



Englobe

Soils Materials Environment

**Submitted To AECOM Canada Ltd.
189 Wyld Street Suite 103, North Bay, Ontario P1B 1Z2
On Behalf of the Ontario Ministry of Transportation**

**Highway 144 Rehabilitation - GWP 5223-14-00
Additional Foundation Investigation
Bridge Replacement – Site No. 46-051
Whitson River Bridge**

FINAL ADDITIONAL FOUNDATION INVESTIGATION AND DESIGN REPORT

Date: February 4, 2016
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Prepared by:

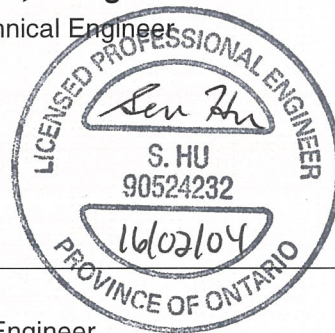

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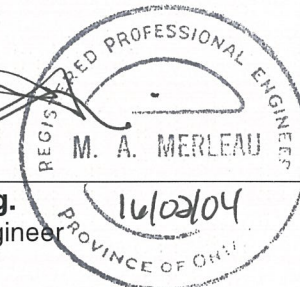


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1 INTRODUCTION

LVM-Merlex, a Division of EnGlobe Corp. (now known as Englobe Corp.) has been retained by AECOM Canada Ltd., on behalf of the Ministry of Transportation of Ontario (MTO), to carry out an additional foundation investigation at the Whitson River Bridge. The bridge is located on Highway 144, some 20 km north of Highway 17, in the Township of Balfour. The existing bridge is a single span concrete girder structure some 25 m in length.

An initial investigation at this site was carried out by Englobe in 2013 to supply subsurface data for the design of a protection system to be implemented at the Whitson River Bridge for the proposed rehabilitation. The results of the initial investigation were supplied in the Final Foundation Investigation and Design Report, Geocres No. 41I-304 (LVM-Merlex Reference No. 12/11/12218), dated March 21, 2014. Following submission of the final report for the initial foundation investigation, it is understood that the bridge structure has been further reviewed and it has been decided that the existing bridge will be replaced rather than rehabilitated.

The initial and additional foundation investigation location was specified by the MTO. The terms of reference for the scope of work are outlined in LVM-Merlex's Proposal for Foundation Engineering 12218-144-R1, dated June 2, 2014. The purpose of this additional investigation was to determine the subsurface conditions in the area of the bridge abutments in order to provide design recommendations for the proposed new abutments as well as to provide subsurface data for the foundation design of a proposed detour and temporary ACROW Modular type structure. Englobe investigated the foundations area by the drilling of boreholes, carrying out in-situ tests, and performing laboratory testing on select samples.

2 SITE DESCRIPTION

The Whitson River Bridge is located on Highway 144, between Stations 12+419.5 to 12+444.5, Township of Balfour (Site No. 46-051). The topography at the site is generally of low relief. The existing highway embankment currently supports two undivided lanes of highway, locally running in an east-west direction. Whitson River flows from north to south at the bridge location. A visual review of the highway at the east and west approaches indicates that, in general, the approaches are in poor condition with significant cracking of the asphalt pavement structure.

The existing 25 m single span concrete girder bridge was constructed in 1961 and rehabilitated in 1995 on the existing highway alignment. It is understood that the structure is in poor service condition.

Infrastructure at the bridge location consists of overhead wires on the north and south sides of the highway. Several buried services are present to the north and the south of the existing embankments.

2.1 SITE PHYSIOGRAPHY AND SURFICIAL GEOLOGY

This project is located in the Geomorphic Sub-province known as the Eastern Sandy Uplands. The topography along this section of Highway 144 is generally flat to slightly rolling. Within the specific project area overburden consists primarily of sands with silts overlying silty clays overlying bedrock.

Bedrock in the area, as indicated on OGS Map 2506, is of the Middle Precambrian Animikie Group which consists of sandstone, shale, argillite, iron formation, tuff, basalt, and limestone. The map of Bedrock Geology of Ontario published by OGS (MRD126 (Rev.1)) indicates that the carbonaceous slate of Onwatin Formation of the Paleoproterozoic Whitewater Group encountered in the area.

3 INVESTIGATION PROCEDURES

The field work for this additional investigation was carried out during the period of September 10th to October 1st, 2014, during which time six (6) sampled boreholes (Borehole Nos. 5 to 10, inclusive) were advanced. Two boreholes were advanced at each end of the existing bridge, behind the existing abutments: one to the left and one to the right of centerline. One borehole was advanced in the area of each of the abutments for the proposed detour. Borehole No. 1 to 4 inclusive were advanced during the period of August 19th to 21st, 2013 for the initial foundation investigation, Geocres 411-304.

The field investigation was carried out using a truck and bombardier mounted CME drilling rig equipped with hollow stem augers, standard augers, and routine geotechnical sampling equipment. Prior to mobilizing the auger drill to the site, the concrete approach slabs were core drilled, where required, with an electric core drill. Soil samples were obtained at the borehole locations at regular intervals of depth using the standard 50 mm O.D. split spoon sampler advanced in accordance with the Standard Penetration Test (SPT) procedures (ASTM D-1586). The SPT method involves advancing a 50 mm O.D. split spoon sampler with the force of a 63.5 kg hammer freely dropping 760 mm mounted in a trip (automatic) hammer. The number of blows per 300 mm penetration was recorded as the "N" value. When cohesive deposits were encountered, the in-situ strength was measured using an "N" size field vane, vane collar, and calibrated torque meter. When shallow refusal was encountered, NQ size diamond coring equipment was used to determine the nature of shallow refusal. All samples taken during this investigation were stored in labeled airtight containers for transport to our North Bay laboratory for visual examination and select laboratory testing.

Groundwater conditions in the open boreholes were observed during the advancement of and immediately following, completion of the individual boreholes. All open boreholes were backfilled upon completion with compacted auger cuttings in the general order they were removed and, where necessary, bentonite pellet backfill was added to the boreholes to bring

them up to grade. At the borehole(s) through the embankment, the upper portion of the hole, where necessary, was backfilled with an asphalt cold patch to seal the existing asphalt surface.

The field work for this investigation was under the full time direction of a senior member of the Englobe engineering staff, who was responsible for locating the boreholes, clearing the borehole locations of underground services, in-situ sampling and testing operations, logging of the boreholes, labeling and preparation of samples for transport to our North Bay laboratory, plus overall drill supervision. All samples received a visual confirmatory inspection in our laboratory. Laboratory testing of select samples included routine testing for natural moisture content determination and particle size analysis, an Atterberg Limits Testing, as well as specific gravity testing. The results of the laboratory testing are presented on the individual Record of Borehole Sheets (Appendix 2). Borehole logs from the initial foundation investigation (Borehole Nos. 1 to 4, inclusive) as well as the additional foundation investigation (Borehole Nos. 1 to 10, inclusive) have been included in Appendix 2. Three unconfined compressive tests (UCT) were carried out by the Mississauga laboratory of Golder Associates Limited on selected three intact rock samples recovered at various depths in Borehole Nos. 4, 5 and 9. A summary of results is presented on the laboratory sheets in Appendix 2 (Figures Nos. L-1 to L-8 and Table No. L-10).

The location of the individual boreholes were determined in the field using highway chainage (established by others) and offset relative to highway centerline. The MTO co-ordinates, northing and easting based on MTM zone 12 NAD83 CSRS, were then established for the boring locations. Elevations contained in this report are referenced to a geodetic datum.

4 SUBSURFACE CONDITIONS

Details of the subsurface conditions revealed by the investigation program are presented on the enclosed Record of Borehole Logs (Appendix 2) and on Drawing Nos. 2 and 3 (Appendix 3). Please note that stratigraphic delineation presented on the borehole logs and soil strata plot are the results of non-continuous sampling, response to drilling progress, the results of SPT and field observations. Typically such boundaries represent transitions from one zone to another and are not an exact demarcation of specific geological unit. Additional consideration should be given to the fact that subsurface conditions may vary markedly between adjacent boreholes and beyond any specific boring location, and are shown on the drawings for illustration purposes only.

4.1 WHITSON RIVER BRIDGE

Plans and profiles illustrating the borehole locations and stratigraphic sequences are shown on Drawing Nos. 2a and 2b, Appendix 3.

During the initial exploration program, four (4) sampled boreholes were put down at this site, as follows;

- Borehole No. 1 was advanced to the east of the east approach slab right of centerline;
- Borehole No. 2 was advanced behind the west abutment right of centerline;
- Borehole No. 3 was advanced behind the east abutment to the left of centerline, and
- Borehole No. 4 was advanced to the west of the west approach slab, left of centerline.

At the time of the initial investigation, the ground surface elevations at Boreholes Nos. 1 to 4 were recorded at 268.5, 268.8, 268.6, and 268.9 m, respectively. As noted, the borehole logs for Borehole Nos. 1 to 4 have been included with this report.

During the course of the additional exploration program, six (6) sampled boreholes were put down at this site, as follows;

- Borehole Nos. 5 and 6 were advanced to the east of the east abutment, left and right of centerline, respectively;
- Borehole Nos. 7 and 8 were advanced to the west of the west abutment, right and left of centerline, respectively;
- Borehole No. 9 was advanced in the area of the proposed east detour bridge abutment, and
- Borehole No. 10 was advanced in the area of the proposed west detour bridge abutment.

At the time of the additional subsurface investigation, the ground surface elevations at Boreholes Nos. 5 to 10 were recorded at 268.5, 268.5, 268.8, 268.8, 264.8, and 264.5 m, respectively.

4.1.1 Pavement Structure

At surface at Borehole Nos. 1 and 4, a pavement structure consisting of 100 to 150 mm of asphalt and 200 to 300 mm crushed gravel was penetrated. At surface at Borehole Nos. 2, 3, 5, 6, 7, and 8, a pavement structure consisting of 75 to 150 mm of asphalt overlying a concrete approach slab some 200 to 300 mm thick was encountered. A layer of crushed gravel some 200 to 300 mm thick was encountered underlying the concrete approach slab at Borehole Nos. 2, 3, and 5.

4.1.2 Embankment Fill

Underlying the pavement structure and concrete approach slab at Borehole Nos. 1 to 8, a deposit of fill consisting of brown sand and gravel to gravelly sand, trace silt was penetrated. Cobble size rock pieces were encountered in this deposit. The natural moisture content measured on samples of this deposit was in the order of 2 to 8%. Gradation analyses were carried out on eight (8) samples of this deposit, the results of which indicated 29 to 48% gravel size particles, 46 to 60% sand size particles, and 6 to 8% silt and clay size particles (Figure No.

L-1, Appendix 3). Based on SPT 'N' values of 7 to 113 blows per 300 mm penetration, the compactness of this deposit was described as loose to very dense, generally dense. This deposit was encountered to depths of 3.4, 4.4, 5.2, 3.0, 5.6, 4.9, 4.0, and 4.3 m below grade at Borehole Nos. 5 to 8, respectively (elevations 265.1, 264.4, 263.4, 265.9, 262.9, 263.6, 264.8, and 264.5 m, respectively).

4.1.3 Sand Fill

Underlying the embankment fill at Borehole Nos. 2, 3, 5, 6, 7, and 8, a deposit of fill described as brown to grey sand some silt was penetrated. The natural moisture content measured on samples of this deposit was in the order of 8 to 30%, indicating a moist to wet moisture condition relative to the estimated optimum moisture content. A gradation analysis was carried out on one (1) sample of this deposit, the results of which indicated 0% gravel size particles, 89% sand size particles, and 11% silt and clay size particles (Figure No. L-2, Appendix 3). Based on SPT 'N' values of 0 (static weight of hammer) to 24 blows per 300 mm penetration, the compactness of this deposit was described as very loose to compact, generally loose. This deposit was encountered to depths of 8.1, 10.1, 11.3, 9.5, and 7.1 m below grade at Borehole Nos. 2, 5, 6, 7, and 8, respectively (Elevations 260.7, 258.4, 257.2, 259.3, and 261.0 m, respectively). Auger refusal was encountered in this deposit at a depth of 10.1 m below grade at Borehole No. 3 (Elevation 258.5 m).

4.1.4 Mixed Fill

Underlying the embankment fill at Borehole No. 4, a deposit of fill described as a mix of cobble and boulder sizes mixed with a grey sand with silt was penetrated. Pieces of wood and concrete were encountered in this deposit. The natural moisture content measured on samples of this deposit was in the order of 21%, indicating a wet condition relative to optimum moisture content. This deposit was encountered to a depth of 4.4 m below grade (Elevation 264.5 m).

4.1.5 Sand

At surface at Borehole No. 9 and underlying the embankment fill at Borehole No. 1, a deposit of brown to grey sand some to with silt was penetrated. The natural moisture content measured on samples of this deposit was in the order of 15 to 54%, indicating a moist wet moisture condition relative to the estimated optimum moisture content. The elevated moisture contents are likely a result of organics mixed with the samples. Gradation analyses were carried out on three (3) samples of this deposit, the result of which indicated 0% gravel size particles, 71 to 84% sand size particle, and 16 to 29% silt and clay size particles (Figure No. L-3, Appendix 3). Based on SPT 'N' values of 0 (static weight of hammer) to 7 blows per 300 mm penetration, the compactness of this deposit was described as very loose to loose. This deposit was encountered to depths of 6.7 and 4.4 m below grade at Borehole Nos. 1 and 9, respectively (Elevations 261.8 and 260.4 m, respectively).

4.1.6 Silt

Underlying the fill at Borehole No. 4, a deposit of grey silt trace clay was penetrated. The natural moisture content measured on a sample of this deposit was in the order of 20%, indicating a wet moisture condition, relative to optimum moisture content. A gradation analysis was carried out on one (1) sample of this deposit, the results of which indicated 0% gravel size particles, 0% sand size particles, 95% silt size particles, and 5% clay size particles (Figure No. L-4, Appendix 3). Based on STP 'N' values of 28 blows per 300 mm penetration, this deposit was described as compact. This deposit was encountered to a depth of 5.8 m below grade (Elevation 263.1 m).

4.1.7 Sand and Silt to Silty Sand

At surface at Borehole No. 10, a deposit of brown sand and silt to silty sand was penetrated. The natural moisture content measured on samples of this deposit was in the order of 18 to 20%, indicating a wet moisture condition relative to the estimated optimum moisture content. Gradation analyses were carried out on two (2) samples of this deposit, the result of which indicated 0 to 1% gravel size particles, 54 to 67% sand size particle, and 33 to 45% silt and clay size particles (Figure No. L-5, Appendix 3). Based on SPT 'N' values of 4 to 5 blows per 300 mm penetration the compactness of this deposit was described as loose. This deposit was encountered to a depth of 2.1 m below grade at Borehole No. 10 (Elevation 262.4 m).

4.1.8 Sandy Silt

Underlying the sand and silt to silty sand at Borehole No. 10, a deposit of grey sandy silt trace clay was penetrated. The natural moisture content measured on a sample of this deposit was in the order of 23%, indicating a moist moisture condition, relative to the estimated optimum moisture content. A hydrometer analysis was carried out on a single sample of this deposit, the results of which indicated 0% gravel size particles, 37% sand size particles, 58% silt size particles, and 55% clay size particles (Figure No. L-6, Appendix 3). Atterberg Limits Testing was carried out on one (1) sample of this deposit, the results of which indicated a liquid limit in the order of 17% and a plastic limit of 14% (Figure No. L-8, Appendix 3). Based on the results of the Atterberg Limits Testing, this deposit was described as inorganic silt (ML). Based on a SPT 'N' value of 12 blows per 300 mm penetration, this deposit was described as compact. This deposit was encountered to a depth of 2.9 m below grade (Elevation 261.6 m).

4.1.9 Silty Clay

Underlying the sand at Borehole Nos. 1 and 9, underlying the silt at Borehole No. 4, underlying the sand fill at Borehole No. 8, and underlying the sandy silt at Borehole No. 10, a deposit of grey silty clay was penetrated. The natural moisture content measured on samples of this deposit was in the order of 30 to 57%, indicating a wet moisture condition relative to the estimated optimum moisture content. Hydrometer analyses were carried out on three samples of this deposit, the results of which indicated 0% gravel size particles, 0 to 1% sand size particles, 23 to 71% silt size particles, and 28 to 77% clay size particles (Figure No. L-7,

Appendix 3). Atterberg Limits Testing was carried out on three (3) samples of this deposit, the results of which indicated a liquid limit in the order of 28 to 61% and a plastic limit of 20 to 22% (Figure No. L-8, Appendix 3). Based on the results of the Atterberg Limits Testing, this deposit was described as silty clay of low to high plasticity (CL to CH). Based on in situ shear strength of 60 to 88 kPa, the consistency of this deposit was described as stiff. This deposit was encountered to depths of 7.8, 7.9, and 7.4 m below grade at Borehole Nos. 8, 9, and 10, respectively (Elevations 261.0, 256.9, and 257.1 m, respectively). Auger refusal was encountered in this deposit at depths of 11.4 and 8.8 m below grade at Borehole Nos. 1 and 4, respectively (Elevations 257.1 and 260.1 m, respectively).

4.1.10 Concrete

A layer of concrete was encountered below the sand fill at Elevation 260.7 m at location of Borehole No. 2. This concrete is likely part of the abutment footing. The borehole was terminated at a depth of 9.3 m below grade at Borehole No. 2 (Elevation 259.5 m).

4.1.11 Bedrock

Underlying the above described sand fill at Borehole Nos. 5, 6, and 7, and underlying the silty clay at Borehole Nos. 4, 8, 9 and 10, bedrock was proven by diamond core drilling. The bedrock was described as black slate. Based on Rock Quality Designation (RQD) values of 49 to 100% the bedrock was described as poor to excellent quality. Photographs of the rock cores are included in Appendix 2. The summarized elevations of the top-of-bedrock are shown on the table below.

BOREHOLE NUMBER	ELEVATION OF TOP-OF BEDROCK (M)
04	260.1
05	258.4
06	257.2
07	259.3
08	261.0
09	256.9
10	257.1

Three unconfined compressive tests (UCT) were carried out on selected three intact rock samples recovered at various depths in Borehole Nos. 4, 5 and 9. Results of testing are summarized on the table below.

BOREHOLE NUMBER	DEPTH/ELEVATION	UNCONFINED COMPRESSIVE STRENGTH
04	11.6 to 11.9 m/ 257.3 to 257 m	22.9 MPa
05	12.64 to 12.88 m/ 255.86 to 255.62 m	79.7 MPa
09	9.26 to 9.46 m/ 255.54 to 255.34 m	100.1 MPa

Sampling in the bedrock was terminated at a depth of 12.2, 13.1, 14.3, 12.5, 11.0, 10.9, and 11.0 m below grade at Borehole Nos. 4 to 10, respectively (Elevations 256.7, 255.4, 254.2, 256.3, 257.8, 253.9, and 253.5 m, respectively). It should be noted that, when encountered, the underlying bedrock surfaces in this area are very erratic in nature, varying in elevation over short horizontal distances.

4.2 GROUNDWATER DATA

Measurements of the groundwater table and cave-in levels were undertaken, where possible, in the open boreholes during the advance of the individual borings and upon completion.

Piezometers were installed at Borehole Nos. 5, 8 and 9 to determine stabilized water levels. These levels are recorded on the individual Record of Borehole Log Sheets (Appendix 2).

The groundwater levels in Borehole Nos. 5, 8, and 9 were measured at Elevations between 262.8 to 264.3 m, some one to three days after completion of the field program. The groundwater was encountered at Elevations 265.2 and 262.0 m below grade at Borehole Nos. 6 and 10 immediately following completion of coring, however this water level was not stabilized and was likely elevated due to the water used during coring operations. The water levels in Whitson River were measured at Elevation 264.2 m in May 2013 by others and at Elevation 262.7 m in September 2014.

The groundwater and river water levels will fluctuate seasonally/yearly.

5 DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

This additional foundation investigation was carried out to provide design recommendations for foundation for the replacement of the existing Whitson River Bridge as well as for foundations for a temporary detour bridge to be used during bridge replacement. The existing bridge is located approximately between Stations 12+419.5 to 12+444.5, in the Township of Balfour, and is identified as Site No. 46-051. The existing bridge is a single span, concrete girder structure, some 25 m in length.

The existing highway, at the bridge location, supports two undivided lanes of traffic, locally running in an east-west direction. Based on data from this foundation investigation, the embankment supporting the existing pavement structure at this site has been constructed with a granular fill overlying native sands, silts, and silty clays.

Based on the historical General Arrangement Drawing D-4476/1, the existing Whitson River Bridge abutments are founded on spread footings supported on bedrock (see Appendix 4).

The abutments for the new bridge will be established behind the existing abutments and the bridge will be lengthened to some 32.5 m. It is understood that, at this time, the bridge design consists of a box girder system. It is further understood that the length of 32.5 m is approaching the maximum allowable span for this type of system. As such, there is limited space for new abutments between the existing abutments, when considering maximum single span structure, without changing the bridge design. The bridge will be replaced with integral abutments.

5.2 FROST PROTECTION

The estimated depth of frost penetration for the area is about 2.1 m. As such, foundation elements, which are subject to frost penetration, must be supplied with a minimum of 2.1 m of earth cover (both horizontally and vertically) for frost protection. If a sufficient depth of earth cover cannot be provided for frost protection, equivalent Expanded Extruded Polystyrene insulation (EEP) may be used in conjunction with available soils cover to provide frost protection. If EEP is used for frost protection, precautions must be taken to protect the insulation from contact with hydrocarbons, solvents, or other destructive products. The buoyancy of the product must also be considered in the design unless sufficient cover weight is provided to counter the effects, which may limit the use of EEP to above the high water level.

5.3 FOUNDATION CONSIDERATIONS

5.3.1 Shallow Foundations

As noted, the existing abutments are supported on spread footings supported directly on bedrock. As indicated on the Record of Boreholes and summarized in the Section 4, the overburden soils present at the bridge abutments consist sand and gravel to gravelly sand fills overlying very loose to compact sand fills. These fill generally extended to bedrock at the

boreholes advanced at the location of the proposed abutments, except at Borehole No. 8, where a layer of silty clays were encountered underlying the sand fills.

Foundations cannot be established on non-engineered fills. As such, should abutments founded on shallow foundations be considered, the foundations should be founded directly on bedrock (Elevations 257.2 to 258.4 m at the east approach and 259.3 to 261.0 m at the west approach). The top of existing bridge abutment footing was probably encountered at Elevation 260.7 m at the location of Borehole No. 2 located at the Western approach. Spread footings founded directly on bedrock can be established above the scour depth. The bedrock is of poor to excellent quality, based on RQD data. Results of three unconfined compressive tests (UCT) indicate that the unconfined compressive strength of rock cores range from 22.9 to 100.1 MPa. As such, a bearing resistance at ULS of 2300 kPa is appropriate for the foundation design. Since bedrock is essentially an unyielding subgrade, a geotechnical reaction at SLS does not apply.

If any bedrock surface supporting foundations slopes greater than 10° off the horizontal, adequate dowels should be installed to resist sliding and a staff member from our office must review the conditions in the field. Alternatively, the contractor could level the footing bearing surface with a hydraulic hammer (hoe-ram).

The depth of excavations (some 7.8 to 11.3 m) will be the major constraining factor for installation of reinforced concrete spread footings. In addition, groundwater control will be critical during excavation and construction operation. Groundwater levels were encountered at Elevations 262.8 to 264.3 m at the time of this additional investigation. As such, up to some 7 m of water must be controlled to allow for construction of shallow foundations. In consideration of the difficulties associated with founding the abutments directly on bedrock, it is understood that shallow foundations directly on bedrock, are not considered to be cost effective at this site. As such, deep foundations are recommended for supporting the proposed abutments, as discussed below.

5.3.2 Deep Foundations

As noted, it is understood that a deep foundation system is the preferred method of supporting the proposed abutments. It is further understood that the existing abutments will be left in place and the new abutments to be constructed behind the existing abutments. At this time it is anticipated that the underside of the abutments will be established at approximately Elevation 264.0 m. Deep foundations will be used to support the new abutments to minimize the excavation depth, shoring requirements, etc.

Based on two boreholes advanced at each of the locations of the proposed abutments, a deep foundation system would encounter refusal on bedrock at Elevations 257.2 to 258.4 m at the east approach and Elevations 259.3 to 261.0 m at the west approach.

Three potential deep foundation systems, H-Piles, caissons, and micropiles, are discussed as follows.

5.3.2.1 *H Piles*

It is understood that steel H pile sections are generally the preferred deep foundation system for integral abutments as steel H piles are generally an economical pile choice. This can, however, vary depending upon market conditions.

In comparison to driven concrete piles, timber piles, auger cast or Franki (expanded base) piles; steel H piles are considered most appropriate driven pile based on the subsurface conditions at this site and from a cost perspective.

However, based on the bedrock elevations encountered during this investigation and historical foundation information, it appears that significant elevations differences are possible across each abutment location. As such, it is likely that the bedrock surface is sloping. Considering the relatively shallow depths of bedrock, there will likely be difficulties setting the pile tip on the bedrock.

The most effective method of setting the piles would be to socket the pile into rock. It is recommended that a 600 mm diameter socket be installed at least 1 m into bedrock. This hole is predrilled and cased from grade and the Contractor fills the socket with 35 MPa grout and sets the H pile or steel pipe pile into the grout. The Contractor can then pull the casing and install a 3 m x 0.6 m sand filled CSP, directly below the grade beam/abutment, for lateral movement consideration at the top of the pile. However, considering the relatively short pile length which will be socketed into bedrock, it must be analyzed as an end bearing caisson (i.e. circular footing).

In order to obtain full axial capacity of a conventional H pile foundation system, the piles should be set into the sound bedrock. At this site, it is anticipated that the pile tips will be set at approximately Elevations 256.2 to 257.4 m at the east approach and Elevations 258.3 to 260.0 m at the west approach, in consideration of socketing the pile some 1 m into bedrock.

For structural design we recommend that a HP310X110 set 1 m into bedrock be considered using the following capacity:

H PILE TYPE	ESTIMATED PILE TIP DEPTH (M)	ESTIAMTED PILE TIP ELEVATION (M)	FACTORED AXIAL RESISTANCE AT ULS (KN) (PER PILE)	RESISTANC E AT SLS (KN) (PER PILE)	FACTORED LATERAL RESISTANCE AT ULS (KN) (PER PILE)
HP 310X110	±10	±256.2 to 260.0	1130	*	75 kN at per cut-off level

* Since bedrock is essentially an unyielding subgrade, a geotechnical reaction at SLS does not apply.

As noted, since the pile will be short and installed into bedrock, it must be analyzed as an end bearing caisson. As such, driving criteria will not apply.

The weak axis of the H Pile can be oriented parallel to the longitudinal axis of the bridge to provide lower stiffness of the foundation for the integral abutment. Additional lateral resistance could be supplied with batter piles (maximum batter 4:1), however battered piles are not appropriate for an integral abutment system at the flexible abutment end. In compact to dense deposits a 600 mm diameter by 3000 mm CSP filled with loose sand would be required around the upper part of each H pile.

5.3.2.2 **Caissons**

Caissons are a deep foundation generally constructed as a cast in place concrete, bored, pile. Caissons are often cased to allow installation to depth in poor subsurface conditions (i.e. loose/wet soils). Considering the relatively shallow depths to bedrock at this site, cased caissons could be considered for support of the proposed abutments.

The optimum method of caisson installation at this site would be to advance a casing to bedrock, then set the casing into the rock. Following which, the soils within the casing can be removed and the casing filled with concrete.

As previously noted, there is limited space between the existing abutments and proposed bridge limits. As such, a limited diameter (i.e. say maximum 600 mm) caisson is considered for support of the proposed abutment.

Considering the relatively shallow depth to bedrock, caissons will develop the bearing as a circular footing from the bedrock. As such, caissons bearing directly on unweathered clean bedrock may be designed with a factored bearing resistance at ULS of 4000 kPa (i.e. 1130 kN for 600 mm diameter caisson). Since bedrock is essentially an unyielding subgrade, a geotechnical reaction at SLS does not apply. If any bedrock surface supporting foundations slopes greater than 10° off the horizontal, adequate rebar dowels should be installed to resist sliding and a staff member from our office must review the conditions in the field. Based on the bedrock profile encountered at the borehole location, the bedrock surface appears to have slopes greater than 10°, as such, must be doweled into the bedrock or socketed a sufficient depth to produce a horizontal bearing surface all on sound bedrock.

5.3.2.3 **Micropiles**

A micropile is a small diameter (typically less than 300 mm diameter) pile and consists of either a cased or uncased cement grouted column, with one or more centrally located high tensile strength steel threaded rod(s) or pipe(s). The capacity of the micropile is dependent upon the grout to ground bond between the grouted portion of the micropile and the bedrock and/or soil. It should be noted that the design of a micropile foundation is proprietary and there are many different micropile configurations and installation methods that may be used to meet different founding and loading conditions.

At this site, it would be possible to advance a micropile through the existing fills and into the bedrock. Since the depth of bedrock varies at the proposed abutment location, the capacity of the micropile should be based on the grout to bedrock bond length, probably ignoring the contribution of the variable overburden thickness. A duplex rotary percussive drill rig with a down the hole hammer (DTH) would be the preferable method to advance the casing through the fill into the bedrock. Once the bedrock is encountered and the casing sealed into bedrock, a smaller diameter bit would be lowered down the casing to allow percussive drilling into the bedrock. Once the required penetration into bedrock is achieved the drill rod is removed and a high strength threaded rod (i.e. Dywidag Threadbar, Williams Anchor, or equivalent) is lowered into the bedrock socket. Tremie methods can then be used to grout the rod into the bedrock from the bottom of the hole up to the underside of abutment elevation, using neat cement grout, typically with a compressive strength of 30 MPa strength or greater. In this case the geotechnical resistance of the micropile would be limited to the grout-to-bedrock bond. A micropile design is presently based on the Federal Highway Administration publication FHWA-SA-97-070, June 2000.

For the basic design, micropiles would be drilled through the fills and/or native soils and a minimum 2 m into bedrock. The centre high tensile strength steel threaded rod(s) of the micropile would be grouted into the bedrock and extended up, through the overburden, and into the new abutments. The micropiles can be designed to withstand various compression and tension loads, depending upon depth of penetration/bond length in the bedrock.

As noted previously micropiles are proprietary and the specific design is carried out by the contractors engineering personnel. The specialist foundation contractors have different drilling equipment and tooling size however they will be able to satisfy the contract if imposed loads and load location are supplied on the contract drawings. As a basic/typical design the factored geotechnical resistance of a micropile constructed with a #14 (43 mm diameter) Dywidag, Grade 75 threaded rod (or equivalent), gravity grouted into a 100 mm diameter by 2.0 m long bedrock socket can be taken as 250 kN. This geotechnical resistance does not account for the upper 0.5 m of rock socket, due to the occasional poor quality rock designation. The capacity of micropiles can be increased by increasing bond length, however this is limited by the yield strength of the rod.

For the foundation design, a #14 (43 mm diameter) Dywidag, Grade 75 threaded rod, can be considered using the following capacity:

BOND DIAMETER (MM)	DEPTH INTO BEDROCK (M)	BOND LENGTH (M)	FACTORED GEOTECHNICAL RESISTANCE (KN)
100	2	1.5	250
200	2	1.5	500
300	2	1.5	750
300	2.5	2.0	850

The design of the micropile system must be carried out by a Professional Engineer registered in the Province of Ontario. The contractors design must be verified with a minimum of one static axial load test, on a sacrificial pile, in accordance with ASTM test procedure.

A table outlining the alternatives of the possible foundation types described above for the new bridge and their relative advantages, disadvantages, and relative costs, as well as comments on the viability of the methods is provided on Table A in Appendix 5.

5.4 DETOUR

It is understood that a temporary detour will be constructed to allow construction of the proposed new bridge. The detour will be established to the north of the existing bridge.

Boreholes No. 9 and 10 were advanced on the east and west banks, respectively, north of the existing bridge to supply information for detour foundation design. A temporary modular bailey type bridge can be supplied up to a 76 m single span. The detour design is typically the responsibility of the contractor.

At this time it is understood that the detour will likely be constructed in one year for use the next year (i.e. left in place over the winter). It is further understood that the detour will not be used over the winter.

This temporary modular bridge structure is not as susceptible to settlement as a permanent concrete structure, as such; temporary shallow foundations could be considered at this location as discussed below.

The grade along the proposed detour alignment drops off towards Whitson River. The grade in the area of the proposed abutments is some 3.5 to 4 m lower than the existing highway centreline. It is likely that the grade will be raised to allow for detour construction. The native soils at the west approach comprise loose sands and silts to silty sands, overlying sandy silts, overlying stiff silty clays overlying bedrock. On the east bank the soils comprise of very loose to loose sands overlying stiff silty clays overlying bedrock.

Frequently temporary modular bridges are supported on a timber crib foundations or concrete spread footings on grade. The existing very loose to loose sands/sands and silts at the approaches are generally not acceptable for supporting foundations for permanent structures. However as noted, the temporary detour bridge structure is not as susceptible to settlement (both total and differential) as permanent structures, as such, shallow foundation may be considered for the temporary detour at this site. Additionally, the existing grade will likely be raised to support the proposed detour structure. If the fill below the on grade footing is constructed with a minimum 2 m of Granular B Type II, compacted to 100% Standard Proctor Dry Density, the stresses will be dissipated substantially through the engineered pad.

Granular B Type II engineered fill compacted to 100% SPDD can offer a reasonably higher geotechnical bearing resistance however, the SLS condition will govern to maintain settlement

to 50 mm or less. With a minimum 2.0 m depth (at east abutment) and 4 m depth (at the west abutment) of engineered fill below the on grade footing, the following geotechnical parameters are applicable for temporary footings of a minimum 0.5 m width at depth of 2.1 m below grade, provided the unsuitable material is removed at the ground surface:

- ▶ Factored Geotechnical Resistance at ULS: 200 kPa
- ▶ Geotechnical Reaction at SLS of 25 mm total settlement: 100 kPa net bearing pressure increase

It is understood that the detailed detour design will be carried out by the Contractor. Based on the existing topographic conditions along the proposed detour alignment at the river banks, some 3.5 m to 4.5 m of grade raise are anticipated to maintain the similar profile elevations to the existing bridge for the temporary modular bridges supported on shallow foundations.

Alternative a deep foundation system can be considered for supporting the proposed detour bridge to minimize the earthwork, shoring requirements, etc. For the temporary detour bridge, the H-pile with a 600 mm diameter socket installed at least 1 m into bedrock is recommended for the deep foundation. Based on two boreholes (i.e. Borehole Nos. 9 and 10) advanced at locations of the proposed temporary detour bridge, the founding levels of the deep foundation system would not be greater than Elevation 255.9 m at the east approach and Elevation 256.1 m at the west approach. A table outlining the alternatives of the possible foundation types for the temporary detour bridge and their relative advantages, disadvantages, and relative costs of the methods is provided on Table B in Appendix 5. It should be noted that slope stability analyses and recommended thickness of rock protection described in Section 5.4.1 haven't yet considered any additional surcharge loads which may be induced by the temporary detour embankment and/or bridge construction. The Contractor should carry out his own slope stability analyses and provide additional rock protection, if necessary, based on the structure type of their proposed detour bridge and/or detour embankment.

5.4.1 Slope Stability

A stability analysis, using the GEO-SLOPE computer program, Slope/W (GeoStudio 2007, version 7.17, Geo-Slope International Ltd.), was carried out for a proposed detour embankment at this location with embankment slopes of some 2.0H:1.0V in granular fill. For the purposes of these analyses, the materials were modeled using the following parameters;

PARAMETER	MATERIAL				
	GRANULAR B TYPE II	EMBANKMENT FILL	SAND AND SILT	SAND	SILTY CLAY
Unit Weight (kN/m ³)	23.0	20	18	18	16.5
Effective Friction Angle (degrees)	34	32	28	29	-
Undrained Shear Strength (kPa)	-	-	-	-	60

The unit weights and friction angles for the slope calculations are based on general representative values for the various soil types, obtained through laboratory testing and tactile analysis. The results of the analyses indicated the factor of safety for assumed 2H:1V side slopes of the detour embankment consisting of OPSS granular B Type II material, if considered, will be in the order of 1.7 (see Figure No. S-1, Appendix 5). Lower factors of safety will occur during excavation and backfilling as discussed in Section 5.6. Short term stability should not be an issue if construction is carried out as described herein. The long term stability of the proposed detour embankment will not be an issue provided it is properly constructed.

A rock protection placed on the 1.5H:1V slope on river banks directly under the new bridge is feasible because the shallow footings of the existing bridge will remain in place below Elevation 264.3 m to provide sufficient geotechnical resistance for stability of embankments. However, the results of our slope stabilities for rock protection carried out along the east river bank indicate the slopes of river banks, consisting of loose native sand and presently at 2H:1V or steeper inclination angles, are marginally stable as shown on Figure No. S-2; therefore it is recommended the slopes of river banks between Cross Stations 0+010 and 0+060 be flattened to 2H:1V or flatter and protected with 0.5 m thick rock protection (R-50 size as per OPSS.PROV 1004) from the submerged toe of river bank up to the high water level plus at least 0.3 m except the area directly below the bridge, to increase the factor safety against slope failure to be in the order of 1.2 as shown Figure Nos. S-3 to S-6. For the remaining areas of the east bank and the west bank, 0.5 m thickness of rock protection is required on slopes which are cut back to 2H:1V except the area directly below the bridge.

It should be noted that the analyses of this slope stability and recommended thickness of rock protection haven't yet considered any additional surcharge loads which may be induced by the temporary detour embankment and/or bridge construction. The Contractor should carry out his own slope stability analyses and provide additional rock protection, if necessary, based on the structure type of the proposed detour bridge and/or detour embankment. A special provision of Notice to Contractor – addressing this issue is contained in Appendix 5.

5.5 EXCAVATION, DEWATERING, AND EMBANKMENT RECONSTRUCTION

All excavations greater than 1.2 m in depth must, at a minimum, be sloped or shored in accordance with the Occupational Health and Safety Act Regulations for Construction Projects. The embankment material, above the water table, is considered a Type 3 soil as defined in the Occupational Health and Safety Act and Regulations for Construction Projects. Temporary open excavations above the groundwater table, could be cut back at an angle of 1H:1V, provided they are monitored continuously, however, below the groundwater table, the side slopes will have to be cut back to an angle of 2H:1V, possibly shallower, dependent upon the Contractors' chosen method of controlling the groundwater.

Final embankment slopes in granular fill, where required (i.e. west approach), should be constructed to match the existing side slopes, or at an angle of 3H:1V as per OPSS 200.010 (November 2009).

Excavations must be maintained in a dewatered condition during excavation and foundation construction, and every reasonable effort must be made to prevent disturbing (piping/boiling) at the founding subgrade. Groundwater control, in accordance with OPSS 517 and 518, will be required to maintain a stable subgrade during excavation.

The water level in the river was recorded at Elevation 262.7 m at the time of this investigation. The depths of excavations at this site will depend on the chosen methodology of abutment construction. Dewatering may be required depending on excavation depths and water level at time of construction. Groundwater levels will fluctuate seasonally/yearly.

Ultimately, the method of excavation, dewatering, and river flow control will be the choice of the contractor; however the importance of maintaining the subgrade in a dewatered stable condition during excavation and construction operations cannot be stressed enough.

5.5.1 **Backfill and Compaction**

The existing backfill at the abutments was very loose to very dense, generally in a compact condition. Prior to backfilling the excavation the existing subgrade should be proofrolled with a minimum of five overlapping passes of a hand operated vibratory compactor with a minimum weight of 400 kg (or a centrifugal force of 50 kN). Backfilling should be carried out in accordance with OPSS 902 and compaction should be carried out in accordance with OPSS 501.

5.6 **CONSTRUCTION CONCERNS**

If a conventional foundation system of spread footings taken down to bear on the underlying bedrock is considered, then a closed sheet pile cofferdam system would have to be employed at the abutment location(s) to control groundwater during construction. Cobble/boulder size rock pieces were encountered in the existing abutment/embankment fill which may create minor difficulties during driving sheet piles. However, these obstructions were generally encountered above the prevailing groundwater table and could be excavated out before or during driving of the sheeting below the water table.

In addition, the rock protection on both river banks is designed without considering the additional surcharge loads induced by the temporary detour bridge construction. The Contractor should carry out his own slope stability analyses and checking based on the structure type of detour bridge he selects and construct any additional bank stabilization required, dependent on his design.

6 STATEMENT OF LIMITATIONS

The design recommendations given in this geotechnical report are applicable only to the project described in the text and only if constructed substantially in accordance with details of alignment and elevations stated in the report. Since all details of the design may not be known, in our analysis certain assumptions had to be made. The actual conditions may however, vary from those assumed, in which case changes and modifications may be required to our geotechnical recommendations. We recommend, therefore, that we be retained and provided the opportunity during the design stage to review the design drawings, site survey information, proposed elevations, etc. to verify that they are consistent with our recommendations or the assumptions made in our analysis. It is further recommended that we be retained to review the final design drawings and specifications relative to the geotechnical recommendations.

If, during construction, conditions in the field vary from those assumed at the design stage, an engineer from this office must be notified immediately.

Proper subgrade preparation, groundwater control, compaction, etc. are all critical aspects of the bearing capacity of native soils. It must be noted that different aspects of the geotechnical design are based on the assumption that Englobe will be retained during site preparation and construction of the proposed works to ensure that both the geotechnical site characteristics and the construction operations/techniques are consistent with our recommendations. Should Englobe not be involved during the full construction phase, our liability is strictly limited to the factual information contained herein only.

The comments in this report are intended solely for the guidance of the design engineer and address the geotechnical conditions only. The number of boreholes required to determine the localized conditions between boreholes directly affecting construction costs, equipment, scheduling, etc. would in fact be greater than what has been carried out for design purposes. Therefore, contractors bidding on this project or undertaking this work should make their own interpretations of the factual borehole results and carry out further work as they deem necessary to assess the scope of the project.

Section 5 of this reported is intended for the use of the client and the design team only and is not intended to be included in the tender documents. Inclusion of the factual information (Sections 1 to 5 inclusive) in the tender documents is furnished merely for the general information of bidders and is not in any way warranted or guaranteed by or on behalf of the owner or the owner's consultants and its subconsultants or the consultants' or subconsultants' employees, and neither the owner nor its consultants or its employees shall be liable for any representations negligent or otherwise contained in the documents.

Appendix 1 Key Plan

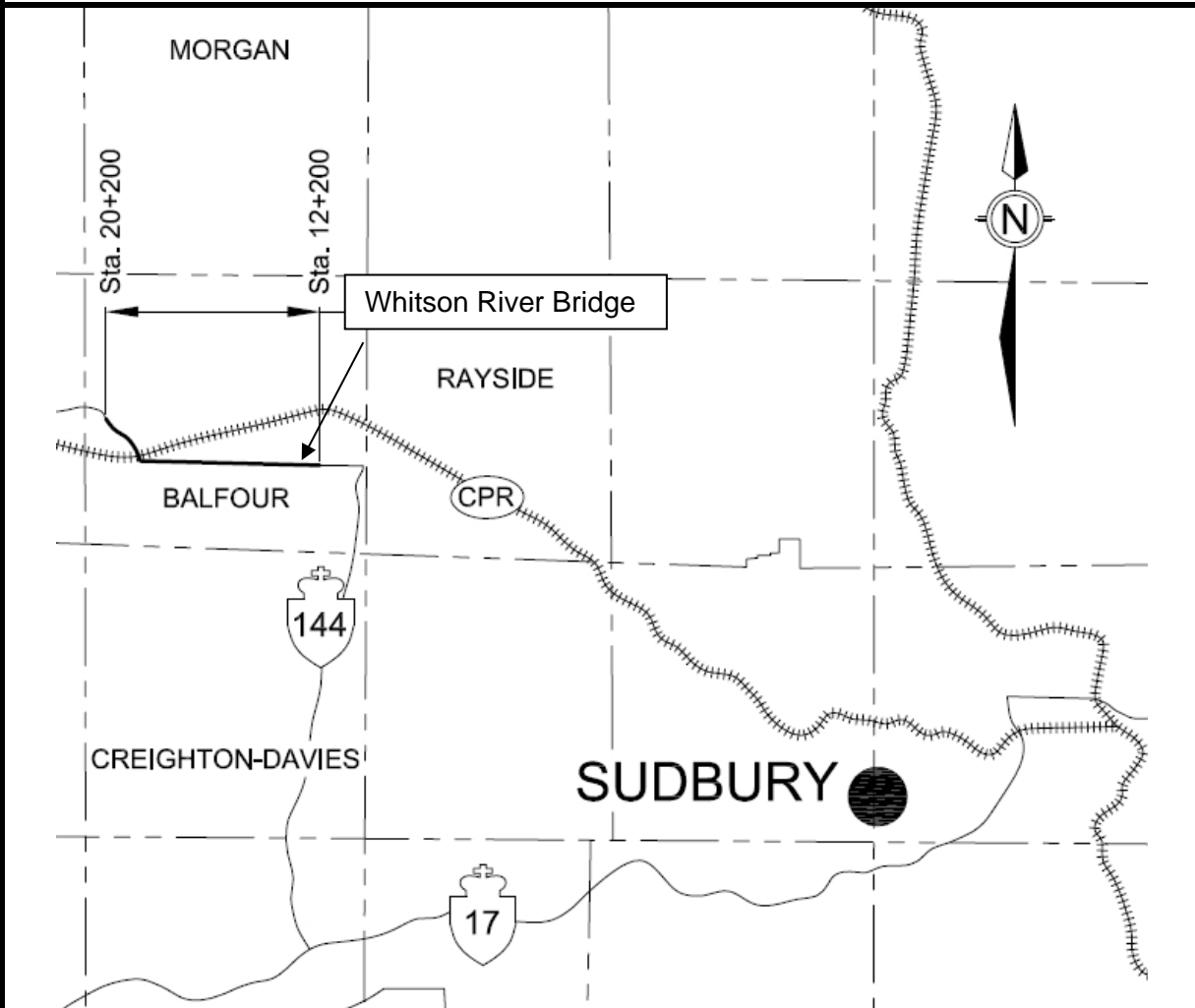
Drawing No. 1

Key Plan

KEY PLAN

Drawing No. 1

NOT TO SCALE



FINAL
ADDITIONAL FOUNDATION INVESTIGATION
AND DESIGN REPORT
GWP 5223-14-00
Highway 144
Whitson River Bridge



Reference No: 12/11/12218-F3

February 2016

Appendix 2 Subsurface Data

Enclosure No. 1	List of Abbreviations and Symbols
Enclosure Nos. 2 to 11	Record of Borehole Sheet
Enclosure No. 12	Photos of Rock Cores

LIST OF ABBREVIATIONS & DESCRIPTION OF TERMS

The abbreviations and terms, used to describe retrieved samples and commonly employed on the borehole logs, on the figures and in the report are as follows:

1. ABBREVIATIONS

AS	Auger Sample
CS	Chunk Sample
DS	Denison type sample
FS	Foil Sample
NFP	No Further Progress
PH	Sampler advanced by hydraulic pressure
PM	Sampler advanced by manual pressure
RC	Rock core with size & percentage of recovery
SS	Split Spoon
ST	Slotted Tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash Sample
WH	Sampler advanced by static weight of hammer and/or rods
Rec	% recovery from individual run of rock core
RQD	Rock quality designation (%)

2. PENETRATION RESISTANCE/"N"

Dynamic Cone Penetration Test (DCPT):

A continuous profile showing the number of blows for each 300 mm of penetration of a 50 mm diameter 60° cone attached to AW rod driven by a 63 kg hammer falling 760 mm.

Plotted as —●—●—●—●—

Standard Penetration Test (SPT) or "N" Values

The number of blows of a 63 kg hammer falling 760 mm required to advance a 50 mm O.D. drive open sampler 300 mm.

3. SOIL DESCRIPTION

a) *Cohesionless Soils:*

"N" (blows/0.3 m)	Relative Density
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

b) *Cohesive Soils:*

Undrained Shear Strength (kPa)	Consistency
Less than 12	very soft
12 to 25	soft
25 to 50	firm
50 to 100	stiff
100 to 200	very stiff
over 200	hard

3. SOIL DESCRIPTION (Cont'd)

c) *Bedrock:*

RQD (%)	Classification
Less than 25	Very poor quality
25 to 50	Poor quality
50 to 75	Fair quality
75 to 90	Good quality
90 to 100	Excellent quality

d) *Method of Determination of Undrained Shear Strength of Cohesive Soils:*

- + 3.2 - Field Vane test in borehole.
The number denotes the sensitivity to remoulding.
- D - Laboratory Vane Test
- " - Compression test in laboratory

For a saturated cohesive soil the undrained shear strength is taken as one-half of the undrained compressive strength.

e) *Soil Moisture:*

Moisture	Described as
Dry	Below optimum moisture content
Moist	Near optimum moisture content
Wet	Above optimum moisture content

4. TERMINOLOGY

Terminology used for describing soil strata is based on the proportion of individual particle sizes present in the samples (please note that, with the exception of those samples subject to a grain-size analysis, all samples were classified visually and the accuracy of visual examination is not sufficient to determine exact grain sizing):

Trace, or occasional	Less than 10%
Some	10 to 20%
With	20 to 30%
Adjective (i.e. silty or sandy)	30 to 40%
And (i.e. sand and gravel)	40 to 60%

Terminology for cobbles and boulders is based on auger response and field observations:

Occasional	Obstructions encountered in borehole, however advance is not impeded
Numerous	Obstructions are essentially continuous over drilled length

SAMPLE DESCRIPTION NOTES:

1. **FILL:** The term fill is used to designate all man-made deposits of natural soil and/or waste materials. The reader is cautioned that fill materials can be very heterogeneous in nature and variable in depth, density and degree of compaction. Fill materials can be expected to contain organics, waste materials, construction materials, shot rock, rip-rap, and/or larger obstructions such as boulders, concrete foundations, slabs, abandoned tanks, etc.; none of which may have been encountered in the borehole. The description of the material penetrated in the borehole therefore may not be applicable as a general description of the fill material on the site as boreholes cannot accurately define the nature of fill material. During the boring and sampling process, retrieved samples may have certain characteristics that identify them as 'fill'. Fill materials (or possible fill materials) will be designated on the Borehole Logs. If fill material is identified on the site, it is highly recommended that testpits be put down to delineate the nature of the fill material. However, even through the use of testpits defining the true nature and composition of the fill material cannot be guaranteed. Fill deposits often contain pockets or seams of organics, organically contaminated soils or other deleterious material that can cause settlement or result in the production of methane gas. It should be noted that the origins and history of fill material is frequently very vague or non-existent. Often fill material may be contaminated beyond environmental guidelines and the material will have to be disposed of at a designated site (i.e. registered landfill). Unless requested or stated otherwise in this report, fill material on this site has not been tested for contaminants however, environmental testing of the fill material can be carried out at your request. Detection of underground storage tanks cannot be determined with conventional geotechnical procedures.
2. **TILL:** The term till indicates a material that is an unstratified, glacial deposit, heterogeneous in nature and, as such, may consist of mixtures and pockets of clay, silt, sand, gravel, cobbles and/or boulders. These heterogeneous deposits originate from a geological process associated with glaciation. It must be noted that due to the highly heterogeneous nature of till deposits, the description of the deposit on the borehole log may only be applicable to a very limited area and therefore, caution must be exercised when dealing with a till deposit. When excavating in till, contractors may encounter cobbles/boulders or possibly bedrock even if they are not indicated on the borehole logs. It must be appreciated that conventional geotechnical sampling equipment does not identify the nature or size of any obstruction.
3. **BEDROCK:** Auger refusal may be due to the presence of bedrock, but possibly could also be due to the presence of very dense underlying deposits, boulders or other large obstructions. Auger refusal is defined as the point at which an auger can no longer be practically advanced. It must be appreciated that conventional geotechnical sampling equipment does not differentiate between nature and size of obstructions that prevent further penetration of the boring below grade. Bedrock indicated on the borehole logs will be labeled 'possibly' or 'probable' etc. based on the response of the boring and sampling equipment, surrounding topography, etc. Bedrock can be proven at individual borehole locations, at your request, by diamond core drilling operations or, possibly, by testpits. It must also be appreciated that bedrock surfaces can be, and most times are, very erratic in nature (i.e. sheer drops, isolated rock knobs, etc.) and caution must be used when interpreting subsurface conditions between boreholes. A bedrock profile can be more accurately estimated, at the clients' request, through a series of closely positioned unsampled auger probes combined with core drilling.
4. **GROUNDWATER:** Although the groundwater table may have been encountered during this investigation and the elevation noted in the report and/or on the record of boreholes, it must be appreciated that the elevation of the groundwater table will fluctuate based upon seasonal conditions, localized changes, erratic changes in the underlying soil profile between boreholes, underlying soil layers with highly variable permeabilities, etc. These conditions may affect the design and type and nature of dewatering procedures. Cave-in levels recorded in borings give a general indication of the groundwater level in cohesionless soils however, it must be noted that cave-in levels may also be due to the relative density of the deposit, drilling operations etc.

METRIC

RECORD OF BOREHOLE NO. 01



REFERENCE 12/11/12218-F3 DATUM Geodetic LOCATION N 5158833.9 E 288453.0 - Station 12+401.7 Balfour Township ORIGINATED BY JL
 PROJECT GWP 5223-14-00, Highway 144, Site No. 46-051 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM DATE (Started) 19 August 2013 TIME (Completed) 5:00:00 PM CHECKED BY MAM
 DATE (Completed) 19 August 2013

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES								
268.5	Ground Surface												
0.0	150 mm Asphalt 300 mm Crushed Gravel FILL - sand and gravel trace silt brown, dry (compact/very dense)		1	SS	72		268						
			2	SS	69		267						47 47 (6)
			3	SS	17		266						
			4	SS	41		265						
265.1			5	SS	19		264						
3.4	SAND - with silt brown, dry (very loose) moist		6	SS	2		263						0 75 (25)
			7	SS	2		262						
			8	SS	2		261						
261.8							260						
6.7	SILTY CLAY grey, wet 6 mm silt varves at 25 mm spacing (stiff)		9	SS	WH		259						
			10	SS	PM		258						
			11	SS	PM								
257.1													
11.4	Auger Refusal End of Borehole												
COMMENTS								+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE					
The stratification lines represent approximate boundaries. The transition may be gradual.								WATER LEVEL RECORDS					
								Date (dd/mm/yy)/Time		Water Depth (m)		Cave In (m)	
								1) 19/8/13 5:00:00 PM		6.7		▽ - 5	
2) 21/8/13 5:05:00 PM		6.7		▽ -									
3) 22/8/13 8:40:00 AM		6.7		▽ -									

MEL-GEO 12218 - BOREHOLE LOGS (ADDITIONAL) GPJ MEL-GEO.GDT 23/12/15

EnGlobe Corp.

120 Progress Court, North Bay, On P1A 0C2 Phone: (705)476-2550 Fax: (705)476-8882 Email: northbay@ivm.ca

METRIC

RECORD OF BOREHOLE NO. 02



REFERENCE	12/11/12218-F3	DATUM	Geodetic	LOCATION	N 5158834.8 E 288407.7 - Station 12+447 Balfour Township	ORIGINATED BY	JL
PROJECT	GWP 5223-14-00, Highway 144, Site No. 46-051			BOREHOLE TYPE	Truck Mounted CME 45B - Hollow Stem Augers	COMPILED BY	AT
CLIENT	AECOM	DATE (Started)	20 August 2013	TIME (Completed)		CHECKED BY	MAM
		DATE (Completed)	20 August 2013				

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)	
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)					
								<div><div><div>○ UNCONFINED</div><div>● QUICK TRIAXIAL</div></div><div><div>+ FIELD VANE</div><div>× LAB VANE</div></div></div>					W _p	W	W _L			
268.8	Ground Surface																	
0.0	100 mm Asphalt 250 mm Concrete 300 mm Crushed Gravel		1	SS	29	<div>▽</div> <div>▽▽</div>												
268.2																		
0.6	FILL - sand and gravel trace silt brown, dry (compact/dense)		2	SS	30													48 46 (6)
			3	SS	27													
			4	SS	50													39 55 (6)
			5	SS	18													
			6	SS	45													37 54 (9)
264.4																		
4.4	FILL - sand some silt brown, dry (loose/compact)		7	SS	17													0 89 (11)
			8	SS	8													
260.7			9	SS	15													
8.1	creosote treated wood in tip CONCRETE (probably footing)																	
259.5																		
9.3	End of Borehole																	
COMMENTS							<div>+³, ×³: Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa</div> <div>○ 3% STRAIN AT FAILURE</div>					WATER LEVEL RECORDS						
												Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)				
The stratification lines represent approximate boundaries. The transition may be gradual												1) 20/8/13 3:30:00 PM			4.1	▽	-	☒
												2) 21/8/13 5:00:00 PM			6.6	▽	-	
												3) 22/8/13 8:35:00 AM			6.6	▽	-	

MEL-GEO 12218 - BOREHOL LOGS (ADDITIONAL).GPJ MEL-GEO.GDT 23/12/15

METRIC

RECORD OF BOREHOLE NO. 03



REFERENCE 12/11/12218-F3 DATUM Geodetic LOCATION N 5158829.5 E 288438.6 - Station 12+416 Balfour Township ORIGINATED BY JL
 PROJECT GWP 5223-14-00, Highway 144, Site No. 46-051 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM DATE (Started) 21 August 2013 TIME (Completed) 11:30:00 AM CHECKED BY MAM
 DATE (Completed) 21 August 2013

SOIL PROFILE		STRATA PLOT	SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)		NUMBER	TYPE			"N" VALUES	20					
268.6	Ground Surface												
0.0	75 mm Asphalt												
268.0	300 mm Concrete												
0.6	250 mm Crushed Gravel		1	SS	27								
	FILL - gravelly sand trace silt												
	brown, dry		2	SS	17								
	(loose/very dense)												
			3	SS	7								
			4	SS	29								
			5	SS	42								
			6	SS	35								
			7	SS	61								
263.4	FILL - sand some silt												
5.2	grey, moist												
	(very loose/loose)		8	SS	6								
			9	SS	5								
			10	SS	1								
258.5	Auger Refusal												
10.1	End of Borehole												
COMMENTS						+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE		WATER LEVEL RECORDS Date (dd/mm/yy)/Time 1) 21/8/13 11:30:00 AM 2) 3)					
The stratification lines represent approximate boundaries. The transition may be gradual.								Water Depth (m) DRY - -		Cave In (m) 6.4 - -			

MEL-GEO 12218 - BOREHOLE LOGS (ADDITIONAL) GPJ MEL-GEO.GDT 23/12/15

METRIC**RECORD OF BOREHOLE NO. 04**

REFERENCE 12/11/12218-F3 DATUM Geodetic LOCATION N 5158830.0 E 288397.6 - Station 12+457 Balfour Township ORIGINATED BY JL
 PROJECT GWP 5223-14-00, Highway 144, Site No. 46-051 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM DATE (Started) 21 August 2013 TIME (Completed) 8:10:00 PM CHECKED BY MAM
 DATE (Completed) 21 August 2013

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
268.9	Ground Surface													
0.0	100 mm Asphalt 200 mm Crushed Gravel FILL - sand and gravel trace silt brown, dry (dense/very dense)		1	SS	45									
			2	SS	78									
			3	SS	44									
			4	SS	51									
265.9														
3.0	FILL - cobbles/boulder size rock mixed with sand with silt grey, wet		5	SS	50/75 mm									
			6	SS	25/25 mm									
264.5	pieces of wood and concrete													
4.4	SILT trace sand grey, wet (compact)		7	SS	28									
263.1														
5.8	SILTY CLAY grey, wet (stiff)		8	SS	WH									
			9	SS	PM									
260.1														
8.8	BEDROCK - black slate fair to excellent to quality		10	RC	Rec=100% RQD=74%									
			11	RC	Rec=100% RQD=98%									
			12	RC	Rec=100% RQD=98%									
256.7														
12.2	End of Borehole													

COMMENTS

Note: Groundwater level in borehole at 0.5 m depth below grade upon completion. Water level NOT Stabilized.

The stratification lines represent approximate boundaries. The transition may be gradual.

+ 3, × 3 : Numbers on right refer to Sensitivity
Numbers on left refer to values greater than 120 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

MEL-GEO 12218 - BOREHOLE LOGS (ADDITIONAL) GPJ MEL-GEO.GDT 23/12/15

METRIC**RECORD OF BOREHOLE NO. 05**

REFERENCE 12/11/12218-F3 DATUM Geodetic LOCATION N 5158829.5 E 288438.6 - Station 12+416 Balfour Township ORIGINATED BY JL
 PROJECT GWP 5223-14-00, Highway 144, Site No. 46-051 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM DATE (Started) 9 September 2014 TIME (Completed) 12:00:00 PM CHECKED BY MAM
 DATE (Completed) 9 September 2014

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)												
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)											
268.5	Ground Surface																									
0.0	150 mm Asphalt		1	AS																						
267.9	250 mm Concrete																									
0.6	200 mm Crushed Gravel																									
	FILL - gravelly sand trace silt																									
	brown, dry		2	SS	22																					
	cobble size rock pieces encountered																									
	(compact/very dense)		3	SS	33																					
			4	SS	113																					
			5	SS	71																					
			6	SS	43																					
			7	SS	34																					
262.9	FILL - sand some silt																									
5.6	grey, moist		8	SS	WH																					
	(very loose)																									
			9	SS	2																					
	silty clay layers encountered below 9.1 m depth		10	SS	3																					
258.4	BEDROCK - black slate																									
10.1	good to excellent quality		11	RC	Rec=100% RQD=75%																					
			12	RC	Rec=100% RQD=94%																					
Continued Next Page																										
COMMENTS							+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE		WATER LEVEL RECORDS <table border="1"> <thead> <tr> <th>Date (dd/mm/yy)/Time</th> <th>Water Depth (m)</th> <th>Cave In (m)</th> </tr> </thead> <tbody> <tr> <td>1) 9/9/14 12:00:00 PM</td> <td>DRY</td> <td>-</td> </tr> <tr> <td>2) 11/9/14 7:15:00 AM</td> <td>5.7</td> <td>-</td> </tr> <tr> <td>3) 12/9/14 1:00:00 PM</td> <td>5.7</td> <td>-</td> </tr> </tbody> </table>						Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)	1) 9/9/14 12:00:00 PM	DRY	-	2) 11/9/14 7:15:00 AM	5.7	-	3) 12/9/14 1:00:00 PM	5.7	-
Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)																								
1) 9/9/14 12:00:00 PM	DRY	-																								
2) 11/9/14 7:15:00 AM	5.7	-																								
3) 12/9/14 1:00:00 PM	5.7	-																								
The stratification lines represent approximate boundaries. The transition may be gradual.																										

MEL-GEO 12218 - BOREHOLE LOGS (ADDITIONAL) GPJ MEL-GEO.GDT 23/12/15

EnGlobe Corp.

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METRIC

RECORD OF BOREHOLE NO. 05



REFERENCE	12/11/12218-F3	DATUM	Geodetic	LOCATION	N 5158829.5 E 288438.6 - Station 12+416 Balfour Township	ORIGINATED BY	JL
PROJECT	GWP 5223-14-00, Highway 144, Site No. 46-051			BOREHOLE TYPE	Truck Mounted CME 45B - Hollow Stem Augers	COMPILED BY	AT
CLIENT	AECOM	DATE (Started)	9 September 2014	TIME		CHECKED BY	MAM
		DATE (Completed)	9 September 2014	(Completed)	12:00:00 PM		

[illegible]

METRIC**RECORD OF BOREHOLE NO. 06**

REFERENCE 12/11/12218-F3 DATUM Geodetic LOCATION N 5158833.9 E 288438.7 - Station 12+416 Balfour Township ORIGINATED BY JL
 PROJECT GWP 5223-14-00, Highway 144, Site No. 46-051 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM DATE (Started) 11 September 2014 TIME (Completed) 11:00:00 AM CHECKED BY MAM
 DATE (Completed) 11 September 2014

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)												
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)											
268.5	Ground Surface																									
0.0	150 mm Asphalt																									
268.1	275 mm Concrete																									
0.4	FILL - sand and gravel trace silt		1	AS																						
	brown, dry		2	SS	20																					
	cobble size rock pieces encountered		3	SS	41																					
	(compact/very dense)		4	SS	50/100 mm																					
			5	SS	52																					
			6	SS	45																					
263.6			7	SS	54																					
4.9	FILL - sand some silt																									
	grey, moist		8	SS	1																					
	(very loose/compact)		9	SS	5																					
			10	SS	5																					
			11	SS	13																					
257.2																										
11.3	BEDROCK - black slate		12	RC	Rec=100% RQD=100%																					
	fair to excellent quality																									
	Continued Next Page																									
COMMENTS							+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE		WATER LEVEL RECORDS <table border="1"> <thead> <tr> <th>Date (dd/mm/yy)/Time</th> <th>Water Depth (m)</th> <th>Cave In (m)</th> </tr> </thead> <tbody> <tr> <td>1) 11/9/14 11:00:00 AM</td> <td>DRY</td> <td>5.3</td> </tr> <tr> <td>2) 11/9/14 3:00:00 PM</td> <td>-</td> <td>-</td> </tr> <tr> <td>3) 30/12/99</td> <td>-</td> <td>-</td> </tr> </tbody> </table>						Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)	1) 11/9/14 11:00:00 AM	DRY	5.3	2) 11/9/14 3:00:00 PM	-	-	3) 30/12/99	-	-
Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)																								
1) 11/9/14 11:00:00 AM	DRY	5.3																								
2) 11/9/14 3:00:00 PM	-	-																								
3) 30/12/99	-	-																								

The stratification lines represent approximate boundaries. The transition may be gradual.

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MEL-GEO 12218 - BOREHOLE LOGS (ADDITIONAL) GPJ MEL-GEO.GDT 23/12/15

METRIC**RECORD OF BOREHOLE NO. 06**

REFERENCE 12/11/12218-F3 DATUM Geodetic LOCATION N 5158833.9 E 288438.7 - Station 12+416 Balfour Township ORIGINATED BY JL
 PROJECT GWP 5223-14-00, Highway 144, Site No. 46-051 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM DATE (Started) 11 September 2014 TIME (Completed) 11:00:00 AM CHECKED BY MAM
 DATE (Completed) 11 September 2014

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
	Continued from Previous Page																
254.2			13	RC	Rec= 60% ROD= 60%		255										
14.3	End of Borehole																

MEL-GEO 12218 - BOREHOLE LOGS (ADDITIONAL) GPJ MEL-GEO.GDT 23/12/15

METRIC**RECORD OF BOREHOLE NO. 07**

REFERENCE 12/11/12218-F3 DATUM Geodetic LOCATION N 5158834.8 E 288405.2 - Station 12+449.2 Balfour Township ORIGINATED BY JL
 PROJECT GWP 5223-14-00, Highway 144, Site No. 46-051 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM DATE (Started) 11 September 2014 TIME
 DATE (Completed) 11 September 2014 (Completed) CHECKED BY MAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
							20	40	60	80	100	20	40	60	
268.8	Ground Surface														
0.0	125 mm Asphalt		1	AS											
268.4	250 mm Concrete														
0.4	FILL - sand and gravel trace silt														
	brown, dry		2	SS	25										
	(compact/very dense)														
	cobble size rock pieces encountered		3	SS	40										
			4	SS	58										
			5	SS	44										
264.8			6	SS	37										
4.0	FILL - sand some silt		7	SS	22										
	brown, moist														
	(very loose/compact)		8	SS	2										
			9	SS	13										
259.3	pieces of rock and wood encountered below 9.3 m depth		10	SS	25/50 mm										
9.5	BEDROCK - black slate		11	RC	Rec=100% RQD=100%										
	excellent quality		12	RC	Rec=100% RQD=100%										
256.3															
12.5	End of Borehole														
COMMENTS							+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE			WATER LEVEL RECORDS Date (dd/mm/yy)/Time Water Depth (m) Cave In (m) 1) - ▽ - 2) - ▽ - 3) - ▽ -					

The stratification lines represent approximate boundaries. The transition may be gradual.

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MEL-GEO 12218 - BOREHOLE LOGS (ADDITIONAL) GPJ MEL-GEO GDT 23/12/15

METRIC**RECORD OF BOREHOLE NO. 08**

REFERENCE 12/11/12218-F3 DATUM Geodetic LOCATION N 5158830.3 E 288404.6 - Station 12+450 Balfour Township ORIGINATED BY JL
 PROJECT GWP 5223-14-00, Highway 144, Site No. 46-051 BOREHOLE TYPE Truck Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM DATE (Started) 10 September 2014 TIME (Completed) 12:00:00 PM CHECKED BY MAM
 DATE (Completed) 10 September 2014

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)												
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)											
268.8	Ground Surface																									
0.0	125 mm Asphalt																									
268.4	250 mm Concrete																									
0.4	FILL - sand and gravel trace silt		1	AS																						
	brown, dry		2	SS	74																					
	(dense/very dense)		3	SS	75																					
	cobble size rock pieces encountered		4	SS	57																					
			5	SS	39																					
			6	SS	41																					
264.5	FILL - sand some silt																									
4.3	brown, moist		7	SS	24																					
	(loose/compact)		8	SS	15																					
	wet																									
261.7	SILTY CLAY																									
7.1	grey, wet		9	SS	40/100 mm																					
261.0	BEDROCK - black slate																									
7.8	good to excellent quality		10	RC	Rec=100% RQD=87%																					
			11	RC	Rec=100% RQD=100%																					
257.8	End of Borehole																									
11.0																										
COMMENTS								+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE		WATER LEVEL RECORDS <table border="1"> <thead> <tr> <th>Date (dd/mm/yy)/Time</th> <th>Water Depth (m)</th> <th>Cave In (m)</th> </tr> </thead> <tbody> <tr> <td>1) 10/9/14 12:00:00 PM</td> <td>DRY</td> <td>-</td> </tr> <tr> <td>2) 11/9/14 11:00:00 AM</td> <td>4.9</td> <td>-</td> </tr> <tr> <td>3) 12/9/14 1:30:00 PM</td> <td>4.9</td> <td>-</td> </tr> </tbody> </table>					Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)	1) 10/9/14 12:00:00 PM	DRY	-	2) 11/9/14 11:00:00 AM	4.9	-	3) 12/9/14 1:30:00 PM	4.9	-
Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)																								
1) 10/9/14 12:00:00 PM	DRY	-																								
2) 11/9/14 11:00:00 AM	4.9	-																								
3) 12/9/14 1:30:00 PM	4.9	-																								

The stratification lines represent approximate boundaries. The transition may be gradual.

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MEL-GEO 12218 - BOREHOLE LOGS (ADDITIONAL) GPJ MEL-GEO.GDT 23/12/15

METRIC**RECORD OF BOREHOLE NO. 09**

REFERENCE 12/11/12218-F3 DATUM Geodetic LOCATION N 5158852.6 E 288454.3 - Station 12+400.7 Balfour Township ORIGINATED BY JL
 PROJECT GWP 5223-14-00, Highway 144, Site No. 46-051 BOREHOLE TYPE Track Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM DATE (Started) 30 September 2014 TIME
 DATE (Completed) 1 October 2014 (Completed) 10:00:00 AM CHECKED BY MAM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)													
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa						WATER CONTENT (%)												
						20	40	60	80	100	20	40	60													
264.8	Ground Surface																									
0.0	SAND - some silt brown (very loose/loose)		1	SS	WH																					
			2a	SS	7																					
			2b	SS																						
	trace wood encountered		3	SS	5																					
			4	SS	2																					
			5	SS	6																					
	trace wood encountered		6	SS	5																					
260.4	SILTY CLAY																									
4.4	grey (stiff)		7	TO	2																					
			8	SS	WH																					
			9	SS	PM																					
			10	SS	50/50 mm																					
256.9	BEDROCK - black slate																									
7.9	excellent quality		11	RC	Rec=100% ROD=100%																					
			12	RC	Rec=100% ROD=91%																					
253.9	End of Borehole																									
10.9																										
COMMENTS						+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE		WATER LEVEL RECORDS <table border="1"> <thead> <tr> <th>Date (dd/mm/yy)/Time</th> <th>Water Depth (m)</th> <th>Cave In (m)</th> </tr> </thead> <tbody> <tr> <td>1) 30/9/14 4:30:00 PM</td> <td>0.9</td> <td>▽</td> </tr> <tr> <td>2) 2/10/14 11:00:00 AM</td> <td>0.5</td> <td>▽</td> </tr> <tr> <td>3)</td> <td>-</td> <td>▽</td> </tr> </tbody> </table>							Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)	1) 30/9/14 4:30:00 PM	0.9	▽	2) 2/10/14 11:00:00 AM	0.5	▽	3)	-	▽
Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)																								
1) 30/9/14 4:30:00 PM	0.9	▽																								
2) 2/10/14 11:00:00 AM	0.5	▽																								
3)	-	▽																								

The stratification lines represent approximate boundaries. The transition may be gradual.

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MEL-GEO 12218 - BOREHOLE LOGS (ADDITIONAL) GPJ MEL-GEO.GDT 23/12/15

METRIC

RECORD OF BOREHOLE NO. 10



REFERENCE 12/11/12218-F3 DATUM Geodetic LOCATION N 5158848.9 E 288412.2 - Station 12+443 Balfour Township ORIGINATED BY JL
 PROJECT GWP 5223-14-00, Highway 144, Site No. 46-051 BOREHOLE TYPE Track Mounted CME 45B - Hollow Stem Augers COMPILED BY AT
 CLIENT AECOM DATE (Started) 1 October 2014 TIME (Completed) 4:30:00 PM CHECKED BY MAM
 DATE (Completed) 1 October 2014

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
264.5	Ground Surface													
0.0	150 mm Organic Soil		1	SS	4									1 54 (45)
	SAND AND SILT to silty sand													
	brown		2	SS	5									
	(loose)													
			3	SS	5									0 67 (33)
262.4	SANDY SILT - trace clay													
2.1	grey		4	SS	12									0 37 58 5
261.6	(compact)													
2.9	SILTY CLAY													
	grey		5	SS	8									0 1 71 28
	(stiff)													
	silt varves 19 to 25 mm thick encountered		6	SS	1									
			7	TO	PM									
			8	SS	WH									
257.1	BEDROCK - black slate													
7.4	poor to excellent quality		9	RC	Rec=100% RQD=49%									
			10	RC	Rec=100% RQD=93%									
			11	RC	Rec=100% RQD=100%									
253.5	End of Borehole													
11.0														
COMMENTS								+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE						
The stratification lines represent approximate boundaries. The transition may be gradual.								WATER LEVEL RECORDS						
								Date (dd/mm/yy)/Time			Water Depth (m)		Cave In (m)	
								1) 1/10/14 4:30:00 PM			2.5		2.8	
								2) -			-		-	
								3) -			-		-	

MEL-GEO 12218 - BOREHOLE LOGS (ADDITIONAL) GPJ MEL-GEO.GDT 23/12/15

EnGlobe Corp.

120 Progress Court, North Bay, On P1A 0C2 Phone: (705)476-2550 Fax: (705)476-8882 Email: northbay@vm.ca

Rock Cores – Borehole No. 4

Photo: 1



Rock Cores – Borehole No. 5

Photo: 2



Rock Cores – Borehole No. 6

Photo: 3



Project: Hwy 144 – Replacement of Whitson River Bridge

Photos Provided By: Englobe

Date: December 2015

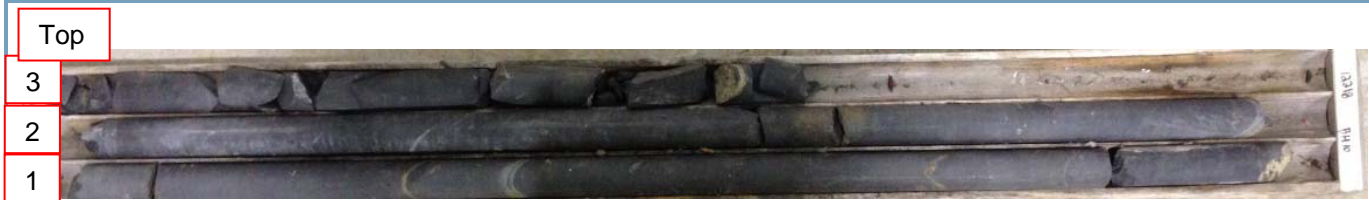
Rock Cores – Borehole No. 9

Photo: 4



Rock Cores – Borehole No. 10

Photo: 5



Project: Hwy 144 – Replacement of Whitson River Bridge

Photos Provided By: Englobe

Date: December 2015

Appendix 3 Lab Data

Drawing Nos. 2a and 2b: Borehole Location and Soil Strata

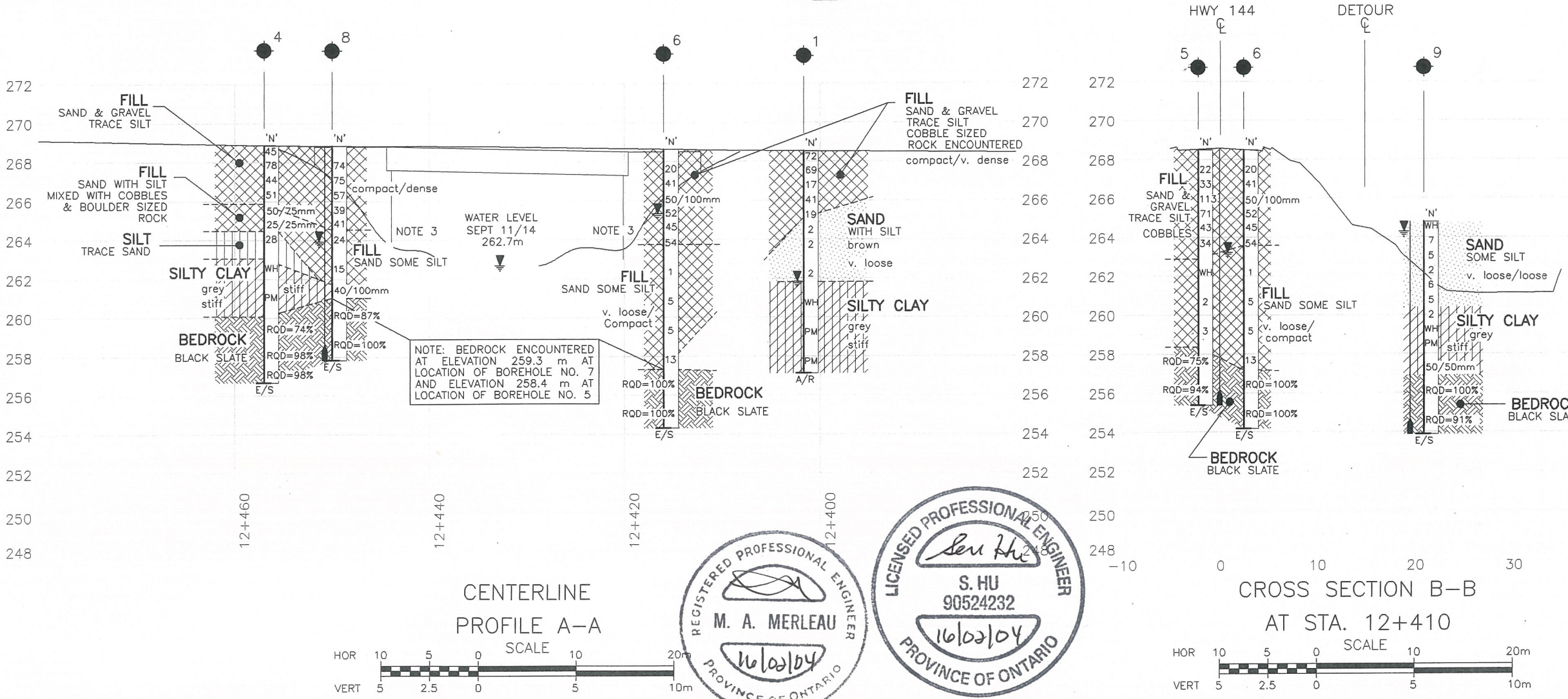
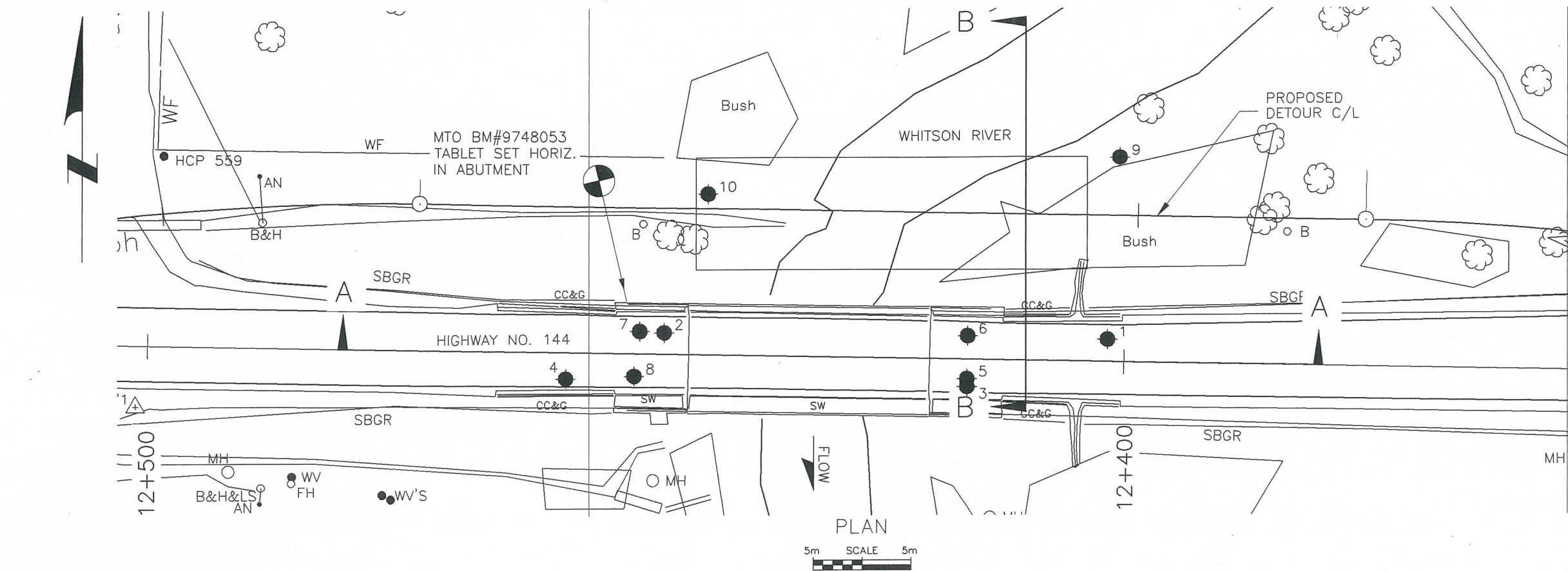
Figure Nos. L-1 to L-7: Grain Size Distribution Curves

Figure No. L-8: Atterberg Limits Summary

Figure No. L-9: Shear Strength Summary Chart

Figure Nos. G1 to G3: Photos and Data of Unconfined
Compression Test

Table No. L-10: Lab Test Summary Sheet

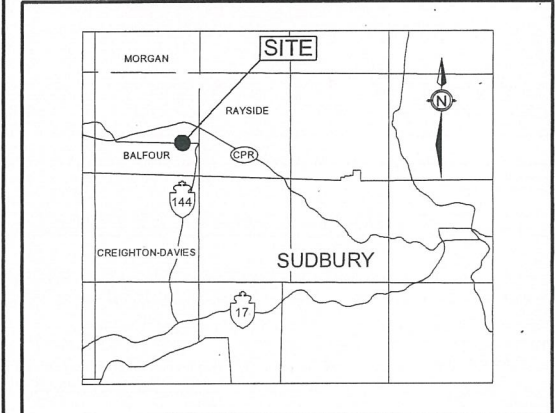


CONT. No.
XXXX-XXXX

GWP. No.
5223-14-00

DRAWING
2a

HWY 144
WHITSON RIVER BRIDGE (SITE NO. 46-051)
BALFOUR TOWNSHIP
BOREHOLE LOCATIONS & SOIL STRATA



- LEGEND
- Borehole
 - ⊕ Dynamic Cone Penetration Test (DCPT)
 - ⊙ Borehole w/ DCPT
 - N Blows/0.3 m (Std Pen Test, 475 J/blow)
 - DCPT Blows/0.3 m (60' Cone, 475 J/blow)
 - ▽ Water Level at Time of Investigation
 - A/R Auger Refusal
 - E/S End of Sampling

Borehole No.	Elev.	O/S	Co-ordinates	
			Northerly	Easterly
Borehole No. 1	268.5	2.3 m Rt	5158833.9	288453.0
Borehole No. 2	268.8	2.2 m Rt	5158834.8	288407.7
Borehole No. 3	268.6	2.8 m Lt	5158829.1	288438.6
Borehole No. 4	268.9	2.7 m Lt	5158830.0	288397.6
Borehole No. 5	268.5	2.0 m Lt	5158829.9	288438.6
Borehole No. 6	268.5	2.4 m Rt	5158834.3	288438.7
Borehole No. 7	268.8	2.3 m Rt	5158834.8	288405.2
Borehole No. 8	268.8	2.3 m Lt	5158830.3	288404.6
Borehole No. 9	264.8	21.0 m Rt	5158852.6	288454.3
Borehole No. 10	264.5	16.5 m Rt	5158848.9	288412.2

NOTE 1: This drawing is for subsurface information only. Surface details and features are for conceptual illustration. The proposed structure location is shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

NOTE 2: The boundaries between soil strata have been established at the borehole locations only. The boundaries illustrated and stratigraphy between boreholes on this drawing are assumed based on borehole data and may vary. They are intended for design only.

NOTE 3: Top of footing elevations are shown in the General Arrangement Drawing D-4476/1 dated March 1960.

NOTE 4: Coordinates based on MTM Zone 12 NAD83 CSRS

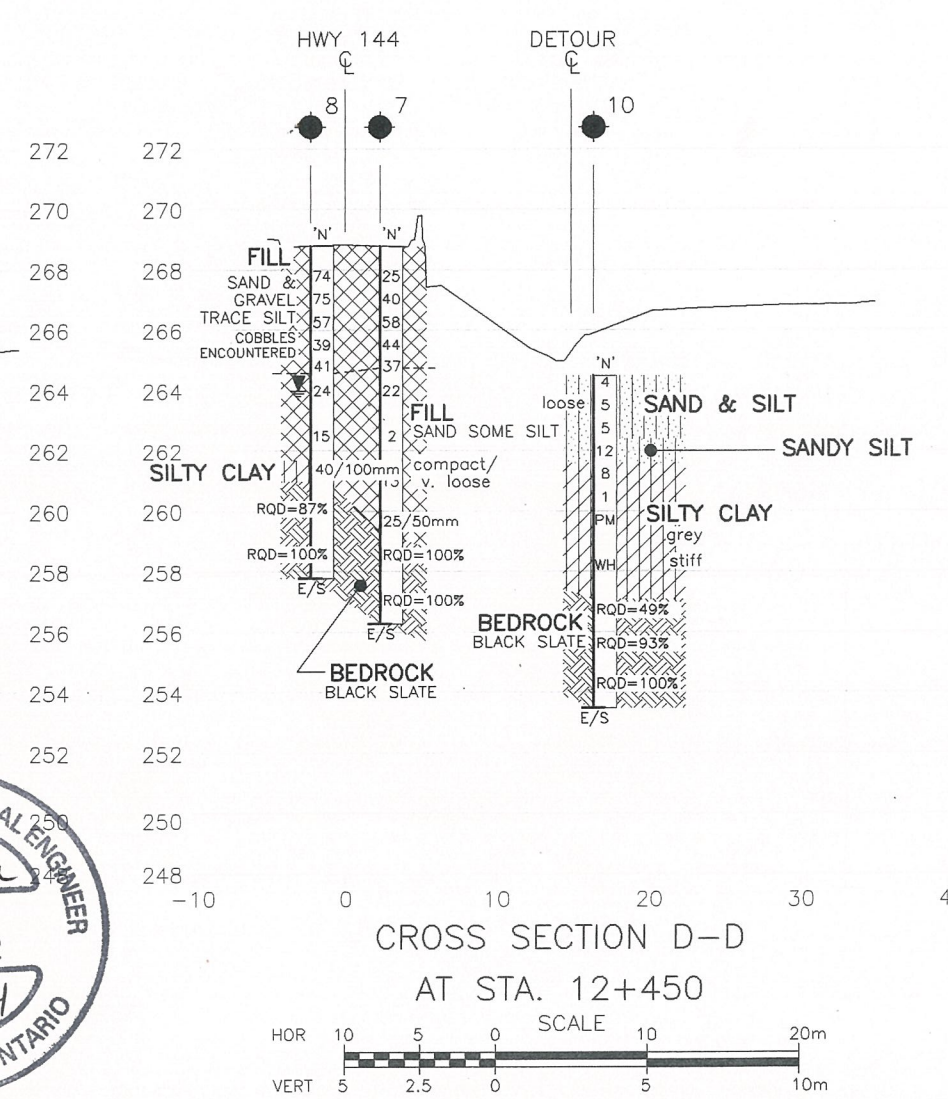
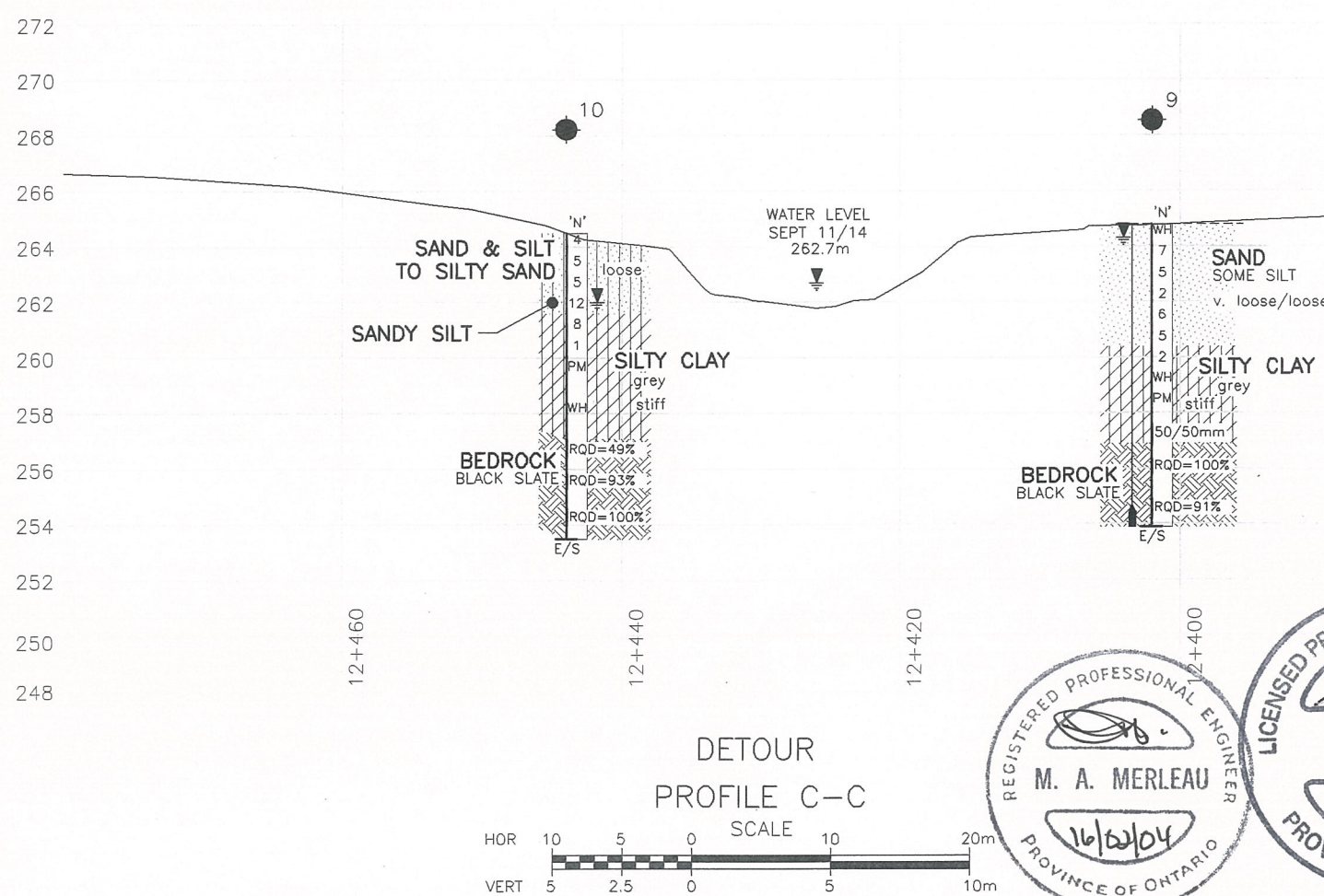
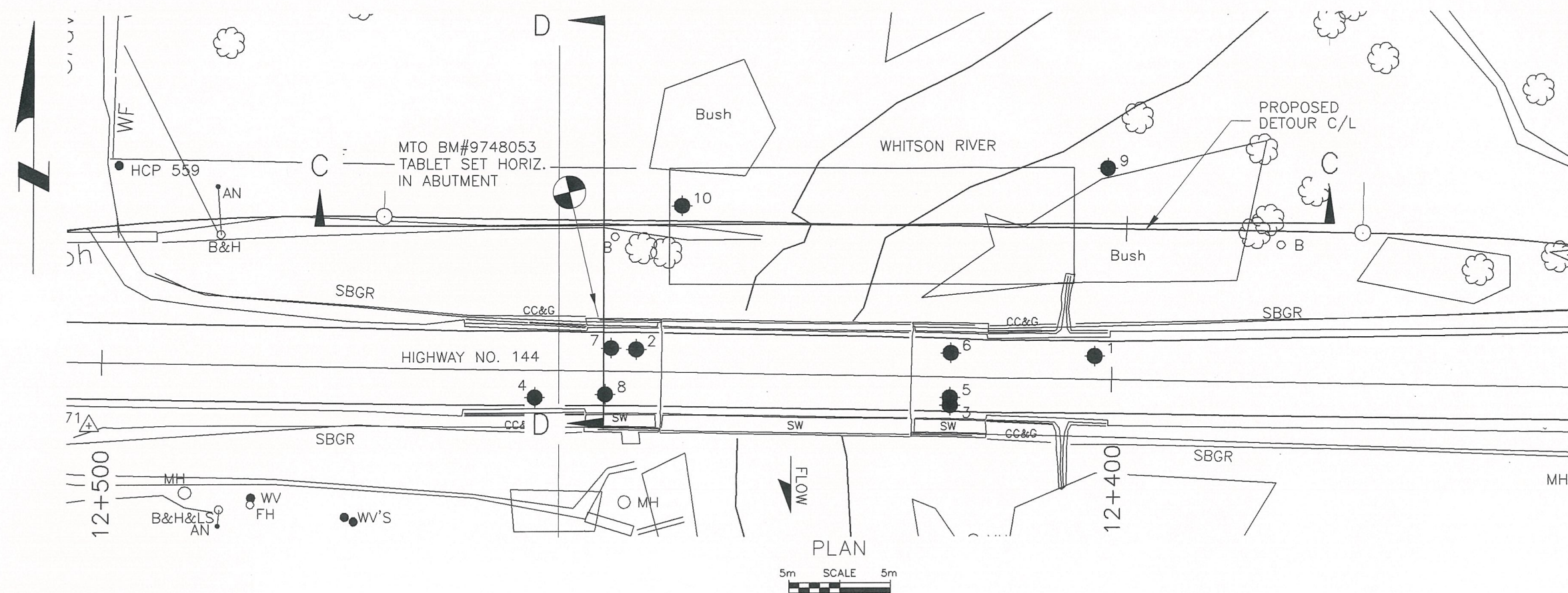
REVISIONS	DATE	BY	DESCRIPTION
1	SEPT 2015	RG	DRAFT
2	JAN 2016	DM	FINAL

HWY NO. 144 - BALFOUR TOWNSHIP

GEOCRES NO.: 411-338

L/V/M REF. NO.: 12/11/12218-F3

DRAWN: RG CHECKED: AT DATE: JANUARY 2016



REGISTERED PROFESSIONAL ENGINEER
M. A. MERLEAU
16/02/04
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
S. HU
90524232
16/02/04
PROVINCE OF ONTARIO

CONT. No.
XXXX-XXXX
GWP. No.
5223-14-00

DRAWING
2b

HWY 144
WHITSON RIVER BRIDGE (SITE NO. 46-051)
BALFOUR TOWNSHIP
BOREHOLE LOCATIONS & SOIL STRATA

KEY PLAN - NOT TO SCALE

LEGEND

- Borehole
- Borehole w/ DCPT
- N Blows/0.3 m (Std Pen Test, 475 J/blow)
- DCPT Blows/0.3 m (60' Cone, 475 J/blow)
- Water Level at Time of Investigation
- A/R Auger Refusal
- E/S End of Sampling

Borehole No.	Elev.	O/S	Co-ordinates	
			Northerly	Easterly
Borehole No. 1	268.5	2.3 m Rt	5158833.9	288453.0
Borehole No. 2	268.8	2.8 m Rt	5158834.8	288407.7
Borehole No. 3	268.6	2.2 m Lt	5158829.1	288438.6
Borehole No. 4	268.9	2.7 m Lt	5158830.0	288397.6
Borehole No. 5	268.5	2.0 m Lt	5158829.9	288438.6
Borehole No. 6	268.5	2.4 m Rt	5158834.3	288438.7
Borehole No. 7	268.8	2.3 m Rt	5158834.8	288405.2
Borehole No. 8	268.8	2.3 m Lt	5158830.3	288404.6
Borehole No. 9	264.8	21.0 m Rt	5158852.6	288454.3
Borehole No. 10	264.5	16.5 m Rt	5158848.9	288412.2

NOTE 1: This drawing is for subsurface information only. Surface details and features are for conceptual illustration. The proposed structure location is shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

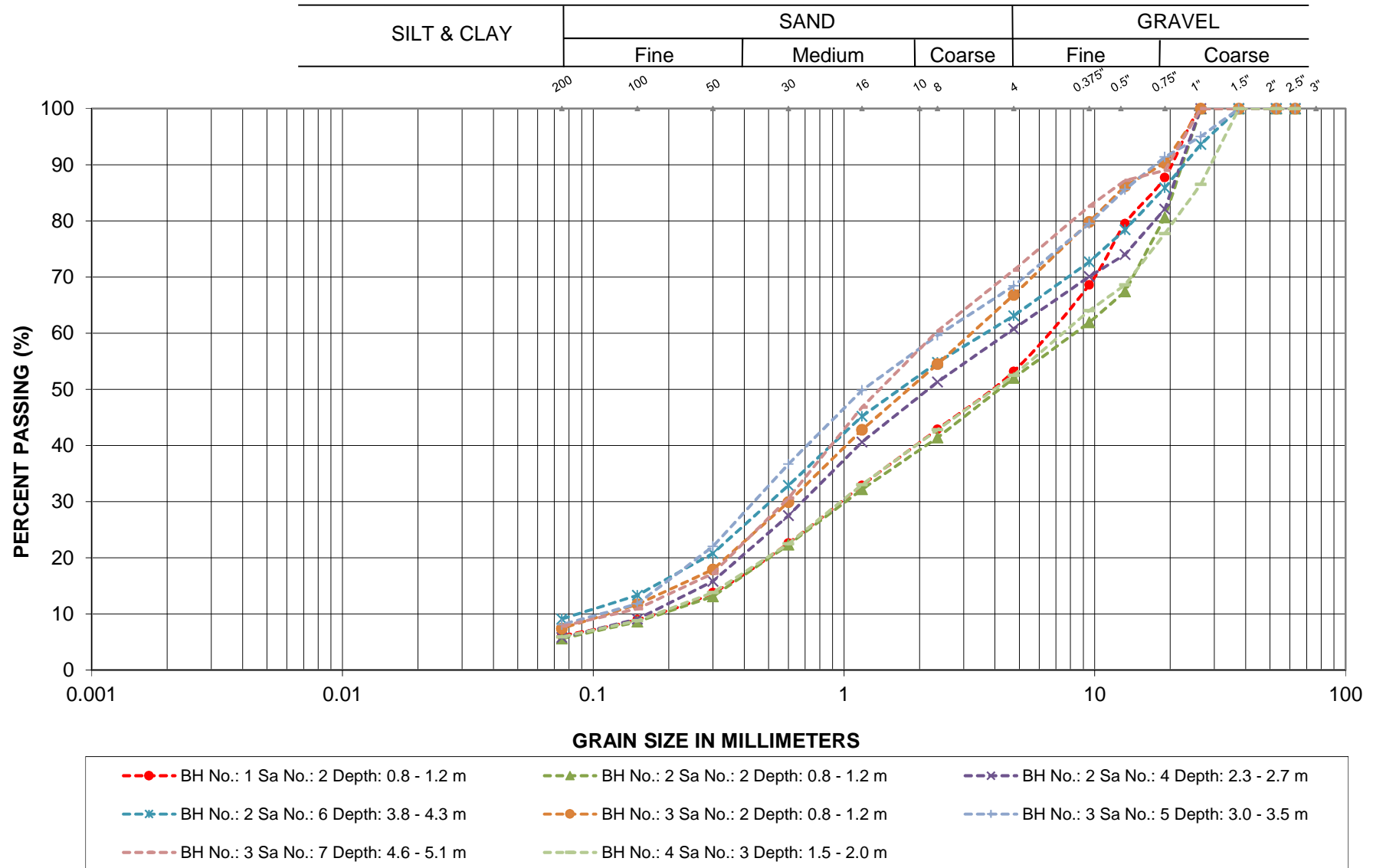
NOTE 2: The boundaries between soil strata have been established at the borehole locations only. The boundaries illustrated and stratigraphy between boreholes on this drawing are assumed based on borehole data and may vary. They are intended for design only.

NOTE 3: Top of footing elevations are shown in the General Arrangement Drawing D-4476/1 dated March 1960.

NOTE 4: Coordinates based on MTM Zone 12 NAD83 CSRS

REVISIONS	DATE	BY	DESCRIPTION
1	SEPT 2015	RG	DRAFT
2	JAN 2016	DM	FINAL

HWY NO. 144 - BALFOUR TOWNSHIP
GEOCRES NO.: 411-338
L/V/M REF. NO.: 12/11/12218-F3
DRAWN: RG CHECKED: AT DATE: JANUARY 2016

GRAIN SIZE ANALYSIS

G.W.P.: 5223-14-00

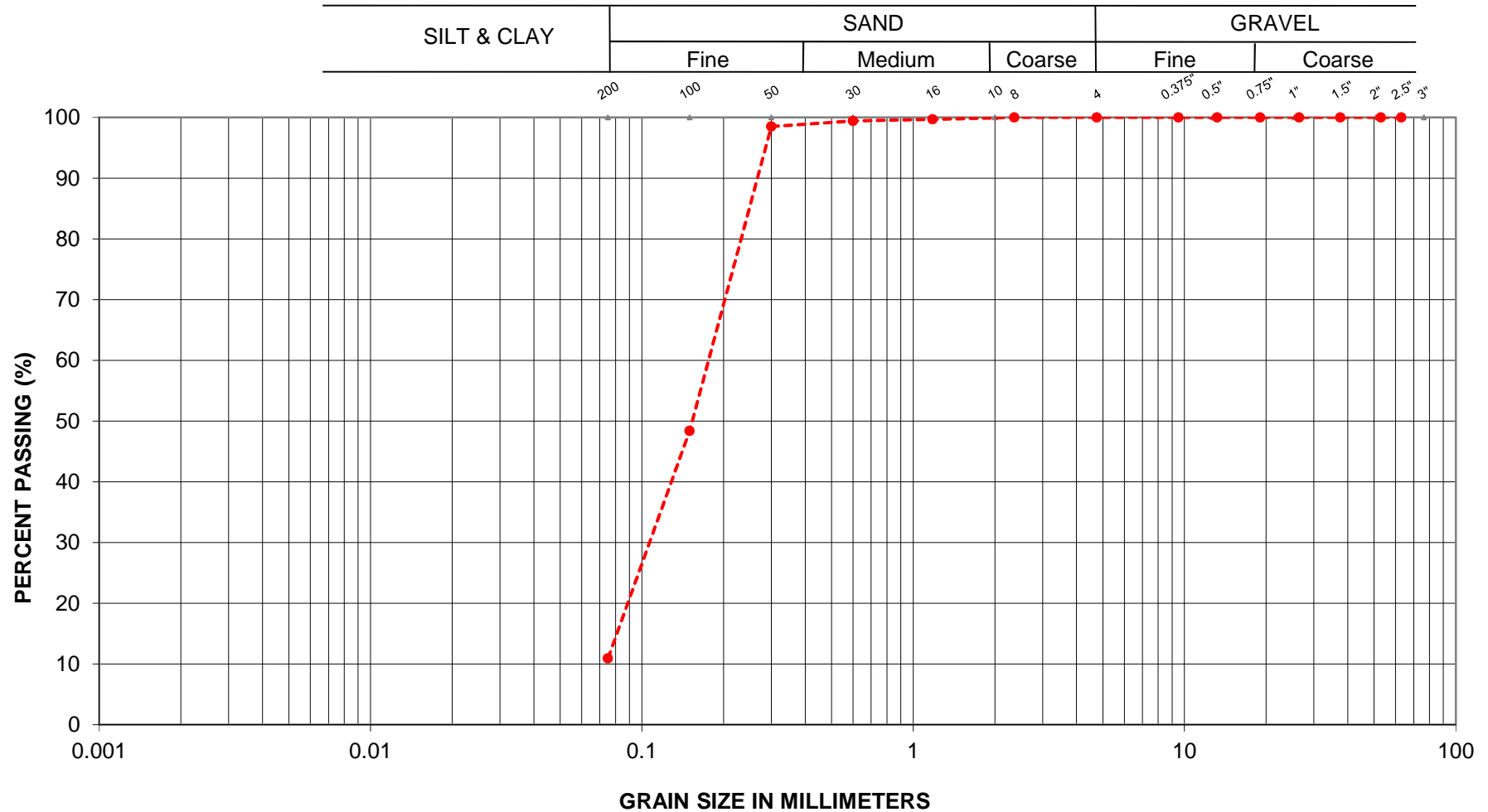
LOCATION: Hwy 144, Whitson River Bridge

EMBANKMENT FILL

ENGLOBE CORP.

FIGURE L-1

GRAIN SIZE ANALYSIS



---●--- BH No.: 2 Sa No.: 7 Depth: 4.6 - 5.1 m

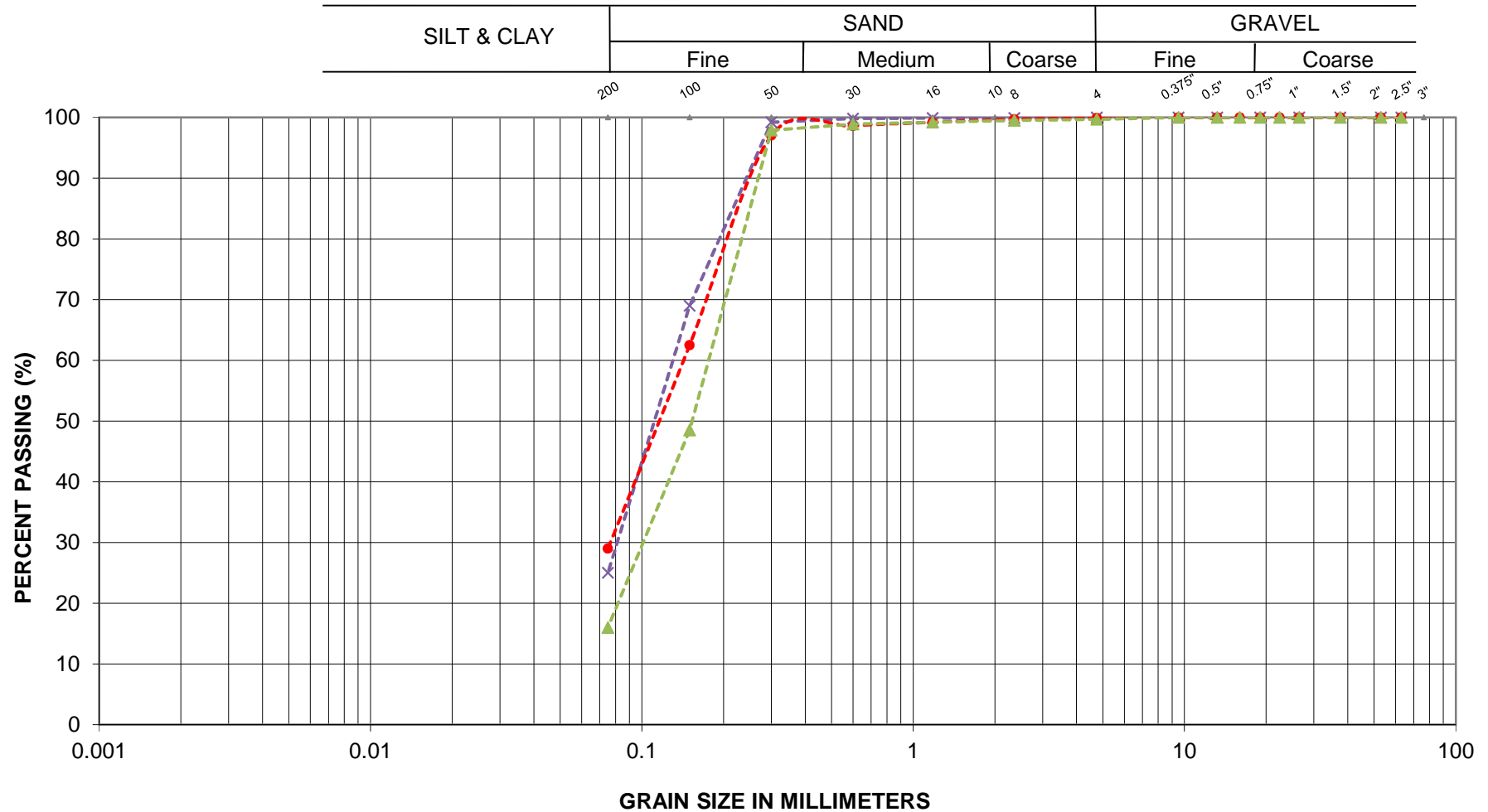
G.W.P.: 5223-14-00

LOCATION: Hwy 144, Whitson River Bridge

SAND FILL

ENGLOBE CORP.

FIGURE L-2

GRAIN SIZE ANALYSIS

---x--- BH No.: 1 Sa No.: 6 Depth: 3.8 - 4.3 m

---●--- BH No.: 9 Sa No.: 2b Depth: 0.8 - 1.2 m

---▲--- BH No.: 9 Sa No.: 5 Depth: 3.0 - 3.5 m

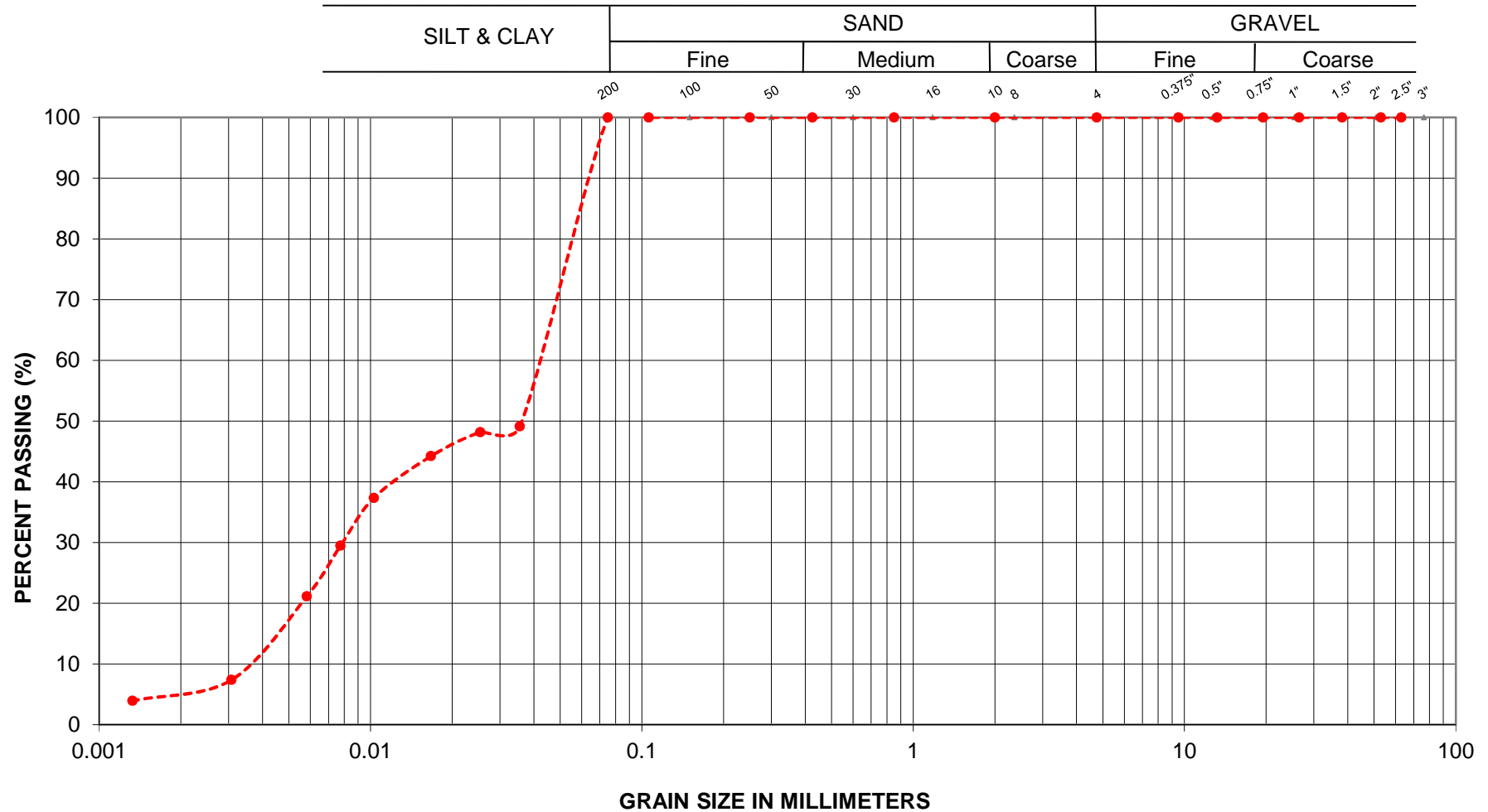
G.W.P.: 5223-14-00

LOCATION: Hwy 144, Whitson River Bridge

SAND

ENGLOBE CORP.

FIGURE L-3

GRAIN SIZE ANALYSIS

---●--- BH No.: 4 Sa No.: 7 Depth: 4.6 - 5.1 m

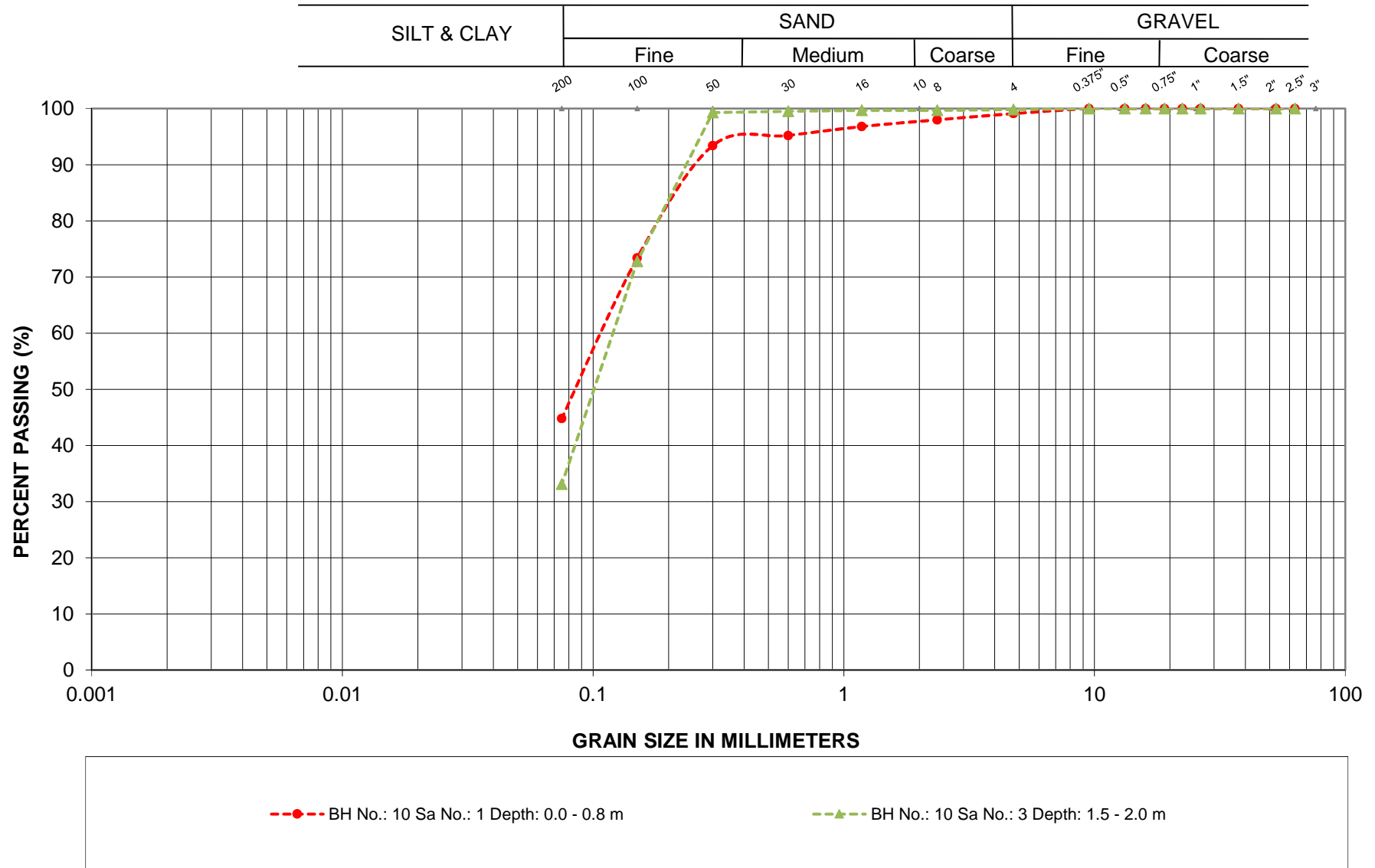
G.W.P.: 5223-14-00

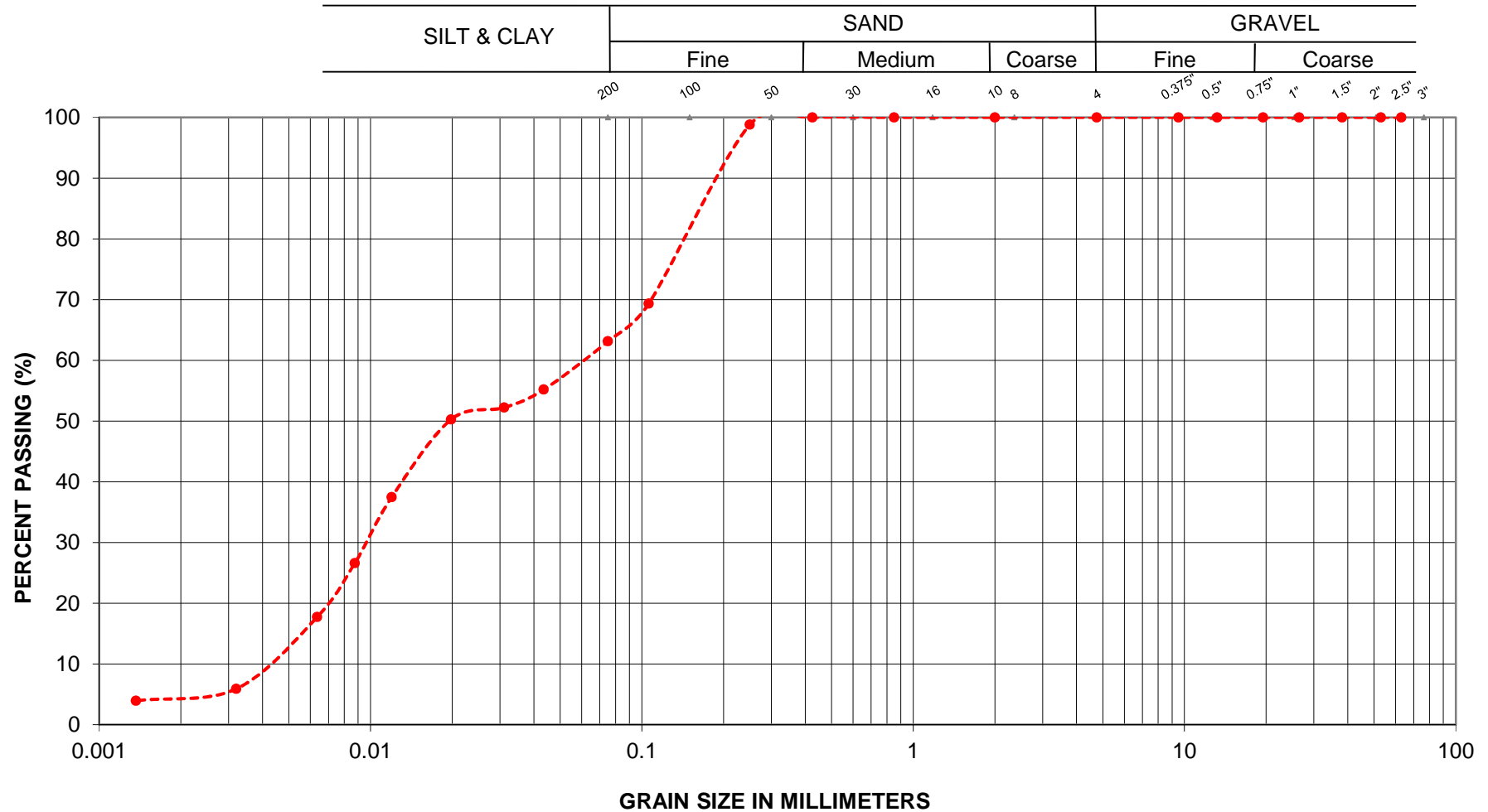
LOCATION: Hwy 144, Whitson River Bridge

SILT

ENGLOBE CORP.

FIGURE L-4

GRAIN SIZE ANALYSIS

GRAIN SIZE ANALYSIS

---●--- BH No.: 10 Sa No.: 4 Depth: 2.3 - 2.7 m

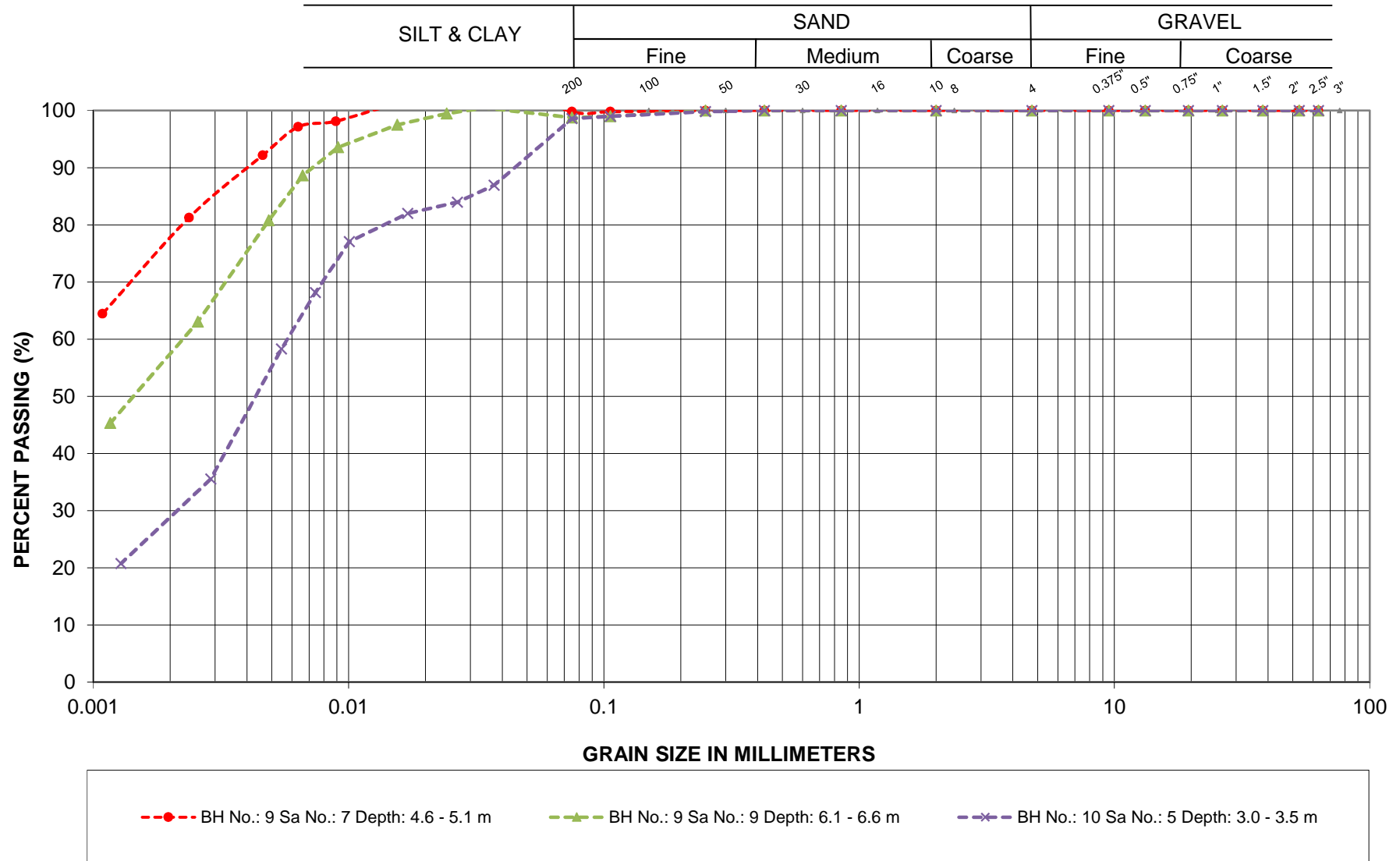
G.W.P.: 5223-14-00

LOCATION: Hwy 144, Whitson River Bridge

SANDY SILT

ENGLOBE CORP.

FIGURE L-6

GRAIN SIZE ANALYSIS

G.W.P.: 5223-14-00

LOCATION: Hwy 144, Whitson River Bridge

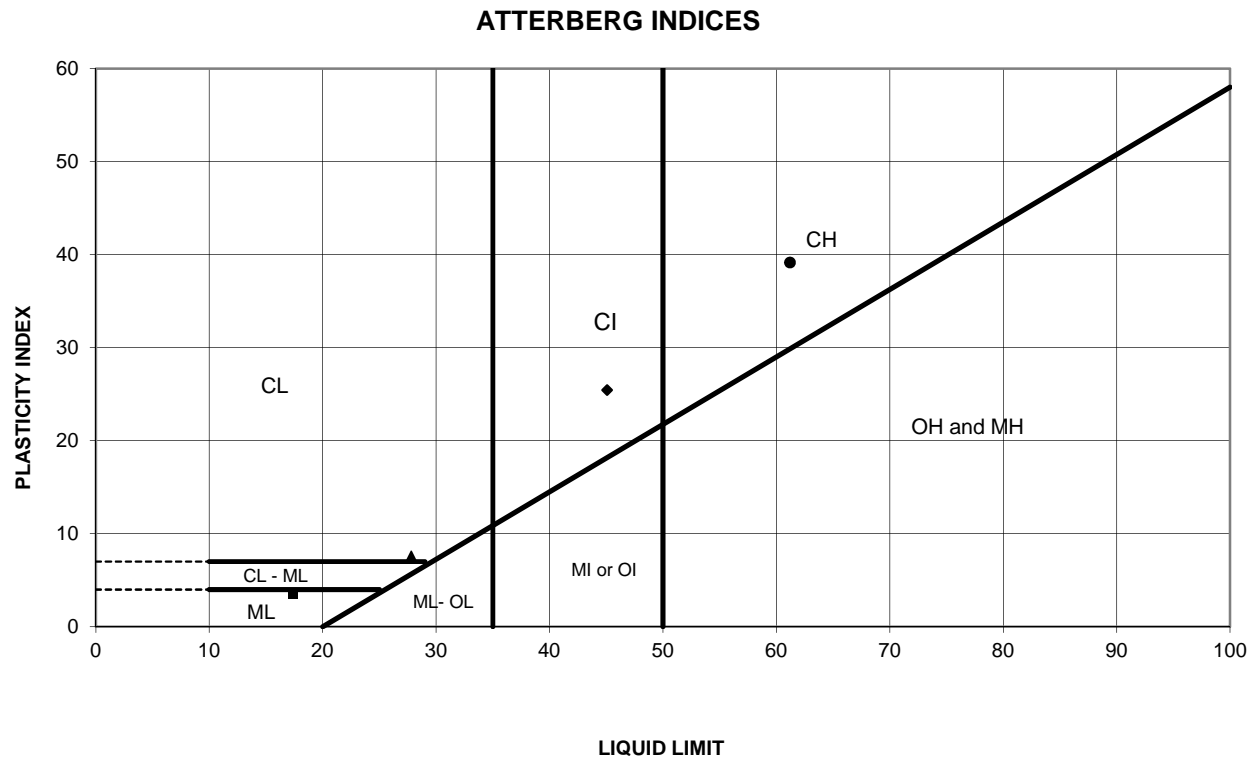
SILTY CLAY

ENGLOBE CORP.

FIGURE L-7

ATTERBERG LIMITS TEST RESULTS

FIGURE L-8

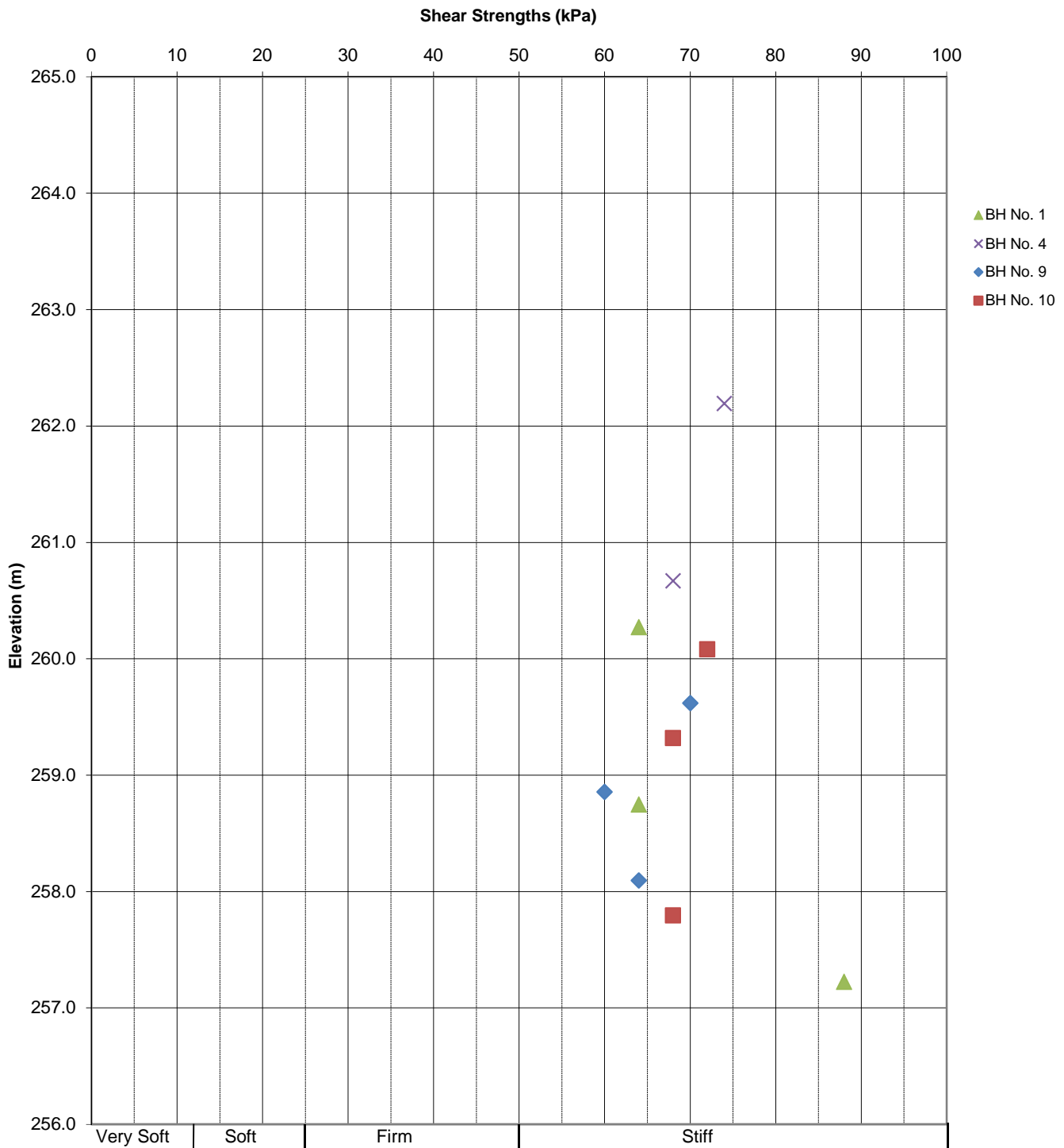


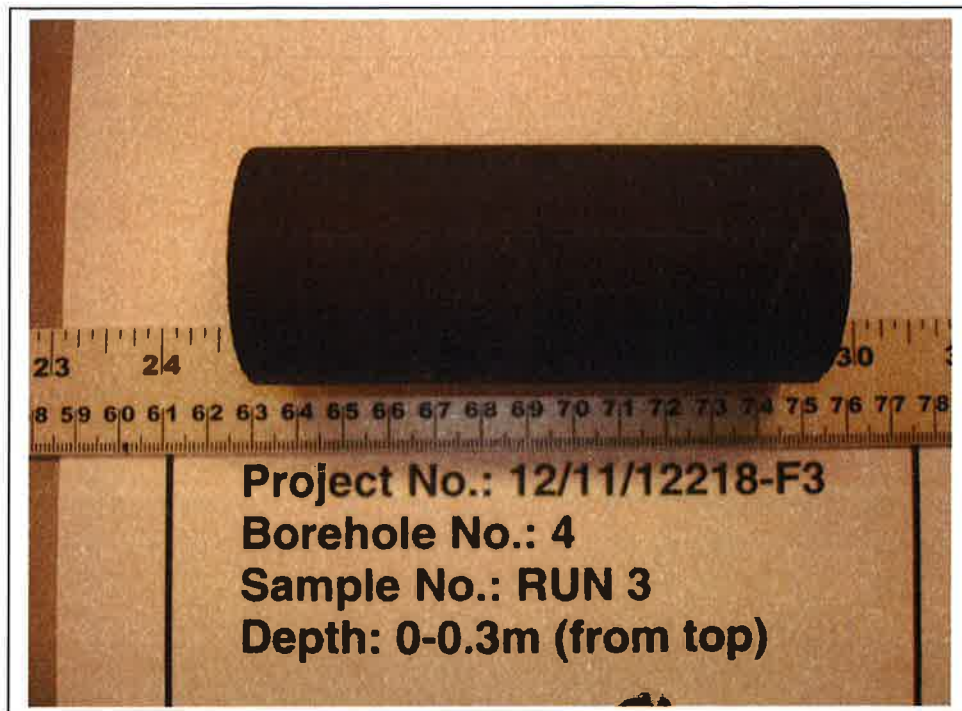
SYMBOL	BH	Sa. No.	Depth(m)	Elev.(m)	Liquid Limit	Plastic Limit	Plasticity Index	NMC %
●	9	7	4.6	260.2	61.2	22.1	39.2	56.9
◆	9	9	6.1	258.7	45.1	19.6	25.4	46.4
■	10	4	2.3	262.2	17.4	13.9	3.5	22.7
▲	10	5	3.0	261.5	27.8	20.3	7.6	30.5

Date: Nov-14
Project: Hwy 144, Whitson River Bridge
G.W.P: 5223-14-00

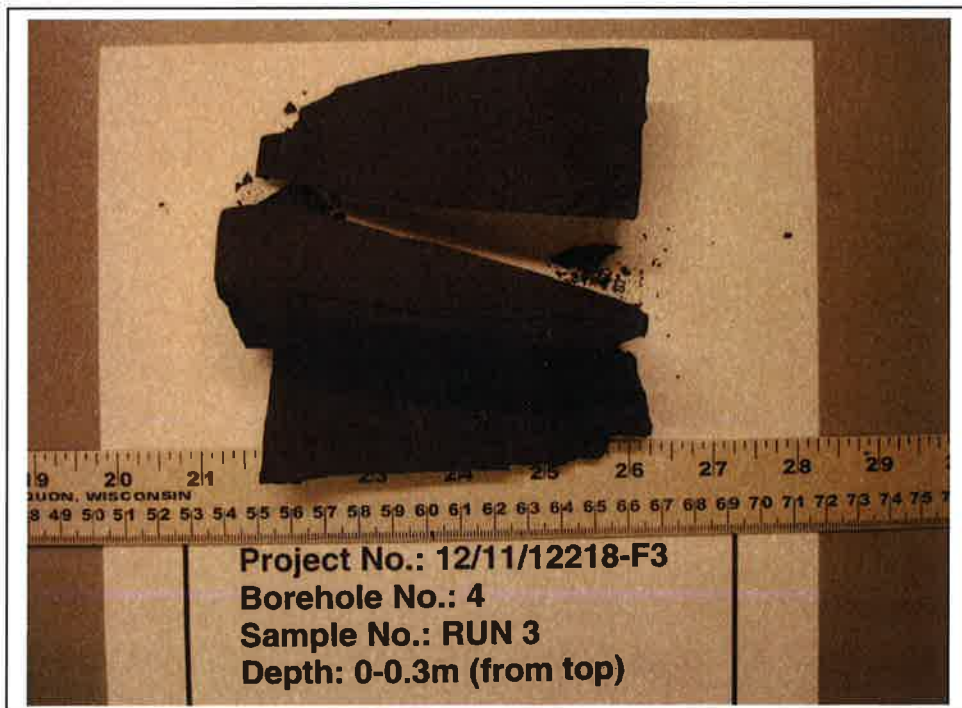
Prep'd: AT
Chkd: MAM
Ref. No.: 12/11/12218-F3

In-Situ Shear Strengths vs. Depth





BEFORE COMPRESSION



AFTER COMPRESSION

Date Dec. 18, 2015
Project 12/11/12218-F3

Golder Associates

Drawn Frank
Chkd. *[Signature]*

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	12/11/12218-F3	SAMPLE NUMBER	Run 3
PROJECT NAME	-	SAMPLE DEPTH, m	0-0.3(from top)
BOREHOLE NUMBER	4	DATE:	2015-12-16

TEST CONDITIONS

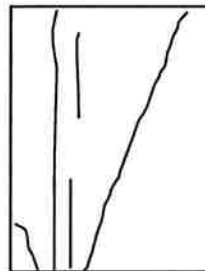
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.34

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	11.10	WATER CONTENT, (specimen) %	0.36
SAMPLE DIAMETER, cm	4.75	UNIT WEIGHT, kN/m ³	25.82
SAMPLE AREA, cm ²	17.71	DRY UNIT WT., kN/m ³	25.72
SAMPLE VOLUME, cm ³	196.67	SPECIFIC GRAVITY	-
WET WEIGHT, g	517.94	VOID RATIO	-
DRY WEIGHT, g	516.08		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	22.9
----------------------	-----	---------------------------	------

REMARKS:

Checked By: *AB*

Golder Associates



BEFORE COMPRESSION



AFTER COMPRESSION

Date Dec. 18, 2015
Project 12/11/12218-F3

Golder Associates

Drawn Frank
Chkd. *[Signature]*

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	12/11/12218-F3	SAMPLE NUMBER	Run 2
PROJECT NAME	-	SAMPLE DEPTH, m	1.04-1.28(from top)
BOREHOLE NUMBER	5	DATE:	2015-12-16

TEST CONDITIONS

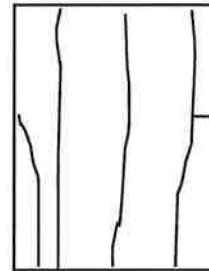
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.31

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.93	WATER CONTENT, (specimen) %	0.43
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m ³	25.80
SAMPLE AREA, cm ²	17.57	DRY UNIT WT., kN/m ³	25.69
SAMPLE VOLUME, cm ³	192.09	SPECIFIC GRAVITY	-
WET WEIGHT, g	505.60	VOID RATIO	-
DRY WEIGHT, g	503.44		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	79.7
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REMARKS:

Checked By: *flb*

Golder Associates



BEFORE COMPRESSION



AFTER COMPRESSION

Date Dec. 18, 2015
Project 12/11/12218-F3

Golder Associates

Drawn Frank
Chkd. *AK*

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	12/11/12218-F3	SAMPLE NUMBER	Run 1
PROJECT NAME	-	SAMPLE DEPTH, m	1.36-1.56(from top)
BOREHOLE NUMBER	9	DATE:	2015-12-16

TEST CONDITIONS

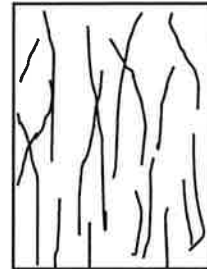
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.28

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.77	WATER CONTENT, (specimen) %	0.47
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m ³	25.85
SAMPLE AREA, cm ²	17.56	DRY UNIT WT., kN/m ³	25.73
SAMPLE VOLUME, cm ³	189.00	SPECIFIC GRAVITY	-
WET WEIGHT, g	498.34	VOID RATIO	-
DRY WEIGHT, g	496.01		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	100.1
----------------------	-----	---------------------------	-------

REMARKS:

Checked By: *Ab*

Golder Associates

Laboratory Tests - Summary Sheet



Borehole No.	Sample No.	Depth	Grain Size Analysis				NMC	Atterberg Limits			SPT 'N'	USCS	Total Unit Weight (kN/m ³)	Remarks
			Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)		LL (%)	PL (%)	IP (%)				
1	1	0.2					2.0				72			
	2	0.8	47	47		6	2.1				69			
	3	1.5					2.8				17			
	4	2.3					4.7				41			
	5a	3.1					3.8				19			
	5b	3.1					8.6				19			
	6	3.8	0	75		25	16.6				2			
	7	4.6					17.9				2			
	8	6.1					30.8				2			
	9	7.6					51.3				WH			
	10	9.1					41.9				PM			
	11	10.7					35.7				PM			
2	1	0.0					3.6				29			
	2	0.8	48	46		6	1.5				30			
	3	1.5					1.6				27			
	4	2.3	39	55		6	2.5				50			
	5	3.1					3.4				18			
	6	3.8	37	54		9	4.6				45			
	7	4.6	0	89		11	7.7				17			
	8	6.1					21.6				8			
	9	7.6					21.0				15			
3	1	0.3					3.3				27			
	2	0.8	33	60		7	2.6				17			
	3	1.5					2.2				7			
	4	2.3					4.5				29			

Laboratory Tests - Summary Sheet



Borehole No.	Sample No.	Depth	Grain Size Analysis				NMC	Atterberg Limits			SPT 'N'	USCS	Total Unit Weight (kN/m ³)	Remarks
			Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)		LL (%)	PL (%)	IP (%)				
3	5	3.1	32	60	8		3.6				42			
	6	3.8					4.5				35			
	7	4.6	29	63	8		4.0				61			
	8	6.1					18.5				6			
	9	7.6					22.0				5			
	10	9.1					24.6				1			
4	1	0.0					3.2				45			
	2	0.8					2.8				78			
	3	1.5	47	47	6		2.5				44			
	4	2.3					5.8				51			
	5	3.1									50/75 mm			
	6	3.8					21.2				25/25 mm			
	7	4.6	0	0	95	5	19.6				28			
	8	6.1					47.8				WH			
	9	7.62									PM			
	10	9									RC			Rec= 100%, RQD= 74%
	11	10.4									RC			Rec= 100%, RQD= 98%
	12	11.6									RC		25.8	Rec= 100%, RQD= 98%, UCS = 22.9 MPa of Unconfined Compression test at depth from 11.6 m to 11.9 m
5	1	0.0					6.2							
	2	0.8					1.8				22			
	3	1.5					2.2				33			
	4	2.3					3.2				113			

Laboratory Tests - Summary Sheet



Borehole No.	Sample No.	Depth	Grain Size Analysis				NMC	Atterberg Limits			SPT 'N'	USCS	Total Unit Weight (kN/m ³)	Remarks
			Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)		LL (%)	PL (%)	IP (%)				
	5	3.1					3.8				71			
	6	3.8					6.1				43			
	7	4.6					4.0				34			
	8	6.1					21.4				WH			
	9	7.6					20.9				2			
	10	9.1					19.1				3			
	11	10.1												RC, Rec= 100%, RQD= 75%
	12	11.6										25.8		RC, Rec= 100%, RQD= 94%, UCS = 79.7 MPa of Unconfined Compression test at depths from 12.64 m to 12.88 m
6	1	0.0					3.5							
	2	0.8					3.6				20			
	3	1.5					2.7				41			
	4	2.3					4.9				50/100 mm			
	5	3.1					5.5				52			
	6	3.8					6.2				45			
	7	4.6					7.5				54			
	8	6.1					24.2				1			
	9	7.6					22.8				5			
	10	9.14					18.8				5			
	11	10.67					22.0				13			
	12	11.3												RC, Rec= 100%, RQD= 100%
	13	12.8												RC, Rec= 100%, RQD= 100%

Laboratory Tests - Summary Sheet



Borehole No.	Sample No.	Depth	Grain Size Analysis				NMC	Atterberg Limits			SPT 'N'	USCS	Total Unit Weight (kN/m ³)	Remarks
			Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)		LL (%)	PL (%)	IP (%)				
7	1	0.0					3.3							
	2	0.8					3.4				25			
	3	1.5					2.7				40			
	4	2.3					4.0				58			
	5	3.1					4.1				44			
	6	3.8					6.5				37			
	7	4.6					19.8				22			
	8	6.1					15.6				2			
	9	7.6					25.7				13			
	10	9.1					30.2				25/50 mm			
	11	9.5												RC, Rec= 100%, RQD= 100%
	12	11.0												RC, Rec= 100%, RQD= 100%
8	1	0.0					2.2							
	2	0.76					2.1				74			
	3	1.52					1.6				75			
	4	2.29					3.5				57			
	5	3.05					3.8				39			
	6	3.81					7.5				41			
	7	4.57					17.6				24			
	8	6.1					28.6				15			
	9	7.62					38.7				40/100 mm			
	10	7.8												RC, Rec= 100%, RQD= 87%
	11	9.3												RC, Rec= 100%, RQD= 100%
9	1	0.0					25.3				WH			
	2a	0.8					54.1				7			

Laboratory Tests - Summary Sheet

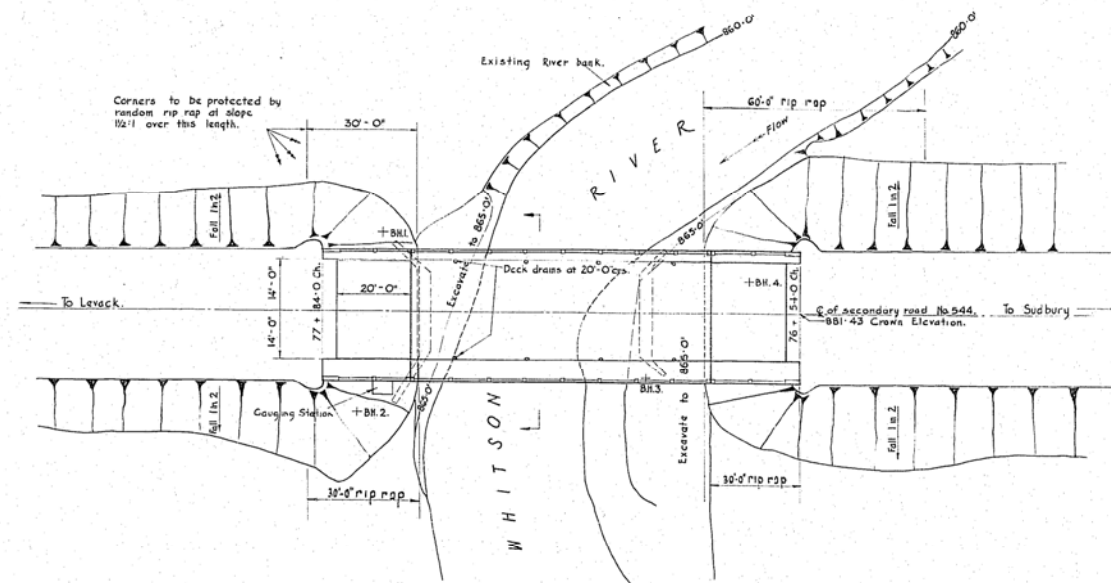


Borehole No.	Sample No.	Depth	Grain Size Analysis				NMC	Atterberg Limits			SPT 'N'	USCS	Total Unit Weight (kN/m ³)	Remarks
			Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)		LL (%)	PL (%)	IP (%)				
	2b	1.0	0	71	29		15.0				7			
	3	1.5					26.4				5			
	4	2.3					29.2				2			
	5	3.1	0	84	16		31.2				6			
	6	3.8					24.2				5			
	7	4.6	0	0	23	27	56.9	22.1	61.2	39.2	2			
	8	5.3					53.3				WH			
	9	6.1	0	1	43	56	46.4	19.6	45.1	25.4	PM			
	10	7.6					33.9				50/50 mm			
	11	7.9										25.9		RC, Rec= 100%, RQD= 100%, UCS = 100.1 MPa of Unconfined Compression test at depths from 9.26 m to 9.46 m
	12	9.4												RC, Rec= 100%, RQD= 91%
10	1	0.0	1	54	45		20.0				4			
	2	0.8					18.8				5			
	3	1.5	0	67	33		18.1				5			
	4	2.29	0	37	58	5	22.7	13.9	17.4	3.5	12			
	5	3.05	0	1	71	28	30.5	20.3	27.8	7.6	8			
	6	3.81					48.4				1			
	7	4.57					53.7				PM			
	8	6.1					36.4				WH			
	9	7.4												RC, Rec= 100%, RQD= 49%
	10	8.2												RC, Rec= 100%, RQD= 93%
	11	9.6												RC, Rec= 100%, RQD= 100%

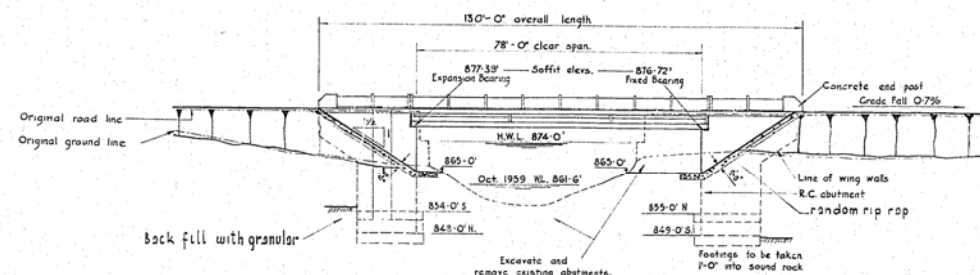
Appendix 4 Historical Data

Enclosure No. 13:

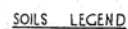
Historical Drawing



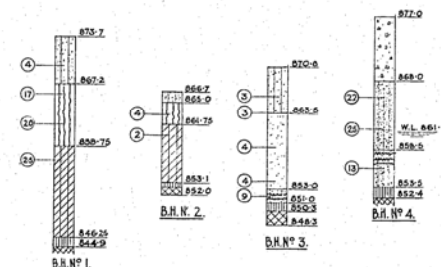
PLAN
Scale: 1 in. to 20 ft.



SOUTH ELEVATION
Scale: 1 in to 20 ft.



- Loose fine grey sand, some organic matter.
Loose grey silt.
Loose brown silty sand.
Medium dense brownish grey silt & sand.
Rotted wood fibre.
Coarse sand, silt & gravel.
Soft grey silty clay.
Dark grey clayey silt.
Bedrock.

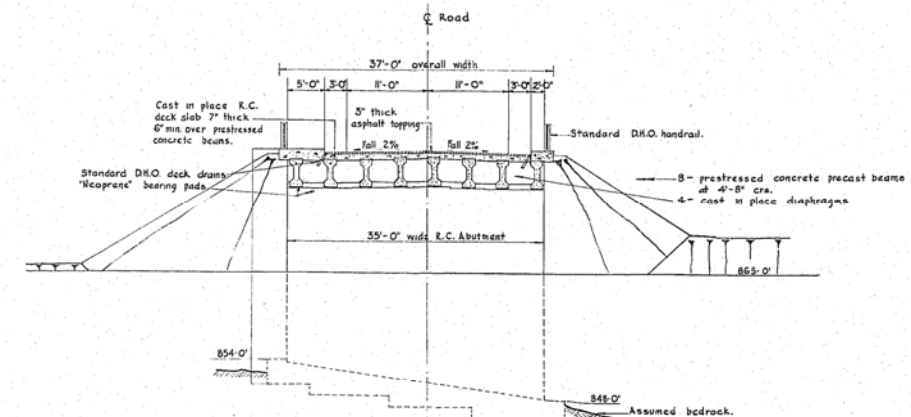


PENETRATION RESISTANCE (3) (blows per foot)
2" SPLIT SPOON.

BORING DATA

The complete soil investigation report BA 971 may be examined at the Bridge Office, Toronto.
The Department does not guarantee the accuracy of this report or the abridged version shown on these plans.

RECORD OF BOREHOLES



CROSS SECTION
Scale: 1 in to 10 ft.

GENERAL NOTES

NOTE TO DISTRICT ENGINEERS:-

NOTE TO DISTRICT ENGINEERS:-
Concrete work on this structure must not be commenced until monuments to fix control points have been erected and checked by the District Engineer.

NOTE TO CONTRACTOR:-
Structure to be built in accordance with the latest form No 9 and the Special Provisions, extra copies of which may be obtained from the District Engineer.

All Construction Joints must be approved by the Bridge Engineer.

CONCRETE DETAILS

Concrete to Footings, Abutments & Wing Walls, Deck Slab, Diaphragms, Sidewalks & Curbs to have a minimum compressive strength of 3000 p.s.i. at 28 days. Concrete to prestressed beams to have a minimum compressive strength of 5000 p.s.i. at 28 days and a minimum compressive strength of 4000 p.s.i. at transfer of prestress.

An approved admixture supplied by the Department will be added to all concrete as specified by the Materials and Research Section, D.H.O.

Maximum aggregate size in footings and abutments below beam seatings $1\frac{1}{2}$ " ; in all other work, $\frac{3}{4}$ "

REINFORCING STEEL:-

Clear cover in footings and abutments to be 2" except where noted otherwise.

Clear cover in Deck Slab, Diaphragms, Sidewalks & Curbs to be 1"
Clear cover in Prestressed Beams to be 1"

CONSTRUCTION NOTES:-

CONSTRUCTION NOTES:-
All exposed edges to be chamfered as shown on the drawings.
No concrete to be placed above bridge seat elevations until
concrete in deck slab has been placed.

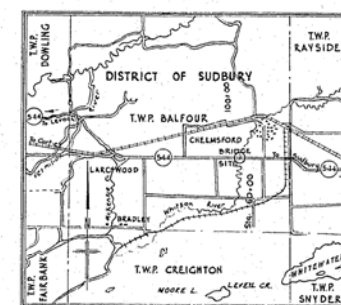
The General Contractor shall be responsible for finishing the bridge seats dead level to the specified elevations with a tolerance of plus or minus $\frac{1}{8}$ inch. If they are cast too high they shall be bush hammered down by the General Contractor. If they are cast too low the General Contractor shall provide full bearing shims to bring them up to the correct elevations. The use of grout is prohibited.

Footings to be taken 1'-0" minimum into sound rock.
Drainage ditches at foot of existing embankments to be maintained.

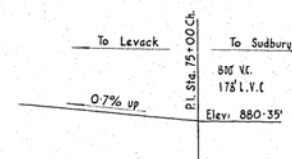
No concrete to be placed in ballast wall until deck has been poured.

J.G.G.	ENCL. ON ABUT. BEAT REVISED. (MAY 31/61 BY J.G.G. TOP PORTION CAUSING STA. ERROR)	RE
J.G.G.	-15 ADDED TO SCHEDULE.	
J.M.	Gauging Station Added	E

J.M.	GAUGING STATION ADDED	C
P.B.M.	NOTE ADDED TO CONSTRUCTION NOTES	M
P.B.A.	END POSTS CORRECTED	B
	DESCRIPTION	



KEY PLAN
Scale: 1 in. to 2 mi.



PROFILE AT CROWN
OF FINISHED PAVEMENT

DESIGN ENGINEER					
BRIDGE ENGINEER					
DESIGN					
DRAWING					
TRACING					
DATE					
LOADINGS					
CHECKING NUMBER					
H20-S16					
D-4476/1					

TWP# 618-51-1-A

Appendix 5 Design Data

Table A:	Foundation Alternatives for New Bridge Abutments
Table B:	Foundation Alternatives for Detour Bridge
Figure No. S-1:	Slope Stability for Detour Embankment Side Slope
Figure No. S-2:	Slope Stability at Station 0+050 on Existing East River Bank
Figure No. S-3:	Slope Stability for Rock Protection (river water level at Elevation 262.7 m) at Station 0+050 on East River Bank
Figure No. S-4:	Slope Stability for Rock Protection (river water level at Elevation 264.2 m) at Station 0+050 on East River Bank
Figure No. S-5:	Slope Stability for Rock Protection (river water level at Elevation 262.7 m) at Station 0+060 on East River Bank
	Notice to Contractor - Obstructions

Table A – Foundation Alternatives for New Bridge Abutments

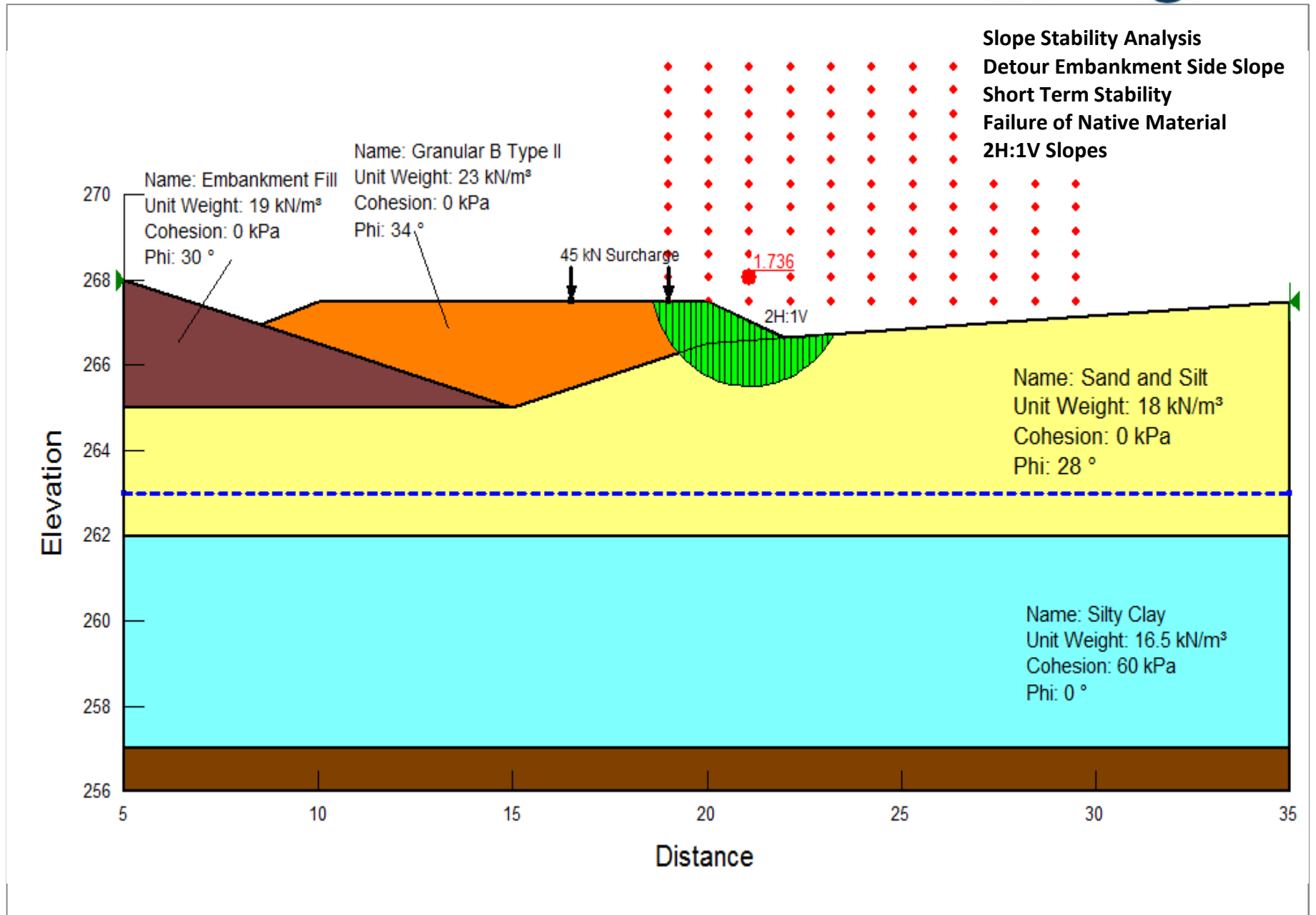
Foundation Type	Founding Level (m)	Advantages	Disadvantages	Remark	Potential Risks/ Relative Costs
Spread footings supported on bedrock	<p>-To depths ranging from 8 m to 9.5 m below ground surface for the west abutment;</p> <p>-To depths ranging from 10 m to 11.5 m for the east abutment</p>	<p>- Relative simple construction after temporary shoring and dewatering systems completed</p> <p>- Negligible settlement</p> <p>- Compatible with existing bridge abutment foundations</p>	<p>-Temporary shoring and dewatering systems required</p> <p>-Time consuming to demolish existing</p> <p>- Trimming bedrock surface and anchorage may be required due to varying bedrock levels</p>	<p>-Feasible, not recommended due to high cost</p> <p>-Additional stability analyses for river protection are not required</p>	<p>- High risk option for difficulty in dewatering system</p> <p>- Possible additional subexcavation needed, due to fractured bedrock near surface</p> <p>- Differential settlement between abutment and embankment fill due to improper backfilling behind new abutment walls</p> <p>- Probably most expensive</p>
Steel H-piles socketed 1 m minimum into bedrock	<p>To depths ranging from 9 m to 10.5 m below ground surface for the west abutment;</p> <p>To depths ranging from 11 m to 12.5 m for the east abutment</p>	<p>-High geotechnical resistance</p> <p>- Negligible settlement</p> <p>- Allow for fixing pile end into bedrock for required lateral resistance</p> <p>-Predrilled steel H piles induced less vibration</p>	<p>-Space behind existing bridge abutment walls/footings is limited for new pile construction</p> <p>-Difficult to ensure clean contact at the pile base for geotechnical resistance</p> <p>-Specialist contractor required</p>	<p>Recommended , but moderate risk</p>	<p>- Moderate risk of encountering existing substructure due to limited space for new pile construction</p> <p>- Moderate expensive</p>
Cast-in-place caissons socketed 1 m minimum into bedrock	<p>To depths ranging from 9 m to 10.5 m below ground surface for the west abutment;</p> <p>To depths ranging from 11 m to 12.5 m for the east abutment</p>	<p>- High geotechnical resistance</p> <p>- Negligible settlement</p> <p>- Allow for fixing pile end into bedrock for required lateral resistance</p> <p>-Bored the piles to induce less vibration</p>	<p>-Space behind existing bridge abutment walls/footings is limited for new pile construction</p> <p>-Difficul to ensure clean contact at the pile base</p> <p>-Specialist contractor required</p>	<p>Considered as alternative due to higher cost</p>	<p>- Moderate risk of encountering existing substructure due to limited space for new pile construction</p> <p>- Probably more expensive than Steel H-piles socketed into bedrock</p>

**Table A – Foundation Alternatives for New Bridge Abutments
(continued)**

Foundation Type	Founding Level (m)	Advantages	Disadvantages	Remark	Potential Risks/ Relative Costs
Drilled micropiles socketed into bedrock for required factored geotechnical resistance	<p>300 mm diameter micropiles to depths ranging from 10.5 m to 12 m below ground surface to achieve 850 kN geotechnical resistance for the west abutment;</p> <p>300 mm diameter micropiles to depths ranging from 12.5 m to 14.0 m below ground surface to achieve 850 kN geotechnical resistance for the east abutment</p>	<p>-High geotechnical resistance</p> <p>- Allow to penetrate through rock slab and/or abutment footing, if encountered</p> <p>-Negligible settlement</p> <p>- Allow for fixing pile end into bedrock for required resistance</p> <p>-Smaller drilling equipment and less induced vibration</p>	<p>--Drilling into required depths into bedrock required at pile location</p> <p>--Specialist contractor required</p>	Considered as alternative due to higher cost	<p>- Low risk Option</p> <p>- Probably more expensive than Steel H-piles socketed into bedrock</p>

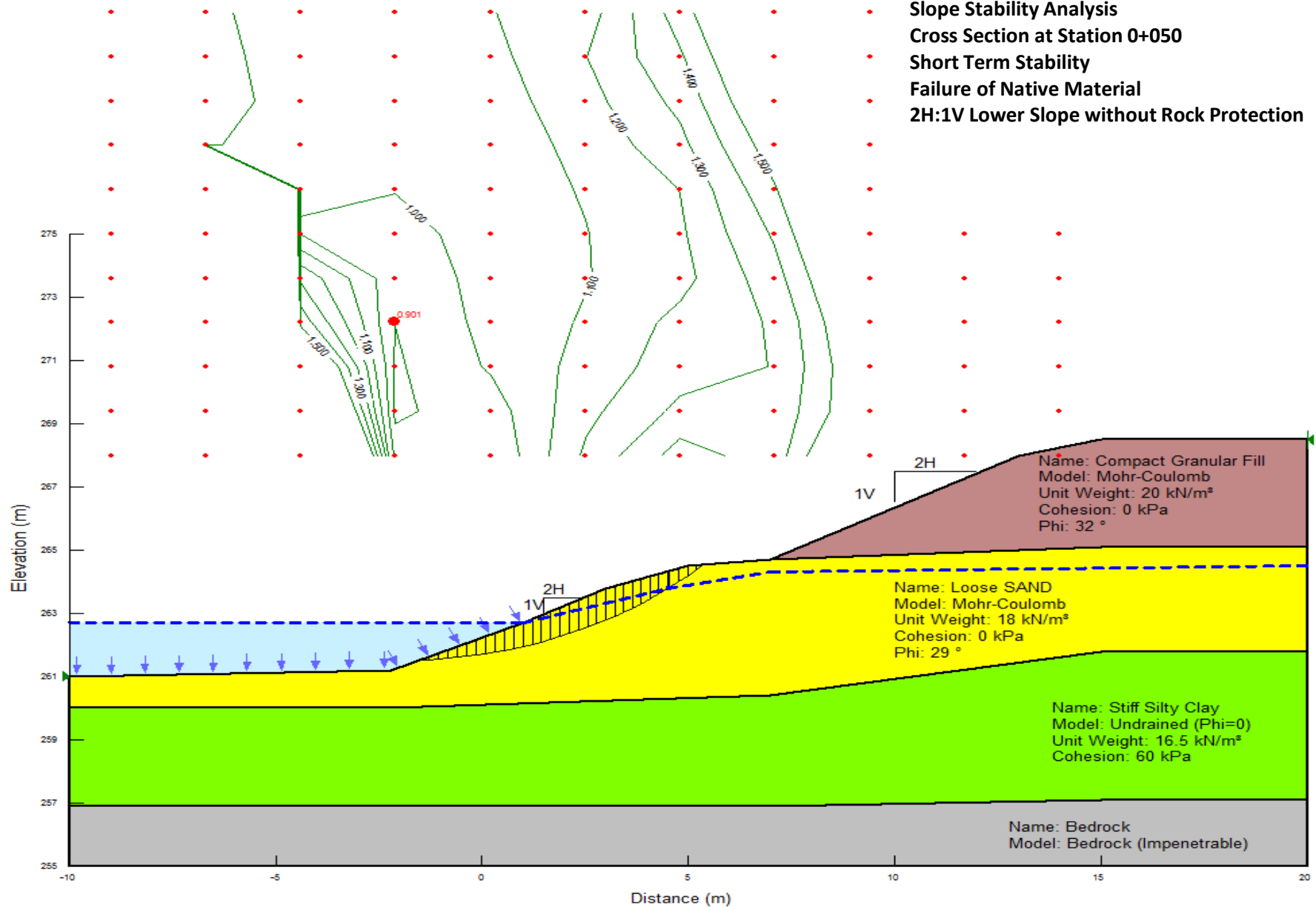
Table B – Foundation Alternatives for Detour Bridge

Foundation Type	Founding Level (m)	Advantages	Disadvantages	Note	Potential Risks/ Relative Costs
Spread footings seated on Granular B Type II engineered fill	2.1 m below grade	- Relative simple construction	<ul style="list-style-type: none"> - Imported embankment fill and OPSS engineered fill required - Relative low geotechnical resistances at ULS and SLS - Some differential settlement anticipated between abutments and approach embankments 	<ul style="list-style-type: none"> -Additional slope stability analyses for rock protections of embankments and river banks are required -Feasible, but additional cost and longer schedule for additional rock protections on embankment and river banks, if required 	<ul style="list-style-type: none"> - High risk of differential settlement between abutments and approach embankments due to surcharge and/or improper backfilling for detour embankments - Probably more expensive and longer schedule
Steel H-piles with 600 mm diameter socket 1 m minimum into bedrock	Founding levels not greater than elevation 255.9 m at the east approach and elevation 256.1 at the west approach	<ul style="list-style-type: none"> -High geotechnical resistances seated on bedrock - Negligible settlement - Additional slope stability analyses on the river banks are not required 	<ul style="list-style-type: none"> -Vibration impact on the river banks need be checked -Specialist contractor required 	<ul style="list-style-type: none"> -Feasible, but moderate risk -Additional stability analyses for rock protection of embankments are not required 	<ul style="list-style-type: none"> - Moderate risk of encountering existing substructure for new pile construction - Moderate expensive



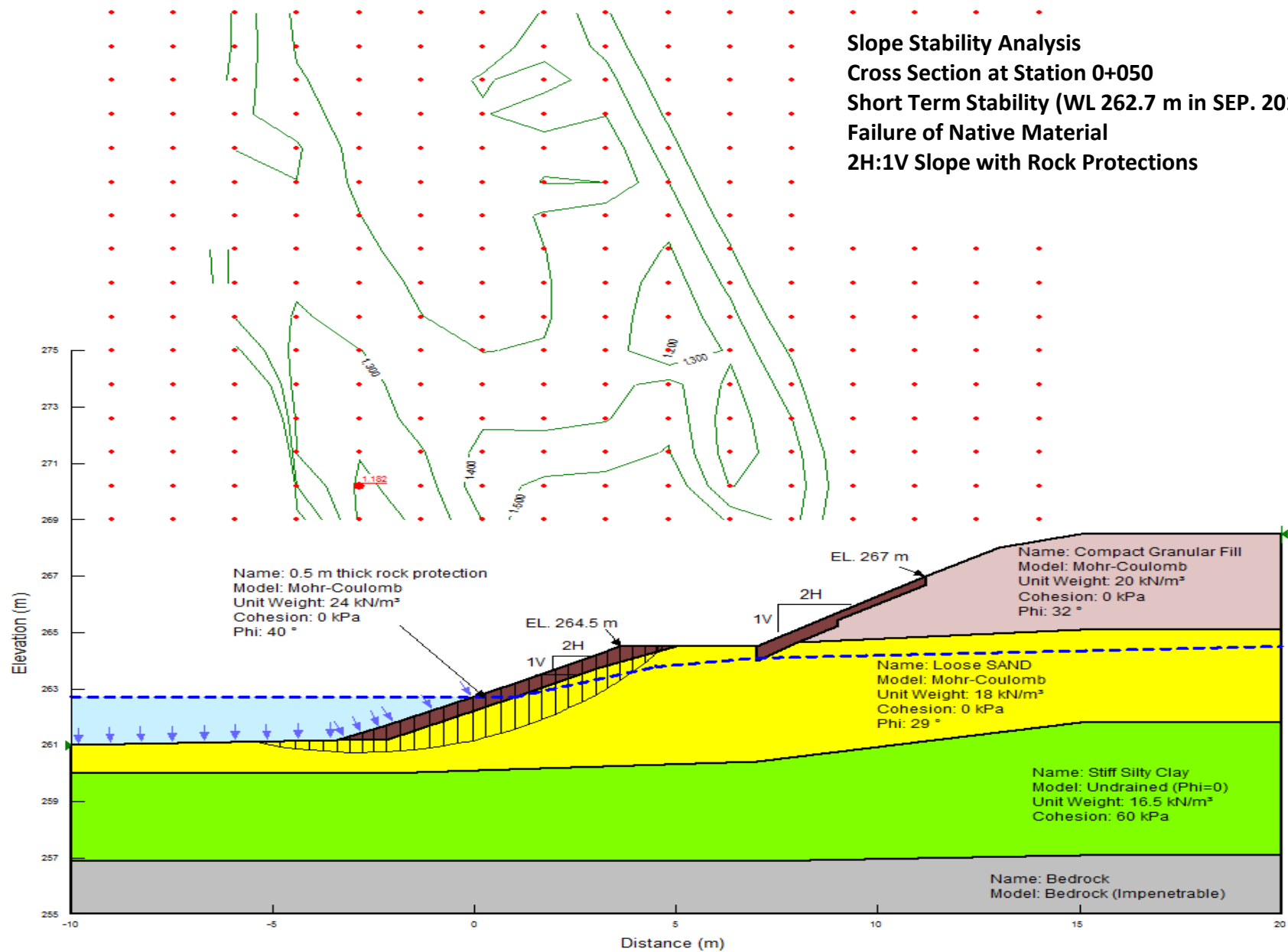
Stability Analysis
 Detour Embankment
 Whitson River Bridge

Slope Stability Analysis
Cross Section at Station 0+050
Short Term Stability
Failure of Native Material
2H:1V Lower Slope without Rock Protection



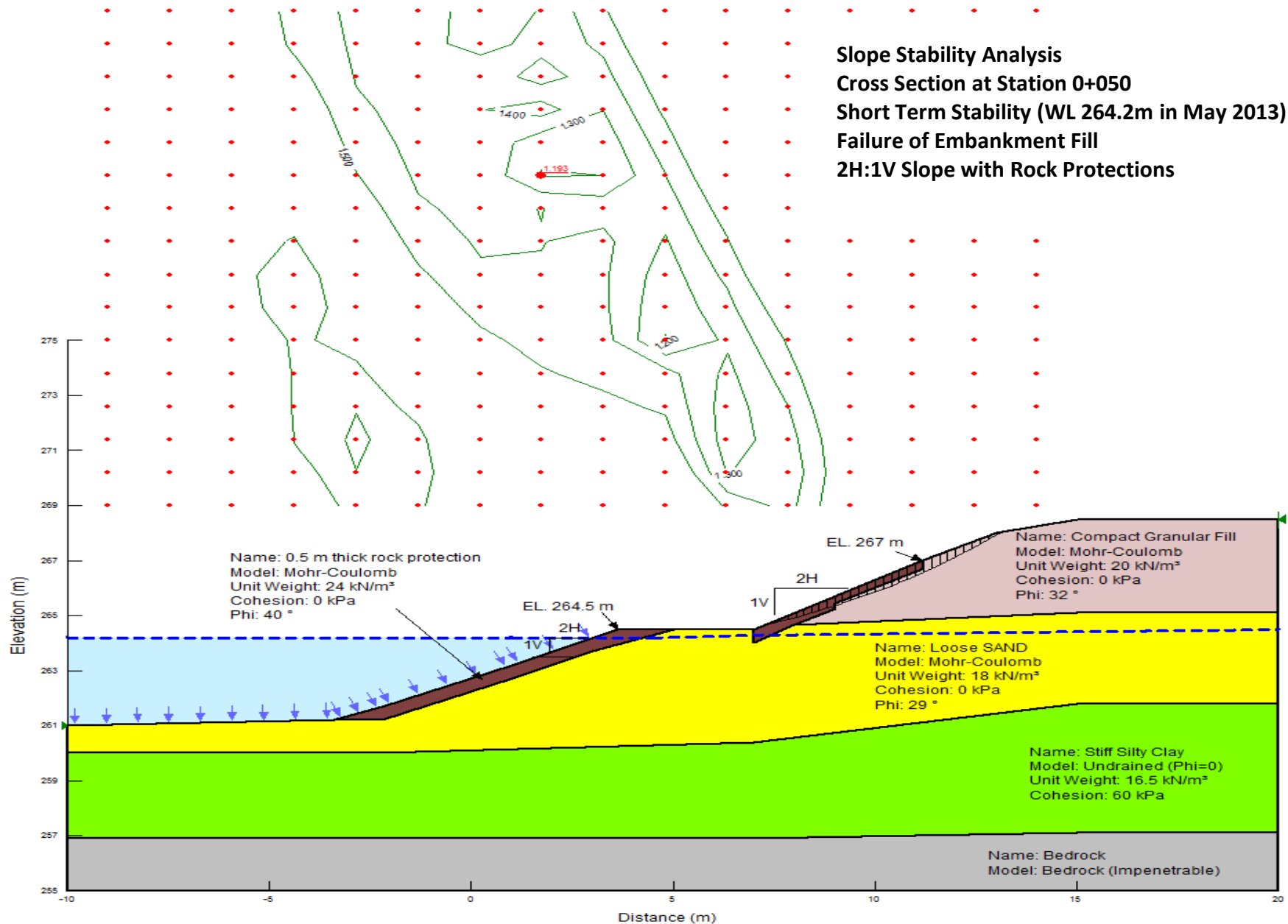
Stability Analysis
Existing East River Bank
Whitson River Bridge

Slope Stability Analysis
Cross Section at Station 0+050
Short Term Stability (WL 262.7 m in SEP. 2014)
Failure of Native Material
2H:1V Slope with Rock Protections



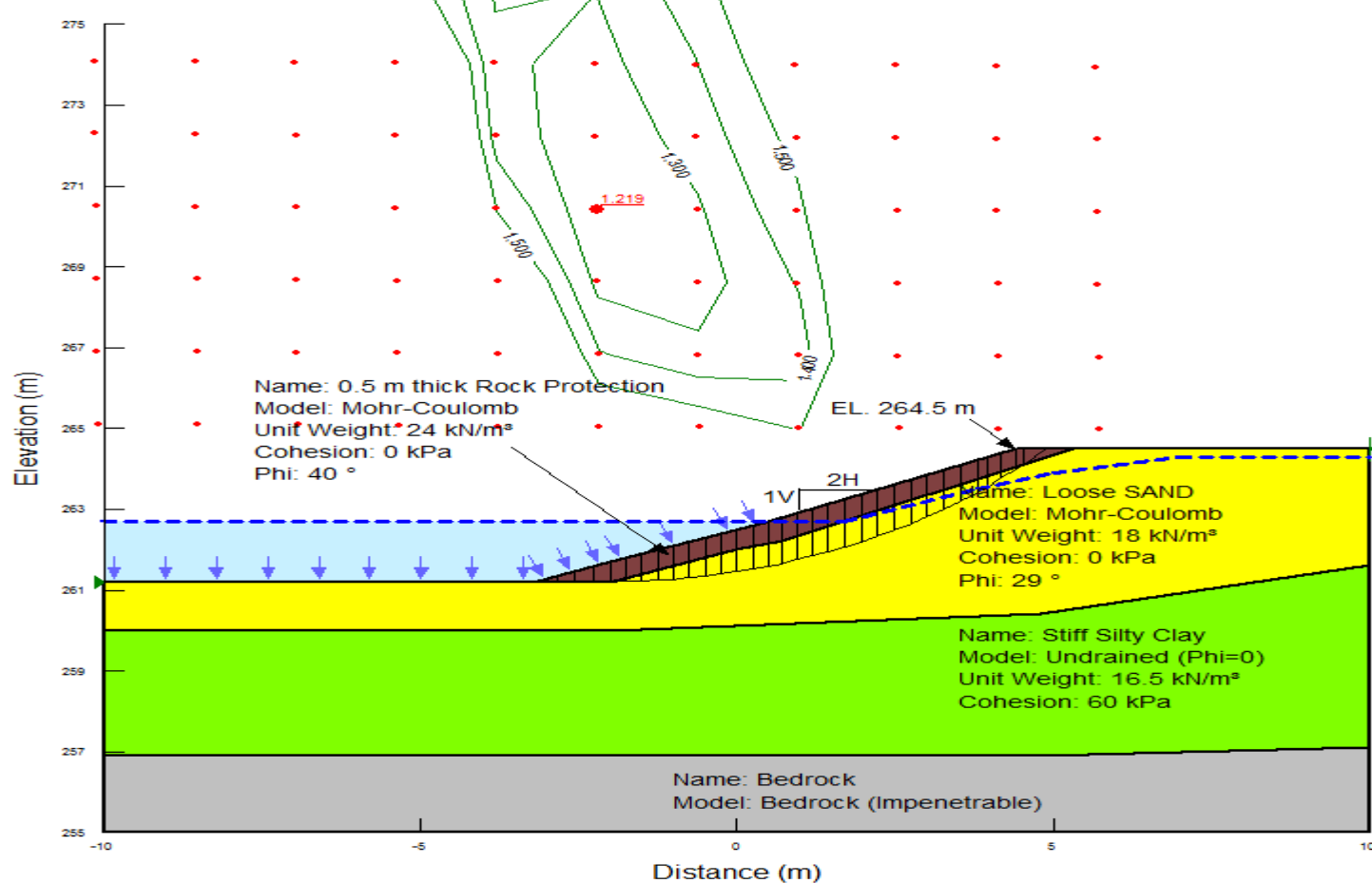
Stability Analysis
East River Bank with Rock Protection
Whitson River Bridge

Slope Stability Analysis
Cross Section at Station 0+050
Short Term Stability (WL 264.2m in May 2013)
Failure of Embankment Fill
2H:1V Slope with Rock Protections



Stability Analysis
East River Bank with Rock Protection
Whitson River Bridge

Stability Analysis
Cross Section at Station 0+060
Short Term Stability (WL 262.7 m in SEP.2014)
Failure of Native Material
2H:1V Slope with Rock Protection



Stability Analysis
East River Bank with Rock Protection
Whitson River Bridge

NOTICE TO CONTRACTOR – Obstructions in Native Soils and Rock Protection on River Banks

Special Provision

The Contractor is notified that, during foundation field investigations at the location of the Whitson River Bridge on Highway 144, cobble/boulder sized rock pieces were encountered in the embankment fill. The contractor shall take into account these materials when designing and installing abutment foundations.

In addition, the rock protection on both river banks is designed without considering the additional surcharge loads induced by the temporary detour bridge construction. The Contractor should carry out his own slope stability analyses and checking based on the structure type he chooses for the proposed detour bridge, addressed rock protection, and bank stabilization, dependent upon his chosen type of detour structure.



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