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**GEOCRES NO. 31E-379  
ASSIGNMENT 5015-E-0045 – WORK ORDER 1  
PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN  
HIGHWAY 141 SHADOW RIVER BRIDGE, SITE NO. 44-159  
ROSSEAU, ONTARIO**

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## **1.0 INTRODUCTION**

Golder Associates Ltd. (Golder) has been retained by AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation, Ontario (MTO) as part of the retainer Assignment 5015-E-0045 to provide preliminary and detail foundation engineering services for the replacement of the Shadow River Bridge (Site No.44-159), located on Highway 141 northwest of Rosseau, Ontario in the Township of Humphrey. Foundation engineering services for this assignment are required under two phases:

- **Phase 1 – Preliminary/Feasibility Investigation and Design Memorandum:** The purpose of the preliminary investigation is to establish the subsurface conditions at the locations of the proposed detour structure to the north of the existing structure and at the existing/replacement bridge alignment, and to provide preliminary recommendations for the permanent and temporary modular bridges along with the feasibility of the construction staging options, including preliminary stability and settlement analysis.
- **Phase 2 – Detail Investigation and Design:** Once the preferred replacement method is chosen, a detailed foundation investigation and design is to be completed.

This memorandum addresses Phase 1 of the project for the Preliminary/Feasibility Foundation Investigation and Design of the project.

As part of the Terms of Reference (TOR), MTO provided Golder with the approximate temporary modular bridge location that was developed by Morrison Hershfield (MH) under a separate assignment. The approximate locations of the existing Shadow River Bridge and Temporary Modular Bridge (TMB) are shown on Drawing 1.

## **2.0 SITE DESCRIPTION AND BACKGROUND INFORMATION**

The Shadow River Bridge site is situated in the Township of Humphrey on Highway 141, approximately 3.1 km northwest of the junction of Highway 141 and Highway 632 in Rosseau, Ontario. The bridge site is located in a low-lying valley/swampy area between bedrock outcrops that are located approximately 150 m east and west of the site. The low-lying swampy area surrounding the bridge is vegetated with grasses and small shrubs. The river flows in a northerly direction and is about 8 m wide at the existing bridge location.



The existing structure is a two-lane, eight-span, timber bridge with a stressed laminated timber deck which is asphalt-surfaced and was constructed in 1975. The structure is founded on timber crib abutments and the abutments and piers are supported on timber piles founded at unknown depths. The existing bridge is supported by eight timber pile bents and is in extremely poor structural condition. The existing grade of the highway at the bridge is at about Elevation 229.2 m. The ice/water level in Shadow River was measured at Elevation 226.4 m on March 10, 2017.

A previous Foundation Investigation and Design Report (GEOCRE 31E00-020, dated June 26, 1986 by the Department of Highways Ontario – Foundations Section) completed for the site indicates the native materials at are comprised of soft to firm varved silty clay up to about 18 m thick, underlain by a 3 m to 6 m thick deposit of generally very loose to compact silt to sand, in turn underlain by granite/gneiss bedrock. Further, the previous drawings indicate that the river was realigned to its current location from its natural meander to accommodate the bridge. At the current bridge location the previous river channel meander ran parallel to Highway 141, north of the existing Highway prior to the realignment.

### **3.0 INVESTIGATION PROCEDURES**

The field work for this subsurface investigation was carried out between March 13 and 24, 2017, at which time eight boreholes (Boreholes S-1 to S-4 and ST-1 to ST-4) were advanced using CME-55 and CME-850 track-mounted drill rigs supplied and operated by Landcore Drilling Inc. of Sudbury, Ontario. Boreholes S-1 to S-4 were advanced adjacent to the existing structure for a proposed three-span replacement option. Boreholes ST-1 to ST-4 were advanced to the north of the existing structure for assessing the feasibility of a temporary modular bridge detour structure.

The boreholes were advanced using 108 mm inner diameter hollow-stem augers, as well as NW/HQ casing and NQ-size core barrel. Soil samples were obtained at intervals of depth of about 0.75 m to 1.5 m, using a 50 mm outer diameter split-spoon sampler operated by an automatic hammer on the drill rig, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Field vane shear tests were carried out in the cohesive soils for assessment of undrained shear strengths in accordance with ASTM D2573 (Standard Test Method for Field Vane Shear Test in Saturated Fine Grained Soils) using 'N' size vanes. A vane collar was used when necessary. Select samples of the cohesive soils were obtained using 76 mm outer diameter thin-walled Shelby Tubes (ASTM D1587) for relatively undisturbed samples. The measured in situ field results (i.e., SPT 'N'-values and field vane shear tests) presented in Section 4.0 report are uncorrected.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations and two standpipe piezometers were installed in Boreholes ST-2 and S-3 to permit monitoring of the groundwater level. The piezometers consist of a 38 mm (1.5 inch) diameter polyvinyl chloride (PVC) pipe, with a slotted screen sealed within a sand filter pack at a selected depth interval within the boreholes. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was partially backfilled with bentonite pellets and/or bentonite grout to ground surface. The piezometer installation details and water level readings are indicated on the borehole records contained in Appendix A. All other boreholes were backfilled upon completion in accordance with Ontario Regulation 903 (Wells, as amended).

The field work was supervised on a full-time basis by a member of Golder's staff, who located the boreholes in the field, directed the drilling and sampling and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's Sudbury Laboratory for further examination and laboratory testing. Index and classification tests consisting of water content, Atterberg limits and grain size distribution were carried out on selected soil samples. In addition two one-dimensional consolidation (oedometer)



tests were carried out on selected soil samples. Unconfined compression strength (UCS) tests were carried out on selected bedrock core samples. The geotechnical laboratory testing was completed according to applicable ASTM and MTO LS standards, as applicable.

The approximate locations of the Phase 1 boreholes were identified by MTO/AECOM prior to drilling. The boreholes were surveyed using a Trimble Geo7 GPS survey unit. The locations of the boreholes are shown on Drawing 1. A summary of the borehole locations (northing and easting coordinates given relative to NAD83 MTM Zone 10, as well as latitude and longitude) and geodetic elevations are provided on the borehole records and summarized below.

Borehole	Northing	Easting	Latitude	Longitude	Elevation	Location
S-1	5014721.8	291120.5	45.27232055	-79.67413458	229.2	West Abutment
S-2	5014732.3	291136.6	45.27241504	-79.67413487	227.2	West Pier
S-3	5014732.7	291152.6	45.27241895	-79.67393096	226.9	East Pier
S-4	5014722.4	291173.5	45.27232667	-79.67366432	229.1	East Abutment
ST-1	5014739.8	291120.2	45.27248221	-79.67434408	227.1	North Detour – West Abutment
ST-2	5014739.3	291130.4	45.27247791	-79.67421408	226.7	North Detour – West Pier
ST-3	5014739.2	291154.8	45.27247748	-79.67390311	226.8	North Detour – East Pier
ST-4	5014739.6	291164.5	45.27248127	-79.67377949	226.9	North Detour – East Abutment

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Site Geology

Published literature indicates that the site is located in the Rosseau Domain of the Algonquin Terrane, which is located in the Grenville Province (*Geology of Ontario*; OGS Special Volume 4). The bedrock of this domain consists of gneiss to granitic gneiss with localized areas of metagabbro.

Based on terrain mapping by the Ontario Geological Survey, the subsurface soils in the vicinity of the site consist of organic terrain and glaciolacustrine deposits consisting of sand, gravel, silt and clay. The site is bordered to the east and west by bedrock outcrops.

### 4.2 Subsurface Conditions

In general the subsurface conditions encountered in the boreholes consist of embankment fill and/or topsoil, underlain by an extensive deposit of clayey silt to silt, underlain by a non-cohesive deposit that varies in composition from silt to sand, in turn underlain by granitic gneiss bedrock. The borehole and drillhole records are provided in Appendix A; detailed geotechnical laboratory testing figures and rock core photographs will be included in the Foundation Investigation Report in Phase 2.

#### 4.2.1 Asphalt

Boreholes S-1 and S-4 were advanced through Highway 141 asphalt near the existing bridge abutments and the thickness of the asphalt was 90 mm and 130 mm respectively.



#### **4.2.2 Topsoil**

Boreholes S-2, S-3, and ST-1 to ST-4 encountered a 0.1 m to 1.1 m thick layer of dark brown sandy/silty topsoil, trace to some gravel, trace to some clay immediately below the ground surface, with its surface ranging from Elevation 227.2 m to 226.7 m. Immediately underlying the topsoil in Borehole ST-3, a 300 mm boulder was encountered, requiring NQ-coring techniques to advance the borehole. The natural water content measured on a sample of the topsoil deposit was 25 percent.

#### **4.2.3 Fill**

Fill materials were encountered in Boreholes S-1 and S-4 underlying the asphalt, and in Borehole ST-2 underlying the topsoil. The fill materials vary in composition from moist, brown silty sand to gravelly sand. In general, the surface of the fill was encountered between Elevation 229.1 m and 226.4 m, and it ranges from 1.7 m to 4.0 m in thickness. In Borehole S-4, a 150 mm thick layer of concrete was encountered at Elevation 228.6 or 0.5 m depth. Directly underlying the concrete, an approximately 1.1 m thick layer of white Styrofoam (lightweight fill) was encountered within the fill. Cobbles up to 75 mm were also encountered within the fill deposit, one such zone required NQ-coring techniques to advance the borehole.

The SPT “N”-values within the fill materials range from 3 to 69 blows per 0.3 m of penetration, indicating a variable, very loose to very dense relative density. In general, the embankment fill in Boreholes S-1 and S-4 has a compact to dense relative density, while the fill in Borehole ST-2 is very loose to loose. One split-spoon sample did not penetrate the full sample depth likely as a result of the presence of gravel, cobbles or rock fill fragments within the fill, as noted on the borehole records. The natural water content measured on samples of the fill range from about 3 to 13 percent.

### **4.3 Clayey Silt to Clay**

A deposit of moist to wet, brown to grey clayey silt to clay was encountered below the embankment fill or topsoil in all boreholes. The deposit is between 10.6 m and 16.7 m thick as encountered in the boreholes, and the surface of this deposit was encountered between Elevation 226.8 m and 224.5 m. The upper and lower portions of the deposit consists in some instances of clayey silt containing trace to some sand, while the majority of the deposit generally consists of silty clay to clay, containing sand. Varves were noted in the deposit in all boreholes but not consistently throughout depth.

The measured SPT “N”-values within the clayey silt to clay deposit range from 0 blows (weight of rods/hammer) to 7 blows per 0.3 m of penetration. In situ field vane tests carried out within this portion of the deposit measured undrained shear strengths ranging from about 8 kPa to 67 kPa. The field vane test results together with the SPT “N” values indicate that the silty clay to clay has a very soft to stiff consistency; however generally the cohesive deposit has a soft to firm consistency.

Atterberg limits tests on selected samples of this deposit yielded liquid limits ranging between about 21 and 69 percent, plastic limits between about 15 and 26 percent and plasticity indices between about 6 and 43 percent, indicating the material is classified as a clayey silt of low plasticity to a clay of high plasticity. The natural moisture content measured on samples of the cohesive deposit ranges from 34 to 96 percent.

Laboratory consolidation (oedometer) tests were carried out on two specimens of the clay, obtained from Shelby tube samples in Boreholes S-1 and ST-4. The preconsolidation stress was estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. A bulk unit weight of 15.6 kN/m<sup>3</sup> and 15.9 kN/m<sup>3</sup> and a specific gravity of 2.78 were measured on the consolidation test samples. The test results are summarized below.



Borehole/ Sample No.	Sample Depth / Elevation	$\sigma_{vo}'$ (kPa)	$\sigma_p'$ (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	$e_o$	$C_c$	$C_r$	$c_v^*$ (cm <sup>2</sup> /s)
S-1/Sample 10	9.3 m/ 219.9 m	95	55	-40	0.6	1.9	0.9	0.09	$4.0 \times 10^{-4}$
ST-4/Sample 4	3.2 m/ 223.7 m	45	55	10	1.2	2.0	1.0	0.1	$1.1 \times 10^{-3}$

\*For the normally consolidated stress range

where:  $\sigma_{vo}'$  is the effective overburden stress in kPa  
 $\sigma_p'$  is the preconsolidation stress in kPa  
OCR is the overconsolidation ratio  
 $e_o$  is the initial void ratio  
 $C_c$  is the compression index  
 $C_r$  is the recompression index  
 $c_v$  is the coefficient of consolidation in cm<sup>2</sup>/s

It should be noted that upon review of the consolidation test results from Borehole S-1 Sample 10, it appears that the sample may be disturbed, and the results should be used with caution. Additional consolidation tests will be completed in the next phase of work.

A summary of the water contents and Atterberg limits versus elevation is plotted on Figure 1, attached. The consolidation test results are shown on Figures 2 and 3 for Boreholes S-1 and ST-4, respectively.

#### 4.4 Sandy Silt to Silt

A deposit of non-cohesive sandy silt to silt was encountered below the clayey silt to clay deposit in Boreholes S-2 to S-4, ST-1 and ST-2. The deposit is between 1.4 m and 5.9 m thick as encountered in the boreholes, and the surface of this deposit was encountered between Elevation 213.7 m and 211.7 m.

The SPT "N"-values measured within the sandy silt to silt deposit range from 1 to 13 blows per 0.3 m of penetration, indicating a very loose to compact relative density. Atterberg limits tests on selected samples within the deposit yielded non-plastic results. The natural water contents measured on samples of the sandy silt to silt deposit range from about 14 to 27 percent.

#### 4.5 Sand to Silt and Sand

A deposit of non-cohesive sand to silt and sand, trace to some gravel, trace clay was encountered below the clayey silt to clay deposit and/or the sandy silt to silt deposit in all boreholes except Borehole S-2. The deposit is between 2.3 m and 5.5 m thick in and the surface of this deposit was encountered between Elevation 215.2 m and 206.5 m.

The SPT "N"-values measured within the sand to silt and sand deposit range from 1 to 38 blows per 0.3 m of penetration, indicating a very loose to dense relative density. The natural water contents measured on samples of the sand to silt and sand deposit range from about 10 to 29 percent.

#### 4.6 Bedrock/Refusal

Bedrock was cored in Boreholes S-2, S-3 and ST-1 to ST-4 and refusal to further casing penetration and split-spoon penetration was encountered in Boreholes S-1 and S-4. The depth to the confirmed/inferred bedrock surface and bedrock surface elevations are presented below.



Borehole No.	Depth to Bedrock (below ground surface) (m)	Bedrock Surface Elevation (m)	Core Length (m)
S-1	17.2	212.0	Casing and split-spoon refusal
S-2	17.7	209.5	3.1
S-3	20.6	206.3	3.1
S-4	24.9	204.2	Casing and split-spoon refusal
ST-1	17.4	209.7	3.0
ST-2	17.5	209.2	3.4
ST-3	20.1	206.7	3.3
ST-4	21.5	205.4	2.7

The retrieved bedrock core is described as fine grained, black/pink, foliated, fresh to slightly weathered, strong to very strong granitic gneiss as presented in the drillhole records. A description of the bedrock properties encountered in the boreholes is provided below.

Borehole No.	Total Core Recovery	Rock Quality Designation	Quality Classification Table 3.10 of CFEM 2006 <sup>1</sup>	Uniaxial Compressive Strength (MPa)	Strength Classification Table 3.5 of CFEM 2006 <sup>1</sup>
S-2	93 – 100 %	85 – 100 %	Good to Excellent	99	Strong (R4)
S-3	100 %	45 – 100 %	Poor to Excellent	159	Very Strong (R5)
ST-1	100 %	83 – 94 %	Good to Excellent	160	Very Strong (R5)
ST-2	94 – 100 %	44 – 100 %	Poor to Excellent	89	Strong (R4)
ST-3	90 – 100 %	83 – 87 %	Good	132	Very Strong (R5)
ST-4	100 %	0 – 40 %*	Very Poor to Poor	139	Very Strong (R5)

\*Broken core sections may have been as a result of drilling and may not be representative of the overall rock quality.

## 4.7 Groundwater Conditions

Groundwater levels were measured in the open boreholes during and upon completion of overburden drilling, prior to commencement of rock coring operations. Two piezometers were installed to monitor the groundwater level: Borehole ST-2 sealed within the clayey silt to clay deposit, and Borehole S-3 sealed in the sand deposit below the clayey silt to clay deposit. Groundwater levels encountered in the boreholes during and shortly after drilling are not considered to be representative of stabilized groundwater levels as wash boring was used to advance the casing/coring equipment and the results are noted on the Record of Borehole. The measured static groundwater levels in the piezometer are presented below.

Borehole No.	Installation	Time and/or Date	Groundwater Depth (m)	Groundwater Elevation (m)
S-3	Piezometer	April 16, 2017	-0.4*	227.3
ST-2	Piezometer	April 16, 2017	0.4	226.3

\*Note: Artesian conditions were noted in within the piezometer in Borehole S-3.

<sup>1</sup> Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4<sup>th</sup> Edition.



Groundwater and river water levels in the area are subject to seasonal fluctuations and to fluctuations after precipitation events and snowmelt. The river ice level / water level of Shadow River was measured at Elevation 226.4 m on March 10, 2017. The water level of Shadow River from GEOCRE 31E00-020 measured in December 1967 was noted to be approximately 226.7 m (743.66 ft).

## 5.0 FOUNDATION ENGINEERING DISCUSSION

### 5.1 Replacement Alternatives

The replacement strategy for the Shadow River Bridge considers three options, as provided by the structural designer, as follows:

- **Option 1:** Constructing a new bridge on the existing alignment using half-and-half staging, without or with an up to 1.3 m grade raise.
- **Option 2:** Constructing a new bridge on the existing alignment, while traffic is diverted onto a single-lane detour with a temporary modular bridge (TMB) located immediately to the north of the existing embankment (essentially a “widening” of the existing embankment), without or with an up to 1.3 m grade raise.
- **Option 3:** Constructing a new bridge on a new alignment shifted partially to the north, and diverting traffic to a single lane on the new bridge while the remainder of the bridge is constructed, without or with an up to 1.3 m grade raise.

A summary of the advantages, disadvantages, relative costs and risks/consequences associated with each bridge replacement option from a foundations perspective is provided in Table 1 following the text of this memo. From a foundations perspective, the preferred option is replacing the bridge on the current alignment using half-and-half staged construction to minimize the risk associated with instability and settlement of existing/new/widened/detour embankments, and reduce the impact on existing bridge piles (i.e. downdrag loading and potential disturbance) while still in operation. However, we understand that there are structural considerations related to safety and traffic that may make half-and-half construction unfeasible. If the half-and-half option is not structurally feasible, the option that has the least amount of filling or alteration to the existing embankment is preferred over other options, to limit post-construction settlement and potential embankment instability as well as reduce overall foundation-related costs.

### 5.2 Foundation Options for Existing Structure and Detour (TMB) Structure

Based on the proposed bridge geometry and the subsurface conditions at this site, deep foundations will be required for support of the abutments of the replacement structure and for the TMB structure (if required). Shallow strip/spread footings are not considered feasible for support of the new abutments and/or the temporary detour structure given the low geotechnical resistance(s) and large settlements associated with the very soft to firm consistency of the cohesive deposit.

A summary of the advantages and disadvantages associated with each deep foundation option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, relative costs and risks/consequences is provided in Table 2 following the text of this memo. For any deep foundation option, the proximity of the new piles to the existing bridge piles will have to be considered. Because the existing piles are founded at unknown depth, installation of new foundation elements may impact the existing piles due to vibrations and/or loss of ground associated with installation procedures.



- **Driven steel H-piles:** Driven steel H-piles are feasible for support of the abutments and associated wingwalls and would permit integral abutment design for the structure(s). The abutments would be supported on end-bearing piles driven to granitic gneiss bedrock. Mitigation measures (such as a granular filter blanket, and/or possibly grouting) would be required to minimize soil migration during pile driving as a result of high/ artesian groundwater pressures within the sandy silt stratum overlying the bedrock, and a minimum separation distance will be required to minimize risk to the existing structure. Excavation/dewatering would be required for pile cap construction. It is noted that the large construction equipment may not be easily supported on the soft underlying soils and may induce unbalanced loading on the existing bridge and any temporary excavations/works. Downdrag loads may need to be considered depending on the replacement staging strategy chosen.
- **Driven steel pipe piles:** End-bearing steel pipe piles driven to bedrock could also be considered as a deep foundation option for support of the abutments. Pipe piles would preclude the use of integral abutments due to the rigidity of such piles. In addition, the use of pipe piles may create greater risks with soil migration due to artesian pressures, depending on the installation method. Other advantages/disadvantages are similar to driven steel H-piles, but given the above noted disadvantages, pipe piles are less preferred than H-piles at this site.
- **Micropiles:** Micropiles socketted into the bedrock could be considered as a deep foundation option for support of the abutments. Micropiles have the advantage of requiring lighter weight equipment for installation, which may be advantageous at this site. Further, they could be used to aid in supporting/underpinning the existing bridge founded on timber piles (at unknown depth), depending on the replacement strategy chosen.
- **Socketted H-Piles:** H-piles are installed by advancing a temporary steel casing and excavating a socket into the bedrock in which a steel H-pile is installed. This option typically requires removal of the casing during concreting, which at this site, is not recommended due to the high risk of ground disturbance during removal and impact on the existing piles. Further, mitigation measures (such as a granular filter blanket, and possibly grouting) would still be required to minimize soil migration during casing and pile installation, and the minimum separation distance from the existing structure would be greater than for small diameter casings where the casing is left in place.
- **Drilled steel casings (small diameter):** Drilled steel casings involve installation of a 305 mm to 750 mm diameter permanent steel casing into the bedrock using wash boring methods, with the casing then filled with concrete. Small diameter casings have an advantage over driven piles in that there would be less disturbance/vibration during installation, resulting in a lower separation distance requirement. Mitigation measures (such as a granular filter blanket) would still be required to minimize soil migration during drilled steel casing installation. Small diameter casings are more advantageous when obstructions are present and when sloping bedrock is present although neither of these conditions are key factors in determining the foundation type at this site. There is typically a cost premium for this type of foundation.
- **Drilled shafts/caissons (large diameter):** Drilled shafts socketted into the bedrock are also considered to be feasible for a deep foundation option at this site. However, caissons are not commonly constructed in Northern Ontario due to constructability issues associated with large-diameter drill holes through wet subgrade soils, challenges associated with seating/sealing large diameter elements at the interface with the moderately sloping bedrock at this site, and the costs associated with creating a socket in the strong to very strong bedrock. Larger diameter units will also increase the risk associated with proximity of the existing piles and would require a larger offset distance from the existing structure/foundation elements.



The following sections provide preliminary recommendations for deep foundation options to support the proposed replacement and TMB structures. From a foundations perspective, the small diameter casings with less disturbance/vibration during installation are preferred while the existing bridge is still in place. Once the existing bridge is removed, steel H-piles are preferred and would permit integral abutment design. However, depending on the replacement strategy chosen, micropiles may be advantageous for half-and-half staged construction if existing bridge stabilization is required or if lighter construction equipment is an advantage.

### 5.2.1 Minimum Separation Distance Between Existing and New Foundations

As noted above, due to the risk of ground loss associated with artesian pressures below the cohesive deposit at this site during pile installation to bedrock, all deep foundation options will require a minimum separation distance to be maintained between the existing and new foundation elements for the proposed bridge and/or TMB, in order to minimize the influence of the pile installation on the performance of the existing pile foundations. In general, the minimum separation distance will increase with larger pile sizes; however, the type of pile (driven versus drilled) and the termination stratum will also affect the minimum separation distance required. A detailed assessment of each of the factors that could influence the effect of the new construction on the existing is beyond the scope of this phase of the assignment; however, a preliminary assessment has been carried out that has considered the following:

- **Effect of soil displacements due to pile driving.** Fellenius et al. (1982), Bozozuk et al. (1978), and Poulos (1994) suggest a minimum separation distance of 9 m to 12 m, and no less than 10 pile diameters is required to minimize soil movements (laterally away from and vertically upwards) due to displacement of the soil during installation of driven piles which could cause heave, tensile forces or bending in existing adjacent piles. These sources note that the effect increases with the number of piles in a group.
- **Effect of soil displacements due to hole drilling (for drilled shafts/caissons).** Poulos (2005) suggests that a minimum separation distance of 6 to 8 hole diameters is required to minimize soil movement (laterally towards and vertically downwards) due to excavation of the pile shaft which could cause ground loss, undermining, settlement and/or additional forces or bending in existing adjacent piles.
- **Effect of vibrations due to pile driving or vibratory liner installation.** Based on the work of Lacy and Gould (1985), Massarsch (2000) and Drabkin et al. (1996), and consideration of the subsurface conditions at the present site, in order to limit vibrations (or Peak Particle Velocity) to a level below which the risk of vibration-induced settlements might occur, it is estimated that a minimum separation distance of about 15 m may be required. As noted previously, it is recommended that measurements of vibrations/peak particle velocities be carried out at the site (as part of test pile installation program as well as during production piling) and that the existing bridge structure and foundations be monitored for vibrations and settlements during construction. Vibration-related offset distances for drilled shaft liner installation may also be mitigated by using non-vibratory casing installation methods (i.e., rotational or oscillatory techniques).
- **Conflicts between existing and new battered pile elements.** Based on the available as-built drawings for the existing bridge, the outer row of piles is battered. The structural engineer should confirm that the existing piles and proposed piles do not present conflict.

Based on consideration of each the above, it is suggested that there be a minimum separation distance between the existing and new structures/foundation elements of not less than 10 pile diameters for driven piles, or 6 hole diameters for drilled shafts/caissons. In addition, for driven piles, a minimum separation distance of 10 m should be implemented. It should be noted that there is still risk involved even after applying the minimum separation



distances and consideration should be given to moving foundation elements further away if possible. Once the existing bridge is no longer in use, the risk is lower for piles in place that are known to be installed to bedrock, depending on the bridge replacement option chosen.

### **5.2.2 Mitigation of Risks Associated with Deep Foundation Installation**

Artesian conditions were encountered in Borehole S3 overlying the bedrock surface and the water level was recorded at Elevation 227.3 m in the piezometer, corresponding to approximately 0.4 m above ground surface at the borehole location, or about 0.9 m above the river level. For any deep foundation type chosen, special measures to mitigate the risk of impact to the existing bridge piles (founded at unknown depth) will likely be required. These could consist of a combination of the following:

- A filter sand blanket constructed immediately below the pile caps to mitigate the loss of fine soil particles along with water flowing upward along the piles as a result of artesian groundwater conditions;
- Post-installation grouting alongside the piles to mitigate the potential for loss of fine soil particles;
- Vibration monitoring of existing structure; and/or
- Settlement monitoring of existing structure.

Further assessment of the artesian groundwater pressures will be made during the detailed design stage to refine these recommendations depending on the final bridge alternative and foundation options chosen.

### **5.3 Consequence and Site Understanding Classification**

As the proposed replacement Shadow River Bridge is located on Highway 141 and will carry large volumes of traffic with the potential to impact alternative transportation corridors, a “typical consequence level” is considered appropriate as outlined in Section 6.5 of the *Canadian Highway Bridge Design Code* (CHBDC 2014) and its *Commentary*. Further, given the scope of work of the foundation field investigation and laboratory testing program as outlined in Sections 3.0 and 4.0, a “typical degree of site and prediction model understanding” has been utilized at this stage of the design.

### **5.4 Seismic Site Classification**

For seismic design purposes, a site classification for seismic site response of E (for soft soil) should be used, based on Table 4.1 in Section 4.4.3 of the 2014 CHBDC.



## 5.5 Preliminary Deep Foundation Recommendations

### 5.5.1 Pile Tip Elevations

The abutments of the replacement structure or TMB may be founded using driven steel H-piles end bearing on the granitic gneiss bedrock. The following pile tip elevations can be used for preliminary purposes, based on the results from the preliminary investigation:

Alignment	Foundation Element	Estimated Design Driven Pile Tip Elevation / Top of Bedrock (m)
Existing Alignment	West Abutment (S-1)	212.0
	West Pier (S-2)	209.5
	East Pier (S-3)	206.3
	East Abutment (S-4)	204.2
North TMB Alignment	West Abutment / Pier (ST-1 & ST-2)	209.7 – 209.2
	East Abutment / Pier (ST-3 & ST-4)	206.7 – 205.4

If micropiles, drilled steel casings or drilled shafts are adopted, they would extend below the surface of the bedrock as identified in the table above; further recommendations on this aspect will be provided during the detail design stage.

### 5.5.2 Geotechnical Axial Resistances

A factored ultimate axial geotechnical resistance of 2,000 kN may be used for the design of steel HP 310x110 piles driven to the surface of the granitic gneiss bedrock. The factored serviceability axial geotechnical resistance (for 25 mm of settlement) will be greater than the factored ultimate axial geotechnical resistance. As the bedrock is considered to be an unyielding material, ULS conditions will govern for this foundation type. The preliminary geotechnical resistances may have to be re-evaluated and modified as necessary during detail design in consideration of the additional subsurface investigation at the foundation elements.

At this preliminary stage, for micropiles embedded into granitic gneiss bedrock, a typical bond length of 2 m for a 273 mm diameter micropile (with a 229 mm diameter rock socket) would yield a factored ultimate axial geotechnical resistance of about 2,000 kN per pile. The structural capacity, the bond length and the factored geotechnical resistance would have to be confirmed during detail design, if required depending on the selected replacement strategy. Additionally, if a larger capacity is required, consideration could be given to using a larger diameter drilled steel casing, and/or increasing the embedment depth within the bedrock.

It should be noted that obstructions (i.e. cobbles/boulders) were encountered within the existing fill and it may be necessary to excavate and replace the fill with OPSS.PROV 1010 Granular A or Granular B Type I, II or III (limiting the maximum particle size to less than 75 mm) at the abutment/pile locations prior to pile driving.

### 5.5.3 Downdrag Loads

The placement of new embankment fill for some of the proposed replacement strategies will induce consolidation settlement of the underlying soft to firm cohesive strata and as a result may (depending on construction sequence) cause downdrag loads on the new/existing piles. For any of the options including a grade raise and/or if the embankment is widened for the detour or alignment shift, downdrag loads on the existing/new piles will need to be considered/evaluated at detail design.



The estimated unfactored downdrag loads acting on a HP 310 x 110 pile, given the approximately 10 m to 16 m thick cohesive deposit, may be taken as 250 kN to 450 kN per pile. The estimated unfactored downdrag load acting on an existing 0.3 m diameter timber pile may be taken as 200 kN to 350 kN per pile; however, given that the tip elevation/length of these existing piles is unknown, the overall capacity of the existing piles is not known. These preliminary downdrag loads may have to be re-evaluated and modified during detail design. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.11.4.10 of the 2014 CHBDC and its Commentary for factored ultimate and serviceability conditions.

## 5.6 Preliminary Assessment of Approach Embankment Settlement and Stability

The existing embankments are approximately 2 m to 2.5 m high (compared to the existing ground surface at the toe of slope) and we understand that there has been on-going settlement of the approaches to this bridge over the years. This is corroborated by the existence of a 1.1 m thickness of expanded polystyrene (EPS) lightweight fill within the east approach embankment as encountered in Borehole S-4. Further, between 3.4 m and 4.1 m of embankment fill was encountered in Boreholes S-1 and S-4 (i.e. a fill thickness greater than the embankment height), suggesting that fill may have been added after the original construction and resulting settlement to maintain the current highway grade. From a foundations perspective, and as discussed in Section 5.1, it is recommended that grade raises be avoided as part of the new construction and that the new structure be replaced and staged on the existing alignment to avoid adding fill for widening to minimize post-construction settlement and embankment instability.

Preliminary settlement analyses have been carried out based on the simplified stratigraphy as encountered in the boreholes using Settle 3D (Version 3.020), produced by Rocscience Inc., to estimate the total factored settlement for the various options. For the purpose of the settlement analyses, to achieve factored settlements in accordance with Table 6.2 of CHBDC (2014), the inverse of the product of the consequence factor,  $\psi$ , and the geotechnical resistance factor  $\phi_{gs}$  (i.e. Factor =  $1/(\psi * \phi_{gs})$ ) was used.

Preliminary slope stability analyses have been carried out for the proposed embankment configuration(s) using the commercially available program GeoStudio 2007 (Version 7.23), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. The Factor of Safety (FoS) is defined as the ratio of forces tending to resist failure to the driving forces tending to cause the failure. For the purpose of the stability analysis, the FoS is equal to the inverse of the product of the consequence factor,  $\psi$ , and the geotechnical resistance factor  $\phi_{gu}$  (i.e.  $FoS = 1/(\psi * \phi_{gu})$ ). Accordingly, a target minimum FoS of 1.5 have been used for the preliminary design of the permanent embankment slopes, as per Table 6.2 of CHBDC (2014).

### 5.6.1 Parameter Selection

The simplified stratigraphy together with the associated strengths and unit weights employed for the different soil types at the approach embankments are summarized below.

Soil Type	$\gamma$ (kN/m <sup>3</sup> )	Settlement Parameters	Effective Friction Angle (°)	Undrained Shear Strength (kPa)
New Granular Fill	21	-	35	-
Existing Embankment Fill	20	-	33	-
Topsoil	12	-	27	1



Soil Type	$\gamma$ (kN/m <sup>3</sup> )	Settlement Parameters	Effective Friction Angle (°)	Undrained Shear Strength (kPa)
Clayey Silt to Clay (generally soft to firm)	15.8	$\sigma_p'$ = See Figure 5 OCR = 1 $e_o = 1.0 - 2.0$ $C_c = 0.5 - 0.9$ $C_r = 0.05 - 0.09$	-	See Figure 4
Sandy Silt to Silt (very loose to compact)	18	E = 5 MPa	28	-
Sand to Silt and Sand (loose to dense)	19	E = 20 MPa	29	-

The field vane shear tests shown on Figures 4 and 5 have been corrected to account for the plasticity index (Bjerrum) and varves noted in the cohesive deposit. It should be noted that further investigation and analysis will be required during detail design to check for the distribution and continuity of weak/very soft layers within the clayey silt to clay deposit, as such layers could control the global stability of the embankments.

## 5.6.2 Results of Analysis

The three replacement strategies being considered are those outlined in Section 5.1, with or without a 1.3 m grade raise. Lower grade raises will result in settlement values lower than those provided below; however, the time rate of settlement will remain the same. Further assessment will be provided during detail design when the detailed grade/geometry is confirmed.

The time rate of settlement for primary consolidation settlement for all options is based on an average coefficient of consolidation ( $c_v$ ) of about  $1.1 \times 10^{-3}$  cm<sup>2</sup>/sec estimated for the cohesive deposit. For the imposed loading conditions and assuming two-way drainage of the 10.6 m (Borehole S-1) to 12.6 m (Borehole S-4) thick cohesive deposit, it is estimated that approximately 90 percent of the primary consolidation settlement will be completed in about 14 years.

The approximate magnitude of secondary consolidation (creep) settlement, where applicable for the options below, is expected to range between about 50 mm to 100 mm per log cycle; however, this will be dependent on the mitigation option proposed and the values provided herein should be considered preliminary only.

### 5.6.2.1 Option 1A – Replace on Existing Alignment (No Grade Raise)

Given that the existing embankment has been in place (and hence the subsoils have been preloaded) since construction in 1975, it is anticipated that if there is no grade raise or widening, there will be no additional post-construction settlements due to primary consolidation of the cohesive soils. We understand that there has been historical embankment settlement within the vicinity of the bridge, likely due to the initial primary settlement as well as secondary compression of the cohesive soils. Consideration could be given to unloading the existing embankment by incorporating a layer, or additional layers, of EPS (similar to the current conditions at the east abutment) to further reduce the impact of creep settlement.

The results of the preliminary stability analysis indicate that the critical sections have a FoS approximately equal to the target 1.5 for granular embankments constructed with slopes of 2H:1V or flatter for both undrained (short-term) and drained (long-term) analysis, provided there is no grade raise or widening. Therefore, stability mitigation measures will not be required for this option. The results of the stability analysis is shown on Figure 6 for a section near the west abutment.



### 5.6.2.2 Option 1B – Replace on Existing Alignment (1.3 m Grade Raise)

Based on the results of the settlement analysis for an approximately 1.3 m grade raise (with associated widened platform) on the existing alignment, the total factored primary consolidation settlement of the foundation soils is estimated to range between about 300 mm and 400 mm (in about 14 years) plus creep. Since the total post construction settlement exceeds the settlement criteria for bridge approaches of 25 mm, settlement mitigation measures such as preloading, surcharging (toe berms may be required) or lightweight fill would have to be considered. Wick drains are not considered to be feasible given the existing embankment configuration. Other ground improvement options may also be considered (rigid inclusions, controlled modulus columns, soil mixing, etc.) but are also likely not feasible/practical due to the embankment configuration.

For an approximately 1.3 m grade raise (for 3.9 m of total fill on the existing embankment), the results of the stability analysis indicate that the critical sections have a FoS less than the target 1.5. Therefore, stability mitigation measures would be required for this option. At this preliminary stage, it is anticipated that toe berms approximately 2 m high by 20 m wide could be required at this site to achieve a FoS of 1.5. Alternatively, lightweight fill could be utilized to offload the embankment and reduce the size of/need for toe berms. Other ground improvement options may also be considered and their feasibility can be assessed at detail design if a grade raise must be implemented. The results of the stability analysis is shown on Figures 7 and 8 for a section near the west abutment.

### 5.6.2.3 Option 2A – Replace on Existing Alignment with TMB Detour (No Grade Raise)

Based on the results of the settlement analysis, for an approximately 2.5 m high detour embankment constructed on a new/adjacent (i.e. widened) alignment to the north, the total factored primary consolidation settlement of the foundation soils is estimated to range between about 800 mm and 1100 mm (in about 14 years) plus creep. Additionally, given the proximity of the detour embankment (i.e. essentially widening of the existing embankment) to the existing alignment, settlement of the north side of the existing embankment will also occur and will be differential across the existing/future embankment cross-section, ranging from approximately 100 mm to 150 mm at the existing centreline to 250 mm to 300 mm at the existing shoulder.

Given that the detour embankments will likely only be utilized for one or two construction seasons and given the relatively slow rate of consolidation of the cohesive soils ( $t_{90}$  of about 14 years), only a portion of the estimated settlements would occur during construction. It is estimated that the detour itself would experience the following factored settlements during construction:

Location	Total Estimated Factored Settlement (mm)	Estimated Factored Settlement to occur over 1-year period (mm)	Estimated Factored Settlement to occur over 2-year period (mm)
2.5 m widened detour embankment (at existing toe of slope)	800 – 1100	250 – 375	375 – 520
Existing centreline to shoulder	100 – 300	25 – 100	50 - 150

Because the total post-construction settlements exceed the settlement criteria for the TMB bridge approaches of 25 mm, settlement mitigation measures such as preloading, surcharging (toe berms may be required) or lightweight fill should be considered. Wick drains and other ground improvement options may also be considered but are likely not feasible/practical due to the embankment configuration.

Because the detour is not permanent, it may be possible to allow settlement to occur while traffic is in operation, depending on the driving surface required. Further, if the detour embankment grade could be lowered, and/or



surcharging to a maximum height of 2.5 m (stability mitigation not required) could be considered to reduce differential settlements during construction; however more detailed analysis would be required once the preferred staging has been determined.

For an approximately 2.5 m embankment constructed for the detour (similar to the existing embankment), the results of the stability analysis indicate that the critical sections have a FoS approximately equal to 1.5 for granular embankments constructed at slopes of 2H:1V or flatter. The results of the stability analysis is shown on Figure 9 for a section near the west abutment.

#### **5.6.2.4 Option 2B – Replace on Existing Alignment with TMB Detour (1.3 m Grade Raise)**

Based on the results of the settlement analysis carried out in Section 5.6.2.2, for an approximately 1.3 m grade raise on the existing alignment, the total factored settlement of the foundation soils is estimated to range between about 300 mm and 400 mm (in about 14 years) plus creep. Based on the results of the settlement analysis carried out in Section 5.6.2.3, for an approximately 2.5 m high detour embankment constructed on a new/widened alignment to the north, the total factored primary consolidation settlement of the detour foundation soils is estimated to range between about 800 mm and 1100 mm. Additionally given the proximity of the detour embankment (i.e. essentially widening of the existing embankment) to the existing alignment, settlement of the north side of the existing embankment will also occur and will be differential across the existing/future embankment cross section ranging from approximately 250 mm to 350 mm at the existing centreline to 350 mm to 450 mm at the existing shoulder.

Given that the detour embankments will likely only be utilized for one or two construction seasons and given the relatively slow rate of consolidation of the cohesive soils ( $t_{90}$  of about 14 years), only a portion of the estimated settlements would occur during construction. It is estimated that the detour itself would experience the following factored settlements during construction:

<b>Location</b>	<b>Total Estimated Factored Settlement (mm)</b>	<b>Estimated Factored Settlement to occur over 1-year period (mm)</b>	<b>Estimated Factored Settlement to occur over 2-year period (mm)</b>
2.5 m widened detour embankment at existing toe of slope	800 – 1100	250 – 375	375-520
Existing with 1.3 m grade raise	250 – 450	100 – 150	150 - 200

Because the total post-construction settlements exceed the settlement criteria for the TMB bridge approaches of 25 mm, settlement mitigation measures should be considered such as preloading, surcharging (larger toe berms may be required) or lightweight fill would have to be considered. Wick drains and other ground improvement options may also be considered but are likely not feasible/practical due to the embankment configuration.

Because the detour is not permanent, it may be possible to allow settlement to occur while traffic is in operation, depending on the driving surface required. Further, if the detour embankment grade could be lowered, surcharging to a maximum height of 2.5 m (stability mitigation not required) could be considered to reduce differential settlements during construction; however more detailed analysis would be required once the preferred staging has been determined. Additionally, lowering of the 1.3 m grade raise on the permanent alignment would be beneficial from a settlement mitigation perspective.



For an approximately 1.3 m grade raise on the existing embankment (for 3.9 m of total fill) and an approximately 2.5 m high detour embankment to the north, the results of the stability analyses indicate that the critical sections have a FoS less than the target 1.5. Therefore, stability mitigation measures are required for this option. Toe berms approximately 2 m high by 15 m wide (from the toe of the detour toe of slope) may be required at this site to achieve a FoS of 1.5. Alternatively, lightweight fill could be utilized to offload the embankment and reduce the size of/need for toe berms. Other ground improvement options may also be considered and their feasibility can be assessed at detail design if a permanent grade raise must be implemented.

#### **5.6.2.5 Option 3A – Shifted Alignment to the North (No Grade Raise)**

Based on the results of the settlement analysis, for an approximately 2.5 m high shifted alignment constructed to the north, the total factored primary consolidation settlement of the foundation soils is estimated to range between about 800 mm and 1100 mm (in about 14 years) plus creep. Additionally, given the proximity of the detour embankment (i.e. essentially widening of the existing embankment) to the existing alignment, settlement of the north side of the existing embankment will also occur and will be differential across the existing/future embankment cross-section, ranging from approximately 100 mm at the existing centreline to 300 mm at the existing shoulder. Because the total post-construction settlements exceed the settlement criteria for bridge approaches of 25 mm, settlement mitigation measures such as preloading, surcharging (toe berms may be required) or lightweight fill would have to be considered. Wick drains and other ground improvement options may also be considered but are likely not feasible/practical due to the embankment configuration.

For an approximate 2.5 m high shifted alignment embankment constructed to the north of the existing, the results of the stability analysis indicate that the critical sections have a FoS approximately equal to 1.5 for granular embankments constructed at slopes of 2 horizontal to 1 vertical (2H:1V) or flatter. The result of the analysis is shown on Figure 9 for a section near the west abutment.

#### **5.6.2.6 Option 3B – Shifted Alignment to the North (1.3 m Grade Raise)**

Based on the results of the settlement analysis for an approximately 1.3 m higher embankment, resulting in 3.9 m of total fill across the existing/shifted final alignment, the total factored primary consolidation settlement of the foundation soils is estimated to range between about 300 mm and 400 mm plus creep for portions on the existing alignment and 1400 mm to 1700 mm (in about 14 years) plus creep for the shifted portion of the alignment. Given the shifted profile of the proposed embankment, there will be differential settlement between the existing embankment (portion of final alignment will overlap) and the new shifted portion of the alignment. Because the total post-construction settlements exceed the settlement criteria for bridge approaches of 25 mm, settlement mitigation measures such as preloading, surcharging (larger toe berms may be required) or lightweight fill would have to be considered. Wick drains and other ground improvement options may also be considered but are likely not feasible/practical due to the embankment configuration.

For a 3.9 m total embankment fill height across the shifted/existing final embankment configuration, the critical sections have a FoS less than the target 1.5. Therefore, stability mitigation measures are required for this option. Toe berms approximately 2 m high by 20 m wide (from the toe of the detour toe of slope) may be required to achieve a FoS of 1.5. Alternatively, lightweight fill could be utilized to offload the embankment and reduce the size of/need for toe berms. Other ground improvement options may also be considered and their feasibility can be assessed at detail design if a grade raise must be implemented.

#### **5.6.2.7 Summary**

The following summary provides an overview of the potential for stability and/or settlement concerns and the anticipated requirement for mitigation measures for each option.



Option	Grade Raise	Stability Mitigation Required (Yes/No)	Settlement Mitigation Required (Yes/No)
Option 1A	None	No	No
Option 1B	~1.3 m on existing alignment	Yes	Yes
Option 2A	~2.5 m for TMB embankment	No	Yes
Option 2B	~2.5 m for TMB embankment and ~1.3 m on existing alignment	Yes	Yes
Option 3A	~ 2.5 m for shifted alignment	No	Yes
Option 3B	~3.9 m for shifted portion off existing alignment and 1.3 m over existing embankment	Yes	Yes

Based on the preliminary stability and settlement analyses and from a foundations perspective, we recommend limiting or eliminating grade raises and limiting or eliminating embankment widening(s) at this site to avoid potentially costly settlement and stability mitigation measures. Further, considering the potential for weak/very soft layers to exist within the clayey silt to clay strata, confirmation of the calculated factors of safety as well as the requirements for stability mitigation will be required following additional investigation as part of the detail design stage.

## 5.7 Construction Considerations

Based on the subsurface conditions encountered at this site, construction considerations that should be considered and addressed during detail design are as follows:

- The very soft to firm cohesive deposit at this site will have very low geotechnical resistances and may present challenges for construction equipment (i.e. cranes pads for pile driving equipment, material stockpile storage, etc.), particularly in proximity to a potentially sensitive water body or to temporary excavations.
- Subgrade soils at this site will be sensitive to disturbance from construction equipment.
- Artesian groundwater conditions observed in the piezometer in Borehole S-3 will need to be investigated further and addressed via a non-standard special provision for the pile installation and pile cap construction. This may include a granular filter blanket below and extending beyond the limits of each pile cap and the potential requirement for grouting along the flanges of the piles, which will be assessed further in detail design.
- Given the condition of the existing structure, the structural engineer will need to consider the development of a detailed monitoring program for the bridge structure during construction to monitor vibrations and vertical/lateral movements.
- Obstructions are noted to be present within the existing embankment fill (concrete, cobbles/boulders, EPS), and may affect the installation of deep foundation or protection system elements.
- The incorporation of lightweight fill materials, if adopted, would require additional detailing and a non-standard specification for their supply and placement.



## 5.8 Recommendations for Further Work During Detail Design

During the detail design phase, additional field investigation and testing will be required based on the final structure configuration, alignment and replacement strategy. We recommend the following scope of work at detail design (i.e., Phase 2 of Golder's scope of work):

- **Abutments:**
  - Additional borehole investigation to confirm the variation in the bedrock surface along the foundation elements, confirm the tip elevation for deep foundations, and obtain additional information on the cohesive soils surrounding the abutment location(s) to refine the preliminary estimate for downdrag loading.
  - Detailed design of micropiles, drilled steel casings or drilled shafts, if required based on the selected strategy and foundation option.
  - Confirmation of the stabilized groundwater elevation from piezometers installed during current phase, as well as additional piezometers if necessary, to assess mitigation requirements related to artesian conditions.
- **Approach embankments:**
  - Further assessment of the thickness and strength/consolidation properties of the weak cohesive deposits within the footprint of the existing, detour and final approach embankments based on the selected staging strategy and highway grading. In particular, if a detour, staging shift or grade raise is selected in the preferred alternative, it is recommended that a suite (or suites) of additional boreholes be completed at selected critical locations to assess the presence, thickness and continuity of weak zones within the cohesive deposit that could critically impact the embankment stability. It is recommended that such a suite consist of at least three boreholes (or Cone Penetration Tests (CPTs)) arrayed along lines perpendicular to the highway (for side slopes) and parallel to the highway (for front slopes). At each location, the boreholes or CPTs should be laid-out as follows: one through the embankment crest, one at the embankment toe, and one beyond the embankment toe.
  - Detailed assessment of the stability of the embankment front slopes and side slopes based on the selected highway profile, and the suites of boreholes/CPTs as described above. As demonstrated in Section 5.6.2.7 above, stability mitigation measures will be required for some geometric alternatives, and may also be required if a critical weak plane is identified in the detail design investigation.
  - Detailed assessment of the estimated magnitude of settlement under the new approach embankments based on the selected strategy, and more detailed development of the selected mitigation alternative(s).
  - Delineation of the limits of the EPS fill on the west side of the structure to aid in the design of the approach embankment reconstruction, or that of any widening. In this regard, consideration might be given to the use of a non-intrusive geophysical investigation to identify the limits of the concrete slab that may have been constructed over top of the EPS.
- **Temporary protection systems and dewatering:**
  - Confirmation of soil conditions along the locations where roadway protection systems will be required for the selected staging strategy.
- **Construction staging impacts on stability/settlement in temporary conditions.**



Although not included in Golder's original scope of work for Phase 2, should a grade raise be the preferred option, consideration should be given to advancing Cone Penetration Tests (CPTs) and conducting additional complex laboratory testing (i.e. triaxial and direct shear) to further refine settlement and strength parameters, and optimize (i.e., potentially make more cost effective) stability and settlement mitigation measures. As noted above, if a detour, staging shift or grade raise is necessary, the potential presence of weaker planes within the cohesive deposit (similar to those identified in the preliminary investigation) could control the global stability of the embankments; additional suites of boreholes and CPTs are recommended at critical section(s) during detailed design, if one of these alternatives is adopted.

## 6.0 CLOSURE

We trust that the preliminary geotechnical information presented in this Technical Memorandum is sufficient for you immediate needs. If any clarification is required, please do not hesitate to contact this office.

Yours truly,

**GOLDER ASSOCIATES LTD.**

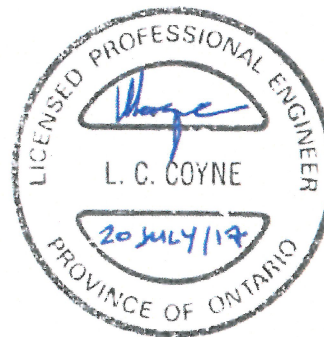


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Attachments: Table 1 – Evaluation of Bridge Replacement Alternatives  
Table 2 – Evaluation of Foundation Alternatives  
Drawing 1 – Borehole Location and Soil Strata  
Drawing 2 – Soil Strata  
List of Symbols and Abbreviations  
Lithological and Geotechnical Rock Description Terminology  
Record of Boreholes S-1 to S-4 and ST-1 to ST-4  
Record of Drillholes S-2, S-3 and ST-1 to ST-4  
Figure 1 – Water Content and Atterberg Limits  
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Figure 4 – Parameters, Undrained Shear Strengths  
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Figure 6 – Stability Analysis – West Abutment – Existing Alignment – No Grade Raise  
Figure 7 – Stability Analysis – West Abutment - Existing Alignment ~ 1.3 m Grade Raise  
Figure 8 – Stability Analysis – West Abutment – Existing Alignment ~1.3 m Grade Raise  
(Toe Berms Required)  
Figure 9 – Stability Analysis – West Abutment – Detour/Widened Alignment – No Grade Raise



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Table 1: Evaluation of Bridge Replacement Alternatives

Replacement Alternative <sup>1</sup>	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
New bridge on the existing alignment constructed using half-and-half staging <sup>2</sup> .	1	<ul style="list-style-type: none"><li>■ Replacement on current alignment without grade raise eliminates need for mitigation to reduce post-construction settlements. However, even minor widening/raising could result in settlement.</li><li>■ No grade raise eliminates potential for inducing downdrag loads on existing timber piles, or new piles for replacement bridge.</li></ul>	<ul style="list-style-type: none"><li>■ Temporary shoring required along highway centreline.</li><li>■ May not be preferred from structural point of view due to poor condition of existing bridge or other considerations (traffic, safety, etc.).</li><li>■ Settlement/stability mitigation would be required for any grade raise/widening.</li><li>■ Creep settlement still occurring which may require mitigation.</li></ul>	<ul style="list-style-type: none"><li>■ Cost of settlement/stability mitigation if any grade raise/widening is adopted; however, potentially less onerous than other staging options.</li><li>■ Costs for temporary protection systems along existing highway centreline.</li></ul>	<ul style="list-style-type: none"><li>■ Low risk of instability and relatively low risk of post-construction settlement if current grade is maintained.</li><li>■ Assumed high risk of existing bridge instability due to poor condition during construction.</li></ul>
Constructing a new bridge on the existing alignment using a temporary modular bridge (TMB) to divert traffic.	2	<ul style="list-style-type: none"><li>■ Would allow for the existing structure to be replaced in one stage without the need for temporary protection systems along the existing highway centreline; however, depending on the offset distance for the detour alignment, temporary protection systems may be required along the outside of the existing highway.</li><li>■ Settlement will occur along detour but this may be acceptable over one construction season.</li></ul>	<ul style="list-style-type: none"><li>■ Approach embankment and abutments of detour TMB would need to be placed well behind/away from the existing abutments to eliminate downdrag loads on existing timber piles; this is also dependent upon length of TMB.</li><li>■ Settlement will occur along the detour approach embankment unless mitigated.</li><li>■ Settlement will also occur under existing/future embankment due to proximity of detour alignment requiring mitigation.</li><li>■ Widened embankment (for detour) may require mitigation achieve adequate factor of safety for global stability depending on fill height.</li><li>■ TMB will require deep foundations.</li></ul>	<ul style="list-style-type: none"><li>■ Cost of settlement/stability mitigation.</li><li>■ Cost of deep foundations required for TMB structure.</li><li>■ Fill/maintenance costs required during construction.</li><li>■ Temporary or permanent property-taking may be required to accommodate detour.</li></ul>	<ul style="list-style-type: none"><li>■ High risk that the detour embankment/widening may induce downdrag loads on existing piles depending on final location.</li><li>■ Settlement and/or stability mitigation required along detour alignment, and possibly along existing/future alignment.</li><li>■ Moderate risk of post-construction settlement of existing/future embankment if current grade is maintained on existing alignment, due to ongoing influence of adjacent detour embankments.</li></ul>
New bridge constructed on a new/partially shifted alignment to the north, allowing existing bridge/new bridge to be used for traffic diversion.	3	<ul style="list-style-type: none"><li>■ Would allow existing bridge to be utilized during construction of shifted portion of the alignment; however, there will be some impact to settlement and stability of the existing embankment from the widened embankment loading.</li></ul>	<ul style="list-style-type: none"><li>■ Long-term differential settlement will occur along the widened embankment alignment, requiring mitigation. Such settlement is also anticipated to impact the existing embankment.</li><li>■ Downdrag loads on existing/new piles from widened embankment fill will occur unless mitigated.</li><li>■ Stability mitigation will be required for widened/new embankment to meet CHBDC 2014 standards.</li></ul>	<ul style="list-style-type: none"><li>■ Cost of fill required for permanent realignment.</li><li>■ Cost of settlement/stability mitigation.</li><li>■ Permanent property-taking may be required to accommodate widening.</li></ul>	<ul style="list-style-type: none"><li>■ High risk that the embankment widening will induce downdrag loads on existing piles, depending on final configuration.</li><li>■ Settlement/stability mitigation required along new alignment and possibly existing alignment depending on magnitude of shift.</li><li>■ Moderate risk of post-construction settlement of existing/widened embankment if current grade is maintained.</li></ul>

Notes:  
1. All options assume no grade raise as a base case, although comments are provided related to implications of grade raise.  
2. Half-and-half staging may not be feasible due to structural, traffic and safety considerations.



Table 2: Evaluation of Foundation Alternatives					
Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Driven steel H-piles	1 (once existing bridge removed)	<ul style="list-style-type: none"><li>■ Relatively straightforward construction.</li><li>■ Higher axial resistance compared to shallow foundation.</li><li>■ Suitable for integral abutment design.</li></ul>	<ul style="list-style-type: none"><li>■ Requires excavation below groundwater level for pile cap construction.</li><li>■ Heavy construction equipment (pile driver) may pose issues on soft soils adjacent to bridge/embankment.</li><li>■ Vibrations during pile driving could impact existing structure; non-standard special provisions may be required to address vibration monitoring during pile installation.</li><li>■ Artesian groundwater pressures may require mitigation, such as a granular filter blanket or grouting to mitigate the potential loss of fine soil particles along the piles.</li><li>■ Minimum separation distance required greater than for drilled units.</li><li>■ Downdrag loads may need to be considered depending on replacement strategy chosen.</li></ul>	<ul style="list-style-type: none"><li>■ Relative costs higher than for shallow foundations (however, these are not feasible at this site), but lower than for micropiles, drilled steel casings and drilled shafts (caissons).</li><li>■ Cost of temporary protection systems, dewatering for pile caps, and vibration monitoring.</li></ul>	<ul style="list-style-type: none"><li>■ Low risk of not achieving design resistance.</li><li>■ Moderate to high risk of ground instability due to heavy construction equipment; mitigation required for temporary works.</li><li>■ Moderate to high risk of vibrations impacting existing bridge/foundations, which are understood to be in poor condition – minimum separation distance and mitigation measures required.</li></ul>
Drilled steel casings – small diameter	2 (while existing bridge in operation)	<ul style="list-style-type: none"><li>■ Relatively straightforward construction.</li><li>■ Less disturbance/vibration compared to driven piles.</li><li>■ Well-suited to penetrating obstructions in soil, and forming socket in strong to very strong bedrock; however, these conditions not key factors at this site.</li></ul>	<ul style="list-style-type: none"><li>■ Requires excavation below groundwater level for pile cap construction.</li><li>■ Heavy construction equipment may pose issues on soft soils adjacent to bridge/embankment.</li><li>■ Artesian groundwater pressures may require mitigation, such as a granular filter blanket or grouting to mitigate the potential loss of fine soil particles along the piles.</li><li>■ Minimum separation distance required less than for driven units.</li><li>■ Downdrag loads may need to be considered depending on replacement strategy chosen.</li></ul>	<ul style="list-style-type: none"><li>■ More expensive than driven piles;; less expensive than larger diameter drilled shafts or micropiles.</li></ul>	<ul style="list-style-type: none"><li>■ Low risk of not achieving design resistance.</li><li>■ Higher risk of ground instability due to heavy construction equipment; mitigation required for temporary works.</li><li>■ Low risk (as compared with driven piles and drilled shafts) of vibrations impacting existing structure – minimum separation distance and mitigation measures required.</li></ul>
Micropiles	3	<ul style="list-style-type: none"><li>■ Lighter weight equipment more suitable for use on soft soils adjacent to bridge/embankment.</li><li>■ Less disturbance/vibration compared to driven piles.</li><li>■ May be more preferred for half-and-half staging option due to poor condition of existing bridge and need for stabilization of existing bridge.</li></ul>	<ul style="list-style-type: none"><li>■ Allows only for semi-integral abutment design.</li><li>■ Requires detailed micropile design/ drawings/specifications.</li><li>■ Pile load tests required to confirm capacity for design.</li><li>■ Requires excavation below groundwater level for pile cap construction.</li></ul>	<ul style="list-style-type: none"><li>■ Additional cost associated with detail micropile design.</li><li>■ Cost for specialist contractor. Typically higher than for driven steel H-piles or pipe piles, and similar to drilled steel casings; may be less expensive than larger diameter drilled shafts.</li><li>■ Additional cost for the micropile pile load tests.</li></ul>	<ul style="list-style-type: none"><li>■ Low risk of not achieving design resistance.</li><li>■ Low risk of ground instability due to lighter weight construction equipment.</li><li>■ Lowest risk of impacting existing bridge due to lower vibrations associated with micropile installation – minimum separation distance and mitigation measures required.</li></ul>

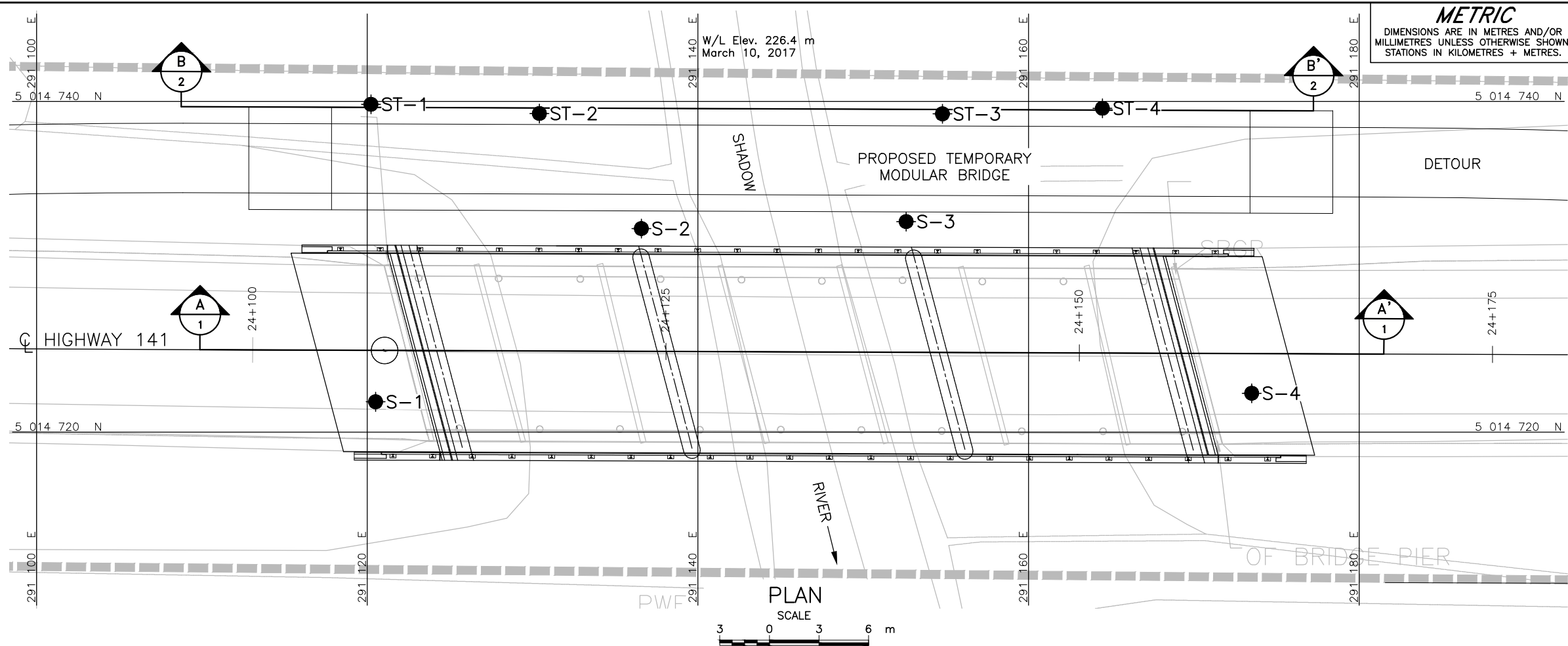


Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Driven steel pipe piles	4	<ul style="list-style-type: none"><li>■ Relatively straightforward construction.</li><li>■ Higher axial resistance compared to shallow foundations.</li><li>■ May be suitable for integral abutment design depending on pile diameter.</li></ul>	<ul style="list-style-type: none"><li>■ Requires excavation below groundwater level for pile cap construction.</li><li>■ Heavy construction equipment (pile driver) may pose issues on soft soils adjacent to bridge/embankment.</li><li>■ Vibrations during pile installation could impact existing structure; non-standard special provisions may be required to address vibration monitoring during pile installation.</li><li>■ Potentially higher risk than for driven H-piles related to migration/loss of fine soil particles due to artesian groundwater pressures; mitigation required, such as a granular filter blanket and/or grouting.</li><li>■ Minimum separation distance required greater than for drilled units.</li><li>■ Downdrag loads may need to be considered depending on replacement strategy chosen.</li></ul>	<ul style="list-style-type: none"><li>■ Relative costs higher than for shallow foundations (however, these are not feasible at this site), but lower than for micropiles, drilled steel casings and drilled shafts (caissons).</li><li>■ Cost of temporary protection systems, dewatering for pile caps and vibration monitoring.</li></ul>	<ul style="list-style-type: none"><li>■ Low risk of not achieving design resistance.</li><li>■ Moderate to high risk of ground instability due to heavy construction equipment; mitigation required for temporary works.</li><li>■ Moderate to high risk of vibrations impacting existing bridge/foundations, which are understood to be in poor condition – minimum separation distance and mitigation measures required.</li></ul>
Drilled shafts (caissons) – large diameter	5	<ul style="list-style-type: none"><li>■ Fewer deep elements required due to higher resistance.</li></ul>	<ul style="list-style-type: none"><li>■ Heavy construction equipment (caisson rig) may pose issues on soft soils adjacent to bridge/embankment.</li><li>■ Not suitable for integral abutment design.</li><li>■ Vibrations during liner installation could impact existing structure; non-standard special provisions may be required to address vibration monitoring; oscillatory equipment may be used to minimize vibrations, but this type of equipment affords slower progress and is more expensive.</li><li>■ Temporary or permanent liner required during drilled shaft installation to support existing soils and mitigate loss of ground; if temporary, potential difficulties in extracting liners due to depth of installation.</li><li>■ Appropriate procedures required to mitigate artesian groundwater pressures (i.e., liners filled with drilling mud during advancement; use of tremie concrete techniques).</li><li>■ Minimum separation distance required less than for driven units but higher due to large diameter</li><li>■ Challenges “seating” caissons into strong to very strong, moderately sloping bedrock.</li><li>■ Downdrag loads may need to be considered depending on replacement strategy chosen.</li></ul>	<ul style="list-style-type: none"><li>■ Relative costs much higher than steel H-piles, pipe piles, and drilled steel casings.</li></ul>	<ul style="list-style-type: none"><li>■ Low risk of not achieving design resistance.</li><li>■ Some risk of not achieving rock seal/socket formation in sloping bedrock.</li><li>■ Moderate to high risk of vibrations/construction activity negatively impacting existing bridge – highest minimum separation distance and mitigation measures required.</li></ul>



Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Socketted H-Piles	Not recommended	<ul style="list-style-type: none"><li>■ Relatively straightforward construction.</li><li>■ Higher axial resistance compared to shallow foundations.</li></ul>	<ul style="list-style-type: none"><li>■ Not recommended due to high potential for disturbance during casing removal.</li><li>■ Requires excavation below groundwater level for pile cap construction.</li><li>■ Heavy construction equipment may pose issues on soft soils adjacent to bridge/embankment.</li><li>■ Artesian groundwater pressures may require mitigation, such as a granular filter blanket or grouting to mitigate the potential loss of fine soil particles along the piles.</li><li>■ Minimum separation distance required less than for driven units but harder to determine based on casing removal.</li><li>■ Downdrag loads may need to be considered depending on replacement strategy chosen.</li></ul>	<ul style="list-style-type: none"><li>■ More expensive than driven piles; less expensive than larger diameter drilled shafts; similar cost to small diameter casings.</li></ul>	<ul style="list-style-type: none"><li>■ Low risk of not achieving design resistance.</li><li>■ Higher risk of ground instability due to heavy construction equipment; mitigation required for temporary works.</li><li>■ Moderate to high risk (as compared with driven piles and drilled shafts) of vibrations impacting existing structure—minimum separation distance required, mitigation measures required.</li></ul>
Shallow Foundations	Not feasible	<ul style="list-style-type: none"><li>■ Conventional construction.</li></ul>	<ul style="list-style-type: none"><li>■ Axial resistances too low for this option to be technically feasible.</li><li>■ Potential for settlement or differential settlement between abutments even with mitigation.</li><li>■ Requires deeper excavation and dewatering (cofferdam) adjacent to the river to allow for construction in-the-dry compared to that for pile caps.</li><li>■ Not suitable for integral abutment design.</li></ul>	<ul style="list-style-type: none"><li>■ Typically lower cost than deep foundations; however, at this site, significant additional costs to mitigate settlement, plus increased cost for deeper excavation and dewatering (cofferdam) to greater depths than excavation for pile caps.</li></ul>	<ul style="list-style-type: none"><li>■ Potential for instability of protection systems, and need to advance shoring with deeper excavation adjacent to existing highway and river compared to excavations for pile caps for deep foundations.</li><li>■ Higher risk of negative impacts on existing bridge (if left in operation during construction)</li></ul>



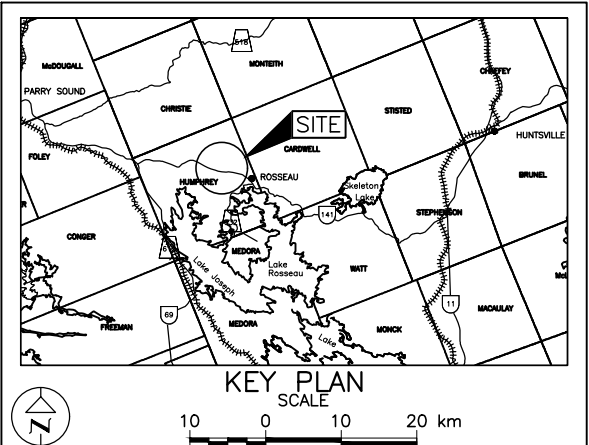


**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
WP No.

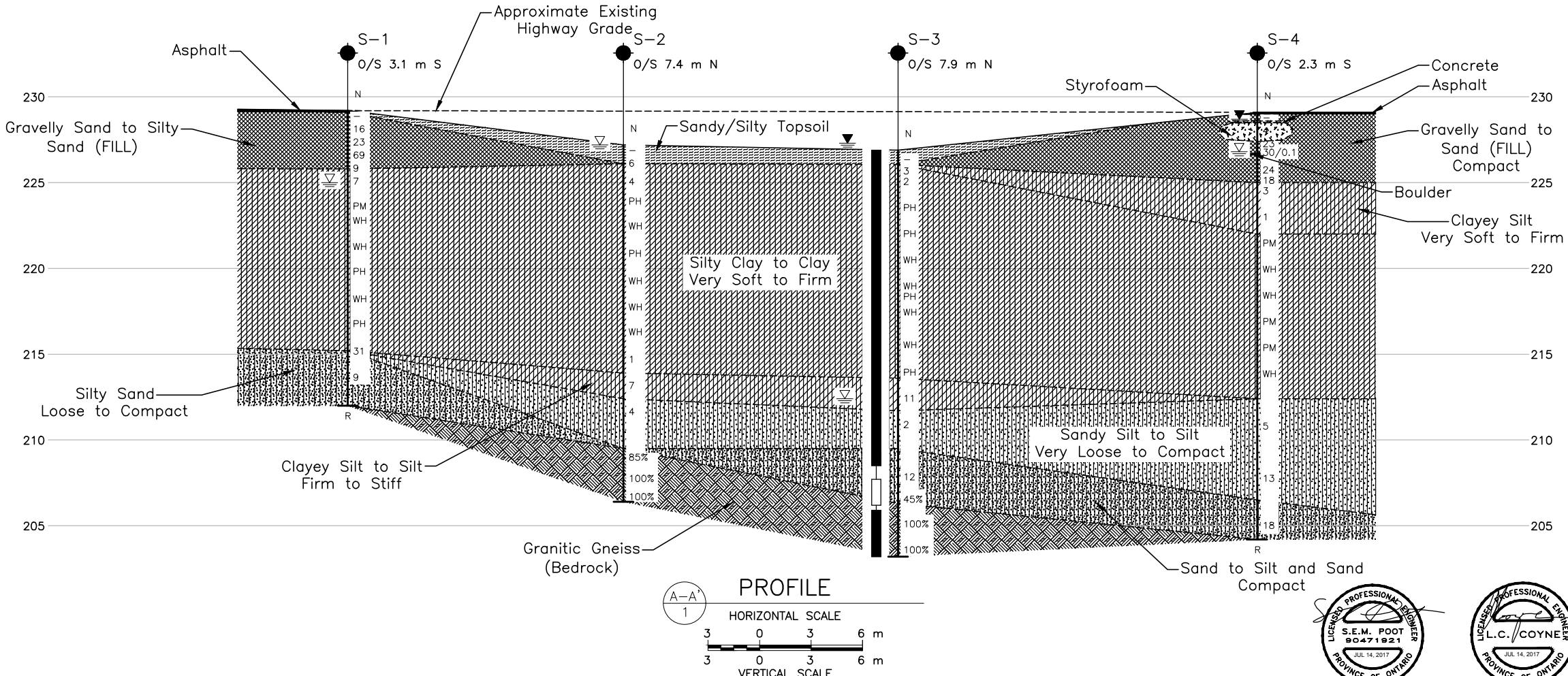
HIGHWAY 141  
SHADOW RIVER BRIDGE  
BOREHOLE LOCATIONS AND SOIL  
STRATA

SHEET



LEGEND

- Borehole - Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- R Refusal
- WL in piezometer, measured on APR 16, 2017
- WL upon completion of drilling



BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
S-1	229.2	5014721.8	291120.5
S-2	227.2	5014732.3	291136.6
S-3	226.9	5014732.7	291152.6
S-4	229.1	5014722.4	291173.5
ST-1	227.1	5014739.8	291120.2
ST-2	226.7	5014739.3	291130.4
ST-3	226.8	5014739.2	291154.8
ST-4	226.9	5014739.6	291164.5

**NOTES**

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

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

**REFERENCE**

Base plans provided in digital format by Aecom, drawing file nos. x1140651\_44-159\_Base.dwg and X\_1140651\_44-159\_ALTERNATE\_TMB.dwg, received FEB 28, 2017.



NO.	DATE	BY	REVISION
Geocres No. 31E-379			
HWY. 141	PROJECT NO. 1651997		DIST. .
SUBM'D.	CHKD. AC	DATE: 7/12/2017	SITE: 44-159
DRAWN: TB	CHKD. SEMP	APPD. LCC	DWG. 1



CONT No.  
WP No.








**HIGHWAY 141**  
SHADOW RIVER BRIDGE

**SOIL STRATA**

**SHEET**



## LEGEND

- |   |  |
|---|--|
|  | Borehole – Current Investigation                                   |
|  | Seal   |
|  | Piezometer   |
| N   | Standard Penetration Test Value                                    |
| 16  | Blows/0.3m unless otherwise stated<br>(Std. Pen. Test, 475 j/blow) |
| REC   | Recovery (%)   |
|  | WL in piezometer, measured on APR 16, 2017                         |
|  | WL upon completion of drilling                                     |

## BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
ST-1	227.1	5014739.8	291120.2
ST-2	226.7	5014739.3	291130.4
ST-3	226.8	5014739.2	291154.8
ST-4	226.9	5014739.6	291164.5

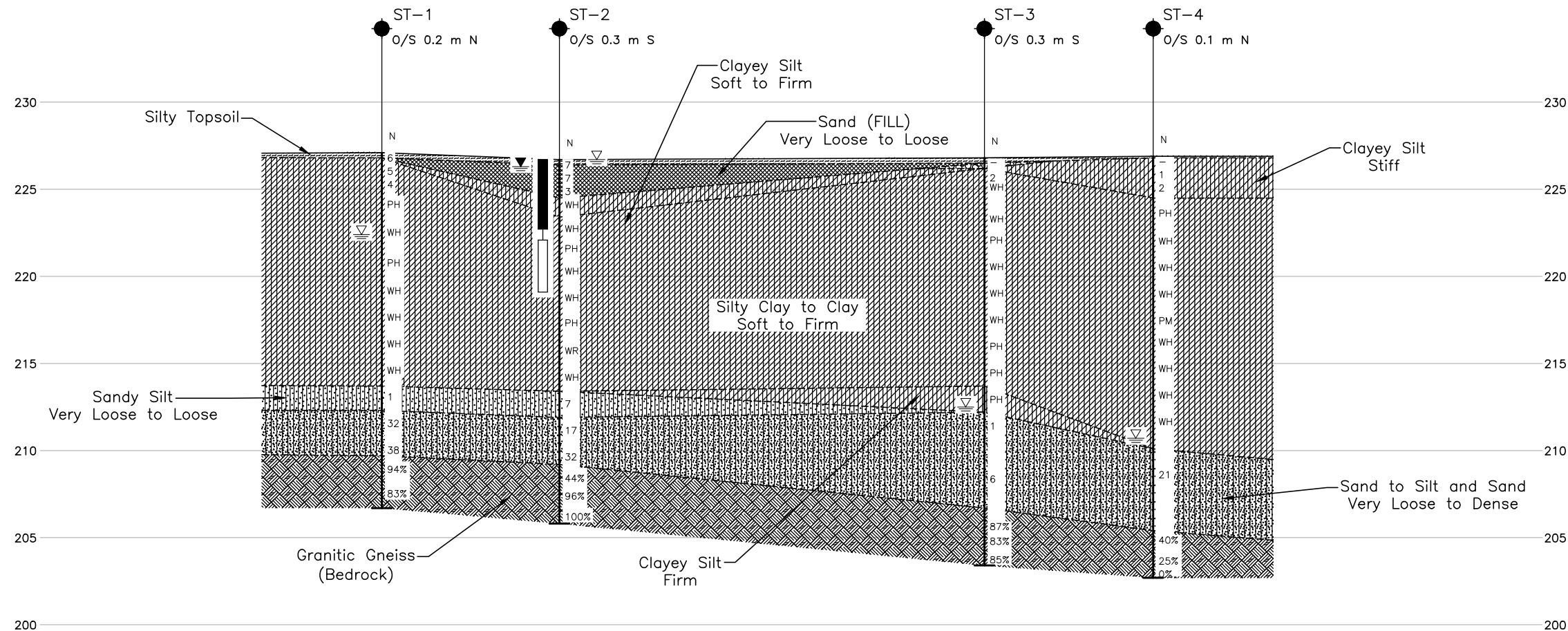
## NOTES

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X\_1140651\_44-159\_ALTERNATE\_TMB.dwg, received FEB 28, 2017.



## PROFILE

Figure 1: Profile of the road. The diagram shows a cross-section of a road with a horizontal scale and a vertical scale. The horizontal scale is marked from 3 to 6 m, and the vertical scale is marked from 3 to 6 m. A circular inset shows a detail of the road surface with a 'B-B' label and a '1' below it.



	-	.			
NO.	DATE	BY	REVISION		
<b>Geocres No. 31E-379</b>					
<b>Hwy. 414</b>		<b>PROJECT NO. 1651997</b>		<b>DIST. .</b>	
SUBM'D..	CHKD. AC	DATE: 7/12/2017		SITE: 44-159	
DRAWN: TB	CHKD. SEMP	APPD. JMJC		DWG. 2	





## LIST OF SYMBOLS

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

### I. GENERAL

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

### II. STRESS AND STRAIN

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

### III. SOIL PROPERTIES

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

### (a) Index Properties (continued)

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

### (b) Hydraulic Properties

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

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

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

### (d) Shear Strength

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

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

Notes: 1  
2

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

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





## LIST OF ABBREVIATIONS

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

### I. SAMPLE TYPE

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

### II. PENETRATION RESISTANCE

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

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

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

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

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

**WH:** Sampler advanced by static weight of hammer

**WR:** Sampler advanced by weight of sampler and rod

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

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

### III. SOIL DESCRIPTION

#### (a) Non-Cohesive (Cohesionless) Soils

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

#### (b) Cohesive Soils Consistency

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

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	relative density (specific gravity, $G_s$ )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
$SO_4$	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

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

### V. MINOR SOIL CONSTITUENTS

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





## LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

### WEATHERINGS STATE

**Fresh:** no visible sign of weathering

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

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

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

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

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

### BEDDING THICKNESS

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

### JOINT OR FOLIATION SPACING

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

### GRAIN SIZE

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

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

### CORE CONDITION

#### Total Core Recovery (TCR)

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

#### Solid Core Recovery (SCR)

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

#### Rock Quality Designation (RQD)

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

### DISCONTINUITY DATA

#### Fracture Index

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

#### Dip with Respect to Core Axis

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

#### Description and Notes

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

#### Abbreviations

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




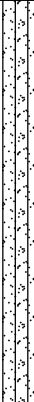
PROJECT 1651997-1104				<b>RECORD OF BOREHOLE No S-1</b>				1 OF 2 <b>METRIC</b>						
W.P. _____		LOCATION N 5014721.8; E 291120.5 MTM ZONE 10 (LAT. 45.27232055; LONG. -79.67413458)				ORIGINATED BY SA								
DIST _____ HWY 141		BOREHOLE TYPE Solid Stem Augers, NW Casing and Wash Boring				COMPILED BY AC								
DATUM GEODETIC		DATE March 14 and 15, 2017				CHECKED BY SEMP								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>p</sub>	W	W <sub>L</sub>		
229.2	GROUND SURFACE													
0.0	ASPHALT (90 mm)													
0.3	Gravelly sand, trace RAP (FILL)		1	AS	-									17 51 26 6
228.5	Silty sand, some gravel (FILL)													
0.7	Brown Moist													
	Gravelly sand, trace silt (FILL)		2	SS	16									
	Loose to very dense													
	Brown Moist to wet													
	A 75 mm cobble encountered at 2.3 m depth.		3	SS	23									
			4	SS	69									24 71 (5)
225.8			5	SS	9									
3.4	CLAY, trace sand													
	Soft to firm													
	Brown to grey		6	SS	7									
	Wet													
	Trace organics above 3.8 m depth.													
			7	TO	PM									
	Varved below 6.1 m depth.		8	SS	WH									
			9	SS	WH									
	No varves noted in Sample 10.		10	TO	PH									
			11	SS	WH									0 0 32 68

Continued Next Page

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

SUD-MTO 001 MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:



PROJECT 1651997-1104				RECORD OF BOREHOLE No S-1				2 OF 2 METRIC									
W.P. _____				LOCATION N 5014721.8; E 291120.5 MTM ZONE 10 (LAT. 45.27232055; LONG. -79.67413458)				ORIGINATED BY SA									
DIST _____ HWY 141				BOREHOLE TYPE Solid Stem Augers, NW Casing and Wash Boring				COMPILED BY AC									
DATUM GEODETIC				DATE March 14 and 15, 2017				CHECKED BY SEMP									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---																
	CLAY, trace sand Soft to firm Brown to grey Wet		12	TO	PH		217										
	Trace organics above 3.8 m depth.							216									
215.2			13A														
14.0	Silty SAND, trace to some gravel Loose to compact Grey Wet		13B	SS	31		215										
								214									
			14	SS	9												
								213									
212.0	Attempted split-spoon at 17.2 m depth.																
17.2	END OF BOREHOLE SPLIT-SPOON REFUSAL AND REFUSAL TO FURTHER CASING ADVANCEMENT  Note:  1. Water level at a depth of 4.2 m below ground surface (Elev. 225.0 m) upon completion of drilling.						212										

SUD-MTO 001 MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:



PROJECT	1651997-1104	RECORD OF BOREHOLE No S-2		1 OF 3	METRIC
W.P.		LOCATION	N 5014732.3; E 291136.6 MTM ZONE 10 (LAT. 45.27241504; LONG. -79.67413487)	ORIGINATED BY	SA
DIST	HWY 141	BOREHOLE TYPE	NW Casing, Wash Boring and NQ Coring	COMPILED BY	AC
DATUM	GEODETIC	DATE	March 22 and 23, 2017	CHECKED BY	SEMP





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

SUD-MTO 001 MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:



PROJECT 1651997-1104		<b>RECORD OF BOREHOLE No S-2</b>				2 OF 3 <b>METRIC</b>											
W.P. _____		LOCATION N 5014732.3; E 291136.6 MTM ZONE 10 (LAT. 45.27241504; LONG. -79.67413487)				ORIGINATED BY SA											
DIST _____ HWY 141		BOREHOLE TYPE NW Casing, Wash Boring and NQ Coring				COMPILED BY AC											
DATUM GEODETIC		DATE March 22 and 23, 2017				CHECKED BY SEMP											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---																
213.9	SILTY CLAY to CLAY, trace sand Soft to firm Grey Wet		10	SS	1												
13.3	CLAYEY SILT to SILT, trace to some sand Firm Grey Wet																
212.4	Sandy SILT Loose Grey Wet		11	SS	7												
14.8																	
209.5	GRANITIC GNEISS (BEDROCK)		12	SS	4												
17.7	Bedrock cored from 17.7 m to 20.8 m depth.  For coring details see Record of Drillhole S-2.																
206.4																	
20.8	END OF BOREHOLE																
	Note:  1. Water level at ground surface (Elev. 227.3 m) upon completion of drilling.																

SUD-MTO 001 MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:



PROJECT: 1651997-1104  
LOCATION: N 5014732.3; E 291136.6  
MTM ZONE 10 (LAT. 45.27241504; LONG. -79.67413487)  
INCLINATION: -90° AZIMUTH: ---

## RECORD OF DRILLHOLE: S-2

SHEET 3 OF 3  
DATUM: GEODETIC

DRILLING DATE: March 22 and 23, 2017

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	RECOVERY		R.Q.D. %	FRACT INDEX METRES	DISCONTINUITY DATA						HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
								FLUSH	TOTAL CORE %			SOLID CORE %	TYPE AND SURFACE DESCRIPTION						Jr	Ja	Jn				k, cm/s	10 <sup>-1</sup>	10 <sup>-2</sup>	10 <sup>-3</sup>																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
	JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate	BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage											PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break	BR - Broken Rock																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
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DEPTH SCALE

1 : 60



LOGGED: SA  
CHECKED: SEMP

SUD-RCK MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 28/04/17 DATA INPUT:



1 OF 4 **METRIC**

DATUM	GEODETIC	DATE	March 22 and 23, 2017	CHECKED BY	SEMP
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Continued Next Page

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

MSUD-MTO 001 MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:



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



PROJECT <u>1651997-1104</u>		<b>RECORD OF BOREHOLE No S-3</b>		3 OF 4 <b>METRIC</b>	
W.P. _____		LOCATION <u>N 5014732.7; E 291152.6 MTM ZONE 10 (LAT. 45.27241895; LONG. -79.67393096)</u>		ORIGINATED BY <u>MA</u>	
DIST _____ HWY <u>141</u>		BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers, NW Casing, Wash Boring and NQ Coring</u>		COMPILED BY <u>AC</u>	
DATUM <u>GEODETIC</u>		DATE <u>March 22 and 23, 2017</u>		CHECKED BY <u>SEMP</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  <b>γ</b>  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>						
	--- CONTINUED FROM PREVIOUS PAGE ---																				
	END OF BOREHOLE																				
	Note:  1. Water level at a depth of 14.5 m below ground surface (Elev. 212.5 m) upon completion of drilling.  2. Water level in piezometer measured at 0.4 m above ground surface (Elev. 227.3 m) on April 16, 2017.																				

SUD-MTO 001 MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:



PROJECT: 1651997-1104  
LOCATION: N 5014732.7; E 291152.6  
MTM ZONE 10 (LAT. 45.27241895; LONG. -79.67393096)  
INCLINATION: -90° AZIMUTH: ---

## RECORD OF DRILLHOLE: S-3

SHEET 4 OF 4  
DATUM: GEODETIC

DRILLING DATE: March 22 and 23, 2017

DRILL RIG: CME 850

DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate										BD - Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage				PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular				PO - Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break				BR - Broken Rock				NOTES WATER LEVELS INSTRUMENTATION
							RECOVERY		R.Q.D. %	FRACT INDEX METRES	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)	RMC -Q' AVG.													
							TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	k, cm/s																
							80 																										

DEPTH SCALE

1 : 60



LOGGED: MA  
CHECKED: SEMP

SUD-RCK MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 28/04/17 DATA INPUT:



PROJECT 1651997-1104				RECORD OF BOREHOLE No S-4				1 OF 3 METRIC									
W.P. _____				LOCATION N 5014722.4; E 291173.5 MTM ZONE 10 (LAT. 45.27232667; LONG. -79.67366432)				ORIGINATED BY MA									
DIST _____ HWY 141				BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing and Wash Boring				COMPILED BY AC									
DATUM GEODETIC				DATE March 13 to 15, 2017				CHECKED BY SEMP									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
229.1	GROUND SURFACE						20	40	60	80	100						
0.0	ASPHALT (130 mm)																
0.1	Gravelly sand, some silt (FILL)		1	AS	-											21	63 (16)
228.6	Brown Moist																
0.6	CONCRETE (150 mm)																
	STYROFOAM White		2	SS	4												
227.4																	
1.7	Gravelly sand to sand, some gravel (FILL)		3	SS	23												
	Compact Brown Moist to wet		4	SS	30/0.1											17	73 (10)
	A 75 mm cobble encountered at 2.4 m depth.		-	RC	REC = 11%												
			5	SS	24												
225.0			6	SS	18												
4.1	CLAYEY SILT, trace to some sand																
	Very soft to stiff Grey Wet		7	SS	3												
			8	SS	1											0	9 63 28
222.0																	
7.1	CLAY		9	TO	PM												
	Very soft to soft Grey																
			10	SS	WH												
	Varved below 8.8 m depth.																
			11	SS	WH												

Continued Next Page

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

SUD-MTO 001 MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:





2 OF 3 **METRIC**

DATUM	GEODETIC	DATE	March 13 to 15, 2017	CHECKED BY	SEMP
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Continued Next Page

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





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

SUD-MTO 001 MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:



PROJECT <u>1651997-1104</u>		<b>RECORD OF BOREHOLE No ST-1</b>		1 OF 3 <b>METRIC</b>	
W.P. _____		LOCATION <u>N 5014739.8; E 291120.2 MTM ZONE 10 (LAT. 45.27248221; LONG. -79.67434408)</u>		ORIGINATED BY <u>SA</u>	
DIST _____ HWY <u>141</u>		BOREHOLE TYPE <u>NW Casing, Wash Boring and NQ Coring</u>		COMPILED BY <u>AC</u>	
DATUM <u>GEODETIC</u>		DATE <u>March 16, 2017</u>		CHECKED BY <u>SEMP</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED	w <sub>p</sub>	w	w <sub>L</sub>		
227.1	GROUND SURFACE							20 40 60 80 100						
0.0	Silty TOPSOIL		1	SS	6									
226.8	Brown													
0.3	Frozen													
	SILTY CLAY, trace to some sand		2	SS	5									
	Stiff													
	Brown		3	SS	4									
	Wet													
224.7	CLAY													
2.4	Soft to firm		4	TO	PH									
	Grey													
	Wet													
			5	SS	WH									
			6	TO	PH									
			7	SS	WH									
			8	SS	WH									
			9	SS	WH									

Varved below 7.6 m depth.

Continued Next Page

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

SUD-MTO 001 MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:



PROJECT <u>1651997-1104</u>		<b>RECORD OF BOREHOLE No ST-1</b>				2 OF 3 <b>METRIC</b>								
W.P. _____		LOCATION <u>N 5014739.8; E 291120.2 MTM ZONE 10 (LAT. 45.27248221; LONG. -79.67434408)</u>				ORIGINATED BY <u>SA</u>								
DIST _____ HWY <u>141</u>		BOREHOLE TYPE <u>NW Casing, Wash Boring and NQ Coring</u>				COMPILED BY <u>AC</u>								
DATUM <u>GEODETIC</u>		DATE <u>March 16, 2017</u>				CHECKED BY <u>SEMP</u>								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100						
	CLAY Soft to firm Grey Wet		10	SS	WH		215							
213.7							214							
13.4	Sandy SILT, trace clay Very loose Grey Wet		11	SS	1		213						NP	0 25 72 3
212.3							212							
14.8	SAND, trace to some gravel, some silt Dense Grey Wet		12	SS	32		211							
							210							12 73 (15)
209.7			13	SS	38		209							RQD = 94%
17.4	GRANITIC GNEISS (BEDROCK)  Bedrock cored from 17.4 m to 20.4 m depth.  For coring details see Record of Drillhole ST-1.		1	RC	REC 100%		208							RQD = 83%
			2	RC	REC 100%		207							
206.7														
20.4	END OF BOREHOLE  Note:  1. Water level at a depth of 4.7 m below ground surface (Elev. 222.5 m) upon completion of drilling.  2. Vane at 1.7 m depth taken from separate borehole located 1.5 m south of Borehole ST-1.													

SUD-MTO 001 MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:



PROJECT: 1651997-1104  
LOCATION: N 5014739.8; E 291120.2  
MTM ZONE 10 (LAT. 45.27248221; LONG. -79.67434408)  
INCLINATION: -90° AZIMUTH: ---

## RECORD OF DRILLHOLE: ST-1

SHEET 3 OF 3  
DATUM: GEODETIC

DRILLING DATE: March 16, 2017  
DRILL RIG: CME 55  
DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	CORRELATION & LOGGING														NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
							FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA						HYDRAULIC CONDUCTIVITY		Diametral Point Load Index (MPa)					RMC -Q' AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
								TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	k, cm/s																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
																			JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate							BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	

DEPTH SCALE

1 : 60



LOGGED: SA  
CHECKED: SEMP





SUD-RCK MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 28/04/17 DATA INPUT:



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

MSUD-MTO 001 MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:



PROJECT		RECORD OF BOREHOLE No ST-2				2 OF 3 METRIC															
1651997-1104																					
W.P. _____		LOCATION N 5014739.3; E 291130.4 MTM ZONE 10 (LAT. 45.27247791; LONG. -79.67421408)				ORIGINATED BY SA															
DIST _____ HWY 141		BOREHOLE TYPE NW Casing, Wash Boring and NQ Coring				COMPILED BY AC															
DATUM GEODETIC		DATE March 20 and 21, 2017				CHECKED BY SEMP															
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL		
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100					W <sub>p</sub> W W <sub>L</sub> 20 40 60			kN/m <sup>3</sup>					
213.4	SILTY CLAY to CLAY Soft to firm Grey Wet		11	SS	WH		214														
13.3	Sandy SILT, trace to some clay Loose Grey Wet		12	SS	7		213														
211.9	SAND, some gravel, some silt Compact to dense Grey Wet		13	SS	17		212														
14.8			14	SS	32		211														
209.2							210														
17.5	GRANITIC GNEISS (BEDROCK)  Bedrock cored from 17.5 m to 20.9 m depth.  For coring details see Record of Drillhole ST-2.		1	RC	REC 94%		209														
			2	RC	REC 100%		208														
			3	RC	REC 100%		207														
205.8							206														
20.9	END OF BOREHOLE  Note:  1. Water level at ground surface (Elev. 226.7 m) upon completion of drilling.  2. Piezometer installed in separate borehole located 0.6 m southeast of Borehole ST-2.  3. Water level in piezometer measured at 0.4 m depth (Elev. 226.3 m) on April 16, 2017.																				



PROJECT: 1651997-1104  
LOCATION: N 5014739.3; E 291130.4  
MTM ZONE 10 (LAT. 45.27247791; LONG. -79.67421408)  
INCLINATION: -90° AZIMUTH: ---

## RECORD OF DRILLHOLE: ST-2

SHEET 3 OF 3  
DATUM: GEODETIC

DRILLING DATE: March 20 and 21, 2017  
DRILL RIG: CME 55  
DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	COLOUR % RETURN	FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA						HYDRAULIC CONDUCTIVITY				Diametral Index (MPa)	RMC -Q AVG.	NOTES WATER LEVELS INSTRUMENTATION
					DEPTH (m)	TOTAL CORE %				SOLID CORE %	TYPE AND SURFACE DESCRIPTION						k, cm/s									
											Jr			Ja	Jn	10 <sup>0</sup>	10 <sup>1</sup>	10 <sup>2</sup>	10 <sup>3</sup>	10 <sup>4</sup>	10 <sup>5</sup>					
		REFER TO PREVIOUS PAGE		209.2																						
18	HQ	GRANITIC GNEISS Strong Slightly weathered to fresh Fine grained Foliated Black/pink  A 75 mm thick clay/silt filled seam was noted at 18.5 m depth.		17.5																						
19	CME 55 NQ Coring			1																						
20				2																						
21				3																						
21		END OF DRILLHOLE		205.8 20.9																						
22																										
23																										
24																										
25																										
26																										
27																										
28																										
29																										

DEPTH SCALE

1 : 60



LOGGED: SA  
CHECKED: SEMP

SUD-RCK MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 28/04/17 DATA INPUT:



1 OF 4 **METRIC**



DATUM	GEODETIC	DATE	March 20 and 21, 2017	CHECKED BY	SEMP
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Continued Next Page

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

MSUD-MTO 001 MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:



PROJECT		1651997-1104		<b>RECORD OF BOREHOLE No ST-3</b>				2 OF 4 <b>METRIC</b>								
W.P.				LOCATION				N 5014739.2; E 291154.8 MTM ZONE 10 (LAT. 45.27247748; LONG. -79.67390311)								
DIST		HWY 141		BOREHOLE TYPE				NW Casing, Wash Boring and NQ Coring								
DATUM		GEODETIC		DATE				March 20 and 21, 2017								
								ORIGINATED BY								
								MA								
								COMPILED BY								
								AC								
								CHECKED BY								
								SEMP								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>p</sub>	W	W <sub>L</sub>				
								20 40 60 80 100	20 40 60 80 100	20 40 60	20 40 60	20 40 60				
--- CONTINUED FROM PREVIOUS PAGE ---																
213.7	SILTY CLAY to CLAY, trace sand Soft to firm Grey Wet		10	TO	PH	▽	214									
13.1	CLAYEY SILT Firm Grey Wet						213									
212.2	SILT and SAND, trace clay Very loose to loose Grey Wet		11	TO	PH		212									
14.6		12	SS	1	211											
					210											
					209											
					208											
					207											
					206											
					205											
					204											
206.7	GRANITIC GNEISS (BEDROCK)		1	RC	REC 100%											RQD = 87%
20.1	Bedrock cored from 20.1 m to 23.4 m depth.  For coring details see Record of Drillhole ST-3.		2	RC	REC 90%											RQD = 83%
			3	RC	REC 100%									RQD = 85%		
203.4																
23.4																

SUD-MTO 001 MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:

Continued Next Page

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



PROJECT <u>1651997-1104</u>	<b>RECORD OF BOREHOLE No ST-3</b>	3 OF 4 <b>METRIC</b>
W.P. _____	LOCATION <u>N 5014739.2; E 291154.8 MTM ZONE 10 (LAT. 45.27247748; LONG. -79.67390311)</u>	ORIGINATED BY <u>MA</u>
DIST _____ HWY <u>141</u>	BOREHOLE TYPE <u>NW Casing, Wash Boring and NQ Coring</u>	COMPILED BY <u>AC</u>
DATUM <u>GEODETIC</u>	DATE <u>March 20 and 21, 2017</u>	CHECKED BY <u>SEMP</u>

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT  <b>γ</b>  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W <sub>p</sub>	W	W <sub>L</sub>		GR	SA	SI	CL	
					○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)											
	--- CONTINUED FROM PREVIOUS PAGE ---  END OF BOREHOLE  Note:  1. Water level at a depth of 14.3 m below ground surface (Elev. 212.5 m) upon completion of drilling.																				

SUD-MTO 001 MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:



SHEET 4 OF 4

DATUM: GEODETIC

DRILLING CONTRACTOR: Landcore Drilling

[illegible]

CHECKED: SEMP



PROJECT

1651997-1104

RECORD OF BOREHOLE

No ST-4

1 OF 4

METRIC

W.P.

LOCATION

N 5014739.6; E 291164.5 MTM ZONE 10 (LAT. 45.27248127; LONG. -79.67377949)

ORIGINATED BY

MA

DIST

HWY

141

BOREHOLE TYPE

108 mm I.D. Hollow Stem Augers, NW Casing, Wash Boring and NQ Coring

COMPILED BY

AC

DATUM

GEODETIC

DATE

March 16 to 17 and 23, 2017

CHECKED BY

SEMP

[illegible]

Continued Next Page

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

SUD-MTO 001 MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:



PROJECT <u>1651997-1104</u>		<b>RECORD OF BOREHOLE No ST-4</b>		2 OF 4 <b>METRIC</b>	
W.P. _____		LOCATION <u>N 5014739.6; E 291164.5 MTM ZONE 10 (LAT. 45.27248127; LONG. -79.67377949)</u>		ORIGINATED BY <u>MA</u>	
DIST _____ HWY <u>141</u>		BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers, NW Casing, Wash Boring and NQ Coring</u>		COMPILED BY <u>AC</u>	
DATUM <u>GEODETIC</u>		DATE <u>March 16 to 17 and 23, 2017</u>		CHECKED BY <u>SEMP</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								<div><div><div>○ UNCONFINED</div><div>● QUICK TRIAXIAL</div></div><div><div>+ FIELD VANE</div><div>× REMOULDED</div></div></div>	20	40	60	80	100	<div><div><div><div><math>w_p</math></div></div></div><div><div><div><math>w</math></div></div></div><div><div><div><math>w_L</math></div></div></div></div>						
	--- CONTINUED FROM PREVIOUS PAGE ---																			
	CLAY, trace sand Soft to firm Grey Wet		10	SS	WH															
							214													
			11	SS	WH															
							213													
			12	SS	WH		212													
							211													
210.1																				
16.8	SAND to Silty SAND Compact Grey Wet						210													
							209													
			13	SS	21												0	82		
							208										9	9		
							207													
							206													
205.4							205													
21.5	GRANITIC GNEISS (BEDROCK)  Bedrock cored from 21.5 m to 24.2 m depth.  For coring details see Record of Drillhole ST-4.		1	RC	REC 100%													RQD = 40%		
			2	RC	REC 100%		204											RQD = 25%		
			3	RC			203											RQD = 0%		

Continued Next Page

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

SUD-MTO 001 MTM ZN INC LAT/LONG: 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:



PROJECT <u>1651997-1104</u>	<b>RECORD OF BOREHOLE No ST-4</b>	3 OF 4 <b>METRIC</b>
W.P. _____	LOCATION <u>N 5014739.6; E 291164.5 MTM ZONE 10 (LAT. 45.27248127; LONG. -79.67377949)</u>	ORIGINATED BY <u>MA</u>
DIST _____ HWY <u>141</u>	BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers, NW Casing, Wash Boring and NQ Coring</u>	COMPILED BY <u>AC</u>
DATUM <u>GEODETIC</u>	DATE <u>March 16 to 17 and 23, 2017</u>	CHECKED BY <u>SEMP</u>

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE   LIQUID CONTENT   LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>					
202.7			3	RC	REC													RQD = 0%		
24.2	END OF BOREHOLE				100%															
	Note:  1. Water level at a depth of 16.2 m below ground surface (Elev. 210.7 m) upon completion of drilling.  2. Vane at 1.7 m depth taken from separate borehole located 2.5 m east of Borehole ST-4.																			

SUD-MTO 001 MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 21/04/17 DATA INPUT:



INCLINATION: -90°      AZIMUTH: --

DRILLING CONTRACTOR: Landcore Drilling

DATUM: GEODETIC

1 : 60



CHECKED: SEMP

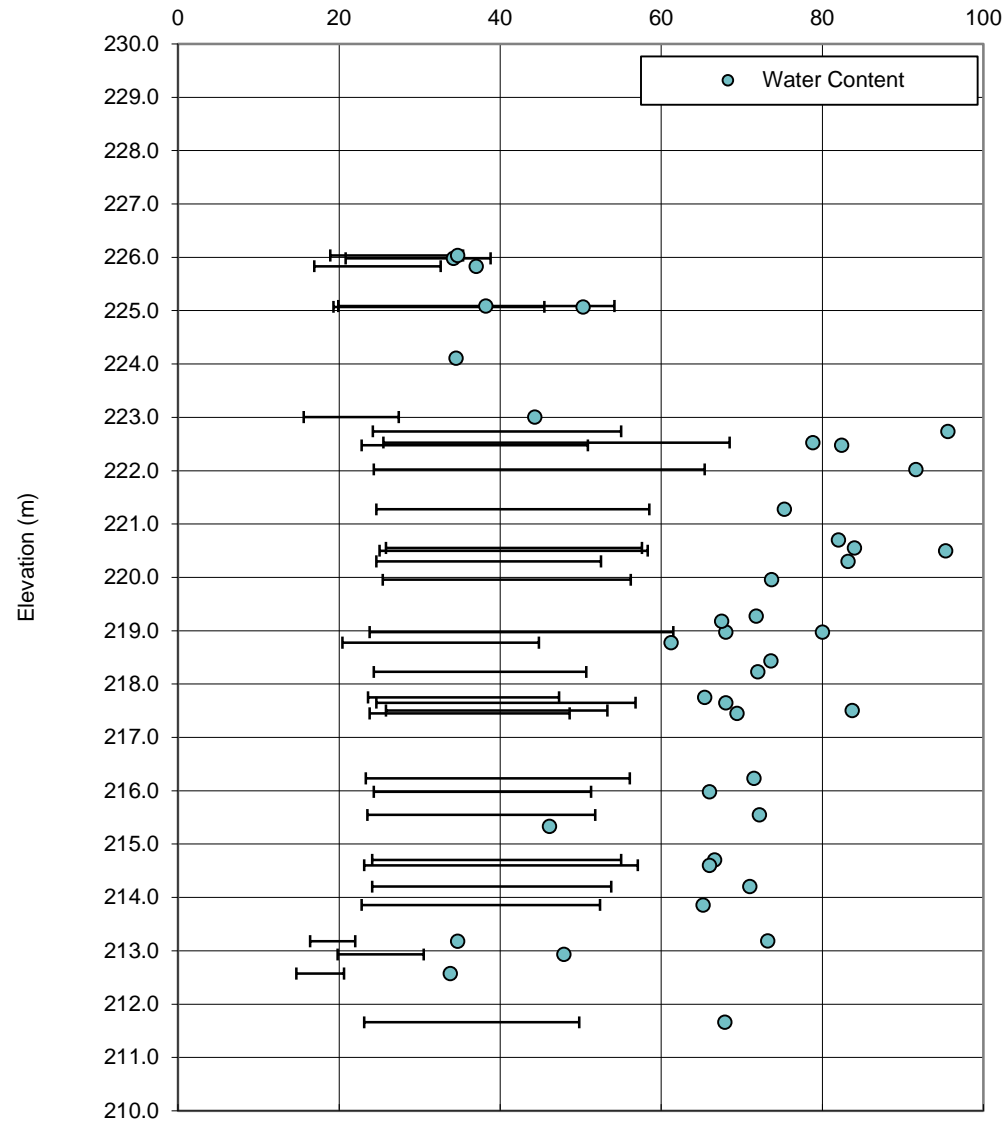
SUD-RCK MTM ZN INC LAT/LONG 1651997.GPJ GAL-MISS.GDT 28/04/17 DATA INPUT:





## Shadow River Water Content and Atterberg Limits

Figure 1



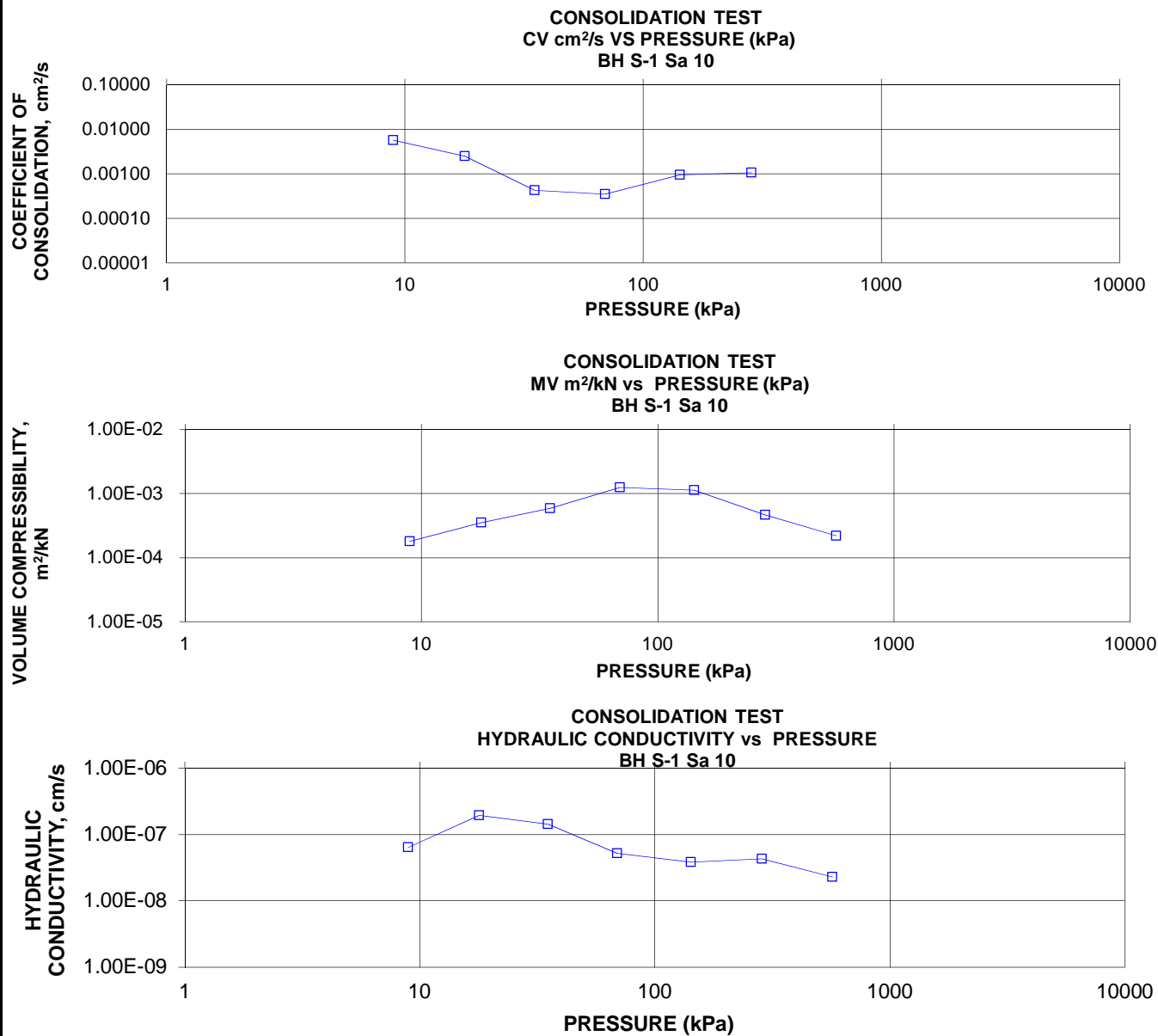


CONSOLIDATION TEST SUMMARY						FIGURE 2 Pg. 1 of 4			
<b>SAMPLE IDENTIFICATION</b>									
Project Number	1651997-1002			Sample Number	10				
Borehole Number	S-1			Sample Depth, m	9.3				
<b>TEST CONDITIONS</b>									
Test Type	Standard			Load Duration, hr	24				
Oedometer Number	1								
Date Started	3/24/17								
Date Completed	4/6/17								
<b>SAMPLE DIMENSIONS AND PROPERTIES - INITIAL</b>									
Sample Height, cm	2.544			Unit Weight, kN/m <sup>3</sup>	15.89				
Sample Diameter, cm	6.357			Dry Unit Weight, kN/m <sup>3</sup>	9.27				
Area, cm <sup>2</sup>	31.74			Specific Gravity, measured	2.785				
Volume, cm <sup>3</sup>	80.75			Solids Height, cm	0.863				
Water Content, %	71.43			Volume of Solids, cm <sup>3</sup>	27.40				
Wet Mass, g	130.82			Volume of Voids, cm <sup>3</sup>	53.35				
Dry Mass, g	76.31								
<b>TEST COMPUTATIONS</b>									
Pressure kPa	Primary Consolidation mm	Corr. Height cm	End of Primary Void Ratio	Average Height cm	t <sub>90</sub> sec	cv, cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s	Total Work kJ/m <sup>3</sup>
0	0.00	2.544	1.947	2.544					
9	0.04	2.538	1.942	2.541	375	0.00365	1.79E-04	6.41E-08	0.007
18	0.06	2.528	1.933	2.533	240	0.00567	3.50E-04	1.95E-07	0.049
35	0.22	2.493	1.903	2.511	540	0.00247	5.92E-04	1.44E-07	0.320
69	0.95	2.364	1.778	2.428	2940	0.00043	1.24E-03	5.19E-08	2.564
143	1.76	2.157	1.535	2.260	3110	0.00035	1.12E-03	3.84E-08	11.843
285	1.38	1.981	1.339	2.069	960	0.00095	4.66E-04	4.32E-08	28.332
570	1.22	1.847	1.154	1.914	735	0.00106	2.21E-04	2.29E-08	62.230
285	-0.08	1.879	1.176	1.863					
143	-0.10	1.908	1.210	1.893					
35	-0.36	1.957	1.267	1.932					
9	-0.28	1.998	1.314	1.978					
Note: k calculated using $\alpha_v$ based on $t_{90}$ values. Void ratio for unloading (or rebound) calculated for the end of increment.									
<b>SAMPLE DIMENSIONS AND PROPERTIES - FINAL</b>									
Sample Height, cm	2.065			Unit Weight, kN/m <sup>3</sup>	16.27				
Sample Diameter, cm	6.36			Dry Unit Weight, kN/m <sup>3</sup>	11.42				
Area, cm <sup>2</sup>	31.74			Specific Gravity, measured	2.785				
Volume, cm <sup>3</sup>	65.54			Solids Height, cm	0.863				
Water Content, %	42.45			Volume of Solids, cm <sup>3</sup>	27.40				
Wet Mass, g	108.70			Volume of Voids, cm <sup>3</sup>	38.14				
Dry Mass, g	76.31								
<div style="display: flex; justify-content: space-between;"> <span>Prepared By: TC</span> <span><b>Golder Associates</b></span> <span>Checked By: AC/AB/MT</span> </div>									

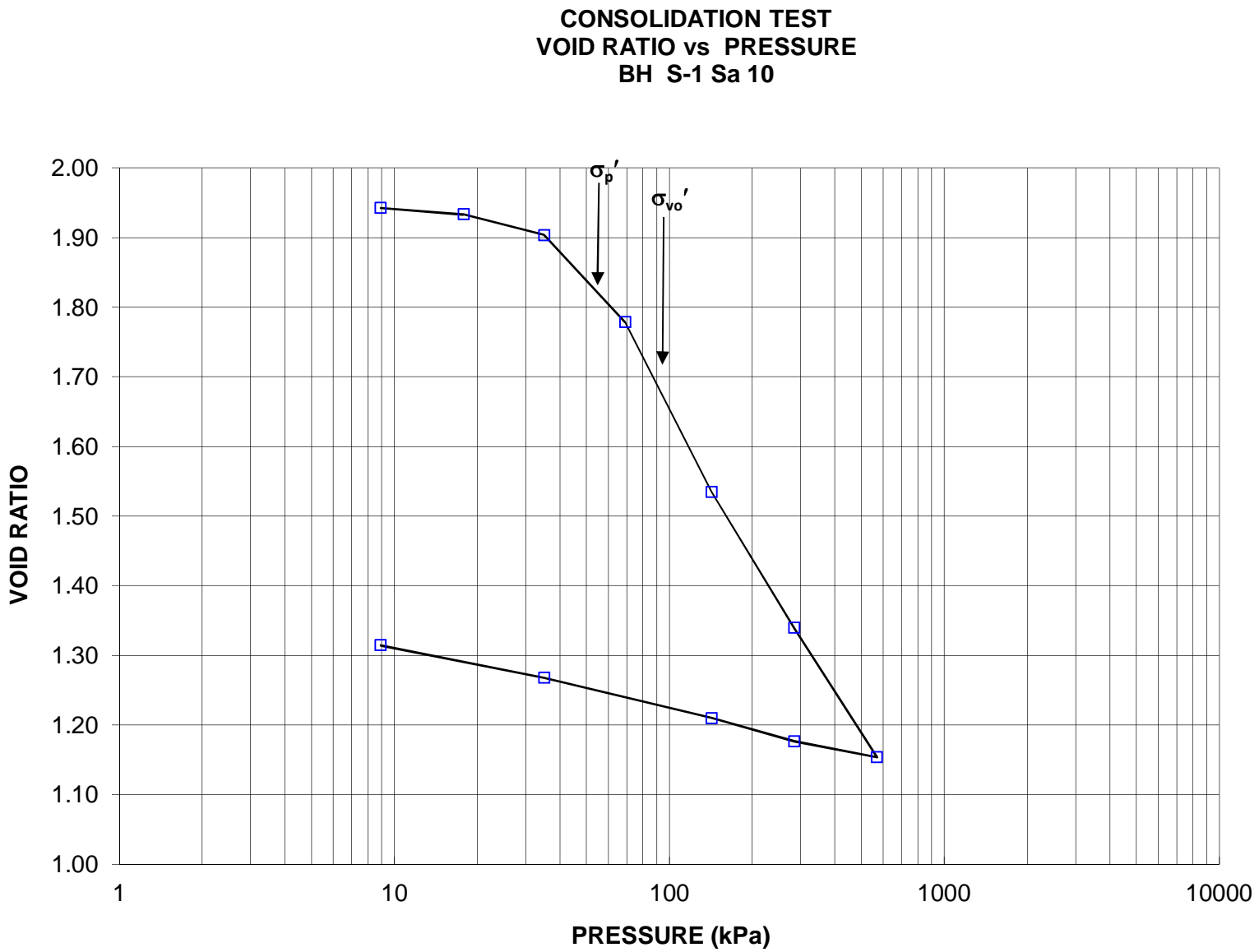


## CONSOLIDATION TEST SUMMARY

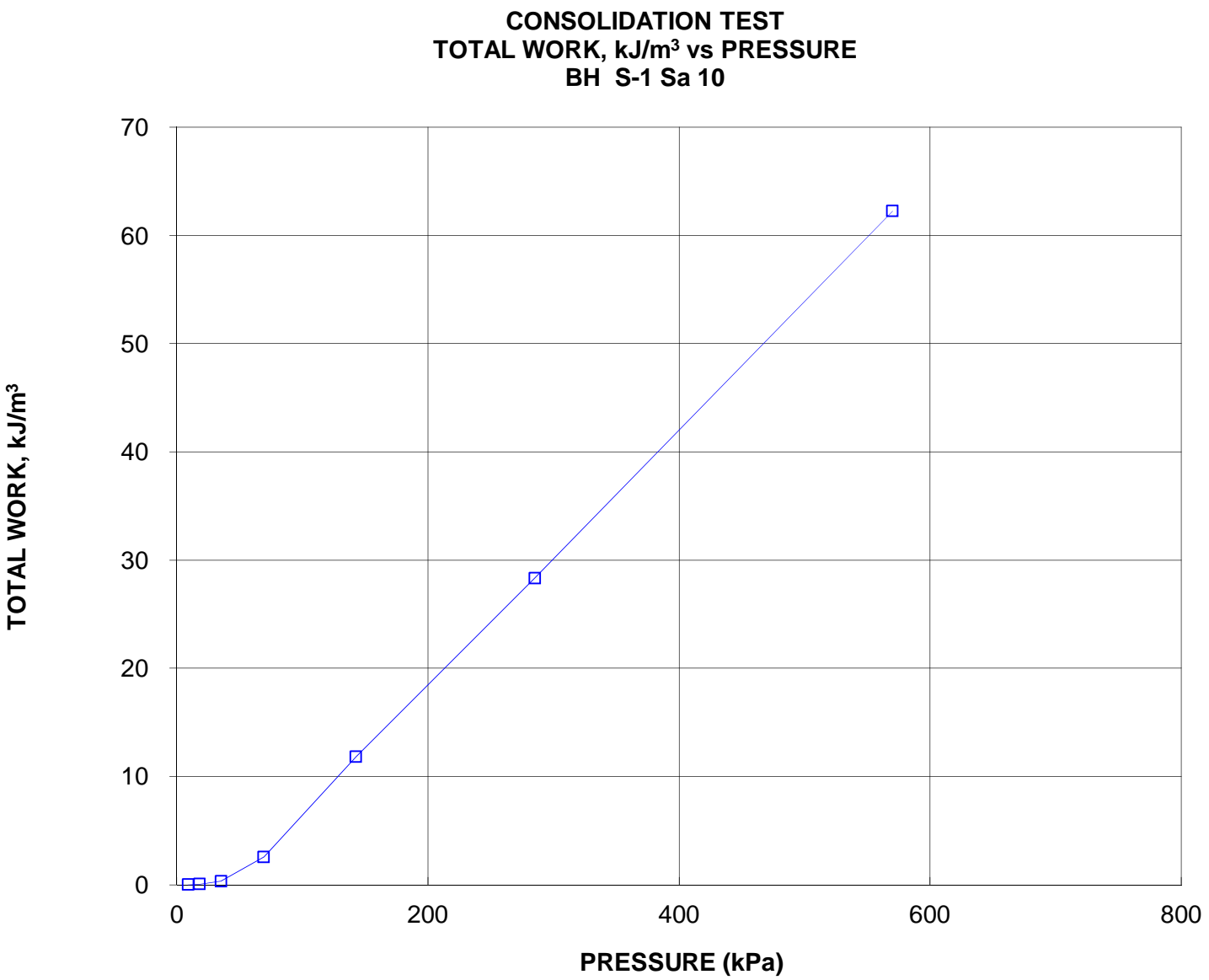
**FIGURE 2**  
Pg. 2 of 4











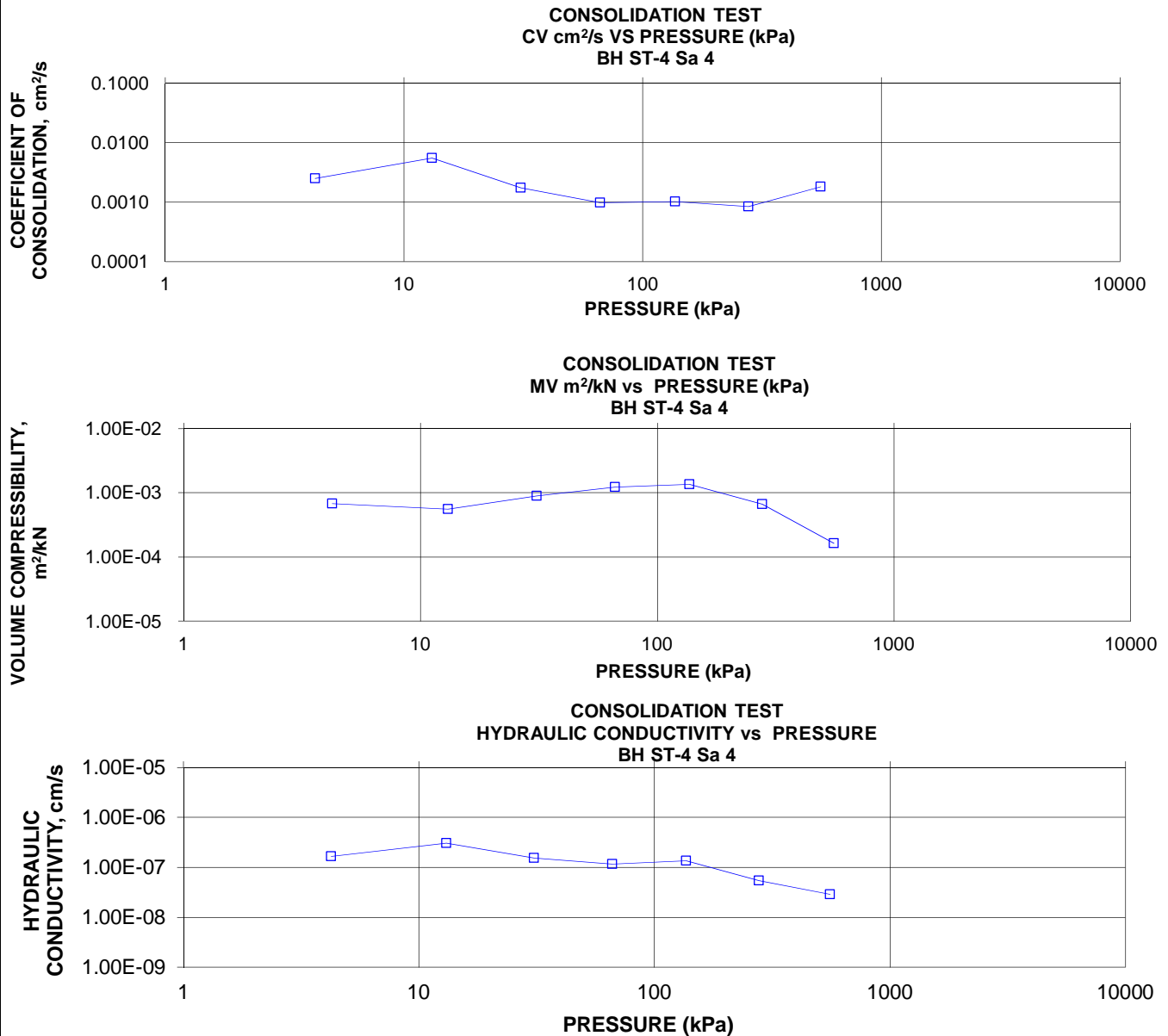


CONSOLIDATION TEST SUMMARY						FIGURE 3 Pg. 1 of 4			
SAMPLE IDENTIFICATION									
Project Number		1651997-1002				Sample Number		4	
Borehole Number		ST-4				Sample Depth, m		3.2	
TEST CONDITIONS									
Test Type		Standard				Load Duration, hr		24	
Oedometer Number		2							
Date Started		March 24, 2017							
Date Completed		April 6, 2017							
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL									
Sample Height, cm		2.522				Unit Weight, kN/m <sup>3</sup>		15.62	
Sample Diameter, cm		6.358				Dry Unit Weight, kN/m <sup>3</sup>		8.95	
Area, cm <sup>2</sup>		31.74				Specific Gravity, Measured		2.781	
Volume, cm <sup>3</sup>		80.06				Solids Height, cm		0.828	
Water Content, %		74.56				Volume of Solids, cm <sup>3</sup>		26.27	
Wet Mass, g		127.55				Volume of Voids, cm <sup>3</sup>		53.78	
Dry Mass, g		73.07							
TEST COMPUTATIONS									
Pressure kPa	Primary Consolidation mm	Corr. Height cm	End of Primary Void Ratio	Average Height cm	t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s	Total Work kJ/m <sup>3</sup>
0	0	2.522	2.047	2.522					
4	0.07	2.510	2.038	2.516	540	0.0025	6.79E-04	1.65E-07	0.006
13	0.08	2.491	2.023	2.500	240	0.0055	5.59E-04	3.03E-07	0.049
31	0.29	2.442	1.975	2.466	735	0.0018	8.94E-04	1.54E-07	0.402
66	0.89	2.281	1.843	2.362	1215	0.0010	1.22E-03	1.16E-07	2.548
137	1.65	2.037	1.557	2.159	960	0.0010	1.33E-03	1.35E-07	12.757
277	1.55	1.856	1.274	1.947	960	0.0008	6.60E-04	5.41E-08	35.628
558	0.88	1.726	1.136	1.791	375	0.0018	1.62E-04	2.88E-08	61.086
277	-0.06	1.757	1.093	1.741					
137	-0.09	1.784	1.133	1.770					
31	-0.28	1.826	1.189	1.805					
4	-0.22	1.860	1.232	1.843					
Note: k calculated using $\alpha_v$ based on t <sub>90</sub> values. Void ratio for unloading (or rebound) calculated for the end of increment									
SAMPLE DIMENSIONS AND PROPERTIES - FINAL									
Sample Height, cm		2.046				Unit Weight, kN/m <sup>3</sup>		15.57	
Sample Diameter, cm		6.36				Dry Unit Weight, kN/m <sup>3</sup>		11.03	
Area, cm <sup>2</sup>		31.74				Specific Gravity, Measured		2.781	
Volume, cm <sup>3</sup>		64.96				Solids Height, cm		0.828	
Water Content, %		41.12				Volume of Solids, cm <sup>3</sup>		26.27	
Wet Mass, g		103.12				Volume of Voids, cm <sup>3</sup>		38.69	
Dry Mass, g		73.07							
Prepared By: TC				Golder Associates				Checked By: AC/AB/MT	

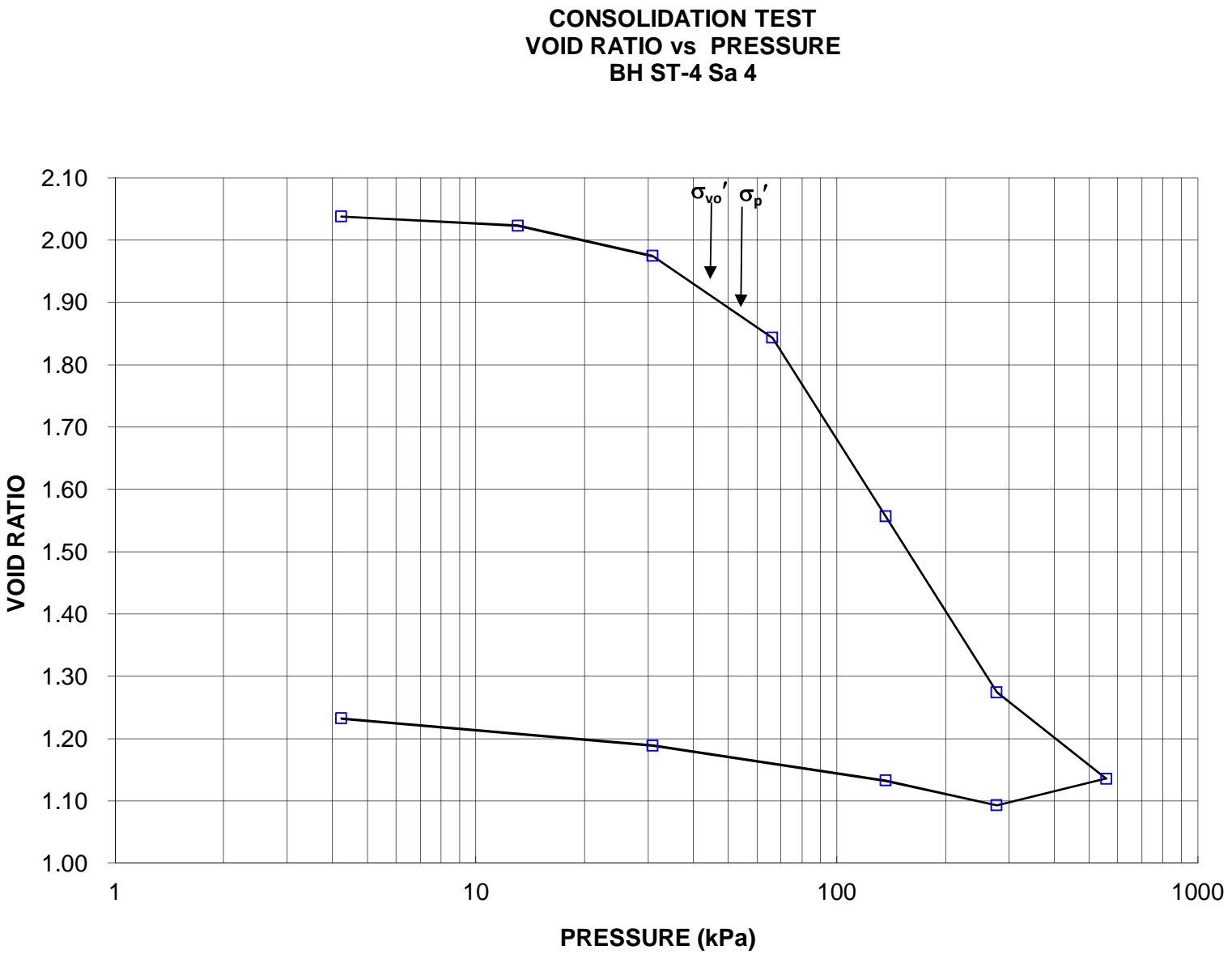


## CONSOLIDATION TEST SUMMARY

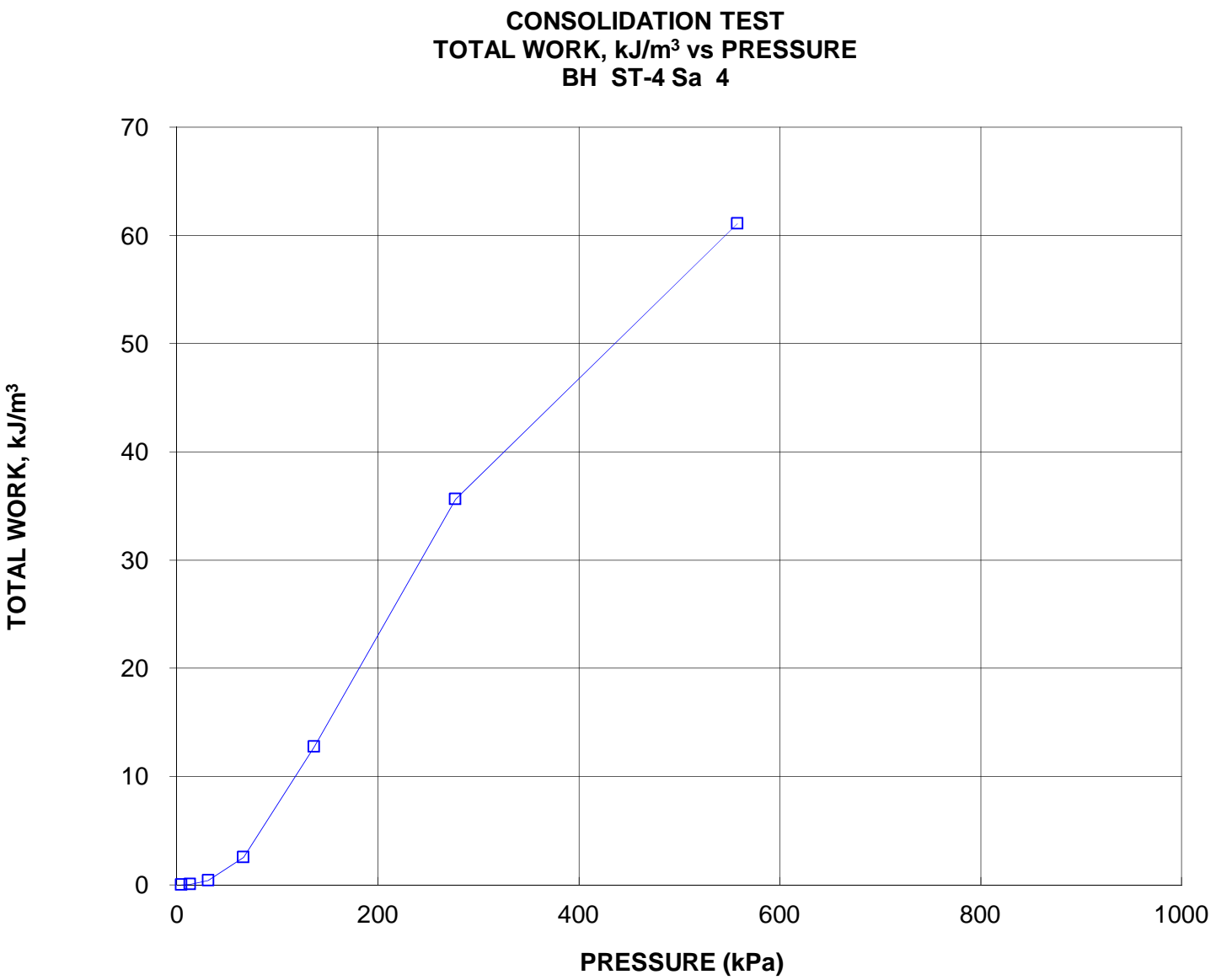
**FIGURE 3**  
Pg. 2 of 4









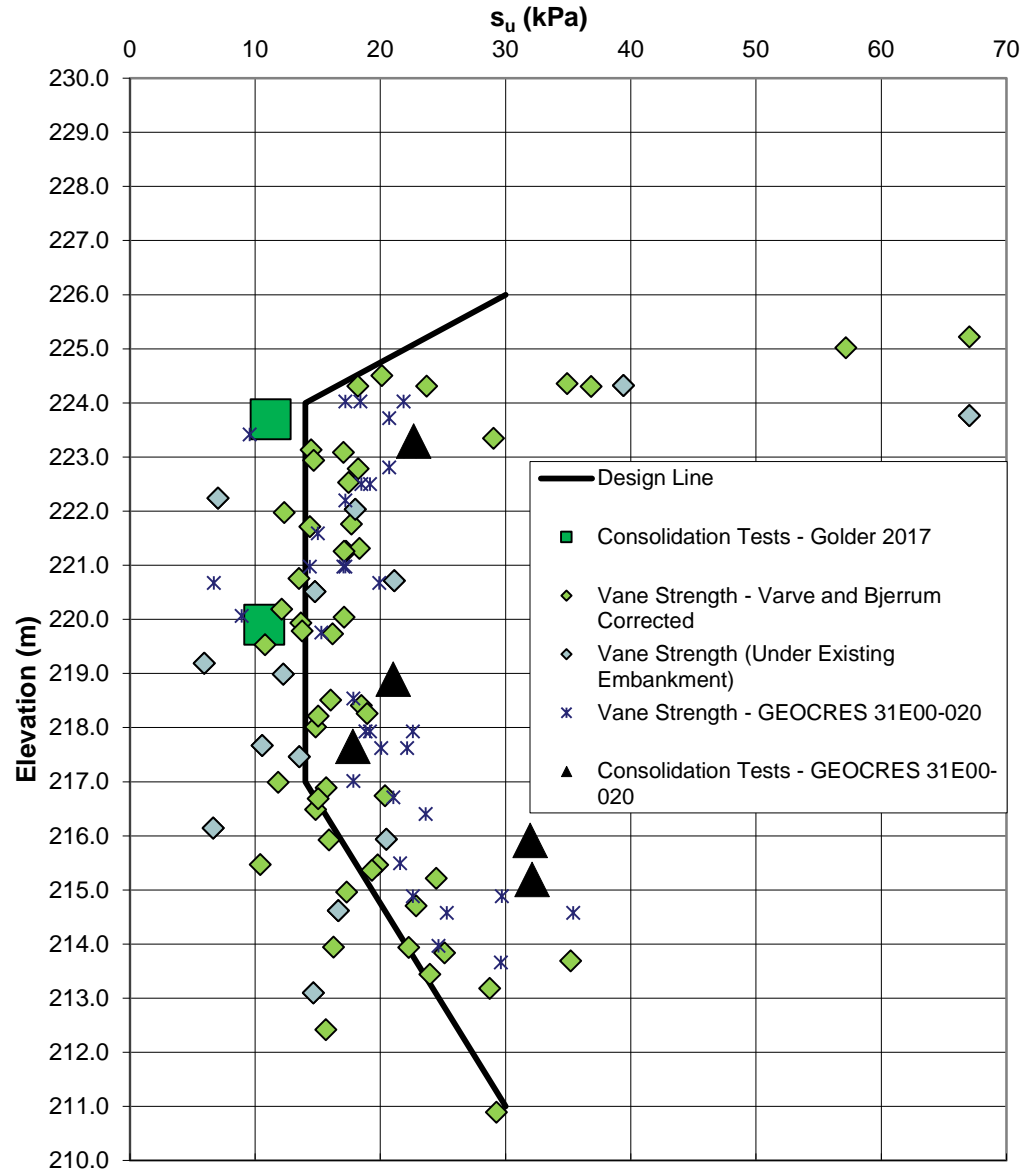






## Shadow River Parameters Undrained Shear Strength

Figure 4

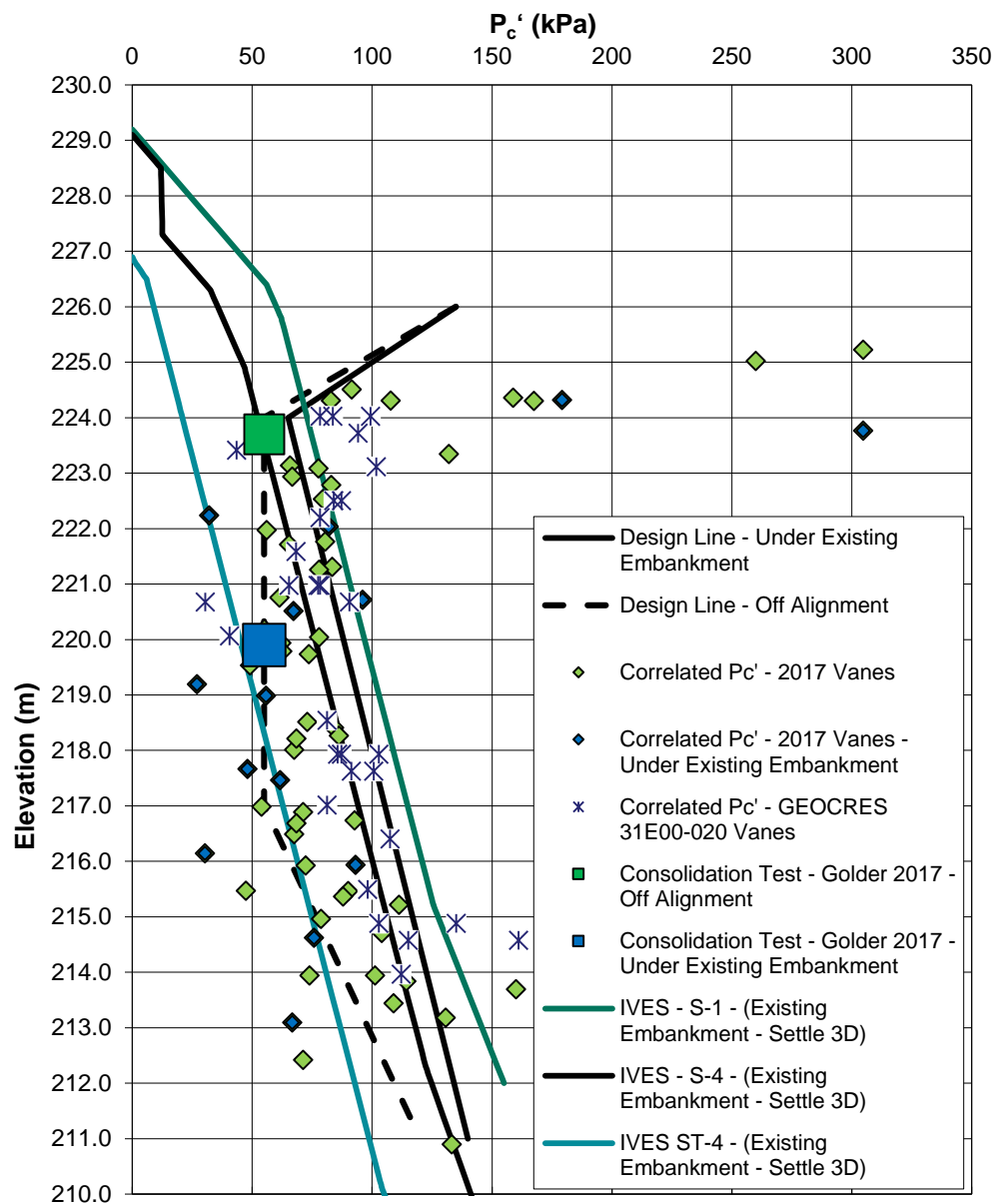






## Shadow River Parameters Preconsolidation Pressure

Figure 5



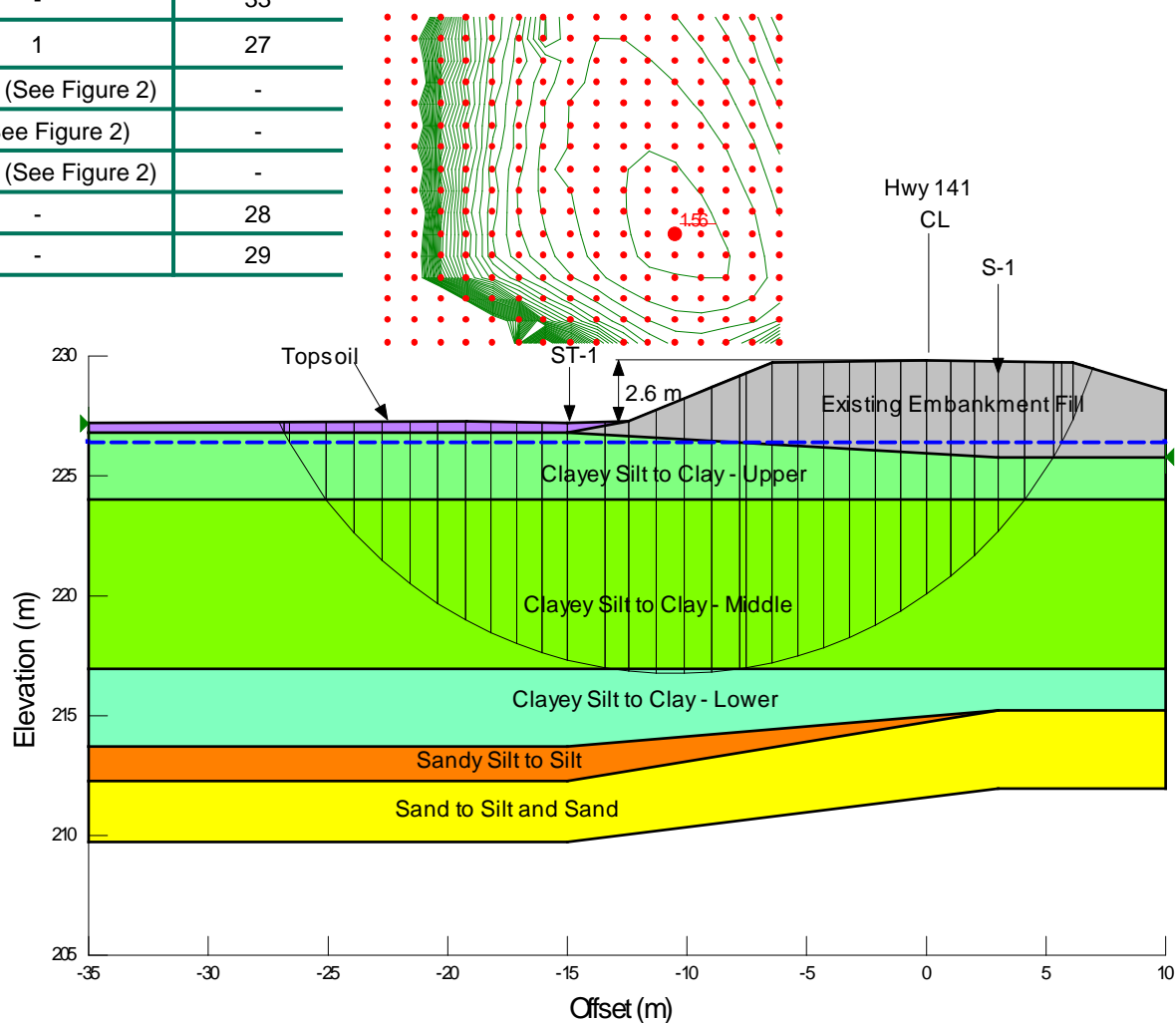




# Stability Analysis West Abutment Existing Conditions – No grade raise

Figure 6

Material Name	Unit Weight (kN/m <sup>3</sup> )	Cohesion (kPa)	Friction Angle (degrees)
Existing Embankment Fill (Granular)	20	-	33
Topsoil	12	1	27
Clayey Silt to Clay – Upper (Above Elev 224 m)	15.8	30 – 14 (See Figure 2)	-
Clayey Silt to Clay – Middle (Elev 224 m – 217 m)	15.8	14 (See Figure 2)	-
Clayey Silt to Clay – Lower (Below Elev 217 m)	15.8	14 – 30 (See Figure 2)	-
Sandy Silt to Silt	18	-	28
Sand and Silt and Sand	19	-	29



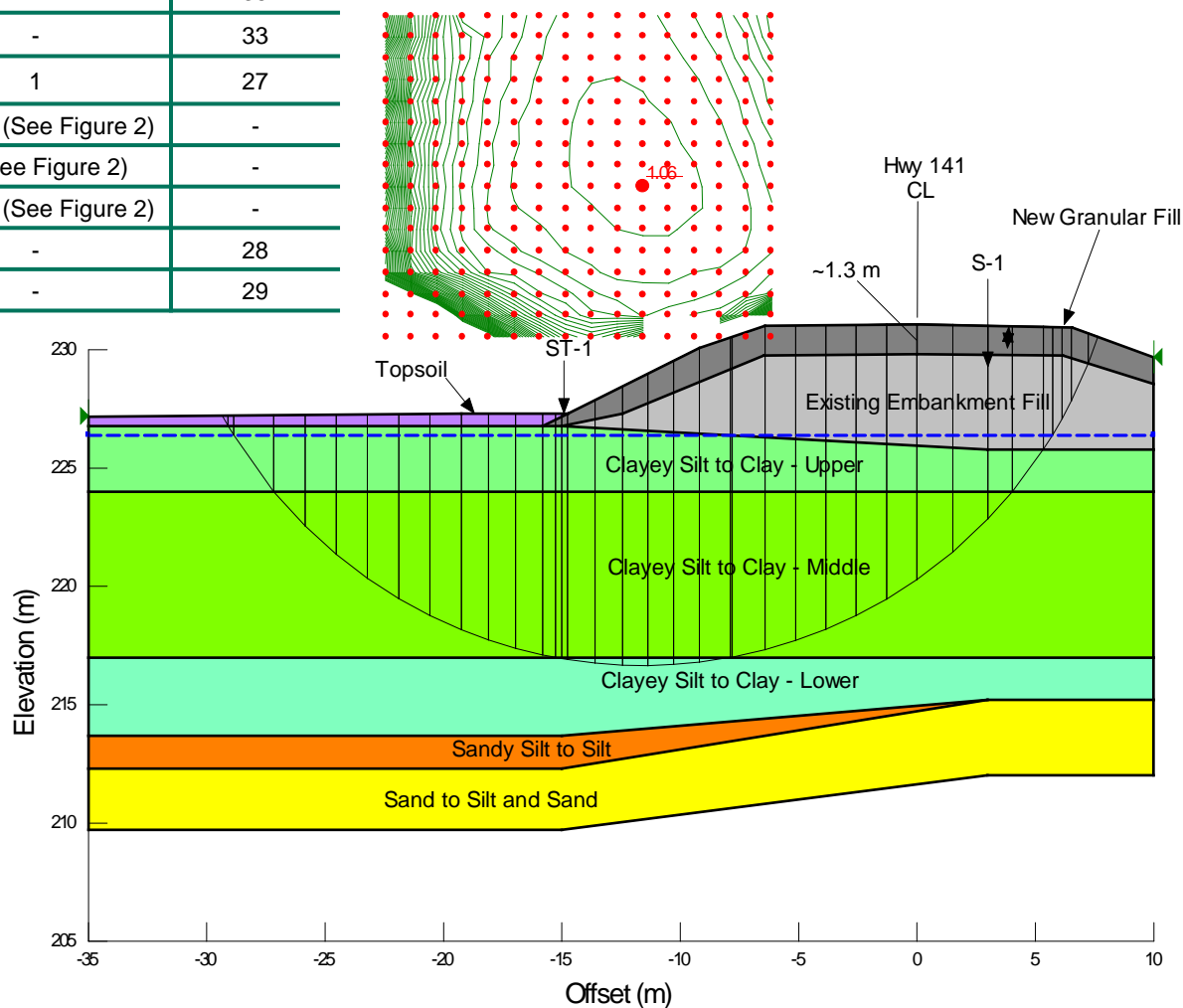




# Stability Analysis West Abutment Existing Alignment ~1.3 m Grade Raise

Figure 7

Material Name	Unit Weight (kN/m <sup>3</sup> )	Cohesion (kPa)	Friction Angle (degrees)
New Embankment Fill (Granular)	21	-	35
Existing Embankment Fill (Granular)	20	-	33
Topsoil	12	1	27
Clayey Silt to Clay – Upper (Above Elev 224 m)	15.8	30 – 14 (See Figure 2)	-
Clayey Silt to Clay – Middle (Elev 224 m – 217 m)	15.8	14 (See Figure 2)	-
Clayey Silt to Clay – Lower (Below Elev 217 m)	15.8	14 – 30 (See Figure 2)	-
Sandy Silt to Silt	18	-	28
Sand to Silt and Sand	19	-	29



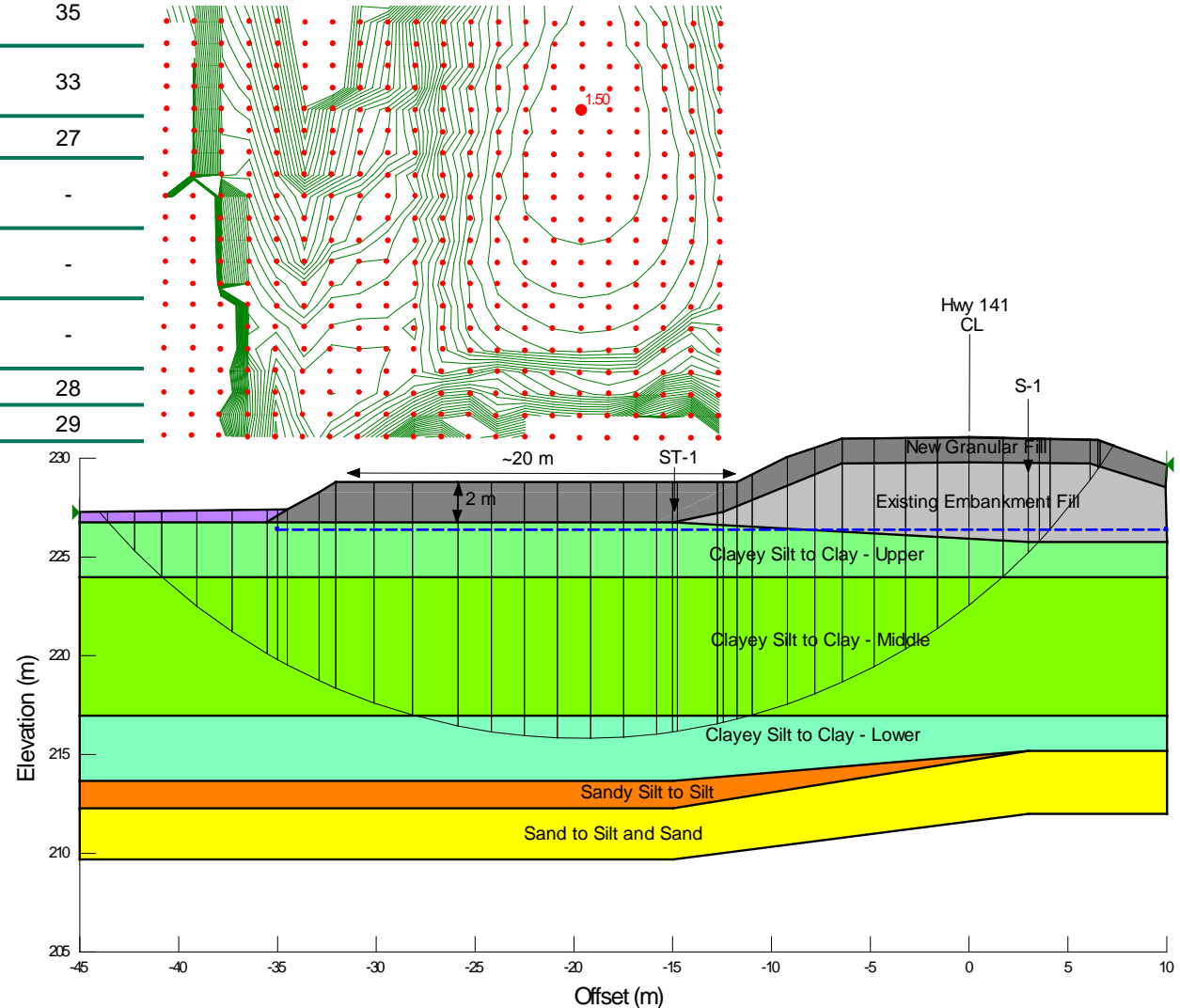




# **Stability Analysis West Abutment Existing Alignment ~1.3 m Grade Raise (Toe Berms Required)**

**Figure 8**

Material Name	Unit Weight (kN/m <sup>3</sup> )	Cohesion (kPa)	Friction Angle (degrees)
New Embankment Fill (Granular)	21	-	35
Existing Embankment Fill (Granular)	20	-	33
Topsoil	12	1	27
Clayey Silt to Clay – Upper (Above Elev 224 m)	15.8	30 – 14 (See Figure 2)	-
Clayey Silt to Clay – Middle (Elev 224 m – 217 m)	15.8	14 (See Figure 2)	-
Clayey Silt to Clay – Lower (Below Elev 217 m)	15.8	14 – 30 (See Figure 2)	-
Sandy Silt to Silt	18	-	28
Sand to Silt and Sand	19	-	29







# Stability Analysis West Abutment Existing Alignment – No Grade Raise – Detour Embankment

Figure 9

Material Name	Unit Weight (kN/m <sup>3</sup> )	Cohesion (kPa)	Friction Angle (degrees)
New Embankment Fill (Granular)	21	-	35
Existing Embankment Fill (Granular)	20	-	33
Topsoil	12	1	27
Clayey Silt to Clay – Upper (Above Elev 224 m)	15.8	30 – 14 (See Figure 2)	-
Clayey Silt to Clay – Middle (Elev 224 m – 217 m)	15.8	14 (See Figure 2)	-
Clayey Silt to Clay – Lower (Below Elev 217 m)	15.8	14 – 30 (See Figure 2)	-
Sandy Silt to Silt	18	-	28
Sand to Silt and Sand	19	-	29

