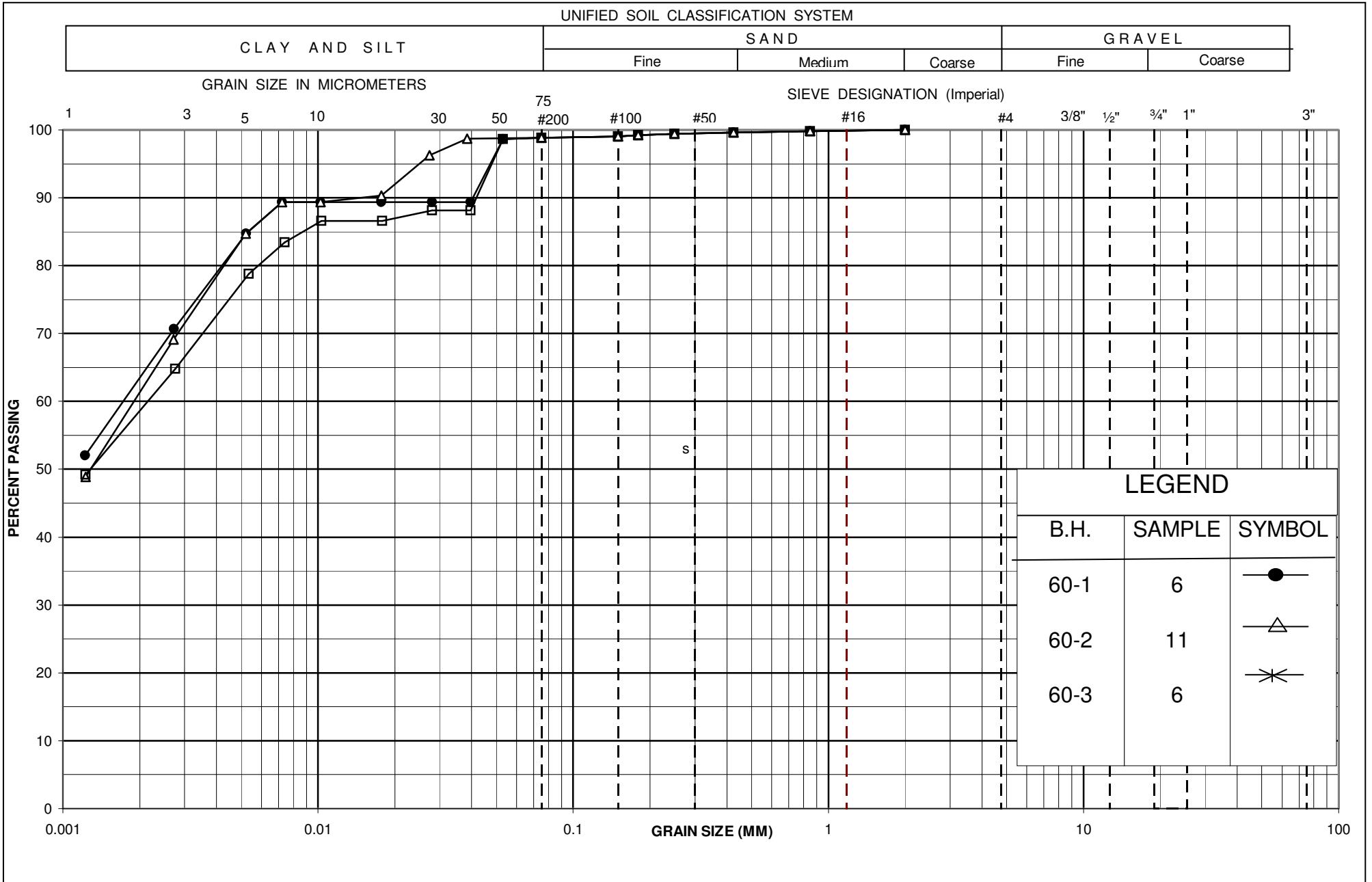
APPENDIX D

Laboratory Data



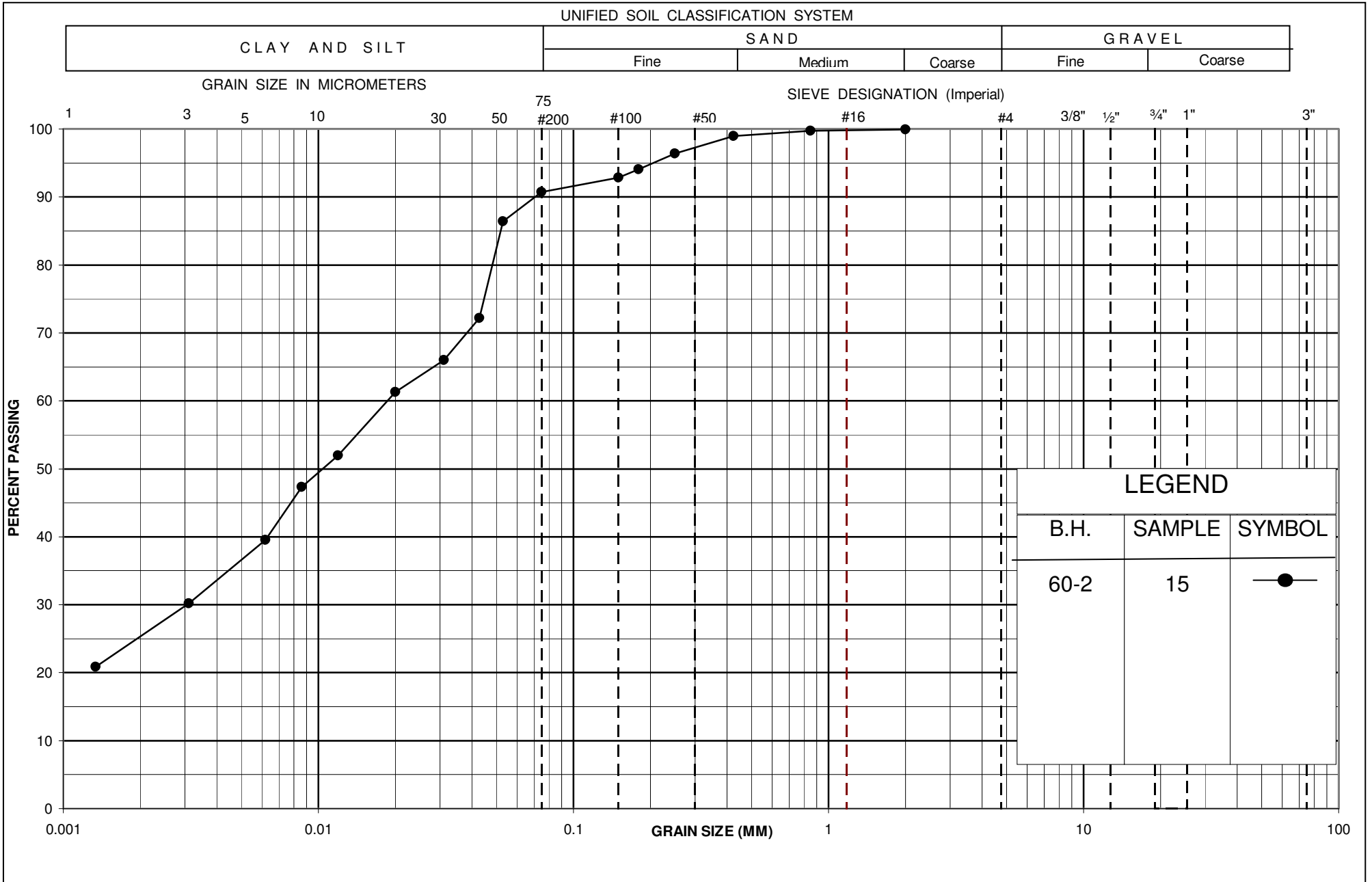
GRAIN SIZE DISTRIBUTION
Sand and Gravel Fill

FIGURE No. 1
WO: 2009-11030
Hwy 11 - Culvert # 60 Extension Replacement and Slope Failure



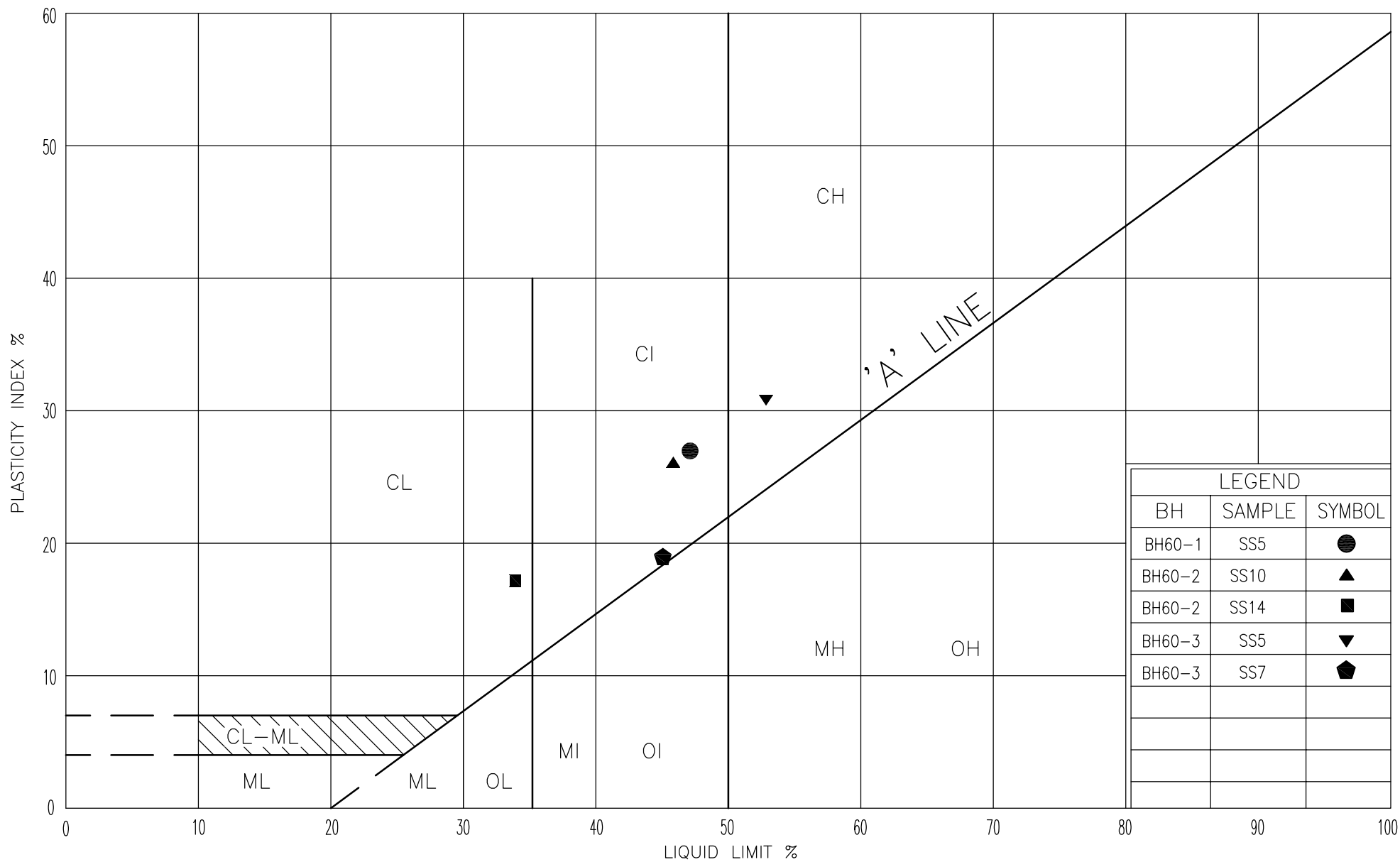
GRAIN SIZE DISTRIBUTION
SILTY CLAY

FIGURE No. 2
WO: 2009-11030
Hwy 11 - Culvert # 60 Extension Replacement and Slope Failure



GRAIN SIZE DISTRIBUTION
CLAYEY SILT

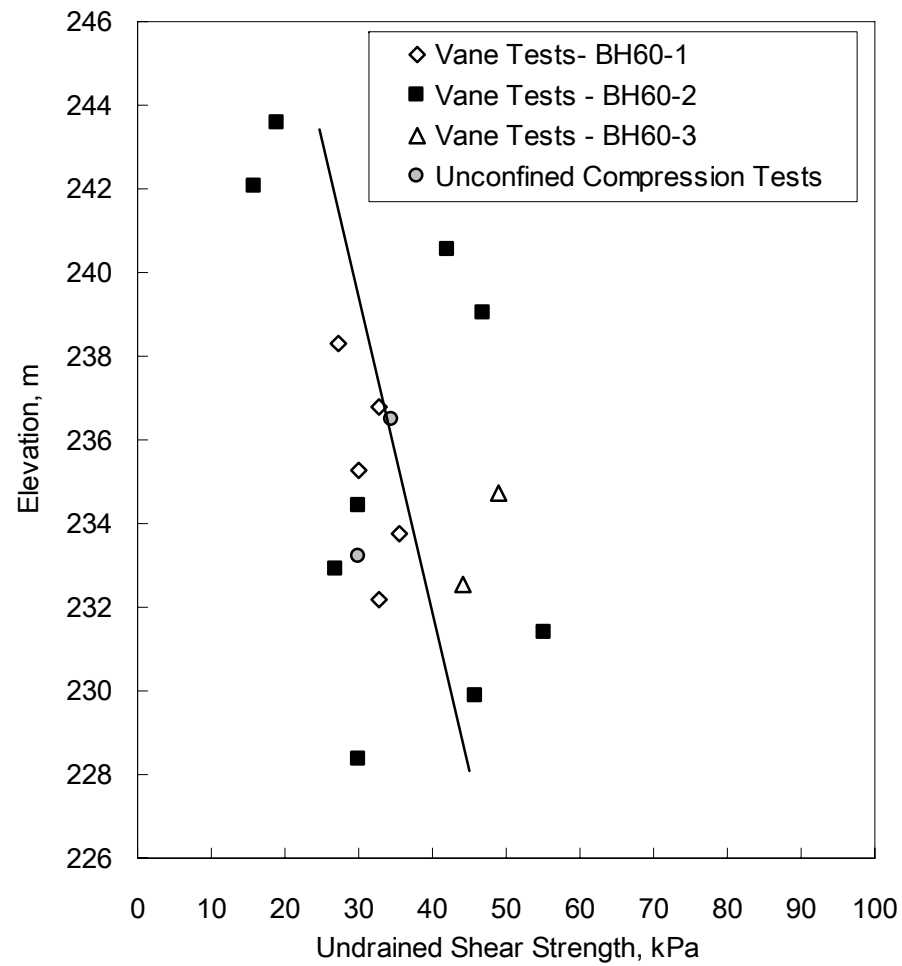
FIGURE No. 3
WO: 2009-11030
Hwy 11 - Culvert # 60 Extension Replacement and Slope Failure



PLASTICITY CHART SILTY CLAY, CL, CI, AND CH

FIGURE No. 4
G.W.P 2009-11030
Hwy 11 - Culvert # 60
Extension Replacement and Slope Failure

Strength Profile Silty Clay



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DATE: July 2009

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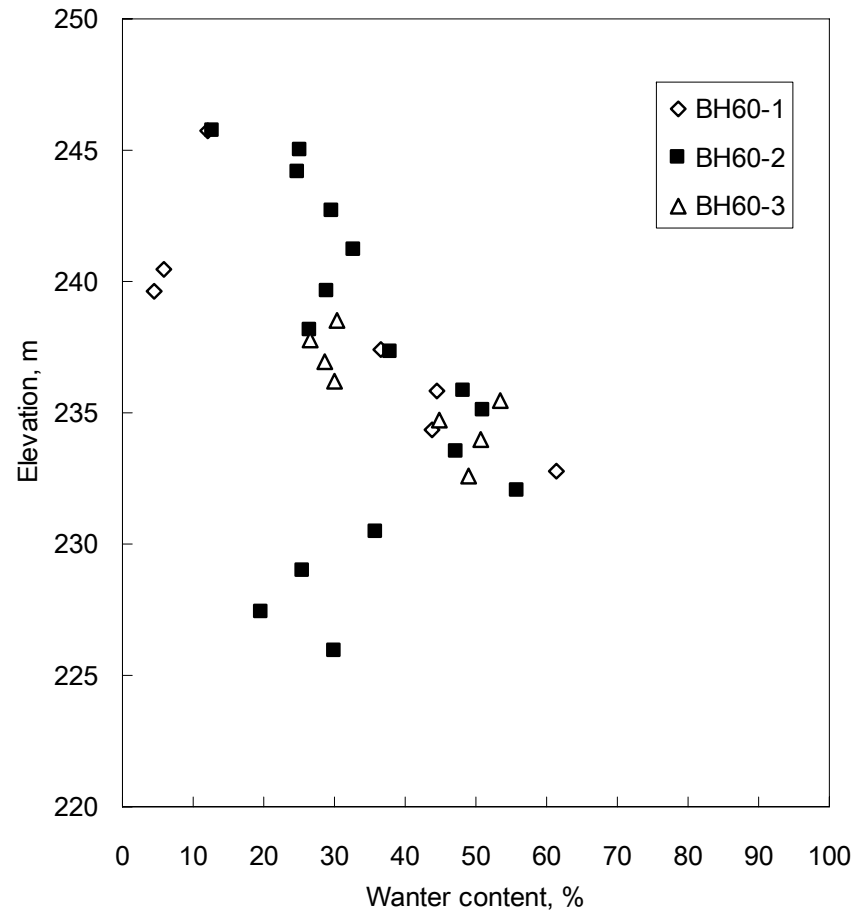
TITLE: Foundation Investigation and Design Report

PROJECT: Englehart Project - HWY 11, Culvert # 60
Replacement and Slope Failure

FIGURE No. 5

PROJECT No.
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Moisture Conten Profile



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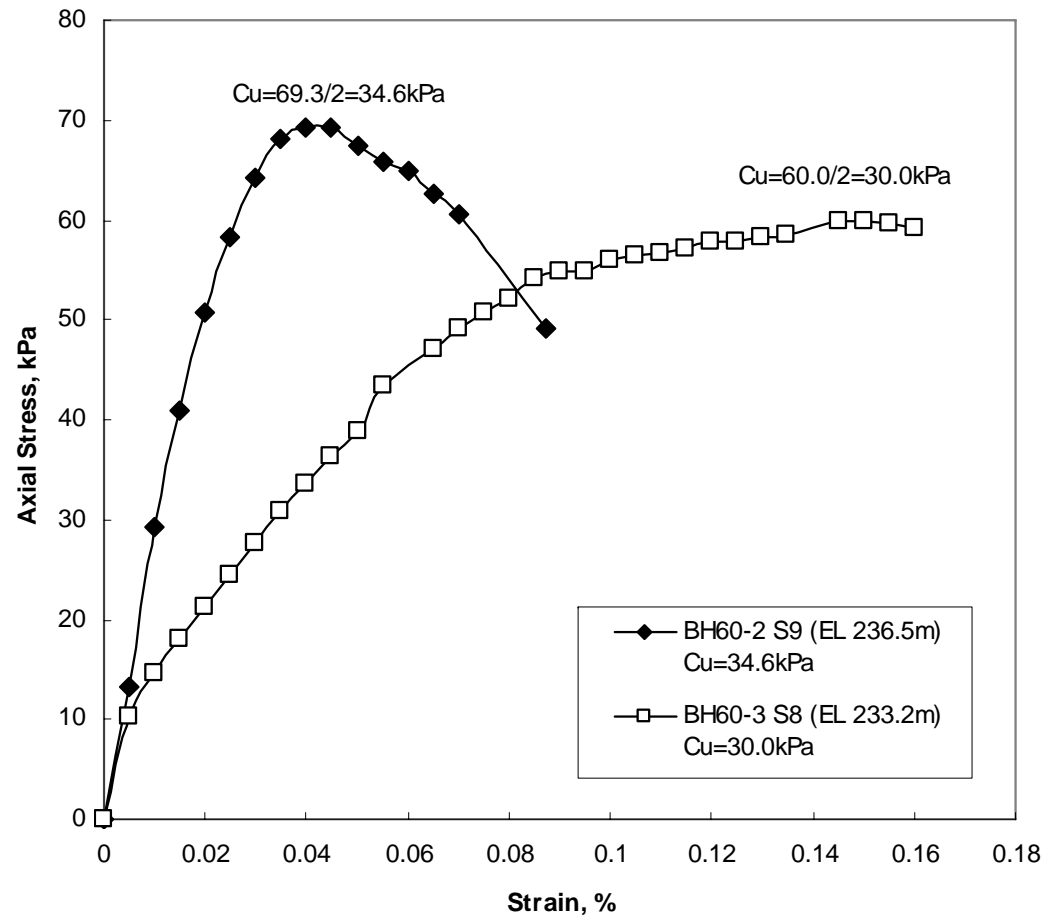
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FIGURE No. 6

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Unconfined Compression Test Silty Clay



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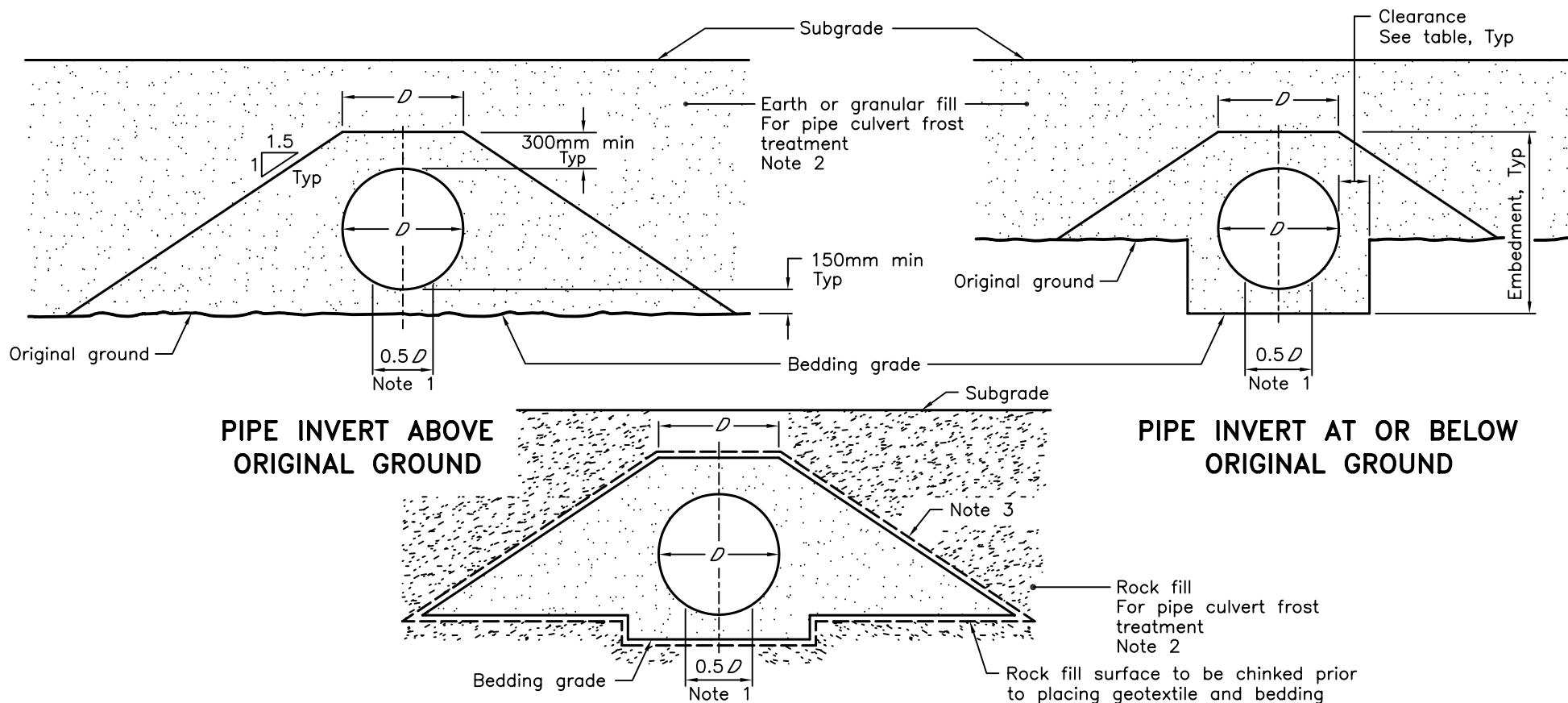
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Replacement and Slope Failure

FIGURE No.7

PROJECT No.
SD000391349a

APPENDIX E

OPSD Specifications



LEGEND:

D – Inside diameter

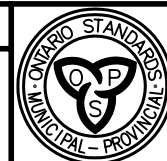
NOTES:

- 1 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
 - 2 Pipe culvert frost treatment according to OPSD-803.030 and 803.031.
 - 3 Embedment material to be wrapped in non-woven geotextile when specified.
- A Granular material placed in the haunch area shall be compacted prior to placing and compacting the remainder of the embedment material.
- B All dimensions are in metres unless otherwise shown.

PIPE EMBEDMENT WITH ROCK FILL UNDER AND OVER THE PIPE

CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING		Nov 2005	Rev	1
FLEXIBLE PIPE EMBEDMENT IN EMBANKMENT				
ORIGINAL GROUND: EARTH OR ROCK				
		OPSD – 802.014		



APPENDIX F

Slope Stability Analyses

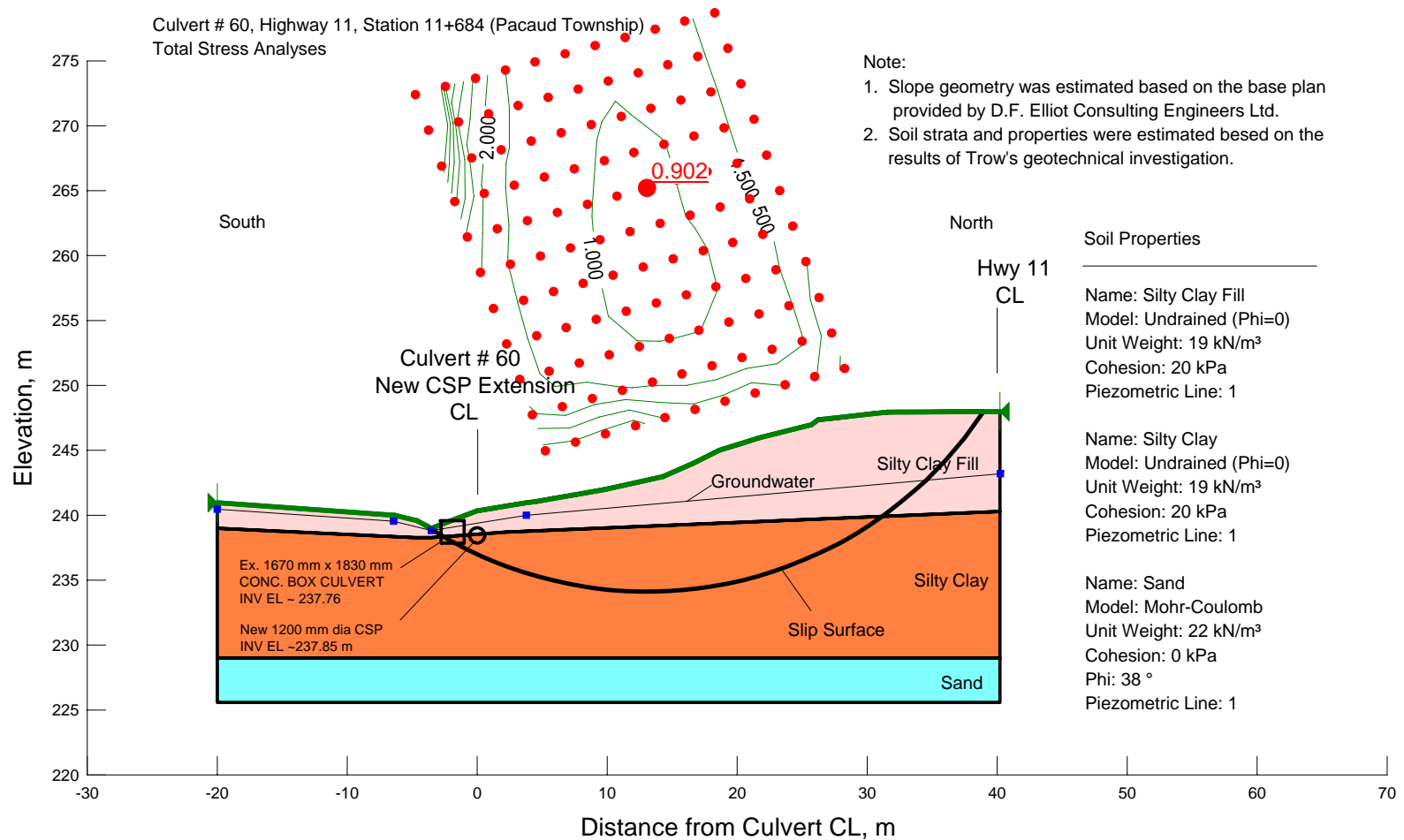


Figure 1 (a). Slope stability analyses before failure (November, 2008) assuming total stress parameters: silty clay fill $C_u=20 \text{ kPa}$ and silty clay $C_u=20 \text{ kPa}$.



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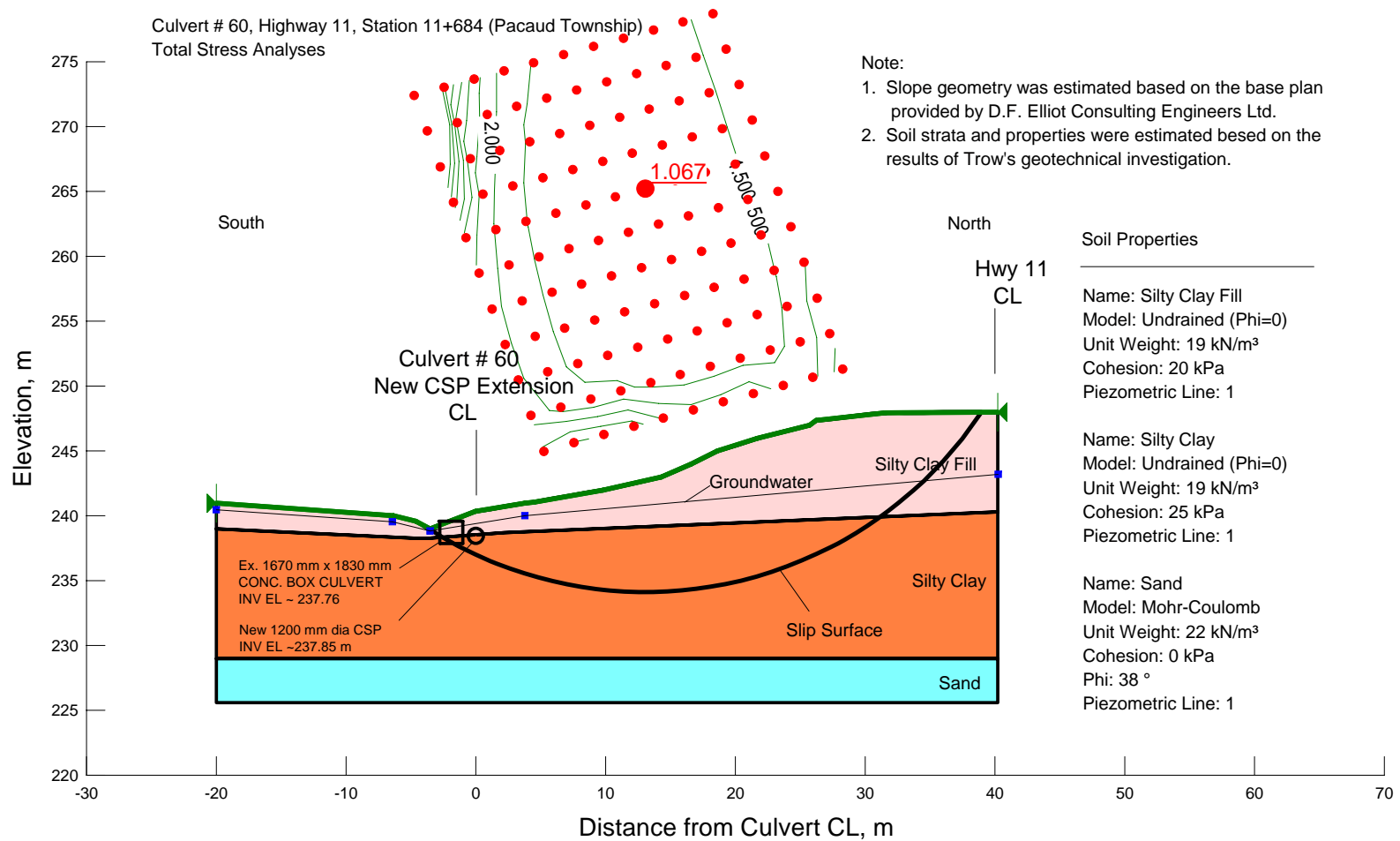


Figure 1 (b). Slope stability analyses before failure (November, 2008) assuming total stress parameters: silty clay fill $C_u=20$ kPa and silty clay $C_u=25$ kPa.



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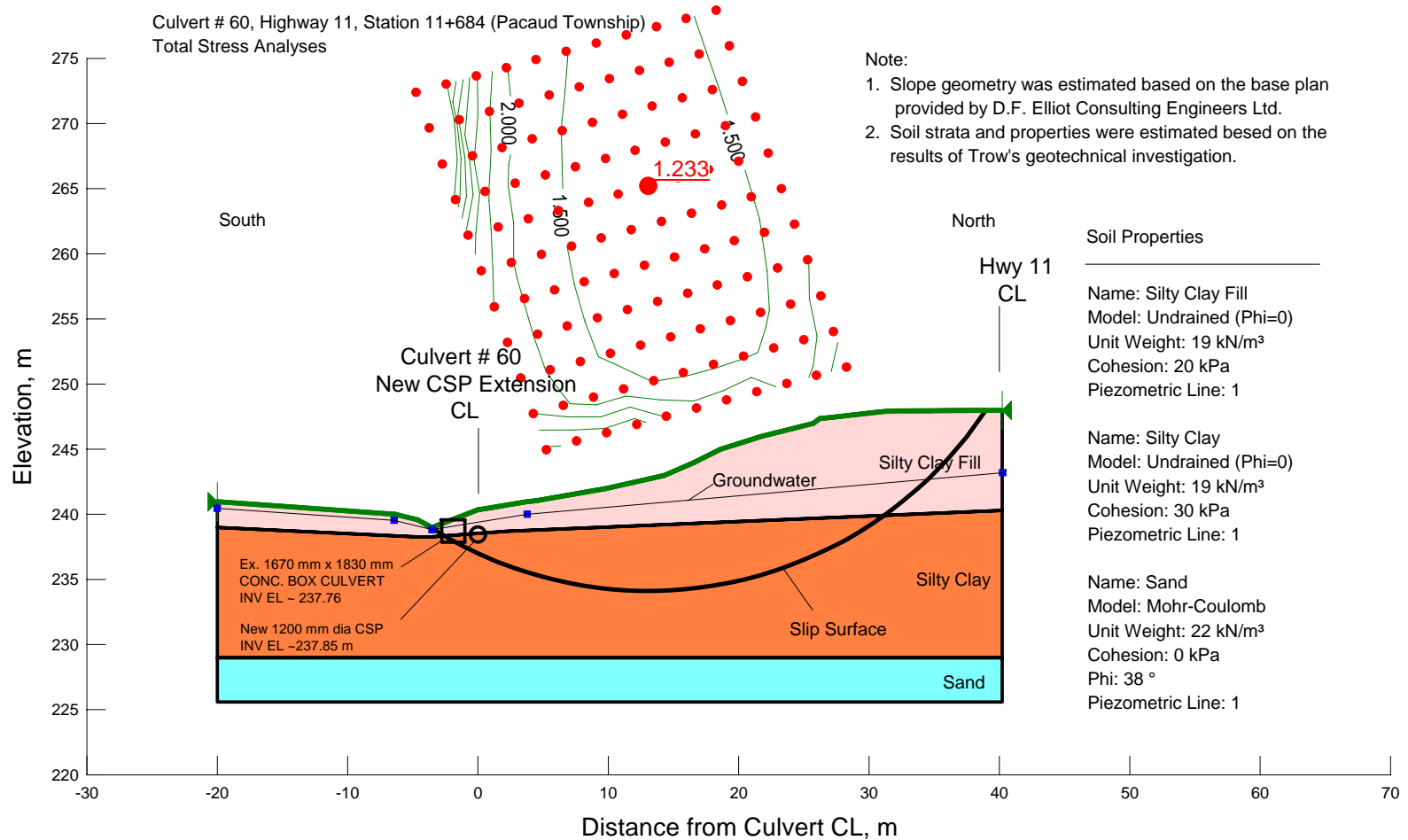


Figure 1 (c). Slope stability analyses before failure (November, 2008) assuming total stress parameters: silty clay fill $C_u=20$ kPa and silty clay $C_u=30$ kPa.



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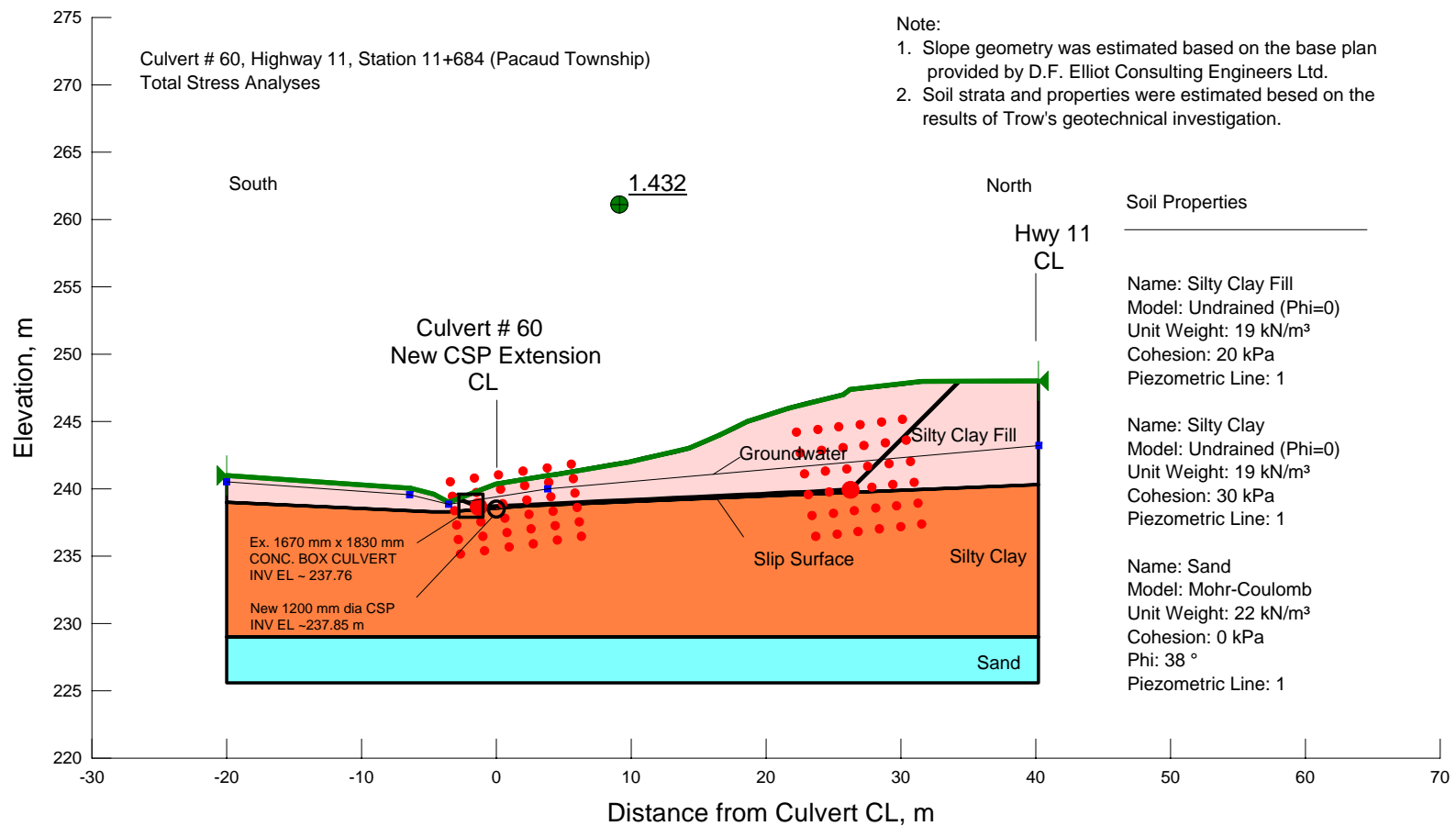


Figure 2. Slope stability analyses before failure (November, 2008) assuming total stress parameters: silty clay fill $C_u=20$ kPa and silty clay $C_u=30$ kPa; and using non-circular slip surface



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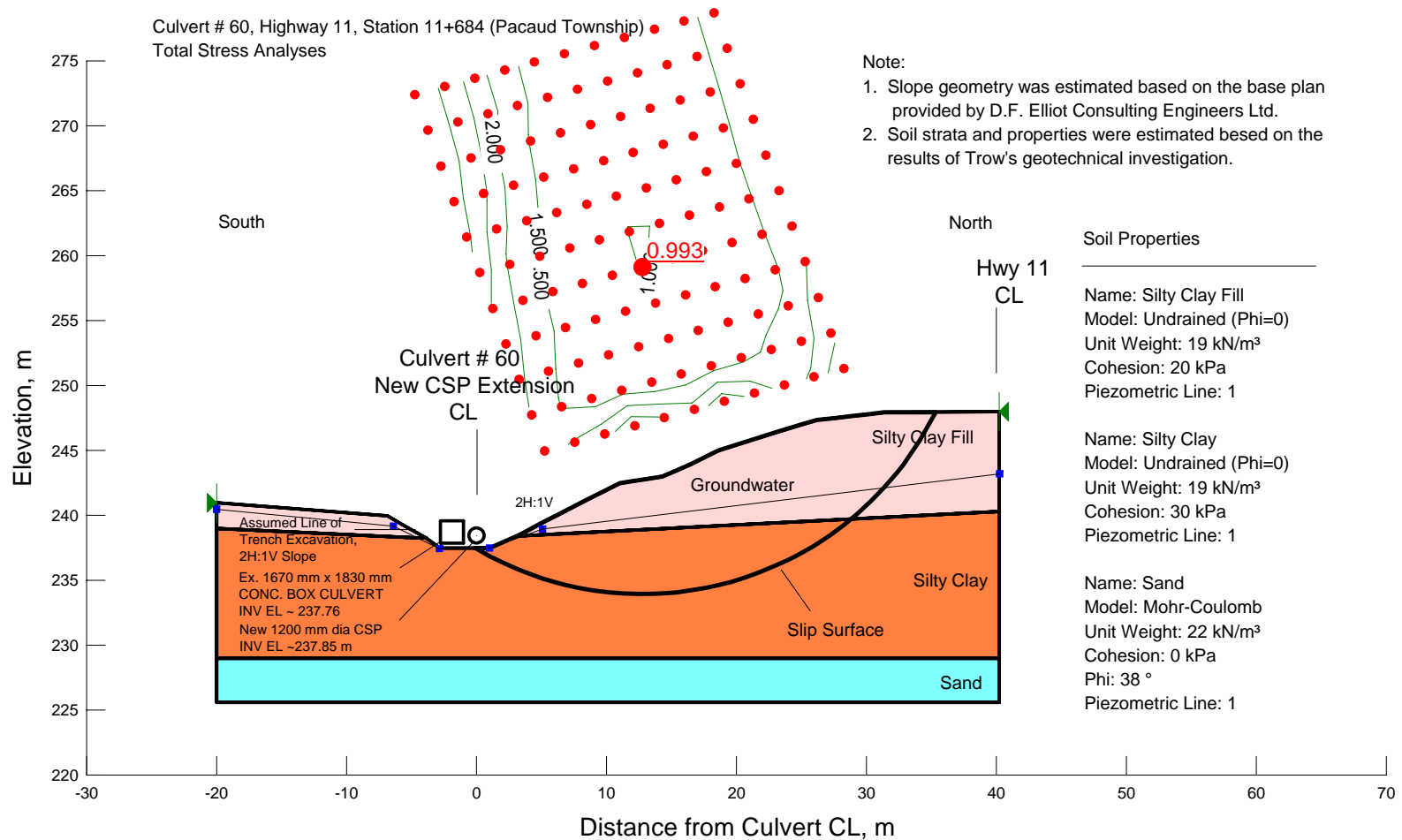


Figure 3. Slope stability analyses assuming trench excavation 2H:1V and total stress parameters: silty clay fill $C_u=20$ kPa and silty clay $C_u=30$ kPa.



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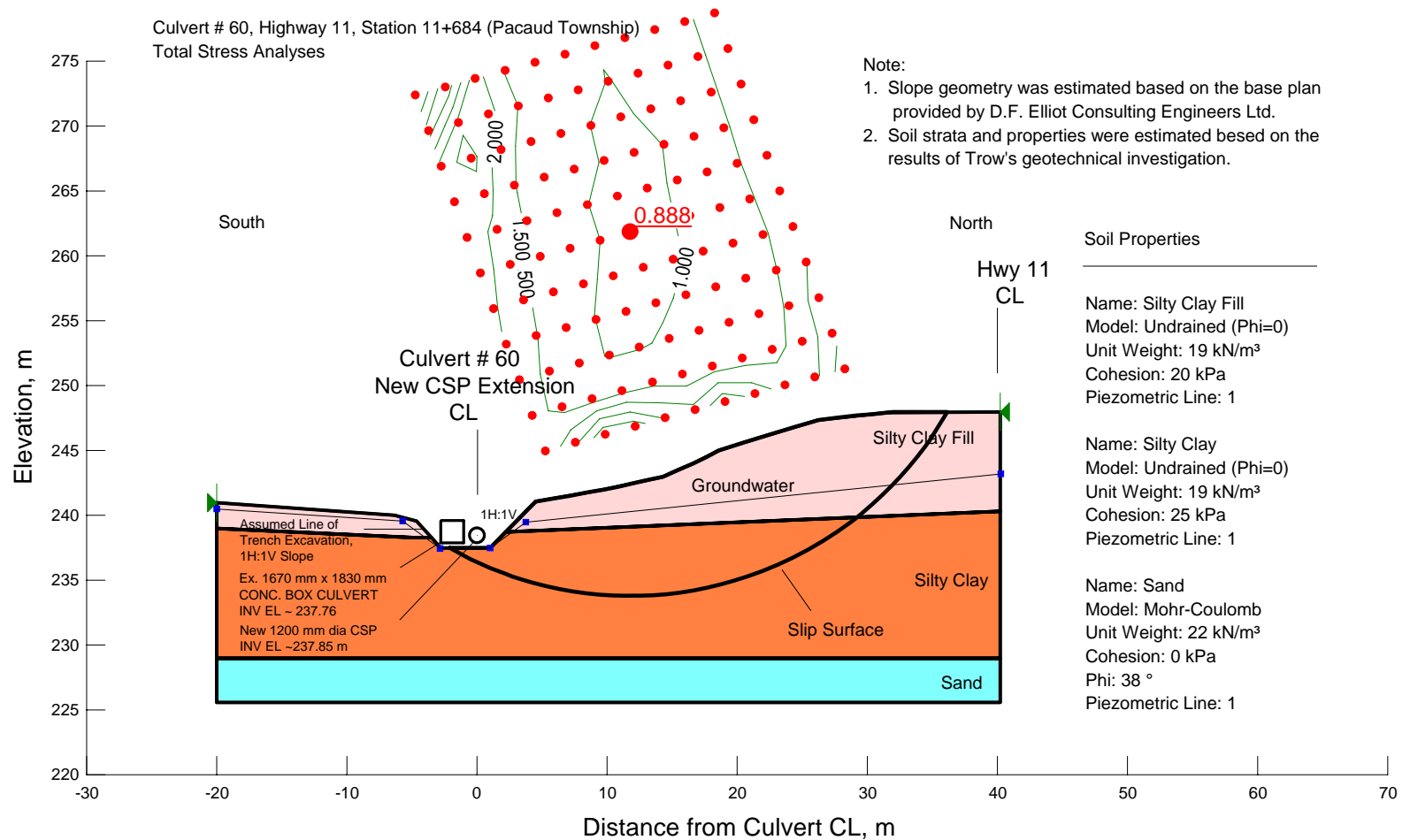


Figure 4(a). Slope stability analyses assuming trench excavation 1H:1V and total stress parameters: silty clay fill $C_u=20$ kPa and silty clay $C_u=25$ kPa.



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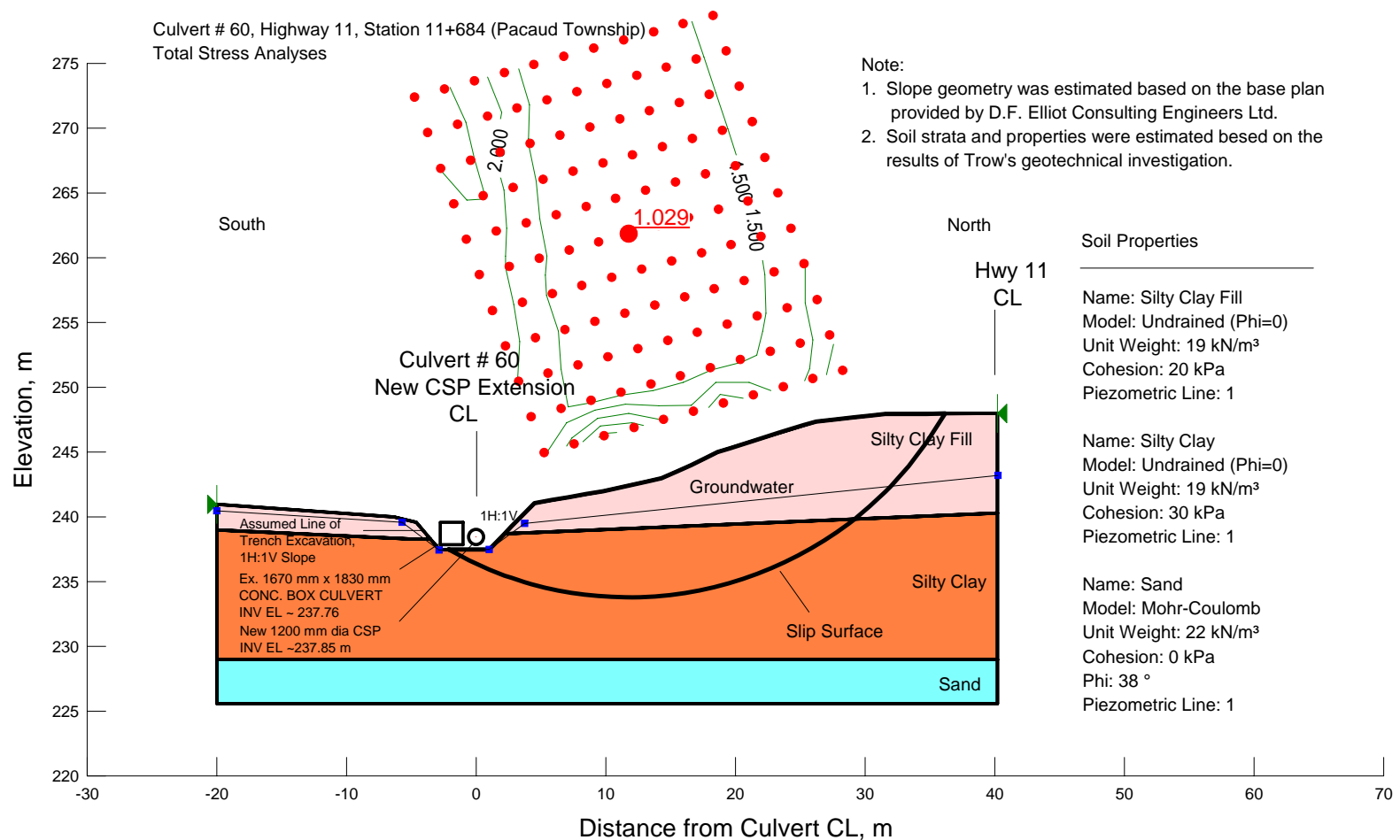


Figure 4(b). Slope stability analyses assuming trench excavation 1H:1V and total stress parameters: silty clay fill $C_u=20$ kPa and silty clay $C_u=30$ kPa.



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TITLE:	Foundation Investigation and Design Report
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SD000391349a

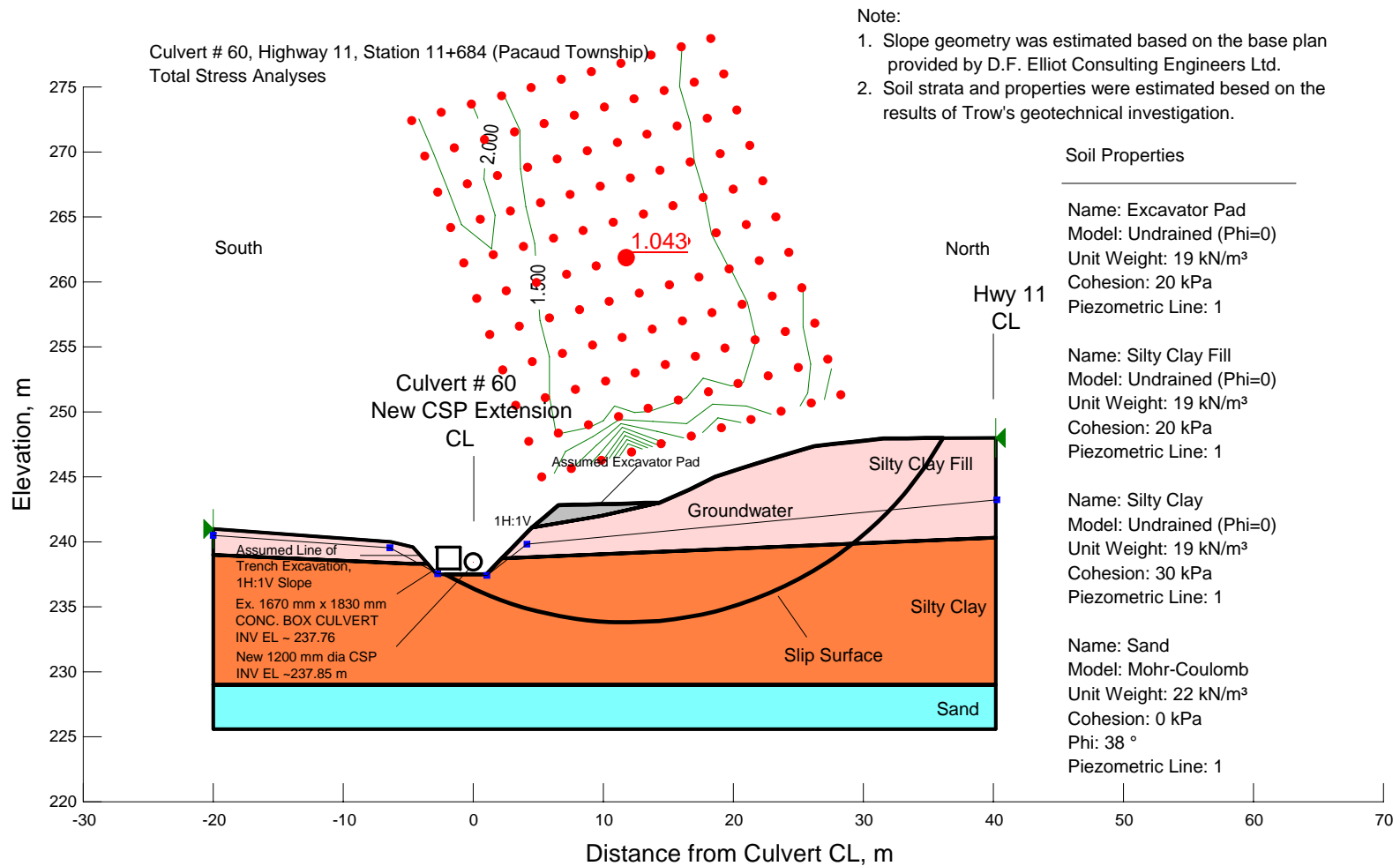


Figure 5. Slope stability analyses assuming trench excavation 1H:1V, excavator pad as surcharge on the slope and total stress parameters: silty clay fill $C_u=20$ kPa and silty clay $C_u=30$ kPa.



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PROJECT: Englehart Project - HWY 11, Culvert # 60
Replacement and Slope Failure

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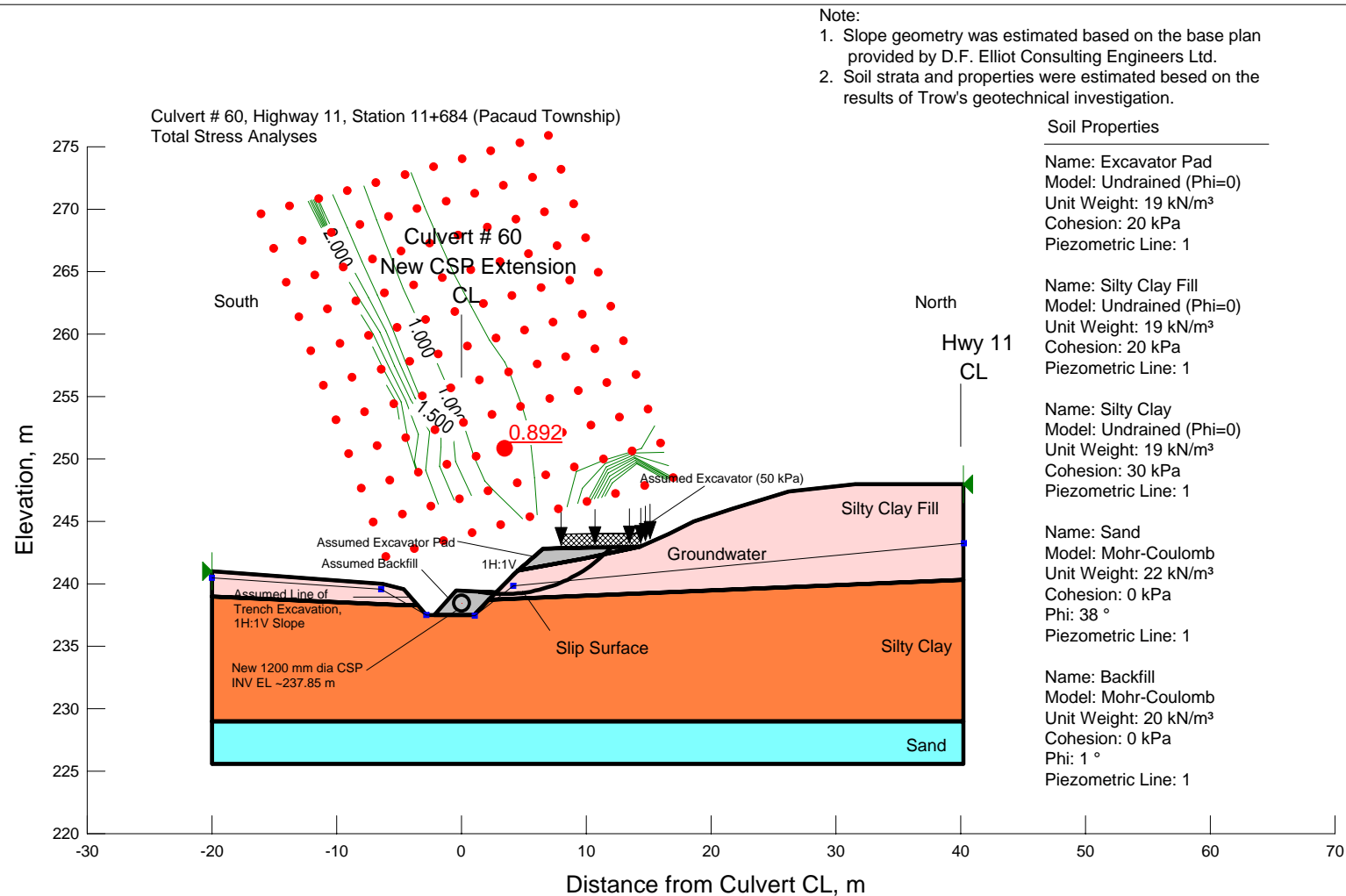


Figure 6(b). Slope stability analyses assuming trench excavation 1H:1V, excavator pad and excavator as surcharge on the slope, culvert backfill and total stress parameters: silty clay fill $C_u=20$ kPa and silty clay $C_u=30$ kPa.



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PROJECT: Englehart Project - HWY 11, Culvert # 60
Replacement and Slope Failure

PROJECT No.
SD000391349a

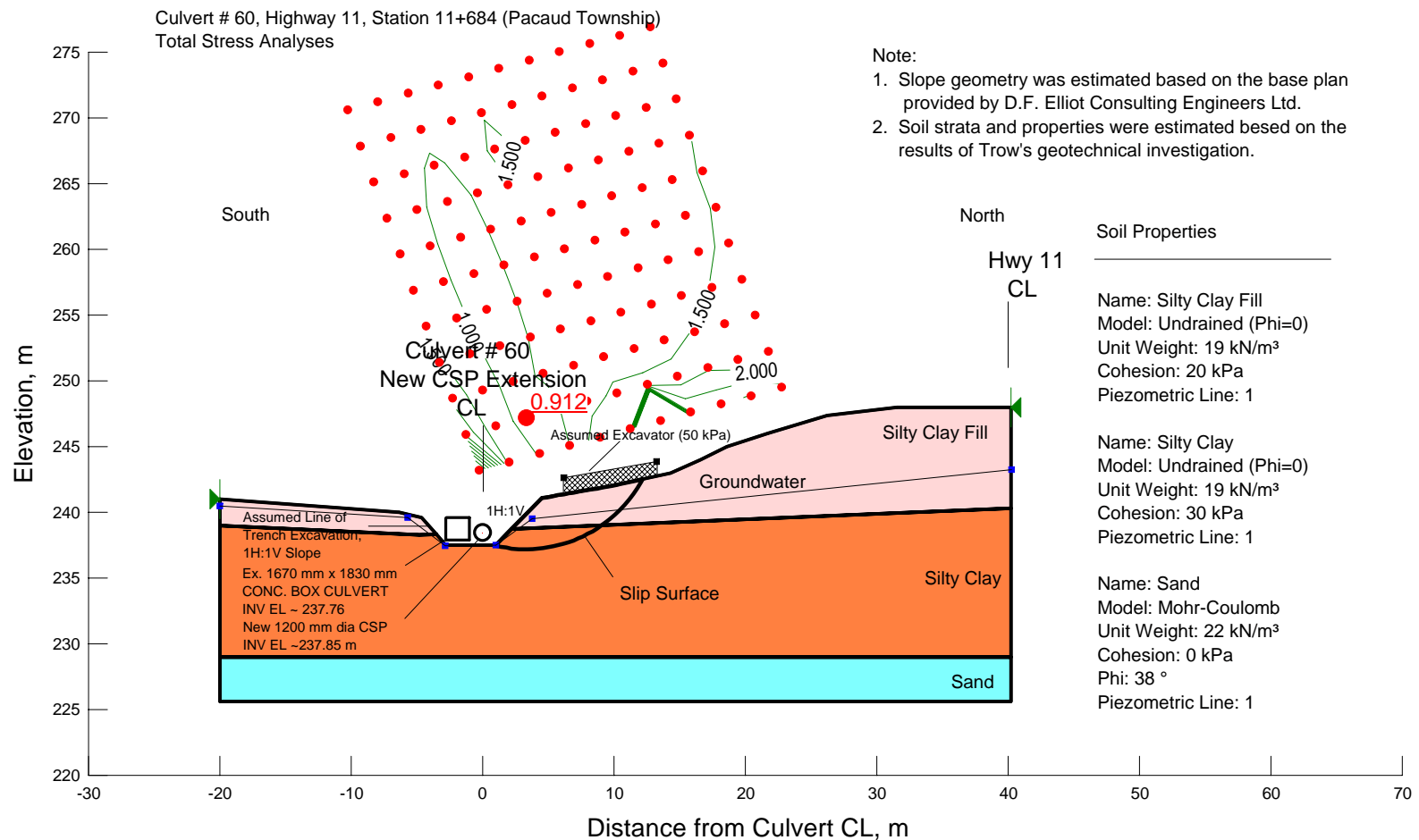


Figure 7(a). Slope stability analyses assuming trench excavation 1H:1V, excavator as surcharge on the slope and total stress parameters: silty clay fill $C_u=20$ kPa and silty clay $C_u=30$ kPa. Critical slip surface for trench slope.



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TITLE: Foundation Investigation and Design Report

PROJECT: Englehart Project - HWY 11, Culvert # 60
Replacement and Slope Failure

PROJECT No.
SD000391349a

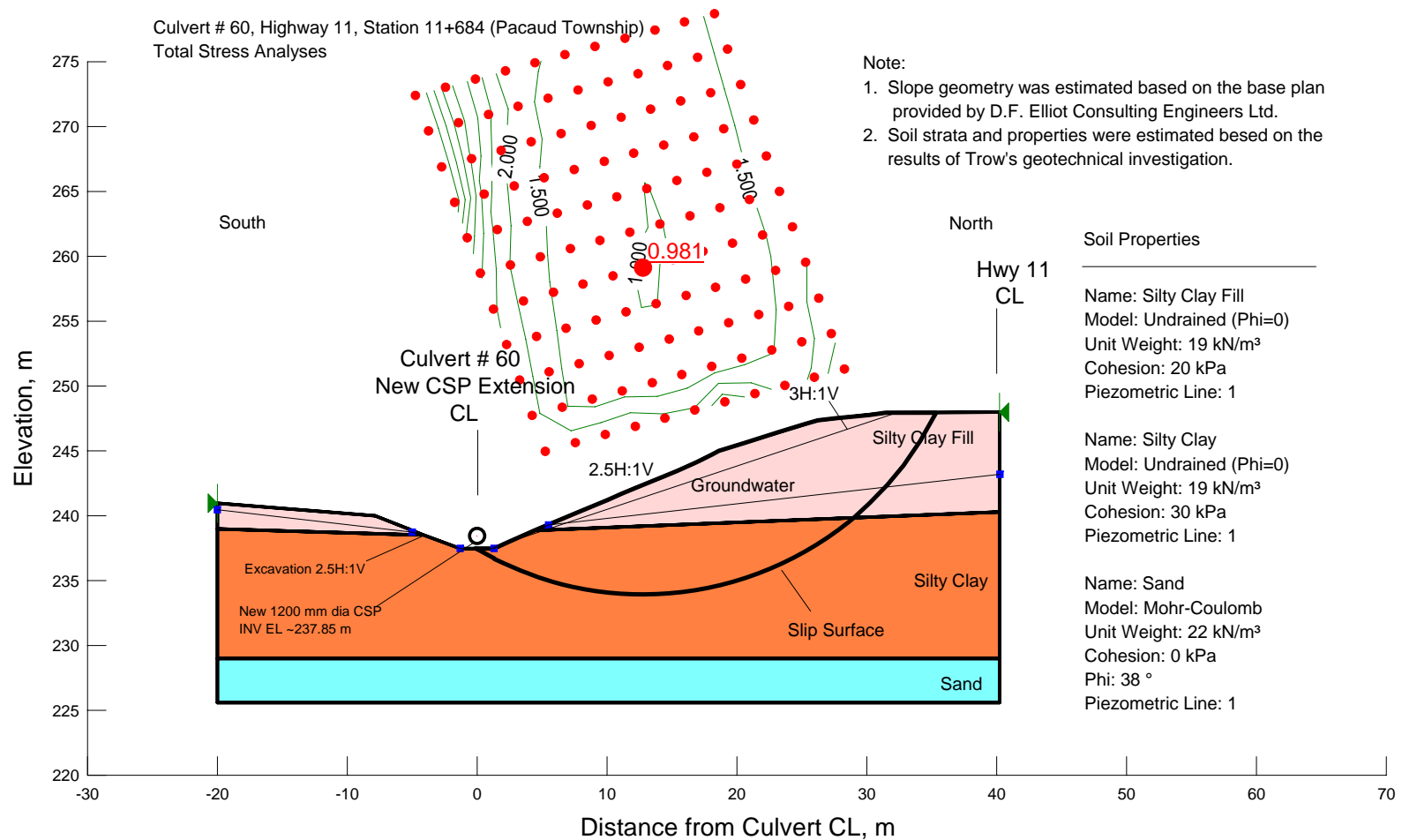


Figure 8. REMEDIATION; Excavation for the failed culvert extension replacement without a support system. Slope stability analysis was performed assuming total stress parameters silty clay fill $C_u=20$ kPa and silty clay $C_u=30$ kPa.



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Replacement and Slope Failure

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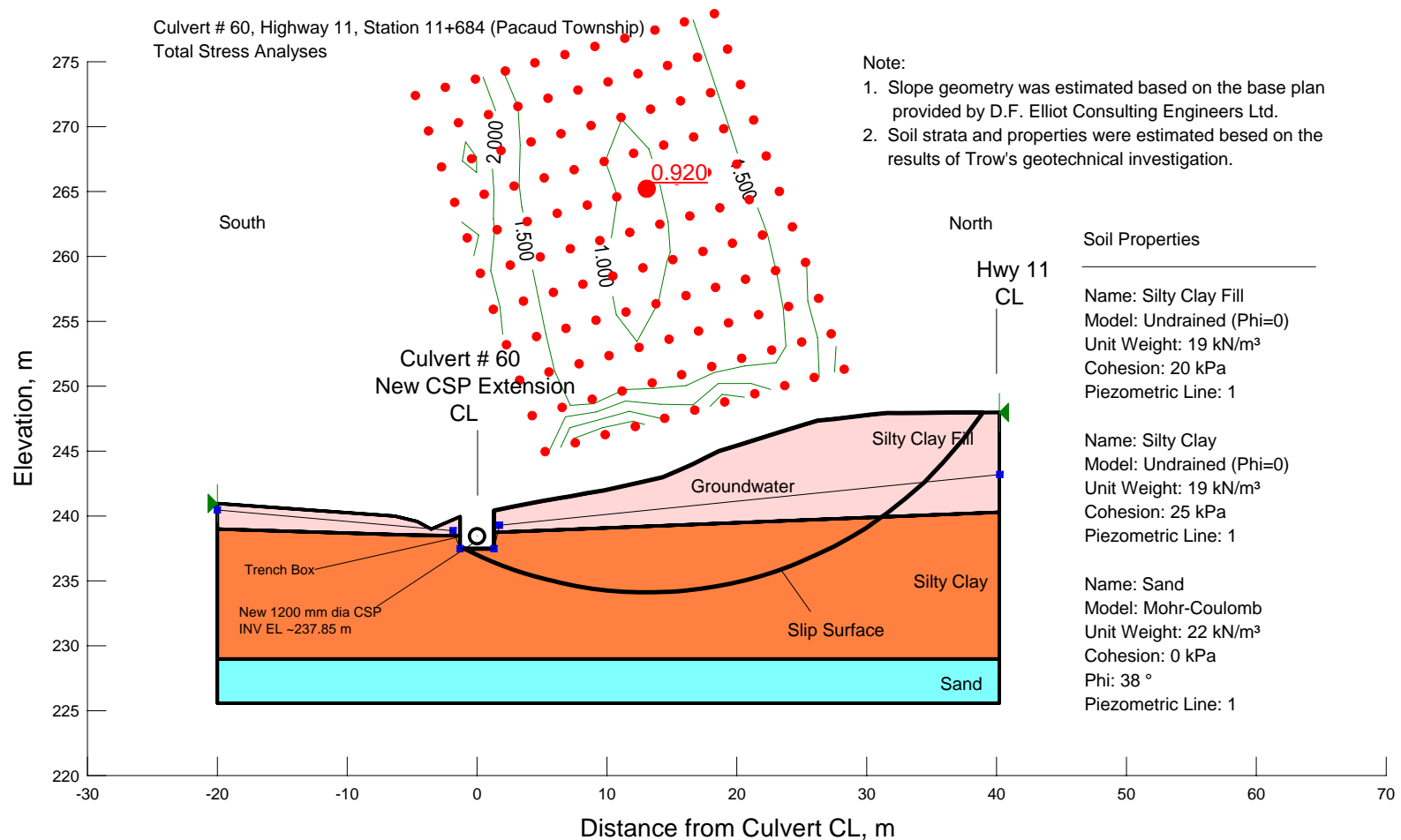


Figure 9(b). REMEDIATION; Excavation for the failed culvert extension replacement using a support system. Slope stability analysis was performed assuming total stress parameters silty clay fill $C_u=20$ kPa and silty clay $C_u=25$ kPa.



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PROJECT: Englehart Project - HWY 11, Culvert # 60
Replacement and Slope Failure

PROJECT No.
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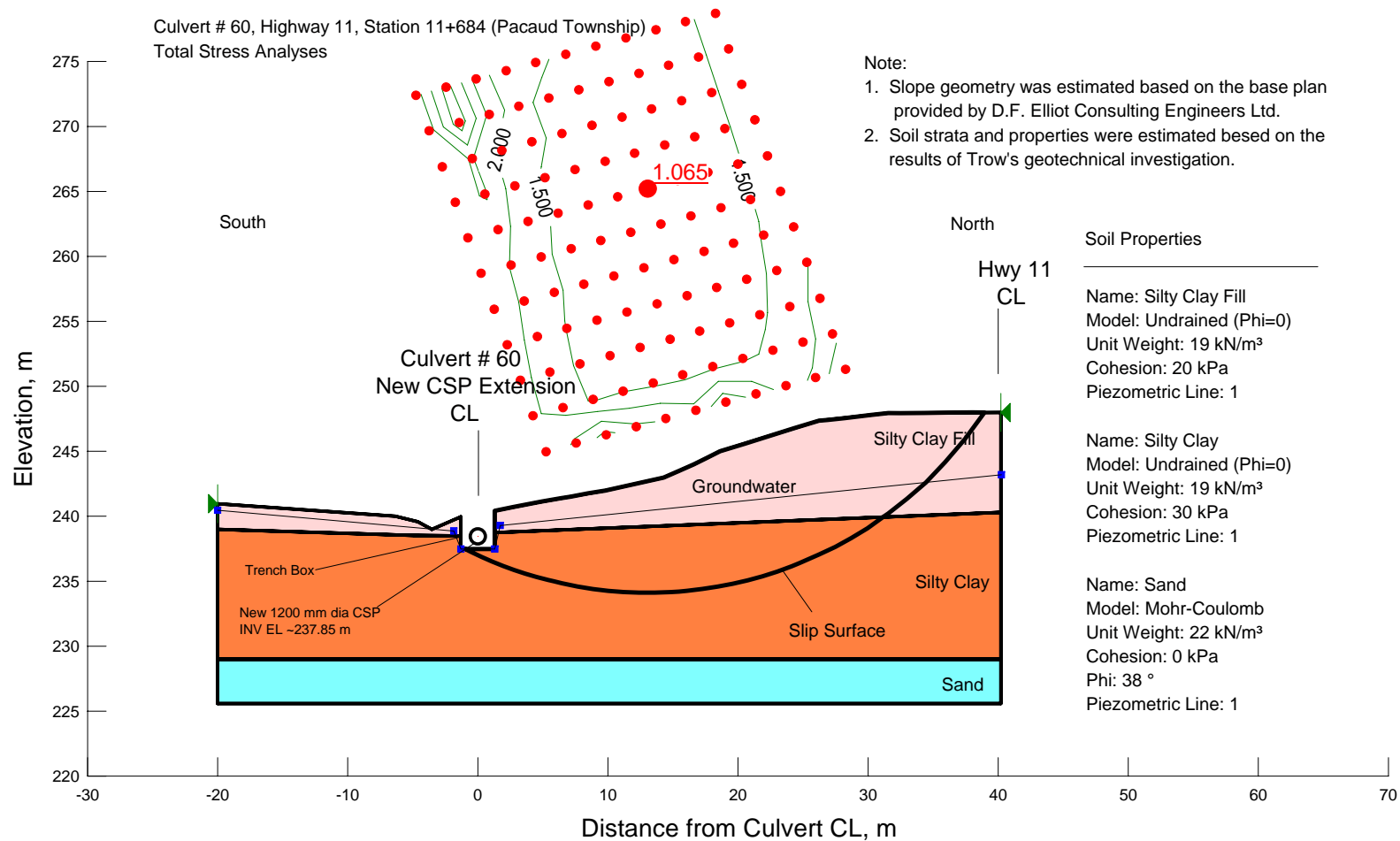


Figure 9(c). REMEDIATION; Excavation for the failed culvert extension replacement using a support system. Slope stability analysis was performed assuming total stress parameters silty clay fill $C_u=20$ kPa and silty clay $C_u=30$ kPa.



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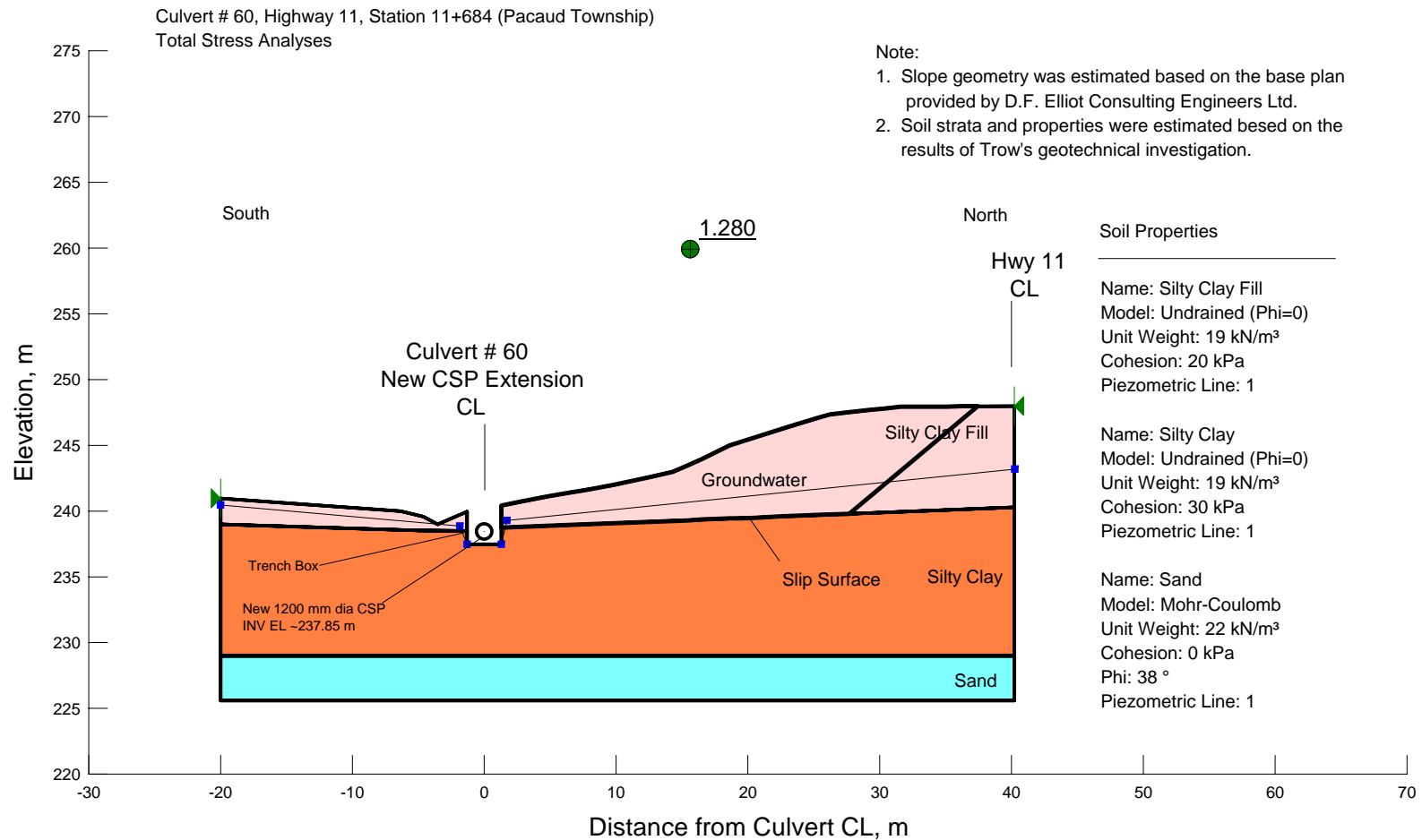


Figure 10. REMEDIATION; Excavation for the failed culvert extension replacement using a support system. Slope stability analysis was performed assuming total stress parameters silty clay fill $C_u=20$ kPa and silty clay $C_u=30$ kPa, and non-circular slip surface.



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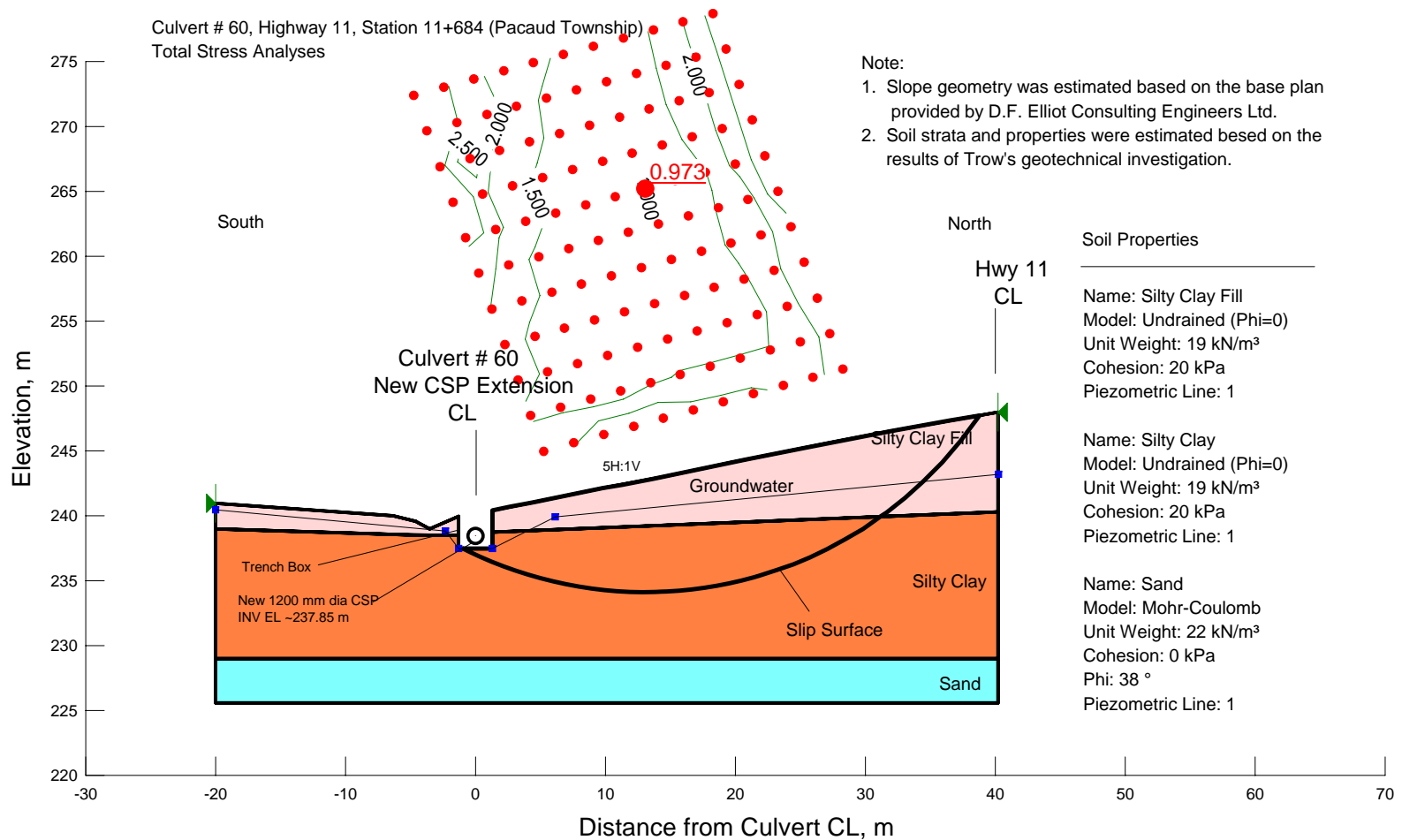


Figure 11(a). REMEDIATION; Excavation for the failed culvert extension replacement using the support system and slope flattening. Slope stability analysis was performed assuming total stress parameters; silty clay fill $C_u=20$ kPa and silty clay $C_u=20$ kPa.



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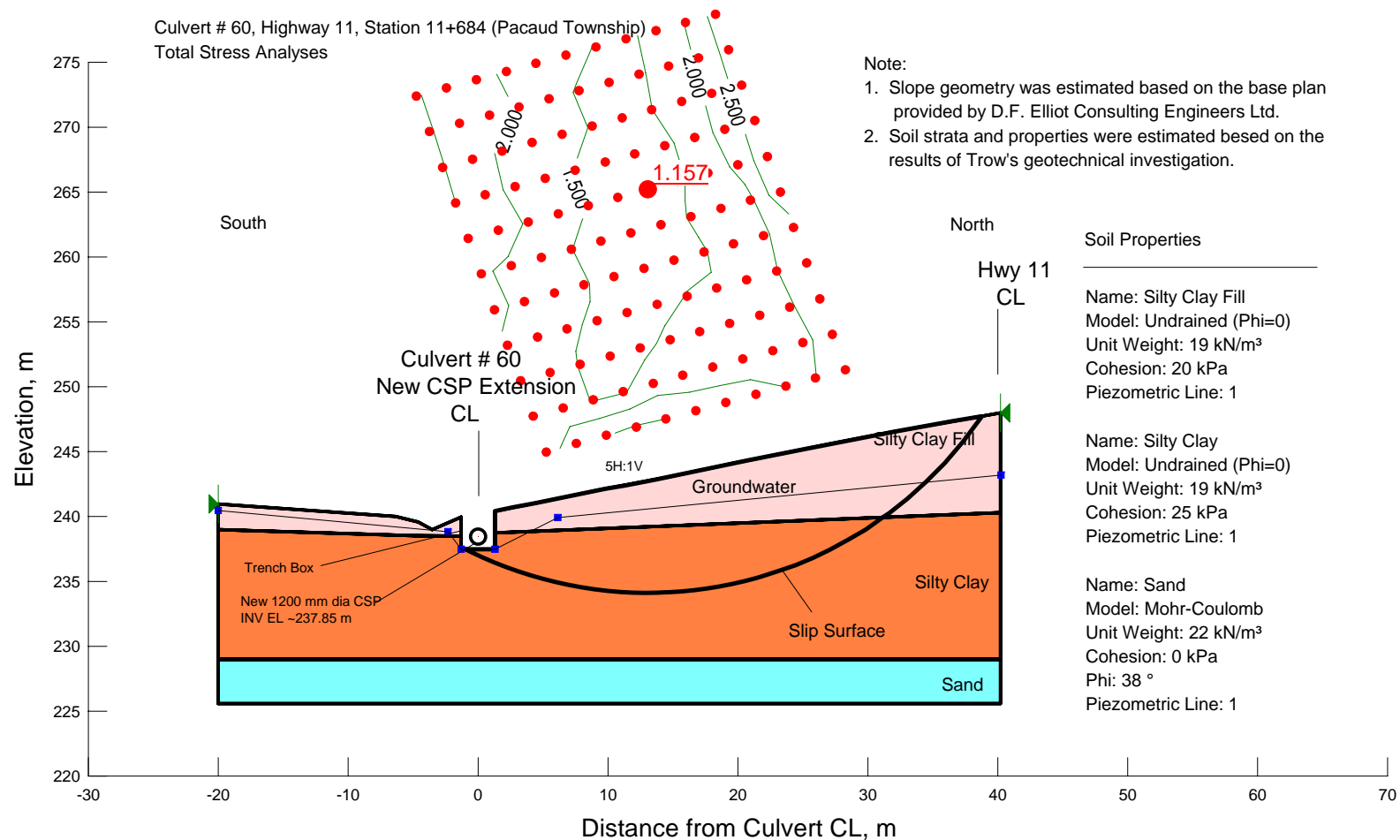


Figure 11(b). REMEDIATION; Excavation for the failed culvert extension replacement using the support system and slope flattening. Slope stability analysis was performed assuming total stress parameters; silty clay fill $C_u=20$ kPa and silty clay $C_u=25$ kPa.



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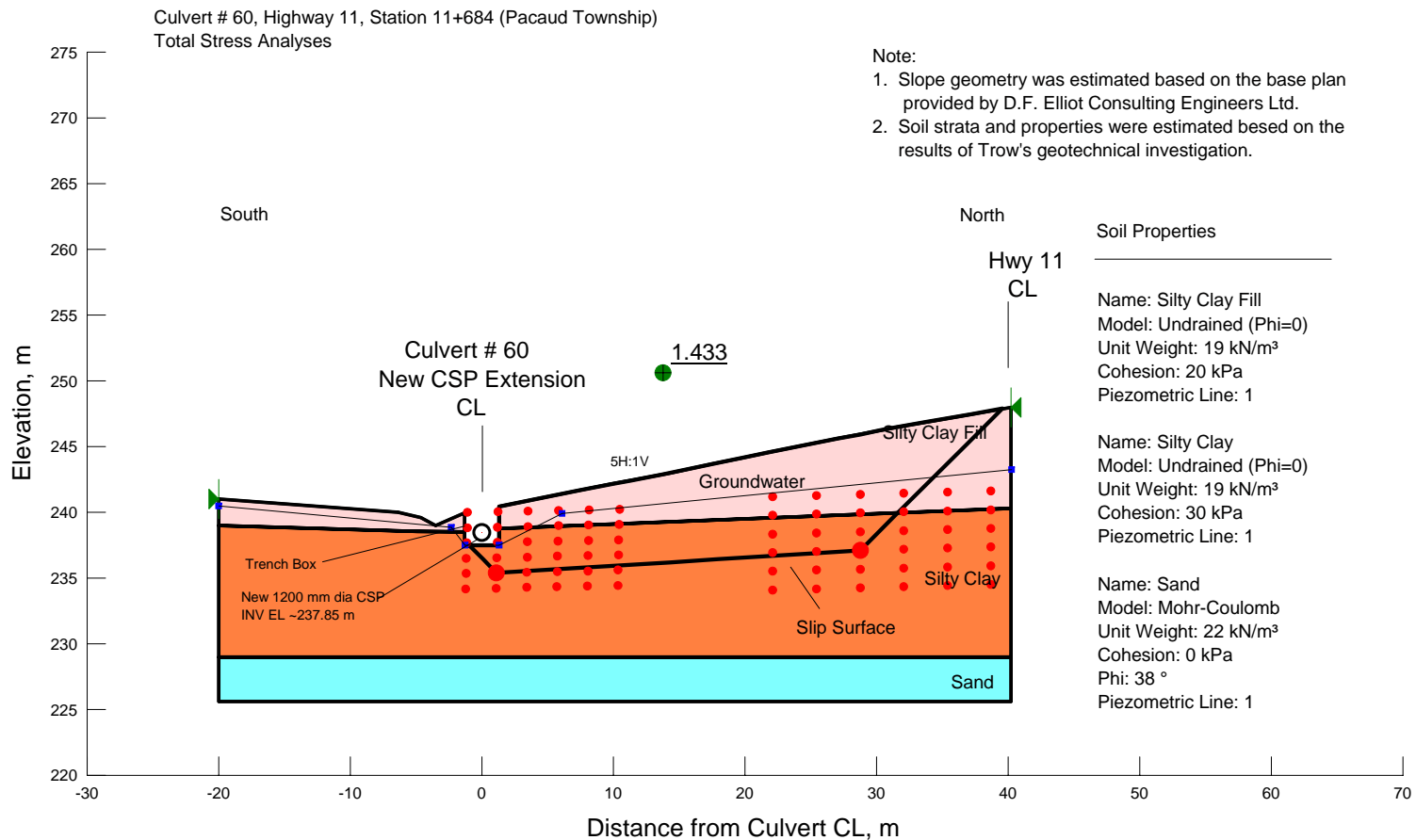


Figure 11(d). REMEDIATION; Excavation for the failed culvert extension replacement using the support system and slope flattening. Slope stability analysis was performed assuming total stress parameters; silty clay fill $C_u=20$ kPa and silty clay $C_u=30$ kPa, and non-circular slip surface.



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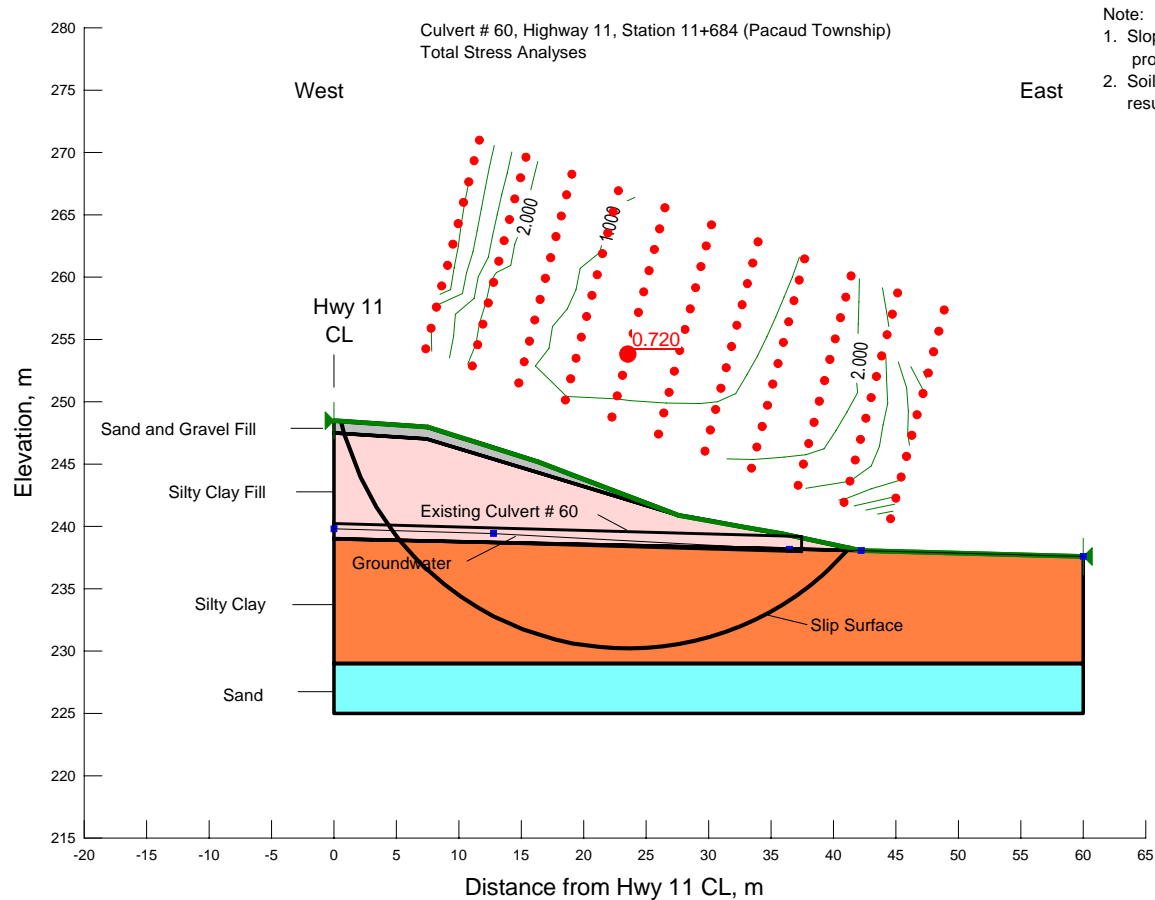
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2. Soil strata and properties were estimated based on the results of Trow's geotechnical investigation.

Soil Properties

Name: Sand and Gravel Fill
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 0 kPa
Phi: 40 °
Piezometric Line: 1

Name: Silty Clay Fill
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 20 kPa
Piezometric Line: 1

Name: Silty Clay
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 20 kPa
Piezometric Line: 1

Name: Sand
Model: Mohr-Coulomb
Unit Weight: 22 kN/m³
Cohesion: 0 kPa
Phi: 38 °
Piezometric Line: 1

Figure 12(a). Slope stability analyses for the HWY 11 embankment slope before excavation of trench assuming total stress parameters; silty clay fill Cu=20 kPa and silty clay Cu=20 kPa.



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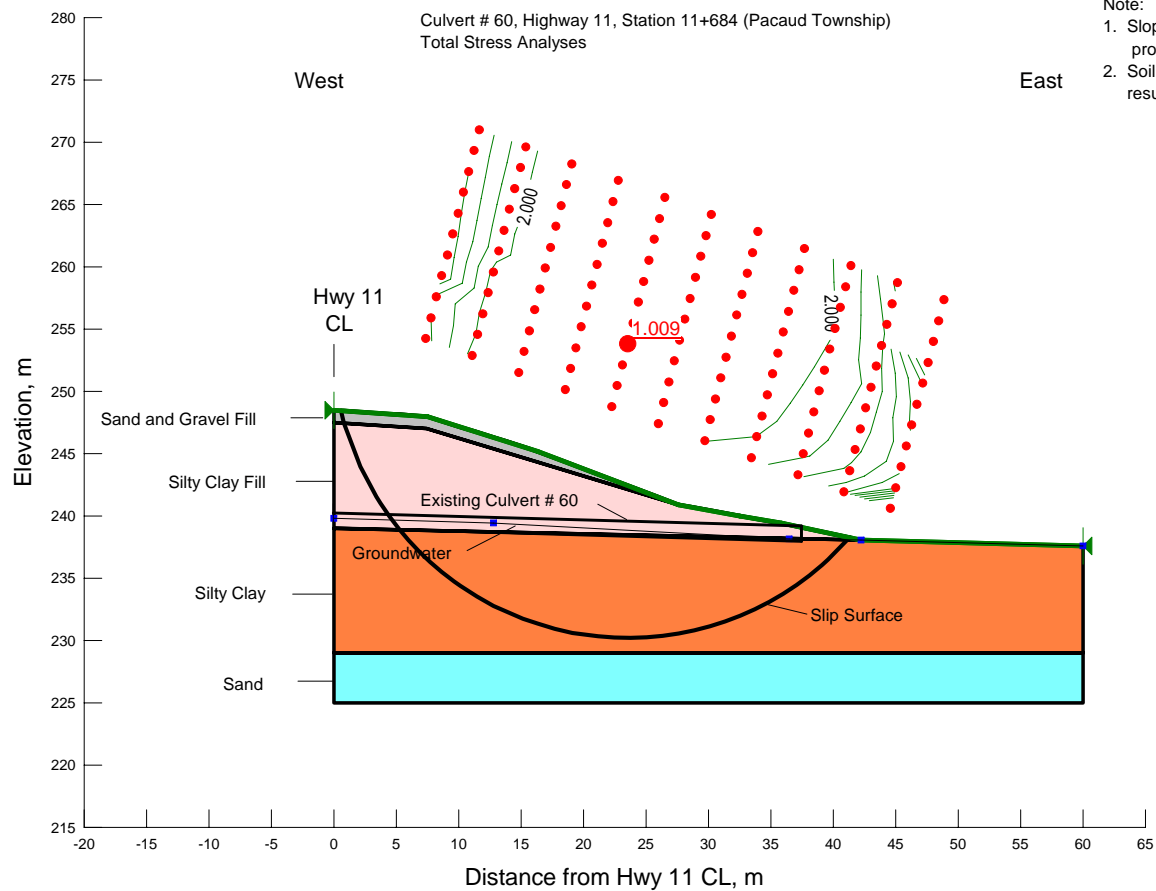
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Soil Properties

Name: Sand and Gravel Fill
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 0 kPa
Phi: 40 °
Piezometric Line: 1

Name: Silty Clay Fill
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 20 kPa
Piezometric Line: 1

Name: Silty Clay
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 30 kPa
Piezometric Line: 1

Name: Sand
Model: Mohr-Coulomb
Unit Weight: 22 kN/m³
Cohesion: 0 kPa
Phi: 38 °
Piezometric Line: 1

Figure 12(b). Slope stability analyses for the HWY 11 embankment slope before excavation of trench assuming total stress parameters; silty clay fill $C_u=20$ kPa and silty clay $C_u=30$ kPa.



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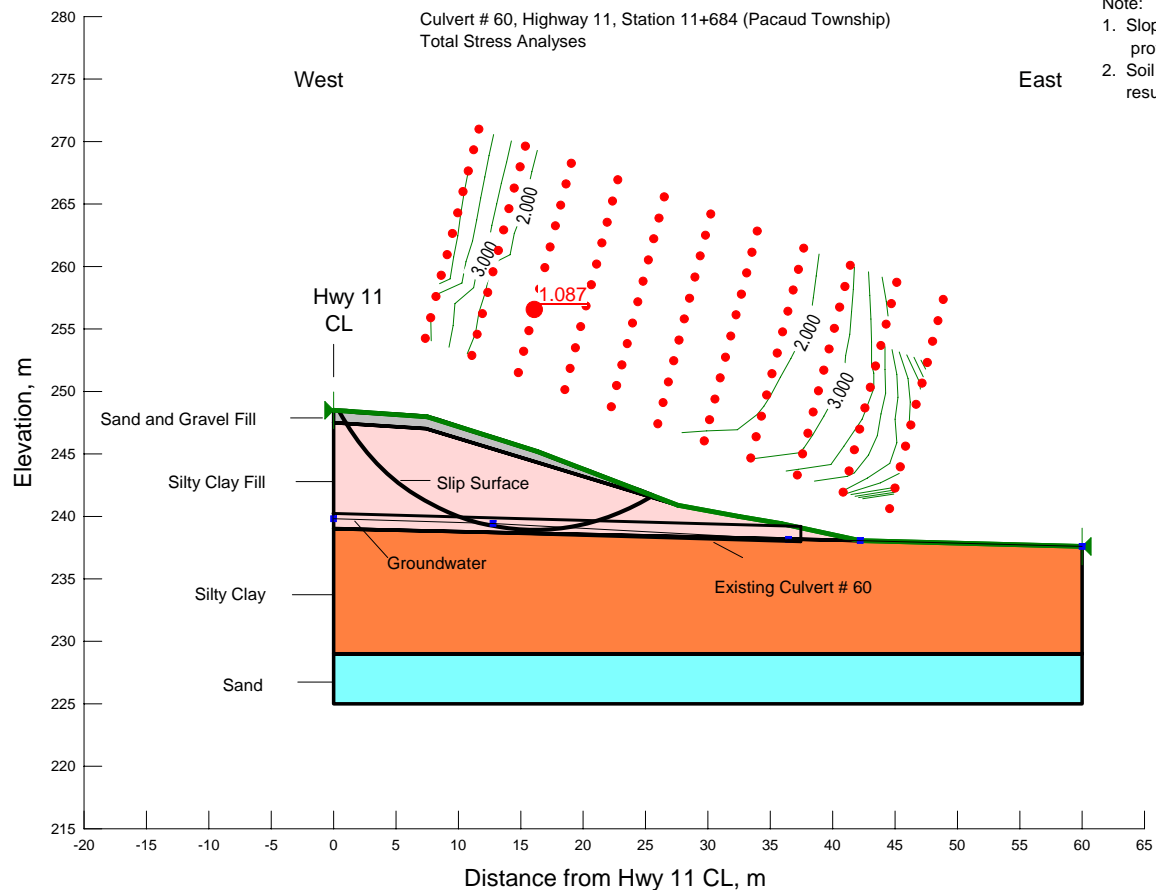
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Soil Properties

Name: Sand and Gravel Fill
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 0 kPa
Phi: 40 °
Piezometric Line: 1

Name: Silty Clay Fill
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 20 kPa
Piezometric Line: 1

Name: Silty Clay
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 35 kPa
Piezometric Line: 1

Name: Sand
Model: Mohr-Coulomb
Unit Weight: 22 kN/m³
Cohesion: 0 kPa
Phi: 38 °
Piezometric Line: 1

Figure 12(c). Slope stability analyses for the HWY 11 embankment slope before excavation of trench assuming total stress parameters; silty clay fill $C_u=20$ kPa and silty clay $C_u=35$ kPa.



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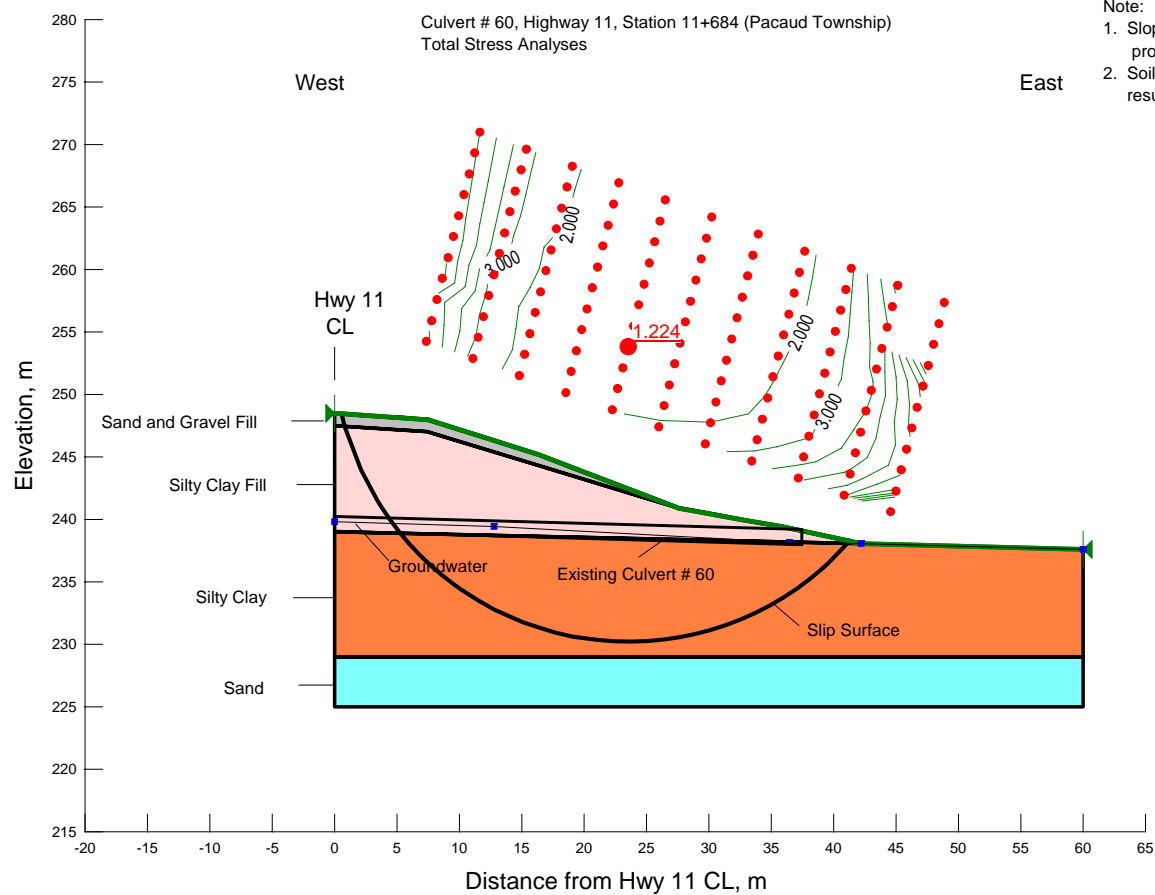
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Soil Properties

Name: Sand and Gravel Fill
 Model: Mohr-Coulomb
 Unit Weight: 21 kN/m³
 Cohesion: 0 kPa
 Phi: 40 °
 Piezometric Line: 1

Name: Silty Clay Fill
 Model: Undrained (Phi=0)
 Unit Weight: 19 kN/m³
 Cohesion: 30 kPa
 Piezometric Line: 1

Name: Silty Clay
 Model: Undrained (Phi=0)
 Unit Weight: 19 kN/m³
 Cohesion: 35 kPa
 Piezometric Line: 1

Name: Sand
 Model: Mohr-Coulomb
 Unit Weight: 22 kN/m³
 Cohesion: 0 kPa
 Phi: 38 °
 Piezometric Line: 1

Figure 12 (d). Slope stability analyses for the HWY 11 embankment slope before excavation of trench assuming total stress parameters; silty clay fill $C_u=30$ kPa and silty clay $C_u=35$ kPa.



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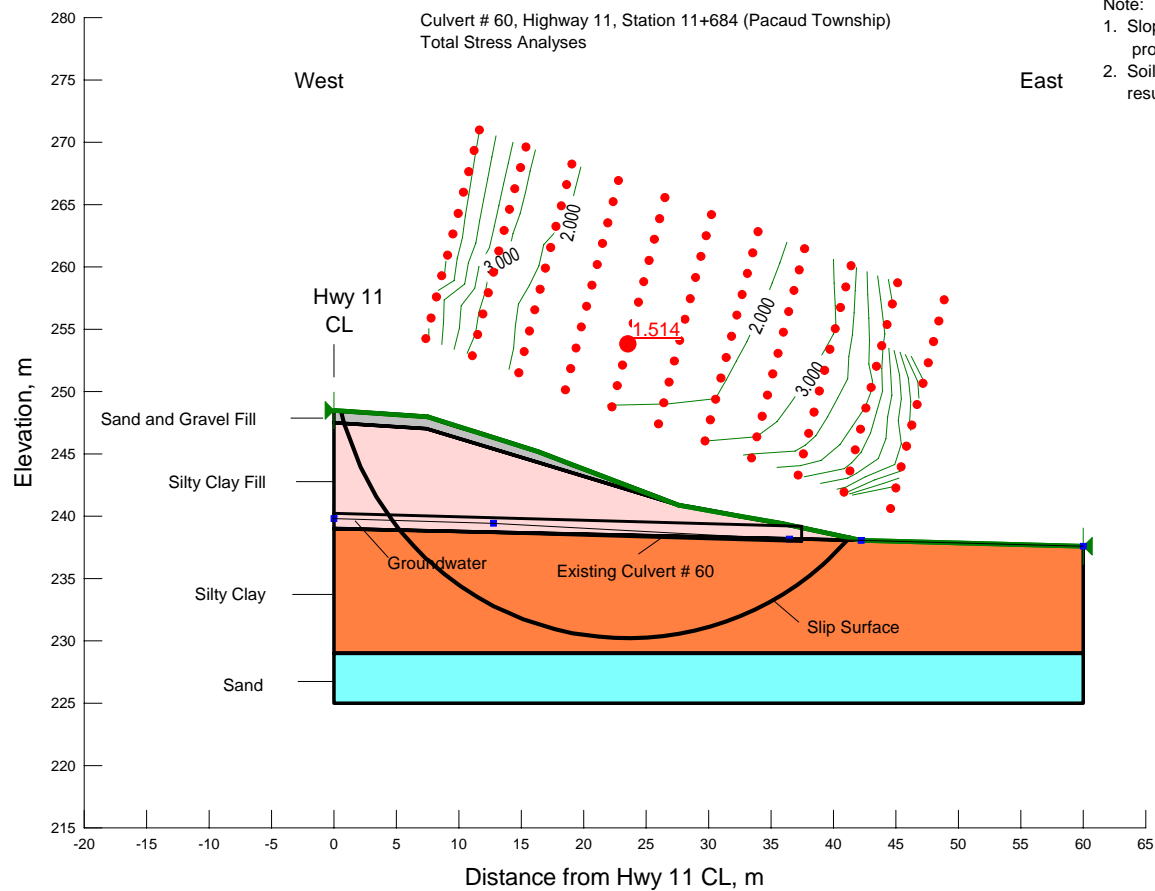
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Soil Properties

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Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 0 kPa
Phi: 40 °
Piezometric Line: 1

Name: Silty Clay Fill
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 30 kPa
Piezometric Line: 1

Name: Silty Clay
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 45 kPa
Piezometric Line: 1

Name: Sand
Model: Mohr-Coulomb
Unit Weight: 22 kN/m³
Cohesion: 0 kPa
Phi: 38 °
Piezometric Line: 1

Figure 12(e). Slope stability analyses for the HWY 11 embankment slope before excavation of trench assuming total stress parameters; silty clay fill $C_u=30$ kPa and silty clay $C_u=45$ kPa.



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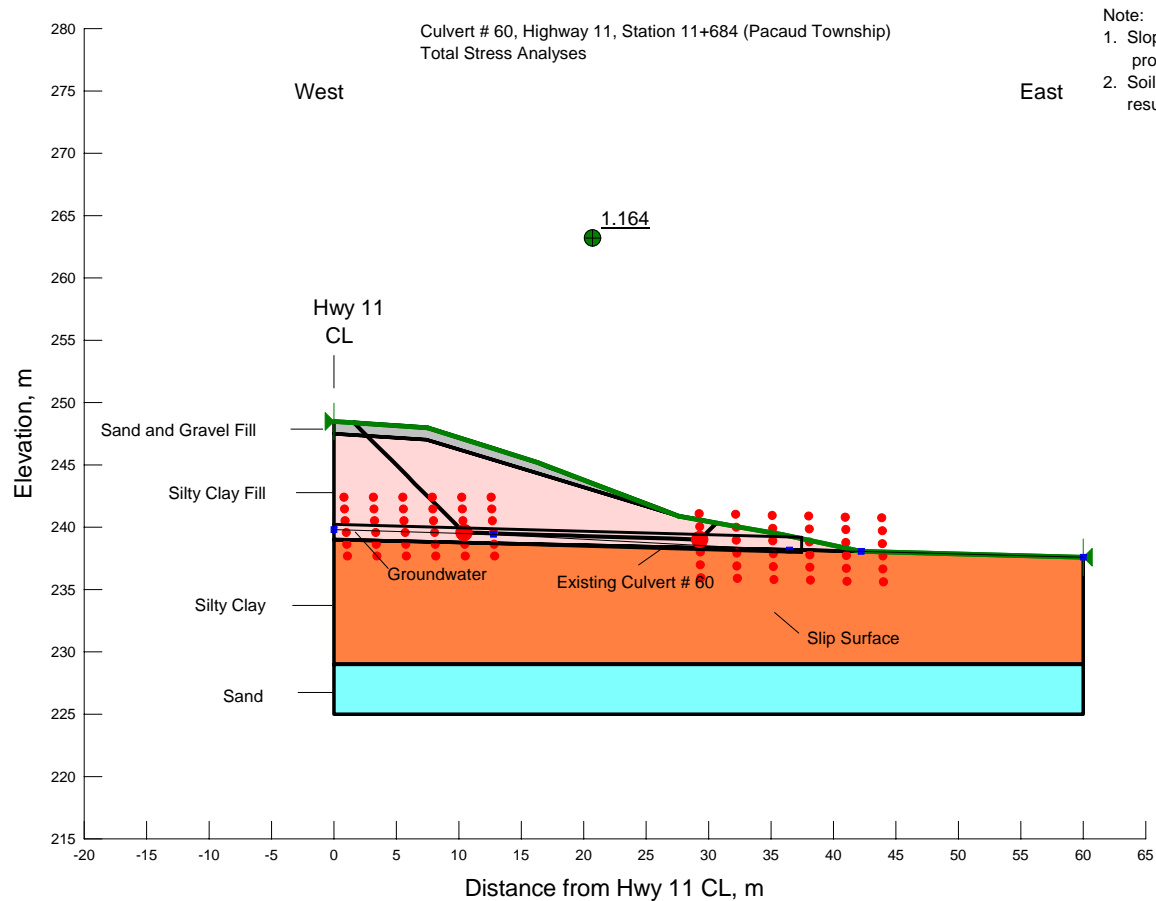
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Soil Properties

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Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 0 kPa
Phi: 40 °
Piezometric Line: 1

Name: Silty Clay Fill
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 20 kPa
Piezometric Line: 1

Name: Silty Clay
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 35 kPa
Piezometric Line: 1

Name: Sand
Model: Mohr-Coulomb
Unit Weight: 22 kN/m³
Cohesion: 0 kPa
Phi: 38 °
Piezometric Line: 1

Figure 13. Slope stability analyses for the HWY 11 embankment slope before excavation of trench assuming total stress parameters; silty clay fill $C_u=30$ kPa and silty clay $C_u=45$ kPa, and non-circular slip surface.



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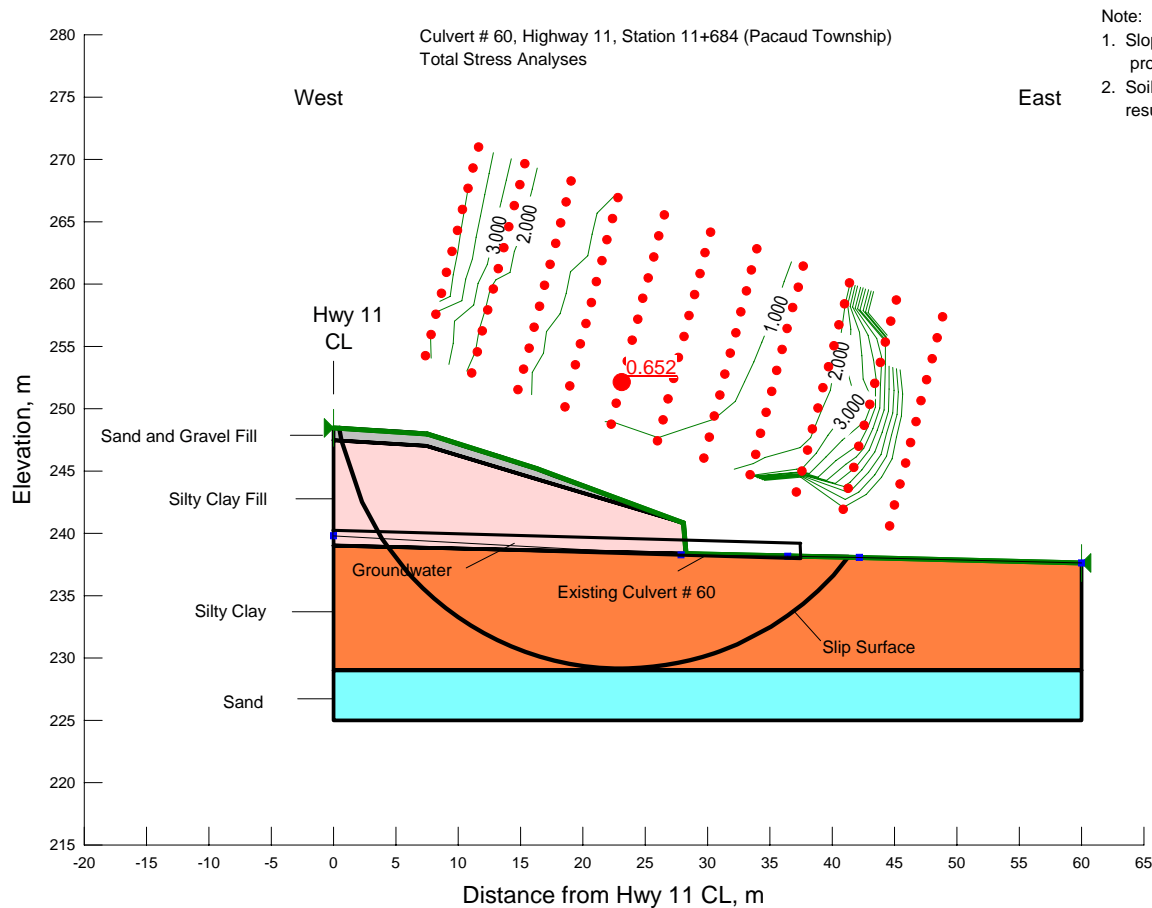
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Soil Properties

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Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 0 kPa
Phi: 40 °
Piezometric Line: 1

Name: Silty Clay Fill
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 20 kPa
Piezometric Line: 1

Name: Silty Clay
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 20 kPa
Piezometric Line: 1

Name: Sand
Model: Mohr-Coulomb
Unit Weight: 22 kN/m³
Cohesion: 0 kPa
Phi: 38 °
Piezometric Line: 1

Figure 14(a). REMEDIATION; Excavation for the failed culvert extension replacement using the support system and slope flattening. Slope stability analyses for HWY 11 embankment slope after excavation of trench assuming total stress parameters; silty clay fill $C_u=20$ kPa and silty clay $C_u=20$ kPa.



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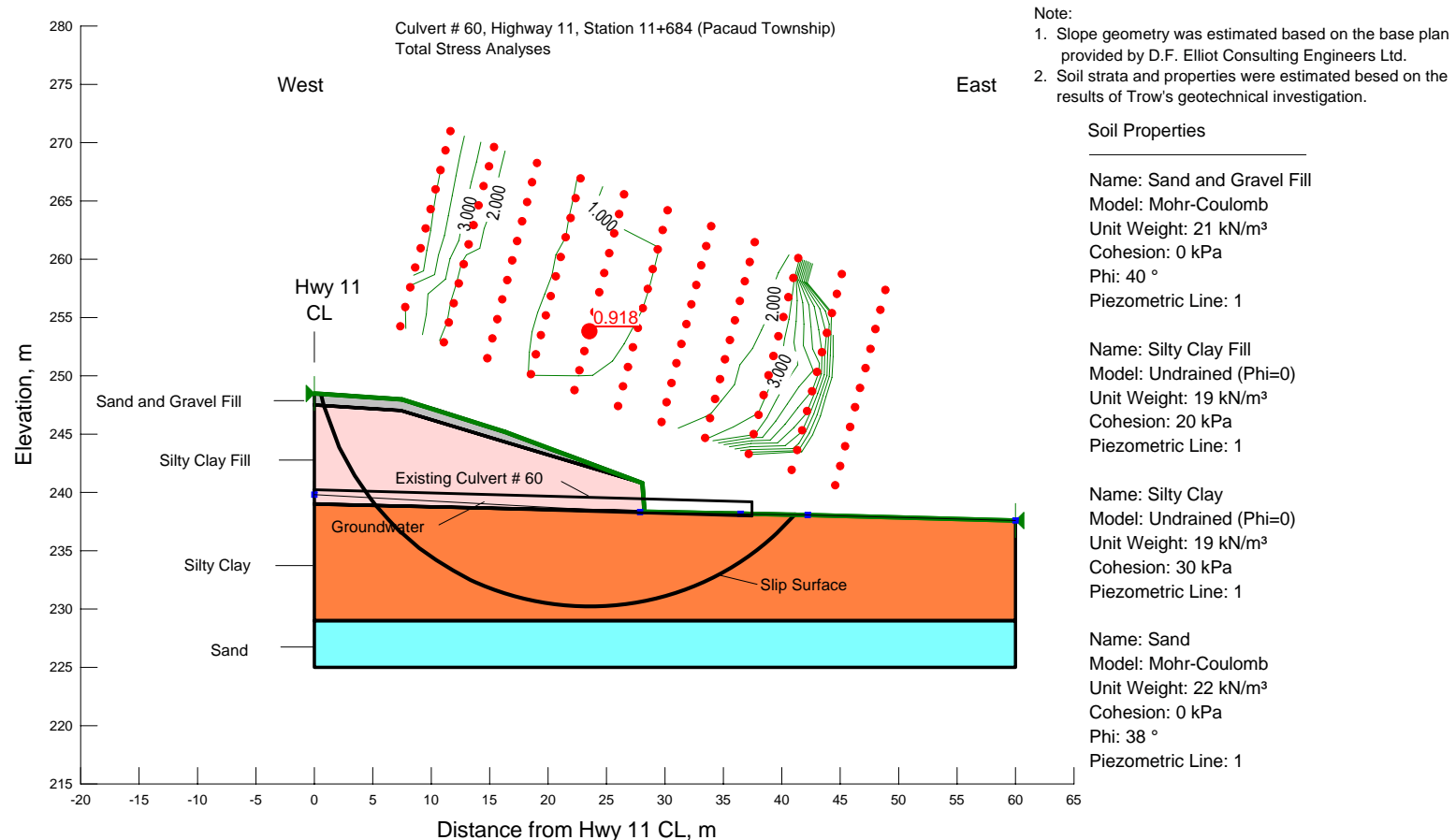


Figure 14 (b). REMEDIATION; Excavation for the failed culvert extension replacement using the support system and slope flattening. Slope stability analyses for HWY 11 embankment slope after excavation of trench assuming total stress parameters; silty clay fill $C_u=20$ kPa and silty clay $C_u=30$ kPa.



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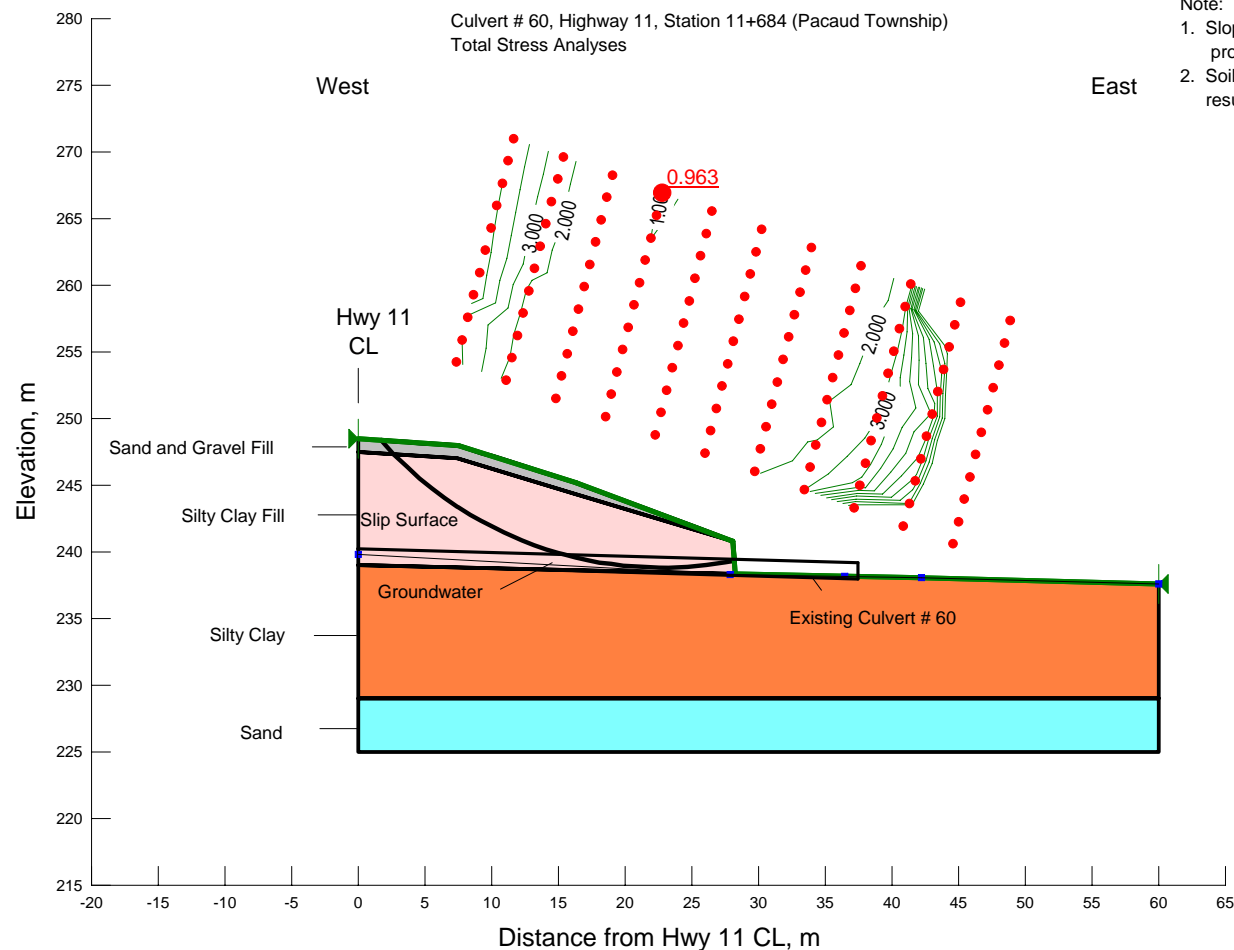
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Model: Mohr-Coulomb
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Cohesion: 0 kPa
Phi: 40 °
Piezometric Line: 1

Name: Silty Clay Fill
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 20 kPa
Piezometric Line: 1

Name: Silty Clay
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 35 kPa
Piezometric Line: 1

Name: Sand
Model: Mohr-Coulomb
Unit Weight: 22 kN/m³
Cohesion: 0 kPa
Phi: 38 °
Piezometric Line: 1

Figure 14(c). REMEDIATION; Excavation for the failed culvert extension replacement using the support system and slope flattening. Slope stability analyses for HWY 11 embankment slope after excavation of trench assuming total stress parameters; silty clay fill $C_u=20$ kPa and silty clay $C_u=35$ kPa.



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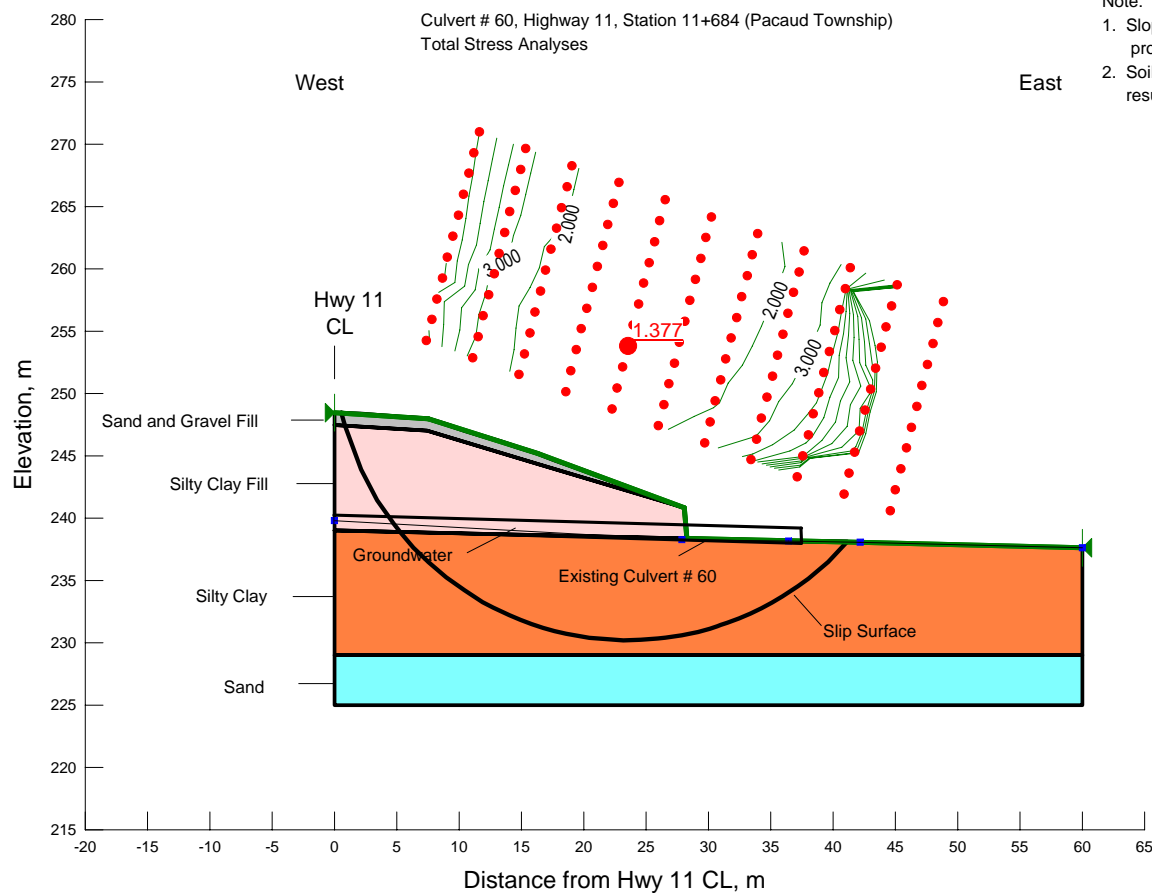
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Soil Properties

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Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 0 kPa
Phi: 40 °
Piezometric Line: 1

Name: Silty Clay Fill
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 30 kPa
Piezometric Line: 1

Name: Silty Clay
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 45 kPa
Piezometric Line: 1

Name: Sand
Model: Mohr-Coulomb
Unit Weight: 22 kN/m³
Cohesion: 0 kPa
Phi: 38 °
Piezometric Line: 1

Figure 14(e). REMEDIATION; Excavation for the failed culvert extension replacement using the support system and slope flattening. Slope stability analyses for HWY 11 embankment slope after excavation of trench assuming total stress parameters; silty clay fill $C_u=30$ kPa and silty clay $C_u=45$ kPa.



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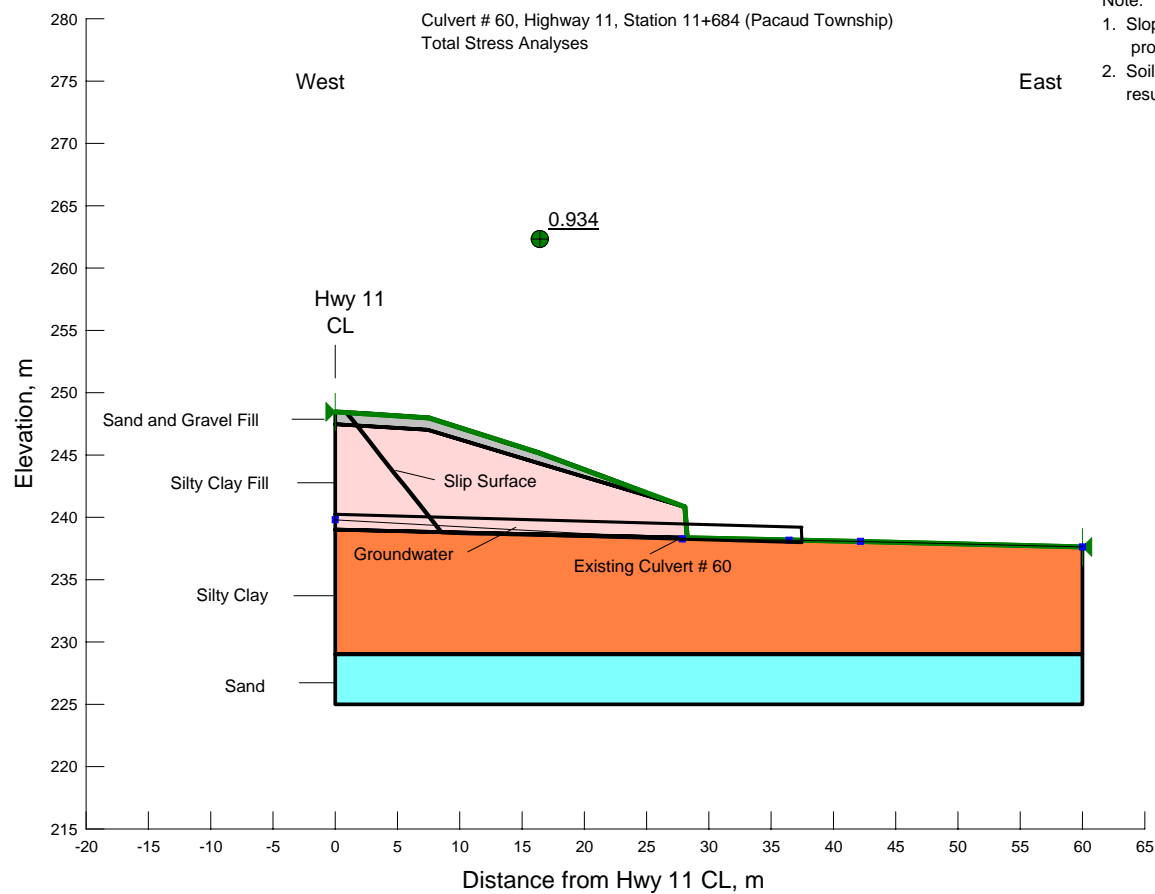
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Soil Properties

Name: Sand and Gravel Fill
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 0 kPa
Phi: 40 °
Piezometric Line: 1

Name: Silty Clay Fill
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 20 kPa
Piezometric Line: 1

Name: Silty Clay
Model: Undrained (Phi=0)
Unit Weight: 19 kN/m³
Cohesion: 35 kPa
Piezometric Line: 1

Name: Sand
Model: Mohr-Coulomb
Unit Weight: 22 kN/m³
Cohesion: 0 kPa
Phi: 38 °
Piezometric Line: 1

Figure 15(a). REMEDIATION; Excavation for the failed culvert extension replacement using the support system and slope flattening. Slope stability analyses for HWY 11 embankment slope after excavation of trench assuming total stress parameters; silty clay fill $C_u=20$ kPa and silty clay $C_u=35$ kPa, and non-circular slip surface.



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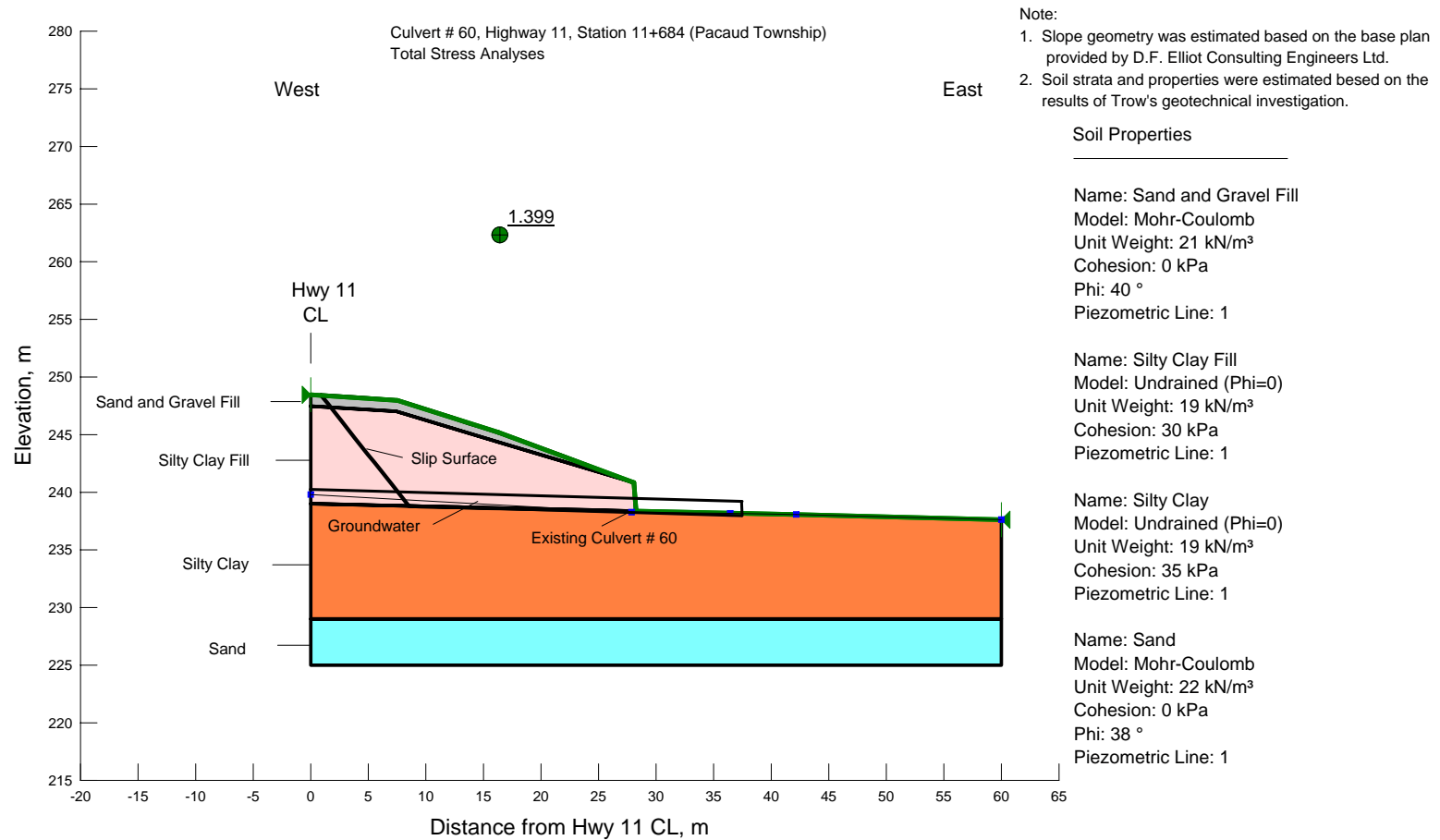


Figure 15(b). REMEDIATION; Excavation for the failed culvert extension replacement using the support system and slope flattening. Slope stability analyses for HWY 11 embankment slope after excavation of trench assuming total stress parameters; silty clay fill $C_u=30$ kPa and silty clay $C_u=35$ kPa, and non-circular slip surface.



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