THURBER ENGINEERING LTD.

FOUNDATION INVESTIGATION AND DESIGN REPORT
STRAWBERRY CREEK BRIDGE #8 REPLACEMENT
WARE ROAD NORTH, DISTRICT OF THUNDER BAY, WARE TOWNSHIP, ONTARIO
LATITUDE: 48.604697°, LONGITUDE: -89.471511°
G.W.P. 6014-18-00, SITE No. 48W-054

GEOCRES Number: 52A-250

Report

to

Ministry of Transportation Ontario

Date: June 5, 2020
File: 27323

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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

This report presents the factual data obtained from a foundation investigation carried out by Thurber Engineering Ltd. (Thurber) for the proposed replacement of the Strawberry Creek #8 Bridge, located on Ware Road North, in Ware Township, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the existing Strawberry Creek #8 bridge site, and based on the data obtained, to provide a borehole location plan, stratigraphic profile, records of boreholes, laboratory test results, and a written description of the subsurface conditions.

Thurber was retained by the Ministry of Transportation (MTO), Northwest Region, to carry out this foundation investigation under the MTO Agreement Number 6017-E-0062, Assignment #2.

2. SITE DESCRIPTION

The site is located on Ware Road North, approximately 3.2 km north of Auto Road, in Ware Township, Thunder Bay District, Ontario. The existing bridge allows Strawberry Creek #8 to flow under Ware Road North in a general west to east direction. Ware Road North runs in a general north-south direction at the bridge site. Ware Road North is a two-lane gravel road that narrows to single lane across the existing bridge. The immediate approaches to the bridge deck are paved with asphalt.

The Ontario Structure Inspection Manual (OSIM) report prepared by MTO on July 18, 2018 indicates that the existing structure is a single-span, wooden rectangular beam / girder bridge with a wooden deck. The inspection report indicates that the bridge has a span of 6.3 m and the structure is approximately 5.5 m wide. Based on a November 2019 survey plan of the site provided by MTO (Plan E-1019-0-7), the ground surface elevation of Ware Road North at the existing bridge is approximately Elevation 366.3 m. The existing bridge is supported on timber sitting on ballast walls. The OSIM report notes that the bridge has a very low clearance. Measurements collected in October 2019 indicate that the surface water level of the creek varies from approximate Elevation 365.4 to 365.2 m from upstream to downstream of the bridge.

The lands surrounding the bridge site are predominantly heavily forested, with mature trees, grass and shrubs. The area along the creek floodplain near the bridge also contains marshy conditions. Photographs of the bridge and surrounding area are presented in Appendix C.

Based on published geological information, the bridge lies within an area consisting of glaciolacustrine plains of silt and clay, surrounded by bedrock knobs, with some sand and gravel pits in the general vicinity. Based on local geological maps, the bedrock in the area consists of metasedimentary rocks and minor metavolcanic rocks.

3. INVESTIGATION PROCEDURES

The field investigation for the replacement bridge was carried out between November 4 and 9, 2019 and consisted of drilling and sampling six (6) boreholes, labeled SC8-01 to SC8-06. The boreholes were drilled to depths ranging from 7.2 to 18.3 m (Elevation 359.1 to 348.0 m). Boreholes SC8-01 and SC8-02 were drilled near the north abutment of the bridge and Boreholes SC8-03 and SC8-04 were drilled near the south abutment. Boreholes SC8-05 and SC8-06 were drilled at the north and south approaches to the bridge respectively. The boreholes were drilled through either the paved bridge approaches or the granular road surface. Flowing artesian groundwater pressure of greater than 3 m above the ground surface were encountered when penetrating the thick native silty clay deposit in Borehole SC8-01 near the north abutment. Due to the difficulty in sealing the artesian pressure in this borehole and following discussion with MTO, the remaining boreholes were terminated near the base of the silty clay deposit to avoid encountering additional flowing artesian conditions.

The borehole logs are included in Appendix A and the approximate locations of the boreholes are shown on the Borehole Locations and Soil Strata drawings included in Appendix D.

Utility clearances were obtained prior to the start of drilling. The ground surface elevations for the boreholes were estimated from field measurements and the survey drawing provided to Thurber by MTO. The coordinate system MTM NAD 83, Zone 15 was used for the boreholes.

A rubber tracked CME 55 drill rig was used to advance the boreholes using hollow stem augers and NW casing. Soil samples were obtained in the boreholes at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) as well as thin-walled tube samples. A Dynamic Cone Penetration Test (DCPT) was conducted at the base of Borehole SC8-04. NQ coring methods were used to advance Borehole SC8-01 into bedrock.

The drilling and sampling operations were supervised on a full-time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's laboratory for further examination and testing.

The rock core was logged, and the Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

The boreholes were decommissioned in general accordance with Ontario Regulation 903 as amended upon completion of drilling.

Completion details of the boreholes are summarized in Table 3.1.

Table 3.1 – Borehole Completion Details

Borehole Number	Borehole Depth / Base Elevation (m)	Completion Details
SC8-01	18.3 / 348.0	Artesian groundwater pressure to greater than 3 m above the ground surface. Sealed pressure and backfilled borehole with mix of Portland cement, bentonite grout and concrete to surface.
SC8-02	8.7 / 357.7	Backfilled with bentonite from 8.7 to 0.3 m, then cement to surface.
SC8-03	7.2 / 359.1	Backfilled with bentonite from 7.2 to 0.3 m, then cement to surface.
SC8-04	10.4 / 356.0	Backfilled with bentonite from 10.4 to 0.3 m, then cement to surface.
SC8-05	11.3 / 354.9	Backfilled with bentonite from 11.3 to 0.3 m, then cement to surface.
SC8-06	8.2 / 358.0	Backfilled with bentonite from 8.2 to 0.3 m, then cement to surface.

4. LABORATORY TESTING

All recovered soil samples were subjected to visual identification (VI) and to natural moisture content determination. Selected samples were also subjected to grain size distribution analyses (sieve and/or hydrometer) and Atterberg Limits tests, where appropriate. One-dimensional consolidation tests were also conducted on two samples of the silty clay. Point load tests were conducted on the rock core. The results of this laboratory testing program are shown on the Record of Borehole sheets included in Appendix A and on the figures included in Appendix B.

In order to assess the potential for sulphate attack on concrete foundations, as well as the potential for corrosion associated with the structure, a sample of the fill soil from Borehole SC8-04 near the south abutment and a sample of the native silty clay soil from Borehole SC8-02 near the north abutment were selected for testing. A surface water sample from the creek on the upstream side of the bridge was also collected. The soil and water samples were submitted to Bureau Veritas Laboratories, a CALA accredited analytical laboratory in Mississauga, Ontario, for analytical testing of corrosivity parameters and sulphate content. The results of the analytical testing are summarized in Section 6 and are presented in Appendix B.

5. SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets and on the Borehole Locations and Soil Strata Drawing included in Appendix D. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following paragraphs. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description and must be used for interpretation of the site conditions. It must be recognized and expected that soil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions encountered at the boreholes consisted of gravelly sand to silty sand fill overlying a thick native deposit of firm to stiff silty clay, overlying compact silt and sand, and very dense gravelly sand, which are underlain by granodiorite bedrock. A buried organic layer was also encountered below the fill in the boreholes on the south side of the bridge, and the upper part of the silty clay was noted to contain organic pockets. Descriptions of the individual strata are presented below.

5.1 Asphalt

A 50 mm thick layer of asphalt was encountered at the ground surface in Boreholes SC8-01 and SC8-03, which were drilled on the edge of the paved bridge approaches.

5.2 Gravelly Sand to Silty Sand Fill

A layer of fill ranging from gravelly sand, silty with trace clay to silty sand, trace to some gravel, trace clay was encountered at the ground surface or below the asphalt in all of the boreholes. The fill extended to depths ranging from 0.7 to 1.4 m (Elev. 365.6 to 364.9 m). The fill was loose to very dense, with SPT ‘N’ values ranging from 5 to greater than 100 blows for 0.3 m of penetration. The measured moisture content in the fill ranged from 4 to 17%.

The results of grain size analyses conducted on samples of the gravely sand fill are illustrated on Figure B1 in Appendix B. The results are summarized as follows:

Soil Particle	Percentage (%)
Gravel	22 to 27
Sand	49 to 52
Silt	21 to 22
Clay	2 to 3
Silt and Clay	26

5.3 Organics

A 0.4 to 0.7 m thick layer of organic soil containing some peat, some sand, trace gravel, and occasional rootlets and wood fragments was encountered below the fill in Boreholes SC8-04 and SC8-06 on the south side of the bridge. The organic layer extended to a depth from 1.4 to 1.8 m (Elev. 364.8 m). The organic soil was very soft, based on SPT ‘N’ values of 2 blows per 0.3 m of penetration. The measured moisture content of the organic soil ranged from 134 to 161%.

5.4 Silty Clay with Organics

A layer of silty clay containing organic pockets and wood fragments was encountered below the fill or organic soil in Boreholes SC8-01 to SC8-04 and SC8-06. The silty clay with organics also contained trace to some sand and trace gravel. The silty clay with organics layer was 0.8 to 1.6 m thick and extended to depths ranging from 2.1 to 3.0 m (Elev. 364.2 to 363.3 m).

The silty clay with organics layer was very soft to very stiff, with SPT ‘N’ values ranging from 0 (weight of hammer) to 30 blows for 0.3 m of penetration, but typically soft. The measured moisture content ranged from 34 to 101%, but typically greater than 40%.

The results of grain size analyses and Atterberg Limit tests conducted on samples of the silty clay with organics are illustrated on Figures B2 and B7 in Appendix B. The results are summarized as follows:

Soil Particle	Percentage (%)
Gravel	0 to 1
Sand	9 to 18
Silt	28 to 43
Clay	48 to 53

Soil Property	Percentage (%)
Liquid Limit	70
Plastic Limit	39
Plasticity Index	31

The results of the Atterberg Limit test indicate that the silty clay is organic with high plasticity, with group symbol OH.

5.5 Silty Clay

A relatively thick deposit of silty clay with trace sand was encountered below the fill and silty clay with organics layers in all of the boreholes. The silty clay ranged to clay with trace to some silt with depth. Where fully penetrated, the silty clay deposit ranged in thickness from 5.0 to 9.0 m and extended to depths from 7.2 to 10.2 m (Elev. 359.1 to 355.9 m). Boreholes SC8-02 and SC8-03 were terminated in the silty clay at depths from 7.2 to 8.7 m (Elev. 359.1 to 357.7 m).

SPT 'N' values recorded in the silty clay ranged from 0 (weight of hammer) to 6 blows per 0.3 m of penetration. In-situ vane shear tests conducted in the silty clay measured undrained shear strengths ranging from 46 to 92 kPa, indicating that the silty clay ranges from firm to stiff. The sensitivity of the silty clay ranged from 1.7 to 3.0, indicating a low to medium sensitivity. The measured moisture content of the silty clay ranged from 36 to 69%, but typically greater than 50%.

The results of grain size analysis and Atterberg Limit tests conducted on samples of the silty clay are provided on the Record of Borehole sheets in Appendix A, and illustrated on Figures B3, B4, B7 and B8 in Appendix B. The results are summarized as follows:

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Soil Particle	Percentage (%)
Gravel	0
Sand	0 to 4
Silt	7 to 29
Clay	70 to 93

Soil Property	Percentage (%)
Liquid Limit	62 to 78
Plastic Limit	23 to 30
Plasticity Index	35 to 50

The results of the Atterberg Limits tests indicate that the silty clay has high plasticity with group symbol CH.

Consolidation tests were performed on two samples of the silty clay (thin walled tube samples), which were collected from Boreholes SC8-02 and SC8-04. The results of the testing are presented in Appendix B and are summarized in the following table.

Borehole	Sample Depth (m)	e_o	C_c	C_r	p_c' (kPa)	p_o' (kPa)	OCR	c_v (m^2/year)	c_{vr} (m^2/year)
SC8-02	6.1 to 6.7	1.982	0.736	0.080	125	51	2.4	1.3 - 1.8	2.6 - 3.0
SC8-04	3.4 to 4.0	1.433	0.656	0.077	220	39	5.7	4.9 - 10.4	21.9 - 28.9

5.6 Silt to Silt and Sand

A deposit of native silt to silt and sand was encountered below the silty clay where it was penetrated in Boreholes SC8-01 and SC8-04 to SC8-06. The thickness of the silt and sand was 3.0 m in Borehole SC8-01, where it extended to a depth 11.7 m (Elev. 354.6 m). A DCPT was conducted in the silt and sand at the base of Borehole SC8-04, and refusal of 100 blows per 0.3 m of penetration was encountered at a depth of 10.4 m (Elev. 356.0 m). Boreholes SC8-05 and SC8-06 were terminated within the silt to silt and sand at depths from 8.2 to 11.3 m (Elev. 358.0 to 354.9 m). Artesian groundwater pressure was encountered within the silt to silt and sand deposit as described in Section 5.9 below.

SPT ‘N’ values measured in the silt to silt and sand deposit ranged from 9 to 17 blows for 0.3 m of penetration, indicating that the deposit is compact. The measured moisture contents in the deposit ranged from 17 to 23%.

The results of grain size analyses conducted on samples of the silt to silt and sand deposit are provided on the Record of Borehole sheets in Appendix A, and illustrated on Figure B5 in Appendix B. The results are summarized as follows:

Soil Particle	Percentage (%)
Gravel	0
Sand	11 to 47
Silt	49 to 82
Clay	4 to 7

5.7 Gravelly Sand

A deposit of gravelly sand that was silty with occasional cobbles and boulders was encountered below the silt and sand in Borehole SC8-01. The gravelly sand layer was 5.0 m thick and extended to a depth of 16.7 m (Elev. 349.6 m), where it was underlain by bedrock. NQ coring methods were required to penetrate cobbles and boulders within this deposit. Four (4) boulders ranging in thickness from 200 to 280 mm were cored through. Photos of the cobbles and boulders that were retrieved from the coring process are shown as Runs #1 to #3 on Photo B1 in Appendix B. Artesian groundwater pressure was encountered within the gravelly sand deposit and measured at a height of greater than 3.0 m above the ground surface, as described in Section 5.9 below.

The SPT ‘N’ values recorded in the gravelly sand ranged from 29 to greater than 100 blows for 0.3 m of penetration, indicating that the deposit is compact to very dense. The measured moisture content ranged from 2 to 12%.

The results of a grain size analysis conducted on a sample of the gravelly sand deposit are provided on the Record of Borehole sheets in Appendix A, and illustrated on Figure B6 in Appendix B. The results are summarized as follows:

Soil Particle	Percentage (%)
Gravel	33
Sand	47
Silt and Clay	20

5.8 Bedrock

The overburden soils described above are underlain by bedrock, which was encountered at a depth of 16.7 m (Elev. 349.6 m) in Borehole SC8-01 near the north abutment. The bedrock was described as granodiorite, grey in colour and slightly weathered. The bedrock was cored 1.6 to a depth of 18.3 m (Elev. 348.0 m) before the borehole was terminated due to high artesian pressure. A photograph of the rock core is included as Run #4 on Photo B1 in Appendix B.

Total Core Recovery (TCR) in the bedrock ranged was 100%, and Solid Core Recovery (SCR) was 95%. The Rock Quality Designation (RQD) determined from the recovered core was 40%, which indicates poor rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 1 to 5, with one broken zone with FI of greater than 10.

The average unconfined compressive strength (UCS) of the rock was 170 MPa, indicating the rock is very strong. The estimated rock strength value is interpreted from point load tests that were conducted on the rock core. A summary of the point load tests results are presented in Appendix B.

5.9 Groundwater Conditions

Groundwater conditions were observed during drilling operations.

Artesian groundwater pressure was encountered within the silt to silt and sand and the gravelly sand deposits. In Borehole SC8-01, the height of the artesian pressure was measured at greater than 3.0 m above the ground surface. The artesian pressure in Boreholes SC8-04 to SC8-06 was sealed before measurements could be taken in order to avoid difficulty backfilling these boreholes. In general, it is anticipated that the source of the artesian groundwater pressure is within the cohesionless deposits below the silty clay, at approximate depths from 7.2 to 10.2 m (Elev. 359.1 to 355.9 m).

The groundwater level near the surface should also be anticipated to reflect the local creek water level. The water level in the creek was measured in October 2019 at Elevation 365.4 to 365.2 m from upstream to downstream of the bridge.

Groundwater levels are short-term observations and seasonal fluctuations of the groundwater levels are to be expected. In particular, the groundwater levels may be at a higher elevation during spring and after periods of significant or prolonged precipitation.

6. CORROSIVITY AND SULPHATE TEST RESULTS

A sample of the fill soil from Borehole SC8-04 and a sample of the native silty clay soil from Borehole SC8-02, as well as a surface water sample from the creek on the upstream side of the bridge were submitted for analytical testing of corrosivity parameters and sulphate. The results of the analytical tests are shown in Table 6.1. The laboratory certificates of analysis are presented in Appendix B.

Table 6.1 - Analytical Test Results

Parameter	Units (soil)	Units (water)	Test Results		
			SC8-02, SS#3, 1.5 to 2.1 m	SC8-04, SS#1, 0.0 to 0.6 m	SC8 Strawberry Creek #8 Bridge
			Silty Clay	Native Silty Clay	Creek Water
Sulphide	%	mg/L	<0.5	<0.5	<0.02
Chloride	µg/g	mg/L	40	28	<10
Sulphate	µg/g	mg/L	<20	330	<10
pH	no unit	no unit	6.58	10.0	9.34
Conductivity	umho/cm	umho/cm	187	488	160
Resistivity	ohm.cm	ohm.cm	5400	2000	6400
Redox Potential	mV	mV	238.2	144.5	186.6

7. MISCELLANEOUS

Thurber obtained subsurface utility clearances prior to drilling. The northing and easting coordinates and ground surface elevations were estimated based on field measurements relative to the survey plan provided by MTO.

RPM Drilling of Thunder Bay, Ontario supplied and operated the drilling, sampling and in-situ testing equipment for the field investigation. The field investigation was supervised on a full-time basis by Mr. Kevin Kweon of Thurber. The overall supervision of the field program was conducted by Mr. Mark Farrant, P.Eng. of Thurber. Geotechnical laboratory testing was carried out in Thurber's geotechnical laboratory.

Interpretation of the field data and preparation of this report was carried out by Mr. Mark Farrant, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

8. GENERAL

This report provides an interpretation of the geotechnical data in the factual report, and presents foundation design recommendations for the proposed replacement of the Strawberry Creek #8 Bridge.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation, and shall not be used or relied upon for any other purposes or by any other parties including the construction contractor. Contractors must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

The site is located on Ware Road North, approximately 3.2 km north of Auto Road, in Ware Township, Thunder Bay District, Ontario. The existing bridge allows Strawberry Creek #8 to flow under Ware Road North in a general west to east direction. Ware Road North runs in a general north-south direction at the bridge site.

The Ontario Structure Inspection Manual (OSIM) report prepared by MTO on July 18, 2018 indicates that the existing structure is a single-span, wooden rectangular beam / girder bridge with a wooden deck. The inspection report indicates that the bridge has a span of 6.3 m and the structure is approximately 5.5 m wide. Based on a November 2019 survey plan of the site provided by MTO (Plan E-1019-0-7), the ground surface elevation of Ware Road North at the existing bridge is approximately Elevation 366.3 m. The existing bridge is supported on timber sitting on ballast walls. The OSIM report notes that the bridge has a very low clearance. Measurements collected in October 2019 indicate that the surface water level of the creek varies from approximate Elevation 365.4 to 365.2 m from upstream to downstream of the bridge.

8.1 Structure Replacement Alternatives

Based on the Terms of Reference for this assignment, it is anticipated that the replacement bridge will likely be a single-span modular bridge located along the same alignment with a similar or longer span than the existing bridge. It is understood that a grade raise will most likely be required to increase the clearance at the bridge. Some embankment widening may also be expected in association with the grade raise. The approximate proposed height of the grade raise provided by MTO is 1 m.

The site conditions near the bridge include marshy conditions along the creek floodplain. In general, the foundation soil stratigraphy at the site consists of gravelly sand to silty sand fill overlying a thick native deposit of firm to stiff silty clay, overlying compact silt and sand, and very dense gravelly sand, which are underlain by granodiorite bedrock. A buried organic layer was also encountered below the fill in the boreholes on the south side of the bridge, and the upper part of the silty clay was noted to contain organic pockets. Bedrock was encountered in one borehole at Elevation 349.6 m. Artesian groundwater pressure of greater than 3 m above the ground surface was also encountered in the cohesionless soils below the thick silty clay deposit. The reported water level in the creek in October 2019 ranged from Elevation 365.4 to 365.2 m from upstream to downstream of the bridge. The near surface groundwater level will likely reflect the creek water level.

Recommendations for the design and installation of foundations for a single span modular bridge with an approximate 1 m high grade raise are presented below.

9. STRUCTURE FOUNDATIONS

The existing 6.3 m long bridge is supported on timber sitting on ballast walls. Based on the subsurface conditions at this site, consideration was given to supporting the replacement bridge on the following foundation types:

- Spread footings placed on engineered fill pads; and
- Driven steel H-piles.

A comparison of the technical advantages and disadvantages of the alternative foundation options is presented in Appendix E.

For driven steel H-piles, it is anticipated that piles driven to bedrock would likely contact high artesian groundwater pressures in the cohesionless soils below the clay and would have difficulty penetrating the boulders encountered in the gravelly sand. Therefore, this alternative is not recommended at this site. Driven steel H-piles may however be feasible to support the abutments primarily through shaft friction in the firm to stiff silty clay, depending on the resistance required. Spread footings on engineered fill pads will eliminate the risk of encountering the artesian pressure below the silty clay.

Recommendations for spread footings founded on engineered fill pads or driven steel H-piles are provided in the following sections.

9.1 Spread Footings on Engineered Fill Pads

9.1.1 Founding Level

The use of concrete spread footings placed on 1 to 2 m thick granular engineered fill pads is considered feasible from a geotechnical perspective to support the replacement bridge.

Accounting for a proposed 1 m grade raise, the base of the spread footings are anticipated to be placed at approximate Elevation 366 m, or 0.5 m below the existing road grade. The footings must be embedded 0.5 m into the granular fill pads. The base of the engineered fill pads may be placed at approximate Elevation 365 m for a 1 m thick pad below the footing, or Elevation 364 m for a 2 m thick pad below the footing, which corresponds to a subgrade consisting of soft silty clay with organic pockets ranging to firm to stiff silty clay. A 2 m thick engineered fill pad below the footings is preferred in order to remove as much of the organic soil as possible below the footings.

9.1.2 Engineered Fill Pad Construction

The engineered fill pads should consist of OPSS Granular A or Granular B Type II placed in 150 mm lifts and compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD) at ±2% of Optimum Moisture Content (OMC). A separation layer consisting of a non-woven geotextile should be placed between the subgrade soils and the engineered fill pads.

The minimum depth of excavation should accommodate the concrete foundation slab and the thickness of engineered fill pad below the slab.

The dimensions of the base of the excavation should be determined by assuming a granular pad at least 1.0 m wider than the spread footing at the level of the footing base and projecting outward and downward at 1H:1V.

The face of the engineered fill pads should be embedded at least 1.0 m below the face of the forward slope, with the edge of footing at a minimum of 2 m behind the crest of the forward slope. Provision of properly designed erosion protection works for the forward and side slopes will be critical to ensure adequate performance of the foundations/engineered fill pads.

It will be beneficial to place/locate the new abutments/spread footings some distance behind the existing timber / ballast wall abutments to take advantage of the potential slope stabilizing effect of the existing foundations. However, if the footprints of the engineered pads overlap the footprints of the existing abutments, then the existing abutments should be removed, to a depth no deeper than the level of the creek.

9.1.3 Geotechnical Resistances

The following geotechnical resistances are recommended for design of minimum 1 and 2 m wide spread footings placed on 1 to 2 m thick granular engineered fill pads prepared as outlined above with the underside at approximate Elevation 365 to 364 m:

Table 9.1 – Geotechnical Resistances for Footings on Engineered Fill Pads

Geotechnical Resistance	Spread Footing on 1 m Thick Fill Pad at Elev. 365 m		Spread Footing on 2 m Thick Fill Pad at Elev. 364 m	
	1 m Wide Footing	2 m Wide Footing	1 m Wide Footing	2 m Wide Footing
Factored Geotechnical Resistance at ULS	125 kPa	90 kPa	150 kPa	125 kPa
Geotechnical Resistance at SLS (for up to 25 mm settlement)	40 kPa	25 kPa	60 kPa	50 kPa

As an approximate 1 m high grade raise is proposed, the placement of new fill at the abutments will result in settlement of the silty clay foundation soils. Discussion of settlement due to the grade raise is provided in Section 13.1 below. A minimum 2-month preloading period should be accommodated in the construction schedule to permit foundation consolidation. The preloading period should commence following construction of the engineered fill pads and prior to placement of the spread footings. The above geotechnical resistances assume that the placement of the spread footings will not take place until after the preloading period is complete in order to minimize settlement of the foundations. Following the preloading period, additional engineered granular fill may need to be added to the fill pads to reach to design footing level.

A consequence factor of 1.0 was utilized in this design adopting the typical consequence level. The geotechnical resistance factor of 0.5 for bearing and 0.8 for settlement, both adopted for typical degree of understanding, were used to obtain the above values, as per Canadian Highway Bridge Design Code (CHBDC) 2014, Section 6.9.

The factored ultimate resistance and settlement are dependent on the footing size, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the footing width or founding/invert elevation differs significantly from that given above.

The geotechnical resistance quoted above is for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance should be calculated as indicated in the CHBDC 2014 Clause 6. 10 .3 and Clause 6.10.4.

The lateral resistance of the concrete footings founded on engineered fill may be computed using an unfactored friction coefficient of 0.6. This is an "ultimate" value and requires a degree of sliding movement to occur to fully mobilize the resistance.

9.2 Driven Steel H-Piles

9.2.1 Axial Geotechnical Resistance and Reaction

Use of driven steel H-piles to support the abutments is considered feasible at this site but not recommended.

The axial factored geotechnical resistance at Ultimate Limit States (ULS) and geotechnical reaction at Serviceability Limit States (SLS) for steel HP 310x110 piles driven within the silty clay deposit or approximately 2 m into the underlying sand and silt are provided in Table 9.2 below. Pile tip protection or driving shoes should not be used as the piles will be largely frictional in nature. It must be noted that driving piles into the sand and silt layer with artesian pressure will potentially promote artesian water travelling upward along the pile shaft, which may cause a reduction in the pile resistance or pile settlement over time. Accordingly, the geotechnical resistances provided below are based on short friction piles in the silty clay or 2 m embedment into the underlying sand and silt with artesian conditions.

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Table 9.2 – Axial Factored Geotechnical Resistance and Reaction for HP 310x110

Location (Borehole)	Pile Length / Tip Elevation (m)	Bearing Stratum at Pile Tip	Factored Geotechnical Resistance at ULS (kN)	Geotechnical Reaction at SLS (kN)
North Abutment (SC8-01, SC8-02)	8 / 358.5	Silty Clay	125	100
	11 / 355.5	Sand and Silt	200	150
South Abutment (SC8-03, SC8-04)	7.5 / 359	Silty Clay	125	100
	10.5 / 356*	Sand and Silt	200	150

*Below bottom of sampled borehole

9.2.2 Pile Installation

Pile installation should be in accordance with OPSS 903.

Pile driving should be controlled in accordance with Standard Drawing SS103-11 (Hiley Formula) and the designer should specify an ultimate pile resistance. The Hiley formula need not be used until the piles are within 2.0 m of the design tip elevation. The appropriate pile driving note is “Piles to be driven in accordance with Standard SS103-11 using an ultimate resistance of ‘R’ kN per pile”. ‘R’ must have a minimum value of twice the design load at ULS.

Pile tip protection or driving shoes should not be used as the piles will be largely frictional in nature.

The alignment of the H-piles should be carefully selected to be outside of the footprint of the existing abutments and away from the creek banks.

9.2.3 Lateral Resistance

The geotechnical lateral resistance acting on a pile in cohesive soils may be estimated using the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$k_s = 67\ S_u / D$ (kN/m^3)

$p_{ult} = 9\ S_u$ (kPa)

Where S_u = undrained shear strength (kPa)

D = pile width or diameter (m)

The geotechnical lateral resistance acting on a pile in cohesionless soils may be calculated using the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

k_s = $n_h \, z / D$ (kN/m3)

p_{ult} = $3 \, \gamma' \, z \, K_p$ (kPa)

Where z = depth of embedment of pile (m)

D = pile width or diameter

(0.310 m for HP 310x110)

n_h = coefficient related to soil relative density (kN/m^3)

γ' = effective unit weight (kN/m^3)

K_p = passive earth pressure coefficient

The above equations and recommended parameters in Table 9.3 below may be used to analyse the interaction between a pile and the surrounding soil. Artesian pressure should be subtracted from the calculated vertical effective stress for the cohesionless deposits below the silty clay. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

Table 9.3 – Soil Parameters for Lateral Pile Resistance

Abutment	Elevation	S_u (kPa)	n_h (kN/m^3)	K_p	Unit Weight* (kN/m^3)	Soil Conditions
South (SC8-01, SC8-02)	366.4 to 365.2	-	2,500	3.0	21	Loose gravelly sand to silty sand fill
	365.2 to 364.2	35	-	-	7*	Soft silty clay with organic pockets
	364.2 to 357.6	35	-	-	6*	Firm to stiff silty clay
	357.8 to 354.6	-	2,000	3.0	10*	Compact to loose silt and sand
	354.6 to 349.6	-	7,500	4.2	11*	Very dense to compact gravelly sand
North (SC8-03, SC8-04)	366.4 to 364.9	-	2,500	3.0	21	Loose to dense gravelly sand to silty sand fill

Abutment	Elevation	S_u (kPa)	n_h (kN/m^3)	K_p	Unit Weight* (kN/m^3)	Soil Conditions
	364.9 to 364.6	25	-	-	4*	Very soft organics / peat
	364.6 to 363.3	35	-	-	7*	Soft silty clay with organic pockets
	363.3 to 358.2	35	-	-	6*	Firm to stiff silty clay
	358.2 to 356.0	-	2,000	3.0	10*	Compact to loose silt and sand

*Bouyant unit weight below groundwater level

The spring constant, K_s, for analysis may be obtained by the expression, $K_s = k_s L D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult} L D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements.

The modulus of subgrade reaction and ultimate lateral resistance may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Section C.6.11.3.4 of CHBDC 2014.

Horizontal loads may be resisted by means of battered piles (i.e. for H-pile case) if load requirements exceed the available lateral pile resistances.

9.3 Downdrag

As an approximate 1 m high grade raise is proposed, the placement of new fill at the abutments will result in development of downdrag forces along the length of abutment piles associated with consolidation of the silty clay foundation under the weight of the new fill. Discussion of settlement due to the grade raise is provided in Section 13.1 below. A minimum 2-month preloading period should be accommodated in the construction schedule to permit foundation consolidation. Pile installation should not commence until after the preloading period in order to minimize downdrag induced pile settlement.

For design purposes, an unfactored downdrag load of 250 kN per pile is recommended to evaluate the impact of downdrag on the abutment piles if foundation preloading cannot be accommodated prior to pile driving.

This downdrag load should be multiplied by a load factor of 1.25 as per CHBDC Commentary Clause C6.11.4.10 to obtain a factored downdrag load. In accordance with Section 6.11.4.10 of the CHBDC and Clause C6.11.4.10 of the Commentary, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag. The factored dead and downdrag load should not exceed the factored structural resistance of a pile at the neutral plane.

9.4 Frost Cover

The depth of frost penetration at this site is approximately 2.3 m, as per OPSD 3090.100. Typically, the base of all footings or concrete pile caps, if employed, must be provided with a minimum of 2.3 m of earth cover as protection against frost action.

Concrete spread footings founded on granular engineered fill pads, provided they consist of non-frost susceptible, free draining granular engineered fill, above the creek water level may be provided with a minimum embedment of 0.5 m. For a modular bridge that can accommodate some movement, these footings do not need to be placed below the depth of frost.

If steel H-piles are adopted, the base of concrete pile caps should be provided with a minimum of 2.3 m of earth cover as protection against frost action. Alternately, consideration could be given to utilizing horizontal header beams instead of concrete pile caps, in which case the frost cover requirement would not apply.

9.5 Recommended Foundation

While both options are feasible, spread footings founded on engineered fill pads placed on the native silty clay are likely more cost-effective than driven steel H-piles, they can accommodate some settlement for a modular bridge, and there is little to no risk of encountering artesian groundwater pressures during construction. Therefore, spread footings founded on engineered fill pads are the recommended foundation option at this site. The 2 m thick engineered fill pad option is recommended in order to remove as much of the organic soil as possible below the footings.

10. EXCAVATION AND GROUNDWATER CONTROL

All excavations must be carried out in accordance with the requirements of the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the existing gravelly sand to silty sand fill above the water level, as well as the native silty clay below the water level may be classified as Type 3 soil. The organic soils above or below the water level are classified as Type

4 soils. Unsupported temporary excavations must be inclined from the bottom of the excavation at no steeper than 1H:1V when made in Type 3 soil, or 3H:1V when made in Type 4 soil. It should be assumed that the groundwater level will be governed by the surface water level in the creek.

The excavation and backfilling for foundations must be carried out in accordance with OPSS 902. Excavations for placement of engineered fill pads will be carried out through the existing gravelly sand fill, silty sand fill, organics/peat and into the native silty clay containing organic pockets to approximate Elevation 365 to 364 m. If pile caps are utilized, the excavations will also extend to approximate Elevation 364 m or lower into the native silty clay. The majority of the excavations are expected to be above the groundwater level, however any excavations below the groundwater or creek water level will require dewatering to lower the water level at least 0.5 m below the base of the excavation and permit construction in the dry and facilitate compaction of the backfill materials.

Selection of the method of excavation is the responsibility of the Contractor and should be based on the Contractor's experience, equipment and interpretation of the site conditions.

The design of the dewatering system, if utilized, is the responsibility of the Contractor. The Contract Documents must alert the Contractor to this responsibility and to design the system in accordance with SP FOUN0003 which amends OPSS 902. SP FOUN0003 has been included in Appendix F.

In accordance with SP FOUN0003, the dewatering system is to be designed in accordance with OPSS.PROV 517. A preconstruction survey is not required at this site, thus Designer Fill-In ** in SP FOUN0003 should be "N/A".

Dewatering must remain operational and effective until the foundations are constructed and backfilled. Suggested wording for an NSSP in this regard is included in Appendix F.

11. LATERAL EARTH PRESSURES

If any new backfill is placed behind the bridge abutments, it should be placed in accordance with OPSS 902. All backfill should consist of free-draining, non-frost susceptible granular materials such as Granular A or B Type II or Type III conforming to the requirements of OPSS.PROV 1010. Reference should be made to the backfill arrangements stipulated in OPSD 3101.150, as appropriate. Compaction equipment to be used adjacent to the walls should be restricted in accordance with OPSS.PROV 501.

Earth pressures acting on the structures may be assumed to be distributed triangularly and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

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

Where: p_h = horizontal pressure on the wall at depth h (kPa)
K = coefficient of lateral earth pressure (see Table 11.1)
γ = unit weight of retained soil (see Table 11.1)
h = depth below top of fill where pressure is computed (m)
q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are given in Table 11.1.

Table 11.1 – Coefficients of Lateral Earth Pressure (K)

Loading Condition	OPSS Granular A or Granular B Type II $\phi = 35°$, $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I or Type III $\phi = 32°$, $\gamma = 21.2$ kN/m^3	
	Horizontal Backfill	Sloping Backfill	Horizontal Backfill	Sloping Backfill
Active K_A (Unrestrained Wall)	0.27	0.38*	0.31	0.46*
At-rest K_0 (Restrained Wall)	0.43	-	0.47	-
Passive K_P	3.7	-	3.3	-

* For abutment walls, if required

The use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) is preferred as it results in lower earth pressures acting on the wall.

In accordance with Clause 6.12.3 of the CHBDC 2014, a compaction surcharge should be added. The magnitude of the surcharge should be 12 kPa at the top of fill which linearly decreases to 0 kPa at a depth of 1.7 m (for Granular B Type I) or at a depth of 2.0 m (for Granular A or B Type II).

12. SEISMIC CONSIDERATIONS

In accordance with the CHBDC 2014, the selection of the seismic site class is based on the soil conditions encountered in the upper 30 m of the stratigraphy. In view of the presence of the relatively thick deposit of firm to stiff silty clay, the site is considered to be classified as Site Class D in accordance with Table 4.1, Clause 4.4.3.2 of the CHBDC. The peak ground acceleration, PGA, for a 2,475-year return period seismic event at this site is 0.037 g as per the National Building Code of Canada (NBCC). The Sa(0.2) and Sa(1.0) values for this return period are 0.063 and 0.019 respectively.

Since the Sa(0.2) value is less than 0.20 and the Sa(1.0) value is less than 0.10, and the structure is not considered to be a lifeline bridge, the new structure is considered as a Seismic performance category 1 based on Table 4.10 of the CHBDC 2014. Therefore, it does not need to be analyzed for seismic loads regardless of its importance and geometry in accordance with Section 4.4.5.1 of the CHBDC 2014.

13. EMBANKMENT RESTORATION AND GRADE RAISE

The existing Ware Road North embankment fill is approximately 1.5 m in height and constructed in a marshy environment. The embankment side slopes are approximately 2H:1V and appear to be in stable condition. A 0.4 to 0.7 m thick layer of organic soil containing some peat was observed below the embankment fill on the south side of the bridge and there may be other zones of buried organic soil that were not observed in the boreholes. A grade raise of approximately 1 m is proposed for Ware Road North at this site. Assessment of the embankment settlement and slope stability with the grade raise are provided in the follow sections.

13.1 Settlement

The results of settlement analysis for a 1 m grade raise on the existing embankment indicate that the centreline of the embankment is anticipated to settle approximately 75 mm during and following the end of embankment construction. The side slopes and embankment toes, which have not experienced the previous loading of the travelled portion of the roadway, will settle approximately 160 mm over the same period. In order to meet the MTO Embankment Settlement Criteria of 25 mm settlement within 20 m of the bridge abutment, it is recommended that a minimum 2-month preloading period be accommodated in the construction schedule to permit foundation consolidation prior to paving. Post-construction settlement after 2 months of preloading is anticipated to not exceed 25 mm.

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As discussed in Section 9, if spread footings are selected for the replacement bridge abutment foundations, the engineered fill pads should be constructed prior to the preloading period. Placement of the spread footings should not be done until the preloading period is complete to allow for most of the embankment settlement to be completed. Additional granular engineered fill may be placed if necessary to reach the design footing level following preloading. If pile foundations are selected, pile installation should not commence until after the preloading period to minimize downdrag induced pile settlement.

13.2 Slope Stability

Stability analysis for the embankment side slopes and forward slopes in front of the abutments has been conducted. The side slope analyses assume the grade raise fill on the top and side of the embankment will be inclined at 2H:1V slopes and are modelled including the buried organic layer encountered on the south side of the bridge. The forward slope analyses assume the spread footings on 2 m thick engineered fill pad foundation option will be utilized, with the footings set back a minimum of 2 m from the crest of the creek slopes. A summary of the stability analyses is provided in Table 13.1 below. The stability analysis figures are included in Appendix G.

Table 13.1 – Slope Stability Analysis Results

Analysis Case	Figure Number	Factor of Safety
Embankment Side Slopes, Existing Condition	1	1.8
Embankment Side Slopes with 1 m Grade Raise, Undrained Analysis	2	2.5
Embankment Side Slopes with 1 m Grade Raise, Drained Analysis	3	1.5
South Forward Slope, Existing Condition	4	1.4
South Forward Slope with 1 m Grade Raise, Undrained Analysis	5	2.1
South Forward Slope with 1 m Grade Raise, Drained Analysis	6	1.5
North Forward Slope, Existing Condition	7	1.4

Analysis Case	Figure Number	Factor of Safety
North Forward Slope with 1 m Grade Raise, Undrained Analysis	8	2.2
North Forward Slope with 1 m Grade Raise, Drained Analysis	9	1.5

The stability analysis results indicate that the embankment side slopes and forward slopes under the conditions analysed will be stable in both the short term (undrained) and long term (drained) conditions and meet MTO's criteria of a Factor of Safety of 1.5 for permanent slopes.

Please note that that assumptions have been made as to the locations of the replacement bridge abutments for the forward slope analysis. When a General Arrangement drawing is available and the actual abutment locations are known, the forward slope analysis should be revisited.

The stability of the embankment and forward slopes will rely on proper steps being taken to mitigate erosion as described in Section 14.

13.3 Embankment Construction

Embankment restoration after completion of the bridge replacement and placement of the grade raise fill should be carried out in accordance with OPSS.PROV 206 and OPSS.PROV 209. The embankment material should consist of imported Granular A, Granular B Type II, or Granular B Type III material.

In general, surface vegetation, topsoil, organic deposits, disturbed material or otherwise loose/soft soils should be stripped from the foundation footprints, and within the embankment footprints. Inspection and approval of the foundation surfaces by qualified geotechnical personnel should be conducted.

14. SCOUR AND EROSION PROTECTION

Erosion protection must be provided at the bridge abutments. Design of the erosion protection measures should consider hydrologic and hydraulic factors and should be carried out by specialists experienced in this field.

Typically, rock protection should be provided over all surfaces with which creek water is likely to be in contact. A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion in general accordance with OPSS.PROV 804.

Consideration may also be given to leaving the existing timber / ballast wall abutments in place if the new abutments are behind the existing cribs.

15. CORROSION AND SULPHATE ATTACK POTENTIAL

The results of the corrosivity and sulphate analytical tests conducted on the samples of the silty sand fill, native silty clay and creek water indicate the following conditions at the locations tested:

- The potential for sulphate attack or corrosion on concrete foundations from the surrounding soil or creek water is considered to be negligible due to low concentrations of sulphate and chloride in the samples tested. The effect of road deicing salt should also be considering while selecting the class of concrete.
- The potential for soil corrosion on metal is considered to be severe in the silty sand fill and mild to in the native silty clay.
- The potential for water corrosion on metal is considered to be mild.
- The effect of road de-icing salt should be considered when selecting corrosion protection measures.

16. CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- Driving steel H-piles below the native silty clay presents a risk of encountering artesian groundwater that may travel up the pile shafts, which may cause a reduction in the pile resistance or pile settlement over time.
- Seasonal fluctuations of the groundwater and creek level are to be expected. In particular, the water level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall, which may impact the construction.
- Erosion protection of the forward slopes in front of the bridge abutments is critical.

- The Contractor's selection of construction equipment and methodology should include assessment of the capability of the existing subsurface soils to support the proposed construction equipment and fill if needed as a crane pad for placement of the replacement bridge. Site conditions may limit the type of equipment suitable for use during construction. The design and safety of any temporary works is the responsibility of the Contractor.

17. CLOSURE

Engineering analysis and preparation of this report was carried out by Mr. Mark Farrant, P.Eng. and Mr. Keli Shi, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

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

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

▼ Water Level

C_{pen} Shear Strength Determination by Pocket Penetrometer

(1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.

(2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

<u>UNIFIED SOILS CLASSIFICATION</u>

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

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

SYMBOLS

CLAYSTONE

SILTSTONE

SANDSTONE

COAL

Bedrock (general)

DISCONTINUITY SPACING

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

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength (MPa)**	**(psi)**	**Field Estimation of Hardness***
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

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RECORD OF BOREHOLE No SC8-01

METRIC

GWP# 6014-18-00	LOCATION Strawberry Creek Bridge #8 N 5 385 269.3 E 343 774.2		ORIGINATED BY KK
DIST Thunder Bay HWY Ware Road N.	BOREHOLE TYPE Solid Stem Augers/Washbore/NQ Coring		COMPILED BY BH
DATUM Geodetic	DATE 2019.11.04 - 2019.11.06	LATITUDE 48.604746 LONGITUDE -89.471476	CHECKED BY MF

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	FIELD VANE (kPa) / SENSITIVITY	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
366.3	GROUND SURFACE							
0.0 / 0.1	**ASPHALT** (50mm)							Artesian Pressure >3 m Above Ground Surface
	Silty **SAND**, trace gravel, trace clay, contains rootlets, Loose, Brown, Moist (FILL)	1	SS	6	366			
365.6 / 0.7	Silty **CLAY**, trace sand, trace gravel, some organic pockets and wood fragments, Stiff to Very Stiff, Dark Grey/Grey, Moist (OH)	2	SS	13	365			0 9 43 48
		3	SS	30			101	
364.2 / 2.1	Silty **CLAY**, trace sand, Firm, Brown, Moist (CH)				364	1.7		
		1	ST					
					363	2.3		
		4	SS	2				0 4 23 73
					362	2.2		
		2	ST					
					361	2.0		
		5	SS	3	360			
					359	2.8		
	becoming stiff	3	ST		358			
357.6 / 8.7	**SILT** and **SAND**, trace clay, Compact to Loose, Grey, Wet					2.0		
		6	SS	12	357			0 47 49 4

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; STRAT PLOT

RECORD OF BOREHOLE No SC8-01

METRIC

GWP# 6014-18-00 | LOCATION Strawberry Creek Bridge #8 N 5 385 269.3 E 343 774.2 | ORIGINATED BY KK

DIST Thunder Bay HWY Ware Road N. | BOREHOLE TYPE Solid Stem Augers/Washbore/NQ Coring | COMPILED BY BH

DATUM Geodetic | DATE 2019.11.04 - 2019.11.06 | LATITUDE 48.604746 | LONGITUDE -89.471476 | CHECKED BY MF

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page				356		
		7	SS	9	355		
354.6 / 11.7	Gravelly **SAND**, silty, occasional cobbles and boulders; Very Dense; Grey; Wet	8	SS	100/ 0.050	354		
	artesian pressure encountered increasing with depth; cored through 4 boulders ranging in size from 200 to 280mm thick	9	SS	100/ 0.050	353, 352, 351		
	becoming compact	10	SS	29	350		33 47 20 (SI+CL)
349.6 / 16.7	**BEDROCK** (Granodiorite); Slightly weathered,very strong, grey; rubble zone(100mm) at 17.8m	1	RUN		349	FI: 3, 5, 4, >10, 1	RUN #1 TCR=100% SCR=95% RQD=40% UCS=170MPa Average
348.0 / 18.3	END OF BOREHOLE AT 18.3m. BOREHOLE BACKFILLED AND ARTESIAN PRESSURE SEALED WITH MIX OF PORTLAND CEMENT , BENTONITE GROUT, AND CONCRETE TO SURFACE.						

Plot headers: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; GROUND WATER CONDITIONS; STRAT PLOT.

+ 3, × 3: Numbers refer to Sensitivity   15 20 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SC8-02

METRIC

GWP# 6014-18-00 | LOCATION Strawberry Creek Bridge #8 N 5 385 267.4 E 343 777.5 | ORIGINATED BY KK

DIST Thunder Bay HWY Ware Road N. | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY BH

DATUM Geodetic | DATE 0209.11.09 - 2019.11.09 | LATITUDE 48.604729 | LONGITUDE -89.471431 | CHECKED BY MF

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa (Field Vane)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
366.4	GROUND SURFACE					
0.0	Gravelly **SAND**, silty, trace clay, occasional rootlets Loose Brown Moist (FILL)	1	SS	5		24 52 22 2
		2	SS	13		
365.2 / 1.1	Silty **CLAY**, trace sand, trace gravel, some organic pockets and wood fragments Stiff to Very Soft Brown/Dark Brown Moist	3	SS	2		
364.2 / 2.2	Silty **CLAY**, trace sand Stiff Brown Moist (CH)	4	SS	2		0 2 23 75
		5	SS	4		
					3.0	
		6	SS	6		
					2.3	
	becoming **CLAY**, trace to some silt, trace sand Firm		ST			0 1 11 88
					2.5	
		7	SS	3		0 0 7 93
357.7 / 8.7	END OF BOREHOLE AT 8.7m. BOREHOLE BACKFILLED WITH BENTONITE TO 0.3m, THEN CEMENT TO SURFACE.					

Plot headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60); UNIT WEIGHT γ kN/m^3; GROUND WATER CONDITIONS; ELEVATION SCALE (366, 365, 364, 363, 362, 361, 360, 359, 358); STRAT PLOT.

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No SC8-03

METRIC

GWP# 6014-18-00 LOCATION Strawberry Creek Bridge #8 N 5 385 258.2 E 343 767.8 ORIGINATED BY KK

DIST Thunder Bay HWY Ware Road N. BOREHOLE TYPE Hollow Stem Augers COMPILED BY BH

DATUM Geodetic DATE 2019.11.09 - 2019.11.09 LATITUDE 48.604646 LONGITUDE -89.471564 CHECKED BY MF

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa / DCPT	WATER CONTENT (%)	GR	SA	SI	CL
366.3	GROUND SURFACE										
0.0	**ASPHALT** (50mm)										
0.1	Gravelly **SAND**, silty, trace clay Dense to Compact Brown Moist (FILL)	1	SS	44	366			27	49	21	3
		2	SS	13							
364.9 1.4	Silty **CLAY**, trace sand, trace gravel, some organic pockets and wood fragments Soft Brown Moist	3	SS	2	365						
		4	SS	3	364		101				
363.3 3.0	Silty **CLAY**, trace sand Stiff Brown Moist (CH)	1	ST		363						
		5	SS	1	362						
		6	SS	WH				0	0	22	78
					361	+ 1.8					
		2	ST		360						
359.1 7.2	END OF BOREHOLE AT 7.2m. BOREHOLE BACKFILLED WITH BENTONITE TO 0.3m, THEN CEMENT TO SURFACE.					+ 2.5					

Legend: SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE. DYNAMIC CONE PENETRATION RESISTANCE PLOT. PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L. UNIT WEIGHT γ kN/m^3. REMARKS & GRAIN SIZE DISTRIBUTION (%).

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-27323.GPJ 2017TEMPLATE(MTO).GDT 6/10/20

Ministry of Transportation Ontario

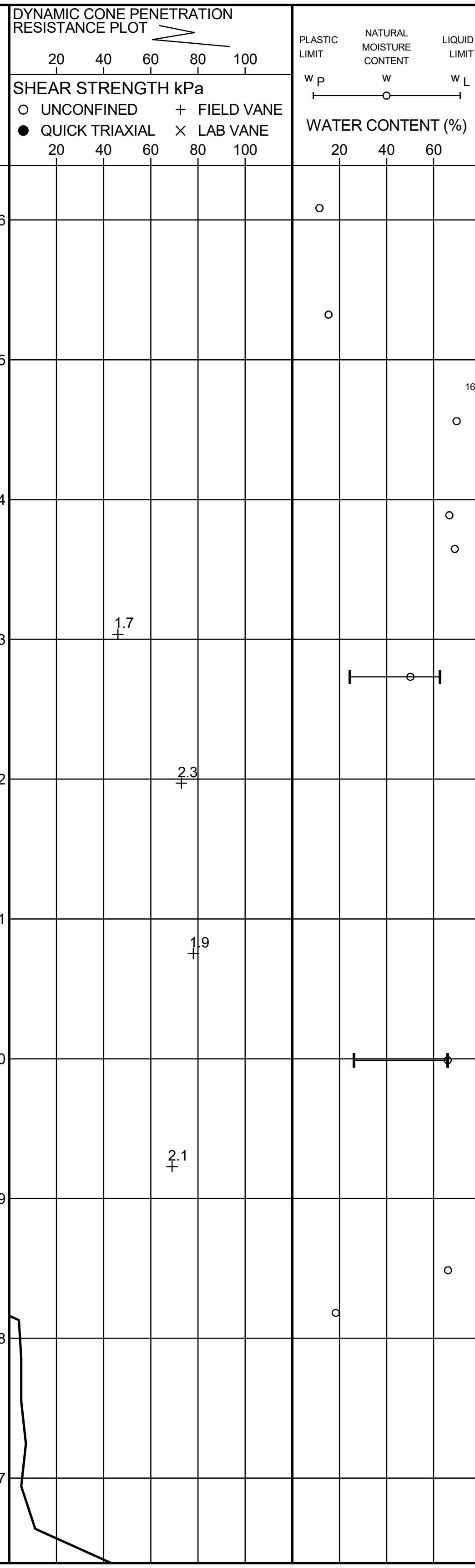
THURBER

RECORD OF BOREHOLE No SC8-04

METRIC

GWP# 6014-18-00	LOCATION Strawberry Creek Bridge #8 N 5 385 256.2 E 343 771.2		ORIGINATED BY KK
DIST Thunder Bay HWY Ware Road N.	BOREHOLE TYPE Hollow Stem Augers/Dynamic Cone Penetration Test		COMPILED BY BH
DATUM Geodetic	DATE 2019.11.08 - 2019.11.08 LATITUDE 48.604628	LONGITUDE -89.471518	CHECKED BY MF

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
366.4	GROUND SURFACE				
0.0	Silty **SAND**, trace to some gravel, trace clay; Dense to Loose; Brown; Moist; (FILL)	1	SS	36	
		2	SS	6	
364.9 / 1.4	**ORGANICS**, some peat, some sand, trace gravel, occasional rootlets and wood fragments; Very Soft; Black; Moist	3	SS	2	
364.6 / 1.8	Silty **CLAY**, some sand, trace gravel, occasional organic pockets; Very Soft; Brown; Moist; (CH)	4	SS	2	0 14 36 50
363.7 / 2.7	Silty **CLAY** trace sand; Firm to Stiff; Brown; Moist; (CH)	1	ST		0 1 29 70
		2	ST		
	becoming **CLAY**, some silt	5	SS	1	0 0 15 85
		6	SS	2	
358.2 / 8.2	**SILT** and **SAND**, trace clay; Grey; Wet; artesian pressure noted				
	END OF SAMPLING. START OF DCPT.				



ELEVATION SCALE: 366, 365, 364, 363, 362, 361, 360, 359, 358, 357

UNIT WEIGHT γ kN/m3

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No SC8-04

METRIC

GWP# 6014-18-00 LOCATION Strawberry Creek Bridge #8 N 5 385 256.2 E 343 771.2 ORIGINATED BY KK

DIST Thunder Bay HWY Ware Road N. BOREHOLE TYPE Hollow Stem Augers/Dynamic Cone Penetration Test COMPILED BY BH

DATUM Geodetic DATE 2019.11.08 - 2019.11.08 LATITUDE 48.604628 LONGITUDE -89.471518 CHECKED BY MF

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
356.0 / 10.4	END OF BOREHOLE AT 10.4m. SEALED ARTESIAN PRESSURE AND BACKFILLED BOREHOLE WITH BENTONITE TO 0.3m, THEN CEMENT TO SURFACE.						356				

$+^3$, $\times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No SC8-05

METRIC

GWP#	6014-18-00	LOCATION	Strawberry Creek Bridge #8 N 5 385 284.2 E 343 782.5	ORIGINATED BY	KK
DIST	Thunder Bay HWY Ware Road N.	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	BH
DATUM	Geodetic	DATE	2019.11.07 - 2019.11.07 LATITUDE 48.604879 LONGITUDE -89.471362	CHECKED BY	MF

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa (sensitivity)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
366.2	GROUND SURFACE								
0.0	Gravelly **SAND**, some silt Very Dense to Loose Brown Moist (FILL)		1	SS	100/0.175		366		
			2	SS	6		365		
364.9 1.2	Silty **CLAY**, trace sand Stiff Brown Moist (CH)		3	SS	5				0 1 20 79
			4	SS	4		364		
			5	SS	3		363		
							362	3.0	
	becoming **CLAY**, some silt, trace sand		6	SS	3		361		0 1 16 83
								2.1	
			1	ST			360		
							359	2.0	
			7	SS	WH		358		0 2 9 89
								2.7	
			8	SS	3		357		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No SC8-05

METRIC

GWP# 6014-18-00 LOCATION Strawberry Creek Bridge #8 N 5 385 284.2 E 343 782.5 ORIGINATED BY KK

DIST Thunder Bay HWY Ware Road N. BOREHOLE TYPE Hollow Stem Augers COMPILED BY BH

DATUM Geodetic DATE 2019.11.07 - 2019.11.07 LATITUDE 48.604879 LONGITUDE -89.471362 CHECKED BY MF

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P – NATURAL MOISTURE CONTENT w – LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
	Continued From Previous Page													
355.9							356							
10.2	**SILT**, some sand, trace clay Compact Grey Wet artesian pressure noted		9	SS	17				○ (~20)		0	11	82	7
354.9							355							
11.3	END OF BOREHOLE AT 11.3m. ARTESIAN PRESSURE SEALED AND BOREHOLE BACKFILLED WITH BENTONITE TO 0.3m, THEN CEMENT TO SURFACE.													

+³, ×³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SC8-06

METRIC

GWP# 6014-18-00 LOCATION Strawberry Creek Bridge #8 N 5 385 241.4 E 343 762.9 ORIGINATED BY KK

DIST Thunder Bay HWY Ware Road N. BOREHOLE TYPE Hollow Stem Augers COMPILED BY BH

DATUM Geodetic DATE 2019.11.07 - 2019.11.07 LATITUDE 48.604496 LONGITUDE -89.471632 CHECKED BY MF

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (FIELD VANE, sensitivity)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
366.2	GROUND SURFACE						
0.0	Gravelly **SAND**, silty, Compact, Brown/Dark Brown, Moist (FILL)	1	SS	24	366		22 52 26 (SI+CL)
365.6 / 0.7	**ORGANICS**, some peat, occasional rootlets and wood fragments, Very Soft, Black, Moist	2	SS	2	365		
364.8 / 1.4	Silty **CLAY**, some sand, trace gravel, some organic pockets and wood fragments, Very Soft, Brown/Grey, Moist	3	SS	WH			1 18 28 53
364.0 / 2.2	Silty **CLAY**, trace sand, Stiff, Brown, Moist (CH)	4	SS	1	364		0 1 28 71
		5	SS	1	363		
					362	2.7	
		1	ST		361	2.0	
		6	SS	WH	360		
					359	2.4	
359.1 / 7.2	**SILT** and **SAND**, trace clay, Grey, Wet, artesian pressure encountered, collected disturbed sample						
358.0 / 8.2	END OF BOREHOLE AT 8.2m. ARTESIAN PRESSURE SEALED AND BOREHOLE BACKFILLED WITH BENTONITE TO 0.3m, THEN CEMENT TO SURFACE.						

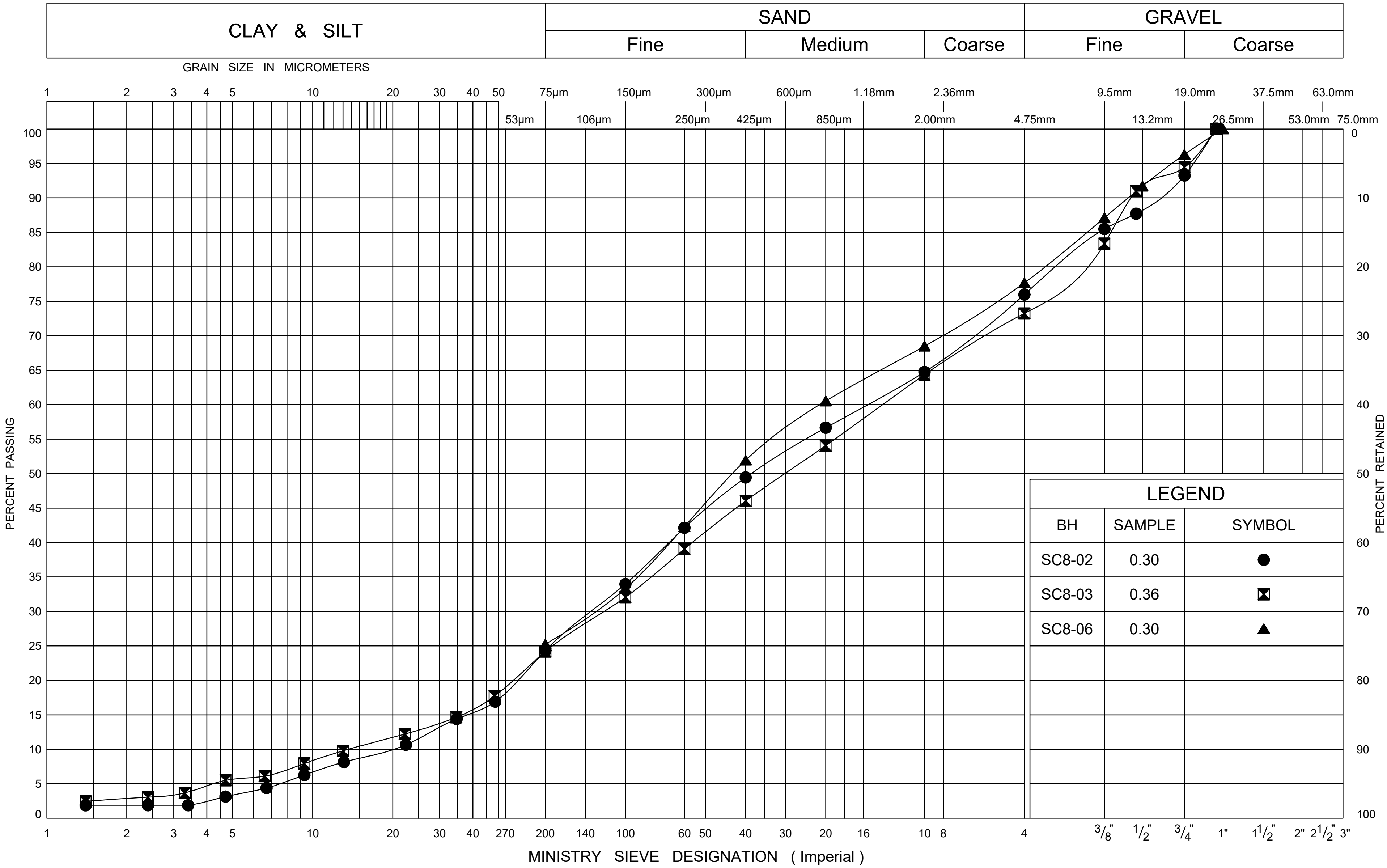
Water content plot: 134 (Sample 2).

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Appendix B

Geotechnical and Analytical Laboratory Test Results



LEGEND		
BH	SAMPLE	SYMBOL
SC8-02	0.30	●
SC8-03	0.36	⊠
SC8-06	0.30	▲

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GRAIN SIZE DISTRIBUTION

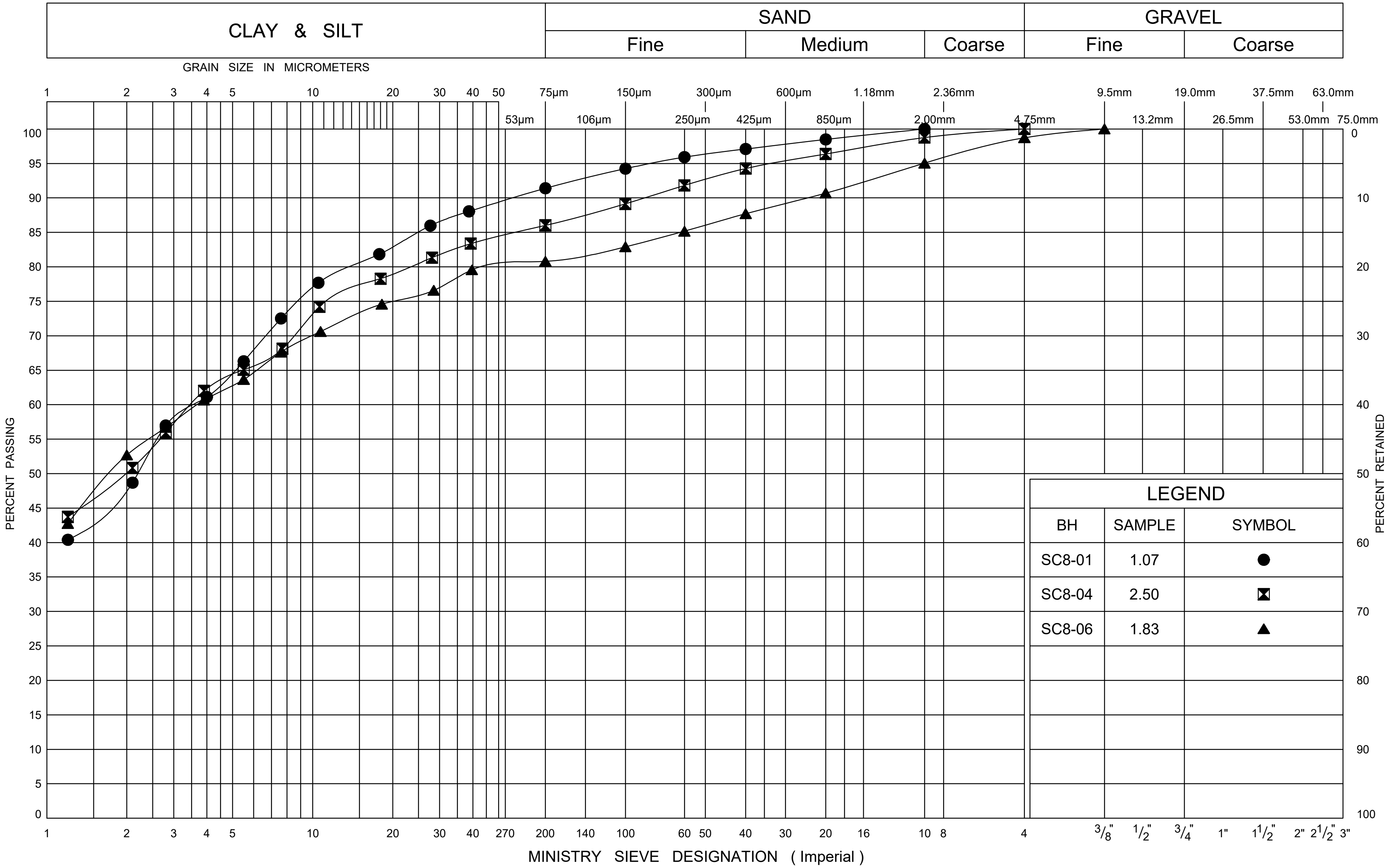
Gravelly SAND FILL

FIG No B1

W P 6014-18-00

Strawberry Creek Bridge #8

ONTARIO MOT GRAIN SIZE 2 MTO-27323.GPJ ONTARIO MOT.GDT 4/1/20



BH	SAMPLE	SYMBOL
SC8-01	1.07	●
SC8-04	2.50	⧖
SC8-06	1.83	▲

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GRAIN SIZE DISTRIBUTION

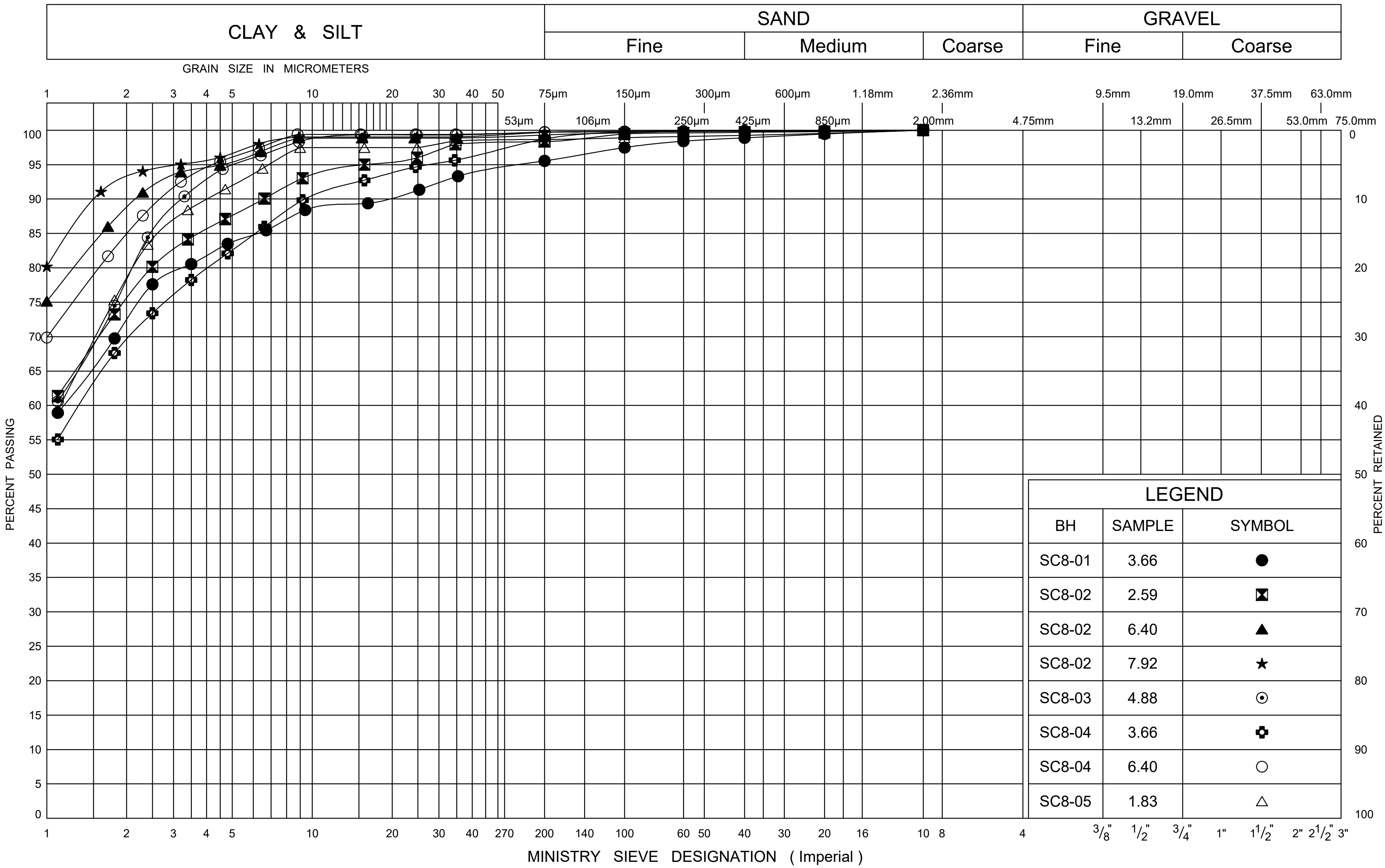
Silty CLAY with Organics

FIG No B2

W P 6014-18-00

Strawberry Creek Bridge #8

78 12 M



BH	SAMPLE	SYMBOL
SC8-01	3.66	●
SC8-02	2.59	⧗
SC8-02	6.40	▲
SC8-02	7.92	★
SC8-03	4.88	⊙
SC8-04	3.66	✿
SC8-04	6.40	○
SC8-05	1.83	△

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GRAIN SIZE DISTRIBUTION

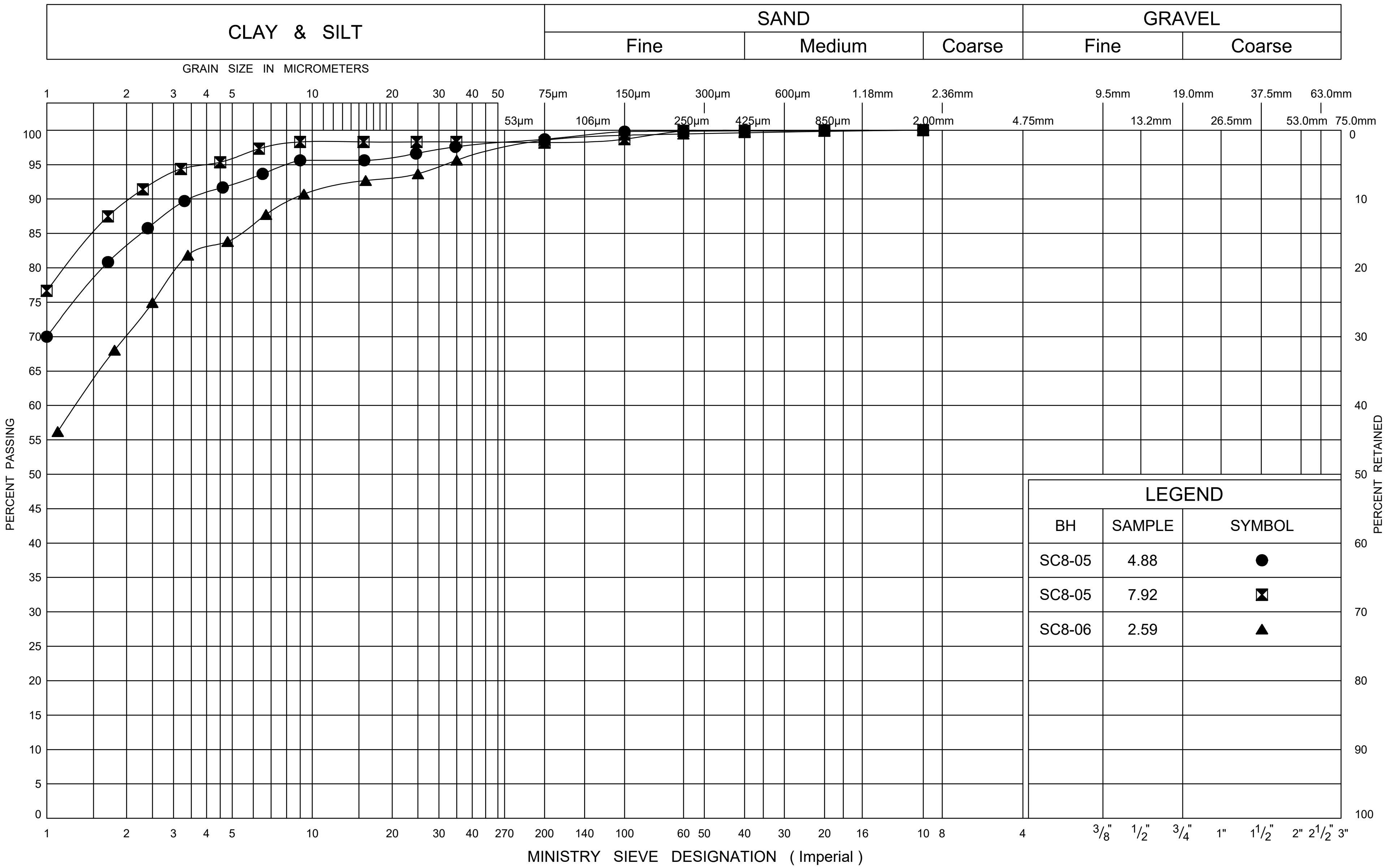
Silty CLAY

FIG No B3

W P 6014-18-00

Strawberry Creek Bridge #8

ONTARIO MOT GRAIN SIZE 2 MTO-27323.GPJ ONTARIO MOT.GDT 4/1/20



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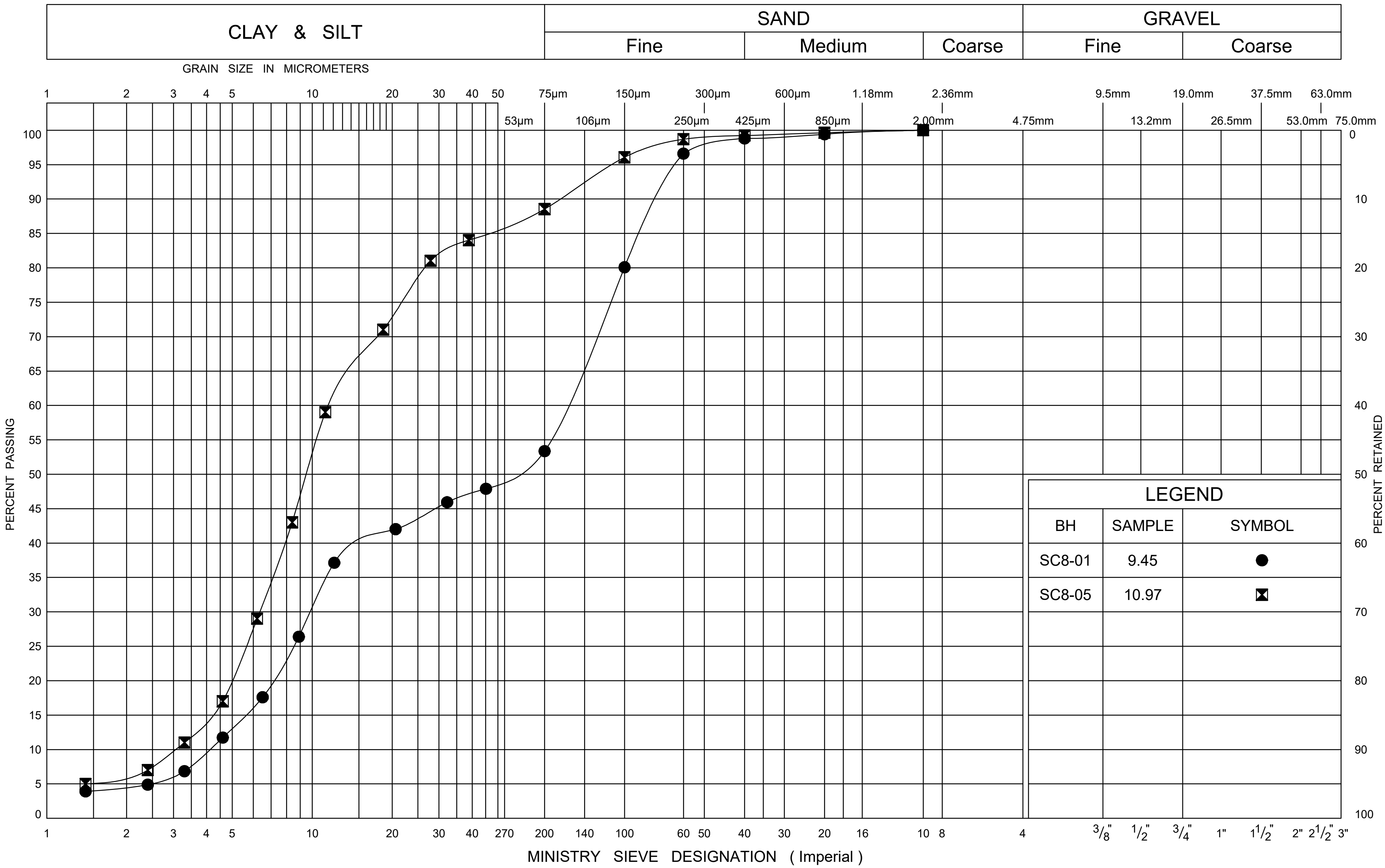
GRAIN SIZE DISTRIBUTION

Silty CLAY

FIG No B4

W P 6014-18-00

Strawberry Creek Bridge #8



BH	SAMPLE	SYMBOL
SC8-01	9.45	●
SC8-05	10.97	⧗

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GRAIN SIZE DISTRIBUTION

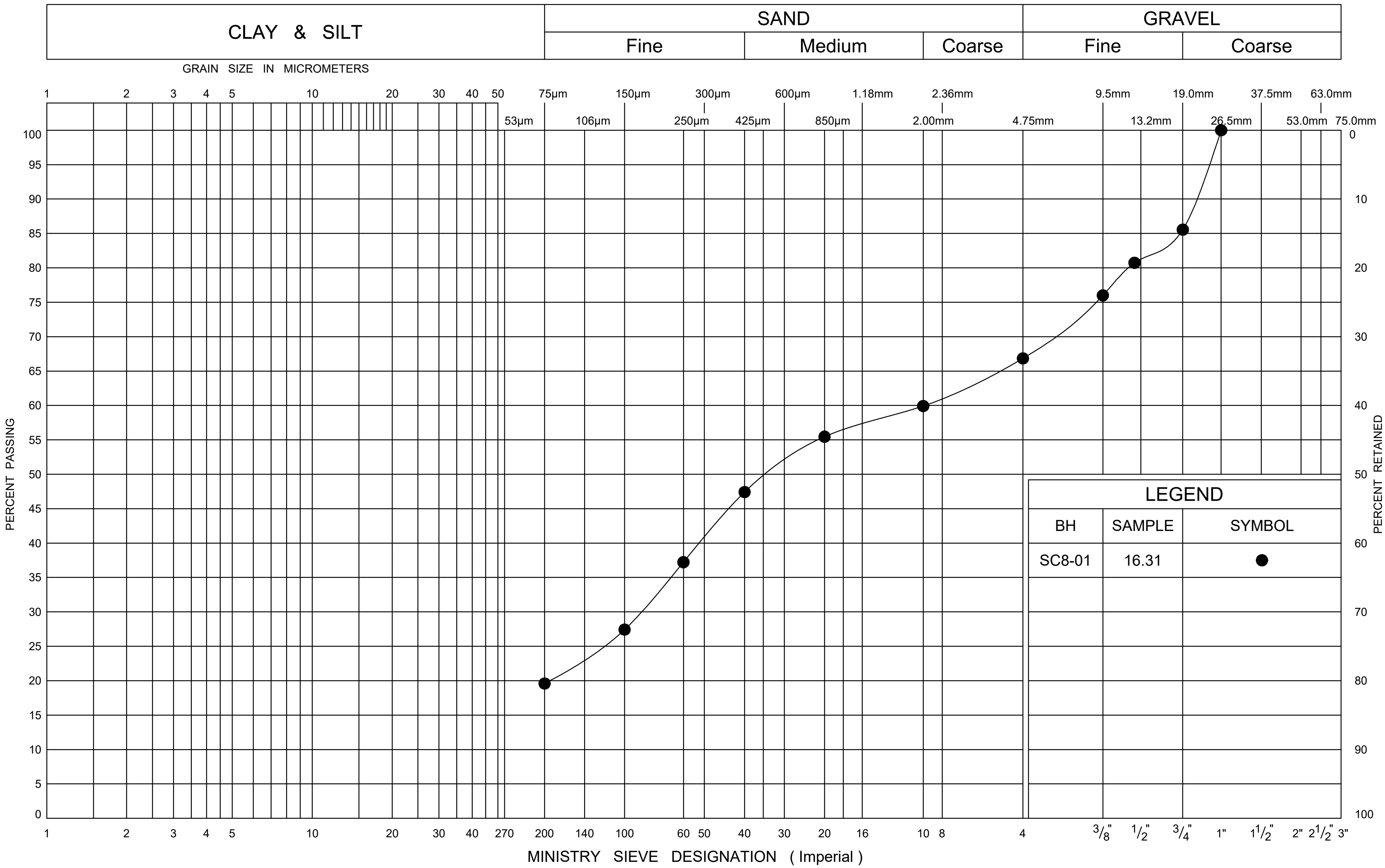
SILT to SILT and SAND

FIG No B5

W P 6014-18-00

Strawberry Creek Bridge #8

78 12 M



ONTARIO MOT GRAIN SIZE 2 MTO-27323.GPJ ONTARIO MOT.GDT 4/1/20

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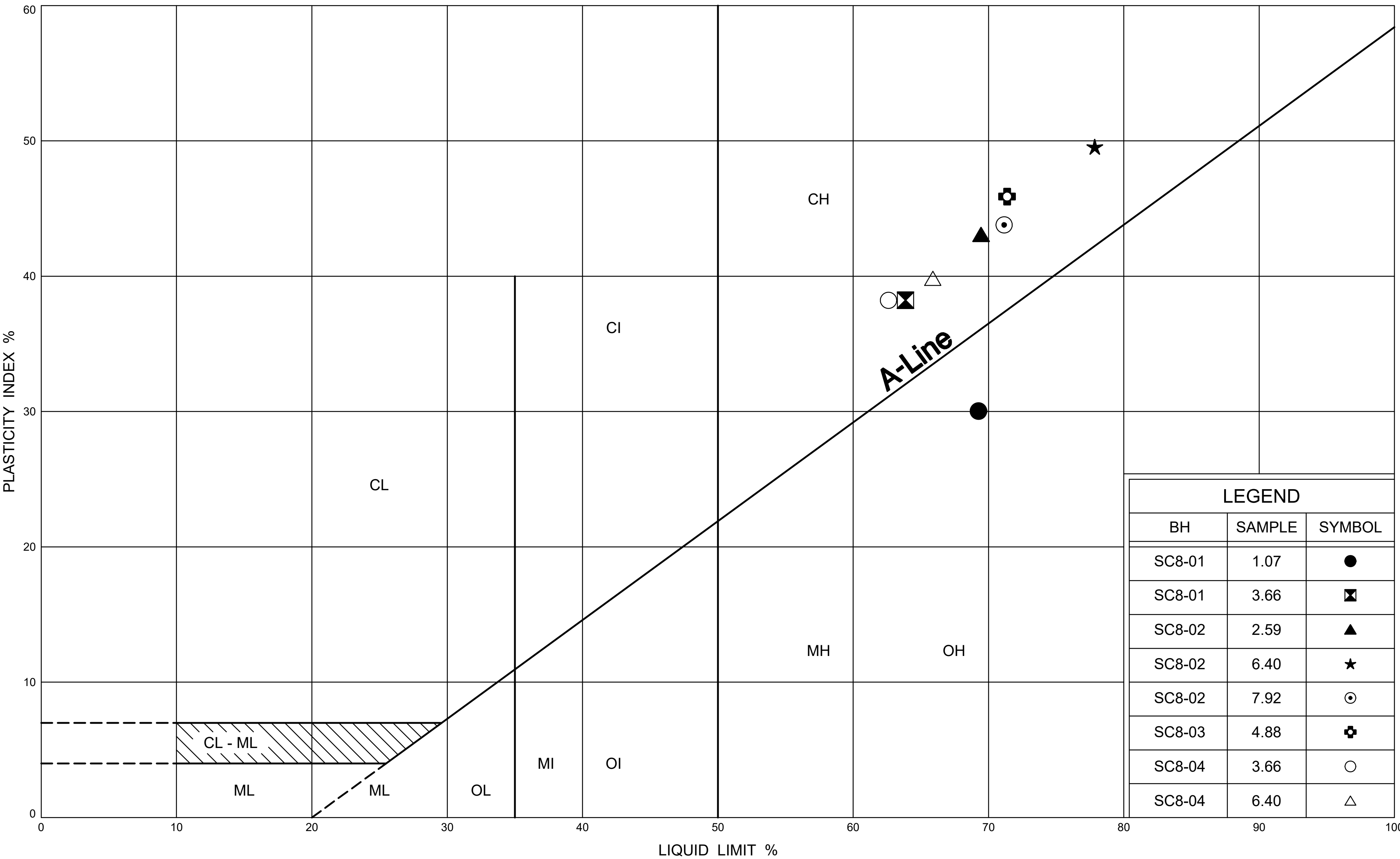
GRAIN SIZE DISTRIBUTION

Gravelly SAND

FIG No B6

W P 6014-18-00

Strawberry Creek Bridge #8



LEGEND		
BH	SAMPLE	SYMBOL
SC8-01	1.07	●
SC8-01	3.66	⧗
SC8-02	2.59	▲
SC8-02	6.40	★
SC8-02	7.92	⊙
SC8-03	4.88	✿
SC8-04	3.66	○
SC8-04	6.40	△

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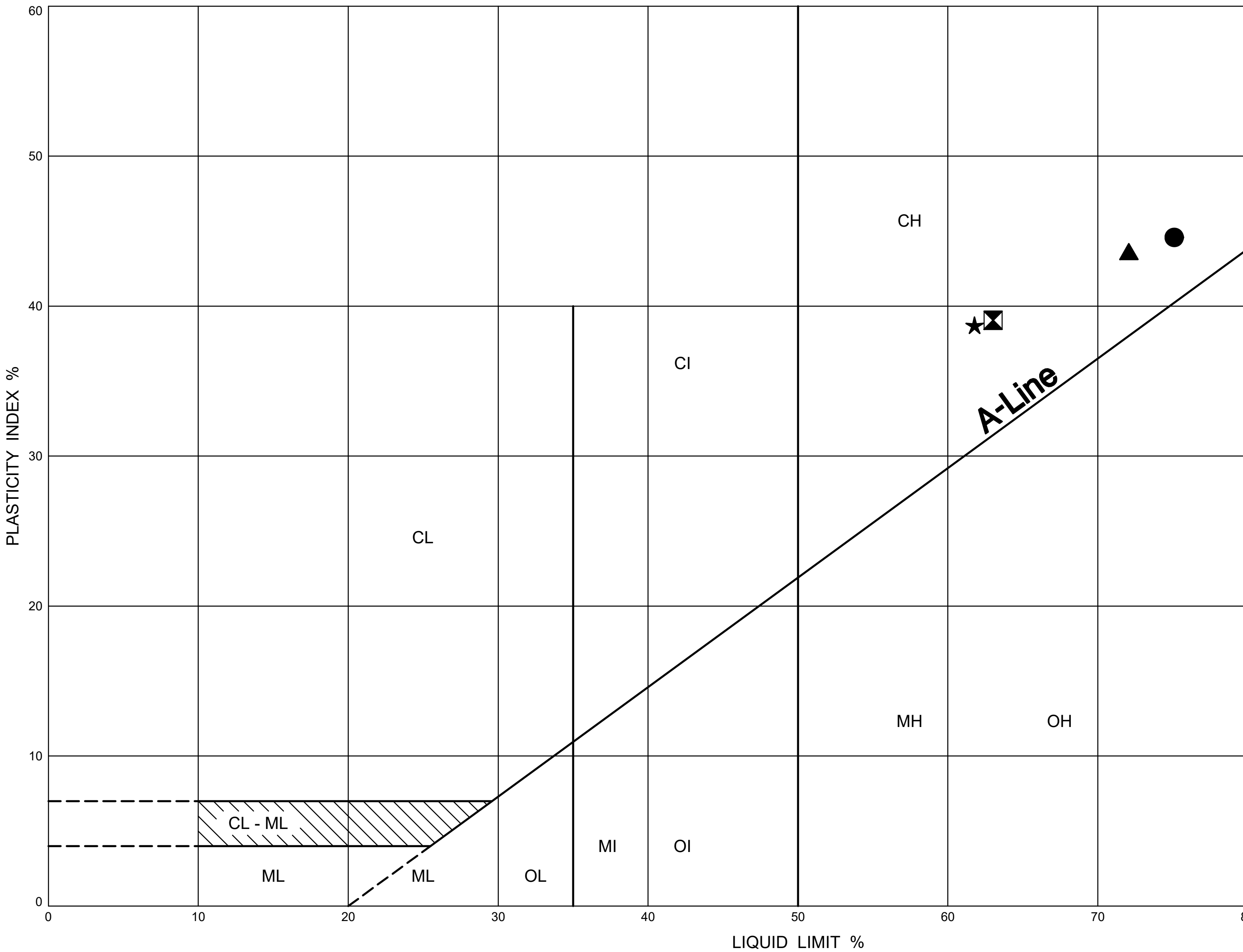
PLASTICITY CHART
Silty CLAY

FIG No B7

W P 6014-18-00

Strawberry Creek Bridge #8

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
SC8-05	1.83	●
SC8-05	4.88	⧗
SC8-05	7.92	▲
SC8-06	2.59	★

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PLASTICITY CHART
Silty CLAY

FIG No B8
W P 6014-18-00
Strawberry Creek Bridge #8

ONTARIO MOT PLASTICITY CHART 2 MTO-27323.GPJ ONTARIO MOT.GDT 4/1/20

THURBER ENGINEERING LTD.

Consolidation Test Report

CLIENT: MTO

FILE NUMBER: 27323

PROJECT: Sunshine Creek #2 and Strawberry Creek #8 Bridges

REPORT DATE: February 3, 2020

TEST DATES: November 27, 2019 - December 07, 2019

SAMPLE: SC8-02 ST1 20'-22'
Silty clay, trace gravel, reddish brown, moist.
LL=78, PL=28, I_p = 50

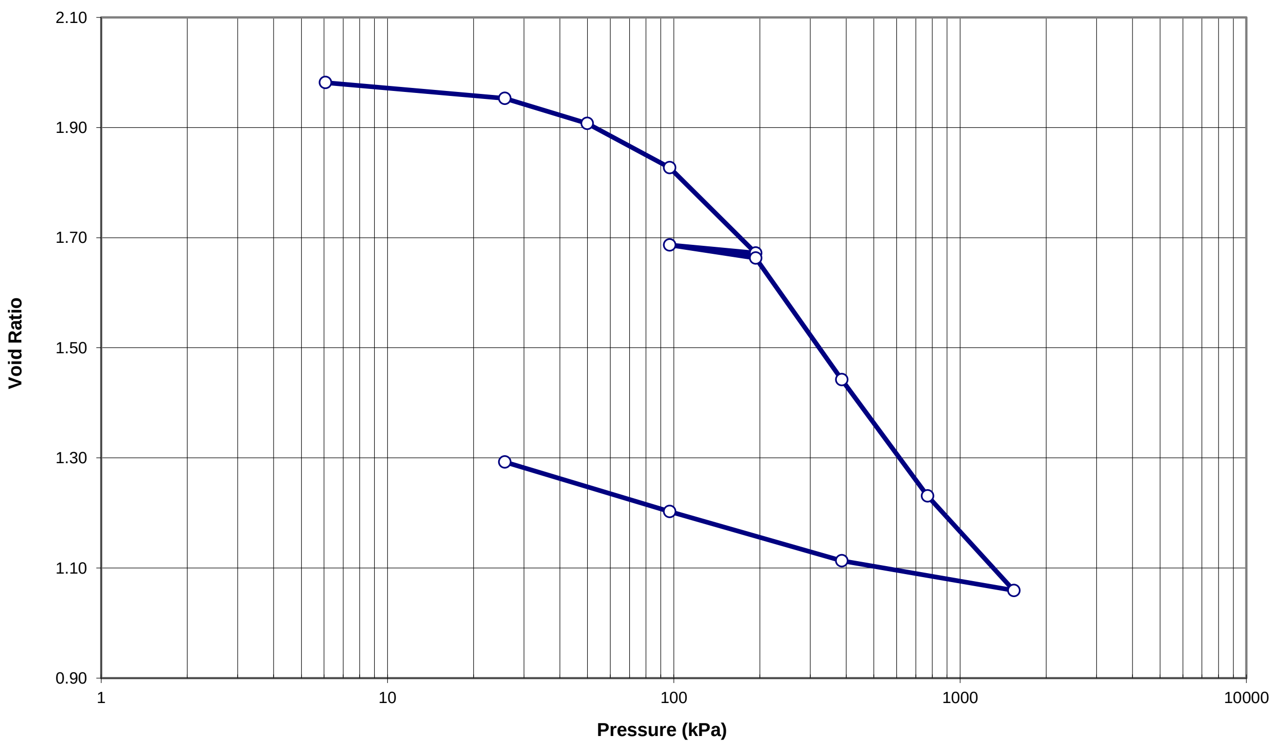
PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-11, method B

	Start of Test	End of Test
Wet Dens. (kg/m^3)	1591.6	1768.7
Dry Dens. (kg/m^3)	932.7	1213.3
Moisture Cont. (%)	70.6	45.8
Void Ratio	1.982	1.293
Saturation (%)	99.1	

Note: A Specific Gravity (Gs) of 2.78 was obtained for the void ratio and saturation calculations.

Project #: 27323
Client: MTO
Project Name: Sunshine Creek #2 and Strawberry Creek #8 Bridges
Sample: SC8-02 ST1 20'-22'

Void Ratio vs. Pressure



TEST DONE BY: BT
REVIEWED BY: KS

THURBER ENGINEERING LTD.

Consolidation Test Report

Sunshine Creek #2 and Strawberry Creek #8 Bridges
27323 SC8-02 ST1 20'-22'

TRIMMING: The Specimen was manually trimmed to the size of consolidation ring, then mounted in a fixed ring consolidometer.

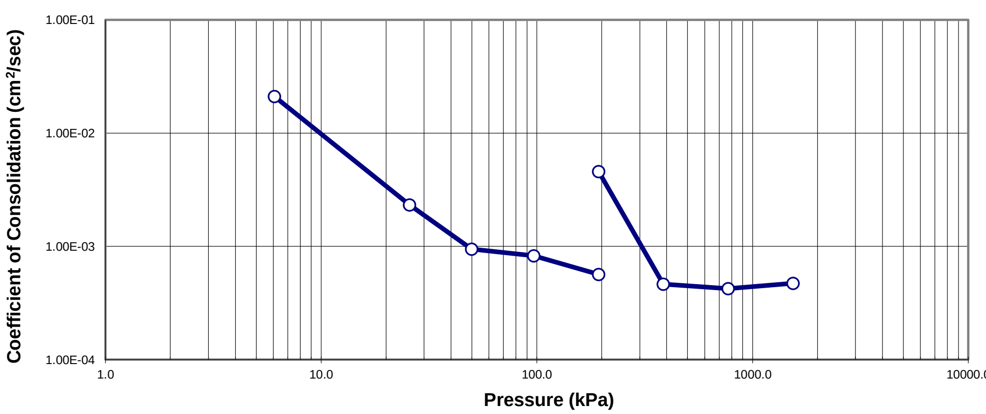
LOADING: A seating load of 6.1 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after 100% primary consolidation was reached at each load increment.

CALCULATIONS: Coefficients of Consolidation were calculated by the square root time method.

Pressure (kPa)	Corr. H. (mm)	Avg. H. (mm)	D_{90} (mm)	t_{90} (min)	c_v (cm^2/s)	Void Ratio	m_v (m^2/kN)	k (cm/s)
0.0	25.400					1.982		
6.1	25.396	25.398	-0.033	1.08	2.11E-02	1.982	2.59E-05	5.36E-08
25.7	25.153	25.275	-0.246	9.73	2.32E-03	1.953	4.88E-04	1.11E-07
49.9	24.767	24.960	-0.306	23.33	9.44E-04	1.908	6.34E-04	5.87E-08
96.6	24.080	24.424	-0.490	25.50	8.26E-04	1.827	5.93E-04	4.81E-08
193.2	22.761	23.421	-0.950	34.22	5.66E-04	1.672	5.67E-04	3.15E-08
96.6	22.886	22.824				1.687		
193.2	22.682	22.784	-0.113	4.00	4.59E-03	1.663	9.23E-05	4.15E-08
385.7	20.801	21.742	-1.368	36.00	4.64E-04	1.442	4.31E-04	1.96E-08
770.7	19.003	19.902	-1.400	33.06	4.23E-04	1.231	2.25E-04	9.32E-09
1540.7	17.539	18.271	-1.187	24.98	4.72E-04	1.059	1.00E-04	4.63E-09
385.7	17.999	17.769				1.113		
96.6	18.760	18.380				1.203		
25.7	19.527	19.144				1.293		

Project #: 27323
Client: MTO
Project Name: Sunshine Creek #2 and Strawberry Creek #8 Bridges
Sample: SC8-02 ST1 20'-22'
Oedometer Consolidation Test

Coefficient of Consoildation vs. Pressure



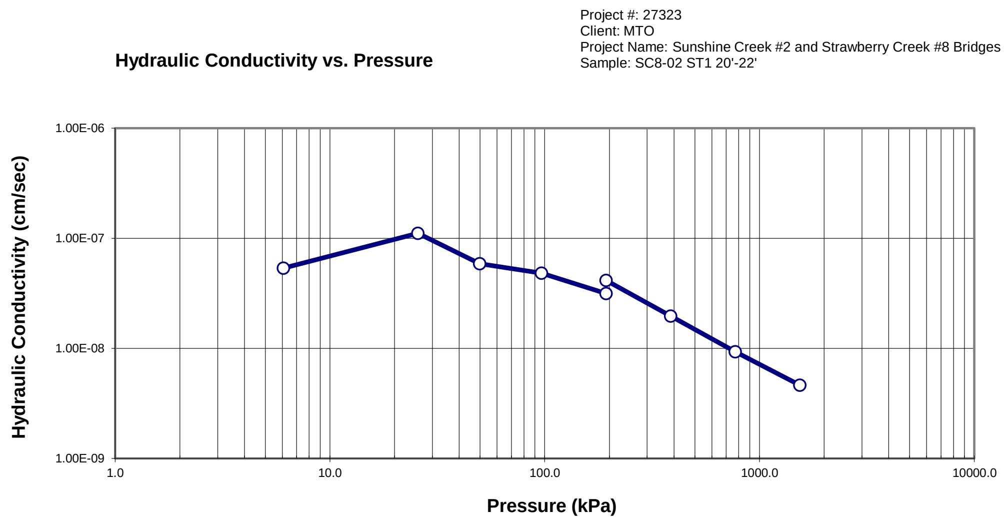
Notes: C_v and k calculated using t_{90} values

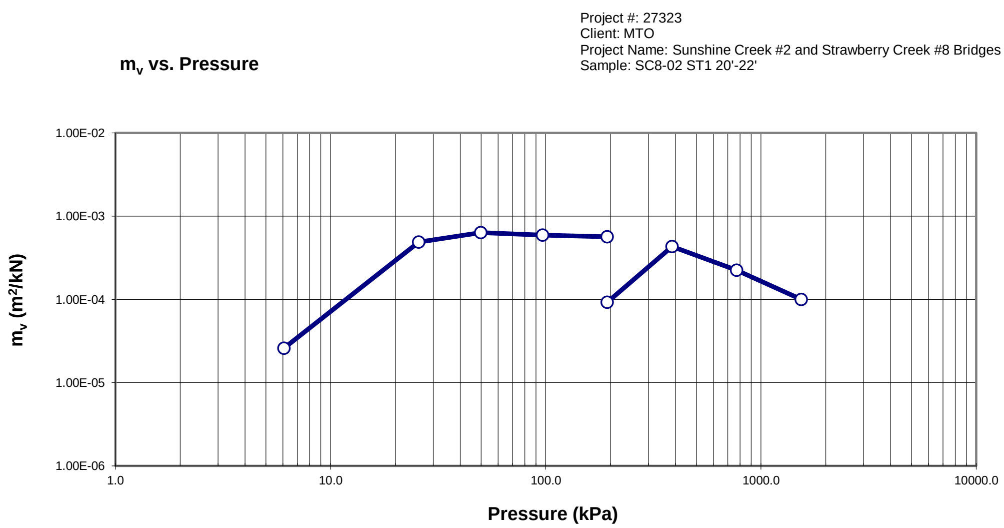
TEST DONE BY: BT
REVIEWED BY: KS

THURBER ENGINEERING LTD.

Consolidation Test Report

Sunshine Creek #2 and Strawberry Creek #8 Bridges
27323 SC8-02 ST1 20'-22'





TEST DONE BY: BT
REVIEWED BY: KS

THURBER ENGINEERING LTD.

Consolidation Test Report

CLIENT: MTO FILE NUMBER: 27323

PROJECT: Sunshine Creek #2 and Strawberry Creek #8 Bridges REPORT DATE: February 4, 2020

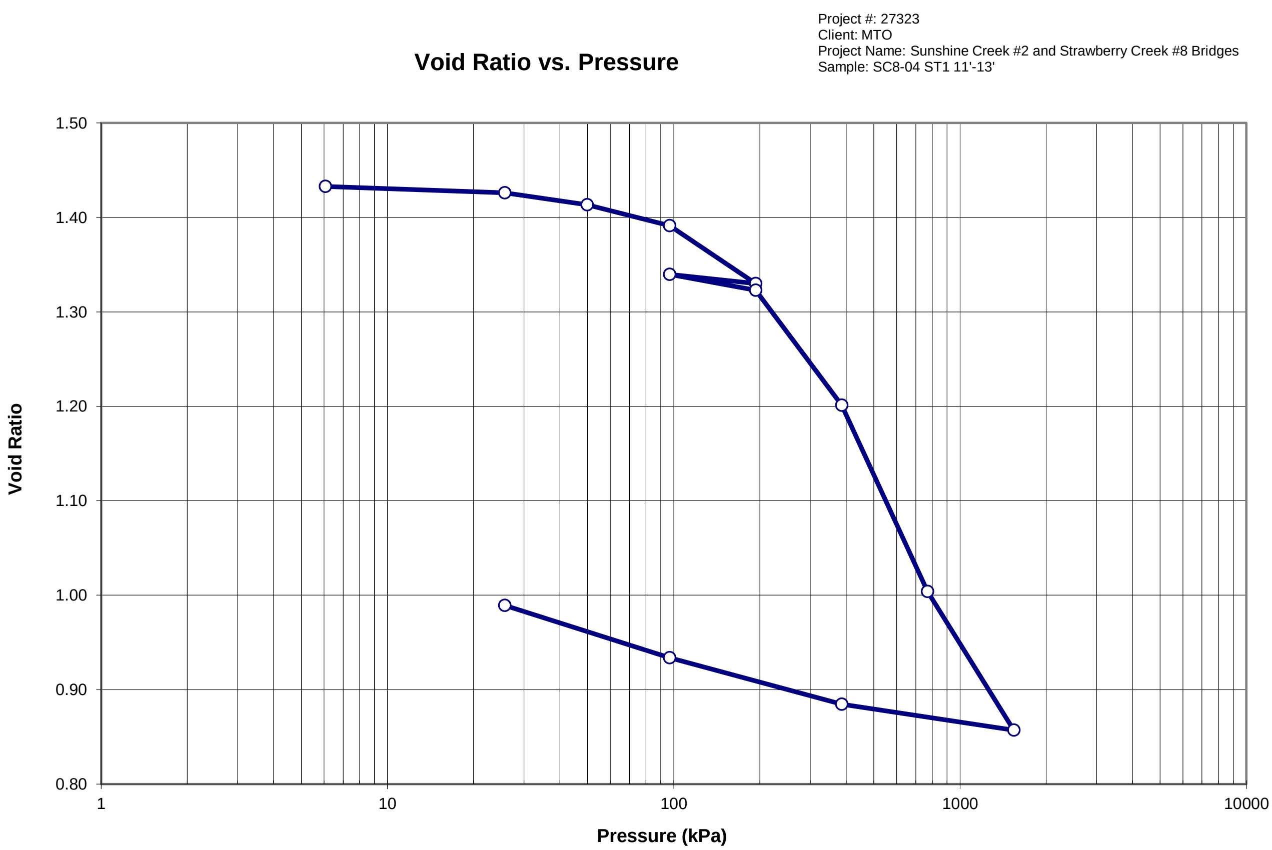
TEST DATES: November 27, 2019 - December 07, 2019

SAMPLE: SC8-04 ST1 11'-13'
Silty clay, trace sand, brown, moist.
LL=62.7, PL=24.4, I_p = 38.2

PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-11, method B

	Start of Test	End of Test
Wet Dens. (kg/m^3)	1723.9	1895.3
Dry Dens. (kg/m^3)	1155.0	1412.7
Moisture Cont. (%)	49.3	34.2
Void Ratio	1.433	0.989
Saturation (%)	96.6	

Note: A Specific Gravity (Gs) of 2.81 was obtained for the void ratio and saturation calculations.



TEST DONE BY: BT
REVIEWED BY: KS

THURBER ENGINEERING LTD.

Consolidation Test Report

Sunshine Creek #2 and Strawberry Creek #8 Bridges

27323 SC8-04 ST1 11'-13'

TRIMMING: The Specimen was manually trimmed to the size of consolidation ring, then mounted in a fixed ring consolidometer.

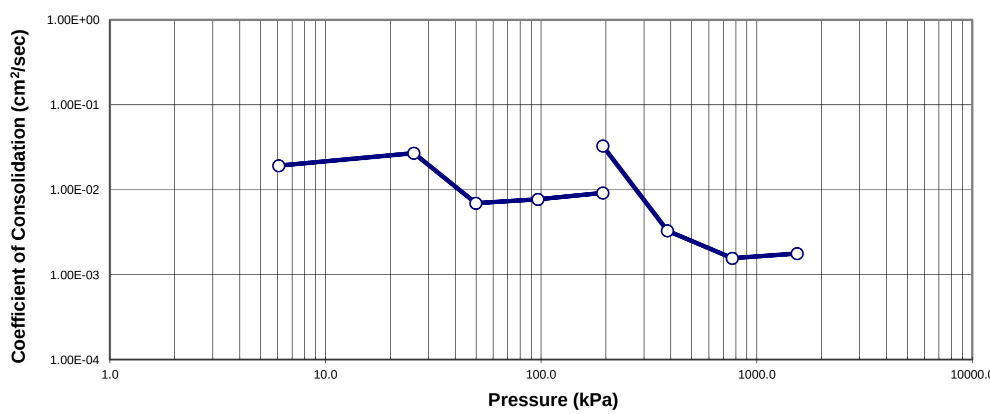
LOADING: A seating load of 6.1 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after 100% primary consolidation was reached at each load increment.

CALCULATIONS: Coefficients of Consolidation were calculated by the square root time method.

Pressure (kPa)	Corr. H. (mm)	Avg. H. (mm)	D_{90} (mm)	t_{90} (min)	c_v (cm^2/s)	Void Ratio	m_v (m^2/kN)	k (cm/s)
0.0	25.400					1.433		
6.1	25.397	25.399	-0.029	1.19	1.92E-02	1.433	1.95E-05	3.66E-08
25.7	25.328	25.363	-0.135	0.85	2.69E-02	1.426	1.39E-04	3.65E-07
49.9	25.195	25.262	-0.125	3.24	6.96E-03	1.413	2.17E-04	1.48E-07
96.6	24.965	25.080	-0.182	2.89	7.69E-03	1.391	1.95E-04	1.47E-07
193.2	24.323	24.644	-0.370	2.34	9.17E-03	1.330	2.66E-04	2.39E-07
96.6	24.424	24.374				1.340		
193.2	24.251	24.338	-0.090	0.64	3.27E-02	1.323	7.33E-05	2.35E-07
385.7	22.980	23.616	-0.615	6.00	3.28E-03	1.201	2.72E-04	8.76E-08
770.7	20.921	21.951	-1.230	10.89	1.56E-03	1.004	2.33E-04	3.57E-08
1540.7	19.387	20.154	-1.000	8.12	1.77E-03	0.857	9.52E-05	1.65E-08
385.7	19.675	19.531				0.885		
96.6	20.189	19.932				0.934		
25.7	20.767	20.478				0.989		

Coefficient of Consoildation vs. Pressure

Project #: 27323
Client: MTO
Project Name: Sunshine Creek #2 and Strawberry Creek #8 Bridges
Sample: SC8-04 ST1 11'-13'



Notes: C_v and k calculated using t_{90} values

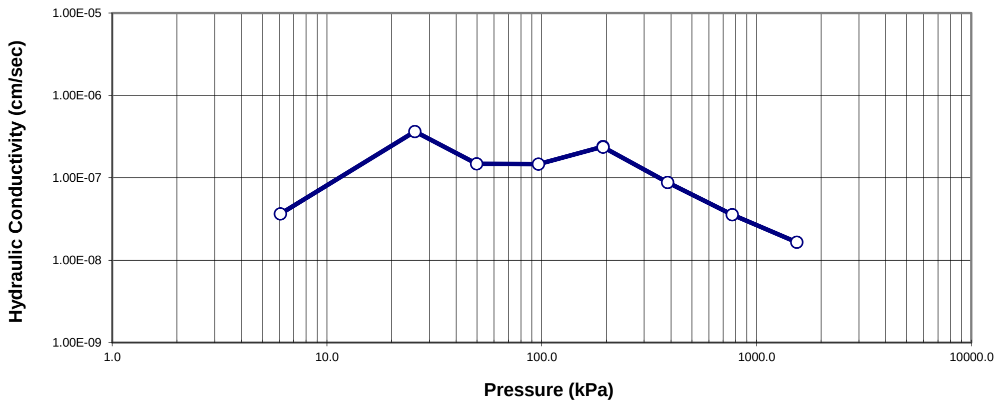
TEST DONE BY: BT
REVIEWED BY: KS

Consolidation Test Report

Sunshine Creek #2 and Strawberry Creek #8 Bridges
27323 SC8-04 ST1 11'-13'

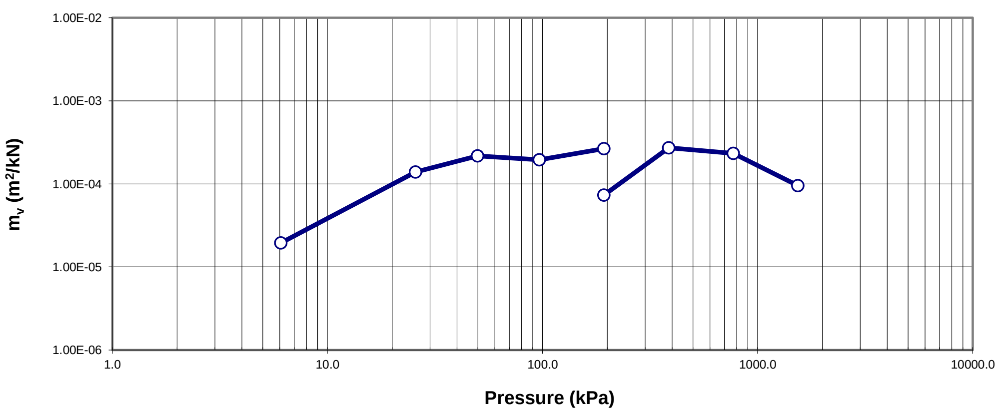
Hydraulic Conductivity vs. Pressure

Project #: 27323
Client: MTO
Project Name: Sunshine Creek #2 and Strawberry Creek #8 Bridges
Sample: SC8-04 ST1 11'-13'



m_v vs. Pressure

Project #: 27323
Client: MTO
Project Name: Sunshine Creek #2 and Strawberry Creek #8 Bridges
Sample: SC8-04 ST1 11'-13'



TEST DONE BY: BT
REVIEWED BY: KS

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	27323	**Date Drilled:**	05-Nov-19
Client:	MTO	**Date Tested:**	18-Feb-20
Project Name:	Strawberry Creek Bridge #8	**Tester:**	RG
Core Size:	NQ **BH No :** SC8-01	**Reviewed by:**	MEF

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	16.8	A	21.1	47.5	48.5	7.1	170.0	Granodiorite	Very Strong
2	1	17.6	D	17.4	47.5	66.6	7.1	171.1	Granodiorite	Very Strong
3										
4					**RUN #1 (AVERAGE) =**			170.5		Very Strong
5										
6										
7										
8										
9										
10										
11										
12										
13										
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30										

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

* Correlation factor to obtain UCS values is 24.

Last Modified: September 14, 2016

Photo B1: Borehole SC8-01 cobbles and boulders (Runs #1 to #3) and bedrock core (Run #4).

Your Project #: 27323
Your C.O.C. #: 744039-03-01

Attention: Mark Farrant

Thurber Engineering Ltd
2010 Winston Park Dr
Suite 103
Oakville, ON
CANADA L6H 5R7

Report Date: 2020/03/19
Report #: R6116722
Version: 3 - Revision

CERTIFICATE OF ANALYSIS – REVISED REPORT

BV LABS JOB #: B9X0440
Received: 2019/11/22, 15:04

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Analytical Method
Chloride (20:1 extract)	2	2019/11/27	2019/11/27	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	2	2019/11/27	2019/11/27	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 2)	2	N/A	2019/11/28	AB SOP-00002	CCME PHC-CWS m
Sulphide in Soil (1)	2	N/A	2019/12/06	AB SOP-00080	EPA9030B/SM4500S2-DF
pH CaCl2 EXTRACT	2	2019/11/25	2019/11/25	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2019/11/22	2019/11/27	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	2019/11/27	2019/11/27	CAM SOP-00464	EPA 375.4 m

Remarks:

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

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

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

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested. When sampling is not conducted by BV Labs, results relate to the supplied samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix “m” indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.
(1) This test was performed by BVLabs Calgary via Mississauga
(2) Offsite analysis requires that subcontracted moisture be reported.

Your Project #: 27323
Your C.O.C. #: 744039-03-01

Attention: Mark Farrant

Thurber Engineering Ltd
2010 Winston Park Dr
Suite 103
Oakville, ON
CANADA L6H 5R7

Report Date: 2020/03/19
Report #: R6116722
Version: 3 - Revision

CERTIFICATE OF ANALYSIS – REVISED REPORT

BV LABS JOB #: B9X0440
Received: 2019/11/22, 15:04

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Antonella Brasil, Senior Project Manager
Email: Antonella.Brasil@bvlabs.com
Phone# (905)817-5817
==
BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

SOIL CORROSIVITY PACKAGE (SOIL)

BV Labs ID		LJR499	LJR499	LJR499	LJR500		
Sampling Date		2019/11/08	2019/11/08	2019/11/08	2019/11/08		
COC Number		744039-03-01	744039-03-01	744039-03-01	744039-03-01		
	UNITS	**SC8-02, SS#3, 5'-7'**	**SC8-02, SS#3, 5'-7' Lab-Dup**	**SC8-02, SS#3, 5'-7' Lab-Dup 2**	**SC8-04, SS#1, 0'-2'**	**RDL**	**QC Batch**
Calculated Parameters							
Resistivity	ohm-cm	5400	N/A	N/A	2000	N/A	6458073
Inorganics							
Soluble (20:1) Chloride (Cl-)	ug/g	40	N/A	N/A	28	20	6465367
Conductivity	umho/cm	187	N/A	N/A	488	2	6465498
Available (CaCl2) pH	pH	6.58	N/A	N/A	10.0	N/A	6460996
Soluble (20:1) Sulphate (SO4)	ug/g	<20	N/A	N/A	330	20	6465370
Sulphide	mg/kg	<0.5 (1)	<0.5	<0.5	<0.5	0.5	6484316
Physical Testing							
Moisture-Subcontracted	%	29	N/A	N/A	13	0.30	6484315

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate
N/A = Not Applicable
(1) Matrix Spike exceeds acceptance limits due to matrix interference. Reanalysis yields similar results.

GENERAL COMMENTS

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

Package 1	8.0°C

Revised Report (2020/03/19): Split reports as per client request .

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

QA/QC Batch	Init	QC Type	Parameter	Date Analyzed	Value	Recovery	UNITS	QC Limits
6460996	KAD	Spiked Blank	Available (CaCl2) pH	2019/11/25		100	%	97 - 103
6460996	KAD	RPD	Available (CaCl2) pH	2019/11/25	0.027		%	N/A
6465367	DRM	Matrix Spike	Soluble (20:1) Chloride (Cl-)	2019/11/27		104	%	70 - 130
6465367	DRM	Spiked Blank	Soluble (20:1) Chloride (Cl-)	2019/11/27		103	%	70 - 130
6465367	DRM	Method Blank	Soluble (20:1) Chloride (Cl-)	2019/11/27	<20		ug/g	
6465367	DRM	RPD	Soluble (20:1) Chloride (Cl-)	2019/11/27	NC		%	35
6465370	ADB	Matrix Spike	Soluble (20:1) Sulphate (SO4)	2019/11/27		NC	%	70 - 130
6465370	ADB	Spiked Blank	Soluble (20:1) Sulphate (SO4)	2019/11/27		106	%	70 - 130
6465370	ADB	Method Blank	Soluble (20:1) Sulphate (SO4)	2019/11/27	<20		ug/g	
6465370	ADB	RPD	Soluble (20:1) Sulphate (SO4)	2019/11/27	2.1		%	35
6465498	KAD	Spiked Blank	Conductivity	2019/11/27		104	%	90 - 110
6465498	KAD	Method Blank	Conductivity	2019/11/27	<2		umho/cm	
6465498	KAD	RPD	Conductivity	2019/11/27	0.36		%	10
6484315	SAY	Method Blank	Moisture-Subcontracted	2019/11/28	<0.30		%	
6484316	éBS	Matrix Spike	Sulphide	2019/12/06		52 (1)	%	75 - 125
6484316	éBS	RPD	Sulphide	2019/12/06	3.0		%	30
			Sulphide	2019/12/06	10		%	30
6484316	éBS	Spiked Blank	Sulphide	2019/12/06		94	%	75 - 125
6484316	éBS	Method Blank	Sulphide	2019/12/06	<0.5		mg/kg	
6484316	éBS	RPD [LJR499-02]	Sulphide	2019/12/06	NC		%	30

N/A = Not Applicable

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

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

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

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

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

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

(1) Recovery or RPD for this parameter is outside control limits. The overall quality control for this analysis meets acceptability criteria.

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Anastassia Hamanov, Scientific Specialist

Ghayasuddin Khan, M.Sc., P.Chem., QP, Scientific Specialist, Inorganics

Veronica Falk, B.Sc., P.Chem., QP, Scientific Specialist, Organics

BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

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

22-Nov-19 15:04
Antonella Brasil
B9X0440
SWE ENV-1164

Page 1 of 1

Only:
Bottle Order #: 744039
COC #: C#744039-03-01
Project Manager: Antonella Brasil

INVOICE TO:

Company Name: #5843 Thurber Engineering Ltd
Attention: Mark Farrant
Address: 2010 Winston Park Dr Suite 103
Oakville ON L6H 5R7
Tel: (905) 829-8666 Ext: 528 Fax: (905) 829-1166
Email: mfarrant@thurber.ca

REPORT TO:

Company Name:
Attention:
Address:
Tel: Fax:
Email:

PROJECT INFORMATION:

Quotation #: B90187
P.O. #:
Project: 27323
Project Name:
Site #:
Sampled By:

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BV LABS DRINKING WATER CHAIN OF CUSTODY

Regulation 153 (2011): ☐ Table 1 ☐ Res/Park ☐ Medium/Fine ☐ Table 2 ☐ Ind/Comm ☐ Coarse ☐ Table 3 ☐ Agri/Other ☐ For RSC ☐ Table ___

Other Regulations: ☐ CCME ☐ Sanitary Sewer Bylaw ☐ Reg 558 ☐ Storm Sewer Bylaw ☐ MISA Municipality ☐ PWQO ☐ Other

Special Instructions

Include Criteria on Certificate of Analysis (Y/N)? ____

ANALYSIS REQUESTED (PLEASE BE SPECIFIC): Field Filtered (please circle): Metals / Hg / Cr VI; Corrosivity Assessment Package

Turnaround Time (TAT) Required:
Please provide advance notice for rush projects

Regular (Standard) TAT: ☒
(will be applied if Rush TAT is not specified):
Standard TAT = 5-7 Working days for most tests.
Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.

Job Specific Rush TAT (if applies to entire submission)
Date Required: Time Required:
Rush Confirmation Number: *(call lab for #)*

	Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered / Metals / Hg / Cr VI	Corrosivity Assessment Package	# of Bottles	Comments
1		SC8-02, SS#3, 5'-7'	Nov. 8/19		Soil		X	2	
2		SC8-04, SS#1, 0'-2'	Nov. 8/19		Soil		X	2	
3		SC2-01, SS#2A/2B, 2'6"-4'6"	Nov. 11/19		Soil		X	2	
4		SC2-02, SS#3, 5'-7'	Nov. 12/19		Soil		X	2	
5									
6									
7									
8									
9									
10									

* RELINQUISHED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	# jars used and not submitted
Mark Farrant	19/11/22		TRUSHNA PATEL	19/11/22; 2019/11/22	15:04	

Laboratory Use Only

Time Sensitive	Temperature (°C) on Recei	Custody Seal	Yes	No
	8 / 8 / 8	Present		✓
		Intact		✓

White: BV Labs Yellow: Client

* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO BV LABS' STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.BVLABS.COM/TERMS-AND-CONDITIONS.

* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.

** SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT WWW.BVLABS.COM/RESOURCES/CHAIN-OF-CUSTODY-FORMS.

SAMPLES MUST BE KEPT COOL (< 10° C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BV LABS

Bureau Veritas Canada (2019) Inc.

Certificate of Analysis

Client:	Bureau Veritas Canada (2019) Inc. 6740 Campobello Road Mississauga, ON L5N 2L8	Report Number:	1921583
		Date Submitted:	2019-11-26
		Date Reported:	2019-12-03
		Project:	B9X0440
Attention:	Antonella Brasil	COC #:	851753
PO#:			
Invoice to:	Bureau Veritas Canada (2019) Inc.		

Dear Antonella Brasil:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

APPROVAL: ______________________________

Sarah Horner, Inorganics Technician

All analysis is completed at Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) unless otherwise indicated.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on the scope of accreditation. The scope is available at: http://www.cala.ca/scopes/2602.pdf.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is licensed by the Ontario Ministry of the Environment, Conservation, and Parks (MECP) for specific tests in drinking water (license #2318). A copy of the license is available upon request.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by the Ontario Ministry of Agriculture, Food, and Rural Affairs for specific tests in agricultural soils.

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required. Unless otherwise stated, measurement uncertainty is not taken into account when determining guideline or regulatory exceedances.

Certificate of Analysis

eurofins | Environment Testing

Client: Bureau Veritas Canada (2019) Inc.
6740 Campobello Road
Mississauga, ON
L5N 2L8
Attention: Antonella Brasil
PO#:
Invoice to: Bureau Veritas Canada (2019) Inc.

Report Number: 1921583
Date Submitted: 2019-11-26
Date Reported: 2019-12-03
Project: B9X0440
COC #: 851753

Group	Analyte	MRL	Units	Guideline	1468740	1468741
				Lab I.D.	1468740	1468741
				Sample Matrix	Soil	Soil
				Sample Type		
				Sampling Date	2019-11-08	2019-11-08
				Sample I.D.	LJR499-SC8-02 SS#3 5'-7'	LJR500-SC8-04 SS#1 0'-2'
Redox Potential	REDOX Potential		mV		238.2	144.5

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Certificate of Analysis

Environment Testing

Client: Bureau Veritas Canada (2019) Inc.
6740 Campobello Road
Mississauga, ON
L5N 2L8

Attention: Antonella Brasil

PO#:

Invoice to: Bureau Veritas Canada (2019) Inc.

Report Number: 1921583
Date Submitted: 2019-11-26
Date Reported: 2019-12-03
Project: B9X0440
COC #: 851753

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 376921 **Analysis/Extraction Date** 2019-12-03 **Analyst** SKH **Method** C SM2580B			
REDOX Potential	197.2 mV	101	

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Your Project #: 27323
Your C.O.C. #: 745256-01-01

Attention: Mark Farrant

Thurber Engineering Ltd
2010 Winston Park Dr
Suite 103
Oakville, ON
CANADA L6H 5R7

Report Date: 2020/03/19
Report #: R6116757
Version: 2 - Revision

CERTIFICATE OF ANALYSIS – REVISED REPORT

BV LABS JOB #: B9W0881
Received: 2019/11/13, 17:10

Sample Matrix: Water
Samples Received: 1

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Analytical Method
Chloride by Automated Colourimetry	1	N/A	2019/11/18	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	1	N/A	2019/11/18	CAM SOP-00414	SM 23 2510 m
pH	1	2019/11/15	2019/11/18	CAM SOP-00413	SM 4500H+ B m
Resistivity of Water	1	2019/11/14	2019/11/19	CAM SOP-00414	SM 23 2510 m
Sulphate by Automated Colourimetry	1	N/A	2019/11/18	CAM SOP-00464	EPA 375.4 m
Sulphide	1	N/A	2019/11/18	CAM SOP-00455	SM 23 4500-S G m

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

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Antonella Brasil, Senior Project Manager
Email: Antonella.Brasil@bvlabs.com
Phone# (905)817-5817
==
BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

RESULTS OF ANALYSES OF WATER

BV Labs ID		LHP067		
Sampling Date		2019/11/10 09:30		
COC Number		745256-01-01		
	UNITS	**SC8 STRAWBERRY CREEK #8 BRIDGE**	**RDL**	**QC Batch**
Calculated Parameters				
Resistivity	ohm-cm	6400	N/A	6442476
Inorganics				
Conductivity	umho/cm	160	1.0	6445661
pH	pH	9.34	N/A	6445667
Dissolved Sulphate (SO4)	mg/L	<10 (1)	10	6446783
Sulphide	mg/L	<0.020	0.020	6448198
Dissolved Chloride (Cl-)	mg/L	<10 (1)	10	6446782

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
N/A = Not Applicable
(1) Due to the sample matrix, sample required dilution. Detection limit was adjusted accordingly.

GENERAL COMMENTS

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

Package 1	9.0°C

Revised Report (2020/03/19) : Split Reports as per client request .

Results relate only to the items tested.

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

QUALITY ASSURANCE REPORT

QA/QC Batch	Init	QC Type	Parameter	Date Analyzed	Value	Recovery	UNITS	QC Limits
6445661	SAU	Spiked Blank	Conductivity	2019/11/18		101	%	85 - 115
6445661	SAU	Method Blank	Conductivity	2019/11/18	<1.0		umho/cm	
6445661	SAU	RPD	Conductivity	2019/11/18	0.11		%	25
6445667	SAU	Spiked Blank	pH	2019/11/18		102	%	98 - 103
6445667	SAU	RPD	pH	2019/11/18	0.80		%	N/A
6446782	DRM	Matrix Spike	Dissolved Chloride (Cl-)	2019/11/18		NC	%	80 - 120
6446782	DRM	Spiked Blank	Dissolved Chloride (Cl-)	2019/11/18		103	%	80 - 120
6446782	DRM	Method Blank	Dissolved Chloride (Cl-)	2019/11/18	<1.0		mg/L	
6446782	DRM	RPD	Dissolved Chloride (Cl-)	2019/11/18	0.43		%	20
6446783	DRM	Matrix Spike	Dissolved Sulphate (SO4)	2019/11/18		99	%	75 - 125
6446783	DRM	Spiked Blank	Dissolved Sulphate (SO4)	2019/11/18		102	%	80 - 120
6446783	DRM	Method Blank	Dissolved Sulphate (SO4)	2019/11/18	<1.0		mg/L	
6446783	DRM	RPD	Dissolved Sulphate (SO4)	2019/11/18	0.0039		%	20
6448198	KAD	Matrix Spike	Sulphide	2019/11/18		108	%	80 - 120
6448198	KAD	Spiked Blank	Sulphide	2019/11/18		83	%	80 - 120
6448198	KAD	Method Blank	Sulphide	2019/11/18	<0.020		mg/L	
6448198	KAD	RPD	Sulphide	2019/11/18	3.7		%	20

N/A = Not Applicable

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

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

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

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

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

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Anastassia Hamanov, Scientific Specialist

BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

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

Page 1 of 1

13-Nov-19 17:10
Antonella Brasil
B9W0881
MIB THU-001

INVOICE TO:		REPORT TO:	
Company Name:	#5843 Thurber Engineering Ltd	Company Name:	Thurber Engineering Ltd.
Attention:	Mark Farrant	Attention:	Mark Farrant
Address:	2010 Winston Park Dr Suite 103 Oakville ON L6H 5R7	Address:	2010 Winston Park Dr. #103 Oakville, ON L6H 5R7
Tel:	(905) 829-8666 Ext: 528 Fax: (905) 829-1166	Tel:	(905) 829 8666, x528 Fax: (905) 829-1166
Email:	mfarrant@thurber.ca	Email:	mfarrant@thurber.ca

PROJECT INFORMATION:
Quotation #: B90187
P.O. #:
Project: 27323
Project Name:
Site #:
Sampled By:

Only:
Bottle Order #: 745256
Project Manager: Antonella Brasil
C#745256-01-01

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BV LABS DRINKING WATER CHAIN OF CUSTODY

Regulation 153 (2011): ☐ Table 1 ☐ Res/Park ☐ Medium/Fine ☐ Table 2 ☐ Ind/Comm ☐ Coarse ☐ Table 3 ☐ Agri/Other ☐ For RSC ☐ Table ___

Other Regulations: ☐ CCME ☐ Sanitary Sewer Bylaw ☐ Reg 558 ☐ Storm Sewer Bylaw ☐ MISA Municipality ___ ☐ PWQO ☐ Other

Special Instructions:

Include Criteria on Certificate of Analysis (Y/N)? ___

ANALYSIS REQUESTED (PLEASE BE SPECIFIC)

	Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered (please circle): Metals / Hg / Cr VI	Redox Potential	pH	Chloride by Automated Colourimetry	Sulphate by Automated Colourimetry	Resistivity of Water	Sulphide	Corrosive Package				# of Bottles	Comments
1	SC8	Strawberry Creek #8 Bridge	Nov. 10, 19	9:30 AM	water								/				3	
2	SC2	Sandstone Creek #2 Bridge	Nov. 13, 19	2:00 PM	water								/				3	
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		

Turnaround Time (TAT) Required:
Please provide advance notice for rush projects

Regular (Standard) TAT: ☐
(will be applied if Rush TAT is not specified):
Standard TAT = 5-7 Working days for most tests.
Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.

Job Specific Rush TAT (if applies to entire submission)
Date Required: ___ Time Required: ___ ☐
Rush Confirmation Number: ___ *(call lab for #)*

* RELINQUISHED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	# jars used and not submitted
Kevin Kwan	19/11/13	5:10 PM	Michelle Huth	2019/11/13	17:10	
			[illegible]	2019/11/15	10:38	

Laboratory Use Only

Time Sensitive	Temperature (°C) on Recei	Custody Seal	Yes	No
	9/9/9 °C	Present		✓
		Intact		N/A

RECEIVED
RC-TBay

White: BV Labs Yellow: Client

* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO BV LABS' STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.BVLABS.COM/TERMS-AND-CONDITIONS.

* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.

** SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT WWW.BVLABS.COM/RESOURCES/CHAIN-OF-CUSTODY-FORMS.

SAMPLES MUST BE KEPT COOL (< 10° C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BV LABS

0/0/0 ice

Certificate of Analysis

Environment Testing

Client:	Bureau Veritas Canada (2019) Inc. 6740 Campobello Road Mississauga, ON L5N 2L8	Report Number:	1921179
		Date Submitted:	2019-11-19
		Date Reported:	2019-11-22
		Project:	B9W0881
		COC #:	851547
Attention:	Antonella Brasil		
PO#:			
Invoice to:	Bureau Veritas Canada (2019) Inc.		

Dear Antonella Brasil:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

APPROVAL: ______________________________

Sarah Horner, Inorganics Technician

All analysis is completed at Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) unless otherwise indicated.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on the scope of accreditation. The scope is available at: http://www.cala.ca/scopes/2602.pdf.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is licensed by the Ontario Ministry of the Environment, Conservation, and Parks (MECP) for specific tests in drinking water (license #2318). A copy of the license is available upon request.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by the Ontario Ministry of Agriculture, Food, and Rural Affairs for specific tests in agricultural soils.

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required. Unless otherwise stated, measurement uncertainty is not taken into account when determining guideline or regulatory exceedances.

eurofins | Environment Testing

Certificate of Analysis

Client:	Bureau Veritas Canada (2019) Inc. 6740 Campobello Road Mississauga, ON L5N 2L8	Report Number:	1921179
Attention:	Antonella Brasil	Date Submitted:	2019-11-19
PO#:		Date Reported:	2019-11-22
Invoice to:	Bureau Veritas Canada (2019) Inc.	Project:	B9W0881
		COC #:	851547

Group	Analyte	MRL	Units	Guideline	Lab I.D.: 1467134 Sample Matrix: Water Sample Type: Sampling Date: 2019-11-10 Sample I.D.: LH067-SC8 STRAWBERRY CREEK #8 BRIDGE	
Redox Potential	REDOX Potential		mV		186.6	

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Certificate of Analysis

eurofins | Environment Testing

Client: Bureau Veritas Canada (2019) Inc.
6740 Campobello Road
Mississauga, ON
L5N 2L8
Attention: Antonella Brasil
PO#:
Invoice to: Bureau Veritas Canada (2019) Inc.

Report Number: 1921179
Date Submitted: 2019-11-19
Date Reported: 2019-11-22
Project: B9W0881
COC #: 851547

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 376353 **Analysis/Extraction Date** 2019-11-21 **Analyst** SKH **Method** C SM2580B			
REDOX Potential	212.7 mV	100	

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

THURBER

Appendix C

Site Photographs

Photo C1: Existing Strawberry Creek #8 bridge looking south along Ware Road North.

(Date taken: November 4, 2019)

Photo C2: Existing Strawberry Creek #8 bridge looking south along Ware Road North.

(Date taken: November 10, 2019)

THURBER

Photo C3: South approach to existing bridge looking north along Ellis Road.

(Date taken: November 10, 2019)

THURBER

Photo C4: Looking north at east side of bridge and northeast embankment from southeast side.

(Date taken: November 10, 2019)

THURBER

Photo C5: Looking south at northwest embankment and west side of bridge.

(Date taken: November 4, 2019)

Photo C6: Looking north at southeast embankment and east side of bridge.

(Date taken: November 10, 2019)

Photo C7: Looking north at southwest embankment and west side of bridge.

(Date taken: November 10, 2019)

THURBER

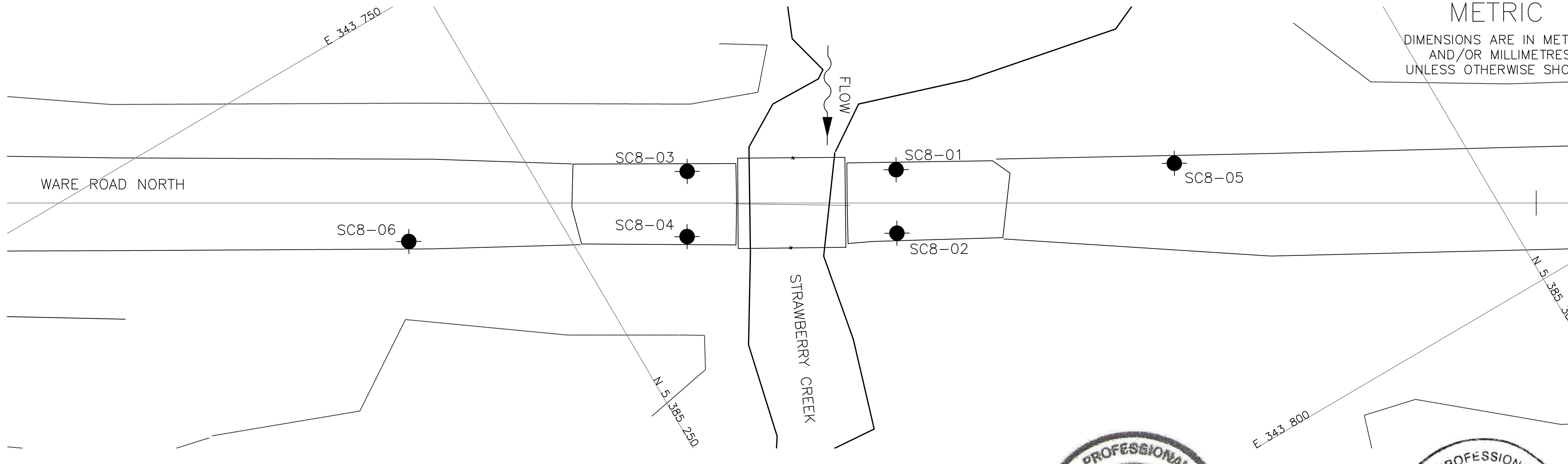
Appendix D

Borehole Locations and Soil Strata Drawing

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

CONT No GWP No 6014–18–00	
STRAWBERRY CREEK BRIDGE #8 REPLACEMENT BOREHOLE LOCATIONS AND SOIL STRATA	SHEET

THURBER ENGINEERING LTD.

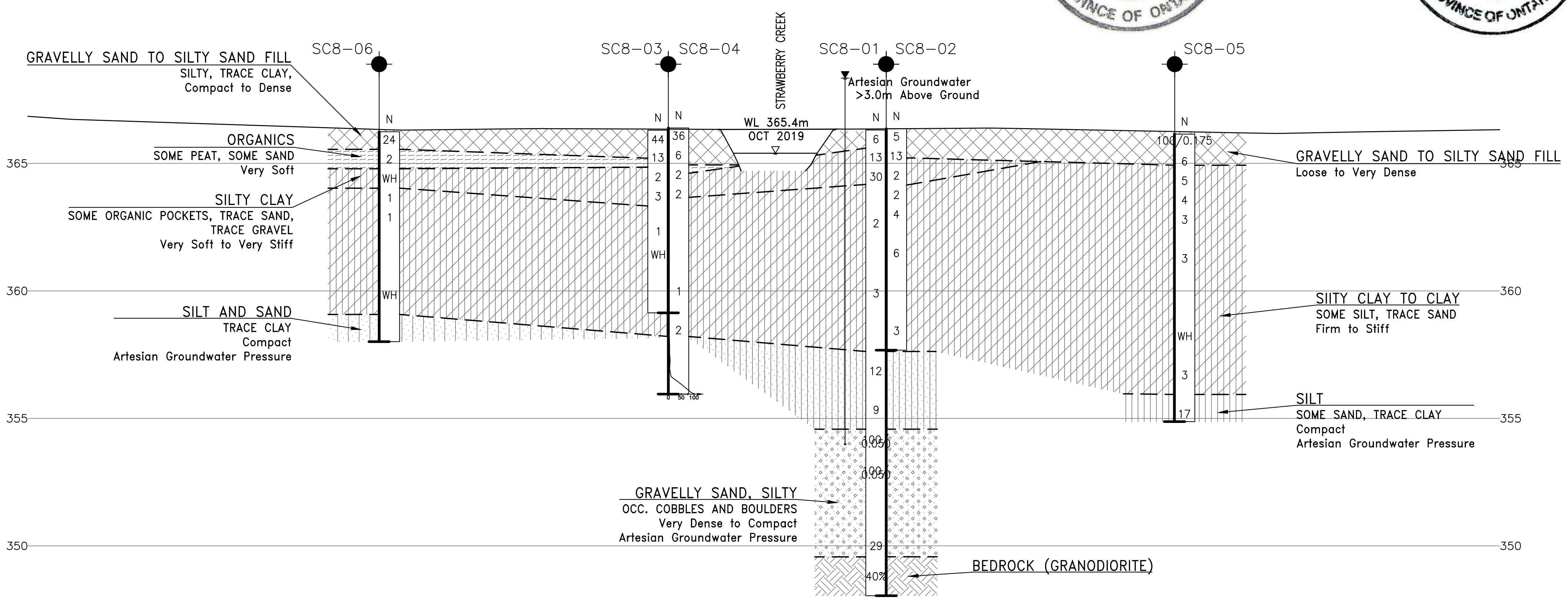


PLAN

6 0 6 12m
SCALE 1:300

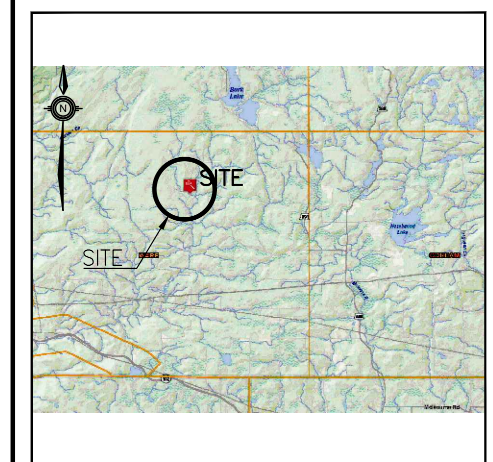
LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
June 5/2020
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
M.E. FARRANT
100053767
June 5/20
PROVINCE OF ONTARIO



PROFILE ALONG WARE ROAD NORTH

6 0 6 12m H 1:300
4 0 4 8m V 1:200



KEYPLAN

LEGEND

	Borehole
	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
SC8–01	366.31	5 385 269.3	343 774.2
SC8–02	366.36	5 385 267.4	343 777.5
SC8–03	366.30	5 385 258.2	343 767.8
SC8–04	366.39	5 385 256.2	343 771.2
SC8–05	366.15	5 385 284.2	343 782.5
SC8–06	366.24	5 385 241.4	343 762.9

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
3) Coordinate system is MTM NAD 83 Zone 15.

GEOCRES No. 52A-250

REVISIONS

DATE	BY	DESCRIPTION

DESIGN MEF	CHK PKC	CODE	LOAD	DATE JUN 2020
DRAWN BH	CHK MEF	SITE 48W–054	STRUCT	DWG 1

FILENAME: H:\Drafting\27000\27323\TED–27323–BHPL–SC8.dwg
PLOTDATE: 6/10/2020 1:48 PM

MINISTRY OF TRANSPORTATION, ONTARIO

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

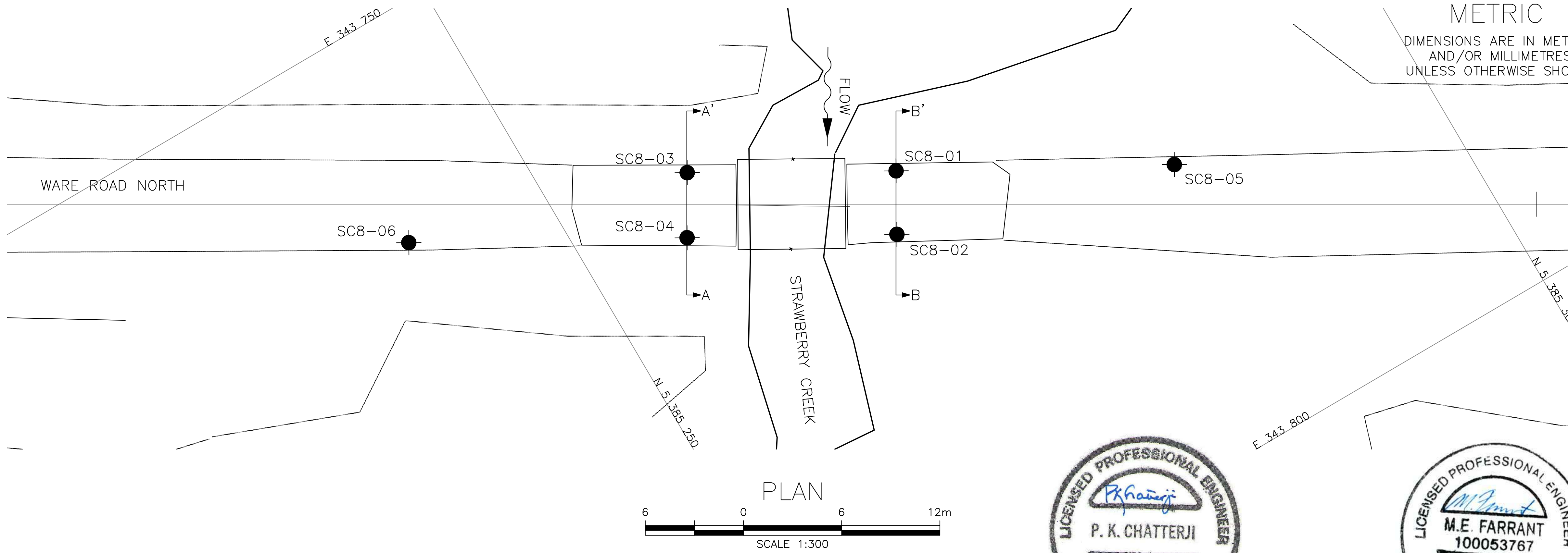
CONT No
GWP No 6014-18-00

STRAWBERRY CREEK BRIDGE #8 REPLACEMENT

BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

THURBER ENGINEERING LTD.

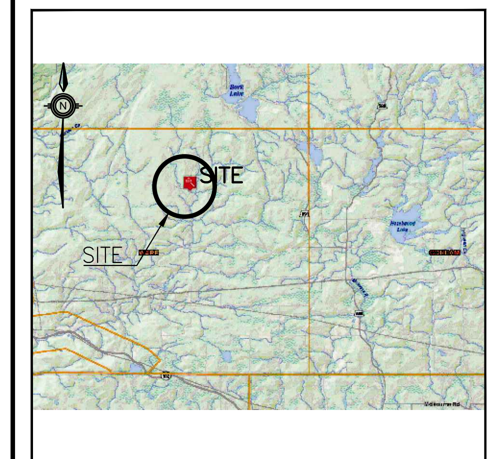


PLAN

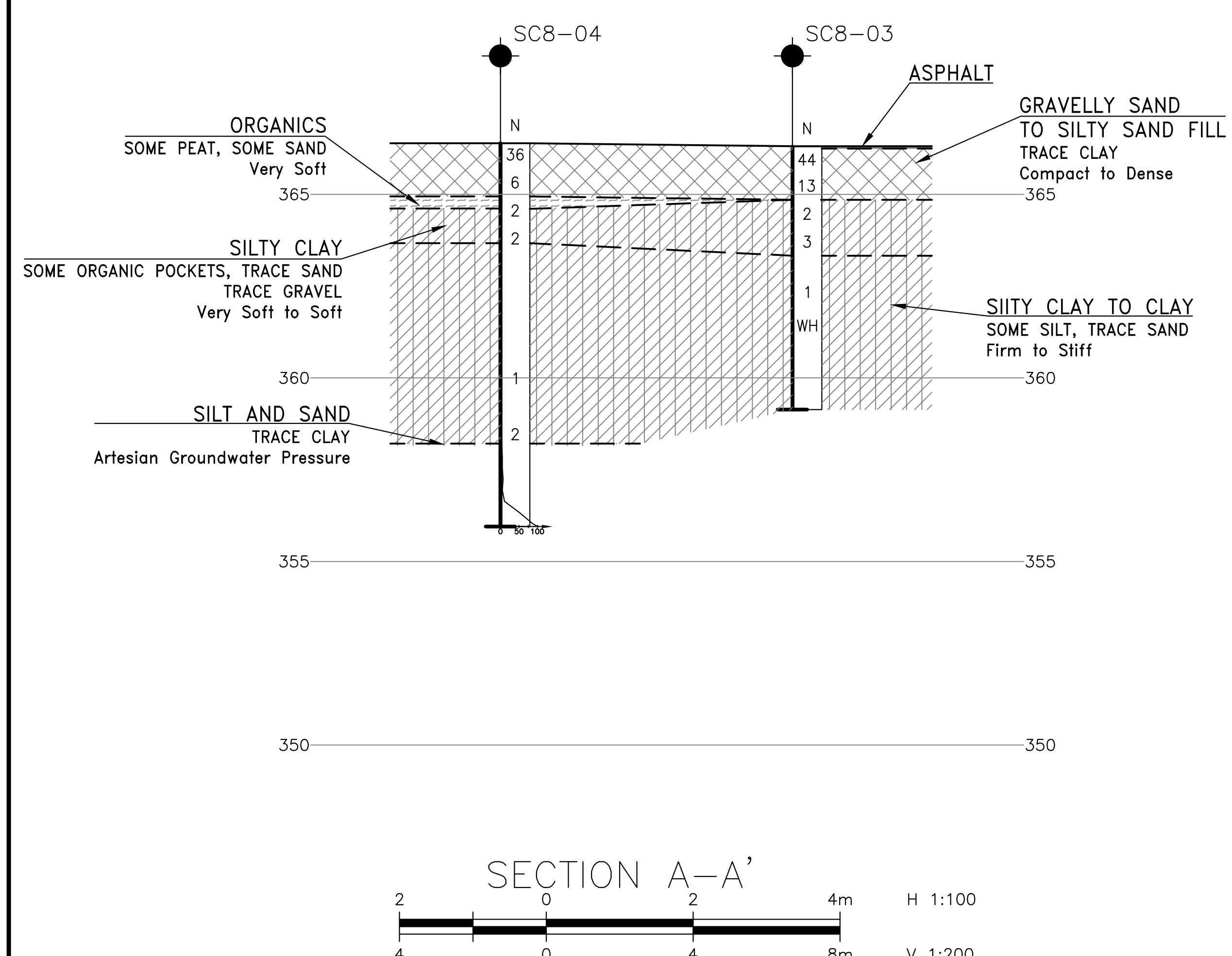
SCALE 1:300

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
June 5/2020
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
M.E. FARRANT
100053767
June 5/20
PROVINCE OF ONTARIO

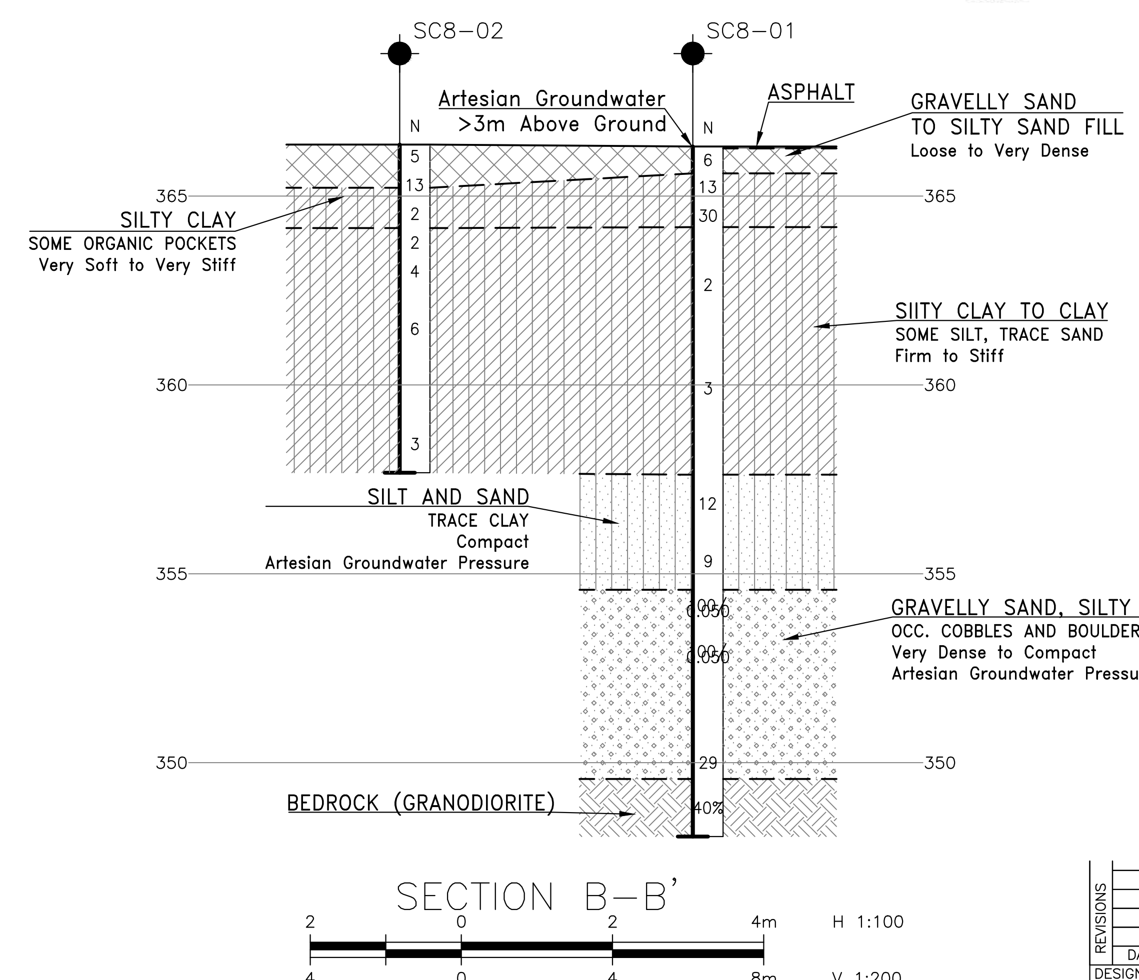


KEYPLAN



SECTION A–A'

H 1:100
V 1:200



SECTION B–B'

H 1:100
V 1:200

LEGEND

Symbol	Description
(borehole symbol)	Borehole
(borehole and cone symbol)	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
∇	Water Level
(artesian symbol)	Head Artesian Water
(piezometer symbol)	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
SC8–01	366.31	5 385 269.3	343 774.2
SC8–02	366.36	5 385 267.4	343 777.5
SC8–03	366.30	5 385 258.2	343 767.8
SC8–04	366.39	5 385 256.2	343 771.2
SC8–05	366.15	5 385 284.2	343 782.5
SC8–06	366.24	5 385 241.4	343 762.9

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
3) Coordinate system is MTM NAD 83 Zone 15.

GEOCRES No. 52A-250

REVISIONS | DATE | BY | DESCRIPTION

DESIGN MEF	CHK PKC	CODE	LOAD	DATE JUN 2020	
DRAWN BH	CHK MEF	SITE 48W–054	STRUCT	DWG 2	

FILENAME: H:\Drafting\27000\27323\TED-27323-BHPL-SC8.dwg
PLOTDATE: 6/10/2020 1:48 PM

Appendix E

Foundation Comparison

THURBER

COMPARISON OF FOUNDATION ALTERNATIVES

Concrete Spread Footings on Engineered Fill Pads	Steel H-Piles Driven to Silty Clay / Sand and Silt
Advantages: i. Generally lower cost than pile foundations. ii. Can accommodate some settlement for a modular bridge iii. Little to no risk of encountering artesian groundwater pressure ***Disadvantages:*** i. Granular fill pads must be protected from scour and erosion. **RECOMMENDED**	***Advantages:*** i. Minimal excavation or dewatering required. ***Disadvantages:*** i. Higher unit cost than footings. ii. Some settlement may occur for piles in silty clay iii. Risk of encountering artesian groundwater pressure in the sand and silt which could travel up the pile shafts, reduce the geotechnical resistance and cause settlement **NOT RECOMMENDED**

Appendix F

List of OPSSs and OPSDs and Suggested Wording for NSSP

THURBER

1. List of OPSS and OPSD Documents Relevant to this Project

- OPSS PROV 206 (Construction Specification for Grading)
- OPSS PROV 209 (Construction Specification for Embankments over Swamps and Compressible Soils)
- OPSS PROV 501 (Construction Specification for Compacting)
- OPSS PROV 517 (Construction Specification for Dewatering)
- Special Provision No. FOUN0003 to OPSS 902 (Dewatering Structure Excavations)
- OPSS PROV 804 (Construction Specification for Seed and Cover)
- OPSS 902 (Construction Specification for Excavating and Backfilling – Structures)
- OPSS PROV 903 (Construction Specification for Deep Foundations)
- OPSS PROV 1010 (Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material)
- OPSD 3090.100 (Foundation Frost Depths for Northern Ontario)
- OPSD 3101.150 (Walls Abutment, Backfill Minimum Granular Requirements)

2. Suggested Wording for NSSPs

- **“Dewatering”**

Effective dewatering shall be designed and provided by the Contractor during structure excavation, footing placement and backfilling to allow the work to proceed in the dry. Excavation below the creek and groundwater level will lead to subgrade softening. The dewatering system must be effective to maintain the water level at a minimum depth of 0.5 m below the final subgrade level throughout construction.

The dewatering system is to be designed in accordance with SP FOUN0003 and OPSS.PROV.517. A preconstruction survey is not required, thus Designer Fill-In ** in SP FOUN0003 should be “N/A”. Special Provision FOUN0003 is included below.

Appendix G

Slope Stability Analysis Figures

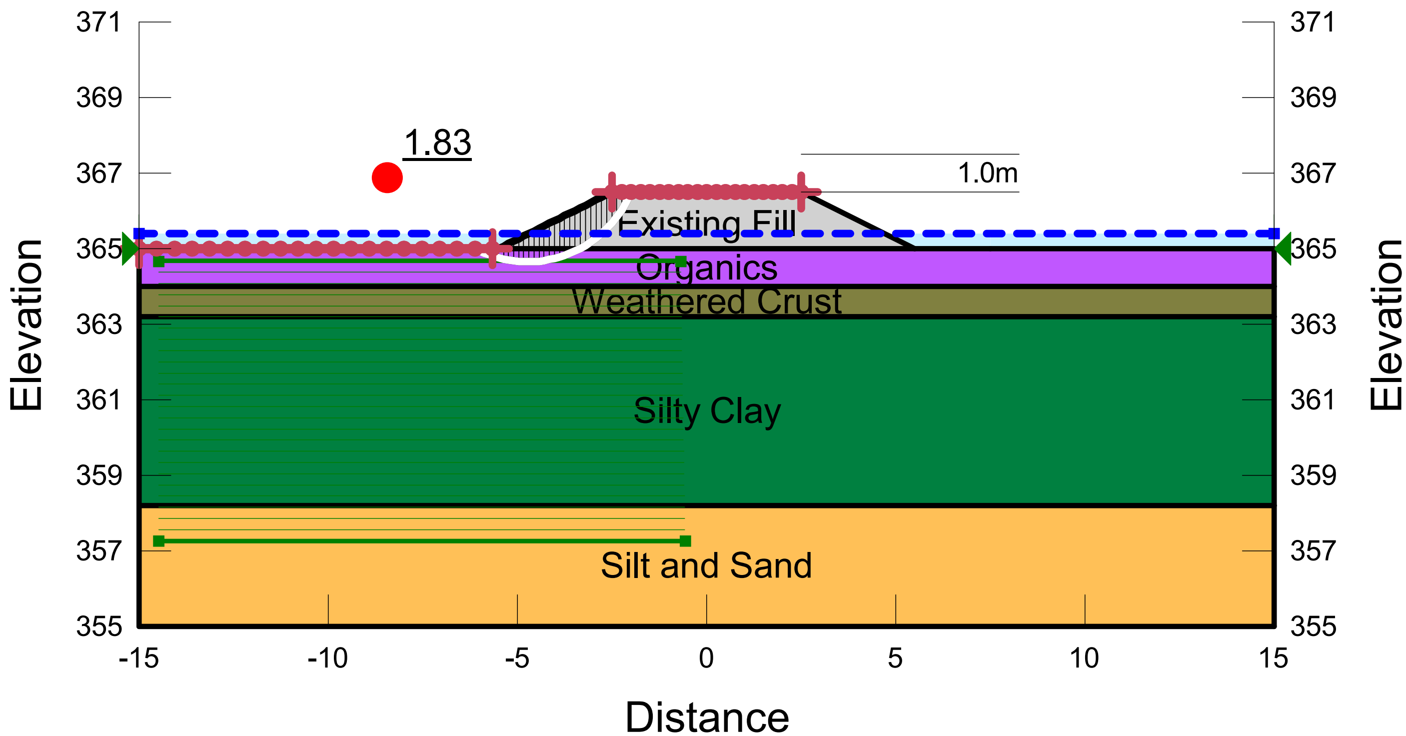
STRAWBERRY CREEK BRIDGE #8
EMBANKMENT SIDE SLOPE
EXISTING CONDITION

FIGURE 1

File: 27323 Existing Side Slope -drained.gsz
Last Edited By: Plaxis3Station
Date: 04/02/2020

Method: Morgenstern-Price
Minimum Slip Surface Depth: 1
Horz Seismic Coef.: 0

Name	Unit Weight	Cohesion'	Phi'
Existing Fill	21	0	30
Organics	14	4	25
Silt and Sand	21	0	30
Silty Clay	16	3	26
Weathered Crust	17	5	26



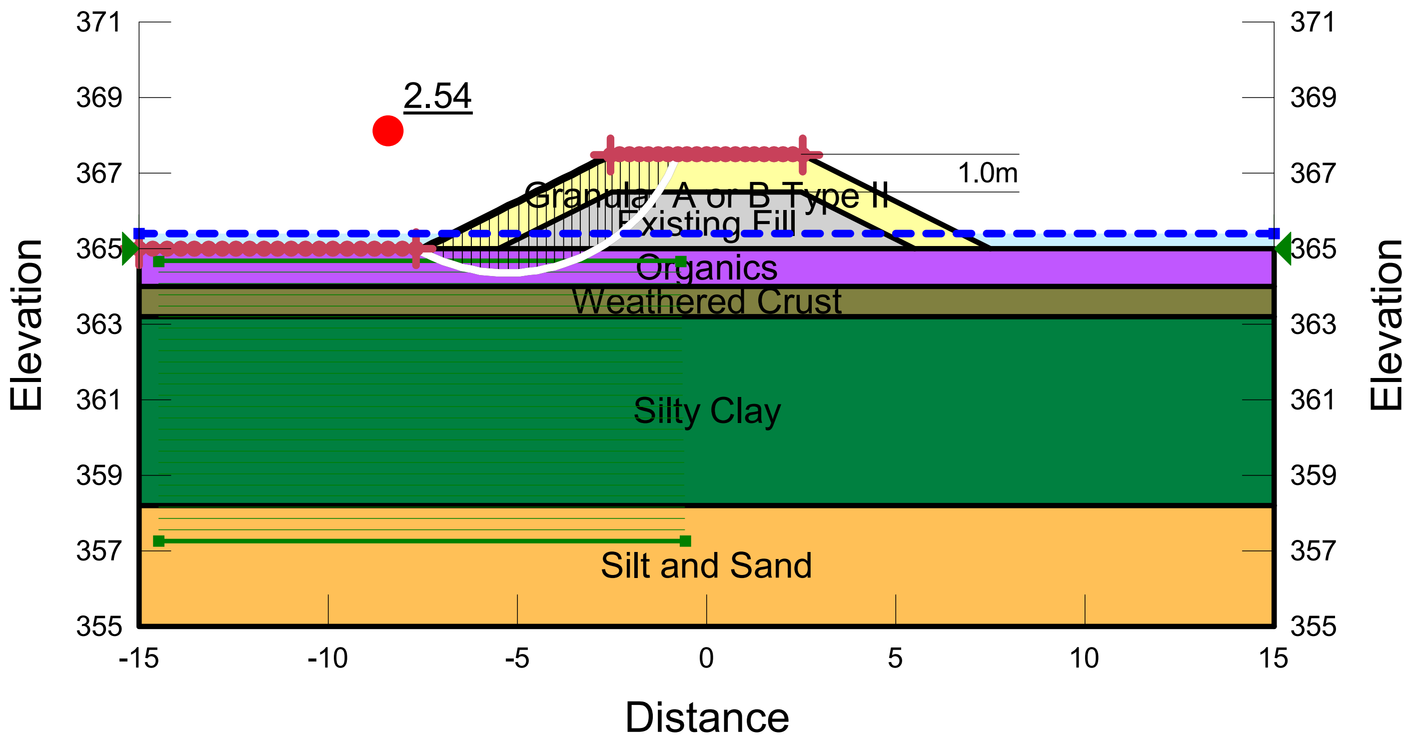
STRAWBERRY CREEK BRIDGE #8
EMBANKMENT SIDE SLOPE
1 M GRADE RAISE - UNDRAINED CONDITION

FIGURE 2

File: 27323 Side Slope -1.0m grade raise -undrained.gsz
Last Edited By: Plaxis3Station
Date: 04/02/2020

Method: Morgenstern-Price
Minimum Slip Surface Depth: 1
Horz Seismic Coef.: 0

Name	Unit Weight	C-Top of Layer	C-Maximum	Cohesion	Cohesion'	Phi'
Existing Fill	21				0	30
Granular A or B Type II	22				0	35
Organics	14			25		
Silt and Sand	21				0	30
Silty Clay	16			35		
Weathered Crust	17	85	35			



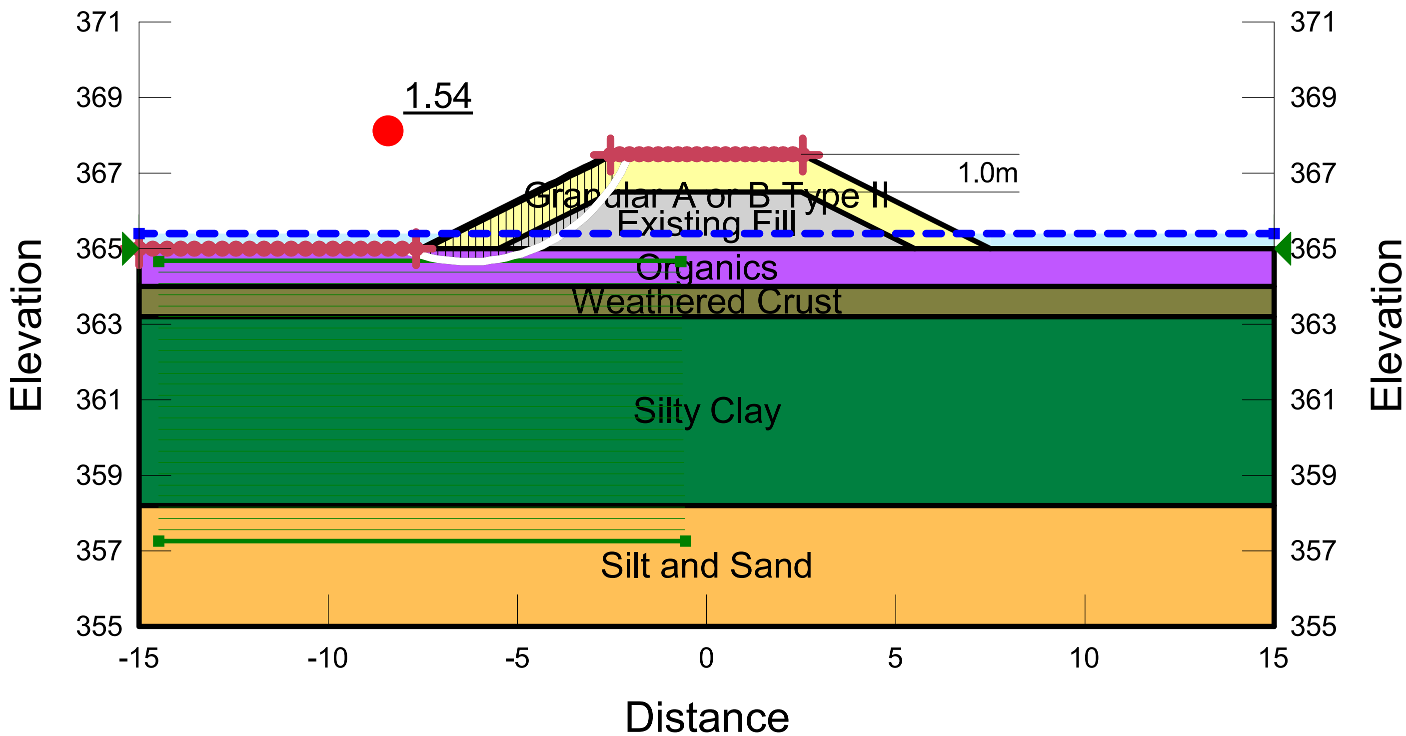
STRAWBERRY CREEK BRIDGE #8
EMBANKMENT SIDE SLOPE
1 M GRADE RAISE - DRAINED CONDITION

FIGURE 3

File: 27323 Side Slope -1.0m grade raise -drained.gsz
Last Edited By: Plaxis3Station
Date: 04/02/2020

Method: Morgenstern-Price
Minimum Slip Surface Depth: 1
Horz Seismic Coef.: 0

Name	Unit Weight	Cohesion'	Phi'
Existing Fill	21	0	30
Granular A or B Type II	22	0	35
Organics	14	4	25
Silt and Sand	21	0	30
Silty Clay	16	3	26
Weathered Crust	17	5	26



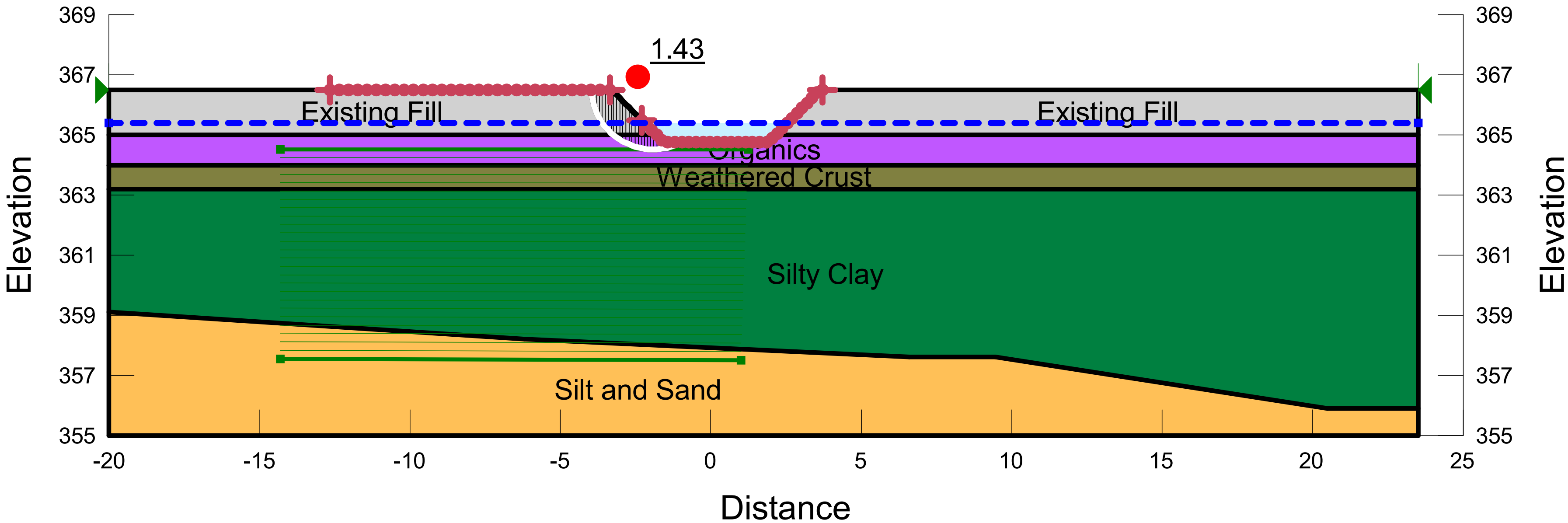
STRAWBERRY CREEK BRIDGE #8
EMBANKMENT SOUTH FORWARD SLOPE
EXISTING CONDITION

FIGURE 4

File Name: 27323 Existing SouthForward Slope -drained.gsz
Last Edited By: Plaxis3Station
Date: 04/02/2020

Method: Morgenstern-Price
Minimum Slip Surface Depth: 1
Horz Seismic Coef.: 0

Name	Unit Weight	Cohesion'	Phi'
Existing Fill	21	0	30
Organics	14	4	25
Silt and Sand	21	0	30
Silty Clay	16	3	26
Weathered Crust	17	5	26



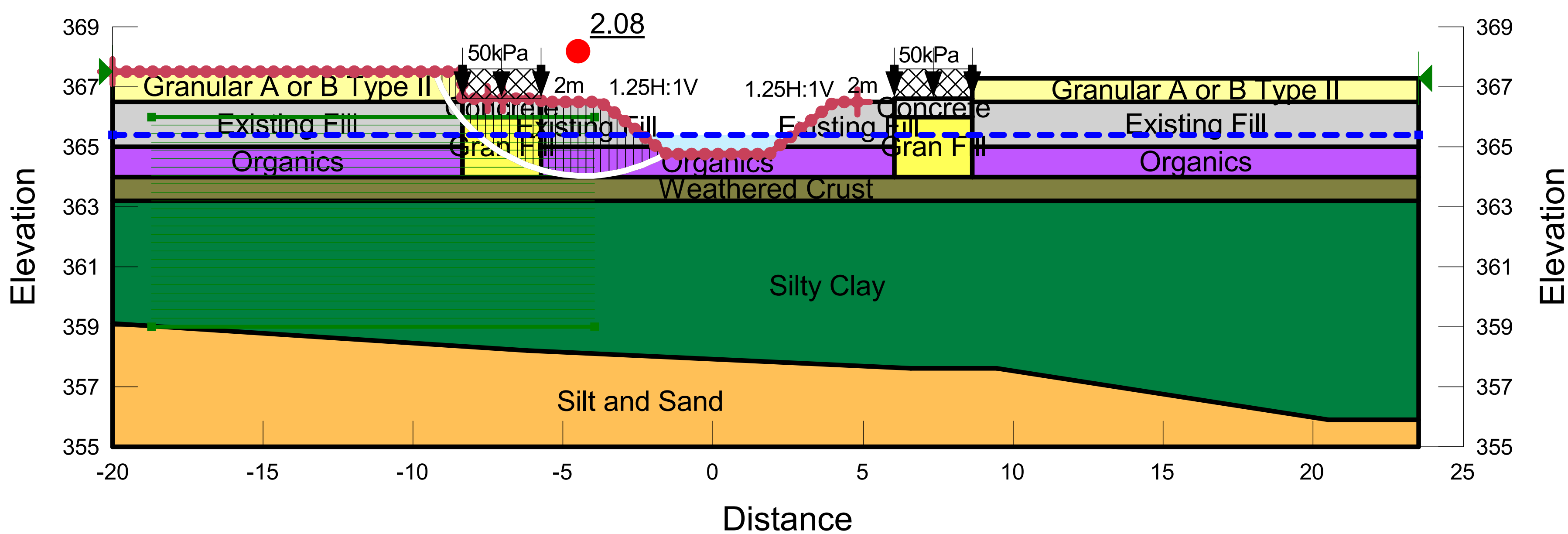
STRAWBERRY CREEK BRIDGE #8
EMBANKMENT SOUTH FORWARD SLOPE
1 M GRADE RAISE - UNDRAINED CONDITION

FIGURE 5

File Name: 27323 South Forward Slope -1.0m grade raise -undrained.gsz
Last Edited By: Plaxis3Station
Date: 04/02/2020

Method: Morgenstern-Price
Minimum Slip Surface Depth: 1
Horz Seismic Coef.: 0

Name	Unit Weight	C-Top of Layer	C-Maximum	Cohesion	Cohesion'	Phi'
Concrete	24				1,000	0
Existing Fill	21				0	30
Gran Fill	22				0	35
Granular A or B Type II	22				0	35
Organics	14			25		
Silt and Sand	21				0	30
Silty Clay	16			35		
Weathered Crust	17	85	35			



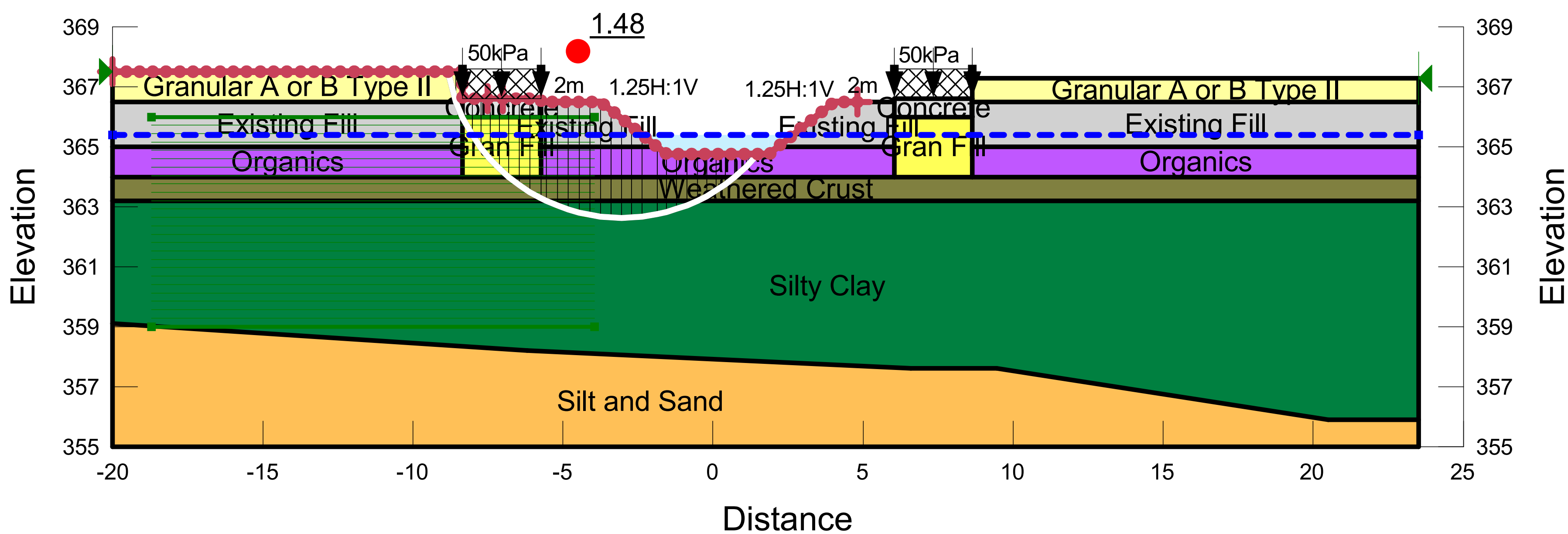
STRAWBERRY CREEK BRIDGE #8
EMBANKMENT SOUTH FORWARD SLOPE
1 M GRADE RAISE - DRAINED CONDITION

FIGURE 6

File Name: 27323 South Forward Slope -1.0m grade raise -drained.gsz
Last Edited By: Plaxis3Station
Date: 04/02/2020

Method: Morgenstern-Price
Minimum Slip Surface Depth: 1
Horz Seismic Coef.: 0

Name	Unit Weight	Cohesion'	Phi'
Concrete	24	1,000	0
Existing Fill	21	0	30
Gran Fill	22	0	35
Granular A or B Type II	22	0	35
Organics	14	4	25
Silt and Sand	21	0	30
Silty Clay	16	3	26
Weathered Crust	17	5	26



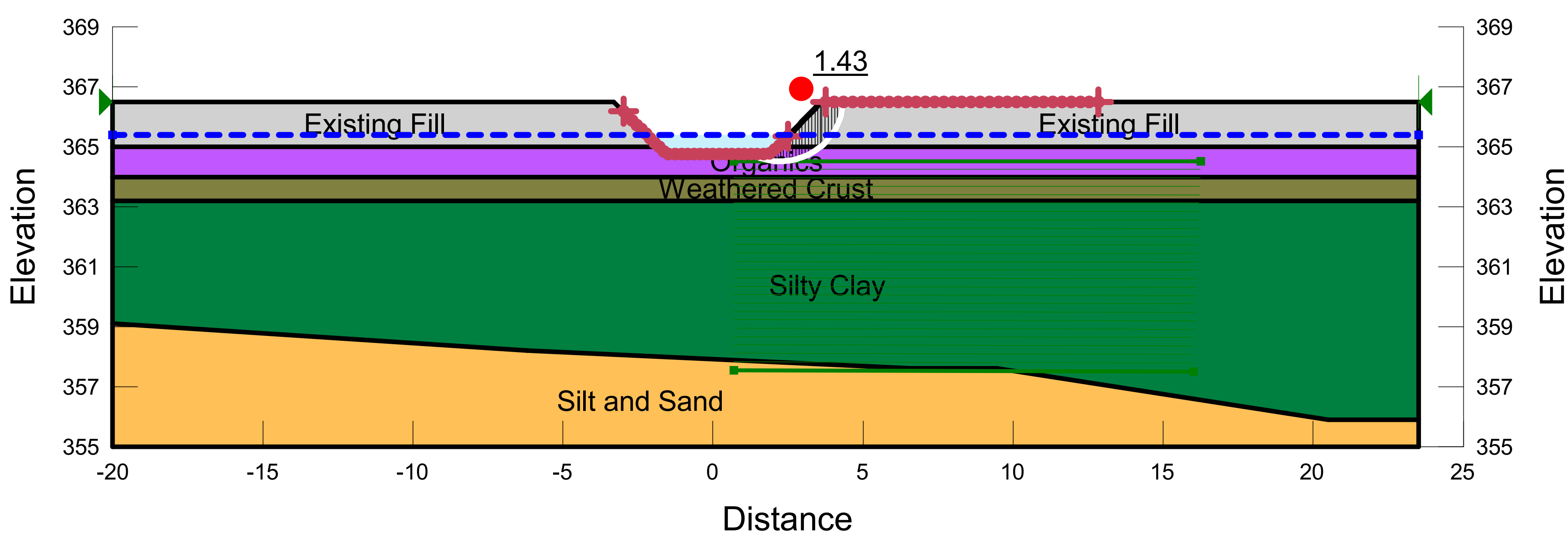
STRAWBERRY CREEK BRIDGE #8
EMBANKMENT NORTH FORWARD SLOPE
EXISTING CONDITION

FIGURE 7

File Name: 27323 Existing North Forward Slope -drained.gsz
Last Edited By: Plaxis3Station
Date: 04/02/2020

Method: Morgenstern-Price
Minimum Slip Surface Depth: 1
Horz Seismic Coef.: 0

Name	Unit Weight	Cohesion'	Phi'
Existing Fill	21	0	30
Organics	14	4	25
Silt and Sand	21	0	30
Silty Clay	16	3	26
Weathered Crust	17	5	26



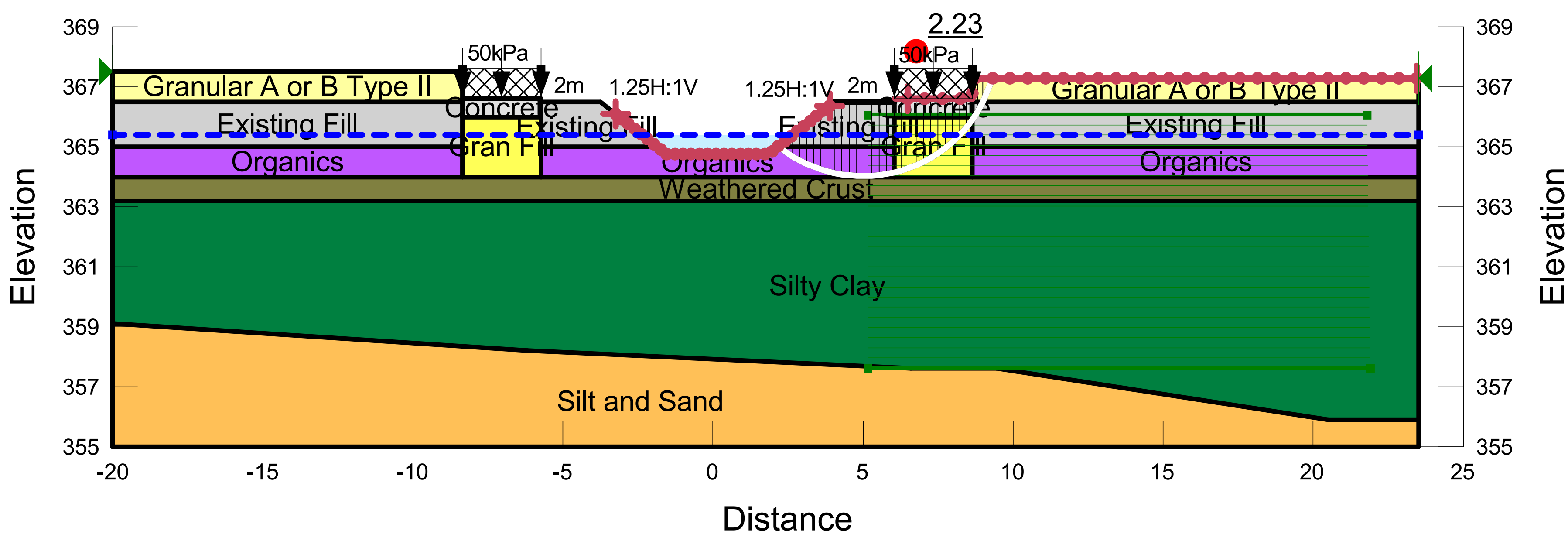
STRAWBERRY CREEK BRIDGE #8
EMBANKMENT NORTH FORWARD SLOPE
1 M GRADE RAISE - UNDRAINED CONDITION

FIGURE 8

File Name: 27323 North Forward Slope -1.0m grade raise -undrained.gsz
Last Edited By: Plaxis3Station
Date: 04/02/2020

Method: Morgenstern-Price
Minimum Slip Surface Depth: 1
Horz Seismic Coef.: 0

Name	Unit Weight	C-Top of Layer	C-Maximum	Cohesion	Cohesion'	Phi'
Concrete	24				1,000	0
Existing Fill	21				0	30
Gran Fill	22				0	35
Granular A or B Type II	22				0	35
Organics	14			25		
Silt and Sand	21				0	30
Silty Clay	16			35		
Weathered Crust	17	85	35			



STRAWBERRY CREEK BRIDGE #8
EMBANKMENT NORTH FORWARD SLOPE
1 M GRADE RAISE - DRAINED CONDITION

FIGURE 9

File Name: 27323 North Forward Slope -1.0m grade raise -drained.gsz
Last Edited By: Plaxis3Station
Date: 04/02/2020

Method: Morgenstern-Price
Minimum Slip Surface Depth: 1
Horz Seismic Coef.: 0

Name	Unit Weight	Cohesion'	Phi'
Concrete	24	1,000	0
Existing Fill	21	0	30
Gran Fill	22	0	35
Granular A or B Type II	22	0	35
Organics	14	4	25
Silt and Sand	21	0	30
Silty Clay	16	3	26
Weathered Crust	17	5	26