



FOUNDATION INVESTIGATION AND DESIGN REPORT

*Shadow River Bridge Site No. 44-159, Highway 141
Township of Humphrey, Ontario
Ministry of Transportation, Ontario
GWP 291-96-00, WP 291-96-01*

Agreement 5015-E-0045, Work Order 1

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NSSP
NSSP
OC
OC
NSSP
NTC

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Dewatering Structure Excavation – FOUND003
Obstructions

PART A

FOUNDATION INVESTIGATION REPORT
SHADOW RIVER BRIDGE – HIGHWAY 141
TOWNSHIP OF HUMPHREY, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 291-96-00, WP 291-96-01

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. The key plan showing the general location of this section of Highway 141 and the location of the investigated area are shown on Drawing 1.

Foundation engineering services for this assignment are required under two phases:

- **Phase 1 – Preliminary/Feasibility Investigation:** 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.
- **Phase 2 – Detail Investigation:** Once the preferred replacement strategy was chosen, a detail foundation investigation for the replacement structure and temporary modular detour structure was carried out.

In December 2017, MH and MTO confirmed a preferred replacement strategy consisting of replacing the existing structure on the same alignment and utilizing a temporary modular structure on a detour to the north of the existing structure. This report addresses Phase 2 of the project for the Detail Foundation Investigation phase of the project.

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 to the 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 southerly direction (based on the General Arrangement drawing provided) 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 and is currently under load restriction. The existing grade of the highway at the bridge is at about Elevation 229.7m at the west abutment and 229.4 m at the east abutment. Photographs of the bridge site area are shown on Photographs 1 to 4, following the text of this report.

We understand that prior to construction of the existing bridge, the river crossed the highway further to the east of the existing bridge site in a culvert and the natural river channel meander was located on the north side of the existing bridge, as outlined in previous contract drawings (74-174, W.P. 323-65-02, sheet 19) The ice/water level in Shadow River was measured by Golder at Elevation 226.4 m on March 10, 2017.

Further, this section of Highway 141 was rehabilitated as part of a larger overall rehabilitation of Highway 141 under contract 2002-5102, which included the placement of a 1 m thick layer of polystyrene lightweight fill covered with an approximately 0.15 m concrete cap incorporated into the approach embankments at the Shadow River Bridge as noted in the previous the previous contract package (2002-5102, W.P. 155-90-00, sheet 99). The approximate stations and extents of the lightweight fill/concrete cap are approximately as follows:

- 24+054 to 24+098 (West Abutment approximately 44 m westerly); and
- 24+162 to 24+224 (East Abutment approximately 62 m easterly).

Based on discussions with MH and MTO in December 2017, we understand that the existing bridge is to be replaced on the existing alignment while traffic is to be routed via a proposed detour temporary modular bridge structure located approximately 21 m (centreline to centreline) to the north of the existing structure.

2.1 Previous Investigations

A Foundation Investigation Report (GEOCRE 31E00-020, dated June 26, 1986 by the Department of Highways Ontario – Foundations Section) completed for the site indicates that the native materials 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.

In 2017, Golder completed a Preliminary Foundation Investigation Technical Memorandum (GEOCRE 31E-379, dated July 20, 2017) for the site, and the investigation results have been incorporated into the following sections of this report.

3.0 INVESTIGATION PROCEDURES

The field work for the preliminary subsurface investigation was carried out between March 13 and 24, 2017 during which time eight boreholes (S-1 to S-4 and ST-1 to ST-4) were advanced near some of the foundation elements of the existing structure and along the proposed temporary modular bridge (TMB) alignment on the side closest to the existing embankment/structure at approximately the locations shown on Drawing 1. The fieldwork for the detail subsurface investigation was carried out between December 18 and 21, 2017 and between January 9 and 26, 2018 during which time an additional ten boreholes (S-5 to S-10, and ST-5 to ST-8), four Cone Penetration Tests (CPTs) CPT-1 to CPT-4, and one Dynamic Cone Penetration Test (DCPT) were advanced to supplement the proposed replacement strategy and preliminary investigation information at approximately the locations shown on Drawing 1. The boreholes, CPTs and DCPTs were advanced by portable, track and truck mount CME 55 and track mount CME850 drill rigs supplied and operated by Landcore Drilling Inc. of Chelmsford, Ontario.

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 rigs or with a cathead and standard weight hammer where portable equipment was employed (Boreholes S7 and S8), 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 for the field vane tests due to prevent the field vane from sinking during the generally soft

consistency of the cohesive soils. Select samples of the cohesive soils were obtained using 76 mm outer diameter thin-walled Shelby Tubes (ASTM D1587) for relatively undisturbed samples.

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, organic content, Atterberg limits and grain size distribution were carried out on selected soil samples. Two, one-dimensional consolidation (oedometer) tests were carried during the preliminary phase of the investigation and an additional four, one-dimensional oedometer tests were carried out on select cohesive soil samples for the detail investigation. Unconfined compression strength (UCS) tests were carried out on selected bedrock core samples. The geotechnical laboratory testing was completed according to ASTM and MTO LS standards, as applicable.

Classification of the rock mass quality of the bedrock with respect to the Rock Quality Designation (RQD) and UCS are described based on Table 3.10 and Table 3.5, respectively, of the Canadian Foundation Engineering Manual (CFEM, 2006¹). The degree of weathering of the bedrock samples (i.e., fresh to slightly weathered) and the strength classification of the intact rock mass based on field identification (i.e., strong to very strong) are described in accordance with Table B.3 and Table B.6, respectively, of the International Society for Rock Mechanics (ISRM²) standard classification system.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations and two standpipe piezometers were installed during the preliminary investigation in Boreholes ST-2 and S-3 to permit monitoring of the groundwater level; and the piezometers were decommissioned during the detail foundation investigation. 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 CPTs, an in situ testing technique used for the nearly continuous characterisation of subsurface soils, were advanced to depths below ground surface ranging from about 15.7 m to 25.3 m. The CPT consists of a special probe equipped with electronic sensing elements to continuously measure tip resistance, local side friction on a sleeve and pore water pressure. It is pushed into the ground at a constant rate (ASTM D5778-07 Standard Test Method for Piezocone Penetration) and a nearly continuous stratigraphic profile, together with inferred engineering properties such as shear strengths and stress history can be interpreted from the results.

At this site, the CPT equipment was advanced using the hydraulic system on the drill rig. Profiles of tip resistance, friction and pore water pressure are presented together with interpreted profiles of undrained shear strength and classification index that is used to infer the soil type (i.e., soil stratigraphy) on the Cone Penetration Test sheets included in Appendix A. Further, three dissipation tests were carried out within the cohesive deposit at this site.

The as-drilled borehole locations and ground surface elevations of the preliminary investigation were surveyed using a Trimble GEO7x prior to profile/survey drawings becoming available. For the detail design investigation,

¹ Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.

² International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.

the as-drilled borehole and CPT locations and ground surface elevations were measured and surveyed by members of our technical staff, referenced to the highway centerline and existing bridge abutments and converted into northing/easting coordinates on the plan drawing. The ground surface elevation of the highway centerline was obtained from the profile drawing provided by MH (drawing x1140651Profile.dwg). Based on discussions with MH we understand the profile, was updated as part of the survey component under separate retainer. The MTM NAD83 Zone 10 northing and easting coordinates, ground surface elevations referenced to Geodetic datum, and borehole depths at each borehole location are presented on the Record of Borehole sheets in Appendix A and summarized below.

Approximate Location	Investigation (Detail or Preliminary)	Borehole Number	MTM NAD83 Northing (m) (Latitude)	MTM NAD83 Easting (m) (Longitude)	Ground Surface Elevation (m)	Borehole/ DCTP Depth (m)
Proposed Replacement Structure						
West Approach	Detail	S-5	5014727.7 (45.272373)	291101.7 (-79.674580)	230.0	15.0
West Abutment	Preliminary	S-1	5014721.8 (45.272321)	291120.5 (-79.674135)	229.7 *	17.2
	Detail	S-6	5014727.4 (45.272371)	291118.5 (-79.674365)	229.7	19.9***
	Detail	CPT-2	5014750.4 (45.2725772)	291120.2 (-79.6743906)	229.7	16.0
West Pier	Preliminary	S-2	5014732.3 (45.272415)	291136.6 (-79.674135)	227.2	20.8***
	Detail	S-7	5014716.0 (45.272268)	291139.2 (-79.674101)	227.7	15.8
East Pier	Preliminary	S-3	5014732.7 (45.272419)	291152.6 (-79.673931)	226.9	23.7***
	Detail	S-8	5014716.0 (45.272269)	291156.8 (-79.673877)	227.8	20.3
East Abutment	Preliminary	S-4	5014722.4 (45.272327)	291173.5 (-79.673664)	229.4 *	24.9
	Detail	S-9	5014727.4 (45.272372)	291172.6 (-79.673676)	229.4	28.7***
	Detail	CPT-4	5014727.4 (45.2723719)	291120.5 (-79.6736502)	229.4	25.3
East Approach	Detail	S-10	5014727.4 (45.272372)	291190.3 (-79.673450)	229.5	20.4

Approximate Location	Investigation (Detail or Preliminary)	Borehole Number	MTM NAD83 Northing (m) (Latitude)	MTM NAD83 Easting (m) (Longitude)	Ground Surface Elevation (m)	Borehole/ DCTP Depth (m)
Temporary Detour and Modular Bridge						
West Detour Embankment	Detail	ST-5	5014749.7 (45.272571)	291083.3 (-79.674815)	227.5	11.8/10.8
West Abutment	Preliminary	ST-1 **	5014739.8 (45.272482)	291120.2 (-79.674344)	227.1	20.4***
	Preliminary	ST-2 **	5014739.3 (45.272478)	291130.4 (-79.674214)	226.7	20.9***
	Detail	ST-6	5014750.3 (45.272577)	291117.9 (-79.674374)	227.1	20.2***
	Detail	CPT-1	5014750.4 (45.2725772)	291120.2 (-79.6743449)	227.1	15.7
East Abutment	Preliminary	ST-3 **	5014739.2 (45.272477)	291154.8 (-79.673903)	226.8	23.4***
	Preliminary	ST-4 **	5014739.6 (45.272481)	291164.5 (-79.673779)	226.9	24.2***
	Detail	ST-7	5014750.1 (45.272576)	291167.1 (-79.673747)	227.6	29.0***
	Detail	CPT-3	5014750.1 (45.2725761)	291169.1 (-79.6737212)	227.6	24.3
East Detour Embankment	Detail	ST-8	5014746.6 (45.272545)	291219.9 (-79.673074)	227.5	20.8

*Boreholes S-1 and S-4 ground surface elevations have been revised from the preliminary foundation investigation technical memorandum.

**Boreholes ST-1 to ST-4 were advanced for the originally proposed TMB adjacent to the existing structure.

***Borehole depth includes between 2.6 m and 3.9 m of bedrock core.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional 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)³. The bedrock of this domain consists of gneiss to granitic gneiss with localized areas of metagabbro.

³ Geology of Ontario, 1991. Ontario Geological Society Special Volume 4, Part 2. Ministry of Northern Development and Mines, Ontario.

Based on terrain and physiography mapping by the Ontario Geological Survey⁴, the subsurface soils in the vicinity of the site consist shallow till and rock ridges, along with 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

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced in the vicinity of the Shadow River bridge replacement and proposed detour structure, with the results of the laboratory tests carried out on selected soil and bedrock samples, are presented on the borehole records in Appendix A, and the laboratory test sheets in Appendix B. The results of the in situ field tests (i.e., SPT 'N' values and field vane results) as presented on the borehole records and in Section 4 are uncorrected. The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile and cross-sections on Drawings 1, 2 and 3 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations. Descriptions of the subsurface conditions encountered in the boreholes are provided in the following sub-sections of this report.

Groundwater levels/conditions encountered in the boreholes during and shortly after drilling may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized. Groundwater levels in the area are subject to seasonal fluctuations and to fluctuations after precipitation events and snowmelt.

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 with sand and gravel layers in places, in turn underlain by granitic gneiss bedrock.

4.2.1 Asphalt Pavement/Embankment Fill

Boreholes S-1, S-4, S-5, S-6, S-9 and S-10 were advanced through the existing Highway 141 embankment and encountered asphalt pavement at the ground surface ranging from Elevation 230.0 m to 229.4 m and the thickness ranges from 90 mm to 180 mm.

In Boreholes S-1, S-4, S-5, S-6, S-9 and S-10 embankment fill comprised of interlayers of silty sand to sand to sand and gravel was encountered underlying the asphalt and in Borehole ST-2 from ground surface. Clayey silt fill was encountered in Boreholes ST-7 and ST-8 from ground surface and in Borehole S-6 underlying the granular embankment fill. Additionally, silty sand fill was encountered underlying the sandy clayey silt fill in borehole ST-8. The surface of the fill was encountered between Elevation 229.8 m and 226.4 m and the overall thickness of the embankment fill deposit ranged from 1.7 m to 4.4 m thick. The clayey silt to sandy clayey silt portion of the fill layers range from 0.9m to 1.5 m thick. Cobbles were encountered in Boreholes S-1, S-4, S-5, S-6 and S-9, ranging from 75 mm to 150 mm in diameter and a 420 mm boulder was encountered in Borehole S-10.

In Boreholes S-4, S-5, S-6, S-9 and S-10 that were advanced through the existing Highway 141 approach embankments, immediately beyond the abutments, a concrete slab immediately underlain by styrofoam

⁴ Chapman, L.J. and Putnam, D.F., 1984. The Physiography of Southern Ontario, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.

(lightweight embankment fill) was encountered. The thickness of the concrete slab ranges from 150 mm to 240 mm and the thickness of the styrofoam ranges from 0.8 m to 1.1 m.

The SPT 'N'-values measured within the silty sand to sand and gravel fill range from 3 blows to 69 blows per 0.3 m of penetration, indicating a very loose to very dense relative density, however the deposit generally has a loose to compact relative density. Where the SPT 'N'-values did not penetrate the entire 0.3 m, it is inferred that the higher 'N'-values are indicative of the presence of the cobbles/boulders within the embankment fill. NQ coring techniques were required to advance the boreholes through these materials within the fill deposit.

The organic content of a sample of the clayey silt portion of the deposit taken near ground surface is 2.8 per cent.

The SPT 'N'-values measured within the layers of clayey silt to sandy clayey silt fill range from 0 blows (weight of hammer) to 2 blows per 0.3 m of penetration, indicating a very soft to soft consistency. One in situ field vane tests carried out within this portion of the deposit measured an undrained shear strength 78 kPa, indicating a stiff consistency.

The grain size distributions of nine samples of the sand and gravel to sand to silty sand fill are presented on Figure B1 in Appendix B.

The natural water content measured on samples of the silty sand to sand and gravel fill range from about 1 per cent to 13 per cent. The natural water content measured on one sample of the clayey silt to sandy clayey silt fill is 36 per cent.

4.2.2 Ice

A 0.1 m thick layer of ice was encountered at Boreholes S-7 and S-8 at "ground" surface, corresponding to Elevation 227.7 m and 227.8 m, respectively.

4.2.3 Topsoil/Peat

Boreholes S-2, S-3, S-7, S-8 and ST-1 to ST-6 encountered a 0.1 m to 1.1 m thick layer of wet, dark brown to black sandy/silty topsoil and/or fibrous peat, trace to some gravel, trace to some clay, immediately below the ground surface, which ranges from Elevations 227.7 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 SPT 'N'-values measured within the topsoil/peat deposit range from 4 blows to 12 blows per 0.3 m of penetration, indicating a soft to stiff consistency, however the higher 'N'-values may be attributed to potentially frozen ground conditions, and may not be representative of the relative density of the deposit.

The natural water content measured on a samples of the topsoil/peat deposit range from 25 per cent to 222 per cent.

4.2.4 Sand

A 2.2 m thick deposit of wet, brown to grey, sand was encountered in Borehole ST-8 at Elevation 225.3 m. Approximately 0.9 m of heave was encountered within the hollow stem augers at 3.0 m depth within this deposit.

The SPT 'N'-values measured within the sand deposit range from 0 (weight of hammer) blows to 2 blows per 0.3 m of penetration, indicating a very loose compactness condition.

The grain size distribution of one sample of the sand deposit is presented on Figure B2 in Appendix B.

The natural water content measured on a sample of the sand deposit is 20 per cent.

4.2.5 Silt to Sandy Silt

A deposit of moist to wet, dark brown to brown to dark grey, silt to sandy silt, trace to some organics, trace to some gravel was encountered underlying the embankment fill in Boreholes S-5, S-6, S-9 and S-10 and in Borehole ST-7 adjacent to the detour alignment. The surface of the deposit was encountered between Elevation 226.5 m and 224.9 m and the thickness of the deposit ranged from 0.6 m to 2.4 m. Pieces of wood 0.3 m and 0.1 m thick were encountered within the silt portion of the deposit in Boreholes S-9 and S-10.

Atterberg limits tests on four selected samples of this deposit yielded non-plastic test results.

The natural moisture content measured on samples of the silt to sandy silt, ranges from 34 per cent to 131 per cent.

Organic content tests were conducted on three samples of this deposit and the organic content ranges from 1.3 per cent to 3.4 per cent.

4.2.6 Clayey Silt to Clay

A cohesive deposit of moist to wet, brown to grey, clayey silt to clay was encountered below the embankment fill, topsoil or silt to sandy silt deposit in all boreholes. The deposit is between 7.8 m and 16.7 m thick as encountered in the boreholes, and the surface of this deposit was encountered between Elevations 227.4 m and 223.0 m. The upper and lower zones of the deposit consists in some places of clayey silt to silt of slight plasticity containing trace to some sand or clayey silt to silt, while the majority of the deposit generally consists of silty clay to clay, containing trace sand. Varves were noted in the deposit in all boreholes but are not present consistently throughout the depth/thickness of the deposit.

The measured SPT "N"-values within the clayey silt to clay deposit range from 0 blows (weight of rods/hammer) to 9 blows per 0.3 m of penetration. In-situ field vane tests carried out within the silty clay to clay portion of the deposit measured undrained shear strengths ranging from about 8 kPa to 65 kPa and 30 kPa to greater than 100 kPa in the clayey silt to silt zones of the deposit. The sensitivity of the cohesive deposit ranged from 1 to 8, and was generally about 3. The field vane test results, together with interpreted undrained shear strength from the CPT plots of S_u (kPa) versus depth, indicate that the silty clay to clay portion of the deposit has a very soft to stiff consistency; however generally a soft to firm consistency, whereas the clayey silt to silt zone of the deposit has a firm to stiff consistency.

Atterberg limits tests carried out on sixty-eight selected samples of this deposit yielded liquid limits ranging between about 21 and 69 per cent, plastic limits between about 14 and 30 per cent and plasticity indices between about 6 and 43 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figures B3.1 to B3.3 in Appendix B and indicate the materials generally consist of clayey silt of low plasticity to clay of high plasticity. The test results of two of these samples from near the bottom of the deposit indicated that the material consisted of clayey silt to silt of slight plasticity.

The grain size distribution of twenty samples of the clayey silt to clay deposit are presented on Figures B4.1 and B4.2 in Appendix B.

The natural moisture content measured on samples of the cohesive deposit ranges from 21 to 96 per cent.

The results of the laboratory consolidation (oedometer) tests carried out on six specimens of the silty clay to clay deposit, obtained from Shelby tube samples in Boreholes S-1, S-6 and S-9 from under the existing embankment, and from Boreholes ST-4, ST-5 and ST-7 from the general alignment, are shown on Figures B5 to B10. 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 ranging from 15.0 kN/m² to 15.9 kN/m² and a specific gravity ranging from 2.72 to 2.79 were measured on the consolidation test samples. The test results are summarized below.

Borehole / Sample No.	Sample Depth / Elev.	$\sigma_{vo'}$ (kPa)	$\sigma_{p'}$ (kPa)	$\sigma_{p'} - \sigma_{vo'}$ (kPa)	OCR	e_o	C_c	C_r	C_v (cm ² /s)
S-1 / Sample 10	9.3 m / 220.4 m	95	55	-40	0.6	1.9	0.9	0.09	4.0×10^{-4} *
ST-4 / Sample 4	3.2 m / 223.7 m	45	55	10	1.2	2.0	1.0	0.1	1.0×10^{-3} *
S-6 / Sample 9	10.4 m / 219.7	100	85	-15	0.9	2.2	1.1	0.1	2.4×10^{-3} **
S-9 / Sample 7	7.3 m / 222.4	85	85	0	1.0	2.1	1.2	0.1	3.0×10^{-3} **
ST-6 / Sample 5	4.9 m / 222.7	45	65	20	1.4	2.2	1.3	0.09	1.1×10^{-3} **
ST-7 / Sample 7	8.7 m / 218.9	50	85	35	1.7	2.2	1.2	0.1	4.3×10^{-3} **

*For the normally consolidated stress range of approximately 60 to 140 kPa

** For the normally consolidated stress range of approximately 80 kPa to 155 kPa

where:

$\sigma_{vo'}$ is the effective overburden stress

$\sigma_{p'}$ is the preconsolidation stress

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 cm²/s

The range of the coefficient of consolidation in the horizontal direction (c_h) obtained from a total of three CPT pore pressure dissipation tests is summarized below.

Ch CPT-Field (cm ² /s)		
Upperbound	Lowerbound	Average
1.19×10^{-3}	9.49×10^{-4}	2.1×10^{-3}

4.2.7 Silt to Sandy Silt

A deposit of non-cohesive wet, grey, silt to sandy silt was encountered below the clayey silt to clay deposit in Boreholes S-2 to S-7, S-10, ST-1, ST-2 and ST-8. The deposit ranges from 0.8 m to 5.9 m thick and the surface of this deposit was encountered between Elevation 218.1 m and 211.7 m.

The SPT “N”-values measured within the sandy silt to silt deposit range from 1 to 18 blows per 0.3 m of penetration, indicating a very loose to compact compactness condition.

Atterberg limits tests on four selected samples within the deposit yielded non-plastic results.

The grain size distribution of seven samples of the sandy silt to silt deposit are presented on Figure B11 in Appendix B.

The natural water contents measured on samples of the sandy silt to silt deposit range from about 14 per cent to 27 per cent.

4.2.8 Silt and Sand to Gravelly Silty Sand to Sand

A deposit of non-cohesive, wet, grey, silt and sand to gravelly silty sand to sand, trace to some gravel, trace clay was encountered below the clayey silt to clay deposit and/or the silt to sandy silt deposit in all boreholes except Boreholes S-2 and S-7. A cobble and a 0.3 m size boulder were encountered within silty sand to sand portions of the deposit in Borehole S-6 and ST-5 near the bottom of the deposit (about Elevations 213.2 m and 216.0 m, respectively). The deposit ranges from 1.7 m to 11.1 and the surface of this deposit was encountered between Elevations 215.7 m and 206.8 m.

The SPT “N”-values measured within the silt and sand to sand deposit range from 1 to 38 blows per 0.3 m of penetration, indicating a very loose to dense relative density, however generally the deposit has a loose to compact compactness condition.

The grain size distributions of fourteen samples of the sandy silt to silt deposit are presented on Figures B12.1 and B12.2 in Appendix B.

The natural water contents measured on samples of the sand to silt and sand deposit range from about 10 per cent to 29 per cent.

4.2.9 Sand and Gravel

A deposit of wet, grey, sand and gravel was encountered at Elevation 215.6 m in Borehole S-5 below the silt and sand to gravelly silty sand to sand deposit and was measured to be 0.6 m in thickness. An SPT 'N'-value of 34 blows per 0.3 m of penetration was measured in this deposit indicating a dense compactness condition.

4.2.10 Bedrock/Refusal

The depth to the confirmed/inferred bedrock surface and bedrock surface elevations are presented below.

Borehole No.	Investigation (Preliminary or Detail)	Nearest Foundation Element	Inferred Depth to Bedrock (below ground surface) (m)	Inferred Bedrock Surface Elevation (m)	Refusal Type or Core Length (m)
S-5	Detail	-	15.0	215.0	Split-spoon refusal
S-1	Preliminary	West Abutment	17.2	212.5	Casing and split-spoon refusal
S-6	Detail		16.9	212.8	3.0
S-2	Preliminary	West Pier	17.7	209.5	3.1
S-7	Detail		15.8	211.9	Split-spoon refusal
S-3	Preliminary	East Pier	20.6	206.3	3.1
S-8	Detail		20.3	207.5	DCPT refusal
S-4	Preliminary	East Abutment	24.9	204.5	Casing and split-spoon refusal
S-9	Detail		25.5	203.9	3.2
S-10	Detail	-			Not Reached
ST-5	Detail	-	11.5	216.0	Casing seated 0.3 m into bedrock or boulder
ST-1	Preliminary	TMB West Abutment	17.4	209.7	3.0
ST-2	Preliminary		17.5	209.2	3.4
ST-6	Detail		16.6	210.5	3.6
ST-3	Preliminary	TMB East Abutment	20.1	206.7	3.3
ST-4	Preliminary		21.5	205.4	2.7

Borehole No.	Investigation (Preliminary or Detail)	Nearest Foundation Element	Inferred Depth to Bedrock (below ground surface) (m)	Inferred Bedrock Surface Elevation (m)	Refusal Type or Core Length (m)
ST-7	Detail		25.1	202.5	3.9
ST-8	Detail	-	20.8	206.7	Split-spoon refusal

The retrieved bedrock core is described as fine to medium grained, black/pink to pinkish grey, foliated, fresh to slightly weathered, strong to very strong, biotite rich, granitic gneiss as presented in the drillhole records in Appendix A. Photographs of the bedrock core samples are shown on Figures B13.1 and B13.2 and the summary of rock core unconfined compression test data is provided on Figures B14.1 to 14.4 in Appendix B. The bedrock properties, as encountered in the cored boreholes, are summarized below.

Borehole No.	Total Core Recovery	Rock Quality Designation	Quality Classification Table 3.10 of CFEM 2006 ⁵	Uniaxial Compressive Strength (MPa)	Strength Classification Table 3.5 of CFEM 2006 ⁵
S-2	93 – 100 %	85 – 100 %	Good to Excellent	99	Strong (R4)
S-3	100 %	45 – 100 %	Poor to Excellent	159	Very Strong (R5)
S-6	97 - 98%	97 - 98 %	Excellent	79	Strong (R4)
S-9	100 %	100 %	Excellent	68	Strong (R4)
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)

⁵ Canadian Geotechnical Society 2006. Canadian Foundation Engineering Manual, 4th Edition. BiTech Publications.

Borehole No.	Total Core Recovery	Rock Quality Designation	Quality Classification Table 3.10 of CFEM 2006 ⁵	Uniaxial Compressive Strength (MPa)	Strength Classification Table 3.5 of CFEM 2006 ⁵
ST-6	96 - 100 %	80 – 100 %	Good to Excellent	114	Very Strong (R5)
ST-7	100 %	60 – 100 %	Fair to Excellent	66	Strong (R4)

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

4.2.11 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 S-3 sealed in the sand deposit below the silt deposit underlying the clayey silt to silt/silty clay to clay deposits and Borehole ST-2 sealed within the clayey silt to clay deposit. Groundwater levels encountered in the boreholes during and shortly after drilling are generally 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 sheets. 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 (screen sealed within the sand deposit)	April 16, 2017 December 18, 2017	-0.4* 0.1	227.3 226.8
ST-2	Piezometer (screen sealed within clayey silt to clay deposit)	April 16, 2017 December 18, 2017 January 11, 2018	0.4 0.9 0.7	226.3 225.8 226.0

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

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 and subsequently at Elevation 226.5 m on January 18, 2018. The water level of Shadow River from GEOCRE 31E00-020 measured in December 1967 was noted to be approximately Elevation 226.7 m (743.66 ft). Based on the drawings provided by MH we understand that the design high water level (50 year) to be approximately Elevation 228.0 m.

4.2.12 Analytical Testing of Soil Samples

A select soil sample from each of the Boreholes S-1, S-4, ST-2 and St-3 was obtained during the field investigation using appropriate sampling protocols and submitted to a specialist analytical laboratory under chain of custody procedures for testing for a suite of inorganic parameters, the results of which are presented on Table B1 in Appendix B.

5.0 CLOSURE

The field drilling program was supervised by Mr. Mat Riopelle, Mr. Shane Albert and Mr. Mike Arthur. The Foundation Investigation Report was prepared by Mr. Adam Core, P.Eng., and the technical aspects were reviewed by Ms. Sarah E. M. Poot, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M.A. Costa, P.Eng., Designated MTO Foundations Contact and Senior Consultant for Golder, conducted an independent quality control review of this report.

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PART B

**FOUNDATION DESIGN REPORT
SHADOW RIVER BRIDGE – HIGHWAY 141
TOWNSHIP OF HUMPHREY, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 291-96-00, WP 291-96-01**

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed replacement of the Shadow River Bridge (Site No. 44-159) located on Highway 141, Northwest of Rosseau, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes and CPTs advanced during the subsurface investigation at this site. The interpretation of the subsurface information and recommendations presented in this Foundation Design Report (Part B) are intended to provide MTO's designers with sufficient information to assess the feasible foundation alternatives and to design the proposed bridge structure foundations and approach embankments, and associated works for a detour alignment/structure on the north side of the existing highway.

The discussion and recommendations contained in this Foundation Design Report (Part B) shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project. The contractor must make their own interpretation based on the factual data in the Foundation Investigation Report (Part A), as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Golder Associates Ltd. (Golder) has been retained by AECOM Canada Ltd. (AECOM) on behalf of MTO to provide recommendations on the foundation aspects for the detail design of the replacement of the Shadow River Bridge. The highway and structural design aspects of this project are being carried out under separate contract between Morrison Hershfield (MH) and MTO.

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 existing bridge is supported on 8 rows of timber pile bents and it is not known if the timber piles extend fully to the bedrock surface. The existing bridge will be replaced with a 46.7 m long, three-span pre-stressed box girder bridge on the existing alignment as depicted on the General Arrangement (GA) drawing provided by MH on November 20, 2017. The proposed highway grade at the new structure will be about 700 mm higher than the existing grade. To facilitate the replacement of the existing bridge, traffic is being staged onto a detour alignment and a 51.8 m long, single-lane Temporary Modular Bridge (TMB) located 21.3 m north of the existing bridge (centreline to centreline). The proposed detour roadway grade at the TMB abutments will be about 2.1 m (west approach) and 1.8 m (east approach) above the existing ground surface along the detour centreline. Beyond the approach embankments of the TMB structure, the detour embankments will be 2.9 m and 2.7 m high above the existing ground surface approximately 40 m west and 60 m east of the TMB abutments, respectively. The overall length of the detour is approximately 340 m, which traverses a low-lying swampy area that extends approximately 50 m west and 100 m east of the TMB.

6.2 Consequence and Site Understanding Classification

The replacement bridge is being designed in accordance with the current Canadian Highway Bridge Design Code CAN/CSA-S6-14 (CHBDC 2014). In accordance with Section 6.5 of CHBDC (2014) and its Commentary, the proposed bridge and its foundation system is considered to be classified as having a "typical consequence level" associated with exceeding limits states design. The degree of understanding, based on the scope of the current foundation investigation and design, is considered 'typical' as described in Clause 6.5.3.2 of CHBDC (2014). The

appropriate corresponding Ultimate Limit States (ULS) and Serviceability Limit States (SLS) consequence factors, Ψ , and geotechnical resistance factors at ULS (ϕ_{gu}) and SLS (ϕ_{gs}), respectively, from Tables 6.1 and 6.2 of the CHBDC have been used for design in this report.

6.3 Foundations

Based on the proposed bridge geometry and the subsurface conditions at this site, deep foundation options are being considered for support of the bridge abutments and piers for both the replacement bridge and the TMB detour bridge. Shallow foundations are not feasible due to the very low geotechnical axial resistances that would be available as a result of the soft clayey foundation soils. A summary of the advantages and disadvantages associated with each foundation option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks/consequences, and relative costs is provided in Table 1 following the text of this report.

- **Driven Steel H-piles:** Driven steel H-piles are feasible for support of the abutments (and any associated wingwalls) and piers and would permit integral abutment design. The abutments and piers would be supported on end-bearing piles driven to granitic gneiss bedrock.
- **Micropiles:** Drilled and grouted micropiles socketed into the bedrock could be considered as a deep foundation option for support of the abutments and piers. 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.
- **Steel Pipe Piles:** Deep foundations comprised of driven steel pipe piles are not considered appropriate for this site due to the higher risk of pile damage associated with seating the piles on the potential sloping bedrock and also due to the higher disturbance to the very soft and sensitive clays caused by the larger displacement pipe piles. Further pipe piles are typically not suitable for integral abutment design.
- **Socketed H-Piles:** If additional lateral fixity is required at the toe of the piles, consideration could be given to installing the H-piles by advancing a temporary steel casing and excavating a socket into the bedrock in which a steel H-pile is installed and grouted. This option typically requires removal of the casing during concreting/grouting, which at this site, is not recommended due to the high risk of ground disturbance to the very soft and sensitive clays during removal and potential for impact on the existing timber piles.
- **Drilled steel casings (small diameter):** Drilled steel casings involve installation of a 305 mm to 750 mm diameter permanent steel casing socketed into the bedrock using wash boring methods (through the overburden) and rotary percussive drilling (through the bedrock), 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. Small diameter drilled 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):** Large diameter drilled shafts (caissons) socketed 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 advancing large diameter drill holes through wet/very soft 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 disturbance to the ground (and potentially the adjacent existing structure) during installation.

Regardless of the deep (pile) foundation option selected, additional mitigation measures will have to be incorporated into the design, including such as a granular filter blanket, and/or possibly grouting will be required to minimize the risk of soil migration up along the pile shaft (during/following pile driving) as a result of high/artesian groundwater pressures that are present within the confined sandy silt aquifer overlying the bedrock. In addition, a minimum separation distance between the detour structure and the existing structure will be required to minimize the risk of settlements caused by detour embankment construction affecting the approach embankments and pile foundations of the existing structure. Excavation/dewatering will be required for pile cap construction. It is noted that large/heavy construction equipment may not be easily supported on the soft clayey foundation soils and may induce unbalanced loading on the existing bridge and any temporary excavations/works. Downdrag loads may also need to be considered in the design of the pile foundations depending on the replacement strategy chosen.

The following sections provide detailed foundation recommendations for the replacement bridge and TMB detour bridge. Steel H-piles driven to refusal on bedrock have been identified as the preferred foundation alternative for this site and as such, the other foundation options described above are not covered further in this report.

6.3.1 Deep Foundations – Steel H-Piles

It is recommended that the replacement bridge (abutments and piers) and the TMB detour bridge (abutments) be supported on steel HP310X110 piles driven to bedrock. The following sections provide details for geotechnical axial resistances, downdrag loads, set criteria and pile driving notes, pile interference, resistance to lateral loads and frost protection.

6.3.2 Design Tip Elevation

The estimated pile tip elevations and the estimated pile lengths have been calculated based on the underside of pile caps as shown on the GA drawing. There should be a provision made in the Contract for dealing with varying pile lengths due to the variability in depth to the bedrock surface and the lengths given below should be considered minimum lengths. When evaluating the quantities (i.e. total length) of piling, it is recommended that the lowest elevation/longest pile length be used for contract purposes (assuming it is easier to cut-off excess pile length than it is to splice/add-on additional pile length).

Bridge	Foundation Element (Relevant Boreholes South Side, North Side)	Proposed Underside of Pile Cap (m)	Estimated Pile Tip Elevation (m)	Estimated Pile Design Length (m)
Replacement Bridge	West Abutment (S-1 and S-6)	226.2	212.5 ¹ - 212.8 ²	13.7 – 13.4
	West Pier (S-7 and S-2)	226.2	211.9 ¹ – 209.5 ²	14.3 - 16.7
	East Pier (S-8 and S-3)	226.2	207.5 ¹ – 206.3 ²	18.7 - 19.9
	East Abutment (S-4 and S-9)	226.2	204.5 ¹ – 203.9 ²	21.7 - 22.3

Bridge	Foundation Element (Relevant Boreholes South Side, North Side)	Proposed Underside of Pile Cap (m)	Estimated Pile Tip Elevation (m)	Estimated Pile Design Length (m)
TMB	West Abutment (ST-1 and ST-6)	228.0 ³	209.7 ² – 210.5 ²	18.3 – 17.5
	East Abutment (ST-4 and ST-7)	228.0 ³	205.4 ² – 202.5 ²	22.6 - 25.5

1. Bedrock elevation inferred from refusal conditions.
2. Bedrock elevation confirmed by bedrock coring.
3. The underside of pile cap elevation for the TMB is to be determined by the proprietary modular bridge designer, as such this is only for estimation purposes.

6.3.3 Geotechnical Axial Resistance

For HP310x110 steel H-piles driven to refusal on the granite gneiss bedrock, a factored ultimate geotechnical resistance of 2,000 kN per pile may be used for design. The factored serviceability geotechnical resistance for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored ultimate geotechnical resistance; as such, the factored ultimate geotechnical resistance will govern for this foundation type.

Artesian conditions were encountered in the piezometer installed in Borehole S-3 in the confined aquifer below the cohesive deposit and the water level was recorded at Elevation 227.3 m (corresponding to 0.4 m above the ground surface or about 0.9 m above the March 2017 river level). A sand blanket filter (see Section 6.6.2) should be constructed at the underside of all of the pile caps to filter any soil fines that may be carried upwards to the surface due to the artesian groundwater and therefore reduce the risk of ground loss and settlement. Since the artesian pressure is low at this site, grouting of piles is not considered to be necessary.

We understand based on discussions with MH that corrugated steel pipes (CSPs) are not required as part of the integral abutment design at this site (see Section 6.3.7).

6.3.4 Pile Installation and Driving Notes

Pile installation should be carried out in accordance with OPSS 903 (Deep Foundations). For end-bearing piles, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on or within the bedrock, and then to gradually increase the energy over a series of blows to seat the pile in the bedrock.

The piles should be fitted with rock points such as Titus Injector or Oslo Point as per Ontario Provincial Standard Drawing OPSD 3000.201 (HP310 Oslo Point), or equivalent to assist in seating the piles on the sloping bedrock. An example NSSP to be included in the contract documents is included in Appendix C.

The pile driving note that should be added to the contract drawings is Note 2 in Clause 3.3.3 of the Structural Manual (MTO, 2014) as follows:

- “Piles to be driven to bedrock”

The piles should be re-tapped to confirm they are seated on the bedrock.

6.3.5 Downdrag Loads

Settlement of the underlying clayey silt to clay deposit will occur either as a result of the grade raise on the existing embankments and/or due to construction of the new detour embankments, unless mitigation measures are carried out. If mitigation is not carried out, downdrag loads (negative skin friction) will be induced on the end-bearing piles supporting the abutments (and possibly the piers if filling is required for access road construction) due to settlement of the surrounding clayey silt to clay and friction/adhesion along the piles. The structural design of the foundation elements (using an HP310X110 pile) should be based on the following estimated unfactored downdrag loads:

Foundation Locations	HP 310 x 110
West Abutment	200 kN
East Abutment	225 kN
TMB West Abutment	240 kN
TMB East Abutment	200kN

Note: if filling is required adjacent to the pier areas of the replacement bridge, dragloads similar to those noted above for the East Abutment should be included in the structural 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 CHBDC (2014) and its Commentary for factored ultimate and serviceability conditions.

6.3.6 Interference between New and Existing Piles

The GA drawing shows the proposed abutment H-piles in close proximity to the existing timber piles, and in some cases, even overlapping. As the existing timber piles will be cut-off after the existing bridge is removed, they will not be required to carry any of the new bridge loading. However, they may physically conflict with the proposed new pile locations and may also impact the lateral resistance of the new piles (in particular if the existing piles are removed if/where conflicts occur). Provision should be made in the design/contract for field adjustment of pile locations to avoid conflicts between the new and existing piles. As a minimum, the new and existing piles should be separated by at least 1 pile diameter to reduce the potential for adverse effects (conflicts) during driving. As the potential effects of the old timber piles being left in place and in close contact with new H-piles is not known and thus a minimum separation distance cannot be provided, consideration should be given to locating the new bridge abutments several metres behind the existing abutments to avoid pile conflicts.

6.3.7 Resistance to Lateral Loads

The design of steel pile foundations subjected to lateral loads should take into account such factors as the batter of the pile (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical

resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case. Lateral loading could be resisted fully or partially by the use of battered piles.

It is understood that an integral abutment foundation design is being considered for the replacement bridge. Typically, the integral abutment design should include the installation of 3 m long corrugated steel pipe (CSP) liners, with the annular space between the pile and the liner backfilled with uniformly graded, loose sand so that the upper portion of the H-piles will be free to flex and move laterally within the limits of the CSP. However, where soft to firm cohesive soils are present within the upper 3 m below the pile cap, it may be possible to achieve sufficient deflection without the need for CSP pipes. The designer should use the values provided below to determine if CSPs are required. If required, the passive lateral resistance over the length of the pile within the CSP liner should be based on the resistance provided by loose sand, otherwise the passive lateral resistance of the pile will be provided by the native soils.

Where ground conditions are generally competent and the lateral loads on piles are relatively small such that the maximum lateral pile deflections will be relatively small, the resistance to lateral loading in front of a single pile can be estimated using subgrade reaction theory as outlined below. However, it should be noted that the response of a pile to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are only appropriate where the maximum pile deflections are less than 1 per cent of the pile diameter, where the loading is static (no cycling) and where the pile material is linear (CFEM, 2006). Where these conditions are not met, the non-linear lateral behavior of the soil should be considered by the use of P-y curves.

The factored serviceability geotechnical response of the soil in front of the piles under lateral loading at this site may be calculated using subgrade reaction theory suggested in the CHBDC (2014) Commentary (Section C6.11.2.2), where the coefficient of horizontal subgrade reaction, k_h , (kPa/m) is based on the equation given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (CFEM 1992).

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where:

- n_h = constant of horizontal subgrade reaction (kPa/m), as given below;
- z = depth below underside of pile cap for loose sand in CSP (m)
- z = depth below ground surface for all other soils (m)
- B = pile diameter or width (m)

For cohesive soils:

$$k_h = \frac{67s_u}{B}$$

where:

- s_u = undrained shear strength of the soil (kPa), as given below.

The following values of n_h and s_u (Terzaghi, 1955) may be incorporated into the calculations of the coefficient of horizontal subgrade reaction k_h for structural analysis for a single vertical pile. The ranges in values reflect the variability in the subsurface conditions, the soil properties and the approximate nature of the analysis and the non-linear nature of the soil behaviour (such that k_h is a function of deflection).

Foundation Element (Relevant Boreholes)	Soil Unit	Elevation (m)	n_h (kPa/m)	S_u (kPa)
All Foundation Elements	Sand Filter Blanket	226.2 – 225.7	1,300	-
West Abutment (S-1 and S-6)	Clayey Silt to Clay	225.7 (underside of filter blanket) – 224	-	20 to 13
		224.0 – 220.0	-	13
		220.0 – 215.7	-	13 to 30
	Sandy Silt to Silt	215.7 – 212.5	4,400	-
West Pier (S-2 and S-7)	Clayey Silt to Clay	225.7 (underside of filter blanket) – 224.0	-	20 to 13
		224.0 – 220.0	-	13
		220.0 – 212.4	-	13 to 30
	Sandy Silt to Silt	212.4 – 209.5	1,300	-
East Pier (S-3 and S-8)	Clayey Silt to Silt	225.7 (underside of filter blanket) – 224.0	-	20 to 13
		224.0 – 220.0	-	13
		220.0 – 211.7	-	13 to 30
	Sandy Silt to Silt	211.7 – 209.5	1,300	-
	Silt and Sand to Sand	209.5 – 206.3	4,400	-
East Abutment (S-4 and S-9)	Silt, trace to some organics	225.7 – 223.8	1,300	-
	Clayey Silt to Clay	223.8 – 220.0	-	13
		220.0 – 212.4	-	13 - 30
	Silt and Sand to Silty Sand	212.4 – 203.9	4,400	-

Foundation Element (Relevant Boreholes)	Soil Unit	Elevation (m)	n_h (kPa/m)	S_u (kPa)
TMB – West Abutment (ST-6)	Clayey Silt to Clay	226.9 - 224.0	-	20 to 13
		224.0 – 220.0	-	13
		220.0 – 214.4	-	13 to 30
	Silt and Sand to Silty Sand	214.4 – 210.5	4,400	-
TMB – East Abutment (ST-7)	Silt	226.1 - 223.8	1,300	-
	Clayey Silt to Clay	223.8 – 220.0	-	13
		220.0 – 213.6	-	13 to 30
	Silt and Sand to Sand	213.6 – 202.5	1,300	-

For a single vertical HP310 x 110 pile advanced at the foundation elements noted below, to the design tip elevations provided in Section 6.3.2, the estimated factored lateral resistance at ULS and the factored lateral resistance at SLS (for 10 mm of horizontal deflection at the pile cap) are presented below. These values are based on analysis carried out using the commercially available program LPILE Plus (Version 2016), produced by Ensoft Inc.

Foundation Element	Lateral Resistance/Reaction (kN)	
	ULS (Factored)	SLS (for 10 mm of deflection) (Factored)
West Abutment	50	20
West Pier	60	20
East Pier	60	20
East Abutment	80	30
TMB West Abutment	60	20
TMB East Abutment	60	20

The lateral resistances given above are based on an assumed fixed-head condition of 1,150 kN unfactored axial load applied at the top of the pile (as noted on GA drawing). The lateral resistance should be reviewed if greater vertical loads are anticipated.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters between rows of driven steel H-Piles. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM7.2, 1986) as follows:

Pile Spacing in Direction of Loading (D = Pile Diameter)	Subgrade Reaction Reduction Factor, R
8D	1.00
6D	0.70
4D	0.40
3D	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above summary. Reduction for group effects is negligible when the centre-to-centre pile spacing exceeds three pile diameters measured in the direction perpendicular to loading.

6.3.8 Frost Protection

The pile caps for the replacement bridge should be provided with a minimum of 1.8 m of soil cover for frost protection as per OPSD 3090.100 (Foundation Frost Penetration Depths for Northern Ontario), or a combination of soil cover and rigid insulation. For polystyrene insulation, the MTO has adopted an equivalency of 25 mm of insulation for every 0.3 m reduction in soil cover. Rip-Rap or erosion protection materials should be excluded from the soil cover for determining insulation thickness. Further, where insulation is required, it should extend laterally 1.8 m beyond the edge of the pile caps and have a minimum of 0.3 m of conventional soil cover.

It is understood that MH anticipates the TMB foundations will be constructed in the fall of the first construction season, then be in operation the following spring/summer but not the following winter. As such, we anticipate that frost protection would not be required based on the underlying granular (i.e., non-frost-susceptible) pad and the flexibility of the TMB. However, if the TMB will be in operation for the following winter, it is recommended that frost protection be provided to mitigate frost-related differential movements over repeated freeze-thaw cycles.

6.3.9 Seismic Considerations

Subsurface ground conditions for seismic site characterization were established based on the results of the borehole investigations. Based on the subsurface conditions, the site may be classified as Site Class E in accordance with Table 4.1 of the CHBDC (2014), in the absence of any geophysical testing. Geophysics testing, if carried out, could provide a more favourable Site Class designation, however, given the soil conditions, a higher Site Class cannot be guaranteed..

Based on the information obtained from the NRCan (2015) Hazard Calculator for this site located at latitude 45.272402 and longitude -79.673968, the following values were obtained for the spectral acceleration for a return period of 2,475 years:

Seismic Hazard Values	2% Exceedance in 50 years (2,475 return period)
Sa (0.2) (g)	0.116
Sa (1.0) (g)	0.050

Based on the values noted above and in accordance with Table 4.10 of the CHBDC (2014), this site should be considered to be located in Seismic Performance Zone 1. In accordance with Section 4.4.5.1 of the CHBDC (2014), no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.4 Embankment Design

Based on the GA drawing provided by MH, the proposed highway grade at the west and east approach embankments for the replacement structure will be at Elevation 230.4 m and 230.1 m, respectively, approximately 700 mm above the existing highway grade. The proposed detour west and east approach embankments will be at Elevation 229.4 m and 229.1 m, respectively, about 2.1 m and 1.8 m above existing grade along the proposed detour centreline, however the detour roadway embankments reach 2.9 m height at 24+050 and 2.7 m height at 24+210 where the detour transitions from the low-lying swampy area to the existing highway embankment. In general, the east side of the site (i.e. east of Shadow River) has the thickest cohesive deposit while the west side has a slightly higher overall embankment height.

The following sections address the stability and settlement of the raised approach embankments on Highway 141 as well as the proposed detour embankments constructed on the new alignment. It is recommended (and has been assumed in all stability and settlement analyses) that the existing fill and organic soils (peat, topsoil and or mixed organic soil and deleterious fills) will be removed from below the footprint of the detour embankment and that the existing fill will be left in place along the existing alignment where the grade raise is being proposed. The geometry of the proposed embankments, existing ground surface and existing river bed included in the stability and settlement analyses are based on the drawings and cross sections provided by MH. The piezometric conditions used in the analyses are based on the groundwater level as encountered during the subsurface investigation and normal and high water levels as appropriate.

For analysis purposes, it has been assumed that the Highway 141 embankment will be raised using granular fill with side slopes of 2 horizontal to 1 vertical (2H:1V) and the detour embankment will be constructed with rock fill with 1.25H:1V side slopes and a granular pavement structure. Rock fill cannot be used for the grade raise/embankment reconstruction along Highway 141 due to the presence of and need for lightweight fill (i.e. rock fill cannot be used as "cover").

6.4.1 Stability

Analyses were performed on the critical sections of the proposed approach embankments for conditions during and after construction to assess the stability of the proposed embankment height considering the geometry and soil stratigraphy. The critical sections at this site are:

- the front slopes for the bridges
- the Highway 141 east and west abutments
- the TMB east and west abutments
- at STA 24+050 where the detour embankment is at its highest at 2.9 m approximately 50 m west of the TMB

6.4.1.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.23), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The stability analyses were performed to check that the target minimum FoS was achieved for the design embankment heights and geometries. In general, circular slip surfaces were analysed in the design; however, when modelling the weak layer in the clayey stratum (see below), other methods of modelling the slip surface, including block specified slip surfaces, were also analyzed to establish the critical case.

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, ψ , and the geotechnical resistance factor ϕ_{gu} (i.e. $FoS = 1/(\psi * \phi_{gu})$). Accordingly, a target minimum FoS of 1.3 has been used for design of the temporary embankment side slopes, and a FoS of 1.5 for the design of the final embankment configuration as per Table 6.2 of CHBDC (2014) for the total stress (short-term undrained) and effective stress (long-term drained) condition, as applicable.

6.4.1.2 Parameter Selection

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

Soil Type	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Effective Friction Angle (°)
New Granular Fill (compacted Granular 'A' or Granular 'B' Type II)	21	-	35
Existing Granular Fill (very loose to compact)	20	-	33
Rock Fill	19	-	40

Soil Type	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Effective Friction Angle (°)
Clayey Silt to Clay	15.1	See Figure 1 and Figure 2	23
Sandy Silt to Silt (very loose to compact)	18	-	28
Silt and Sand to Sand (loose to compact)	19	-	29
Sand and Gravel (compact)	20	-	30

Based on results obtained from the CPT testing at this site, a 0.8 m thick very soft zone with an undrained shear strength of 8 kPa has been modelled within the clayey silt to clay deposit between Elevation 223.5 m and Elevation 222.7 m.

For rock fill and granular soils, effective stress parameters were employed in the analyses assuming drained conditions. The effective stress parameters (effective friction angle) for the granular soils were estimated from empirical correlations using the results of the SPTs, in conjunction with engineering judgement based on experience in similar soil conditions.

For cohesive deposits, total stress parameters were employed in the analyses assuming undrained conditions. The total stress parameters (i.e., average mobilized undrained shear strength – s_u) for the cohesive soils were assessed based on the results of in-situ field vane shear and CPT tests, and estimated from correlations with laboratory test data (i.e., consolidation tests), where appropriate. From the consolidation tests performed in the clayey soils, the following correlation proposed by Mesri (1975) was employed to estimate the undrained shear strength:

$$s_u = 0.22\sigma'_p$$

where:

$$s_u = \text{average mobilized undrained shear strength (kPa)}$$

$$\sigma'_p = \text{preconsolidation pressure (kPa)}$$

Where appropriate, Bjerrum's correction factor for plasticity was employed to estimate the average mobilized undrained shear strength from the results of the in-situ field vane tests as follows:

$$s_{u(mob)} = \mu s_{u(FV)} \quad (\text{after Bjerrum, 1973})$$

where:

$$s_{u(mob)} = \text{average mobilized undrained shear strength (kPa)}$$

$$s_{u(FV)} = \text{undrained shear strength from field vane test (kPa)}$$

$$\mu = \text{Bjerrum's correction factor based on Plasticity Index}$$

Where varved clay was encountered, an additional reduction factor of 25 per cent was employed to account for the angle (or orientation) of minimum shearing resistance (Milligan and Lo, 1967).

6.4.1.3 Results of Analysis

The results of the analyses indicate that the side slopes of the Highway 141 embankment (south sides), both existing and under the influence of the proposed 0.7 m grade raise, as well as the proposed detour embankments (north sides), will achieve a minimum Factor of Safety (FoS) of 1.3 for the short-term undrained analysis and greater than 1.5 for the long-term drained analysis. The results of the stability analyses are shown on Figures 3 to 10. The stability analyses assume that a minimum of 1 m of lightweight (EPS) fill, as is currently in place in the existing embankment, is incorporated into the embankment design (to mitigate settlement as noted below). Since adequate factors of safety for global embankment stability are achieved, monitoring of excess pore pressures and/or settlement and lateral movement is not required from a stability perspective.

Location/Section	Analysis	Figure	Minimum calculated FoS
West Abutment – Detour	Short-Term (Undrained)	Figure 3	1.54
West Abutment – Existing South	Short-Term (Undrained)	Figure 4	1.88
West Abutment – 0.7 m Grade Raise on Existing	Short-Term (Undrained)	Figure 5	1.49
West Abutment – 0.7 m Grade Raise on Existing	Long –Term (Drained)	Figure 6	2.15
East Abutment – Detour	Short-Term (Undrained)	Figure 7	1.47
East Abutment – Existing South	Short-Term (Undrained)	Figure 8	1.87
East Abutment – 0.7 m Grade Raise on Existing	Short-Term (Undrained)	Figure 9	1.44
Max Detour Embankment Height – STA 24+050	Short-Term (Undrained)	Figure 10	1.38

The front slopes of the bridges were also analyzed and achieved adequate factors of safety.

6.4.2 Settlement

Settlement of the approach and detour embankments can be expected as a result of the loading from the new fills on the soft, compressible foundation soils at this site. In addition, depending on the type of fill materials employed in the construction, settlements may also occur due to compression of the new embankment fill itself. The following sections summarize the methodology, criteria, simplified stratigraphy, unit weights and deformation parameters employed for the different soil types in the approach areas. The maximum estimated settlement of the foundation soils in these areas (due to the loading imposed by the new embankment fills) is presented along with a discussion on the rates of settlement.

6.4.2.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach and detour embankments using both the commercially available program Settle3D (Version 4.0), developed by Rocscience Inc., as well as hand/spreadsheet calculations.

The sources of settlement were considered to include:

- immediate settlement of the native granular soils;
- primary time dependent consolidation of the cohesive deposits (using Terzaghi's one-dimensional consolidation theory);
- secondary time dependent (creep) compression of the cohesive deposits (long-term); and,
- self-weight compression of the embankment fill materials (long-term).

6.4.2.2 Parameter Selection

The immediate compression of the non-cohesive deposits was modelled by estimating elastic moduli of deformation based on the SPT 'N'-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990).

The consolidation settlement of the cohesive deposits was assessed using the results of the laboratory consolidation tests as well as the laboratory index testing to evaluate deformation parameters (i.e., recompression and compression indices) using empirical correlations proposed in literature by Koppula (1986), Terzaghi and Peck (1967), Kulhawy and Mayne (1990) and Azzouz et al. (1976). The correlations by Koppula (1986) and Terzaghi and Peck (1967) relating the natural water content and liquid limit to the compression index was found to be the most consistent with the laboratory consolidation (oedometer) tests.

The preconsolidation stress in the cohesive deposit was evaluated from the laboratory consolidation tests as well as from the results of the in-situ field vane tests. The following correlation relating in-situ undrained shear strength to preconsolidation stress (Mesri, 1975) was employed:

$$\sigma'_p = \frac{S_{u(mob)}}{0.22}$$

where:

$$\begin{aligned} \sigma'_p &= \text{preconsolidation pressure (kPa)} \\ S_{u(mob)} &= \mu S_{u(FV)} \\ S_{u(mob)} &= \text{average mobilized undrained shear strength (kPa)} \\ S_{u(FV)} &= \text{undrained shear strength from field vane test (kPa)} \\ \mu &= \text{Bjerrum's correction factor based on Plasticity Index} \end{aligned}$$

The coefficient of consolidation, c_v (cm²/s), required in the time-rate settlement analysis was evaluated from the results of the laboratory consolidation tests, from in-situ dissipation testing carried out in the CPTs, and also estimated from the NAVFAC (1986) correlation with liquid limit assuming normally consolidated soils.

In addition to primary consolidation within the cohesive deposits (i.e., silty clay to clay), secondary compression may also occur. Secondary compression is referred to as creep settlement and occurs over a long period of time, after full dissipation of excess pore pressure under a constant effective stress. The following relationships have

been employed for estimating the magnitude of creep settlement over the life of the embankment following the completion of primary settlement at each location.

$$S_c = HC_{\alpha\epsilon} \log\left(\frac{t}{t_{EOP}}\right)$$

where:

- S_c = secondary consolidation (creep) settlement (mm)
 $C_{\alpha\epsilon}$ = modified secondary compression index as estimated from laboratory consolidation tests and empirical correlations
 H = initial thickness of the normally consolidated portion of the compressible clay deposit (mm)
 t = post-construction period of interest (20 years)
 t_{EOP} = time to reach end of primary consolidation (years)

For clayey soils, in addition to estimating the modified secondary compression index from laboratory consolidation tests, the following empirical correlation by Mesri (1973) was also utilized to estimate $C_{\alpha\epsilon}$ from water content:

$$C_{\alpha\epsilon} = \frac{w_n}{10,000}$$

where: w_n = natural water content (%)

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

Soil Type	γ (kN/m ³)	E (MPa)	eo	Cc	Cr	σ_p' (kPa)
Upper Silt (very loose to loose)	18	5	n/a			
Clayey Silt to Clay	15.1	n/a	See Figures 1 and 2			
Sandy to Silt to Silt (very loose to compact)	18	5	n/a			
Silt and Sand to Sand (loose to compact)	19	10	n/a			
Sand and Gravel (compact)	20	32	n/a			

The coefficient of consolidation, c_v , required in the time-rate settlement analysis for the normally consolidated soils at this site is estimated to be 2.1×10^{-3} cm²/s (based on the consolidation tests and CPT dissipation tests). For secondary (creep) compression, the modified secondary compression index, $C_{\alpha\epsilon}$ is estimated to be 0.008.

6.4.2.3 Settlement of Approach Embankment Fill

Where rock fill is used for the construction of the proposed detour approach embankments, there will be settlement due to compression of the rock fill itself under self-weight, in addition to the settlement of the underlying foundation soils as described above. The magnitude of settlement of the rock fill depends on the following factors:

- type of rock/strength of particles
- size and shape of rock particles
- gradation of rock fill
- total height/thickness of rock fill (stress level)
- method of construction and sequence of placement (including lift thickness, compactive effort and state of packing)

The settlement of rock fill occurs as a result of re-arrangement of rock particles under load and wetting and as a result of localized crushing of rock particles at point contacts. The magnitude of both the short-term and long-term post-construction settlement of the rock fill is a function of the height of fill as well as the method of fill placement (i.e., compacted versus dumped rock fill) as outlined in the MTO Guideline “Rock Fill Settlement and Rock Fill Quantity Estimates” (September 2010).

Rock fill should be placed, whenever possible, in a controlled manner (i.e., not end-dumped) in accordance with OPSS.PROV 206 (Grading). Blading, dozing and ‘chinking’ the rock fill to form a dense, compact mass is required to minimize voids and bridging and reduce settlements and should be used to construct rock fill embankments above the existing groundwater table. Where rock fill cannot be placed in a controlled manner (i.e., below the groundwater table), the post-construction settlement of the rock fill is expected to be greater.

6.4.2.4 *Short-Term Rock Fill Settlement*

The magnitude of short-term post-construction settlement associated with compacted and end-dumped rock fill may be estimated in accordance with the MTO Guideline “Rock Fill Settlement and Rock Fill Quantity Estimates” (September 2010), as follows:

Height of Rock Fill, H	Short-Term Rock Fill Settlement	
	Compacted Rock Fill	Dumped Rock Fill
Up to 5 m	0.5% H	1.0% H
>5 m to 10 m	0.75% H	1.5% H
>10 m to 15 m	1.0% H	2.0% H

Approximately 90 per cent of the short-term settlement may be expected to occur within the first six months following construction of the embankment to full height. The short-term settlement is expected to be fully completed within one year following the completion of embankment construction to full height.

6.4.2.5 *Settlement Performance Requirements*

The settlement performance criterion for design of high fill embankments is in accordance with MTO Foundations Guideline, “Embankment Settlement Criteria for Design” (March, 2010).

Where new embankments approach structural elements, the following post-construction settlement and differential settlement criteria are considered acceptable for settlements to occur within 20 years post-paving for the bridge approach embankments at this site.

Location	Maximum Limits During Pavement Design Life	
	Distance from Transition Point (i.e., Abutment)	Total Post-Construction Settlement
Transition/Taper to Bridge Abutments	0 m to 20 m	25
	20 m to 50 m	50
	50 m to 75 m	75

These criteria have been used for determining whether mitigation measures are required to limit post-construction settlement of the approach embankments.

In general, new embankments not approaching a structural element are to be designed as follows:

Location	Maximum Limits During Pavement Design Life	
	Total Settlement	Differential Settlement Rate
Non-Freeway (Highway 141)	200 mm	100:1
	20 m to 50 m	50
	50 m to 75 m	75

The total settlement and differential settlement rate are considered to be applicable over a 20-year period following completion of construction (i.e., final paving). These performance criteria form part of the overall design performance for the Highway 141 embankments approaching the new bridge as well as for the detour embankments.

6.4.2.6 Settlement Results

Raised Highway 141 Approach Embankments

A summary of the results of the settlement analyses under the proposed 0.7 m grade raise carried out along the existing alignment for primary and secondary (creep) consolidation settlement of the cohesive soils is presented below. The settlement values presented are factored values in accordance with the requirements of the CHBDC (2014). It should be noted that the settlement analysis accounts for the 1 m thickness of lightweight (EPS) fill that is already present in the existing embankment, but assumes the proposed grade raise will be constructed with conventional fill (granular fill, not rock fill).

Location	Proposed Grade Raise (m)	Estimated Factored Post-Construction Settlement (mm)		
		Primary	Creep in 20 Years Post Paving	Total Post Construction (at 20 year design life)
50 m West of West Abutment	0.15	30	35	65
20 m West of West Abutment	0.45	110	35	145
West Abutment	0.70	180	35	215
East Abutment	0.70	225	45	270
20 m East of East Abutment	0.45	130	45	175
50 m East of East Abutment	0.15	45	45	90

Since the estimated settlement is greater than the settlement criteria, mitigation measures will be required on the existing embankment to reduce/eliminate post-construction settlements. The results of the time-rate consolidation settlement analysis for the critical section on the Highway 141 approach embankments are shown on Figure 11.

Detour

A summary of the results of the settlement analyses for the primary consolidation settlement only of the cohesive soils below the approach embankments to the detour bridge as well as along the detour embankments is presented below. The settlement values presented are factored values in accordance with the requirements of the CHBDC (2014). Since the detour embankment is anticipated to be in use for a short time only (maximum 3 construction seasons), long-term creep settlement and long-term rock fill settlement have not been assessed.

Detour Location	Proposed Embankment Height (m)	Thickness of Clay Deposit (m)	Estimated Factored Post-Construction Settlement (mm)		
			Primary	Short-Term Rock Fill	Total
24+050 – Approx. 50 m west of West Abutment	2.9	9.3	575	20	595
Detour West Abutment	2.1	11.2	580	10	590
Detour East Abutment	1.8	10.2	610	15	625
24+240 – Approx. 90 m East of East Abutment	2.7	10.4	785	15	800

Since the estimated settlement is greater than the settlement criteria, some mitigation measures will be required on the detour embankment to reduce/eliminate post-construction settlements over the construction life for the detour roadway. The results of the time-rate consolidation settlement analysis for the critical sections on the detour embankment (noted above) are shown on Figure 12.

6.4.2.7 Settlement Mitigation

Raised Highway 141 Embankments

Based on a c_v of 2.1×10^{-3} cm²/s and two-drainage in the 9.2 m to 12.6 m thick clayey stratum below the Highway 141 alignment, it is estimated that it will take about 2.7 years to 5.1 years to reach 90% consolidation. Creep will still occur beyond this time, even with the 1 m thickness of EPS currently in place, as a result of the grade raise and the fact that the clayey silt to clay soils are normally consolidated.

For the anticipated construction schedule of reconstruction (grade raise) and bridge replacement in one construction season, preloading and/or surcharging will not have sufficient time to appreciably reduce the post-construction settlement to meet the criteria. Wick drains are also not considered to be practical given that the existing embankments are already in place and would require removal of the existing EPS fill prior to wick drain installation. Other mitigation options such as ground improvement (controlled modulus columns, soil mixing, etc.) are also not practical given the presence of the existing EPS fill within the embankments. Given the project constraints, offsetting the proposed 0.7 m grade raise with additional lightweight fill is considered to be the most practical and economical option. Lightweight fill could consist of expanded polystyrene (EPS) fill or cellular concrete. In this case, by fully offsetting the proposed grade raise with lightweight fill, the creep settlement would be eliminated, which would not be the case for other preload/surcharge or wick drain options. If lightweight fill is used for the grade raise, as described below, then the post-construction settlement of the clayey silt to clay subsoils are estimated to be less than the settlement criteria.

As part of the 2002-5102 construction contract, a 1 m thickness of EPS was installed within the highway embankments to unload the foundation soils and reduce the on-going and excessive roadway settlements observed in the vicinity of the bridge. Based on the historical information, the EPS fill extends from STA 24+054 to the west abutment and from the east abutment to STA 24+224. Overlying the existing EPS fill is a 150 mm thick pre-cast concrete distribution slab that is separated from the EPS with a polyethylene sheeting. Based on discussions with the MTO project team, it is our understanding that since the installation of the EPS in the existing embankment (in 2002), there have been no obvious signs of further on-going distress or settlement at the bridge abutments.

Due to the high costs associated with placement of lightweight fill, it may be advantageous to leave some or all of the existing EPS in place. Further, it may be difficult/expensive to dispose of the existing EPS in a landfill if complete replacement of the existing EPS was to be considered. In order to re-use the existing EPS fill, it is recommended that MTO retain the original supplier to carry-out sufficient testing to confirm that the existing EPS still meets the strength, deformation and unit weight characteristics required for the new design. The following risks are associated with re-use of the existing EPS fill:

- The existing EPS will need to be protected from construction equipment traffic during the reconstruction and bridge replacement.
- The in situ condition of existing EPS cannot be assessed during design phase (i.e. no testing method(s), etc.).

- The extents of the existing EPS are based upon previous contract document information only (i.e. no as-built records are known to exist).
- The EPS is susceptible to degradation from exposure to hydrocarbons (i.e., gasoline/diesel).

If the EPS cannot be re-used, then we recommend full removal of the EPS and replacement with new EPS and covering with a concrete protection slab. In this case, the EPS fill should be at least 1.7 m thick behind the west and east abutments and taper to 1 m thick at STA 24+050 (west) and STA 24+225 (east). The EPS should taper to 0 m thick at a transition slope/step of about 5H:1V beyond the stations provided. EPS is typically provided in blocks 0.3, 0.5 m or 1 m thick, but other thickness can also be obtained, to accommodate the taper requirements. A minimum 1 m thickness of conventional granular cover should be provided over the top of the EPS/concrete slab and on the side slopes. Appropriate staggered/stepped layout of the EPS blocks, ties and spacers should be used to ensure the overall EPS mass acts as a single unit. The entire top and sides of the EPS mass should be covered with a minimum 6 mil (0.15 mm) thick polyethylene sheet for protection from hydrocarbon-based products that may infiltrate the pavement/shoulder structure. Further, a 150 mm reinforced concrete protection slab should be placed on the top surface of the EPS to provide a more uniform distribution of traffic loading over the EPS mass. If EPS fill is employed as part of the reconstruction, an NSSP can be provided for Rigid Expanded Polystyrene Embankment Fill, if required.

If cellular concrete is utilized instead of EPS, it is recommended that a minimum thickness of 2.3 m of cellular concrete be placed behind the abutments (for a distance of 10 m), tapering to 1.2 m thick at the aforementioned stations, then tapering to 0 m over a transition of 5H:1V beyond these stations. The cellular concrete used shall have a unit weight of no greater than 5 kN/m³. If cellular concrete is employed as part of the reconstruction, an NSSP should be included in the contract documents for Cellular Concrete; an example is included in Appendix C.

If MTO determines that the existing EPS exhibits the same characteristics as specified in the NSSP for new construction, then we recommend leaving a portion of the existing EPS in place and adding new lightweight fill consisting of cellular concrete to offset the proposed grade raise. In this case, cellular concrete is the preferred lightweight fill material to transition into the existing EPS fill mass as it can more readily be placed over top of and adjacent to the existing EPS fill without the need to field cut and fit new EPS blocks into the existing geometry. Utilizing new EPS blocks will be problematic in the transition zone as there could be gaps between the old and new fill mass which could result in deflection cracking in the overlying pavement. Further, EPS block size compatibility could be an issue and special EPS block sizes may need to be manufactured for this purpose, increasing the cost and possibly creating delays during construction.

As excavation for the proposed pile cap/filter blanket is required behind the existing abutments, we recommend removal of EPS to a minimum distance of 10 m behind the abutments. A 2.3 m thick layer of cellular concrete should be placed immediately behind the abutment until it abuts the existing EPS. Then the cellular concrete should “cap” the existing EPS to the limits of the EPS as shown on Figure 13. .

In this case of capping the existing EPS with cellular concrete, we recommend that the existing 150 mm thick pre-cast concrete cap be removed in order to be able to fully offset the loading from the grade raise. If it is advantageous to leave the slab in, to avoid risk of damaging the EPS as well as to provide protection from construction equipment, then it is recommended that at a minimum the existing concrete slab be removed to at least 20 m behind each abutment. Beyond this point, where the slab is left in place, the post-construction settlement will slightly exceed the settlement criteria as follows:

Location	Existing Concrete Cap Left in Place (Y/N)	Proposed Grade Raise (m)	Estimated Factored Post-Construction Settlement (mm)		
			Primary	Creep in 20 Years Post Paving	Total Post Construction (20 year design life)
50 m West of West Abutment	Y	0.15	0	0	0
20 m West of West Abutment	Y	0.45	10	35	45
West Abutment to 20 m west of abutment	N	0.70	0	0	0
East Abutment to 20 m east of abutment	N	0.70	0	0	0
20 m East of East Abutment	Y	0.45	40	45	85
50 m East of East Abutment	Y	0.15	30	45	75

The buoyancy of the lightweight fill has been assessed using a design 50 year high water level at Elevation 228.02 m and a Factor of Safety of greater than 2.0 is achieved for either a 1.7 m thickness of EPS (underside at Elevation 228.0 m [west] and 228.4 m [east]) or a 2.3 m thickness of cellular concrete (underside at Elevation 227.5 m [west] and 227.3 m [east]).

So long as the proposed grade raise is fully offset by the use of lightweight fill, settlement monitoring along the reconstructed Highway 141 will not be required.

Detour

Based on a c_v of $2.1 \times 10^{-3} \text{ cm}^2/\text{s}$ and two-drainage in the 9.3 m to 11.2 m thick clayey stratum below the detour alignment, we estimate that it will take about 2.7 years to 4.0 years to reach 90% consolidation. It is understood based on discussions with MTO and MH that a preload and maintenance option is preferred for the detour alignment due to the short operational life. Other options that could be considered are extended preloading and/or surcharging, however these may not fit within the overall project schedule and may require additional measures such as monitoring, toe berms, etc. The estimated factored settlements relative to the proposed construction period (i.e. operation life of the detour) are summarized below:

Construction Schedule	Duration Detour in place	Estimated Range of Factored Settlements (mm)
September 2018 – Detour Construction	0 – 1 month	75 - 125
Overwinter Period – December 2018 to end of April 2019	1 – 6 months	170 - 230
Detour in Operation May 2019 to November 2019	6 – 13 months	100 - 150

The results of the predicted time-rate consolidation settlement on the detour embankment at the sections analysed are shown on Figure 12. Assuming that construction of the detour is completed in September 2018, it is understood that paving is to take place in May 2019 just prior to opening of the detour to traffic. Based on the estimated settlement of up to 230 mm during the approximately 6 month overwinter period, we recommend that a 200 mm surcharge be placed on top of the Granular 'A' base course (following completion of the detour construction and prior to the start of the overwinter period) limited to within 20 m of the proposed TMB abutments to reduce the amount of "top up" that will be required in the Spring of 2019.

Provision should be made in the contract for the detour embankment fill to be in place as a preload during the overwintering period of approximately 6 months. If 6 months of preloading cannot be achieved in the schedule, additional settlement, granular top-up, and re-grading will be required to take place during detour operation. Provision should also be made in the contract package for additional granular material to top up the detour embankment following the overwintering period and during the operational life of the detour. If permitted, a gravel driving surface would facilitate re-grading and topping up of the detour as a pronounced 'bump' is expected to develop at the transition between the detour TMB abutments (which are founded on deep foundations on bedrock) and the immediately adjacent settling detour approach embankments. If paving is required, then asphalt padding and/or asphalt removal/regrading/repaving will be required over the life of the detour and provision should be made in the contract package for several overlays. Another option to reduce the impacts of the "bump" at the abutments would be to construct a flexible approach slab. Further, consideration should be given to delaying the installation of the guide rails until the completion of the overwintering period and also to not have them be rigidly connected to the bridge structure.

So long as the detour is only in operation for one construction season, settlement monitoring is not anticipated to be required.

6.5 Lateral Earth Pressures

The lateral earth pressures acting on the bridge abutment walls and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill material, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of walls for this site. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II, should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting). Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSS 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement), OPSS 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), and OPSS 3190.100 (Walls, Retaining and Abutment, Wall Drain).

- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specification as outlined in the Northern Region Directive (2002) for backfill of structures adjacent to rock embankments. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (Walls, Abutment, Backfill, Rock).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2014) Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand operated compaction equipment should be used to compact the backfill soils immediately behind the walls as per OPSS.PROV 501. Other surcharge loadings should be accounted for in the design as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.8 m behind the back of the wall (in accordance with Figure C6.20 (a) of the Commentary to the CHBDC). For unrestrained walls, granular fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing in accordance with Figure C6.20(b) of the Commentary to the CHBDC (2014).
- The lateral earth pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used:

Fill Type	Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At Rest, K_0	Active, K_a
Granular 'A'	22 kN/m ³	0.43	0.27
Granular 'B' Type II	21 kN/m ³	0.43	0.27

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the foundation design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:
 - rotation of approximately 0.002 about the base of a vertical wall (where the rotation is calculated as the horizontal displacement divided by the height of the wall)
 - horizontal translation of 0.001 times the height of the wall
 - or a combination of both
- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

6.6 Construction Considerations

6.6.1 Subgrade Preparation and Embankment Construction

For the replacement of the existing bridge, excavations for the pile caps will extend to approximately Elevation 225.7 m or approximately 0.5 m below the proposed pile cap elevation for sand filter blanket installation at the abutment and pier locations.

For the detour embankment, it is recommended that the sandy clayey silt fill and organic soils be removed from below the footprint of the proposed embankments and excavations up to 1.5 m below ground surface (as encountered in Boreholes ST-7 and ST-8) may be required. Excavations will extend to Elevation 226.9 m (at least 0.2 m deep) at the west abutment/approach and to Elevation 226.1 m (at least 1.5 m deep) at the east abutment/approach to remove the silty clay fill and/or organic soils

Fill for construction of the raised Highway 141 embankments should consist of lightweight fill as discussed above. Fill for construction of the new detour embankments could consist of Granular 'A' or Granular 'B' Type I or Type II meeting the specifications of OPSS.PROV 1010 (Aggregates) or with rock fill. The rock fill should be restricted in size to 300 mm within the upper 0.5 m to provide a more suitable transition to the Granular 'A' base material, and also in the immediate vicinity of the proposed TMB abutments to permit pile driving. Further given the relatively low embankment heights, consideration should be given to limiting the largest particle size to less than 1.0 m. An example NSSP is provided in Appendix C. The embankment fill should be placed and compacted in accordance with OPSS.PROV 501 (Compacting) and OPSS.PROV 206 (Grading). Rock fill should not be dumped in final position, as it should be deposited on and pushed forward over the end of the layer being constructed. Voids and bridging should be minimized by blading, dozing and 'chinking' the rock to form a dense, compacted mass. Where new fill is to tie into the existing fill along and beyond the approaches of the detour embankment (i.e. where the detour embankment ties into the existing Highway 141 roadway embankment), the new fill should be "keyed-in" or benched into the existing fills in accordance with OPSS 208.010 (Benching of Earth Slopes). Embankment side slopes should be constructed no steeper than 2H:1V in granular fill or 1.25H:1V for rock fill.

The abutment front slopes and side slopes adjacent to the river require erosion protection in accordance with OPSS 511 (Rip Rap, Rock Protection and Granular Sheeting). Erosion protection should be placed on the slopes to at least 0.5 m above the design high water level. Subject to confirmation and modifications as necessary based on the hydrology assessment (by others), erosion protection could consist of a minimum 0.6 m thick layer of R-10 Rip Rap (180 mm size as per OPSS.PROV 1004 [Aggregates Miscellaneous]) or Rock Protection. The designer should address the potential for hydraulic scour below the pile caps in the design of the bridge foundations.

To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS 802 (Topsoil) and OPSS.PROV 804 (Seed and Cover) should be carried out as soon as possible after construction of the embankments. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw, or gravel sheeting as per OPSS 511 (Rip Rap, Rock Protection and Granular Sheeting), and OPSS.PROV 1004 (Aggregates – Miscellaneous) will be required to reduce the potential for erosion and to reduce the potential for the requirement of remedial works on the side slopes in the spring prior to topsoil dressing and seeding.

6.6.2 Filter Blanket

Given the measured height of the artesian groundwater level above the ground surface at this site and the fact that the deep foundations supporting the bridge foundation elements will be driven through the artesian aquifer (as noted during the subsurface investigation), it is recommended that a drainage/filter blanket consisting of a minimum 0.5 m thick layer of sand, meeting the specifications for concrete fine aggregate (OPSS.PROV 1002, Aggregates - Concrete), be placed below the underside of the pile caps encasing all of the piles at the abutments/piers. The base of the filter blanket, 0.5 m below the underside of the pile cap, will extend to Elevation 225.7 m. The concrete fine aggregate layer should extend horizontally a minimum 0.5 beyond each of the pile caps. This requirement applies to all of the foundation elements for both the replacement bridge and the detour TMB.

6.6.3 Use of Heavy Equipment

In order to drive piles and construct the replacement bridge and temporary modular bridge (TMB), the use of heavy construction equipment will be required. The impact of the heavy equipment loads on the underlying soft clayey silt to clay soils must be considered during selection of the methodology and equipment employed for construction. Further, this equipment may also require the construction of temporary access roads to access the bridge foundation element locations. It is assumed that tracked pile driving rigs will be required (in the 60T to 100T range) as well as a larger wheeled crane for launching the TMB (in the 220T range).

Prior to commencement of construction, the Contractor shall retain a Geotechnical Consultant to assess the impact of the proposed equipment loads and methodology and sequence of construction, and to determine the requirements and/or restrictions necessary to safely support the equipment loads on the soft foundation soils at the site (including for construction along the detour and existing alignment). The stress on the existing, proposed and temporary embankments due to the load from construction equipment and traffic as well as the stress resulting from equipment required for pile driving operations, cofferdam installation or bridge launching must be considered. In addition, consideration must be given to the use of temporary piles below the crane outriggers, if required to satisfy global stability. At no time shall any heavy equipment be parked or material stockpiles be placed on the existing or detour embankments, and in particular where the existing EPS fill will be left in place. An Operational Constraint (OC) addressing the equipment pads/access road construction has been provided in Appendix C.

We understand that the large crane for the TMB launch will likely be set up on the east detour approach near the TMB east abutment. The results of a preliminary global stability analysis that includes consideration of loading from the new detour embankment and temporary crane pads, indicates that a maximum contact stress of only 20 kPa can be applied (assuming two 1 m wide pads) in order to maintain a FoS of 1.3. This relatively low contact stress (to maintain global stability) may necessitate the use of temporary pile foundations to support the heavier equipment loads. At STA 24+050 where the detour embankment is at its highest at 2.9 m approximately 50 m west of the TMB, the results of a preliminary global stability analysis using the above noted assumptions (two 1 m wide pads), indicate that a maximum contact stress of only 12.5 kPa can be applied to maintain a FoS of 1.3.

Given the above, the individual crane pad locations and access roads, equipment type and loading conditions, etc. must be evaluated by the Contractor's Geotechnical Consultant including review of the embankment/pad/access road fill thicknesses and permissible ground pressures/contact stresses and minimum

setback distances to ensure an adequate Factor of Safety is maintained against bearing capacity as well as global stability during construction.

Further, stockpiling of material shall not be permitted on the detour embankment within 100 m of the TMB. Stockpiles shall not be permitted on the portion of the existing embankment where the EPS will be left in place. An Operational Constraint (OC) addressing stockpiling of material has been provided in Appendix C.

6.6.4 Temporary Modular Bridge Staging

The configuration of the temporary support for the TMB during the overwintering period shall be the responsibility of the Contractor and should be reviewed and designed by the Contractor's Geotechnical Engineer. Given the relatively low bearing pressures (contact stresses) that can be applied to the detour embankment fills to maintain an adequate Factor of Safety for global stability, and the risk of differential settlements between loaded areas, it may be necessary to utilize deep foundations to support the TMB bridge during the overwintering period.

6.6.5 Control of Groundwater and Surface Water

Excavations for the replacement bridge abutments and piers will extend below the groundwater level. Due to the proximity of the abutments/piers to the river, a groundwater cut-off (cofferdam or similar measure) is recommended to minimize dewatering requirements. Groundwater control will be required to maintain the integrity of the abutment excavations for placement of insulation for frost protection (based on the GA drawing) and the sand filter blanket and to prevent basal heave/disturbance due to groundwater pressures. Dewatering should be in accordance with OPSS 902 (Excavation for Structure) and the MTO NSSP FOUND0003; reference to this NSSP is provided in Appendix D. MH should fill in the data for "return period" in this NSSP. This NSSP also contains reference to OPSS.PROV 517 (Dewatering).

Water takings in excess of 50,000 L/day are regulated by the Ontario Ministry of Environment and Climate Change (MOECC). Certain takings of groundwater and stormwater for construction dewatering purposes with a combined total less than 400,000 L/day qualify for self-registration on the MOECC's Environmental Activity and Sector Registry (EASR). Registry on the EASR replaces the need to obtain a PTTW for water taking and a Section 53 approval for discharge of water to the environment. A "Water Taking Plan" and a "Discharge Plan" are required by the MOECC if water is taken in accordance with an EASR. In all cases, discharge under the EASR must be in accordance with a Discharge Plan (to be developed by a qualified professional). The contractor will be responsible for obtaining any required discharge approvals. A Category 3 PTTW would be required for water takings in excess of 400,000 L/day. Depending on the time of year that the work is carried out, an EASR will likely be required, however MH should review the potential volume of pumping to determine if a PTTW will be required.

The silt and clayey silt that will be exposed within the excavation at the abutments/piers will be susceptible to disturbance from construction traffic and/or ponded water. A concrete working slab or concrete tremie plug (if designed as part of and in conjunction with the dewatering/temporary works) may be required to be placed in the base of the excavation above the subgrade depending on the sequencing/timing of placement of the filter blanket.

Surface water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation, but all surface water should be directed away from the excavations. Seepage from the granular fills should be expected, particularly after precipitation events.

6.6.6 Excavations and Temporary Protection Systems

Excavations will be required to remove existing fill and organic soils below the detour embankments and to allow for construction of the approach embankments. Excavations are also required (as described above) at the abutments and piers for installation of the piles and construction of the pile caps and associated abutment walls and pier columns.

Open-cut excavations must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) and Regulation for Construction Activities. All fill above the groundwater level is classified as Type 3 soil according to OHSA. All fill below the groundwater level and all organic and native soils are classified as Type 4 soil according to the OHSA. Temporary excavations (i.e., those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V for Type 3 soils and 3H:1V for Type 4 soils. Open excavations below the groundwater level are not recommended, except for “strip” excavations of limited extent (maximum 3 m wide by 5 m long) for the sub-excavation and replacement of the fill/organics below the detour embankment alignment where it is adjacent to the existing embankment. It is not necessary to remove any portion of the existing embankment slope for this purpose. Where open cut excavations are not possible due to space constraints or poor soils, temporary protection systems and/or cofferdams may be required. It is considered that a driven, interlocking sheet-pile system would be suitable for the temporary excavation support at the abutments/piers, based on the subsurface soil and groundwater conditions. An interlocking sheet pile system would provide both ground support and groundwater control. Temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems) with the lateral movement of the temporary shoring systems meeting the requirements of Performance Level 2, provided that any existing adjacent structures or utilities can tolerate this magnitude of deformation. If excavations must be completed for removals in close proximity to the existing or new foundations, it is recommended that such protection systems meet Performance Level 1b as specified in OPSS.PROV 539.

The design of the temporary support systems may be designed using the following parameters:

Soil Type	Unit Weight	Undrained Shear Strength	Internal Angle of Friction	Coefficient of Earth Pressure ¹		
	(γ , kN/m ³)	(S_u , kPa)	(ϕ , degrees)	Active, K_a	At Rest, K_o	Passive ² , K_p
Existing Granular FILL (Loose to Compact)	20	-	33	0.29	0.46	3.39
Upper Silt (Very Loose to Loose)	18	-	28	0.36	0.53	2.77
Clayey Silt to Clay (Very Soft to Stiff)	15.1	Varies – See Figures 1 and 2	26	0.39	0.56	2.56
Sandy Silt to Silt (Loose to Compact)	18	-	28	0.36	0.53	2.77

Soil Type	Unit Weight	Undrained Shear Strength	Internal Angle of Friction	Coefficient of Earth Pressure ¹		
	(γ , kN/m ³)	(S_u , kPa)	(ϕ , degrees)	Active, K_a	At Rest, K_o	Passive ² , K_p
Silt and Sand to Sand (Very Loose to Compact)	19	-	29	0.35	0.52	2.88
Sand and Gravel (Dense)	20	-	30	0.33	0.50	3.00

1. The lateral earth pressure coefficients presented above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are expected behind or in front of the walls, the coefficients should be corrected accordingly, as per the CFEM (2006) or other appropriate reference.
2. The total passive resistance below the base of the excavation (i.e. adjacent to the temporary protection system) may be calculated based on the values of K_p indicated above but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.16 of the CHBDC (2014) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

The design water level can be assumed to be at Elevation 226.3 m, but the design should also be checked against the high water level of Elevation 228.02 m. Design of the temporary support system should include an evaluation of base stability, soil squeezing stability and hydraulic uplift stability as defined in the CFEM (2006). The subsurface soils (clayey silt to clay) at this site are sensitive to disturbance from vibration and/or driving operations for sheet pile installation, which should be considered in the design and installation of the temporary protection systems.

Installation of sheet-piles for cofferdams may be impeded by the presence of cobbles inferred to be present within the embankment fill in Boreholes S-1, S-4, S-5, S-6, S-9 and S-10. It may be necessary to excavate and replace the existing fill material (where practical) in the areas of sheet-pile installation in a series of narrow trenches of limited width to remove cobble size materials. In general, the narrowest suitable excavator bucket should be used. The replacement fill could consist of excavated fill material (with maximum particles size of 75 mm) or imported granular material such as OPSS.PROV 1010 Granular 'A' or Granular 'B' Type I or II provided that 100 per cent of the material passes the 75 mm size. Excavation and replacement fill placement should be carried out in the same day to avoid leaving any trench open overnight. An NSSP be included in the contract documents to address obstructions; a sample NSSP is included in Appendix C that amends OPSS 902 (Excavation for Structure).

The selection and design of the excavations, temporary protection systems or cofferdams and groundwater control systems will be ultimately the responsibility of the Contractor.

6.6.7 Obstructions

As noted on the Record of Boreholes, cobbles and boulders were encountered within the existing embankment fill and also within the silt and sand to sand deposit near the bedrock surface. Further, pieces of wood were encountered (0.1 m to 0.3 m thick) within the existing embankment fill. The contractor should be alerted as the cobbles/boulders and pieces of wood could affect the installation of deep foundations, excavations for foundations

and installation of cofferdams/temporary protection systems. An notice to contractor should be included in the Contract Documents to identify to the contractor the presence of cobbles and/or boulders within the embankment fill and overburden soils along with the potential for wood material in the embankment fill; an example notice to contractor is included in Appendix C, outlining the potential for obstructions.

6.6.8 Existing Structure Monitoring

We recommend that the abutments of the existing structure be monitored for settlement and lateral movement during construction of the TMB and detour embankments, especially during construction works adjacent to the existing structure, such as excavation operations, installation of temporary protection/cofferdams and installation of deep foundations, for the following reasons:

- The existing bridge is supported by timber piles to unknown depth;
- The existing structure is old and is in poor condition; and,
- The existing structure will have to carry traffic during construction of the detour alignment and TMB.

The monitoring program (i.e. types of instruments and frequency of monitoring) for the existing bridge structure should be developed by the structural engineering team.

6.6.9 Analytical Testing for Construction Materials

The results of analytical testing carried out on soils samples from Boreholes S-1, S-4, ST-2 and ST-3 from the approximate foundation element elevations for the replacement structure and detour TMB are presented in Table B1 in Appendix B. The suite of parameters tested is intended to allow the design engineer to assess the requirements for the appropriate type of cement to be used in construction and the need for corrosion protection of steel reinforcing elements.

For potential sulphate attack on concrete, the results of the soil analysis were compared to Table 3 in CSA A23109, and indicate that the relative degree of sulphate attack is low (less than the moderate range). However, given that the bridge is located on Highway 141 and will be exposed to de-icing salts it is recommended that C-1 class exposure concrete be considered. Further, the resistivity results indicate that the soil has a severe level of corrosiveness ($R > 2000$) potential based on the Transportation Research Board Guidelines (Transportation Research Board, National Research Council, 1998 as referenced in the MTO Gravity Pipe Design Guidelines, 2014).

It should be noted that the river water levels in the area are subject to seasonal fluctuations and variations due to precipitation events and the water and/or soil chemistry could also be variable. These recommendations are provided as guidance only; the structural designer should take the results of the laboratory testing and the potential for corrosion into consideration when selecting materials for bridge construction.

7.0 CLOSURE

The Foundation Design Report was prepared by Mr. Adam Core, P.Eng., and the technical aspects were reviewed by Ms. Sarah E. M. Poot, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Paul

Dittrich, Ph.D., P.Eng., a senior geotechnical engineer and Principal of Golder and also a Designated MTO Foundations Contact for Golder, conducted an independent quality control review of this report.

Signature Page

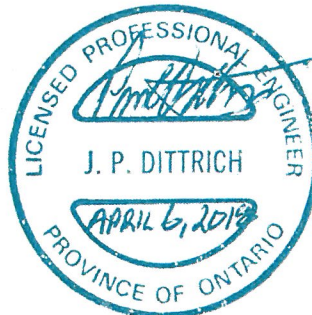
Golder Associates Ltd.



Adam Core, P.Eng.
Geotechnical Engineer



Sarah E.M. Poot, P.Eng.
Associate, Senior Geotechnical Engineer



J. Paul Dittrich, Ph.D., P.Eng.
MTO Foundations Designated Contact, Principal

AC/SEMP/JMAC/kp

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ASTM D2573	Standard Test Method for Field Vane Shear Test in Cohesive Soil
ASTM D5778	Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils

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OPSD 3000.201	Foundation, Piles, HP 310 Olso Point
OPSD 3090.100	Foundation, Frost Penetration Depths for Northern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSD 3121.150	Walls Retaining, Backfill Minimum Granular Requirement

OPSD 3190.100 Walls, Retaining and Abutment, Wall Drain

Ontario Provincial Standard Specifications:

OPSS.PROV 206	Construction Specifications for Grading.
OPSS.PROV 209	Construction Specification for Embankments Over Swamps and Compressible Soils
OPSS.PROV 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip Rap, Rock Protection and Granular Sheeting
OPSS.PROV 517	Construction Specification for Dewatering of Pipeline, Utility and Associated Structure Excavation
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 802	Construction Specification for Topsoil
OPSS 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling - Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS.PROV 1002	Material Specification for Aggregates – Concrete
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Water Resource Act

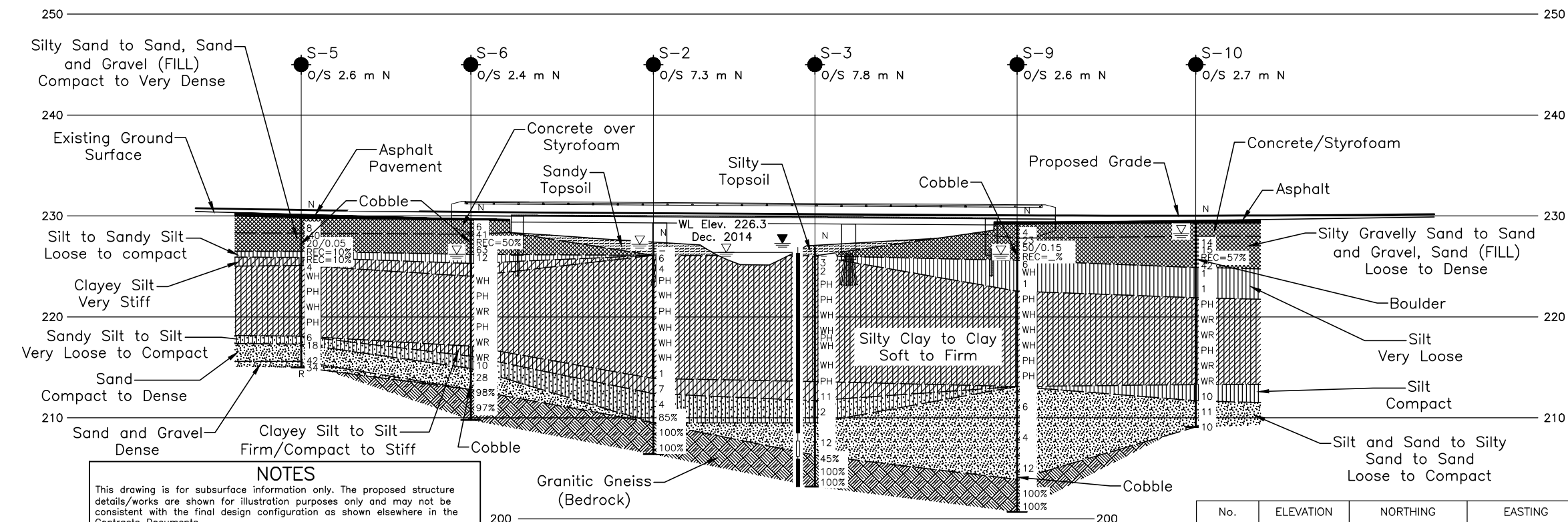
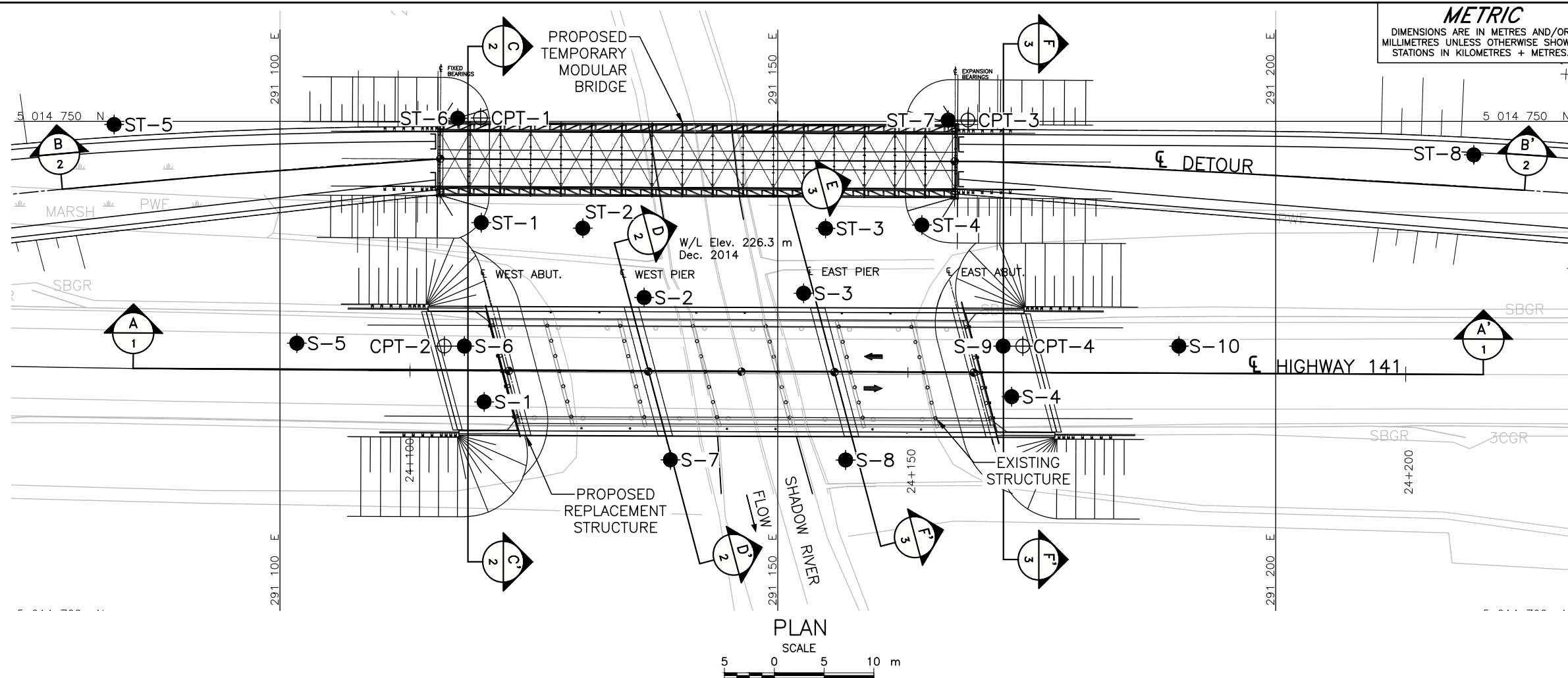
Regulation 903 Wells (as amended)

Table 1: Evaluation of Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Driven steel H-piles	1	<ul style="list-style-type: none"> ■ Straightforward construction. ■ Higher axial resistance compared to shallow foundation. ■ Suitable for integral abutment design. 	<ul style="list-style-type: none"> ■ Requires excavation below groundwater level for pile cap construction. ■ Loading from heavy construction equipment (pile driver) may pose issues with global stability on soft soils adjacent to bridge/embankment. ■ Artesian groundwater pressures will require use of a granular filter blanket or grouting to mitigate the potential loss of fine soil particles due to migration up along the surface of the piles. ■ Downdrag loads will need to be considered for TMB structure and possibly replacement structure depending on settlement mitigation measures adopted. ■ Relatively low lateral capacity compared to large diameter caissons. 	<ul style="list-style-type: none"> ■ 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). ■ Increased costs for temporary protection systems, dewatering for pile caps, and vibration monitoring. 	<ul style="list-style-type: none"> ■ Low risk of not achieving design resistance since piles being driven to refusal on bedrock. ■ Moderate to high risk of ground instability due to heavy construction equipment; mitigation or temporary support required for temporary works. ■ 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.
Drilled steel casings – small diameter	2	<ul style="list-style-type: none"> ■ Relatively straightforward construction. ■ Less disturbance/vibration compared to driven piles. ■ Well-suited to penetrating obstructions in soil and forming socket in strong to very strong bedrock; however, these conditions are not key factors at this site. 	<ul style="list-style-type: none"> ■ Requires excavation below groundwater level for pile cap construction. ■ Loading from heavy construction equipment may pose issues with global stability on soft soils adjacent to bridge/embankment. ■ Artesian groundwater pressures will require use of a granular filter blanket or grouting to mitigate the potential loss of fine soil particles due to migration up along the surface of the piles. ■ Minimum separation distance required may be less than for driven units. ■ Downdrag loads will need to be considered depending on replacement strategy chosen. ■ May not allow for semi-integral abutment design. 	<ul style="list-style-type: none"> ■ More expensive than driven piles; less expensive than larger diameter drilled shafts or micropiles. 	<ul style="list-style-type: none"> ■ Low risk of not achieving design resistance since piles socketed into bedrock. ■ Higher risk of ground instability due to heavy construction equipment; mitigation or temporary support required for temporary works. ■ Lower risk (as compared with driven piles and drilled shafts) of vibrations impacting existing structure – minimum separation distance and mitigation measures required.
Micropiles	3	<ul style="list-style-type: none"> ■ Lighter weight equipment more suitable for use on soft soils adjacent to bridge/embankment. ■ Less disturbance/vibration compared to driven piles. ■ May be more preferred for half-and-half bridge construction staging option due to poor condition of existing bridge and need for stabilization of existing bridge. 	<ul style="list-style-type: none"> ■ Allows only for semi-integral abutment design. ■ Requires detailed micropile design/ drawings/specifications. ■ Pile load tests required to confirm capacity for design. ■ Requires excavation below groundwater level for pile cap construction. 	<ul style="list-style-type: none"> ■ Additional cost associated with detail micropile design. ■ 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. ■ Additional cost for the micropile pile load tests. 	<ul style="list-style-type: none"> ■ Low risk of not achieving design resistance since micropiles socketed into bedrock. ■ Lower risk of ground instability due to lighter weight construction equipment. ■ Lowest risk of impacting existing bridge due to lower vibrations associated with micropile installation – minimum separation distance and mitigation measures required.
Driven steel pipe piles	4	<ul style="list-style-type: none"> ■ Relatively straightforward construction. ■ Higher axial resistance compared to shallow foundations. ■ May be suitable for integral abutment design depending on pile diameter. 	<ul style="list-style-type: none"> ■ Requires excavation below groundwater level for pile cap construction. ■ Loading from heavy construction equipment (pile driver) may pose issues with global stability on soft soils adjacent to bridge/embankment. 	<ul style="list-style-type: none"> ■ 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). 	<ul style="list-style-type: none"> ■ Low risk of not achieving design resistance since piles being driven to refusal on bedrock. ■ Moderate to high risk of ground instability due to heavy construction equipment; mitigation or temporary support required for temporary works.

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
			<ul style="list-style-type: none"> ■ Vibrations during pile installation could impact existing structure; non-standard special provisions may be required to address vibration monitoring during pile installation. ■ Artesian groundwater pressures will require use of a granular filter blanket or grouting to mitigate the potential loss of fine soil particles due to migration up along the surface of the piles. ■ Minimum separation distance required greater than for drilled units. ■ Downdrag loads will need to be considered depending on replacement strategy chosen. 	<ul style="list-style-type: none"> ■ Increased costs for temporary protection systems, dewatering for pile caps and vibration monitoring. 	<ul style="list-style-type: none"> ■ 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.
Drilled shafts (caissons) – large diameter	5	<ul style="list-style-type: none"> ■ Fewer deep foundation elements required due to higher resistance of large diameter shafts/caissons. 	<ul style="list-style-type: none"> ■ Loading from heavy construction equipment (caisson rig) may pose issues with global stability on soft soils adjacent to bridge/embankment. ■ Not suitable for integral abutment design. ■ Vibrations during liner installation effect existing soft soils and may 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. ■ 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. ■ Appropriate procedures required to mitigate artesian groundwater pressures (i.e., liners filled with drilling mud during advancement; use of tremie concrete techniques). ■ Minimum separation distance required less than for driven units but higher due to large diameter. ■ Challenges “seating” caissons into strong to very strong, moderately sloping bedrock. ■ Downdrag loads will need to be considered depending on replacement strategy chosen. 	<ul style="list-style-type: none"> ■ Relative costs much higher than for steel H-piles, pipe piles, and drilled steel casings. 	<ul style="list-style-type: none"> ■ Low risk of not achieving design resistance since caissons socketed into bedrock. ■ Some risk of not achieving rock seal/socket formation in sloping bedrock. ■ Moderate to high risk of vibrations/construction activity negatively impacting existing bridge – highest minimum separation distance and mitigation measures required. ■ High risk of ground instability due to heavy construction equipment; mitigation or temporary support required for temporary works.
Socketted H-Piles	Not recommended	<ul style="list-style-type: none"> ■ Relatively straightforward construction. ■ Higher axial resistance compared to shallow foundations. 	<ul style="list-style-type: none"> ■ Not recommended due to high potential for disturbance during temporary casing removal. ■ Requires excavation below groundwater level for pile cap construction. ■ Loading from heavy construction equipment may pose issues with global stability on soft soils adjacent to bridge/embankment. ■ Artesian groundwater pressures will require use of a granular filter blanket or grouting to mitigate the potential loss of fine soil particles due to migration along the surface of the piles. 	<ul style="list-style-type: none"> ■ More expensive than driven piles; less expensive than larger diameter drilled shafts; similar cost to small diameter casings. 	<ul style="list-style-type: none"> ■ Low risk of not achieving design resistance since piles socketed into bedrock. ■ Higher risk of ground instability due to heavy construction equipment; mitigation or temporary support required for temporary works. ■ Moderate to high risk (as compared with driven piles and drilled shafts) of vibrations impacting existing structure–minimum separation distance required, mitigation measures required.

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
			<ul style="list-style-type: none">■ Minimum separation distance required would be greater than for driven pile elements due to requirements for use of temporary liner during construction.■ Downdrag loads will need to be considered depending on replacement strategy chosen.		
Shallow Foundations	Not feasible	<ul style="list-style-type: none">■ Conventional construction.	<ul style="list-style-type: none">■ Axial resistances too low for this option to be technically feasible.■ Potential for total settlement and differential settlement between abutments/piers even with mitigation.■ Requires deeper excavation and dewatering (cofferdam) adjacent to the river to allow for construction in-the-dry compared to that for pile caps.■ Not suitable for integral abutment design.	<ul style="list-style-type: none">■ 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.	<ul style="list-style-type: none">■ 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.■ Higher risk of negative impacts on existing bridge (if left in operation during construction)



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

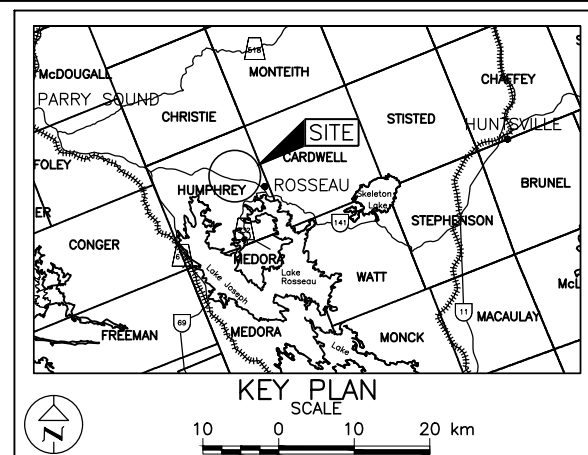
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No.	ELEVATION	NORTHING	EASTING
ST-4	226.9	5014739.6	291164.5
ST-5	227.5	5014749.7	291083.3
ST-6	227.1	5014750.3	291117.9
ST-7	227.6	5014750.1	291167.1
ST-8	227.5	5014746.6	291219.9

CONT No.
WP No.291-96-01

HIGHWAY 141
SHADOW RIVER BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

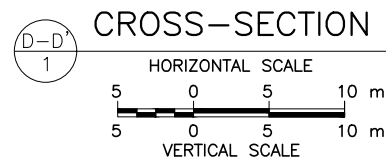
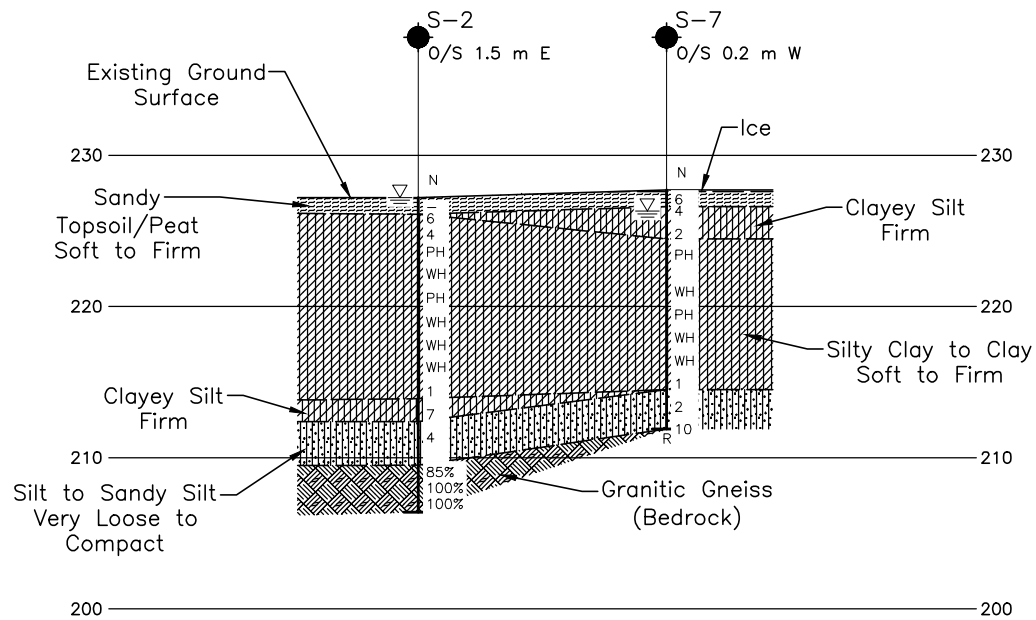
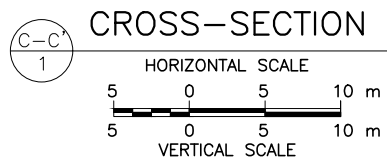
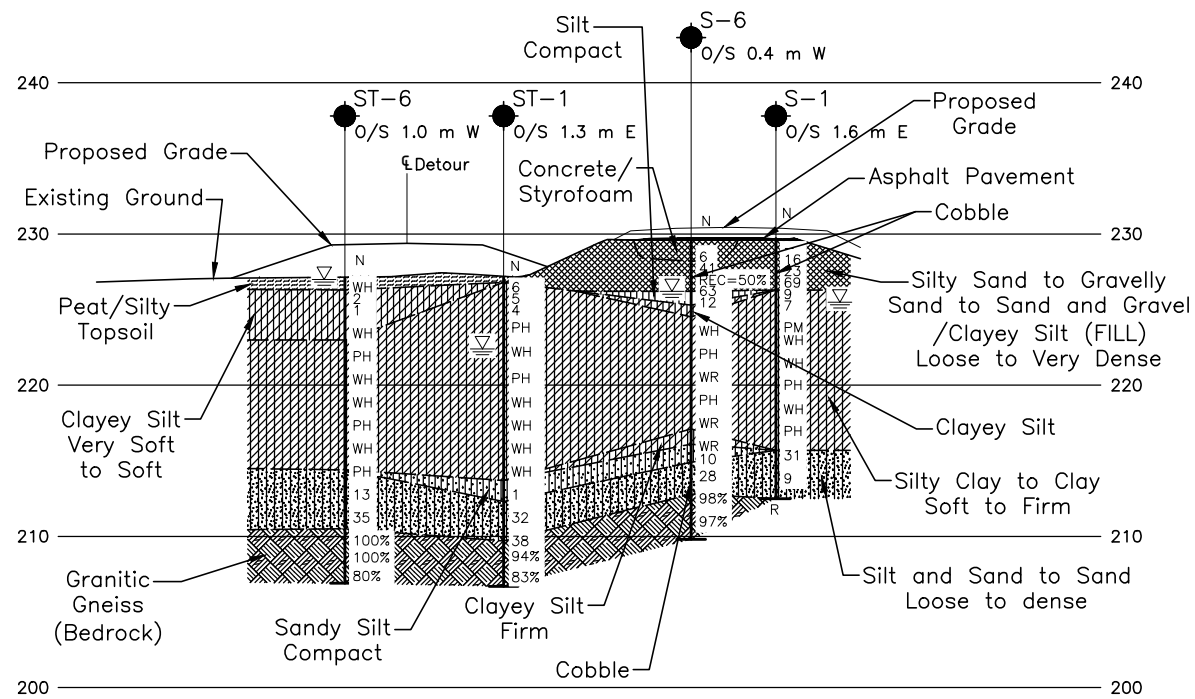
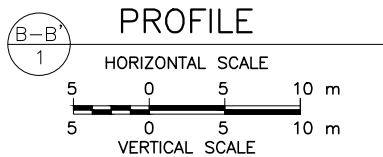
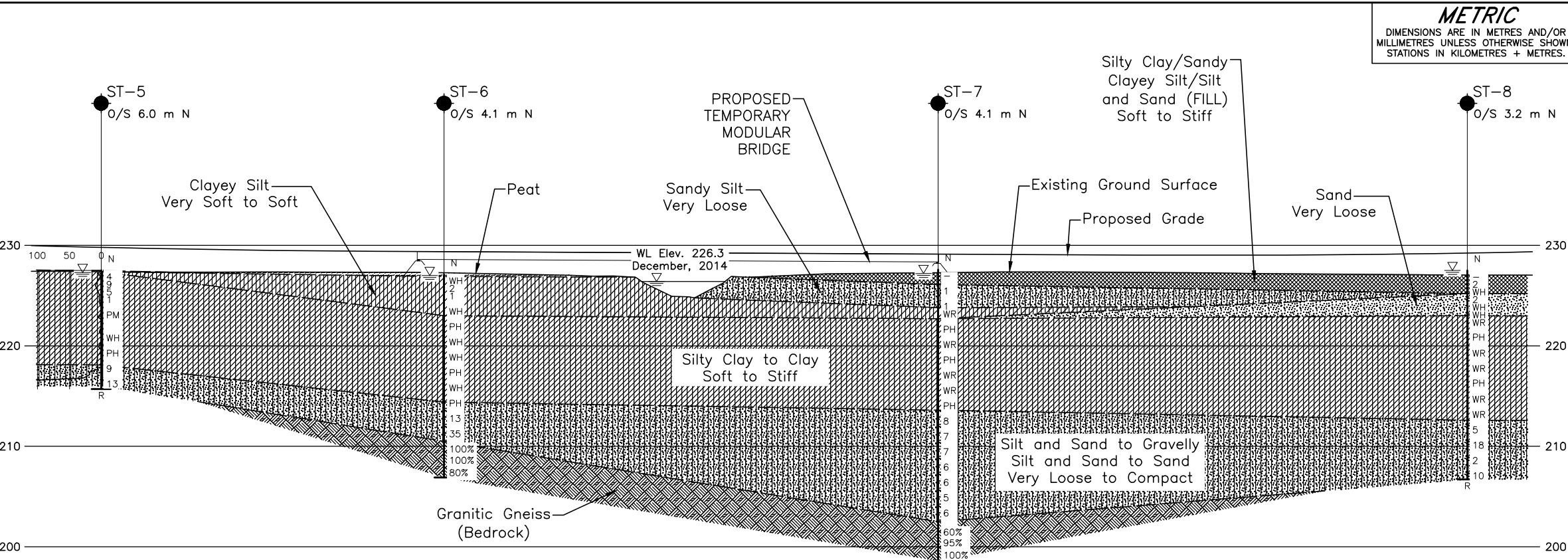


LEGEND

- Borehole
- Cone Penetration Test (CPT)
- 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
CPT-1	227.1	5014750.4	291120.2
CPT-2	229.7	5014727.4	291116.5
CPT-3	227.6	5014750.1	291169.1
CPT-4	229.4	5014727.4	291174.6
S-1	229.7	5014721.8	291120.5
S-2	227.2	5014732.3	291136.6
S-3	226.9	5014732.7	291152.6
S-4	229.4	5014722.4	291173.5
S-5	230.0	5014727.7	291101.7
S-6	229.7	5014727.4	291118.5
S-7	227.7	5014716.0	291139.2
S-8	227.8	5014716.0	291156.8
S-9	229.4	5014727.4	291172.6
S-10	229.5	5014727.4	291190.3
ST-1	227.1	5014739.8	291120.2
ST-2	226.7	5014739.3	291130.4
ST-3	226.8	5014739.2	291154.8

NO.	DATE	BY	REVISION
Geocres No. 31E-386			
HWY. 141	PROJECT NO. 1651997		DIST. .
SUBM'D.	CHKD. AC	DATE: 3/28/2018	SITE: 44-159
DRAWN: TB	CHKD. SEMP	APPD. JMAC	DWG. 1



CONT No.
WP No.291-96-01

HIGHWAY 141
SHADOW RIVER BRIDGE
SOIL STRATA

SHEET



LEGEND

- Borehole
- 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 upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
S-1	229.7	5014721.8	291120.5
S-2	227.2	5014732.3	291136.6
S-6	229.7	5014727.4	291118.5
S-7	227.7	5014716.0	291139.2
ST-1	227.1	5014739.8	291120.2
ST-5	227.5	5014749.7	291083.3
ST-6	227.1	5014750.3	291117.9
ST-7	227.6	5014750.1	291167.1
ST-8	227.5	5014746.6	291219.9

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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-386			
HWY. 141	PROJECT NO. 1651997		DIST.
SUBM'D.	CHKD. AC	DATE: 3/28/2018	SITE: 44-159
DRAWN: TB	CHKD. SEMP	APPD. JMAC	DWG. 2

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

CONT No.
WP No.291-96-01

HIGHWAY 141
SHADOW RIVER BRIDGE

SOIL STRATA

SHEET



LEGEND

- Borehole
- 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-3	226.9	5014732.7	291152.6
S-4	229.4	5014722.4	291173.5
S-8	227.8	5014716.0	291156.8
S-9	229.4	5014727.4	291172.6
ST-4	226.9	5014739.6	291164.5
ST-7	227.6	5014750.1	291167.1

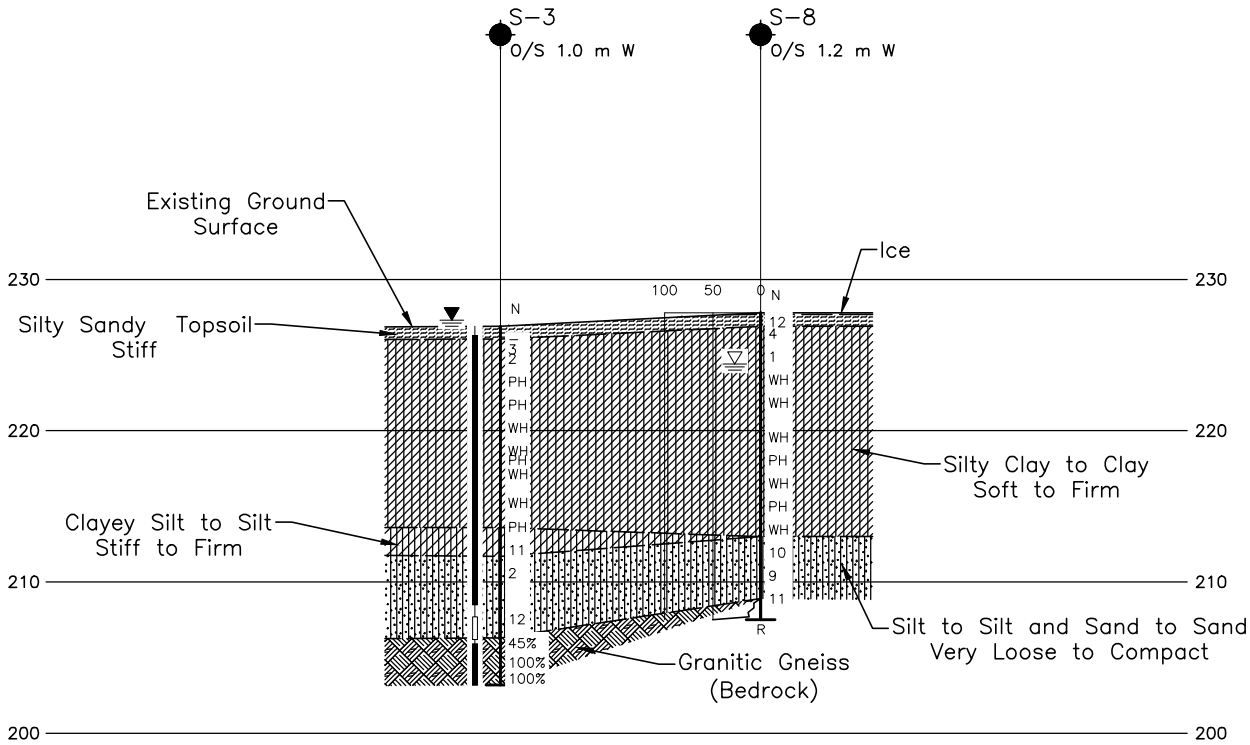
NOTES

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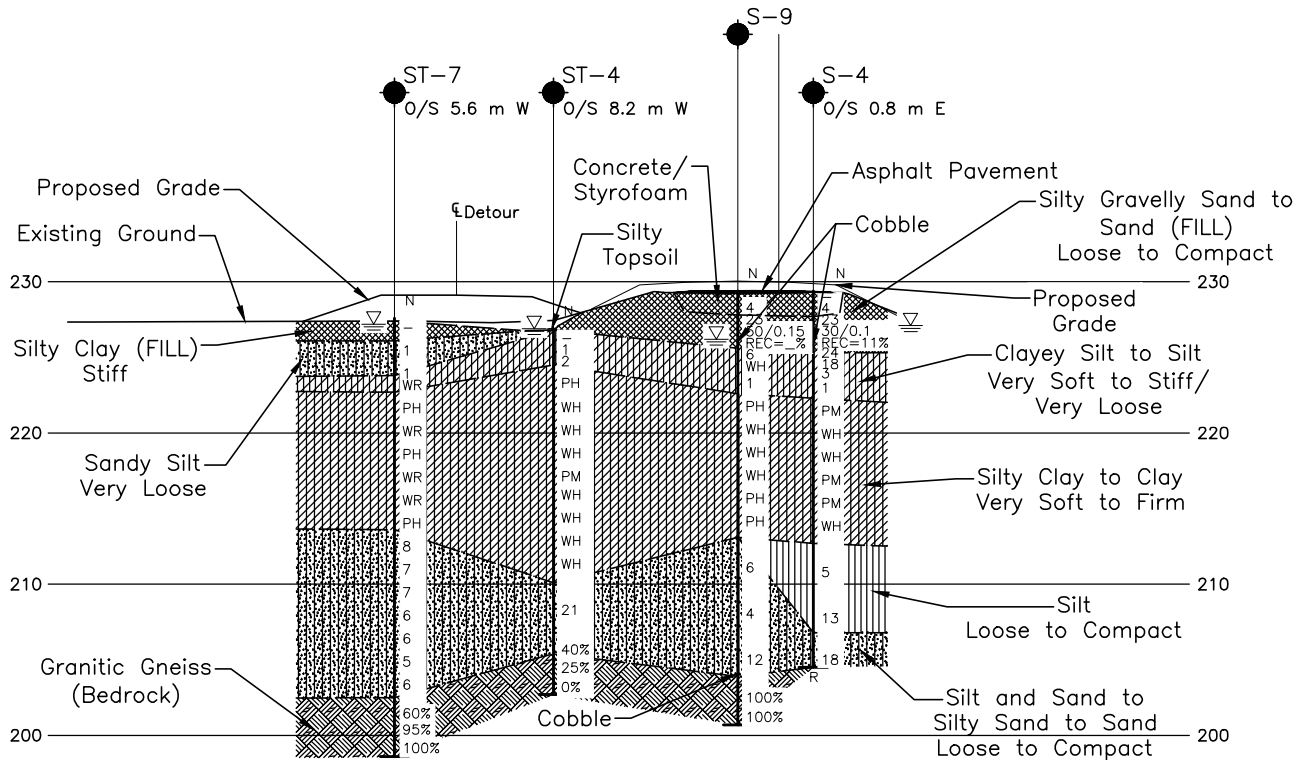
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

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.



CROSS-SECTION
E-E' 1
HORIZONTAL SCALE
5 0 5 10 m
VERTICAL SCALE
5 0 5 10 m



CROSS-SECTION
F-F' 1
HORIZONTAL SCALE
5 0 5 10 m
VERTICAL SCALE
5 0 5 10 m



NO.	DATE	BY	REVISION
Geocres No. 31E-386			
HWY. 141	PROJECT NO. 1651997	DIST.	
SUBM'D.	CHKD. AC	DATE: 3/28/2018	SITE: 44-159
DRAWN: TB	CHKD. SEMP	APPD. JMAC	DWG. 3

Site Photographs



Photograph 1: Highway 141 – At East Approach Looking West (March 2017)



Photograph 2: Highway 141 – At West Approach Looking East (March 2017)

Project No.	1651997-WO1
Date:	February 2018

Golder Associates Ltd.

Inputted by:	LP
Checked by:	AC

Site Photographs



Photograph 3: Highway 141 – Looking West from near West TMB Abutment (January 2018)

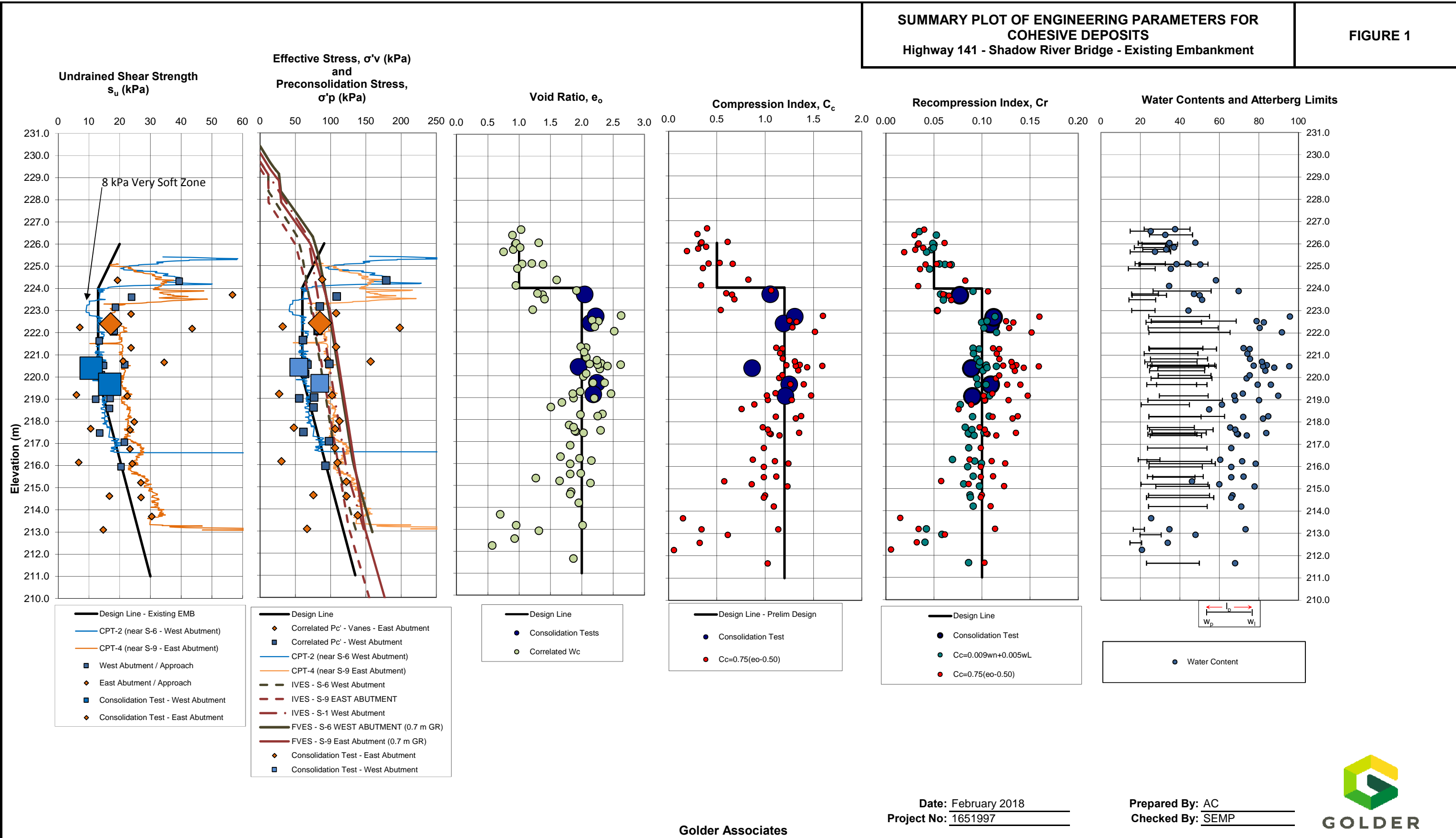


Photograph 4: Highway 141 – Looking West at existing bridge and detour area (January 2018)

Project No.	1651997-WO1
Date:	February 2018

Golder Associates Ltd.

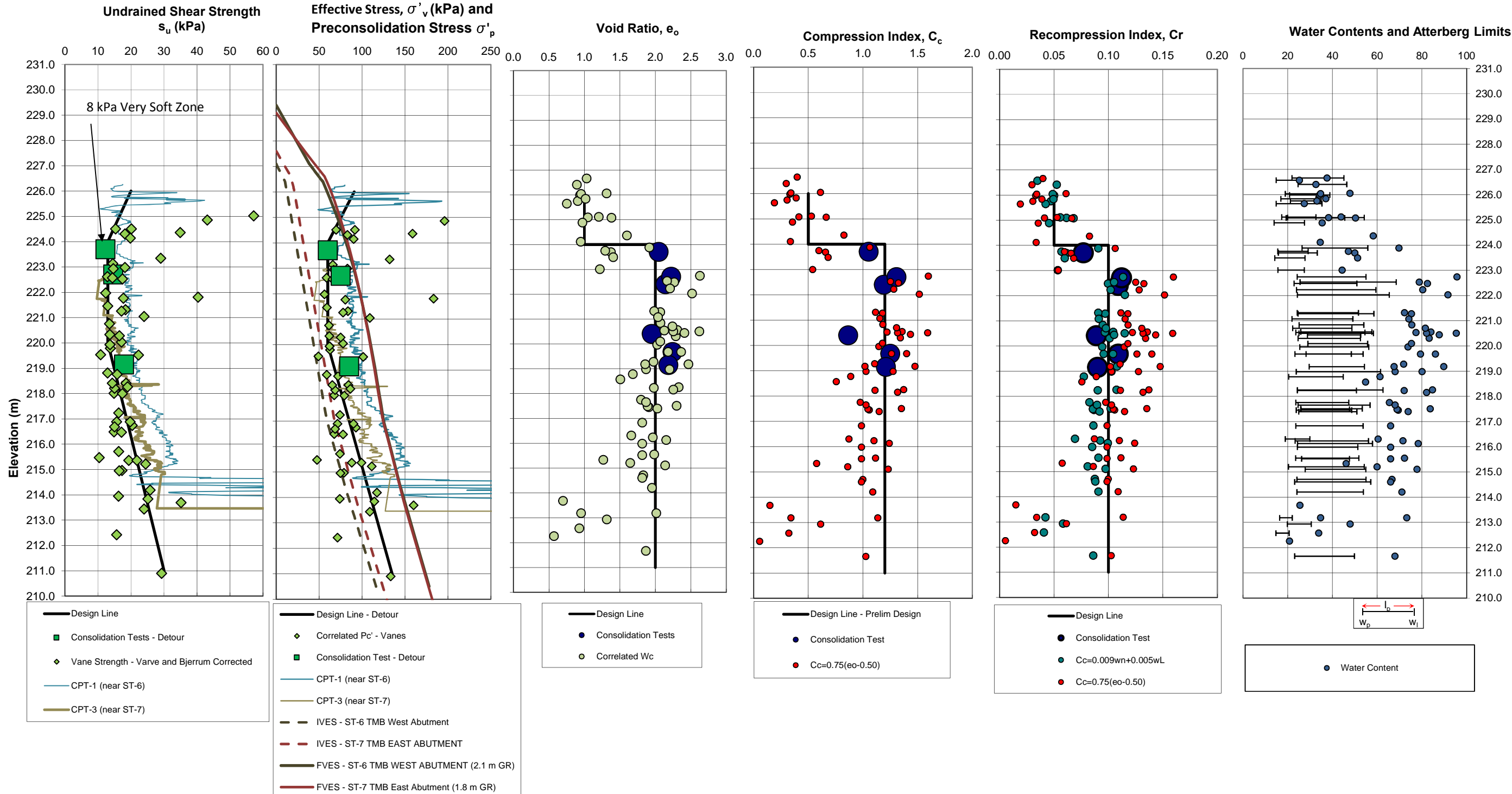
Inputted by:	LP
Checked by:	AC



SUMMARY PLOT OF ENGINEERING PARAMETERS FOR COHESIVE DEPOSITS

Highway 141 - Shadow River Bridge - Detour

FIGURE 2



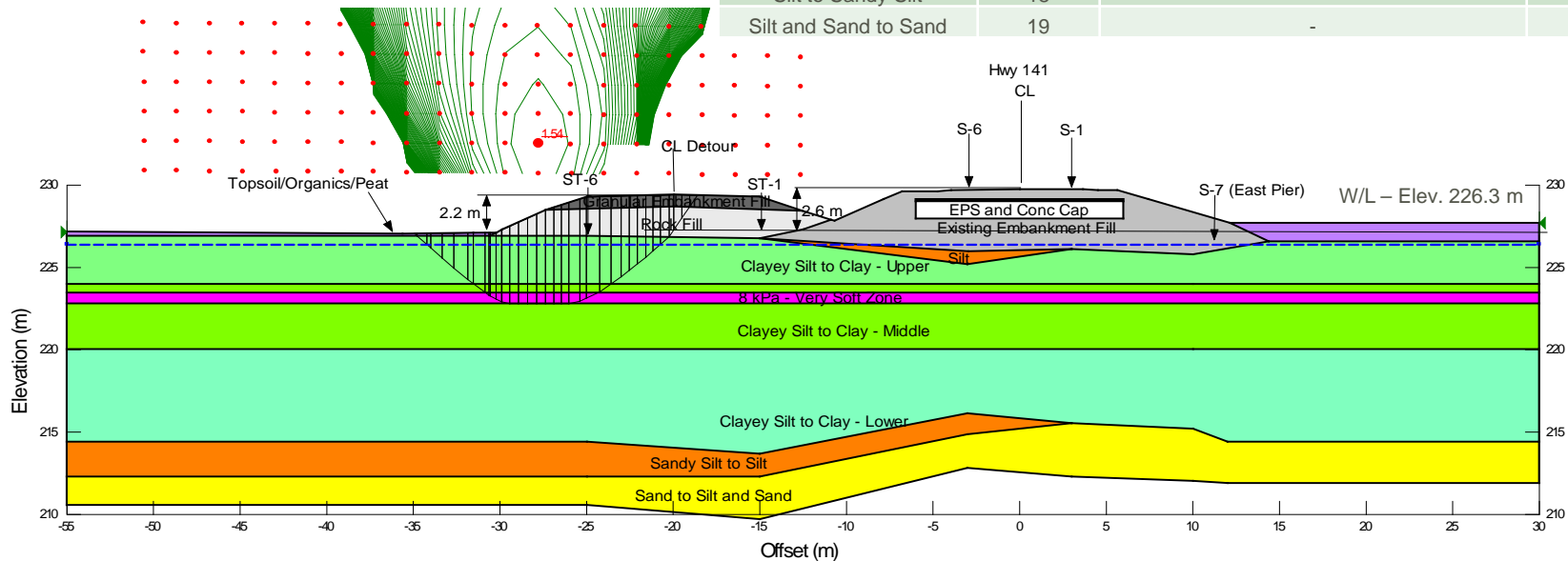
Stability Analysis

West Abutment – Detour

Short-Term (Undrained) Analysis

Figure 3

Material Name	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
Existing Granular Fill	20	-	33
New Embankment Fill	21	-	35
Rock Fill	19	-	40
EPS	0.5	-	-
Concrete	24	-	-
Clayey Silt to Clay	15.1	20 – 13 (Above Elev. 224 m) 13 – (Elev. 224 m to 220 m) 8 (Very Soft Zone Elev. 222.8 m – 223.5 m) 13 – 30 (Below Elev. 220 m)	-
Silt to Sandy Silt	18	-	28
Silt and Sand to Sand	19	-	29

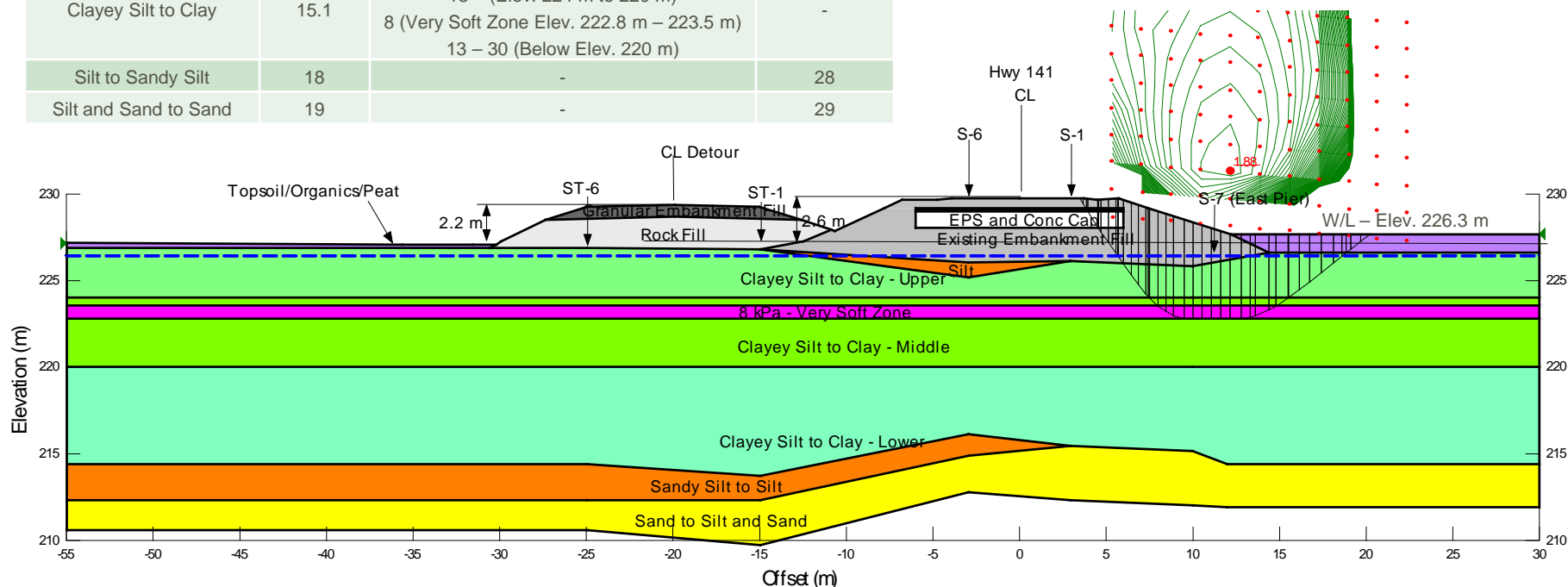


Stability Analysis

West Abutment – Existing South Short-Term (Undrained) Analysis

Figure 4

Material Name	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
Existing Granular Fill	20	-	33
New Embankment Fill	21	-	35
Rock Fill	19	-	40
EPS	0.5	-	-
Concrete	24	-	-
Clayey Silt to Clay	15.1	20 – 13 (Above Elev. 224 m) 13 – (Elev. 224 m to 220 m) 8 (Very Soft Zone Elev. 222.8 m – 223.5 m) 13 – 30 (Below Elev. 220 m)	-
Silt to Sandy Silt	18	-	28
Silt and Sand to Sand	19	-	29

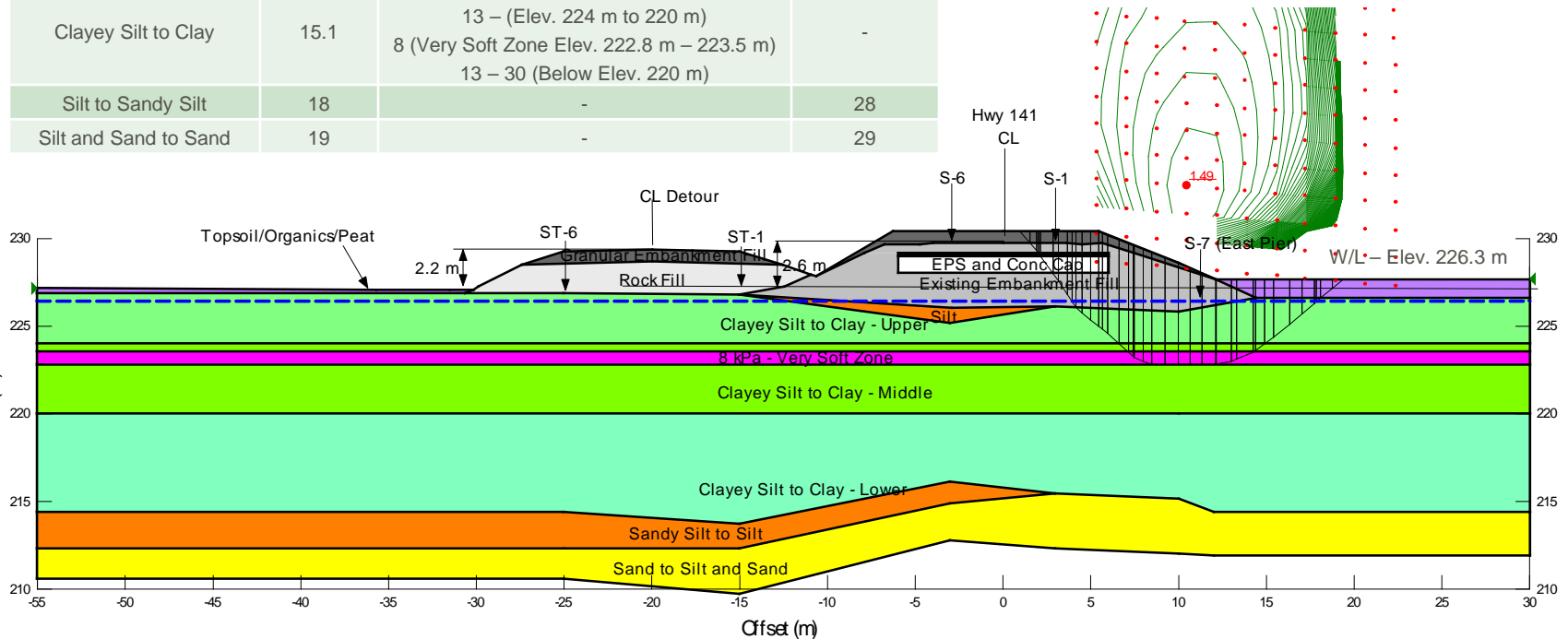


Stability Analysis

West Abutment, 0.7 m Grade Raise on Existing Short-Term (Undrained) Analysis

Figure 5

Material Name	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
Existing Granular Fill	20	-	33
New Embankment Fill	21	-	35
Rock Fill	19	-	40
EPS	0.5	-	-
Concrete	24	-	-
Clayey Silt to Clay	15.1	20 – 13 (Above Elev. 224 m) 13 – (Elev. 224 m to 220 m) 8 (Very Soft Zone Elev. 222.8 m – 223.5 m) 13 – 30 (Below Elev. 220 m)	-
Silt to Sandy Silt	18	-	28
Silt and Sand to Sand	19	-	29

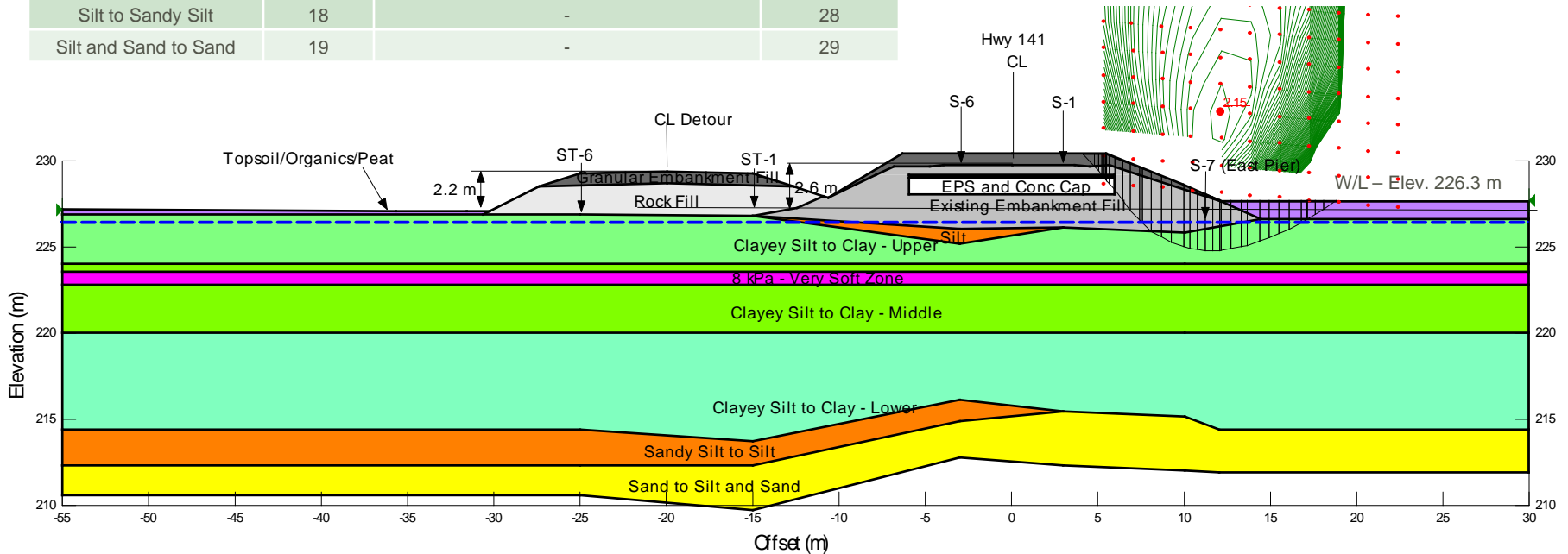


Stability Analysis

West Abutment, 0.7 m Grade Raise on Existing Long Term (Drained) Analysis

Figure 6

Material Name	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
Existing Granular Fill	20	-	33
New Embankment Fill	21	-	35
Rock Fill	19	-	40
EPS	0.5	-	-
Concrete	24	-	-
Clayey Silt to Clay	15.1	-	23
Silt to Sandy Silt	18	-	28
Silt and Sand to Sand	19	-	29



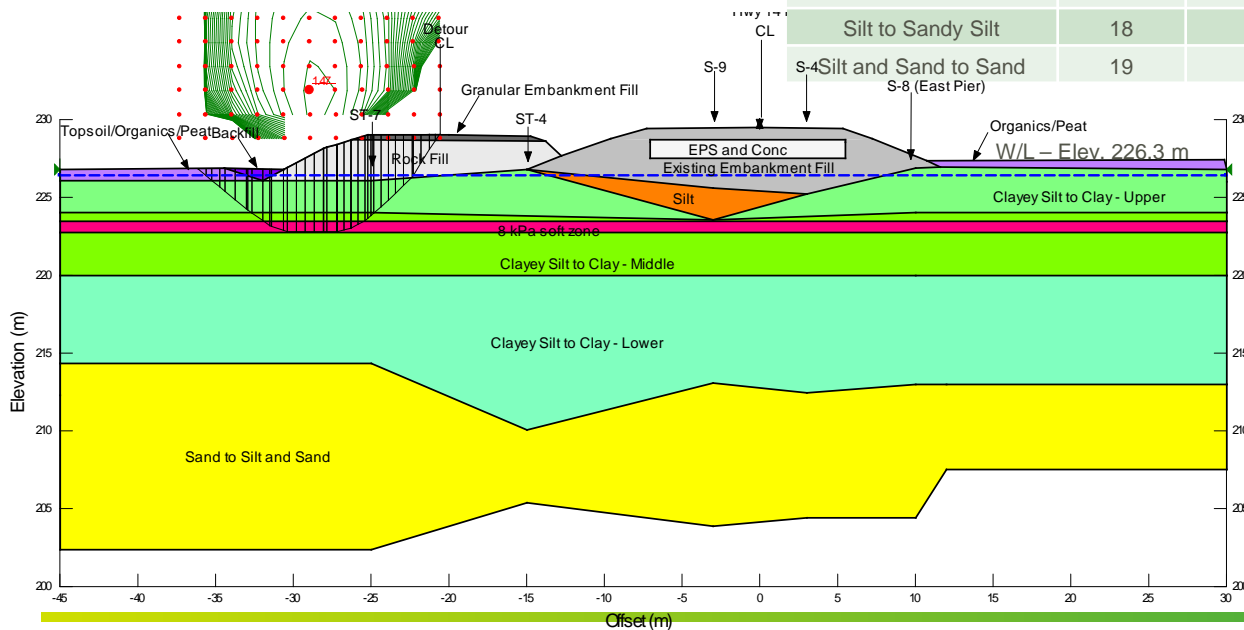
Stability Analysis

East Abutment – Detour

Short-Term (Undrained) Analysis

Figure 7

Material Name	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
Existing Granular Fill	20	-	33
New Embankment Fill	21	-	35
Rock Fill	19	-	40
EPS	0.5	-	-
Concrete	24	-	-
Clayey Silt to Clay	15.1	20 – 13 (Above Elev. 224 m) 13 – (Elev. 224 m to 220 m) 8 (Very Soft Zone Elev. 222.8 m – 223.5 m) 13 – 30 (Below Elev. 220 m)	-
Silt to Sandy Silt	18	-	28
Silt and Sand to Sand	19	-	29



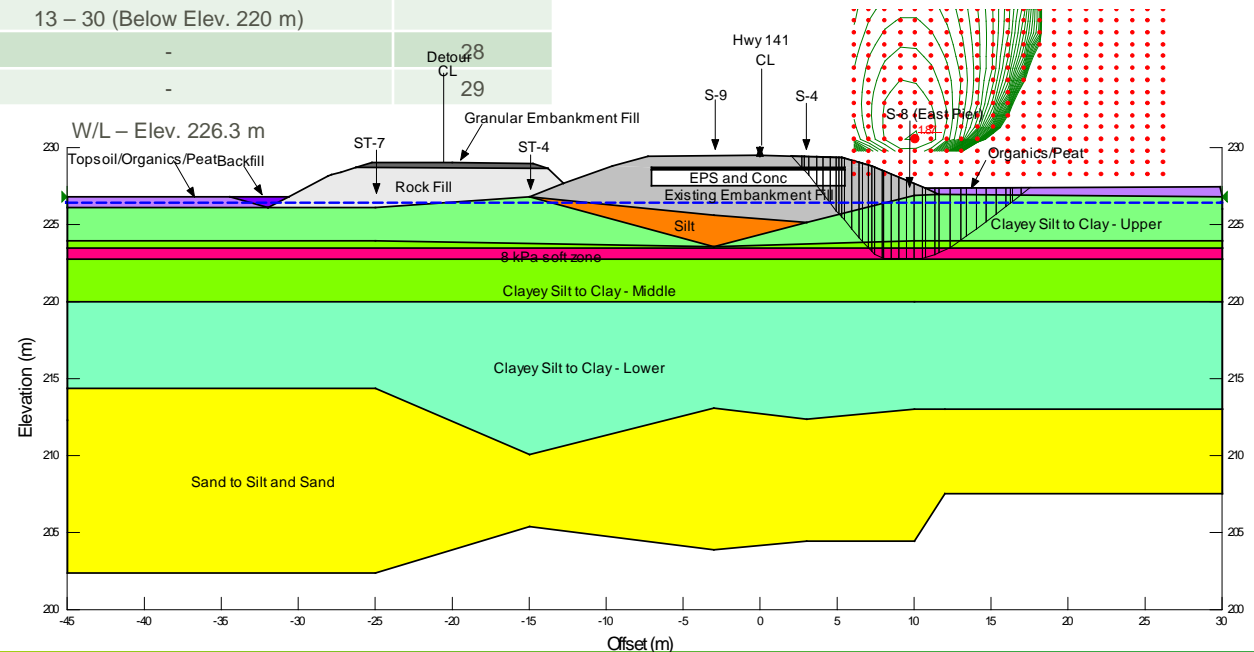
Stability Analysis

East Abutment – Existing South

Short-Term (Undrained) Analysis

Figure 8

Material Name	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
Existing Granular Fill	20	-	33
New Embankment Fill	21	-	35
Rock Fill	19	-	40
EPS	0.5	-	-
Concrete	24	-	-
Clayey Silt to Clay	15.1	20 – 13 (Above Elev. 224 m) 13 – (Elev. 224 m to 220 m) 8 (Very Soft Zone Elev. 222.8 m – 223.5 m) 13 – 30 (Below Elev. 220 m)	-
Silt to Sandy Silt	18	-	28
Silt and Sand to Sand	19	-	29

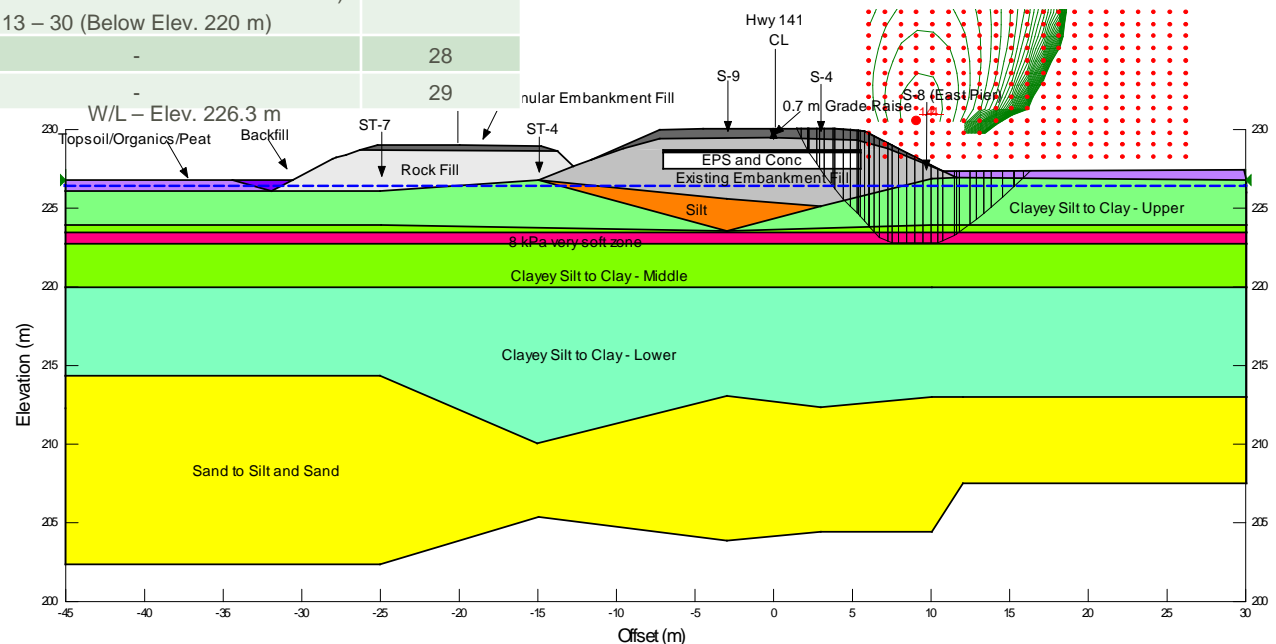


Stability Analysis

Figure 9

East Abutment – 0.7 m Grade Raise on Existing Short Term (Undrained) Analysis

Material Name	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
Existing Granular Fill	20	-	33
New Embankment Fill	21	-	35
Rock Fill	19	-	40
EPS	0.5	-	-
Concrete	24	-	-
Clayey Silt to Clay	15.1	20 – 13 (Above Elev. 224 m) 13 – (Elev. 224 m to 220 m) 8 (Very Soft Zone Elev. 222.8 m – 223.5 m) 13 – 30 (Below Elev. 220 m)	-
Silt to Sandy Silt	18	-	28
Silt and Sand to Sand	19	-	29

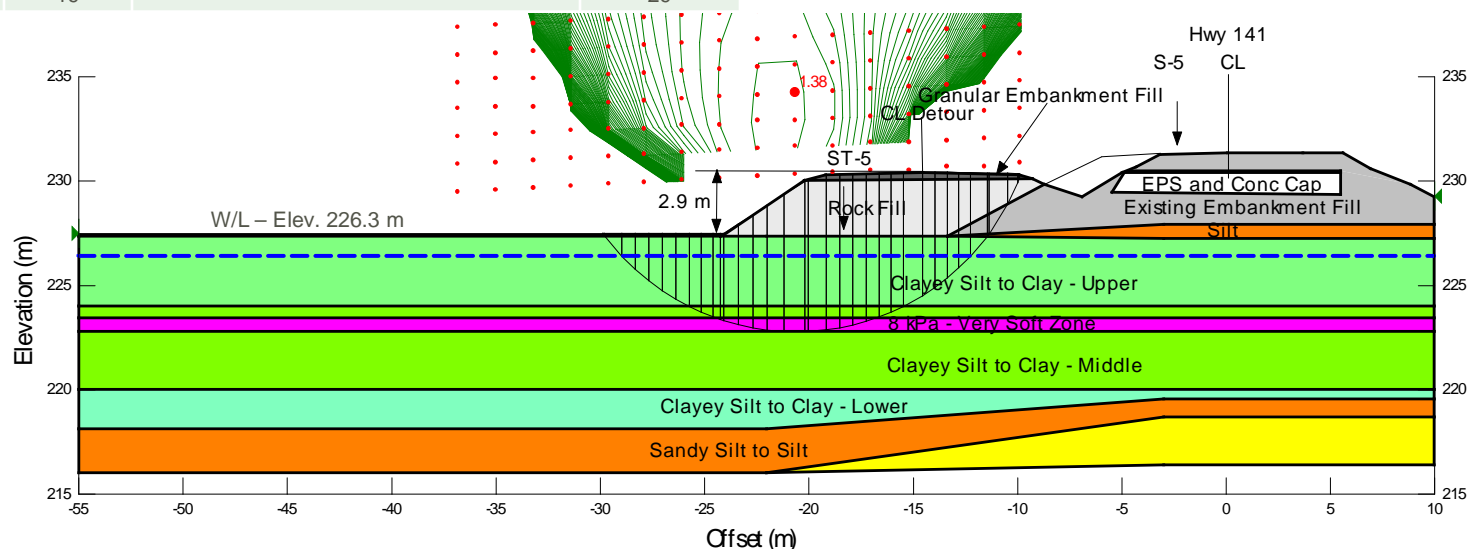


Stability Analysis

Figure 10

Max Detour Embankment Height – STA 24+050
Short-Term (Undrained) Analysis

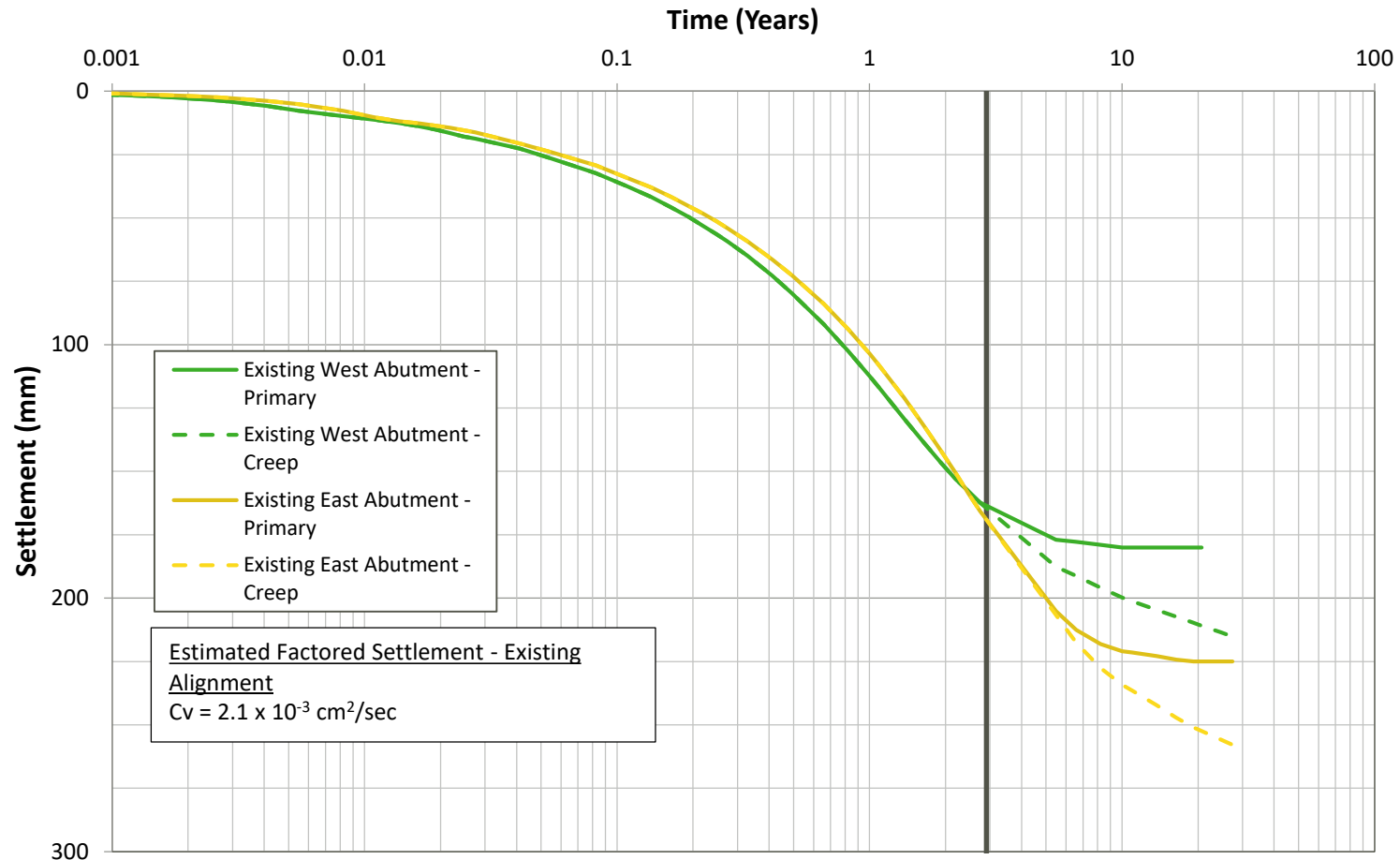
Material Name	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
Existing Granular Fill	20	-	33
New Embankment Fill	21	-	35
Rock Fill	19	-	40
EPS	0.5	-	-
Concrete	24	-	-
Clayey Silt to Clay	15.1	20 – 13 (Above Elev. 224 m) 13 – (Elev. 224 m to 220 m) 8 (Very Soft Zone Elev. 222.8 m – 223.5 m) 13 – 30 (Below Elev. 220 m)	-
Silt to Sandy Silt	18	-	28
Silt and Sand to Sand	19	-	29



Settlement Analysis

Figure 11

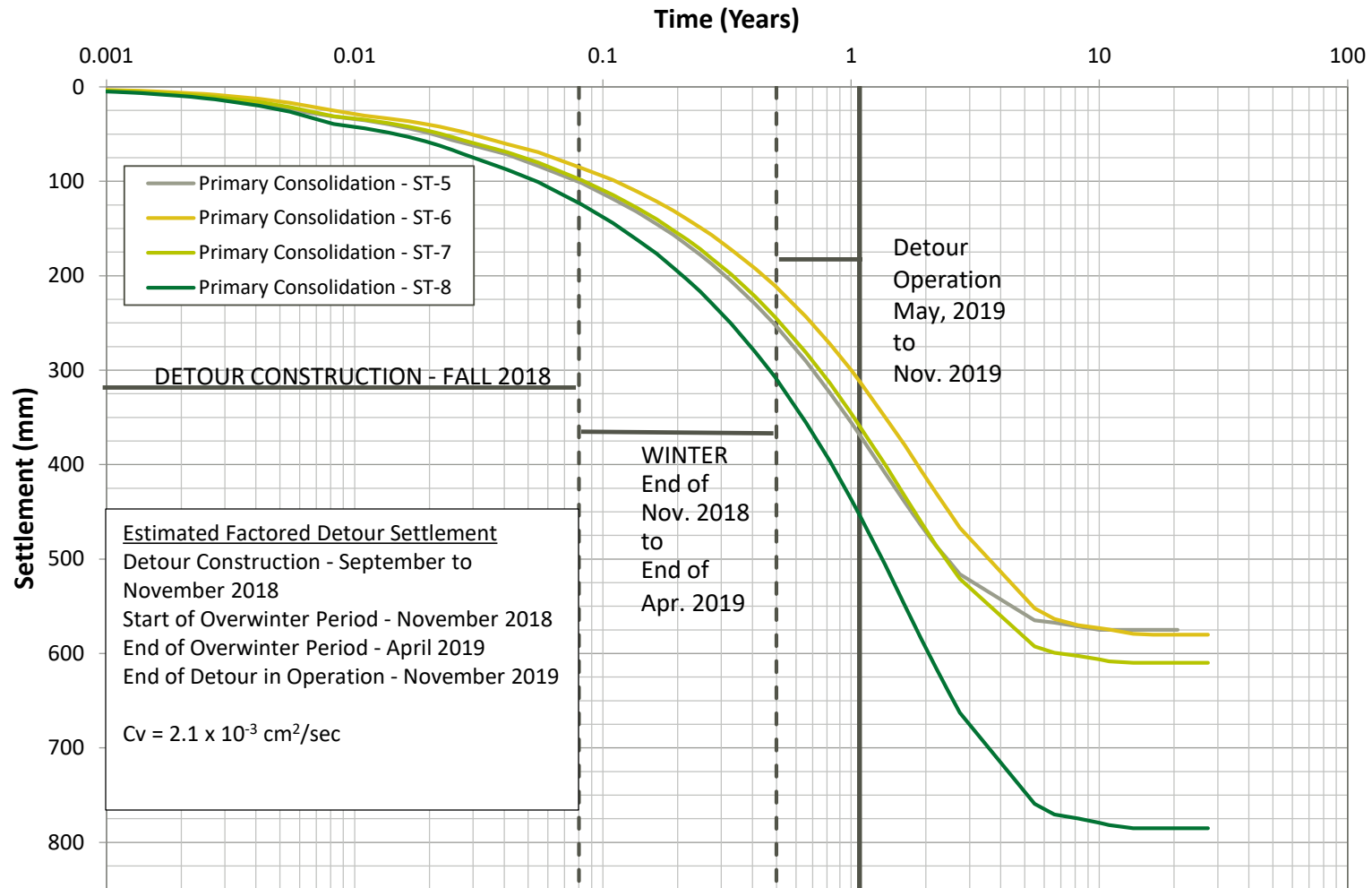
Existing Alignment – Grade Raise



Settlement Analysis

Detour Embankment

Figure 12



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

CONT No.	
WP No.	
HIGHWAY 141 SHADOW RIVER BRIDGE	SHEET
LIGHTWEIGHT FILL – DETAILS	



LEGEND

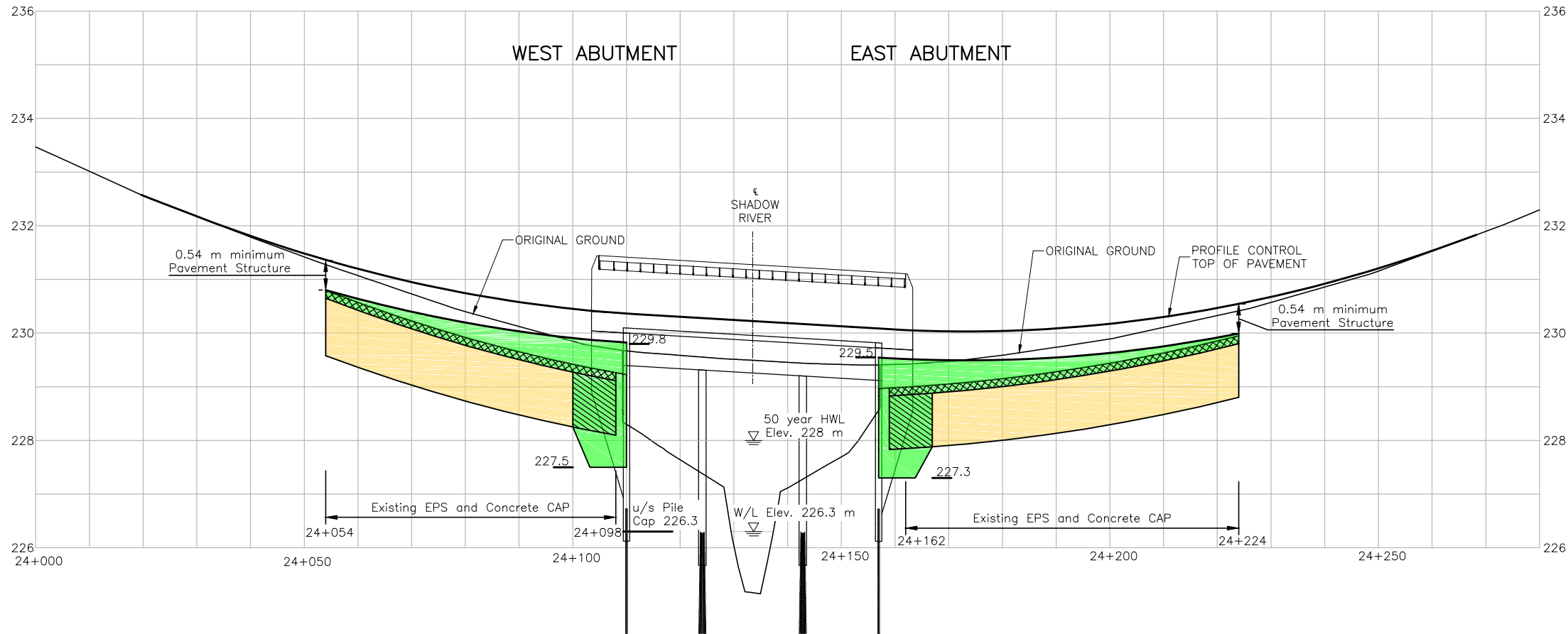
- New Cellular Concrete
- Existing EPS – Approximate location
- Concrete Cap to be removed
- EPS to be removed

NOTES

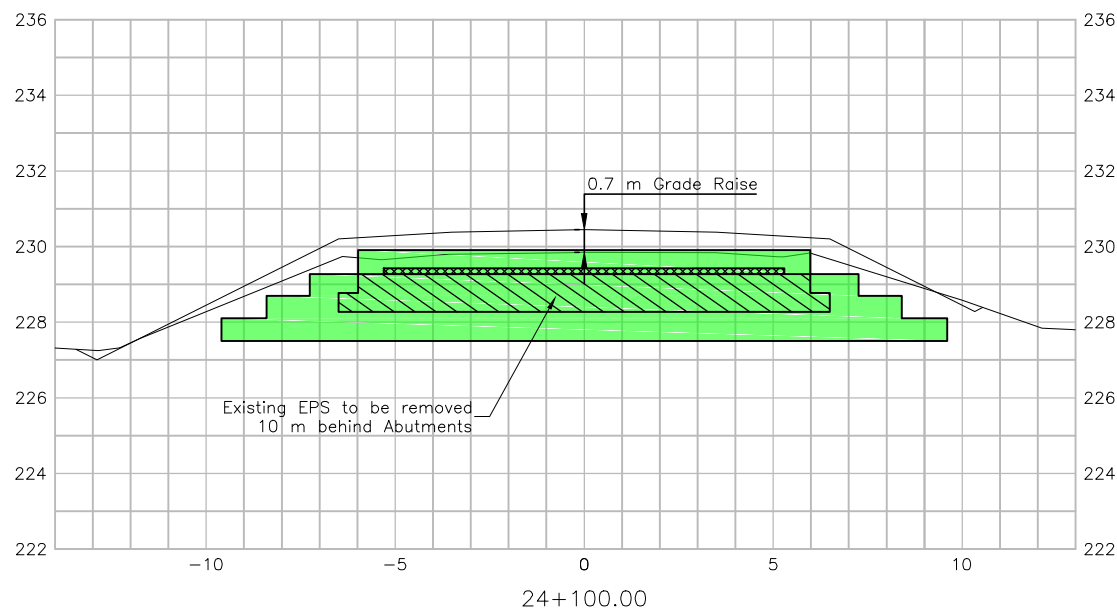
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

REFERENCE

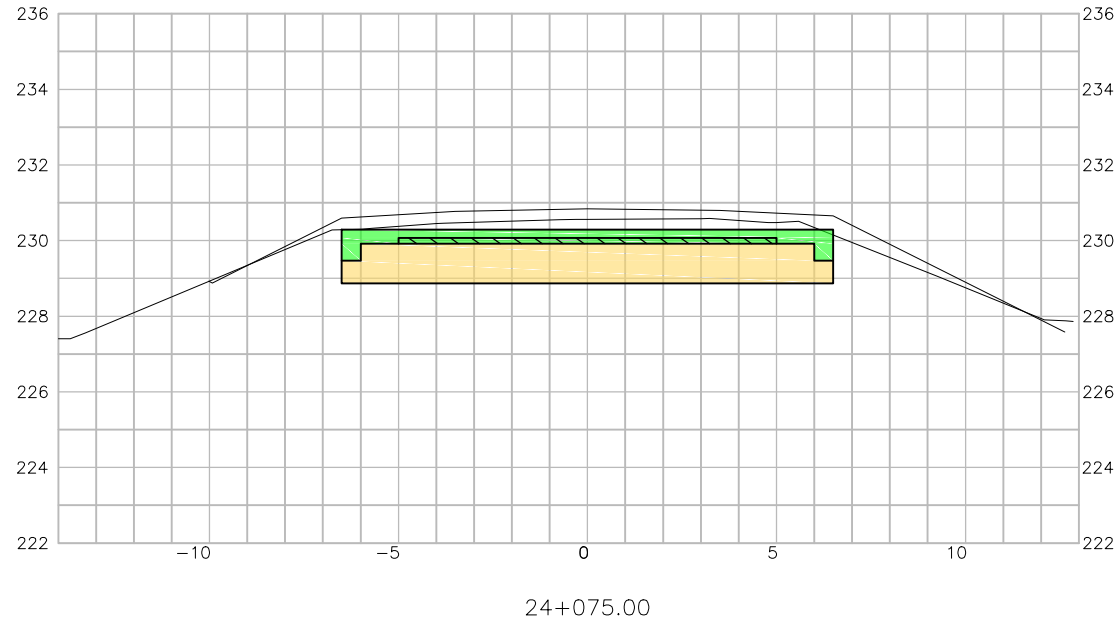
Base plans provided in digital format by MH, drawing file no. Hwy 141 Prelim Cross Sections.dwg, received JAN 09, 2018 and x1140651Profile.dwg, received DEC 14, 2017.



ABUTMENT/APPROACH PROFILE



TYPICAL SECTION 1



TYPICAL SECTION 2

NO.	DATE	BY	REVISION
Geocres No. 31E-379			
HWY. 141	PROJECT NO. 1651997	DIST.	
SUBM'D.	CHKD. AC	DATE: 3/9/2018	SITE: 44-159
DRAWN: TB	CHKD.	APPD.	FIG. 13

APPENDIX A

Record of Boreholes

LIST OF SYMBOLS

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

I. GENERAL

π	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

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	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
γ'	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_{α}	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
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	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 ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1
2

$\tau = c' + \sigma' \tan \phi'$
shear strength = (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 cm² 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

Compactness	N
Condition	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 ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
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)
γ	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

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

<u>Description</u>	<u>Bedding Plane Spacing</u>
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

<u>Description</u>	<u>Spacing</u>
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

<u>Term</u>	<u>Size*</u>
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	

1 OF 2 **METRIC**

CHECKED BY SEMP


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

PROJECT		16519971651997-1002		RECORD OF BOREHOLE No S-1		2 OF 2 METRIC											
W.P.		291-96-01		LOCATION		N 5014721.8; E 291120.5 MTM ZONE 10 (LAT. 45.272321; LONG. -79.674135)											
DIST		HWY 141		BOREHOLE TYPE		Solid Stem Augers, NW Casing and Wash Boring											
DATUM		GEODETIC		DATE		March 14 and 15, 2017											
						ORIGINATED BY SA											
						COMPILED BY AC											
						CHECKED BY SEMP											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
215.7	CLAY, trace sand Soft to firm Brown to grey Wet		12	TO	PH		217										
14.0	Silty SAND, trace to some gravel Loose to compact Grey Wet		13A	SS	31		216										
			13B														
							215										
			14	SS	9		214										
							213										
212.5	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. 225.5 m) upon completion of drilling.																



\\SUD-MTO\001\MTM\ZN\INC\LAT\LONG S:\CLIENTS\MTO\1651997\AECOM_5015-E-0045_NE RETAINER\02 DATA\GINT\1651997.GPJ GAL-MISS.GDT 3/2/18 TB

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

PROJECT		16519971651997-1002				RECORD OF BOREHOLE No S-2				2 OF 3		METRIC						
W.P.		291-96-01		LOCATION		N 5014732.3; E 291136.6 MTM ZONE 10 (LAT. 45.272415; LONG. -79.674135)				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 _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
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													
213.3	CLAYEY SILT to SILT, trace to some sand Firm Grey Wet																	
212.4	Sandy SILT Loose Grey Wet		11	SS	7													
212.4																		
209.5	GRANITIC GNEISS (BEDROCK)																	
209.5	Bedrock cored from 17.7 m to 20.8 m depth. For coring details see Record of Drillhole S-2.		1	RC	REC 93%												RQD = 85%	
			2	RC	REC 100%												RQD = 100%	
			3	RC	REC 100%												RQD = 100%	
206.4	END OF BOREHOLE																	
20.8	Note: 1. Water level at ground surface (Elev. 227.3 m) upon completion of drilling.																	

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTOT\1651997 AECOM_5015-E-0045_NE RETAINER\02_DATA\GINT\1651997 GPJ GAL-MISS.GDT 3/2/18 TB

PROJECT: 16519971651997-1002
LOCATION: N 5014732.3; E 291136.6
MTM ZONE 10 (LAT. 45.272415; LONG. -79.674135)
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	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 NOTE: For additional abbreviations refer to list of abbreviations & symbols.										HYDRAULIC CONDUCTIVITY k, cm/s	Diametral Point Load Index (MPa)	RMC -Q AVG																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
							FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
								TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja				Jn	10°	10°	10°	2°	4°	8°																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
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DEPTH SCALE

1 : 60



GOLDER

LOGGED: SA

CHECKED: SEMP

SUD-RCK MTM ZN INC LAT/LONG S:\CLIENTS\MTM\1651997 AECOM_5015-E-0045_NE RETAINER02_DATA\GINT\1651997.GPJ GAL-MISS.GDT 3/2/18 TB



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

PROJECT <u>16519971651997-1002</u>		RECORD OF BOREHOLE No S-3		2 OF 4 METRIC																																
W.P. <u>291-96-01</u>		LOCATION <u>N 5014732.7; E 291152.6 MTM ZONE 10 (LAT. 45.272419; LONG. -79.673931)</u>		ORIGINATED BY <u>MA</u>																																
DIST <u> </u> 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			SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																															
--- CONTINUED FROM PREVIOUS PAGE ---																																				
213.6	SILTY CLAY to CLAY, trace sand Soft to firm Grey Wet		11	TO	PH																															
13.3	CLAYEY SILT to SILT Stiff to firm Grey Wet																																			
211.7	SILT, some sand, trace to some clay Very loose Grey Wet		12	SS	11																															
15.2																																				
209.5	SAND, some silt Compact Grey Wet		13	SS	2																															
17.4																																				
206.3	GRANITIC GNEISS (BEDROCK)		14	SS	12																															
20.6	Bedrock cored from 20.6 m to 23.7 m depth. For coring details see Record of Drillhole S-3.																																			
203.2			1	RC	REC 100%																															
23.7																																				
			2	RC	REC 100%																															
			3	RC	REC 100%																															

Continued Next Page

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

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTM\1651997 AECOM_5015-E-0045_NE RETAINER\02_DATA\GINT\1651997 GPJ GAL-MISS.GDT 3/2/18 TB

PROJECT <u>16519971651997-1002</u>		RECORD OF BOREHOLE No S-3				3 OF 4 METRIC	
W.P. <u>291-96-01</u>		LOCATION <u>N 5014732.7; E 291152.6 MTM ZONE 10 (LAT. 45.272419; LONG. -79.673931)</u>				ORIGINATED BY <u>MA</u>	
DIST <u> </u> 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 γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W		
						20	40	60	80	100						
	--- CONTINUED FROM PREVIOUS PAGE --- END OF BOREHOLE Note: 1. Water level at a depth of 14.5 m below ground surface (Elev. 212.4 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 and 0.1 m below ground surface (Elev. 226.8 m) on December 18, 2017.															

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SHEET 4 OF 4

DATUM: GEODETIC

DRILLING CONTRACTOR: Landcore Drilling

- Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
- Fault	FO - Foliation	CU - Curved	K - Slickensided	
- Shear	CO - Contact	UN - Undulating	SM - Smooth	NOTE: For additional abbreviations refer to list of abbreviations & symbols.
- Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
- Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

CHECKED: SEMP

S:\SUD-RCK MTM ZN INC\LAT\LONG S\CLIENTS\IMTO\1651997 AECOM 5015-E-0045 NE RETAINER\02 DATA\GIN\1651997.GPJ GAL-MISS.GDT 3/2/18 TB

PROJECT		16519971651997-1002		RECORD OF BOREHOLE No S-4		1 OF 3 METRIC											
W.P.		291-96-01		LOCATION		N 5014722.4; E 291173.5 MTM ZONE 10 (LAT. 45.272327; LONG. -79.673664)											
DIST		HWY 141		BOREHOLE TYPE		108 mm I.D. Hollow Stem Augers, NW Casing and Wash Boring											
DATUM		GEODETIC		DATE		March 13 to 15, 2017											
				ORIGINATED BY		MA											
				COMPILED BY		AC											
				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	20 40 60 80 100	20 40 60	W _p	W	W _L	γ	GR	SA	SI	CL
229.4	GROUND SURFACE																
0.0	ASPHALT (130 mm)																
0.1	Gravelly sand, some silt (FILL)		1	AS	-		229			o				21	63	(16)	
228.9	Brown Moist																
0.6	CONCRETE (150 mm)		2	SS	4		228										
	STYROFOAM White																
227.7																	
1.7	Gravelly sand to sand, some gravel (FILL)		3	SS	23		227			o				17	73	(10)	
	Compact Brown Moist to wet		4	SS	30/0.1												
	A 75 mm cobble encountered at 2.4 m depth.		-	RC	REC = 11%												
			5	SS	24		226										
225.3			6	SS	18												
4.1	CLAYEY SILT, trace to some sand						225										
	Very soft to stiff Grey Wet		7	SS	3												
							224										
			8	SS	1									0	9	63	28
							223										
							222										
222.3	CLAY		9	TO	PM												
7.1	Very soft to soft Grey						221										
							220										
	Varved below 8.8 m depth.		10	SS	WH												
							219										
			11	SS	WH												
							218										

Continued Next Page

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

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTM\1651997 AECOM_5015-E-0045_NE RETAINER\02_DATA\GINT\1651997 GPJ GAL-MISS.GDT 3/2/18 TB




2 OF 3 METRIC

CHECKED BY SEMP

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

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PROJECT <u>16519971651997-1002</u>		RECORD OF BOREHOLE No S-4				3 OF 3 METRIC											
W.P. <u>291-96-01</u>		LOCATION <u>N 5014722.4; E 291173.5 MTM ZONE 10 (LAT. 45.272327; LONG. -79.673664)</u>				ORIGINATED BY <u>MA</u>											
DIST <u> </u> HWY <u>141</u>		BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers, NW Casing and Wash Boring</u>				COMPILED BY <u>AC</u>											
DATUM <u>GEODETIC</u>		DATE <u>March 13 to 15, 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 γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W _p	W			W _L
	--- CONTINUED FROM PREVIOUS PAGE ---																
	SILT and SAND, trace clay Compact Grey Wet		17	SS	18												
	Attempted split-spoon at 24.9 m depth.					205											
204.5 24.9	END OF BOREHOLE SPLIT-SPOON REFUSAL AND REFUSAL TO FURTHER CASING ADVANCEMENT Note: 1. Water level at a depth of 2.3 m below ground surface inside casing (Elev. 227.1 m) upon completion of drilling.																

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S:\SUD-MTO 001 MTM ZN INC LAT\LONG S:\CLIENTS\MTO\1651997 AECOM 5015-E-0045 NERETAINER\02_DATA\GIN\1651997.GPJ GAL-MISS.GDT 3/2/18 TB


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

PROJECT		RECORD OF BOREHOLE No S-5				2 OF 2 METRIC										
16519971651997-1002																
W.P. 291-96-01		LOCATION N 5014727.7; E 291101.7 MTM ZONE 10 (LAT. 45.272373; LONG. -79.67458)				ORIGINATED BY MA										
DIST _____ HWY 141		BOREHOLE TYPE NW Casing and NQ Coring				COMPILED BY AC										
DATUM GEODETIC		DATE January 18, 2018				CHECKED BY SEMP										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---															
11.9	Sandy SILT, trace clay Compact Grey Wet		11	SS	18											0 22 73 5
217.3																
12.7	SAND, some gravel Dense Brown Wet															
215.6																
14.4	SAND and GRAVEL Dense Grey Wet		12	SS	42											
215.0																
15.0	END OF BOREHOLE SPLIT-SPOON REFUSAL Note: 1. Water level at ground surface inside casing (Elev. 230.0 m) upon completion of drilling as influenced by introduction of water for wash boring.		13	SS	34											

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PROJECT		16519971651997-1002		RECORD OF BOREHOLE No S-6		2 OF 2 METRIC															
W.P.		291-96-01		LOCATION		N 5014727.4; E 291118.5 MTM ZONE 10 (LAT. 45.272371; LONG. -79.674365)															
DIST		HWY 141		BOREHOLE TYPE		Solid Stem Augers, NW Casing and NQ Coring															
DATUM		GEODETIC		DATE		January 16 and 17, 2018															
						ORIGINATED BY MA															
						COMPILED BY AC															
						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					W _p W W _L 20 40 60			kN/m ³					
217.1	SILTY CLAY to CLAY Soft to firm Grey Wet																				
12.6	CLAYEY SILT Firm Grey Wet																				
216.1			11	SS	WR																
13.6	Sandy SILT, trace gravel, trace clay Compact Grey Wet		12	SS	10																
214.9																					
14.8	SAND, some clay, some silt, some gravel Compact Grey Wet		13	SS	28																
212.8																					
16.9	GRANITE (BEDROCK) Bedrock cored from 16.9 m to 19.9 m depth. For coring details see Record of Drillhole S-6.		1	RC	REC 98%																
209.8																					
19.9	END OF BOREHOLE Note: 1. Water level at a depth of 3.5 m below ground surface (Elev. 226.2 m) upon completion of drilling.																				

PROJECT: 16519971651997-1002
LOCATION: N 5014727.4; E 291118.5
MTM ZONE 10 (LAT. 45.272371; LONG. -79.674365)
INCLINATION: -90° AZIMUTH: —

RECORD OF DRILLHOLE: S-6

SHEET 1 OF 1
DRILLING DATE: January 16 and 17, 2018
DATUM: GEODETIC

DRILL RIG: CME 55
DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	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	φ ^o	ψ ^o	τ ^o			σ ^o																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
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17	CME 55 NQ Coring	NW	REFER TO PREVIOUS PAGE		212.8																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									

DEPTH SCALE

1 : 60



GOLDER

LOGGED: MA
CHECKED: SEMP

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

PROJECT <u>16519971651997-1002</u>		RECORD OF BOREHOLE No S-7		2 OF 2 METRIC													
W.P. <u>291-96-01</u>		LOCATION <u>N 5014716.0; E 291139.2 MTM ZONE 10 (LAT. 45.272268; LONG. -79.674101)</u>		ORIGINATED BY <u>SA</u>													
DIST <u> </u> HWY <u>141</u>		BOREHOLE TYPE <u>Portable Equipment, NW Casing and Wash Boring</u>		COMPILED BY <u>AC</u>													
DATUM <u>GEODETIC</u>		DATE <u>December 18 and 19, 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 γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L				
	--- CONTINUED FROM PREVIOUS PAGE ---																
214.5	CLAY, trace sand Very soft to firm Grey Wet		9	SS	1		215								----- ○		
13.2	SILT, some sand, trace clay Very loose to compact Grey Wet		10	SS	2		214								○		
							213										
211.9			11	SS	10		212										
15.8	END OF BOREHOLE SPLIT-SPOON REFUSAL Note: 1. Water level at a depth of 1.4 m below ground surface (Elev. 226.3 m) upon completion of drilling.																




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PROJECT 16519971651997-1002			RECORD OF BOREHOLE No S-8			1 OF 2 METRIC											
W.P. 291-96-01			LOCATION N 5014716.0; E 291156.8 MTM ZONE 10 (LAT. 45.272269; LONG. -79.673877)			ORIGINATED BY SA											
DIST _____ HWY 141			BOREHOLE TYPE Portable Equipment, NW Casing and Wash Boring			COMPILED BY AC											
DATUM GEODETIC			DATE December 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	20 40 60 80 100	20 40 60	W _p W W _L	γ	GR SA SI CL					
227.8	GROUND SURFACE																
0.0	ICE																
0.1	Sandy TOPSOIL, some silt, some gravel Stiff Brown Moist		1	SS	12		227										
226.9	SILTY CLAY to CLAY, trace sand Soft to firm Grey Wet		2	SS	4		226	4									
0.9			3	SS	1		225										
			4	SS	WH		224										
			5	SS	WH		223	4									
			6	SS	WH		222	3									
			7	TO	PH		221										
			8	SS	WH		220										
							219	3									
							218	2									
							217										
							216	3									

Continued Next Page

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

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MT01\1651997 AECOM_5015-E-0045_NE RETAINER02_DATA\GINT\1651997 GPJ GAL-MISS.GDT 3/2/18 TB

PROJECT		16519971651997-1002		RECORD OF BOREHOLE No S-8		2 OF 2 METRIC																
W.P.		291-96-01		LOCATION		N 5014716.0; E 291156.8 MTM ZONE 10 (LAT. 45.272269; LONG. -79.673877)																
DIST		HWY 141		BOREHOLE TYPE		Portable Equipment, NW Casing and Wash Boring																
DATUM		GEODETIC		DATE		December 20 and 21, 2017																
				ORIGINATED BY		SA																
				COMPILED BY		AC																
				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					W _p W W _L 20 40 60			kN/m ³						
213.0	SILTY CLAY to CLAY, trace sand Soft to firm Grey Wet		9	TO	PH		215															
								214														
				10	SS	WH																
14.8	SILT and SAND, trace clay Loose to compact Grey Wet						213															
				11	SS	10		212														
							211															
							210															
208.9	REFUSAL TO FURTHER CASING PENETRATION START OF DCPT		13	SS	11		209															
18.9								208														
207.5	END OF BOREHOLE DCPT REFUSAL (Hammer Bouncing)																					
20.3	Note: 1. Water level at a depth of 3.4 m below ground surface (Elev. 224.4 m) upon completion of drilling.																					

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

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



\\SUD-MTO\001\MTM\ZN\INC\LAT\LONG S:\CLIENTS\MTO\1651997\AECOM_5015-E-0045_NE RETAINER\02 DATA\GINT\1651997.GPJ GAL-MISS.GDT 3/2/18 TB

Continued Next Page

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

PROJECT <u>16519971651997-1002</u>		RECORD OF BOREHOLE No S-9				3 OF 3 METRIC								
W.P. <u>291-96-01</u>		LOCATION <u>N 5014727.4; E 291172.6 MTM ZONE 10 (LAT. 45.272372; LONG. -79.673676)</u>				ORIGINATED BY <u>MA</u>								
DIST <u> </u> HWY <u>141</u>		BOREHOLE TYPE <u>Solid Stem Augers, NW Casing and NQ Coring</u>				COMPILED BY <u>AC</u>								
DATUM <u>GEODETIC</u>		DATE <u>January 9, 10 and 17, 2018</u>				CHECKED BY <u>SEMP</u>								
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					W _p W W _L	
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			20 40 60		
203.9	SILT and SAND to SILTY SAND, trace clay Loose to compact Grey Wet		15	SS	12		205							
25.5	A 75 mm cobble encountered at 25.4 m depth.						204							
	GRANITIC GNEISS (BEDROCK)						203							
	Bedrock cored from 25.5 m to 28.7 m depth. For coring details see Record of Drillhole S-9.		1	RC	REC 100%		202							
200.7			2	RC	REC 100%		201							
28.7	END OF BOREHOLE													
	Note: 1. Water level at a depth of 3.2 m below ground surface (Elev. 226.2 m) upon completion of drilling.													

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SHEET 1 OF 1

DATUM: GEODETIC

DRILL RIG: CME 850

DRILLING CONTRACTOR: Landcore Drilling

CHECKED: SEMP

S:\SUD-RCK\MTM\ZN\INC\LAT\LONG S:\CLIENTS\MTM\1651997\AECOM 5015-E-0045 NE RETAINER\02 DATA\GINT\1651997.GPJ GAL-MISS.GDT 3/2/18 TB



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

PROJECT <u>16519971651997-1002</u>		RECORD OF BOREHOLE No S-10				2 OF 2 METRIC														
W.P. <u>291-96-01</u>		LOCATION <u>N 5014727.4; E 291190.3 MTM ZONE 10 (LAT. 45.272372; LONG. -79.67345)</u>				ORIGINATED BY <u>MR</u>														
DIST <u> </u> HWY <u>141</u>		BOREHOLE TYPE <u>76 mm I.D. Hollow Stem Augers, NW Casing and Wash Boring</u>				COMPILED BY <u>AC</u>														
DATUM <u>GEODETIC</u>		DATE <u>January 16, 2018</u>				CHECKED BY <u>SEMP</u>														
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20	40	60	80	100						20	40	60
<div style="display: flex; justify-content: space-between;"> <div> <p>--- CONTINUED FROM PREVIOUS PAGE ---</p> <p>SILTY CLAY to CLAY Firm Grey Wet</p> </div> <div style="border: 1px solid black; width: 100px; height: 100px; background: repeating-linear-gradient(45deg, transparent, transparent 2px, black 2px, black 4px);"></div> </div>																				
			9	TO	PH		217													
							216													
			10	SS	WR															
							215													
			11	SS	WR		214													
							213													
213.3 16.2	SILT, trace clay Compact Grey Wet		12	SS	10		212													
211.7 17.8	SILT and SAND, trace clay Compact Grey Wet		13	SS	11		211													
							210													
209.1 20.4	END OF BOREHOLE		14	SS	10															
<p>Note:</p> <p>1. Water level at a depth of 1.3 m below ground surface (Elev. 228.2 m) upon completion of drilling.</p>																				

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

PROJECT 16519971651997-1104			RECORD OF BOREHOLE No ST-1			2 OF 3 METRIC														
W.P. 291-96-01			LOCATION N 5014739.8; E 291120.2 MTM ZONE 10 (LAT. 45.272482; LONG. -79.674344)			ORIGINATED BY SA														
DIST HWY 141			BOREHOLE TYPE NW Casing, Wash Boring and NQ Coring			COMPILED BY AC														
DATUM GEODETIC			DATE March 16, 2017			CHECKED BY SEMP														
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			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		ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL			
							20 40 60 80 100	20 40 60 80 100	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60						
--- CONTINUED FROM PREVIOUS PAGE ---																				
213.7	CLAY Soft to firm Grey Wet		10	SS	WH		215										NP	0 25 72 3		
213.4	Sandy SILT, trace clay Very loose Grey Wet		11	SS	1		214													
212.3	SAND, trace to some gravel, some silt Dense Grey Wet		12	SS	32		213										12 73 (15)			
212.3			13	SS	38		212													
209.7			13	SS	38		211													
209.7	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%		210										RQD = 94%			
209.7			2	RC	REC 100%		209													
206.7			2	RC	REC 100%		208													
206.7	END OF BOREHOLE Note: 1. Water level at a depth of 4.7 m below ground surface (Elev. 222.4 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.						207										RQD = 83%			

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INCLINATION: -90° AZIMUTH: —

DRILLING CONTRACTOR: Landcore Drilling





DATUM: GEODETIC

1 : 60



CHECKED: SEMP

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

PROJECT 16519971651997-1002		RECORD OF BOREHOLE No ST-2				2 OF 3 METRIC									
W.P. 291-96-01		LOCATION N 5014739.3; E 291130.4 MTM ZONE 10 (LAT. 45.272478; LONG. -79.674214)				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 γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					W _p W W _L		
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			20 40 60			
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								211							
			14	SS	32			210							
209.2	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							
17.5								208							
			2	RC	REC 100%			207							
205.8			3	RC	REC 100%			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 below ground surface as follows: 0.4 m (Elev. 226.3 m), April 16, 2017 0.9 m (Elev. 225.8 m), December 18, 2017 0.7 m (Elev. 226.0 m), January 11, 2018.														

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INCLINATION: -90° AZIMUTH: —

DRILLING CONTRACTOR: Landcore Drilling

DATUM: GEODETIC

1 : 60



CHECKED: SEMP

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PROJECT <u>16519971651997-1104</u>			RECORD OF BOREHOLE No ST-3			1 OF 4 METRIC		
W.P. <u>291-96-01</u>			LOCATION <u>N 5014739.2; E 291154.8 MTM ZONE 10 (LAT. 45.272477; LONG. -79.673903)</u>			ORIGINATED BY <u>MA</u>		
DIST <u> </u> 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			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
226.8	GROUND SURFACE							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L WATER CONTENT (%)
0.0	Silty TOPSOIL, some clay, some sand		1	AS	-		226	
226.5	Brown Moist							
226.2	BOULDER							
0.6	SILTY CLAY to CLAY, trace sand		2	SS	2		225	
	Soft to firm							
	Grey		3	SS	WH		224	
	Wet							
	Poor recovery in Samples 2 and 3.						223	
			4	SS	WH		222	
			5	TO	PH		221	
			6	SS	WH		220	
			7	SS	WH		219	
			8	SS	WH		218	
			9	TO	PH		217	
							216	
							215	

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

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2 OF 4 METRIC

CHECKED BY SEMP

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

PROJECT <u>16519971651997-1104</u>		RECORD OF BOREHOLE No ST-3				3 OF 4 METRIC											
W.P. <u>291-96-01</u>		LOCATION <u>N 5014739.2; E 291154.8 MTM ZONE 10 (LAT. 45.272477; LONG. -79.673903)</u>				ORIGINATED BY <u>MA</u>											
DIST <u> </u> 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 LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W			W _L
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>					WATER CONTENT (%)					
	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.																

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PROJECT: 16519971651997-1104
LOCATION: N 5014739.2; E 291154.8
MTM ZONE 10 (LAT. 45.272477; LONG. -79.673903)
INCLINATION: -90° AZIMUTH: —

RECORD OF DRILLHOLE: ST-3

SHEET 4 OF 4
DATUM: GEODETIC

DRILLING DATE: March 20 and 21, 2017

DRILL RIG: CME 850

DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR FLUSH % RETURN	RECOVERY			R.Q.D. %	FRACT INDEX METRES	DISCONTINUITY DATA					HYDRAULIC CONDUCTIVITY k, cm/s			Diametral Point Load Index (MPa)	RMC -Q AVG																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
	JN	FLT						SHR	VN	CJ			BD	FO	CO	OR	CL	PL	CU	UN			ST	IR	PO	K	SM	Ro	MB	BR																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
																															TOTAL CORE %	SOLID CORE %	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	10 m	10 cm	10 mm																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
21	CME 55 NQ Coring	HQ	REFER TO PREVIOUS PAGE		206.7																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			

DEPTH SCALE

1 : 60



LOGGED: MA
CHECKED: SEMP



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


PROJECT 16519971651997-1104		RECORD OF BOREHOLE No ST-4		2 OF 4 METRIC	
W.P. 291-96-01		LOCATION N 5014739.6; E 291164.5 MTM ZONE 10 (LAT. 45.272481; LONG. -79.673779)		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	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	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	20	40	60					
--- CONTINUED FROM PREVIOUS PAGE ---																				
	CLAY, trace sand Soft to firm Grey Wet		10	SS	WH															
				11	SS	WH														
				12	SS	WH														
							211		4											
							210													
							209													
			13	SS	21												0 82 9 9			
							208													
							207													
							206													
210.1 16.8	Silty SAND to SAND Compact Grey Wet																			

Continued Next Page

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

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PROJECT <u>16519971651997-1104</u>				RECORD OF BOREHOLE No ST-4				3 OF 4 METRIC									
W.P. <u>291-96-01</u>		LOCATION <u>N 5014739.6; E 291164.5 MTM ZONE 10 (LAT. 45.272481; LONG. -79.673779)</u>				ORIGINATED BY <u>MA</u>											
DIST <u> </u> 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 CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	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 ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> W_p W W_L </div>					
202.7			3	RC	REC												RQD = 0%
24.2	END OF BOREHOLE Note: 1. Water level at ground surface (Elev. 226.9 m) inside casing after sitting overnight, upon completion of drilling. 2. Vane at 1.7 m depth taken from separate borehole located 2.5 m east of Borehole ST-4.																

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INCLINATION: -90° AZIMUTH: —

DRILLING CONTRACTOR: Landcore Drilling

DATUM: GEODETIC

[illegible]

1 : 60



CHECKED: SEMP

PROJECT 16519971651997-1002			RECORD OF BOREHOLE No ST-5			1 OF 2 METRIC														
W.P. 291-96-01			LOCATION N 5014749.7; E 291083.3 MTM ZONE 10 (LAT. 45.272571; LONG. -79.674815)			ORIGINATED BY SA														
DIST HWY 141			BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing, Wash Boring			COMPILED BY AC														
DATUM GEODETIC			DATE January 9 and 10, 2018			CHECKED BY SEMP														
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			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		ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	γ	GR	SA	SI	CL			
227.5	GROUND SURFACE		A																	
0.0	PEAT (Fiborous) Black Wet		1	SS	4															
0.1	SILTY CLAY, trace to some sand Soft to stiff Brown to grey Moist to wet Trace organics in Sample 1		B																	
			2	SS	9															
			3	SS	5															
			4	SS	1															
	Clayey silt lens encountered in Sample 4																			
			5	TO	PM															
			6	SS	WH															
			7	TO	PH															
218.1			A																	
9.4	SILTY SAND to SAND, trace to some clay Compact Grey Wet		8	SS	9															
			B																	
			9	SS	13															
216.0	REFUSAL BEDROCK/BOULDER																			
215.7																				
11.8																				

Continued Next Page

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

SUD-MTO 001 MTM ZNI INC LAT/LONG S:\CLIENTS\MTM\1651997 AECOM_5015-E-0045_NE RETAINER\02_DATA\GINT\1651997 GPJ GAL-MISS.GDT 3/2/18 TB

PROJECT <u>16519971651997-1002</u>		RECORD OF BOREHOLE No ST-5				2 OF 2 METRIC											
W.P. <u>291-96-01</u>		LOCATION <u>N 5014749.7; E 291083.3 MTM ZONE 10 (LAT. 45.272571; LONG. -79.674815)</u>				ORIGINATED BY <u>SA</u>											
DIST <u> </u> HWY <u>141</u>		BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers, NW Casing, Wash Boring</u>				COMPILED BY <u>AC</u>											
DATUM <u>GEODETIC</u>		DATE <u>January 9 and 10, 2018</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 W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p W W _L				
	--- CONTINUED FROM PREVIOUS PAGE ---																
	END OF BOREHOLE Note: 1. Casing seated 0.3 m into probable bedrock or boulder 2. Water level at a depth of 0.2 m below ground surface (Elev. 227.3 m) upon completion of drilling inside casing, after being left overnight (January 9, 2018). 3. Advanced DCPT located 3.0 m east of Borehole ST-5. DCPT refusal (Hammer Bouncing) at 10.9 m depth (Elev. 216.6 m).																

SUD-MTO 001 MTM ZNI INC LAT/LONG S:\CLIENTS\MTM\1651997 AECOM_5015-E-0045_NE RETAINER\02_DATA\GINT\1651997 GPJ GAL-MISS.GDT 3/2/18 TB

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

PROJECT 16519971651997-1002		RECORD OF BOREHOLE No ST-6				2 OF 2 METRIC											
W.P. 291-96-01		LOCATION N 5014750.3; E 291117.9 MTM ZONE 10 (LAT. 45.272577; LONG. -79.674374)				ORIGINATED BY SA											
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 January 10 and 11, 2018				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 kN/m³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60				
214.4 12.7	--- CONTINUED FROM PREVIOUS PAGE --- SILTY CLAY to CLAY Soft to firm Grey Wet No recovery in Sample 9, unable to push Shelby past 12.7 m depth SILT and SAND to SAND, trace clay Compact Grey Wet Approximately 0.3 m of heave encountered at 15.2 m depth.		9	TO	PH		215										
			10	SS	13		214										
			11	SS	35		213									0	58 39 3
210.5 16.6	GRANITIC GNEISS (BEDROCK) Bedrock cored from 16.6 m to 20.2 m depth. For coring details see Record of Drillhole ST-6.		1	RC	REC 100%		210										RQD = 100%
			2	RC	REC 96%		209										RQD = 100%
			3	RC	REC 100%		208										RQD = 80%
206.9 20.2	END OF BOREHOLE Note: 1. Water level at a depth of 0.1 m below ground surface (Elev. 227.0 m) upon completion of drilling.						207										

PROJECT: 16519971651997-1002
LOCATION: N 5014750.3; E 291117.9
MTM ZONE 10 (LAT. 45.272577; LONG. -79.674374)
INCLINATION: -90° AZIMUTH: —

RECORD OF DRILLHOLE: ST-6

SHEET 1 OF 1
DRILLING DATE: January 10 and 11, 2018
DATUM: GEODETIC

DRILL RIG: CME 55
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 NOTE: For additional abbreviations refer to list of abbreviations & symbols.																																																																																																																																																																																																																																																																
								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	φ _o	ψ _o	τ _o	σ _o																																																																																																																																																																																																																																																																																													
	FLUSH	80 60 40 20								80 60 40 20	80 60 40 20									15 10 5 2	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02	0.5 0.1 0.05 0.02

UCS = 114 MPa

DEPTH SCALE

1 : 60



GOLDER

LOGGED: SA
CHECKED: SEMP

PROJECT <u>16519971651997-1002</u>		RECORD OF BOREHOLE No ST-7		1 OF 4 METRIC									
W.P. <u>291-96-01</u>		LOCATION <u>N 5014750.1; E 291167.1 MTM ZONE 10 (LAT. 45.272576; LONG. -79.673747)</u>		ORIGINATED BY <u>MR</u>									
DIST <u> </u> HWY <u>141</u>		BOREHOLE TYPE <u>76 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>January 9, 10 and 11, 2018</u>		CHECKED BY <u>SEMP</u>									
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE		"N" VALUES	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED		WATER CONTENT (%) W _p W W _L				
227.6 0.0	GROUND SURFACE Silty clay, trace sand, trace gravel, trace organics (FILL) Stiff Brown Wet		1	AS	-		227					NP OC=3.4%	
226.1 1.5	Sandy SILT, trace organics (peat/rootlets), trace gravel Very loose Brown to black Wet		2	SS	1		226						
	Wood encountered in Sample 3.		3	SS	1		225						
223.8 3.8	CLAYEY SILT, trace sand Very soft Grey Wet		4	SS	WR		224					96.2	
							223						
222.7 4.9	SILTY CLAY to CLAY Soft Grey Wet						222						
			5	TO	PH		221						
							220						
			6	SS	WR		219					87.9	
							218						
			7	TO	PH		217					15.1	
							216						
	Varved below 9.9 m depth		8	SS	WR								
			9	SS	WR								

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTM\1651997 AECOM_5015-E-0045_NE RETAINER\02_DATA\GINT\1651997 GPJ GAL-MISS.GDT 3/2/18 TB

Continued Next Page



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

PROJECT <u>16519971651997-1002</u>			RECORD OF BOREHOLE No ST-7			2 OF 4 METRIC														
W.P. <u>291-96-01</u>			LOCATION <u>N 5014750.1; E 291167.1 MTM ZONE 10 (LAT. 45.272576; LONG. -79.673747)</u>			ORIGINATED BY <u>MR</u>														
DIST <u> </u> HWY <u>141</u>			BOREHOLE TYPE <u>76 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>January 9, 10 and 11, 2018</u>			CHECKED BY <u>SEMP</u>														
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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) Wp — W — Wl			γ	GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100												
	SILTY CLAY to CLAY Soft Grey Wet							20 40 60 80 100												
213.6			10	TO	PH			215												
14.0	SILT and SAND to SAND, some silt, trace clay Loose Grey Wet		11	SS	8			214												
			12	SS	7			213											0 58 30 2	
			13	SS	7			212												
			14	SS	6			211												
			15	SS	6			210												
			16	SS	5			209												
			17	SS	6			208											0 82 16 2	
								207												
								206												
								205												
								204												

Continued Next Page

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

SUD-MTO 001 MTM ZNI INC LAT/LONG S:\CLIENTS\MT01\1651997 AECOM_5015-E-0045_NE RETAINER02_DATA\GINT\1651997 GPJ GAL-MISS.GDT 3/2/18 TB

PROJECT <u>16519971651997-1002</u>		RECORD OF BOREHOLE No ST-7				3 OF 4 METRIC						
W.P. <u>291-96-01</u>		LOCATION <u>N 5014750.1; E 291167.1 MTM ZONE 10 (LAT. 45.272576; LONG. -79.673747)</u>				ORIGINATED BY <u>MR</u>						
DIST <u> </u> HWY <u>141</u>		BOREHOLE TYPE <u>76 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>January 9, 10 and 11, 2018</u>				CHECKED BY <u>SEMP</u>						
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa		WATER CONTENT (%)		
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	W _p W W _L		
202.5	SILT and SAND to SAND, some silt, trace clay Loose Grey Wet						203					
25.1	GRANITIC GNEISS (BEDROCK) Bedrock cored from 25.1 m depth to 29 m depth. For coring details see Record of Drillhole ST-7.		1	RC	REC 100%		202					RQD = 60%
			2	RC	REC 100%		201					RQD = 95%
							200					
			3	RC	REC 100%		199					RQD = 100%
198.6	END OF BOREHOLE											
29.0	Note: 1. Water level at a depth of 0.7 m below ground surface (Elev. 226.9 m) upon completion of drilling, inside casing after being left overnight (January 10, 2018).											

SUD-MTO 001 MTM ZNI INC LAT/LONG S:\CLIENTS\MTM\1651997 AECOM_5015-E-0045_NE RETAINER\02_DATA\GINT\1651997 GPJ GAL-MISS.GDT 3/2/18 TB

INCLINATION: -90° AZIMUTH: —

DRILLING CONTRACTOR: Landcore Drilling

DATUM: GEODETIC

1 : 60



CHECKED: SEMP

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

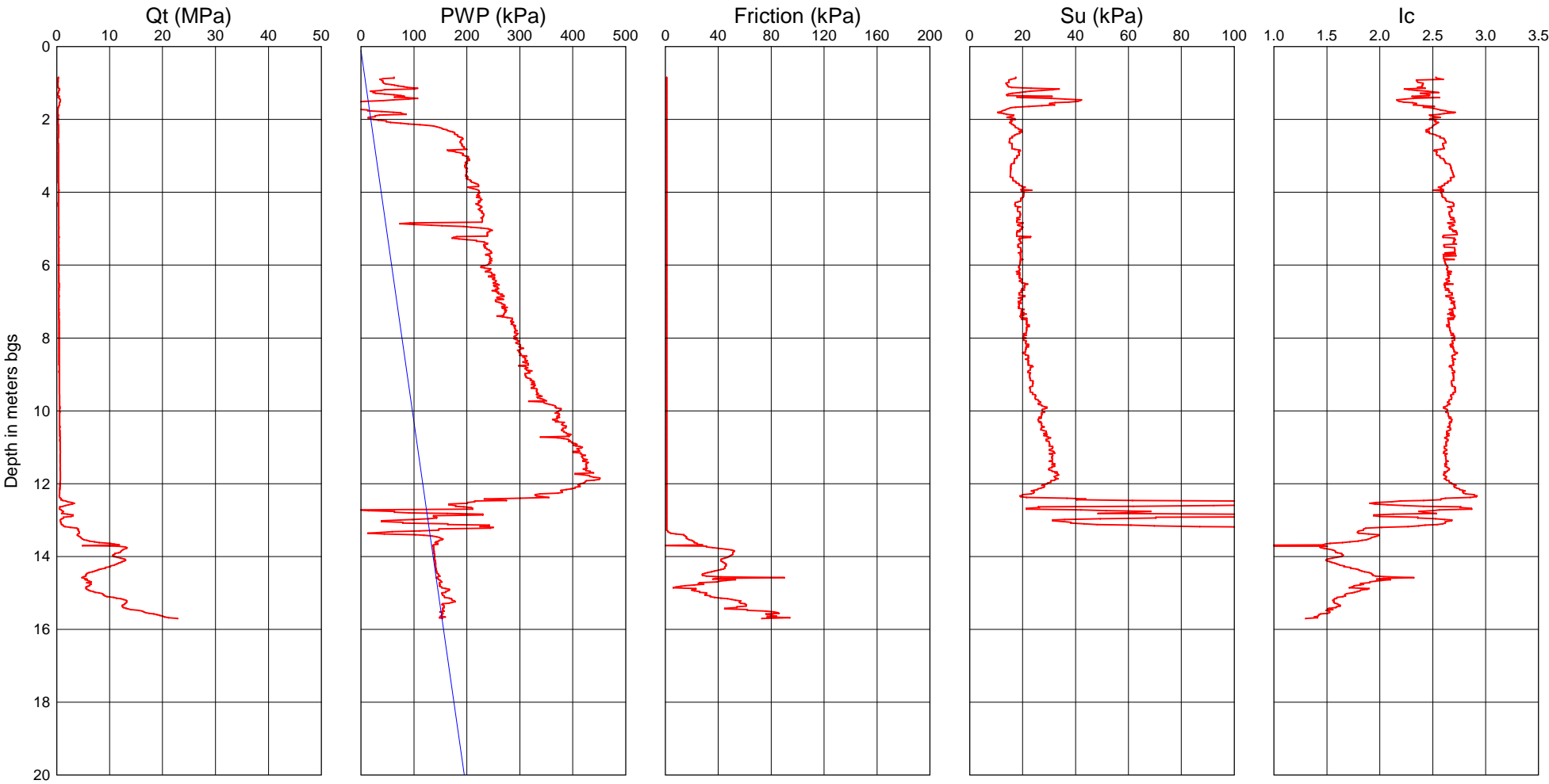
PROJECT <u>16519971651997-1002</u>		RECORD OF BOREHOLE No ST-8		2 OF 2 METRIC															
W.P. <u>291-96-01</u>		LOCATION <u>N 5014746.6; E 291219.9 MTM ZONE 10 (LAT. 45.272545; LONG. -79.673074)</u>		ORIGINATED BY <u>MR</u>															
DIST <u> </u> HWY <u>141</u>		BOREHOLE TYPE <u>76 mm I.D. Hollow Stem Augers, NW Casing and Wash Boring</u>		COMPILED BY <u>AC</u>															
DATUM <u>GEODETIC</u>		DATE <u>January 11 and 15, 2018</u>		CHECKED BY <u>SEMP</u>															
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	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 ---								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED											
212.6	CLAY Soft to firm Grey Wet		12	SS	WR		215												
14.9							214												
							213												
211.2	Sandy SILT Loose Grey Wet		14	SS	5		212												
16.3							211												
							210												
	Gravelly SILT and SAND to SAND, trace clay Very loose to compact Grey Wet		15	SS	18		209												
							208												
							207												
206.7	END OF BOREHOLE SPLIT-SPOON REFUSAL																		
20.8	Note: 1. Water level at ground surface (Elev. 227.5 m) upon completion of drilling.																		

Cone Penetration Test - CPT-1

Test Date : 1/26/2018
Location : 2.3 m East of Borehole ST-6

Operator : Golder Associates Ltd.

Ground Surf. Elev. : 227.10
Water Table Depth : 0.10



Qt normalized for
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$
 $N_k = 15.5$
 $\gamma = 15.1 \text{ kN/m}^3$

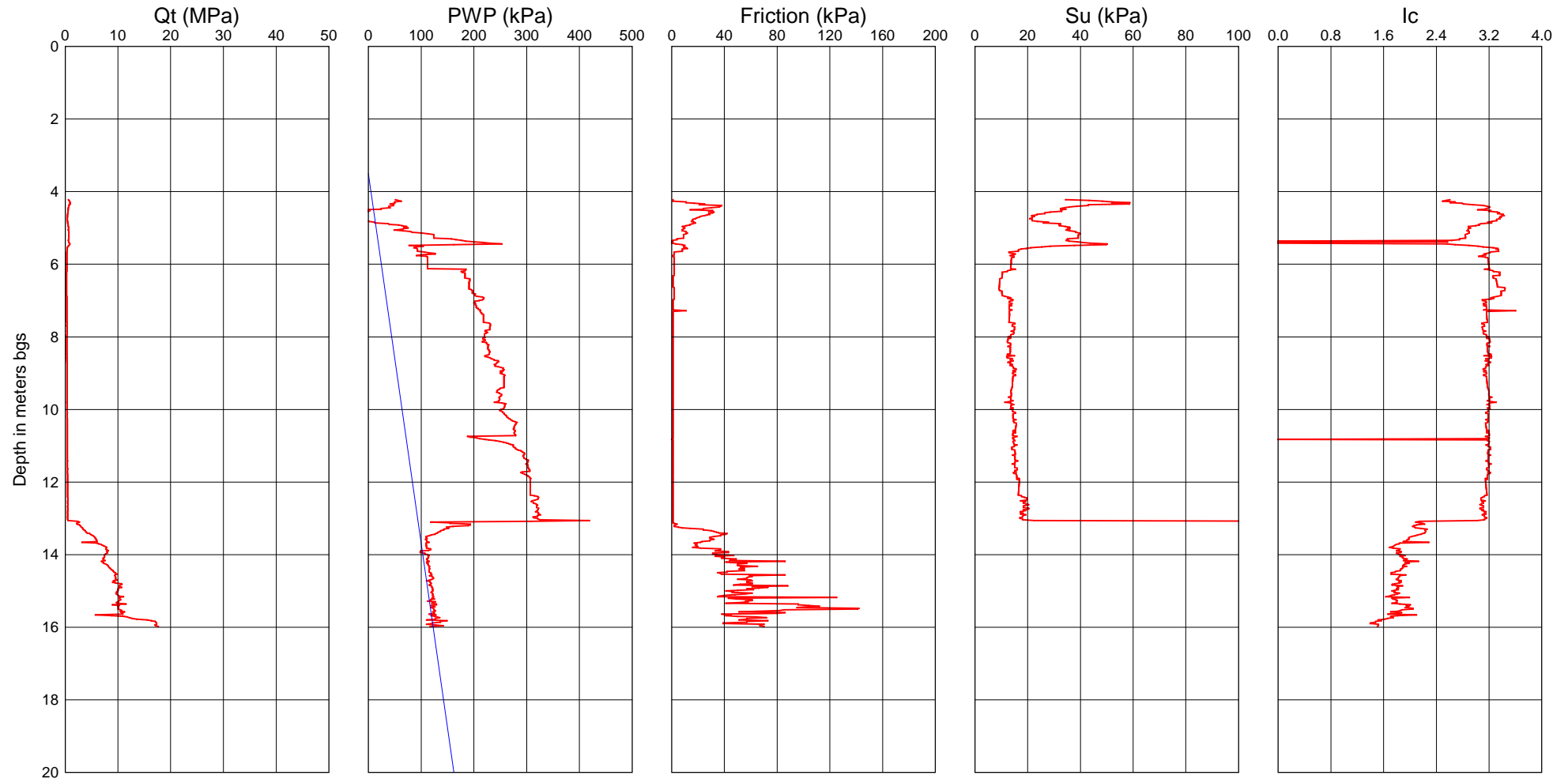
After Robertson and (Fear) Wride (1998)
 $I_c < 1.31$ - Gravelly sands
 $1.31 < I_c < 2.05$ - Clean to silty sand
 $2.05 < I_c < 2.60$ - Silty sand to sandy silt
 $2.60 < I_c < 2.95$ - Clayey silt to silty clay
 $2.95 < I_c < 3.60$ - Clays

Cone Penetration Test - CPT-2

Test Date : 1/25/2018
Location : 2 m West of Borehole S-6

Operator : Golder Associates Ltd.

Ground Surf. Elev. : 229.70
Water Table Depth : 3.50



Qt normalized for
unequal end area effects

$S_u = (Q_t - \sigma_{v0}) / N_k$
 $N_k = 15.5$
 $\gamma = 15.1 \text{ kN/m}^3$

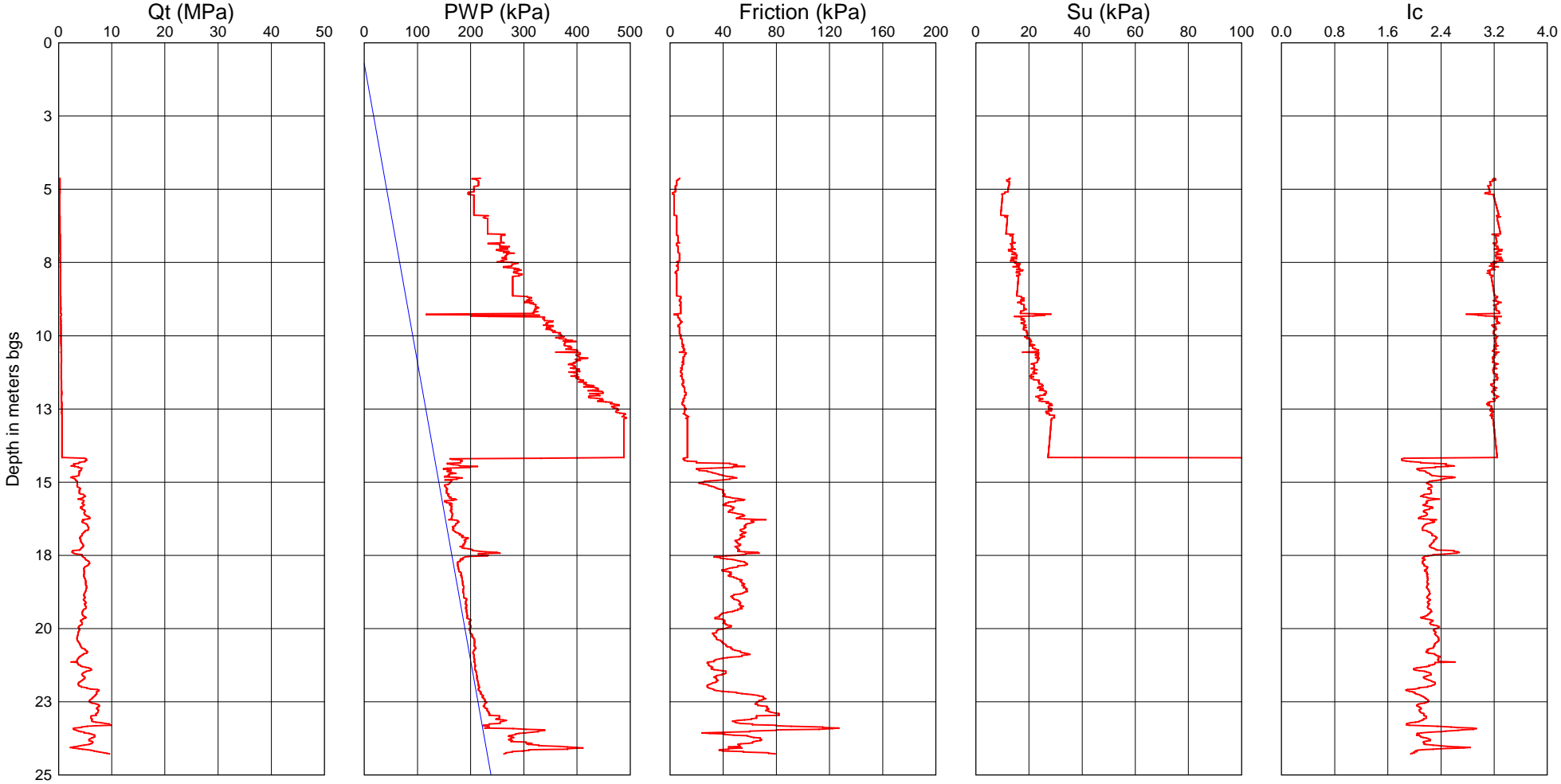
After Robertson and (Fear) Wride (1998)
 $I_c < 1.31$ - Gravelly sands
 $1.31 < I_c < 2.05$ - Clean to silty sand
 $2.05 < I_c < 2.60$ - Silty sand to sandy silt
 $2.60 < I_c < 2.95$ - Clayey silt to silty clay
 $2.95 < I_c < 3.60$ - Clays

Cone Penetration Test - CPT-3

Test Date : 1/19/2018
Location : 2 m East of Borehole ST-7

Operator : Golder Associates Ltd.

Ground Surf. Elev. : 227.60
Water Table Depth : 0.70



Qt normalized for
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$
 $N_k = 15.5$
 $\gamma = 15.1 \text{ kN/m}^3$

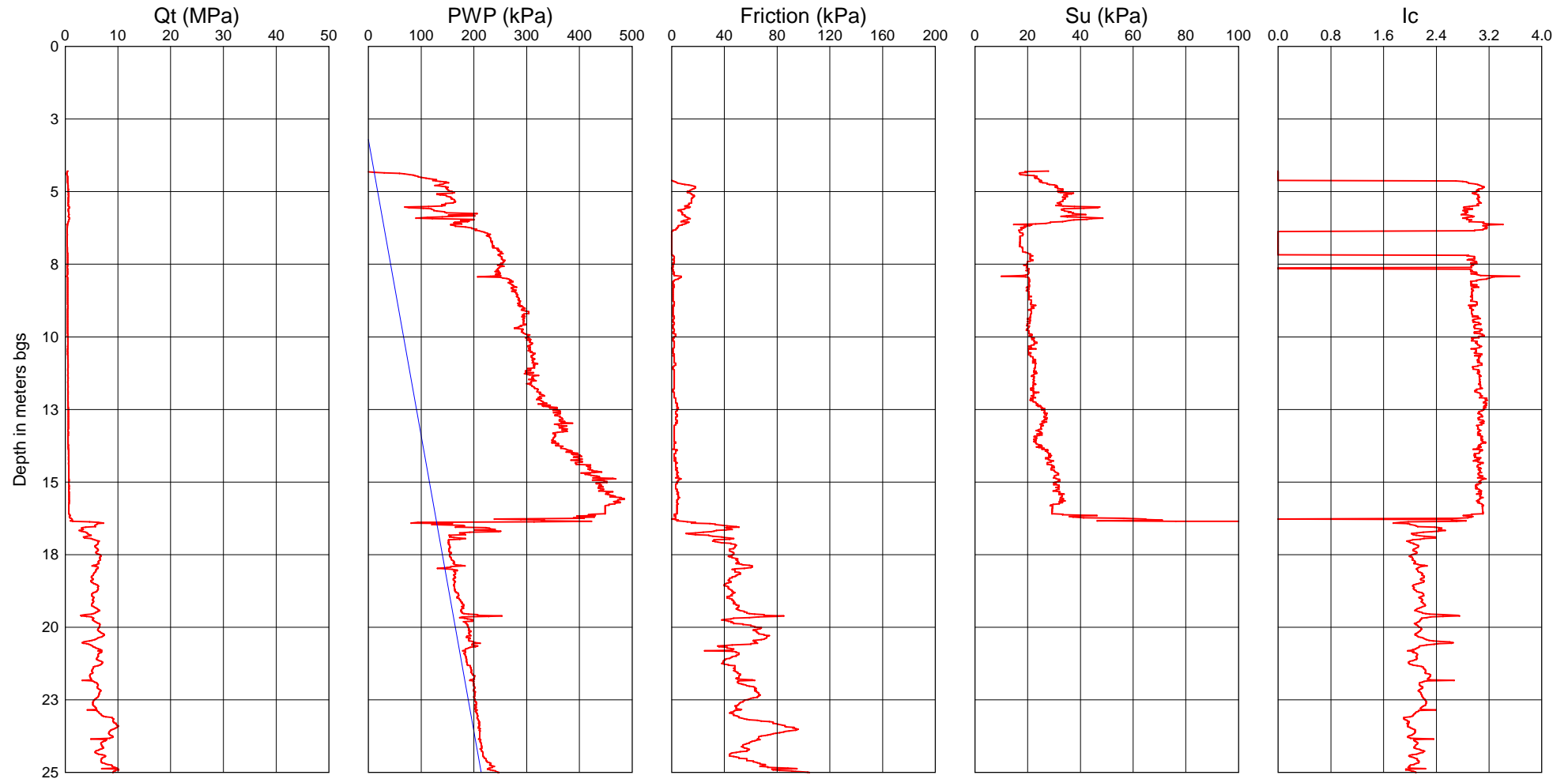
After Robertson and (Fear) Wride (1998)
 $I_c < 1.31$ - Gravelly sands
 $1.31 < I_c < 2.05$ - Clean to silty sand
 $2.05 < I_c < 2.60$ - Silty sand to sandy silt
 $2.60 < I_c < 2.95$ - Clayey silt to silty clay
 $2.95 < I_c < 3.60$ - Clays

Cone Penetration Test - CPT-4

Test Date : 1/24/2018
Location : 2 m East of Borehole S-9

Operator : Golder Associates Ltd.

Ground Surf. Elev. : 229.40
Water Table Depth : 3.20



Qt normalized for
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$
 $N_k = 15.5$
 $\gamma = 15.1 \text{ kN/m}^3$

After Robertson and (Fear) Wride (1998)
 $I_c < 1.31$ - Gravelly sands
 $1.31 < I_c < 2.05$ - Clean to silty sand
 $2.05 < I_c < 2.60$ - Silty sand to sandy silt
 $2.60 < I_c < 2.95$ - Clayey silt to silty clay
 $2.95 < I_c < 3.60$ - Clays

APPENDIX B

Laboratory Test Results

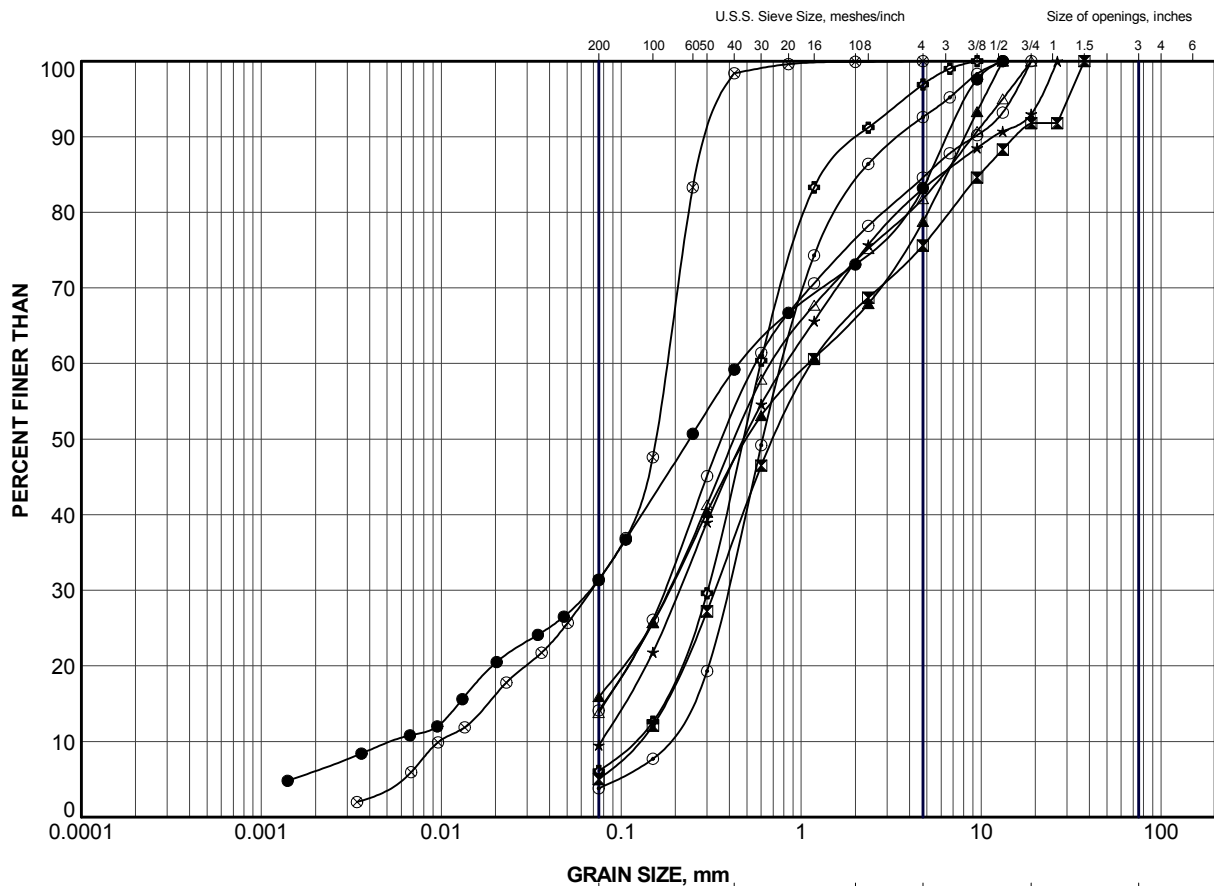
Location	Parameter	Units	Result
West Abutment (S-1, Sa #6)	Chloride (CL)	ug/g	410
	Sulphate (SO4)	ug/g	ND
	Conductivity (EC)	umho/cm	771
	Resistivity	ohm-cm	1,300
	pH	n/a	6.03
East Abutment (S-4, Sa #7)	Chloride (CL)	ug/g	830
	Sulphate (SO4)	ug/g	ND
	Conductivity (EC)	umho/cm	1540
	Resistivity	ohm-cm	650
	pH	n/a	5.69
TMB West Abutment (ST-2, Sa #4)	Chloride (CL)	ug/g	160
	Sulphate (SO4)	ug/g	ND
	Conductivity (EC)	umho/cm	337
	Resistivity	ohm-cm	3000
	pH	n/a	7.62
TMB East Abutment (ST-3, Sa #2)	Chloride (CL)	ug/g	49
	Sulphate (SO4)	ug/g	ND
	Conductivity (EC)	umho/cm	136
	Resistivity	ohm-cm	7300
	pH	n/a	6.01

Notes:

- Samples obtained on March 20, 2017.
- N.D = Not Detected
- Analytical testing carried out by Maxxam.

PROJECT					
HIGHWAY 141 SHADOW RIVER BRIDGE					
TITLE					
SUMMARY OF ANALYTICAL SOIL TESTING					
PROJECT No. 1651997			FILE No. ----		
DESIGN	LP	Feb 18	SCALE	NTS	REV.
CADD	-- --		TABLE B1		
CHECK	AC	Feb 18			
REVIEW					





CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	S-1	1	229.4
⊠	S-1	4	227.1
▲	S-4	1	229.1
★	S-4	4	227.1
⊙	S-6	2	227.9
⊕	S-9	2	227.6
○	S-10	3	225.4
△	ST-2	2	225.7
⊗	ST-8	3	225.7

PROJECT

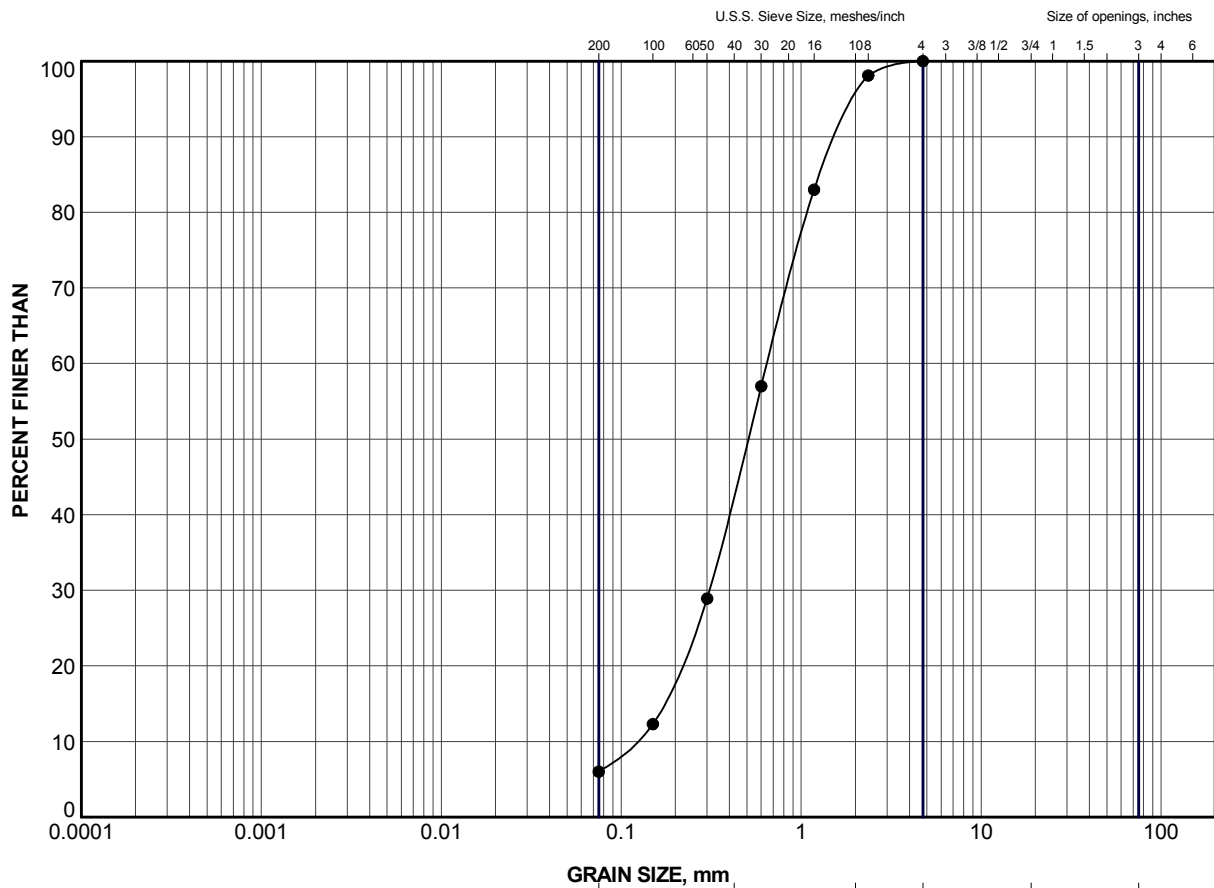
HIGHWAY 141
SHADOW RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION
SILTY SAND to GRAVELLY SAND (FILL)



PROJECT No. 1651997			FILE No. 1651997.GPJ		
DRAWN	TB	Mar 2018	SCALE	N/A	REV.
CHECK	AC/SEMP	Mar 2018	FIGURE B1		
APPR	JMAC	Mar 2018			



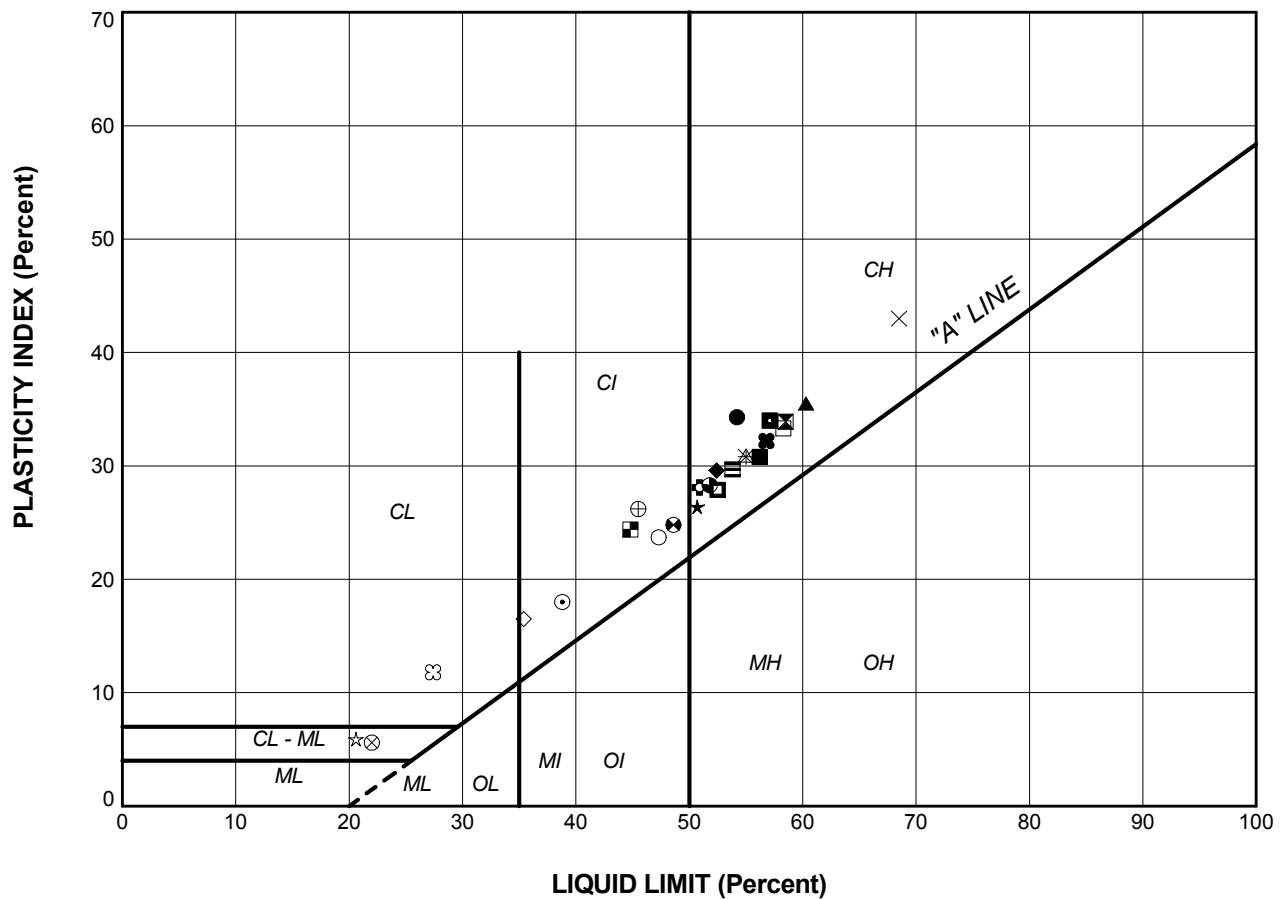
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	ST-8	6	223.4

PROJECT					
HIGHWAY 141 SHADOW RIVER BRIDGE					
TITLE					
GRAIN SIZE DISTRIBUTION SAND					
PROJECT No.		1651997		FILE No. 1651997.GPJ	
DRAWN	TB	Mar 2018	SCALE	N/A	REV.
CHECK	AC/SEMP	Mar 2018	FIGURE B2		
APPR	JMAC	Mar 2018			



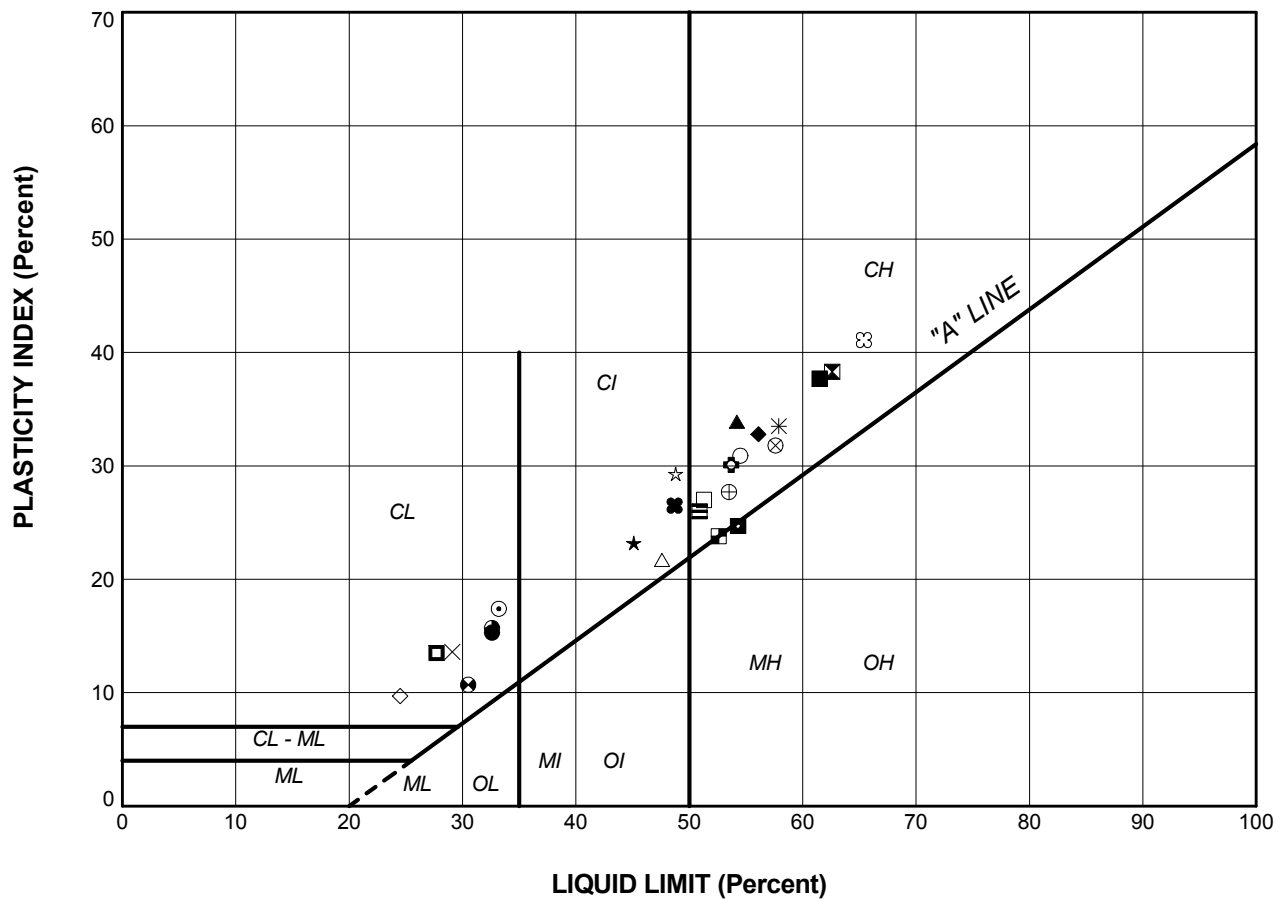


LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	S-1	6	54.2	19.9	34.3
⊠	S-1	9	58.5	24.6	33.9
▲	S-1	10	60.3	24.8	35.5
★	S-1	11	50.7	24.3	26.4
⊕	S-2	2	38.8	20.8	18.0
⊗	S-2	5	50.9	22.8	28.1
○	S-2	8	47.3	23.6	23.7
△	S-2	10	55.0	24.1	30.9
⊗	S-2	11	22.0	16.4	5.6
⊕	S-3	3	45.5	19.3	26.2
□	S-3	6	58.3	25.0	33.3
⊗	S-3	9	48.6	23.8	24.8
⊕	S-3	10	51.8	23.5	28.3
★	S-3	12	20.6	14.7	5.9
⊗	S-4	8	27.4	15.6	11.8
■	S-4	10	56.2	25.4	30.8
◆	S-4	14	52.4	22.8	29.6
◇	ST-1	2	35.4	18.9	16.5
×	ST-1	5	68.5	25.5	43.0
⊗	ST-1	8	56.8	24.6	32.2
■	ST-1	10	57.1	23.1	34.0
*	ST-2	5	55.0	24.2	30.8
□	ST-2	7	52.5	24.6	27.9
⊠	ST-2	8	44.8	20.4	24.4
⊠	ST-2	11	53.8	24.1	29.7

PROJECT					
HIGHWAY 141 SHADOW RIVER BRIDGE					
TITLE					
PLASTICITY CHART CLAYEY SILT to CLAY					
PROJECT No.		1651997		FILE No.	
DRAWN		TB		Mar 2018	
CHECK		AC/SEMP		Mar 2018	
APPR		JMAC		Mar 2018	
SCALE		N/A		REV.	
FIGURE B3.1					

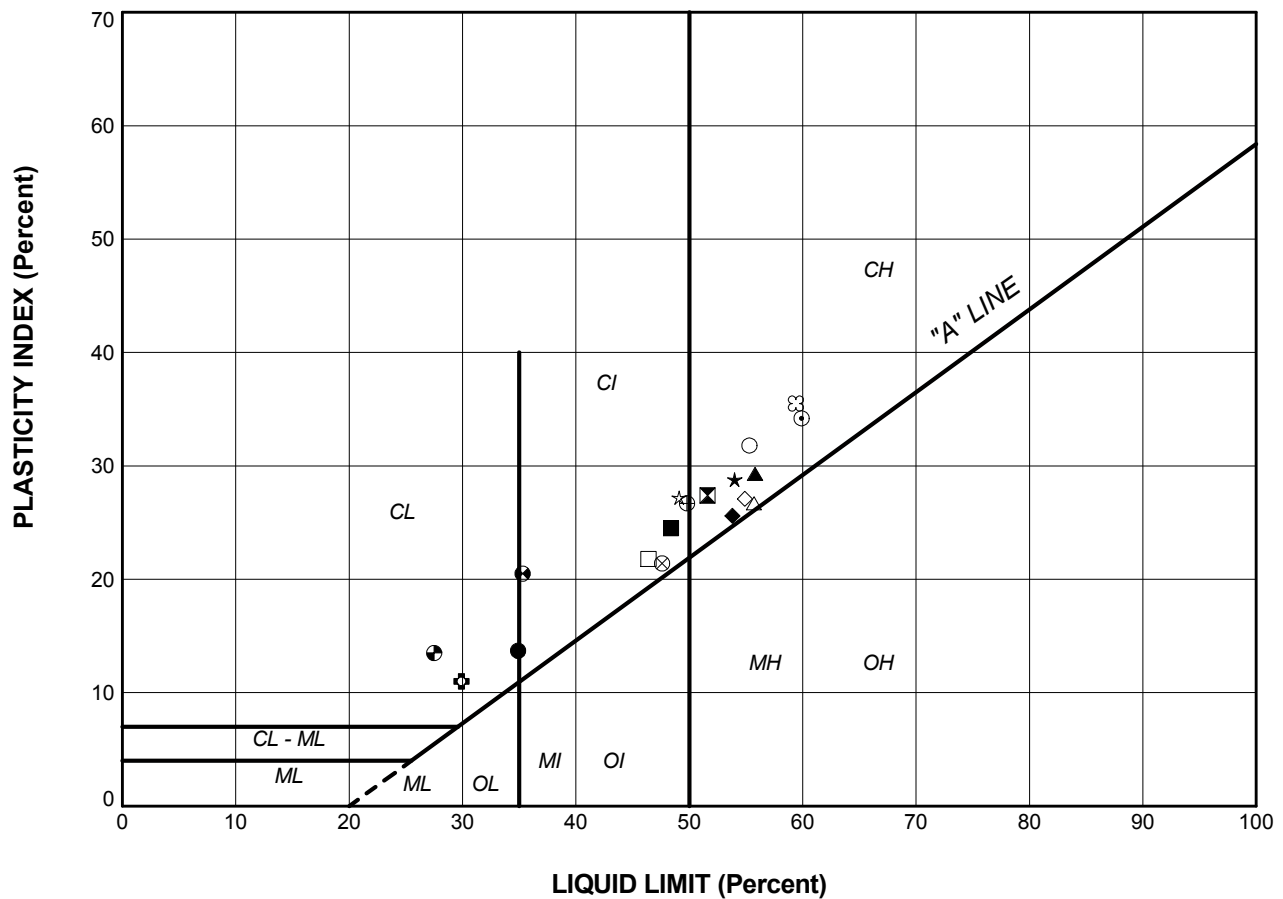




LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	S-7	3	32.6	17.3	15.3
⊠	S-7	7	62.6	24.3	38.3
▲	S-7	9	54.2	20.3	33.9
★	S-8	2	45.1	21.9	23.2
⊙	S-8	4	33.2	15.8	17.4
⊕	S-8	8	53.7	23.6	30.1
○	S-9	8	54.5	23.6	30.9
△	S-9	10	47.6	25.9	21.7
⊗	ST-3	6	57.6	25.8	31.8
⊕	ST-3	8	53.5	25.8	27.7
□	ST-3	9	51.3	24.3	27.0
⊗	ST-3	11	30.5	19.8	10.7
⊕	ST-4	2	32.6	16.9	15.7
★	ST-4	4	48.8	19.5	29.3
⊗	ST-4	5	65.4	24.3	41.1
⊠	ST-4	7	61.5	23.8	37.7
◆	ST-4	9	56.1	23.3	32.8
◇	ST-6	1B	24.5	14.8	9.7
×	ST-6	4	29.1	15.5	13.6
⊗	ST-6	6	48.7	22.2	26.5
⊠	ST-6	7	54.3	29.6	24.7
✱	ST-6	8	57.9	24.4	33.5
⊠	ST-7	4	27.7	14.2	13.5
⊠	ST-7	6	52.6	28.8	23.8
⊠	ST-7	8	50.9	24.9	26.0

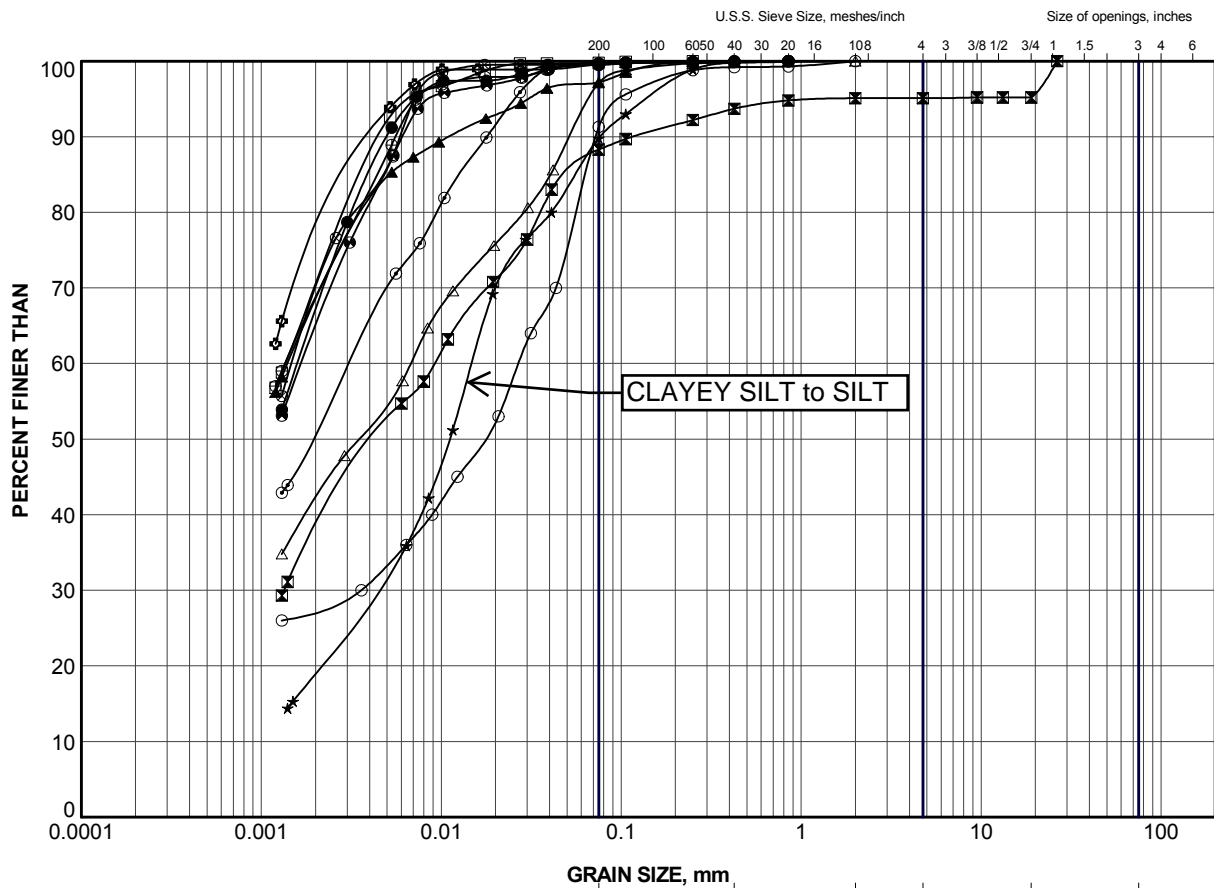
PROJECT					
HIGHWAY 141 SHADOW RIVER BRIDGE					
TITLE					
PLASTICITY CHART CLAYEY SILT to CLAY					
PROJECT No.		1651997		FILE No.	
DRAWN		TB		Mar 2018	
CHECK		AC/SEMP		Mar 2018	
APPR		JMAC		Mar 2018	
SCALE		N/A		REV.	
FIGURE B3.2					



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	S-5	5B	34.9	21.2	13.7
⊠	S-5	8	51.6	24.2	27.4
▲	S-6	6	55.8	26.5	29.3
★	S-6	8	54.0	25.2	28.8
⊙	S-6	9	59.9	25.7	34.2
⊕	S-6	11	29.9	18.9	11.0
○	S-9	7	55.3	23.5	31.8
△	S-10	7	55.7	29.0	26.7
⊗	S-10	10	47.6	26.2	21.4
⊕	ST-4	12	49.8	23.1	26.7
□	ST-5	2	46.4	24.6	21.8
⊗	ST-5	3	35.3	14.8	20.5
⊕	ST-5	4	27.5	14.0	13.5
★	ST-5	6	49.1	21.9	27.2
⊗	ST-6	5	59.4	23.9	35.5
■	ST-7	7	48.4	23.9	24.5
◆	ST-8	9	53.8	28.2	25.6
◇	ST-8	12	54.9	27.8	27.1

PROJECT					
HIGHWAY 141 SHADOW RIVER BRIDGE					
TITLE					
PLASTICITY CHART CLAYEY SILT to CLAY					
PROJECT No.		1651997		FILE No.	
DRAWN		TB		Mar 2018	
CHECK		AC/SEMP		Mar 2018	
APPR		JMAG		Mar 2018	
SCALE		N/A		REV.	
FIGURE B3.3					



CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	S-1	11	218.7
⊠	S-2	2B	226.0
▲	S-2	5	222.5
★	S-2	11	213.2
⊙	S-3	3	225.1
⊕	S-3	6	220.5
○	S-4	8	223.3
△	S-7	3	225.1
⊗	S-8	8	216.9
⊕	ST-2	5	222.7
□	ST-3	8	217.5
⊗	ST-4	9	216.2

PROJECT

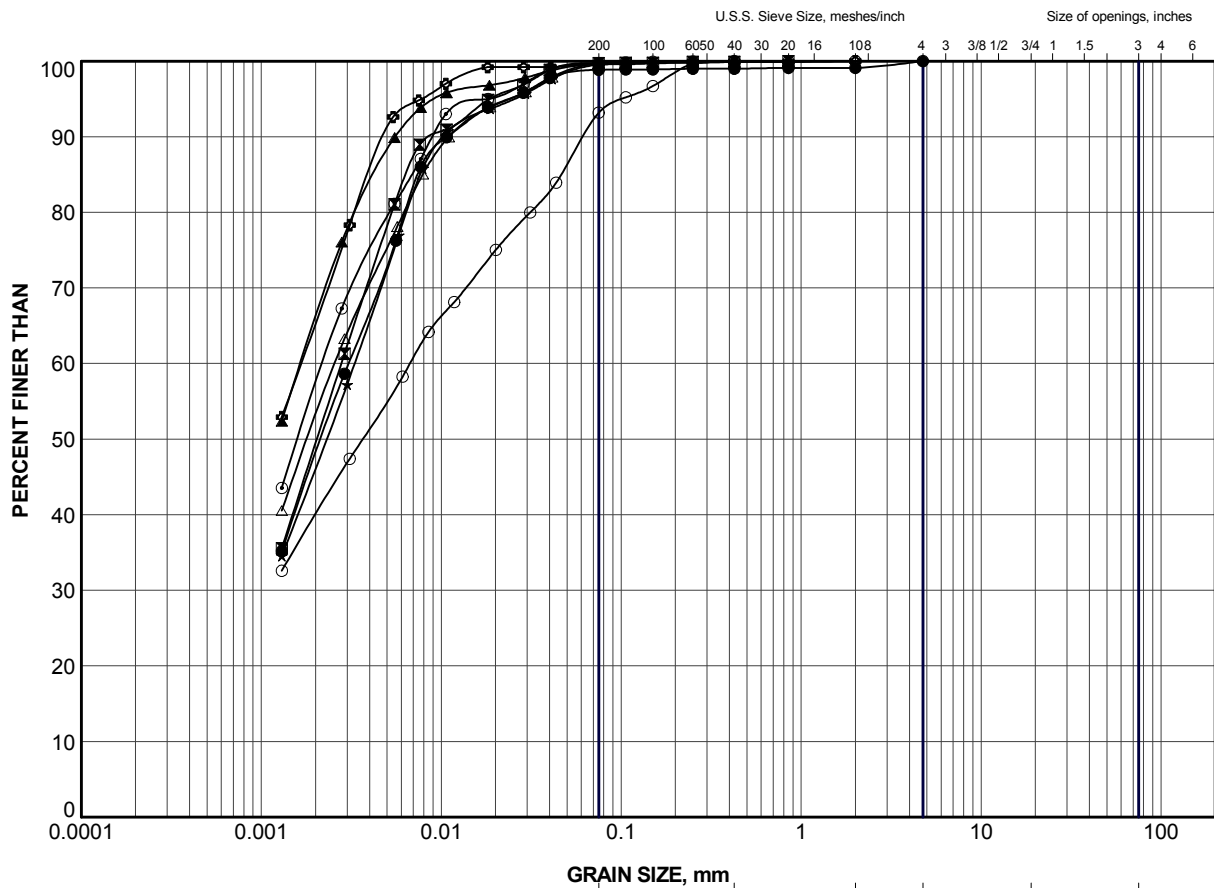
HIGHWAY 141
SHADOW RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION
CLAYEY SILT to CLAY




PROJECT No.		1651997	FILE No.		1651997.GPJ
DRAWN	TB	Mar 2018	SCALE	N/A	REV.
CHECK	AC/SEMP	Mar 2018	FIGURE B4.1		
APPR	JMAC	Mar 2018			



GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

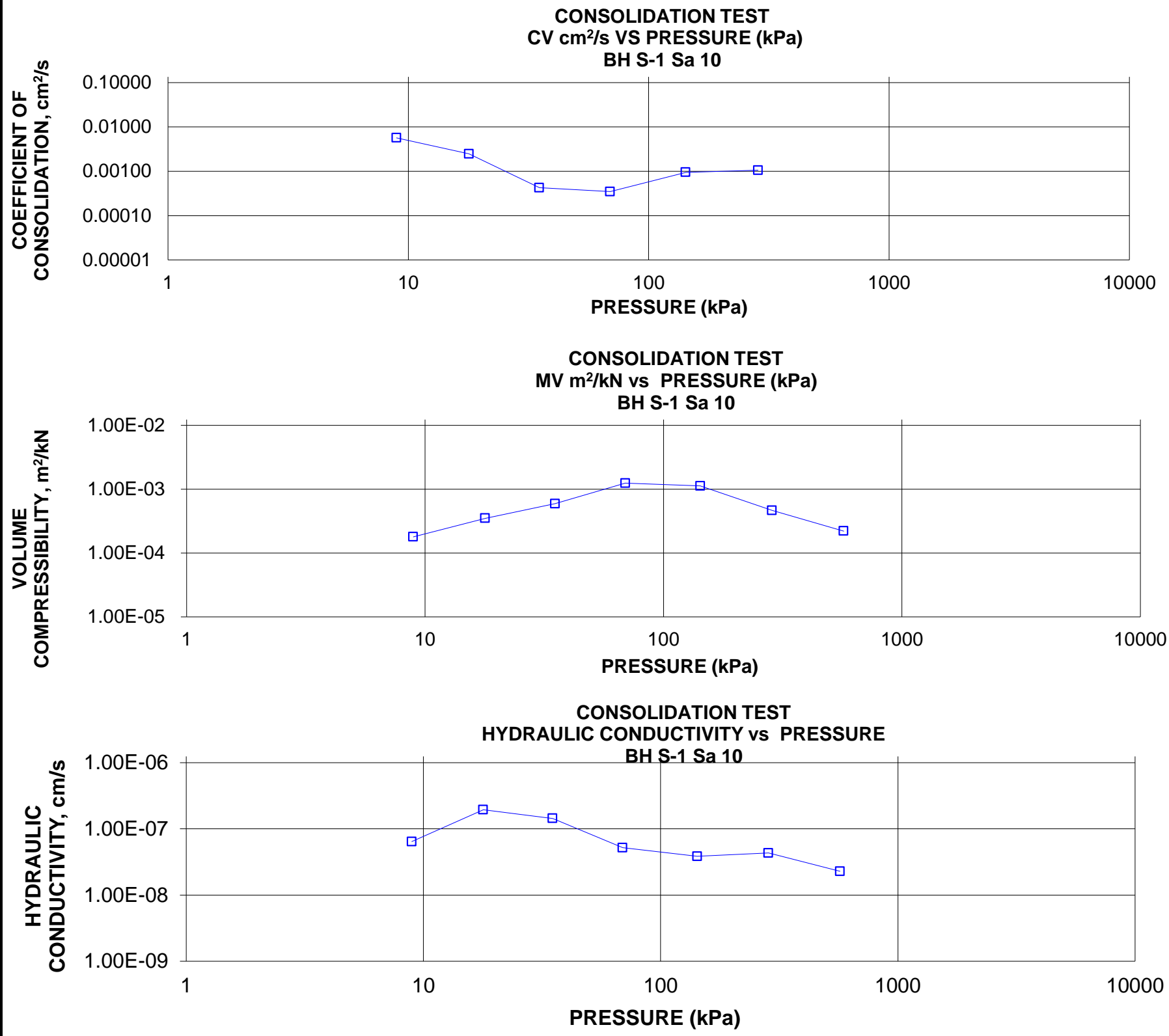
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	S-5	8	221.3
⊠	S-6	8	220.9
▲	S-9	8	220.6
★	S-9	10	217.5
⊙	S-10	7	220.1
⊕	ST-1	8	217.7
○	ST-5	4	224.9
△	ST-6	6	220.7

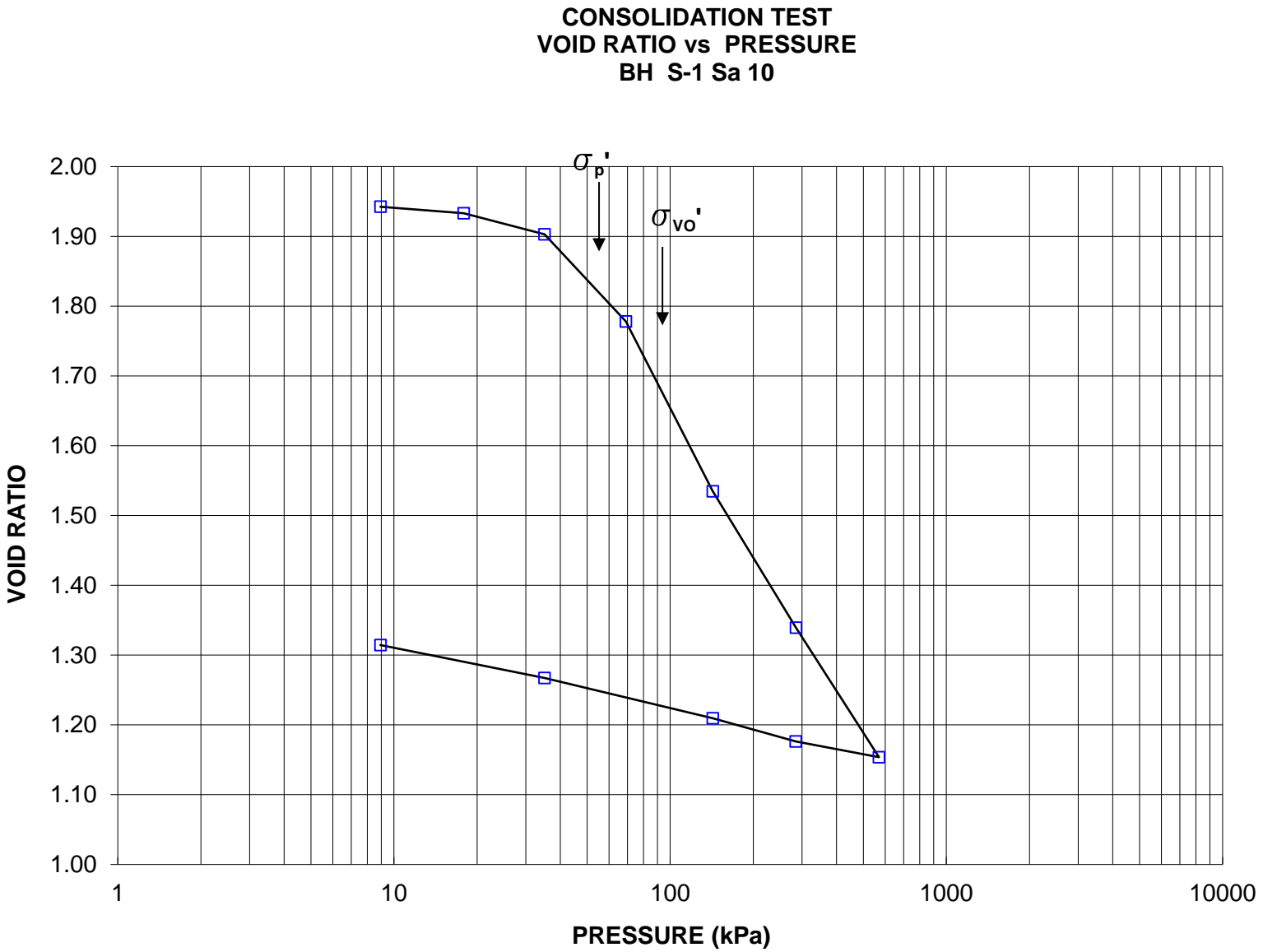
PROJECT					
HIGHWAY 141 SHADOW RIVER BRIDGE					
TITLE					
GRAIN SIZE DISTRIBUTION CLAYEY SILT to CLAY					
PROJECT No.		1651997		FILE No. 1651997.GPJ	
DRAWN	TB	Mar 2018	SCALE	N/A	REV.
CHECK	AC/SEMP	Mar 2018			
APPR	JMAC	Mar 2018			
 Golder Associates SUDBURY, ONTARIO			FIGURE B4.2		

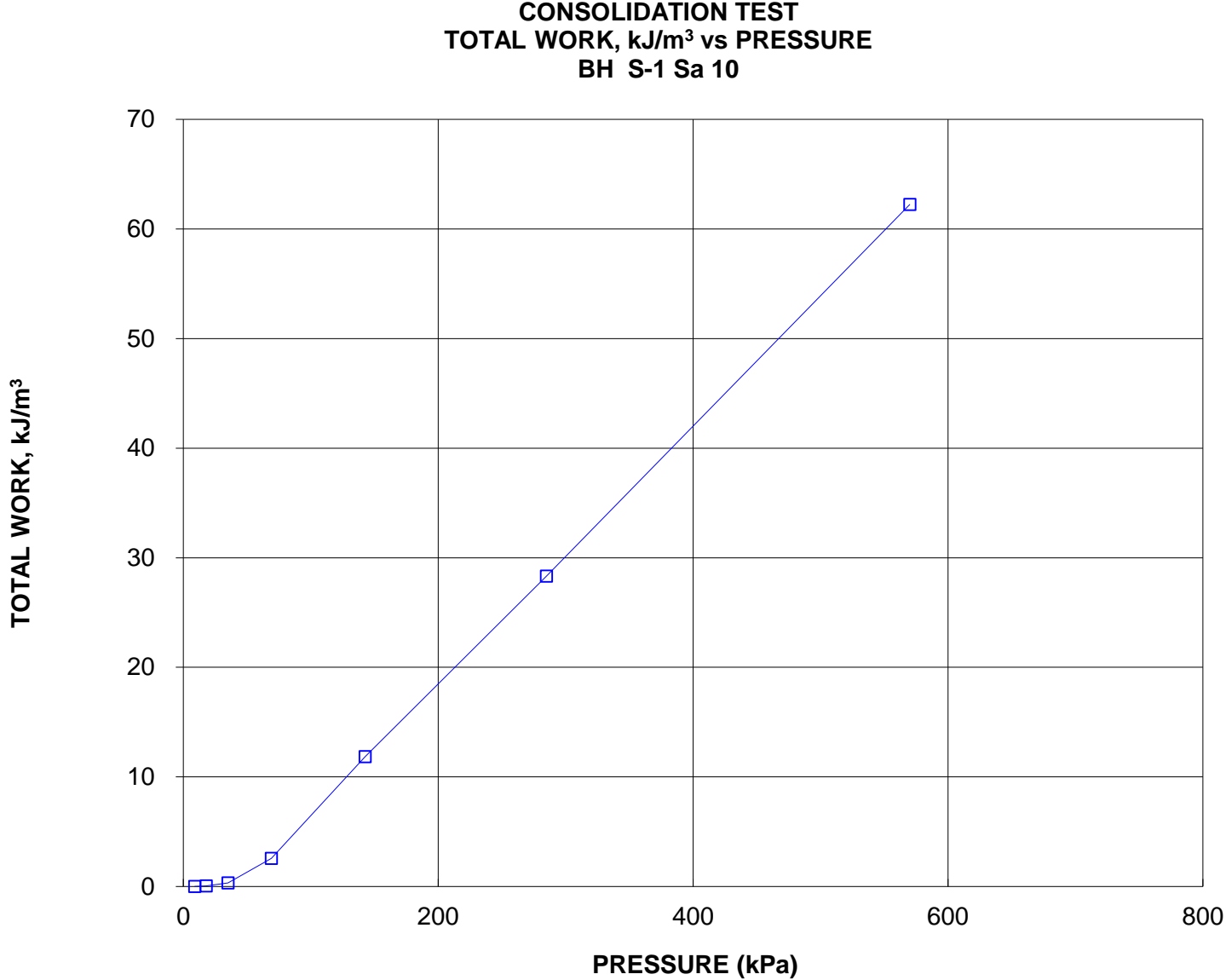
CONSOLIDATION TEST SUMMARY					FIGURE B5 Pg. 1 of 4				
SAMPLE IDENTIFICATION									
Project Number		1651997-1002			Sample Number		10		
Borehole Number		S-1			Sample Depth, m		9.3		
TEST CONDITIONS									
Test Type		Standard			Load Duration, hr		24		
Oedometer Number		1							
Date Started		3/24/17							
Date Completed		4/6/17							
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL									
Sample Height, cm		2.544			Unit Weight, kN/m ³		15.89		
Sample Diameter, cm		6.357			Drv Unit Weight, kN/m ³		9.27		
Area, cm ²		31.74			Specific Gravity, measured		2.785		
Volume, cm ³		80.75			Solids Height, cm		0.863		
Water Content, %		71.43			Volume of Solids, cm ³		27.40		
Wet Mass, g		130.82			Volume of Voids, cm ³		53.35		
Dry Mass, g		76.31							
TEST COMPUTATIONS									
Pressure	Primary	Corr.	End of Primary	Average					Total
kPa	Consolidation	Height	Void	Height	t ₉₀	cv.	mv	k	Work
	mm	cm	Ratio	cm	sec	cm ² /s	m ² /kN	cm/s	kJ/m3
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 cv based on t ₉₀ values. Void ratio for unloading (or rebound) calculated for the end of increment.									
SAMPLE DIMENSIONS AND PROPERTIES - FINAL									
Sample Height, cm		2.065			Unit Weight, kN/m ³		16.27		
Sample Diameter, cm		6.36			Drv Unit Weight, kN/m ³		11.42		
Area, cm ²		31.74			Specific Gravity, measured		2.785		
Volume, cm ³		65.54			Solids Height, cm		0.863		
Water Content, %		42.45			Volume of Solids, cm ³		27.40		
Wet Mass, g		108.70			Volume of Voids, cm ³		38.14		
Dry Mass, g		76.31							
Prepared By: TG					Golder Associates			Checked By: MT	

CONSOLIDATION TEST SUMMARY

FIGURE B5
Pg. 2 of 4







CONSOLIDATION TEST SUMMARY
ASTM D2435/D2435M

FIGURE B6
pg 1 of 4

SAMPLE IDENTIFICATION

Project Number	1651997(1003)	Sample Number	9
Borehole Number	S-6	Sample Depth, ft	10.06-10.67

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	10		
Date Started	01/18/2018		
Date Completed	02/05/2018		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.89	Unit Weight, kN/m ³	15.03
Sample Diameter, cm	6.36	Drv Unit Weight, kN/m ³	8.26
Area, cm ²	31.72	Specific Gravity, measured	2.73
Volume, cm ³	60.08	Solids Height, cm	0.584
Water Content, %	82.00	Volume of Solids, cm ³	18.53
Wet Mass, g	92.09	Volume of Voids, cm ³	41.54
Dry Mass, g	50.6	Degree of Saturation, %	99.9

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	1.894	2.241	1.894				
6.00	1.887	2.229	1.891				
10.61	1.892	2.238	1.890	66	1.15E-02	-6.10E-04	-6.86E-07
20.50	1.889	2.233	1.891	109	6.95E-03	1.63E-04	1.11E-07
39.70	1.874	2.207	1.882	135	5.56E-03	4.16E-04	2.27E-07
78.42	1.830	2.132	1.852	194	3.75E-03	5.96E-04	2.19E-07
20.50	1.848	2.162	1.839				
39.70	1.842	2.153	1.845	60	1.20E-02	1.48E-04	1.74E-07
78.42	1.822	2.118	1.832	101	7.05E-03	2.73E-04	1.89E-07
117.71	1.720	1.943	1.771	5539	1.20E-04	1.38E-03	1.62E-08
155.51	1.609	1.753	1.664	7935	7.40E-05	1.55E-03	1.12E-08
309.77	1.413	1.418	1.511	634	7.63E-04	6.70E-04	5.01E-08
620.30	1.275	1.181	1.344	421	9.09E-04	2.36E-04	2.10E-08
1237.65	1.168	0.999	1.221	194	1.63E-03	9.11E-05	1.45E-08
2473.22	1.078	0.844	1.123	240	1.11E-03	3.86E-05	4.22E-09
619.30	1.100	0.883	1.089				
155.51	1.136	0.944	1.118				
39.70	1.179	1.017	1.157				
10.66	1.213	1.076	1.196				

Note:

Consolidation loading and unloading schedule assigned by the client
 cv and k are approximate only based on t₉₀ estimated from Square Root of Time Method (ASTMD2435/2435M)
 Sample swelled under 6.0kPa.
 Specimen taken 15cm from the bottom of tube

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.21	Unit Weight, kN/m ³	18.27
Sample Diameter, cm	6.36	Drv Unit Weight, kN/m ³	12.90
Area, cm ²	31.72	Specific Gravity, measured	2.73
Volume, cm ³	38.47	Solids Height, cm	0.584
Water Content, %	41.68	Volume of Solids, cm ³	18.53
Wet Mass, g	71.69	Volume of Voids, cm ³	19.94
Dry Mass, g	50.6		

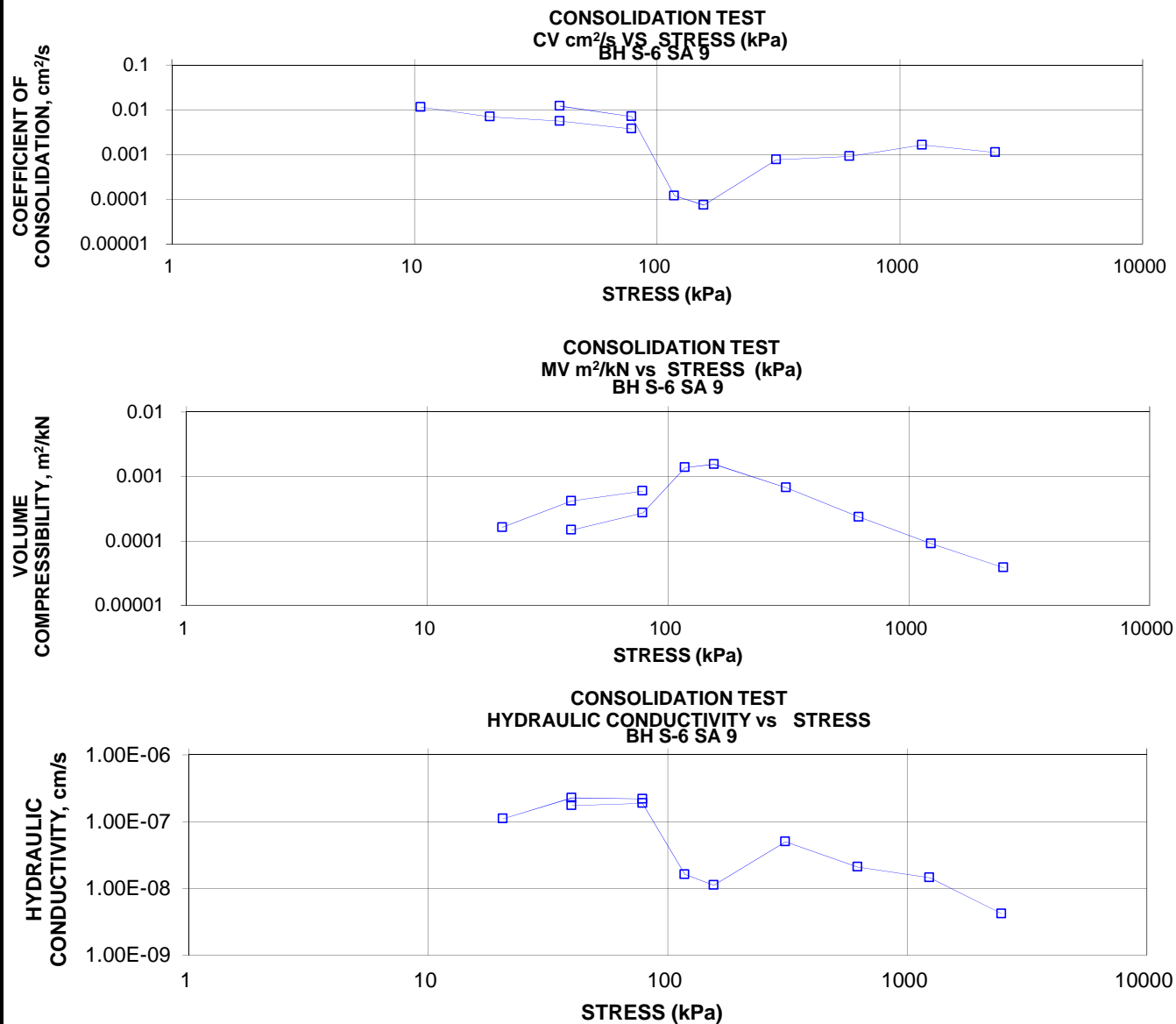
Prepared By: LH

Golder Associates

Checked By: MM

CONSOLIDATION TEST SUMMARY

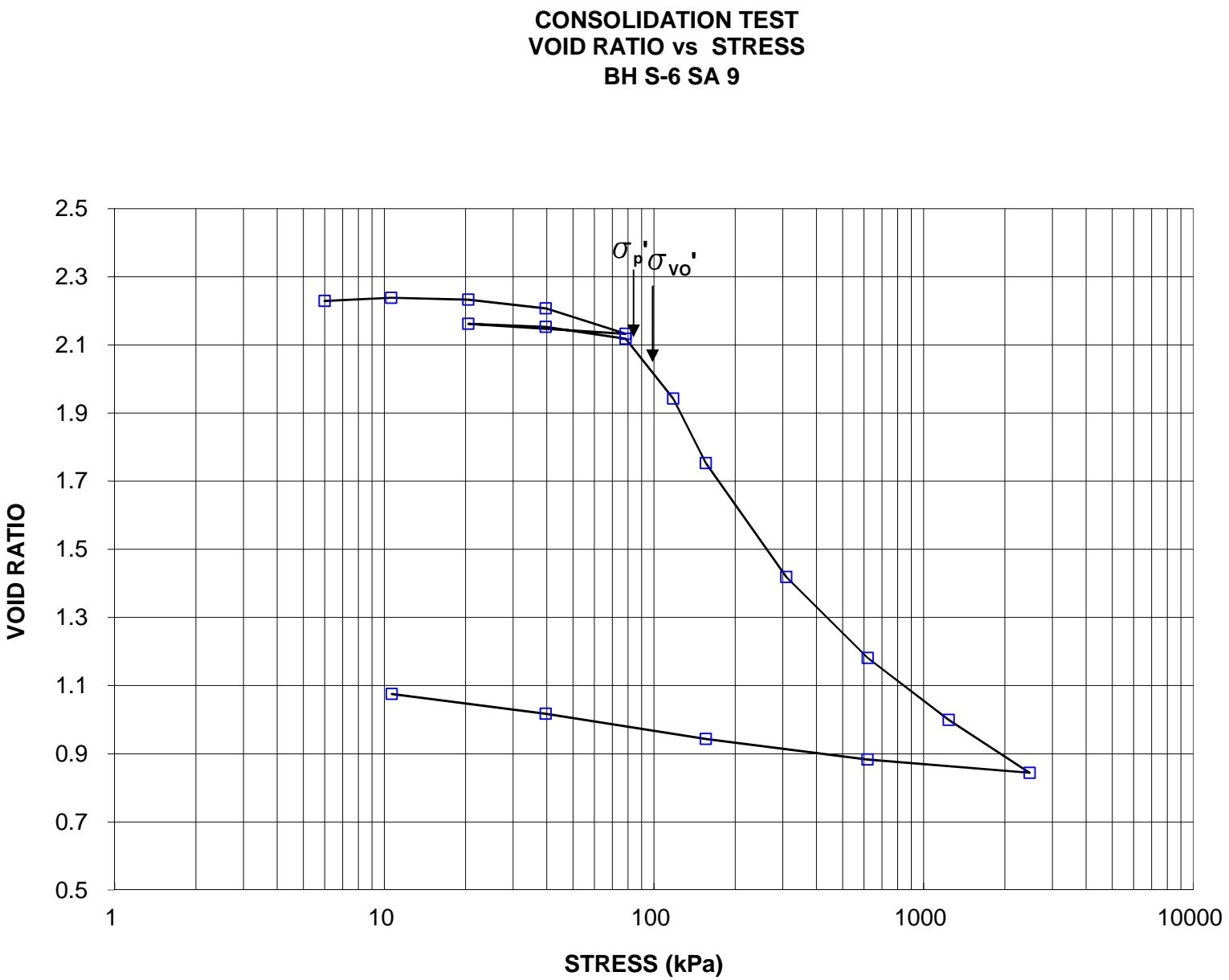
FIGURE B6
pg 2 of 4

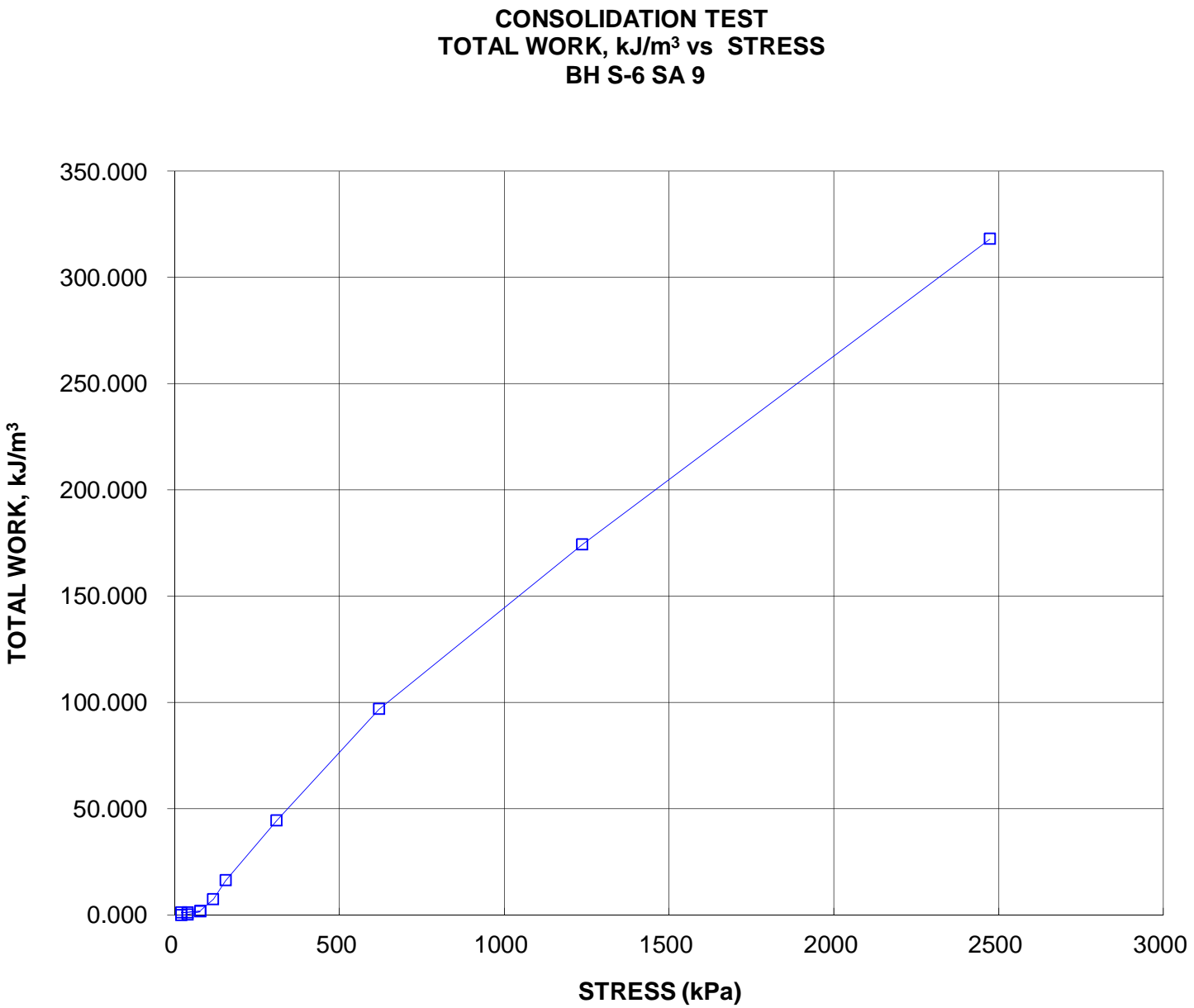


Project No. 1651997(1003)
Prepared By: LH

Golder Associates

Checked By: MM





CONSOLIDATION TEST SUMMARY
ASTM D2435/D2435M

FIGURE B7
pg 1 of 4

SAMPLE IDENTIFICATION

Project Number	1651997(1003)	Sample Number	7
Borehole Number	S-9	Sample Depth, ft	7.01-7.62

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	4		
Date Started	01/18/2018		
Date Completed	02/04/2018		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m ³	15.17
Sample Diameter, cm	6.34	Drv Unit Weight, kN/m ³	8.49
Area, cm ²	31.53	Specific Gravity, measured	2.72
Volume, cm ³	80.02	Solids Height, cm	0.808
Water Content, %	78.61	Volume of Solids, cm ³	25.48
Wet Mass, g	123.78	Volume of Voids, cm ³	54.54
Dry Mass, g	69.3	Degree of Saturation, %	99.9

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	2.538	2.141	2.538				
6.24	2.549	2.154	2.543				
10.89	2.552	2.159	2.551				
20.74	2.548	2.153	2.550	148	9.32E-03	1.84E-04	1.68E-07
40.25	2.535	2.138	2.542	173	7.92E-03	2.50E-04	1.94E-07
79.13	2.493	2.086	2.514	331	4.05E-03	4.27E-04	1.69E-07
20.74	2.518	2.116	2.506				
40.25	2.511	2.107	2.515	135	9.93E-03	1.45E-04	1.41E-07
79.13	2.485	2.075	2.498	154	8.59E-03	2.68E-04	2.25E-07
117.52	2.334	1.888	2.409	5189	2.37E-04	1.55E-03	3.59E-08
156.83	2.185	1.704	2.259	7935	1.36E-04	1.49E-03	2.00E-08
312.49	1.924	1.381	2.055	1717	5.21E-04	6.60E-04	3.37E-08
622.89	1.735	1.147	1.830	1017	6.98E-04	2.40E-04	1.64E-08
1244.57	1.588	0.965	1.661	667	8.77E-04	9.35E-05	8.04E-09
2483.91	1.465	0.813	1.527	421	1.17E-03	3.88E-05	4.47E-09
622.89	1.495	0.850	1.480				
156.83	1.542	0.909	1.519				
40.25	1.601	0.981	1.572				
10.89	1.649	1.041	1.625				

Note:

Consolidation loading and unloading schedule assigned by the client

cv and k are approximate only based on t₉₀ estimated from Square Root of Time Method (ASTMD2435/2435M)

Sample swelled under 10.89kPa.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.65	Unit Weight, kN/m ³	18.20
Sample Diameter, cm	6.34	Drv Unit Weight, kN/m ³	13.07
Area, cm ²	31.53	Specific Gravity, measured	2.72
Volume, cm ³	51.99	Solids Height, cm	0.808
Water Content, %	39.21	Volume of Solids, cm ³	25.48
Wet Mass, g	96.47	Volume of Voids, cm ³	26.51
Dry Mass, g	69.3		

Prepared By: LH

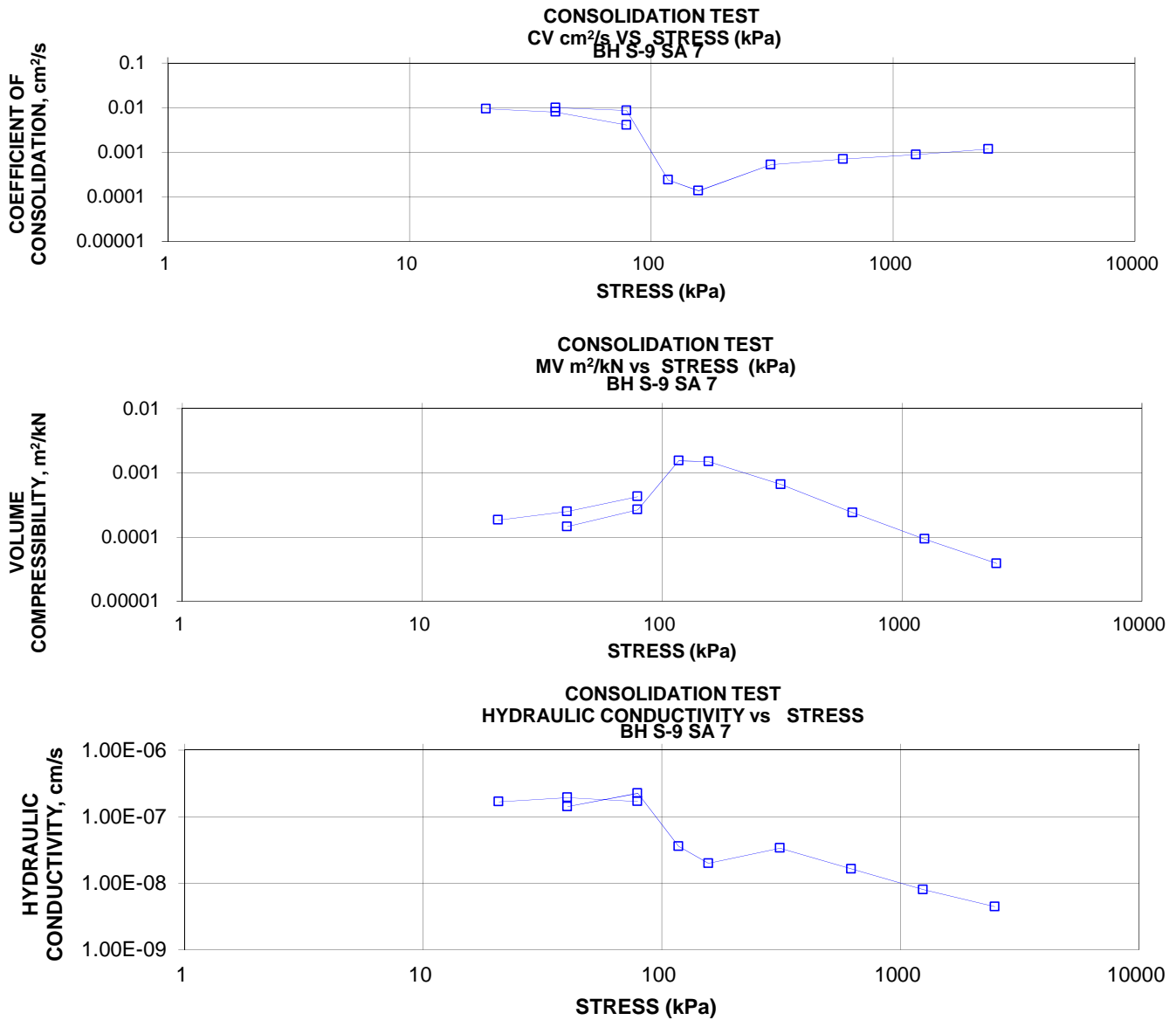
Golder Associates

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CONSOLIDATION TEST SUMMARY

FIGURE B7

pg 2 of 4

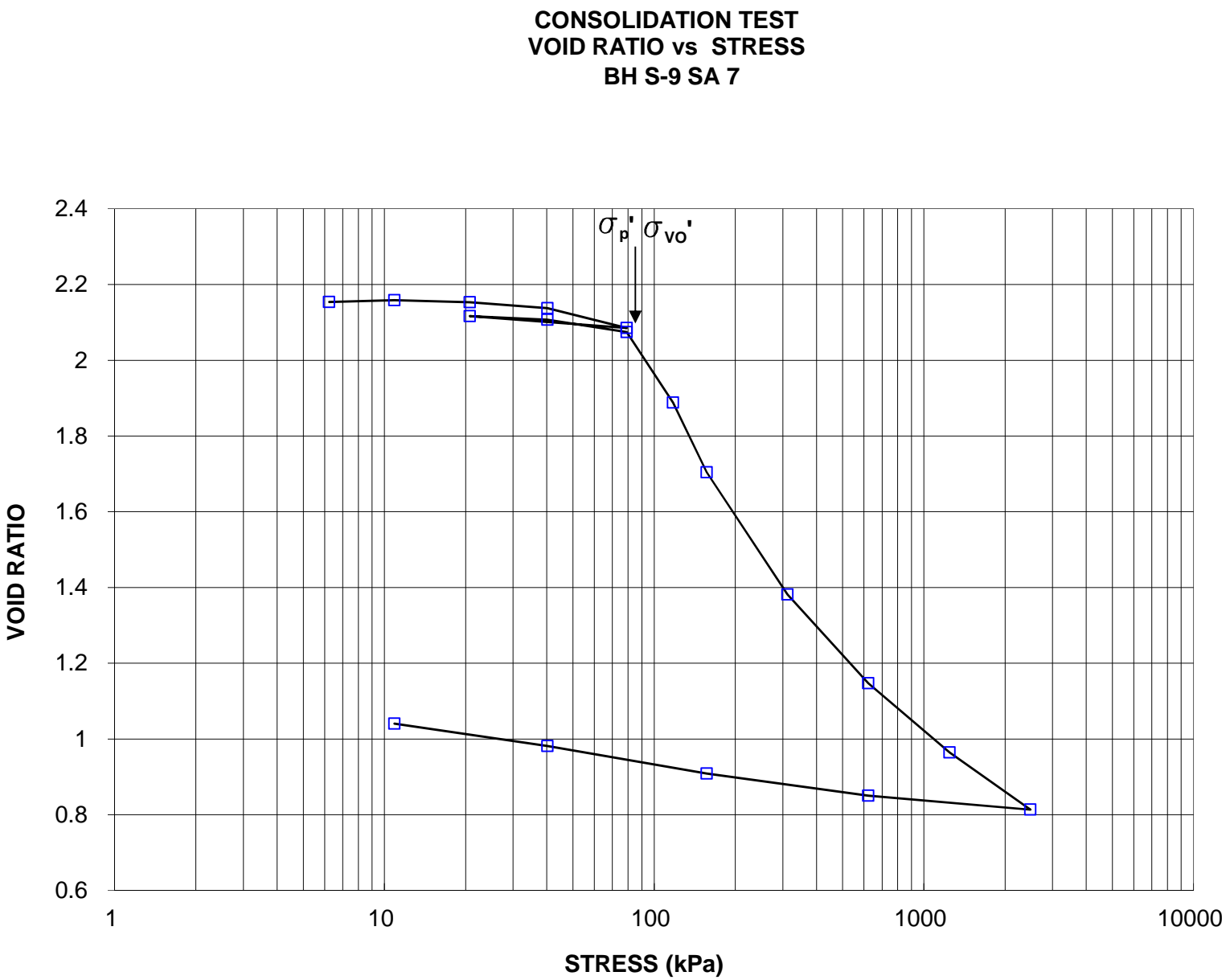


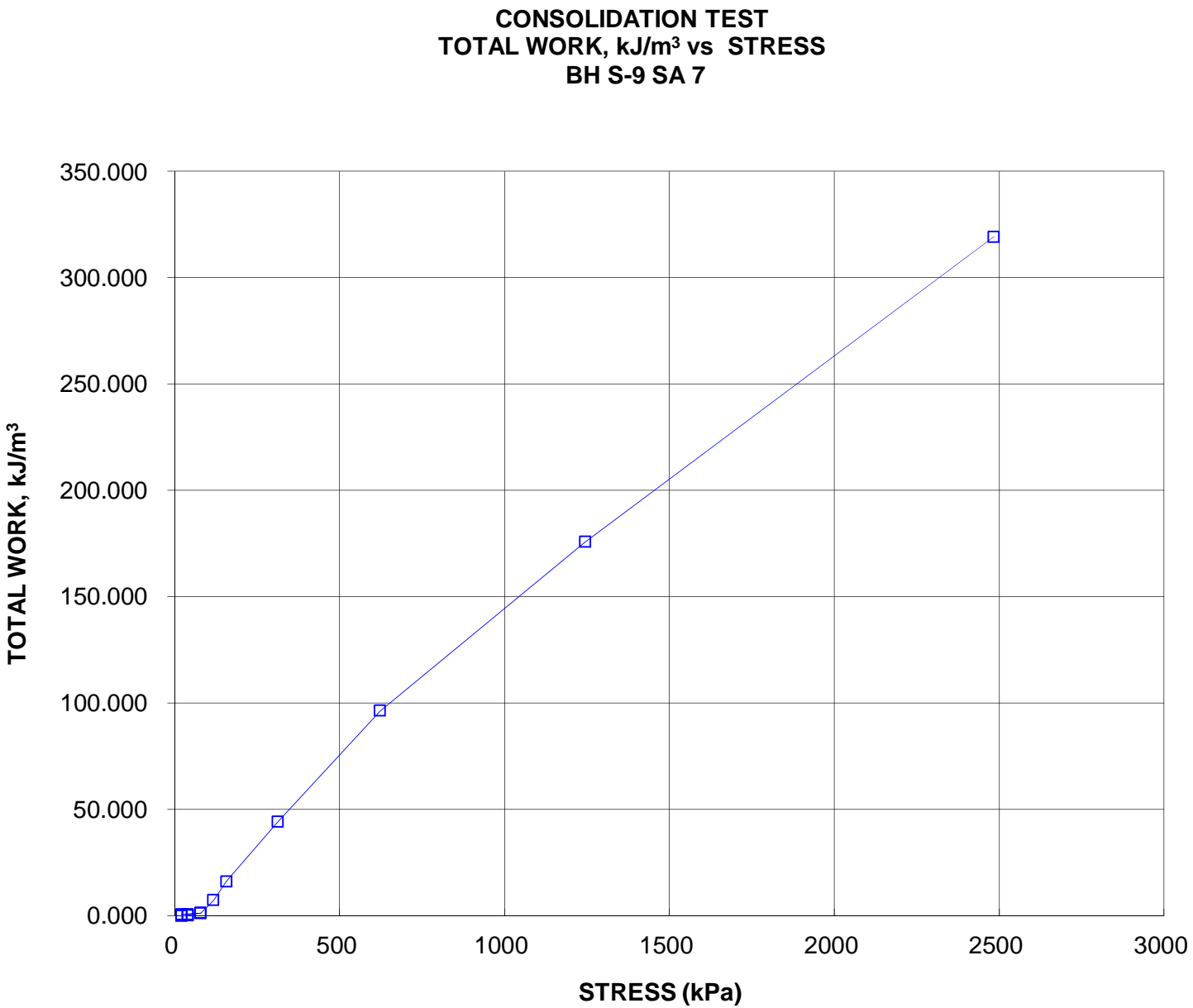
Project No. 1651997(1003)

Prepared By: LH

Golder Associates

Checked By: MM

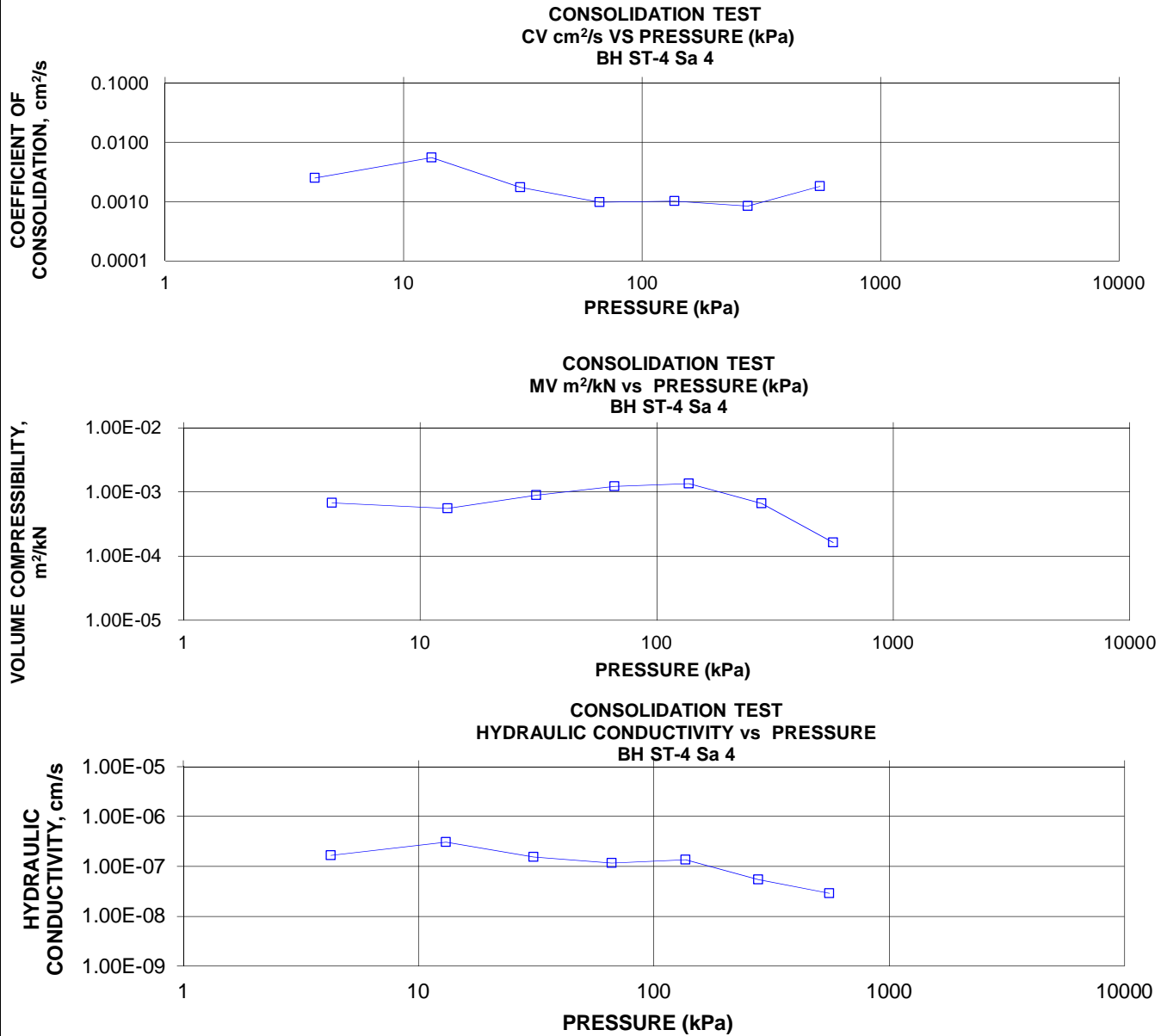




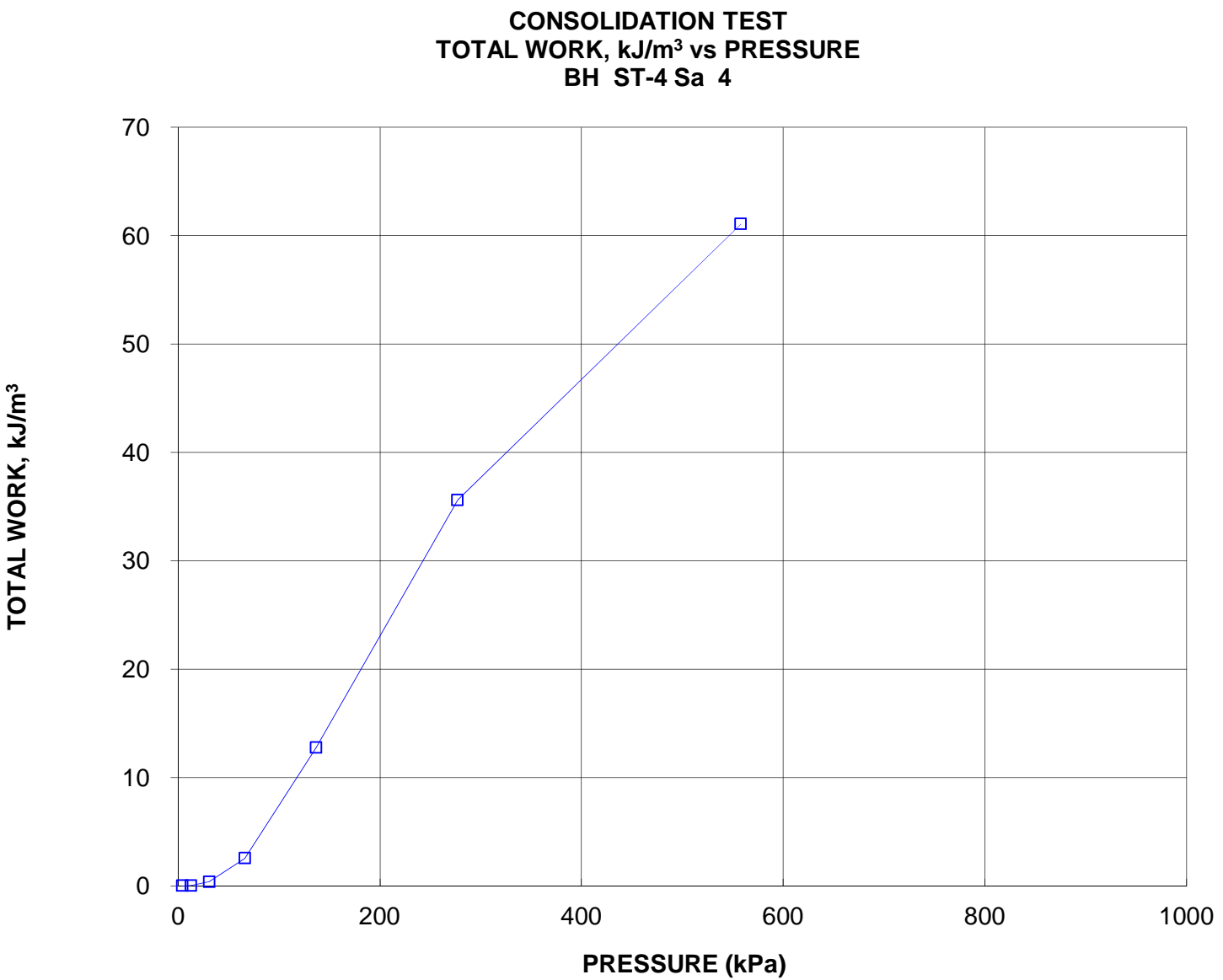
CONSOLIDATION TEST SUMMARY						FIGURE B8 Pg. 1 of 4			
SAMPLE IDENTIFICATION									
Project Number		1651997-1002			Sample Number		4		
Borehole Number		ST-4			Sample Depth, m		3.2		
TEST CONDITIONS									
Test Type		Standard			Load Duration, hr		24		
Oedometer Number		2							
Date Started		March 24, 2017							
Date Completed		April 6, 2017							
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL									
Sample Height, cm		2.522			Unit Weight, kN/m ³		15.62		
Sample Diameter, cm		6.358			Dry Unit Weight, kN/m ³		8.95		
Area, cm ²		31.74			Specific Gravity, Measured		2.781		
Volume, cm ³		80.06			Solids Height, cm		0.828		
Water Content, %		74.56			Volume of Solids, cm ³		26.27		
Wet Mass, g		127.55			Volume of Voids, cm ³		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 ₉₀ sec	cv, cm ² /s	mv m ² /kN	k cm/s	Total Work kJ/m ³
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 cv based on t ₉₀ 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 ³		15.57		
Sample Diameter, cm		6.36			Dry Unit Weight, kN/m ³		11.03		
Area, cm ²		31.74			Specific Gravity, Measured		2.781		
Volume, cm ³		64.96			Solids Height, cm		0.828		
Water Content, %		41.12			Volume of Solids, cm ³		26.27		
Wet Mass, g		103.12			Volume of Voids, cm ³		38.69		
Dry Mass, g		73.07							
<div style="display: flex; justify-content: space-between;"> Prepared By: TG Golder Associates Checked By: MT </div>									

CONSOLIDATION TEST SUMMARY

FIGURE B8
Pg. 2 of 4







CONSOLIDATION TEST SUMMARY
ASTM D2435/D2435M

FIGURE B9
pg 1 of 4

SAMPLE IDENTIFICATION

Project Number	1651997/1541608	Sample Number	5
Borehole Number	ST-6	Sample Depth, ft	4.57-5.18

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	12		
Date Started	01/18/2018		
Date Completed	02/05/2018		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.55	Unit Weight, kN/m ³	14.98
Sample Diameter, cm	6.34	Drv Unit Weight, kN/m ³	8.31
Area, cm ²	31.58	Specific Gravity, measured	2.73
Volume, cm ³	80.46	Solids Height, cm	0.790
Water Content, %	80.34	Volume of Solids, cm ³	24.96
Wet Mass, g	122.90	Volume of Voids, cm ³	55.50
Dry Mass, g	68.15	Degree of Saturation, %	98.6

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	2.548	2.223	2.548				
6.07	2.544	2.218	2.546				
10.73	2.539	2.212	2.541	487	2.81E-03	4.46E-04	1.23E-07
20.66	2.520	2.187	2.529	470	2.89E-03	7.59E-04	2.15E-07
39.94	2.471	2.126	2.495	1162	1.14E-03	9.85E-04	1.10E-07
78.84	2.281	1.886	2.376	1815	6.60E-04	1.91E-03	1.24E-07
20.66	2.307	1.918	2.294				
39.94	2.300	1.909	2.303	240	4.69E-03	1.45E-04	6.64E-08
78.84	2.257	1.855	2.278	360	3.06E-03	4.36E-04	1.31E-07
118.30	2.057	1.602	2.157	7459	1.32E-04	1.99E-03	2.57E-08
156.27	1.960	1.479	2.008	10375	8.24E-05	1.01E-03	8.12E-09
311.33	1.756	1.222	1.858	1848	3.96E-04	5.14E-04	2.00E-08
621.38	1.590	1.011	1.673	1070	5.55E-04	2.11E-04	1.15E-08
1240.66	1.440	0.822	1.515	694	7.01E-04	9.46E-05	6.50E-09
2481.11	1.338	0.692	1.389	454	9.01E-04	3.25E-05	2.87E-09
621.38	1.360	0.720	1.349				
156.39	1.400	0.771	1.380				
40.19	1.452	0.837	1.426				
10.73	1.496	0.893	1.474				

Note:

Consolidation loading and unloading schedule assigned by the client

cv and k are approximate only based on t₉₀ estimated from Square Root of Time Method (ASTMD2435/2435M)

Sample swelled under 6.07kPa.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.50	Unit Weight, kN/m ³	19.05
Sample Diameter, cm	6.34	Drv Unit Weight, kN/m ³	14.14
Area, cm ²	31.58	Specific Gravity, measured	2.73
Volume, cm ³	47.26	Solids Height, cm	0.790
Water Content, %	34.67	Volume of Solids, cm ³	24.96
Wet Mass, g	91.78	Volume of Voids, cm ³	22.29
Dry Mass, g	68.15		

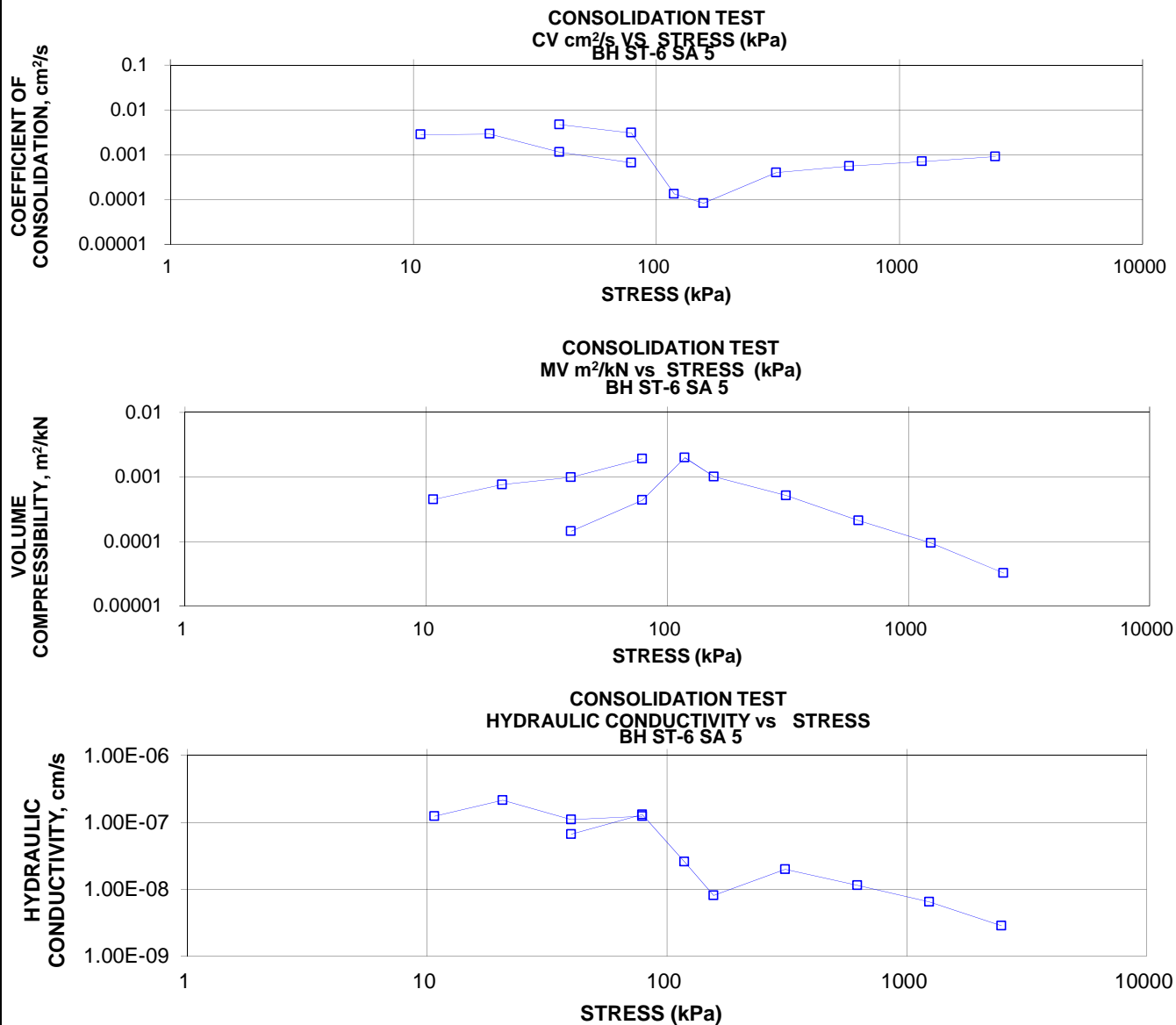
Prepared By: LH

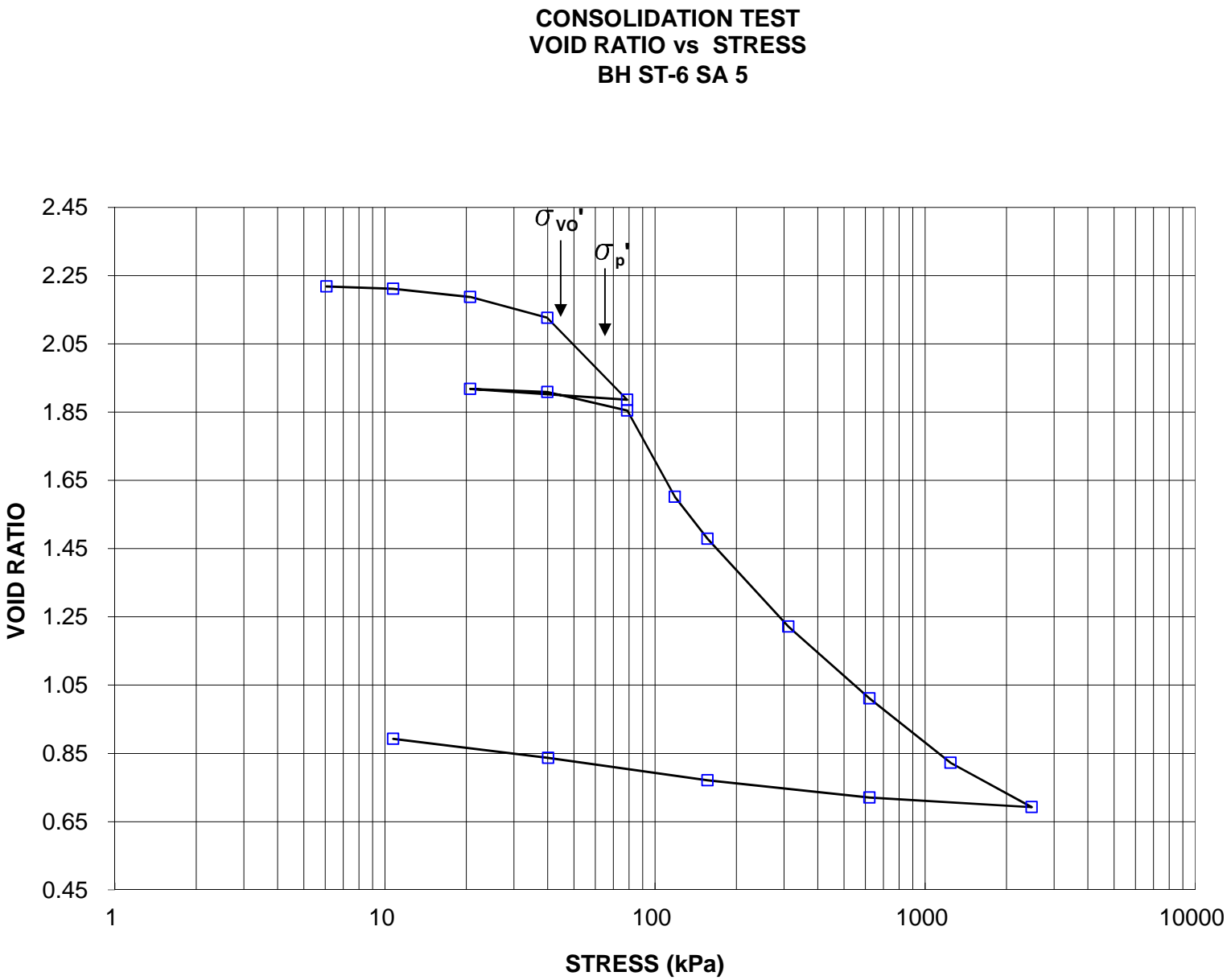
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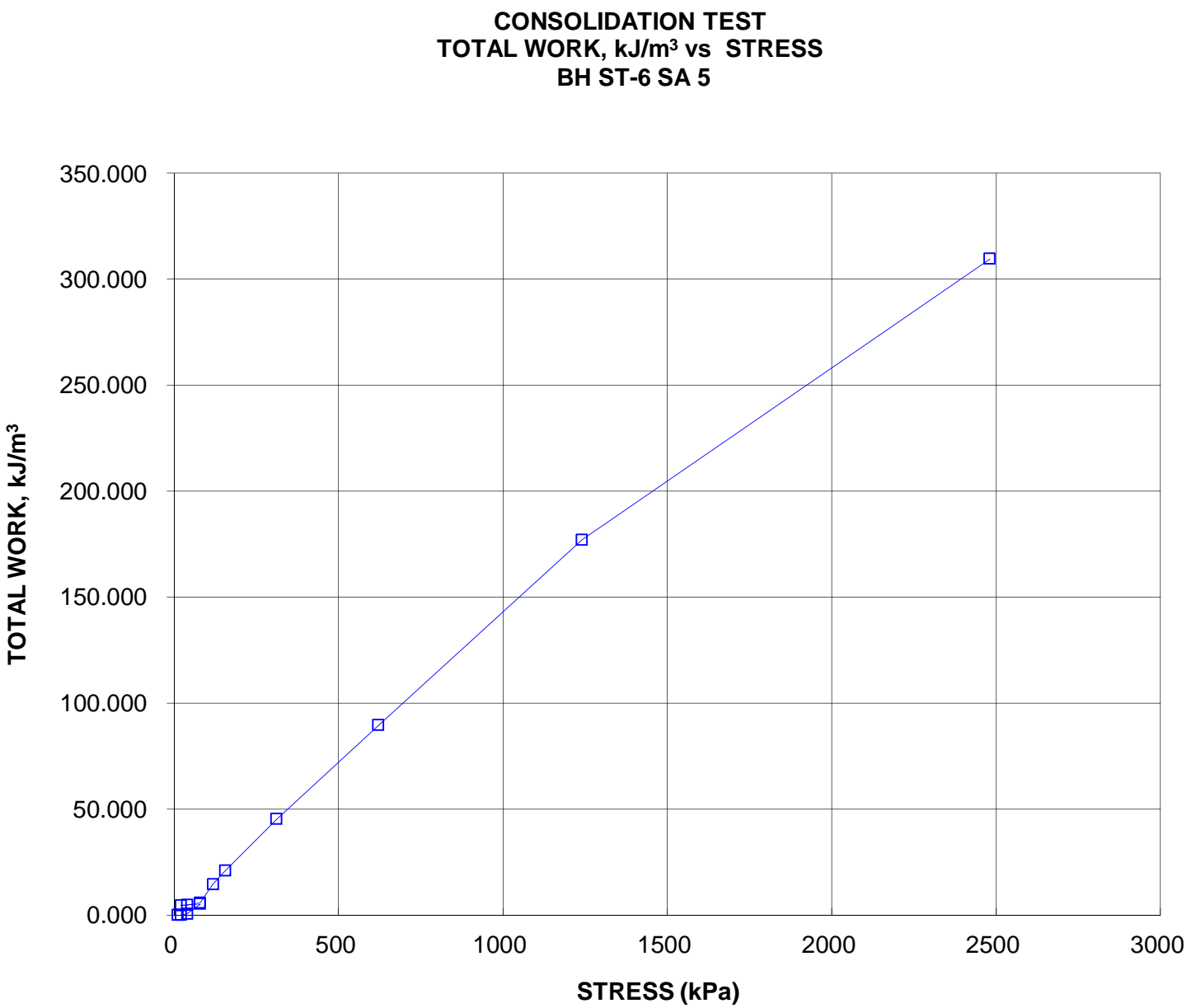
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CONSOLIDATION TEST SUMMARY

FIGURE B9
pg 2 of 4







CONSOLIDATION TEST SUMMARY
ASTM D2435/D2435M

FIGURE B10
pg 1 of 4

SAMPLE IDENTIFICATION

Project Number	1651997/1541608	Sample Number	7
Borehole Number	ST-7	Sample Depth, ft	8.38-8.99

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	11		
Date Started	01/18/2018		
Date Completed	02/04/2018		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m ³	15.09
Sample Diameter, cm	6.33	Drv Unit Weight, kN/m ³	8.41
Area, cm ²	31.47	Specific Gravity, measured	2.73
Volume, cm ³	80.06	Solids Height, cm	0.800
Water Content, %	79.33	Volume of Solids, cm ³	25.16
Wet Mass, g	123.18	Volume of Voids, cm ³	54.90
Dry Mass, g	68.69	Degree of Saturation, %	99.3

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	2.544	2.182	2.544				
5.85	2.536	2.172	2.540				
10.72	2.535	2.170	2.535	60	2.27E-02	8.88E-05	1.98E-07
20.53	2.520	2.152	2.528	154	8.79E-03	5.89E-04	5.08E-07
40.06	2.490	2.114	2.505	217	6.13E-03	6.14E-04	3.69E-07
79.00	2.428	2.037	2.459	252	5.09E-03	6.24E-04	3.11E-07
20.53	2.440	2.051	2.434				
40.06	2.436	2.047	2.438	34	3.71E-02	6.44E-05	2.34E-07
79.00	2.420	2.027	2.428	101	1.24E-02	1.62E-04	1.96E-07
117.30	2.249	1.812	2.334	6615	1.75E-04	1.76E-03	3.02E-08
156.79	2.054	1.570	2.152	5587	1.76E-04	1.93E-03	3.33E-08
312.71	1.835	1.295	1.945	1058	7.58E-04	5.53E-04	4.11E-08
624.53	1.681	1.102	1.758	1033	6.34E-04	1.94E-04	1.21E-08
1247.59	1.553	0.943	1.617	505	1.10E-03	8.04E-05	8.65E-09
2494.41	1.451	0.815	1.502	304	1.57E-03	3.23E-05	4.99E-09
624.53	1.473	0.843	1.462				
156.79	1.513	0.892	1.493				
40.06	1.561	0.952	1.537				
10.72	1.600	1.001	1.580				

Note:

Specimen taken 10-15cm from the bottom of the tube

cv and k are approximate only based on t₉₀ estimated from Square Root of Time Method (ASTMD2435/2435M)

Sample swelled under 5.85kPa.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.60	Unit Weight, kN/m ³	18.38
Sample Diameter, cm	6.33	Drv Unit Weight, kN/m ³	13.38
Area, cm ²	31.47	Specific Gravity, measured	2.73
Volume, cm ³	50.36	Solids Height, cm	0.800
Water Content, %	37.40	Volume of Solids, cm ³	25.16
Wet Mass, g	94.38	Volume of Voids, cm ³	25.19
Dry Mass, g	68.69		

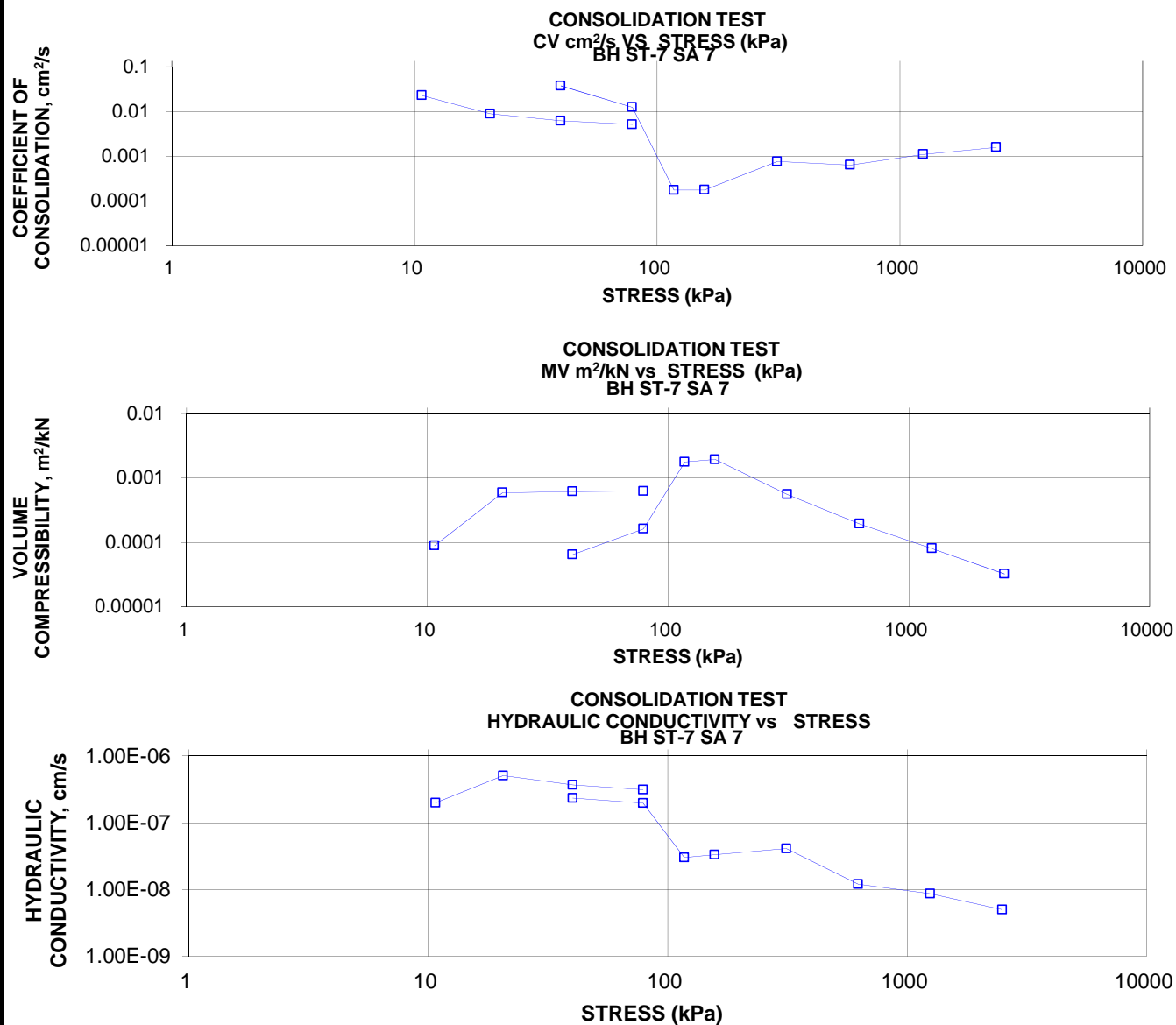
Prepared By: LH

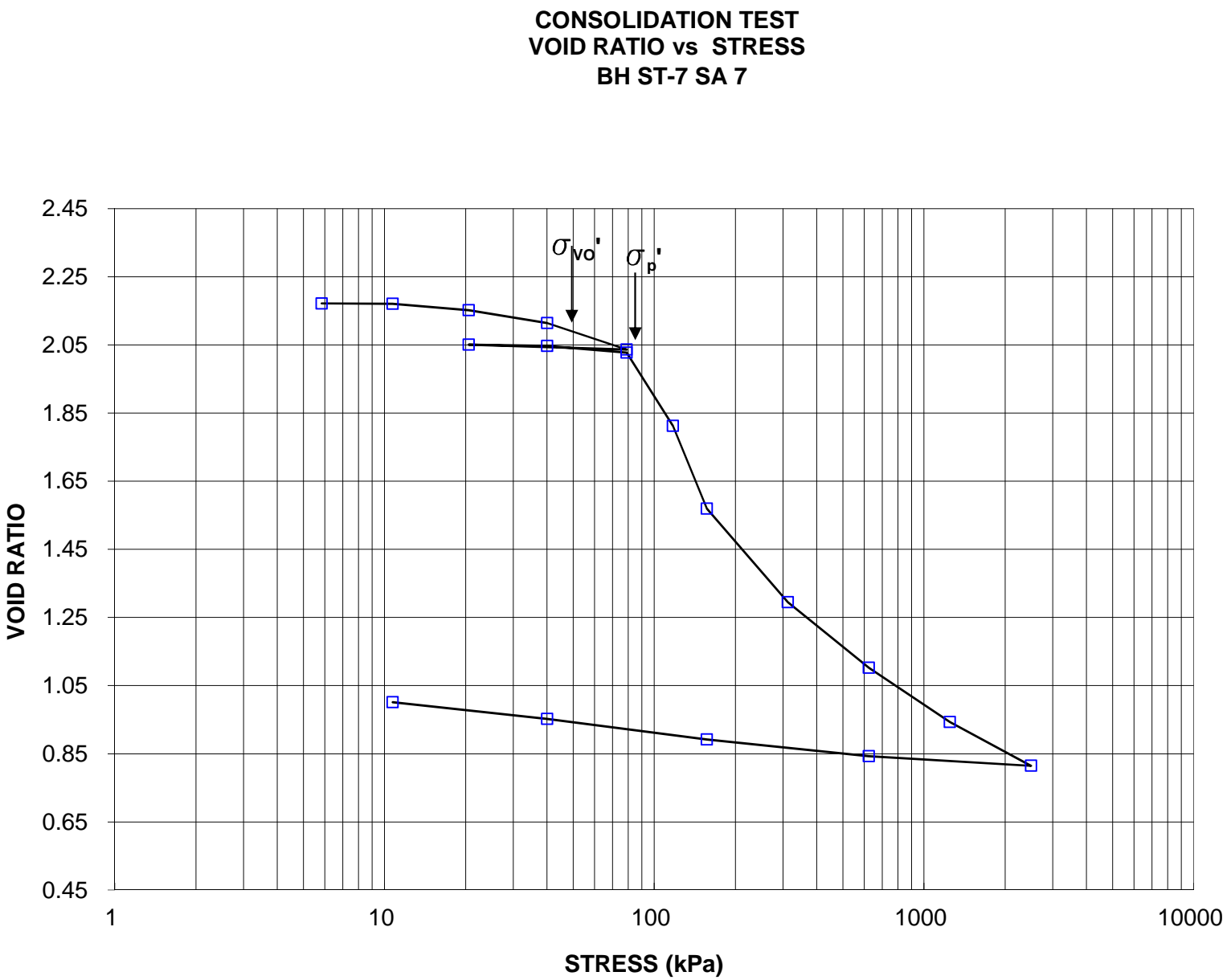
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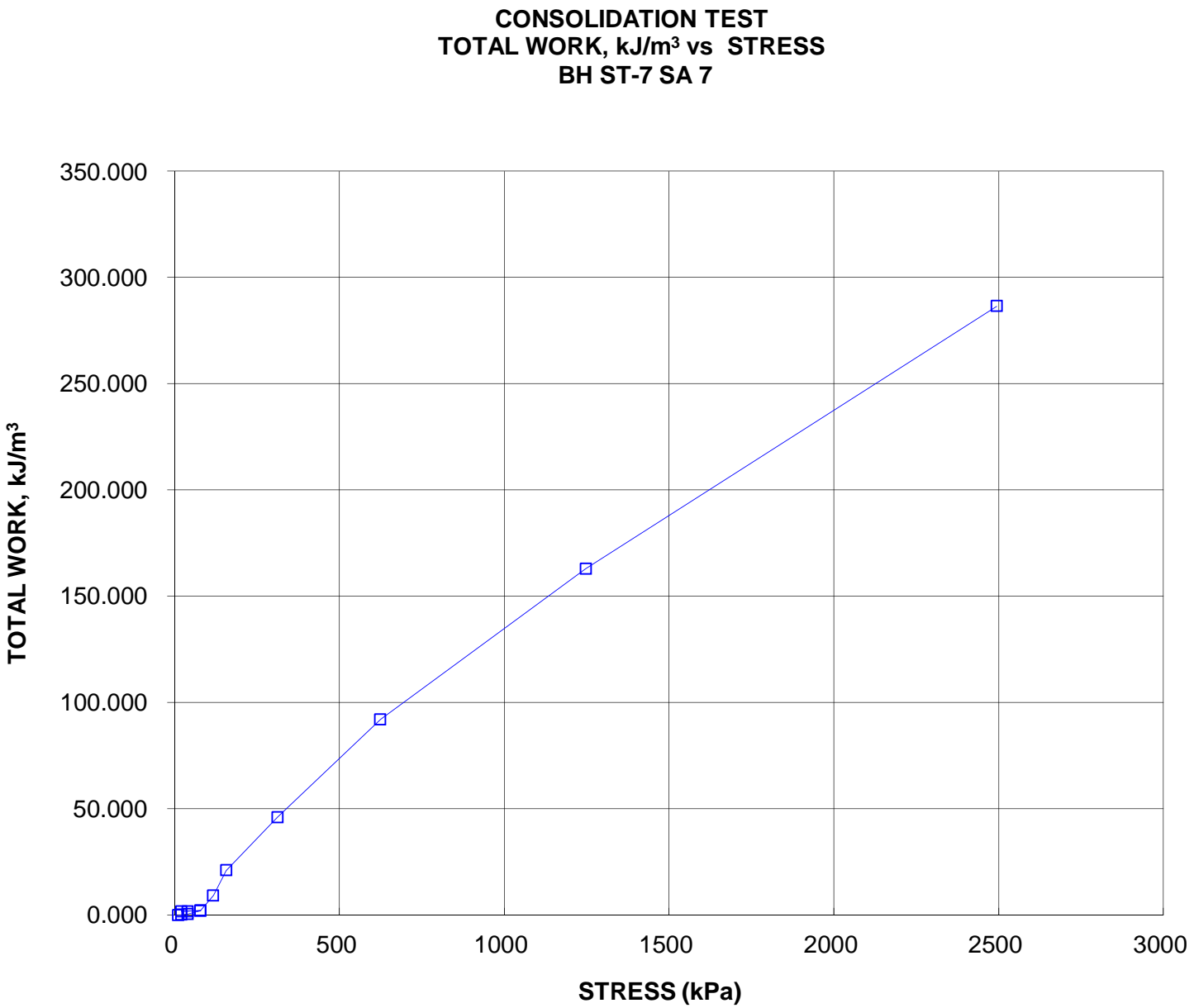
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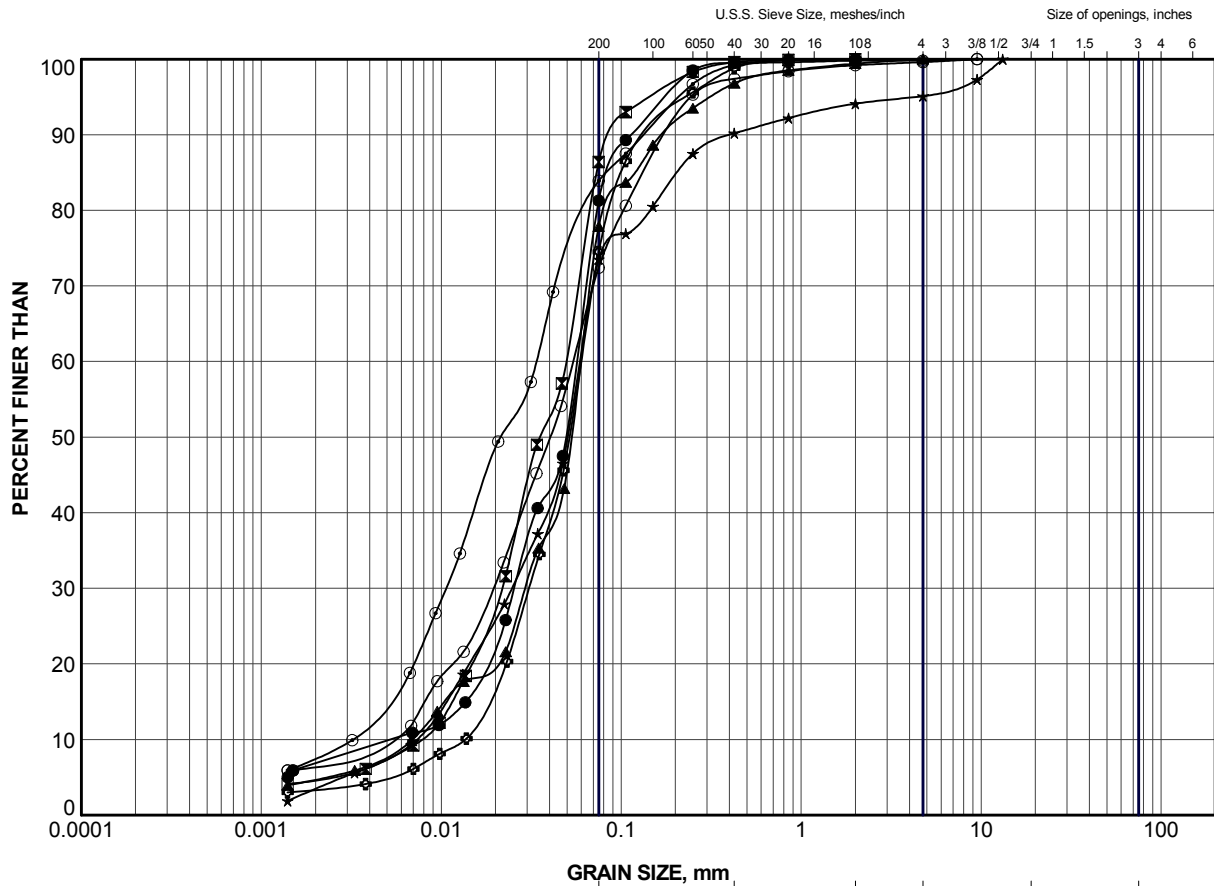
CONSOLIDATION TEST SUMMARY

FIGURE B10
pg 2 of 4








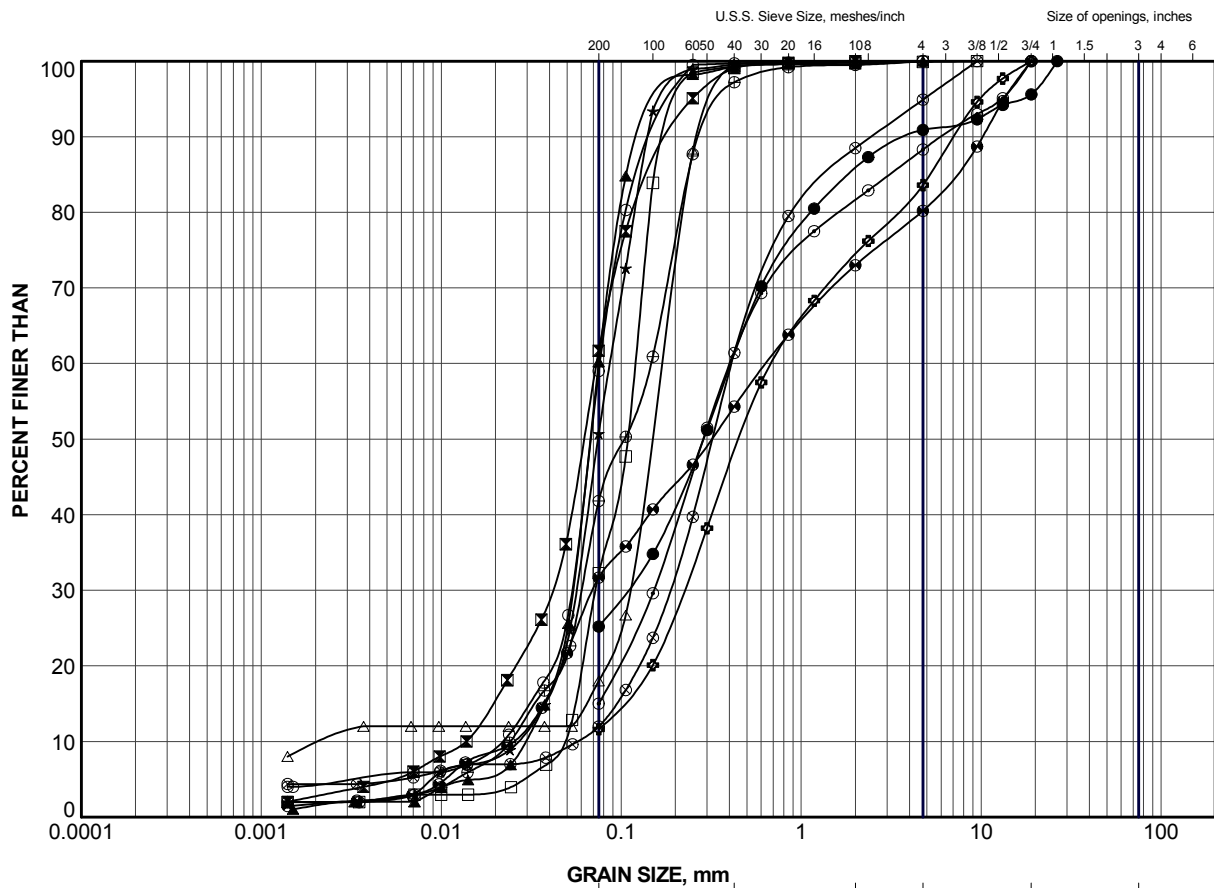


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	S-3	13	210.9
⊠	S-4	15	211.1
▲	S-5	11	217.5
★	S-6	12	215.7
⊙	S-7	10	213.7
⊕	ST-1	11	213.1
○	ST-2	12	212.7

PROJECT					
HIGHWAY 141 SHADOW RIVER BRIDGE					
TITLE					
GRAIN SIZE DISTRIBUTION SANDY SILT to SILT					
PROJECT No.		1651997		FILE No. 1651997.GPJ	
DRAWN	TB	Mar 2018	SCALE	N/A	REV.
CHECK	AC/SEMP	Mar 2018			
APPR	JMAC	Mar 2018			
 Golder Associates SUDBURY, ONTARIO			FIGURE B11		



CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	S-1	14	214.2
⊠	S-4	17	205.3
▲	S-8	11	212.3
★	S-9	13	211.4
⊙	ST-1	13	210.0
⊕	ST-2	14	209.6
○	ST-3	12	211.4
△	ST-4	13	208.6
⊗	ST-5	9	216.5
⊕	ST-6	10	213.1
□	ST-7	11	212.8
⊗	ST-8	15	210.4

PROJECT

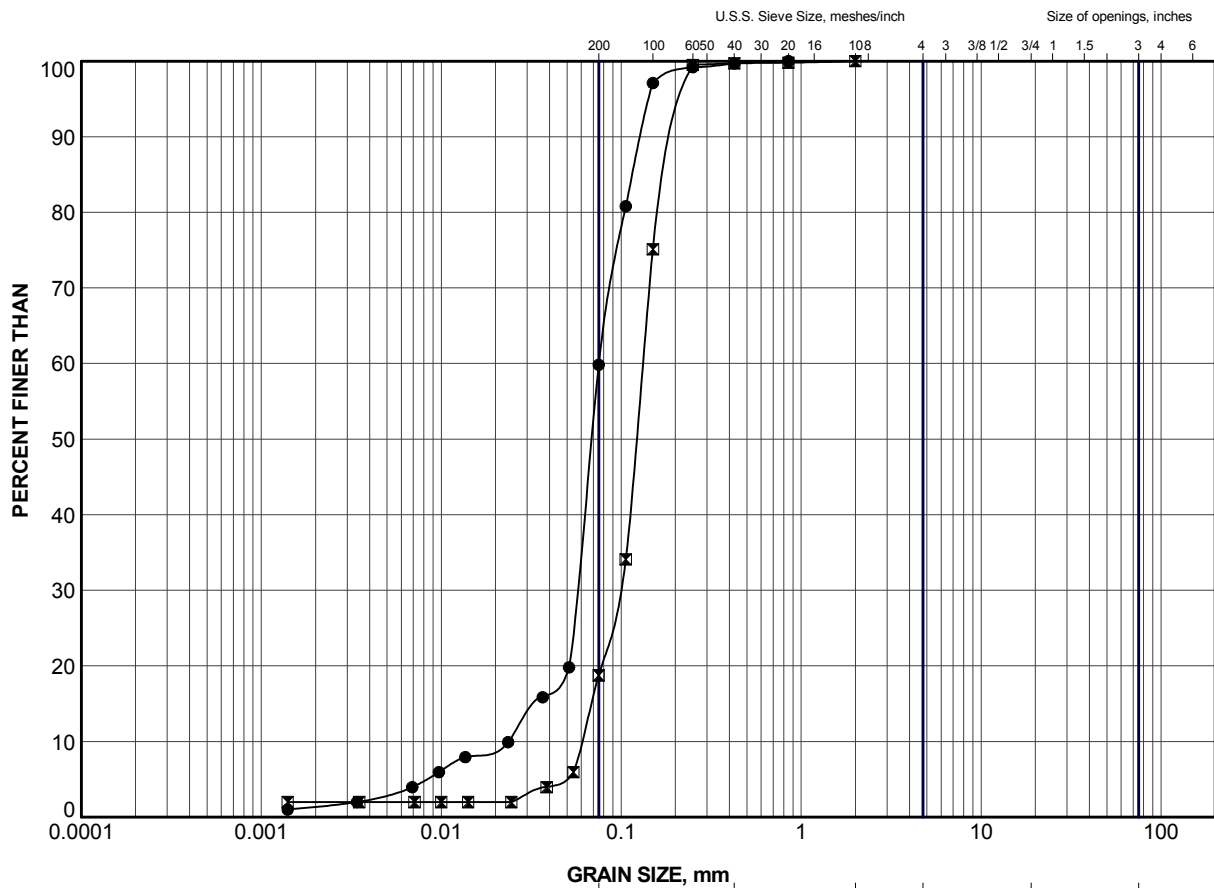
HIGHWAY 141
SHADOW RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION
SILT and SAND to SAND




PROJECT No.		1651997	FILE No.		1651997.GPJ
DRAWN	TB	Mar 2018	SCALE	N/A	REV.
CHECK	AC/SEMP	Mar 2018	FIGURE B12.1		
APPR	JMAC	Mar 2018			



GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	S-10	13	210.9
×	ST-7	14	208.2

PROJECT					
HIGHWAY 141 SHADOW RIVER BRIDGE					
TITLE					
GRAIN SIZE DISTRIBUTION SILT and SAND to SAND					
PROJECT No.		1651997		FILE No. 1651997.GPJ	
DRAWN	TB	Mar 2018	SCALE	N/A	REV.
CHECK	AC/SEMP	Mar 2018			
APPR	JMAC	Mar 2018			
 Golder Associates SUDBURY, ONTARIO			FIGURE B12.2		

Borehole S-2



Box 1: 17.7 m – 20.8 m

Borehole S-3



Box 1: 20.6 m – 23.7 m

Borehole S-6



Box 1: 16.9 m – 19.9 m

Borehole S-9

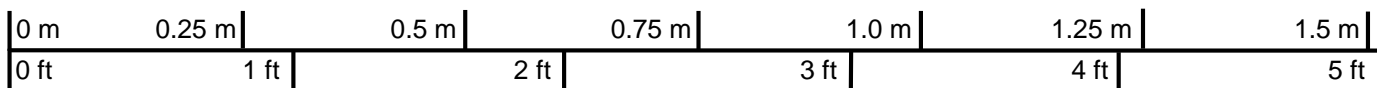


Box 1: 25.5 m – 28.7 m

Borehole ST-1



Box 1: 17.4 m – 20.4 m



Scale

PROJECT

**HIGHWAY 141
SHADOW RIVER BRIDGE**

TITLE

BEDROCK CORE PHOTOGRAPHS



GOLDER

PROJECT No. 1651997

FILE No. ----

DESIGN LP Feb 18

SCALE NTS

REV.

CADD -- Feb 18

CHECK AC Feb 18

REVIEW

FIGURE B13.1


Figure 1 shows three long, dark, cylindrical objects, likely cores, laid out horizontally on a wooden surface. The objects are labeled "END OF CORE 66" and "END OF CORE 67".

A photograph showing three sections of a dark, cylindrical drill core. The sections are arranged horizontally and are labeled with white tape. The top section is labeled '697' 1st RUN'. The middle section is labeled '747' 2nd RUN'. The bottom section is labeled '767' END. 3rd RUN'. The core material appears to be a dark, possibly metallic or mineral, material with some visible texture and some lighter-colored areas. The sections are resting on a light-colored wooden surface.

A photograph showing three long, narrow, dark, textured objects, possibly ancient scrolls or artifacts, laid out horizontally on a wooden surface. The objects have a mottled, dark brown and black appearance, suggesting they might be made of wood or a similar material. They are positioned parallel to each other, with the top one slightly offset to the left. The wooden surface they rest on is light-colored and shows some wear and grain.

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

PROJECT	<h2 style="margin: 0;">HIGHWAY 141</h2> <h2 style="margin: 0;">SHADOW RIVER BRIDGE</h2>
TITLE	<h1 style="margin: 0;">BEDROCK CORE PHOTOGRAPHS</h1>

 GOLDER	PROJECT No. 1651997		FILE No. ----			
	DESIGN	LP	Feb 18	SCALE	NTS	REV.
	CADD	-- --		FIGURE B13.2		
	CHECK	AC	Feb 18			
	REVIEW					

Golder Associates Ltd.

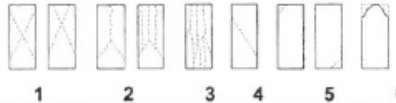
33 Mackenzie Street
Sudbury, Ontario, Canada P3C 4Y1
Telephone: (705) 524-6081
Fax: (705) 524-1984

**SUMMARY OF ROCK CORE TEST DATA**

PROJECT NO.: 1541608-14000
PROJECT NAME: MTO/5015-E-0012/NER Retainer
TYPE OF UNIT: Rock Core
TESTED BY: JP
DATE TESTED: January 31, 2018

GOLDER LAB NUMBER	S105	S106			
BOREHOLE NUMBER:	ST-6	ST-7			
SAMPLE NUMBER:	N/A	N/A			
DEPTH OF TESTED CORE (ft)	62'9"	94'2"			
LENGTH AS CUT (mm)	100.6	101.1			
DIAMETER (mm)	47.5	47.5			
DENSITY (kg/m3)	2633	2625			
COMPRESSIVE STRENGTH (KN)	202.3	116.9			
CORRECTED STRENGTH (MPa)	114.0	66.0			
TYPE OF FRACTURE	3	3			

Type of Fracture



COMMENTS:

Input by: SM
Reviewed by: [Signature]

PROJECT

**HIGHWAY 141
SHADOW RIVER BRIDGE**

TITLE

SUMMARY OF ROCK CORE TEST DATA

PROJECT No. 1651997			FILE No. ----		
DESIGN	LP	Feb 18	SCALE	NTS	REV.
CADD	--		FIGURE B14.1		
CHECK	AC	Feb 18			
REVIEW					

Golder Associates Ltd.

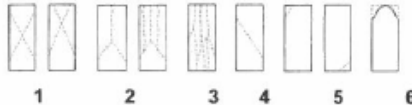
33 Mackenzie Street
Sudbury, Ontario, Canada P3C 4Y1
Telephone: (705) 524-8851
Fax: (705) 524-1994

**SUMMARY OF ROCK CORE TEST DATA**

PROJECT NO.: 1651997/1003 Shadow River
PROJECT NAME: AECOM/5015-E-0045/NE Retainer
TYPE OF UNIT: Rock Core
TESTED BY: JP
DATE TESTED: January 31, 2018

GOLDER LAB NUMBER	S107	S108			
BOREHOLE NUMBER:	S-6	S-9			
SAMPLE NUMBER:	N/A	N/A			
DEPTH OF TESTED CORE (ft)	59'8"	86'11"			
LENGTH AS CUT (mm)	101.1	100.7			
DIAMETER (mm)	47.8	47.2			
DENSITY (kg/m3)	2596	2664			
COMPRESSIVE STRENGTH (KN)	141.8	118.8			
CORRECTED STRENGTH (MPa)	79.2	67.8			
TYPE OF FRACTURE	3	3			

Type of Fracture



COMMENTS:

Input by: SM
Reviewed by: [Signature]

PROJECT

**HIGHWAY 141
SHADOW RIVER BRIDGE**

TITLE

SUMMARY OF ROCK CORE TEST DATA

PROJECT No. 1651997			FILE No. ----		
DESIGN	LP	Feb 18	SCALE	NTS	REV.
CADD	--		FIGURE B14.2		
CHECK	AC	Feb 18			
REVIEW					

Golder Associates Ltd.

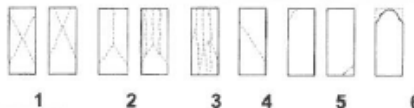
33 Meckenzie Street
Sudbury, Ontario, Canada P3C 4Y1
Telephone: (705) 524-8861
Fax: (705) 524-1984

**SUMMARY OF ROCK CORE TEST DATA**

PROJECT NO.: 1651997 1003
PROJECT NAME: AECOM/5015-E-0045/NE Retainer
TYPE OF UNIT: Rock Core
TESTED BY: JM
DATE TESTED: March 29, 2017

GOLDER LAB NUMBER	C225	C232			
BOREHOLE NUMBER:	ST-1	ST-4			
SAMPLE NUMBER:	n/a	n/a			
DEPTH OF TESTED CORE (ft)	58.5	72			
LENGTH AS CUT (mm)	95.5	95.3			
DIAMETER (mm)	47.5	47.5			
DENSITY (kg/m ³)	2586	2629			
COMPRESSIVE STRENGTH (KN)	283.2	245.8			
CORRECTED STRENGTH (MPa)	159.8	138.9			
TYPE OF FRACTURE	1	1			

Type of Fracture



COMMENTS:

Input by: SB
Reviewed by: [Signature]

PROJECT

**HIGHWAY 141
SHADOW RIVER BRIDGE**

TITLE

SUMMARY OF ROCK CORE TEST DATA

PROJECT No. 1651997			FILE No. ----		
DESIGN	LP	Feb 18	SCALE	NTS	REV.
CADD	--		FIGURE B14.3		
CHECK	AC	Feb 18			
REVIEW					

Golder Associates Ltd.

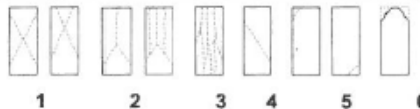
33 Mackenzie Street
Sudbury, Ontario, Canada P3C 4Y1
Telephone: (705) 524-8881
Fax: (705) 524-1664

**SUMMARY OF ROCK CORE TEST DATA**

PROJECT NO.: **1651997 1003**
PROJECT NAME: **AECOM/5015-E-0045/NE Retainer**
TYPE OF UNIT: **Rock Core**
TESTED BY: **SA**
DATE TESTED: **March 29, 2017**

GOLDER LAB NUMBER	C241	C248	C254	C262	
BOREHOLE NUMBER:	S2	S3	ST-3	ST-2	
SAMPLE NUMBER:	0	0	0	0	
DEPTH OF TESTED CORE (ft)	59'-10"	69'-6"	67'-2"	65'-2"	
LENGTH AS CUT (mm)	94.4	91.9	94.7	94.6	
DIAMETER (mm)	47.4	47.5	47.4	47.0	
DENSITY (kg/m3)	2678	2670	2662	2661	
COMPRESSIVE STRENGTH (KN)	174.8	280.9	232.2	153.6	
CORRECTED STRENGTH (MPa)	99.1	158.5	131.6	88.5	
TYPE OF FRACTURE	2	3	4	2	

Type of Fracture



COMMENTS:

Input by: SB
Reviewed by:

PROJECT

**HIGHWAY 141
SHADOW RIVER BRIDGE**

TITLE

SUMMARY OF ROCK CORE TEST DATA

PROJECT No. 1651997			FILE No. ----		
DESIGN	LP	Feb 18	SCALE	NTS	REV.
CADD	--		FIGURE B14.4		
CHECK	AC	Feb 18			
REVIEW					

APPENDIX C

Contract Specifications

CELLULAR CONCRETE - Item No.

Special Provision

CELLULAR CONCRETE

1.0 SCOPE OF WORK

This specification covers the requirements for the supply and placement of lightweight cellular concrete used as embankment fill in accordance with the contract drawings. The cellular concrete shall be placed in dry conditions and above the groundwater table.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications.

Ontario Provincial Standard Specifications, Construction

OPSS.PROV 517	Dewatering
OPSS.PROV 539	Temporary Protection System

Ontario Provincial Standard Specifications, Material

OPSS 1301	Cementing Materials
OPSS 1302	Water
OPSS.PROV 1303	Admixtures for Concrete
OPSS.PROV 1350	Concrete – Materials and Production

American Society for Testing and Materials (ASTM)

ASTM C 150	Portland Cement
ASTM C 869	Standard Specification for Foaming Agents Used in Making Preformed Foam for Cellular Concrete
ASTM C 796	Standard Test Method for Foaming Agents for Use in Producing Cellular Concrete Using Preformed Foam
ASTM C 495-99a	Standard Test Method for Compressive Strength of Lightweight Insulating Concrete

Ministry of Transportation Publication:

LS-407 Method of Test for Compressive Strength of Moulded Cylinders

3.0 DEFINITIONS

For the purpose of this special provision, the following definitions apply:

Cellular Concrete: Cellular concrete is a material with flowable consistency during placement, produced by the substitution of a uniform cellular structure of air cells (voids) for some or all of the aggregate particles found in standard concretes.

Production Lot: The quantity of cellular concrete produced for a continuously placed lift of cellular concrete.

Contractors Engineer: A Foundation Engineer, licensed in the Province of Ontario, with a minimum of five (5) years experience related to the design and/or construction of cellular concrete of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality assurance services for the work at a minimum of two (2) projects of similar scope to the Contract.

4.0 SUBMISSION AND DESIGN REQUIREMENTS

4.1 Prequalification of Cellular Concrete Product

Prior to the commencement of work, the Contractor shall submit to the Contract Administrator a statement from the Supplier verifying that the Supplier has successfully completed the MTO Prequalification Process for Lightweight Fill and confirming that the product has been prequalified for use as lightweight fill by the MTO.

4.2 Qualifications

The Contractor shall submit a resume of the contractor's experience in the production and placement of cellular concrete. The resume shall include the qualifications of contractor's superintendent and/or foreman. The resume shall be submitted to the Contract Administrator for information purposes a minimum of three weeks prior to the start of cellular concrete construction.

The Contractor shall have satisfactorily completed at least five (5) projects of similar nature and complexity during the last three (3) years.

Workers, including the Contractor's superintendent and/or foreman, shall be qualified by the foaming agent manufacturer for production of foam for use in cellular concrete and thoroughly trained and experienced in the production and placement of cellular concrete.

At the commencement of the work, the Contractor shall have on site a representative of the cellular concrete supplier to advise on recommended construction procedures. The Contractor shall also have on site a representative of the cellular concrete supplier at all times during the placement of cellular concrete.

4.3 Submission of Shop Drawings

At least six weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the shop drawings and a method statement that provides full details of the materials and construction procedure.

The contractor shall submit method statements with full details of the following:

- a) Foundation excavation and preparation;
- b) Forming each cellular concrete lift;
- c) Placement of cellular concrete including lift thickness and plan dimensions on a lift by lift basis;
- d) Control of cellular concrete placement to minimize discharge of liquid slurry or foam breakdown products into lifts;
- e) Verification that liquid slurry has been fully removed from discharge hoses or pipes between plant and discharge point prior to each placement event;
- f) Obtaining and measuring the required surface slope at the top of the cellular concrete mass;
- g) Protecting the top cellular concrete surface from damage during pavement structure placement and compaction;

- h) Placement of pavement subbase material;
- i) Placement of side slope cover;
- j) Placement of cellular concrete adjacent to/above the existing EPS/concrete slab including pre-filling of any gaps;
- k) Placement of bond break between concrete abutment wall concrete and cellular concrete;
- l) Protection of cellular concrete during precipitation and freezing weather; and
- m) Quality Control Plan.

Submittals shall also include:

- a) Mix design(s) identifying proportions of foaming agents, water, air, and Portland cement;
- b) Expected density and rate of strength gain showing data from 7 day and 28 day tests on similar mix designs;
- c) Expected in-place volumetric change during and following placement and during curing;
- d) Site layout of mixing plant, delivery hoses and discharge points; and
- e) Results of Quality Control (QC) testing (as outlined further in Section 8.1)

4.4 Submission of Certificate of Conformance

The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Contractors Engineer upon completion of the cellular concrete work and prior to any other backfilling. The Certificate shall state that the work has been carried out in conformance with the contract documents, specifications and/or stamped working drawings.

4.5 Submission of Environmental Protection Strategy

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of an environmental protection strategy as specified under Section 7.5.

5.0 MATERIALS

5.1 Cementing Materials

Cementing materials shall be according to OPSS 1301. Supplementary cementing materials shall not be used.

5.2 Water

Water shall be free of contamination and any deleterious substance. Water shall conform to OPSS 1302.

5.3 Admixtures

Admixtures shall conform to OPSS.PROV 1303.

5.4 Foaming Agents

Foaming agents shall conform to the requirements of ASTM C 869 when tested in accordance with the provisions of ASTM C 796. The Subcontractor shall be pre-qualified and approved in writing by the foaming agent manufacturer referencing this Project.

5.5 Cellular Concrete Properties

Cellular concrete shall be prequalified by the MTO Lightweight Fill Committee Prequalification Process and have the following properties:

- a) Minimum unconfined compressive strength at 28 days of 1 MPa.
- b) Wet cast density of 510 kg/m³ (5.0 kN/m³) (+/-10%)
- c) Must not contain any other waste or process by-product including fly ash.
- d) In place density of 510 kg/m³ (5.0 kN/m³) as measured on cores obtained from the full lift thickness

6.0 EQUIPMENT

The specialized batching, mixing, and placing equipment shall be automated and certified for the purpose by the manufacturer of the cellular concrete material. Dry-mix equipment must be able to receive bulk cement and produce over 100 cubic metres per hour on-site, continuously, from one piece of equipment, and pump through hoses or pipes up to a flat lineal distance of 200 metres. Bulk cement shall be weighed on a scale that operates within a tolerance of one and one-half percent (1.5%) per batch. Wet-mix equipment must be able to receive slurry on-site into the equipment and process it continuously during ready-mix supply, and pump through hoses or pipes up to a flat lineal distance of 200 metres.

Cellular concrete must be pumped by a positive displacement pump (Peristaltic or similar). A foam generator shall be used to continuously produce pre-formed foam which shall be injected and mixed with the cementitious slurry downstream of the positive displacement slurry pump. The equipment shall be calibrated to produce a precise and predictable volumetric rate of foam with stable uniform microbubbles.

7.0 CONSTRUCTION

7.1 Foundation Excavation and Subgrade Preparation

Excavation to the subgrade level shall be carried out to the design elevations and horizontal and vertical limits shown on the drawings. Any softened, loosened or deleterious materials at the subgrade elevation shall be subexcavated and replaced with additional cellular concrete. The existing EPS shall be partially removed as shown on the drawings. The existing concrete cap covering the existing EPS shall be partially removed as shown on the drawings.

The prepared subgrade shall be good competent level ground adjacent to the abutment. Water, snow and ice must be removed from the area prior to placement. Elsewhere, the cellular concrete shall be placed directly on the existing EPS or on the existing concrete cap.

7.2 Dewatering

The prepared subgrade shall be free of standing water during placement of cellular concrete and until backfill is placed on top of the cellular concrete. If necessary, dewatering shall be continuous during placement of materials.

Dewatering shall be according to OPSS.PROV 517.

7.3 Protection System

The construction of all protection schemes shall be according to OPSS.PROV 539 and paid for under the appropriate tender item. Where the stability, safety or function of an existing roadway, railway, other works, or proposed works may be impaired due to the method of operation, such protection as may be required shall be provided by the Contractor.

7.4 Cellular Concrete Placement

Any items to be fully or partially encased in the cellular concrete shall be properly set and stable prior to the installation of the cellular concrete. The Contractor shall provide positive means of preventing uplift and any other movement of embedded items during installation of cellular concrete. The cellular concrete shall be placed adjacent to/over the existing EPS/concrete cap such that the EPS is not moved/misaligned from its current position. All gaps in the existing EPS/concrete cap shall be pre-filled with spray foam prior to placing cellular concrete. The polyethylene sheeting shall be reinstated above the existing EPS in the area(s) where the concrete cap is removed. A bond break shall be placed between the abutment wall and cellular concrete.

Cellular concrete shall under no circumstances be placed during freezing conditions.

Cellular concrete must not be placed during precipitation and shall be protected from precipitation until initial set has been achieved.

Once mixed, the cellular concrete shall be conveyed promptly to the location of placement without excessive handling. The maximum flat line pumping distance through hoses or pipes shall be limited to 200 m regardless of wet-mix or dry-mix placement methods. Initial discharge of cellular concrete that has accumulated in the discharge lines during prior placements or any cellular concrete mix that has not been fully aerated shall be wasted prior to discharge into the intended lift. Cellular concrete shall not be discharged into the intended lift after the foam generator has been turned off.

The maximum lift thickness shall be determined based on density and any other considerations that may affect placement but shall not exceed 650 mm. Cellular concrete shall be cast in a formed area within 1 to 2 hours, to permit undisturbed curing. Foot traffic within the cellular concrete mass shall not be permitted.

Finished surface elevation shall be within ± 25 mm of the design grades shown on the drawings. Cellular concrete surface slopes greater than 1%, if required, shall be created in accordance with the Contractor's approved method statement and shop drawings. Flat benches or slopes of less than 2% will not be permitted for cellular concrete. Sloped concrete caps, trimming of cellular concrete or other pre-approved methods shall be used to obtain appropriate drainage slopes.

Loading of, or traffic on, the cellular concrete shall be prevented until the material has attained sufficient strength to withstand the loads with no damage. Backfilling can commence when cellular concrete supports foot traffic without leaving an indentation.

7.5 Environmental Protection

The Contractor shall handle materials and conduct the work in a manner that will ensure protection of the natural environment and prohibit cellular concrete from entering surface or ground water. The Contractor shall take measures as necessary to prevent the material from entering the natural environment and/or leaking outside of the intended placement location, and shall have established methods for stopping flow of the product as required, and for prompt remediation of any leaks or spills. These measures and any other contingency planning requirements shall be documented in an Environmental Protection Strategy.

8.0 QUALITY CONTROL

8.1 Sampling Frequency and Methods

Fresh cellular concrete shall be collected for density testing once per production run, or once for every 25 cubic metres, or once per 15 minutes, whichever is more frequent. The unit weight shall be maintained within +/- 10% of the design unit weight and shall be adjusted as required to obtain the specified density at the point of placement. Routine density testing shall be carried out on samples of fresh cellular concrete collected from the standing pool of fresh cellular concrete and directly from the delivery hose discharge point with points of collection recorded for each sampling and testing instance.

Cellular concrete samples shall also be captured, cured, and tested at the point of placement to verify the specified compressive strength and the dry unit weight. One sample shall be taken for each placement lift, or every 100 m³, whichever is more frequent. One sample is comprised of one set of six cellular concrete cylinders, with cylinders cast in 75 mm by 150 mm cylindrical plastic molds. Cellular concrete cylinders shall be cured and tested for density and compressive strength as per ASTM C495-99a and LS 407. One cylinder shall be collected from the fresh cellular concrete by hand from the full depth of the standing pool of cellular concrete, one directly from the delivery hose discharge point during production placement with the remaining four in accordance with the approved quality control plan.

Three core samples shall be obtained for each placement volume or lift, or every 100 m³, whichever is more frequent prior to placement of subsequent lifts. Each core sample shall be measured for weight and volume, wrapped in plastic and stored for curing with other quality control test cylinders.

8.2 Production Data Records

The following data shall be recorded for each placement, lift or 100 m³ of in-place cellular concrete, whichever is more frequent:

- a) Confirmation that fluids have been removed from hoses and pipes prior to placement;
- b) Lengths of hoses and pipes from plant to discharge point;
- c) Times of foam generator start and stop;
- d) Times of slurry flow start and stop;
- e) Quantity of materials wasted from hoses and pipes prior to and following production discharge;
- f) Fluid concrete sampling times, locations, sample numbers and curing conditions;
- g) Core sampling times, locations, sample numbers and curing conditions;
- h) Production volume discharged based on flow rates, wasted materials, and plant input quantities;
- i) Dimensions and volumes of placement based on physical measurements at completion of placement and prior to subsequent lifts or backfilling;
- j) Measurements and photographs of slope of cellular concrete surface at completion; and
- k) Results of all laboratory tests on cured cellular concrete cylinders and cores.

8.3 Acceptance/Rejection

Failure of any one of the samples to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the production lot or any alternative mitigation accepted by the Contract Administrator shall be at the Contractor's expense.

9.0 MEASUREMENT FOR PAYMENT

Measurement for payment shall be the calculated neat volume in cubic meters specified to consist of cellular concrete within the theoretical lines and grades shown in the stamped working drawings. In no case will placed volumes be determined by multiplying the known volume of slurry by the ratio of slurry density to average cellular concrete density (expansion ratio).

10.0 PAYMENT

10.1 Basis of Payment

Payment at the contract price for the above tender item shall be full compensation for all labour, materials, and equipment to do the work as described above.

ROCK EMBANKMENT – Item No.

Non-Standard Special Provision

Amendment to OPSS.PROV 206, November 2014

206.07 CONSTRUCTION

Section 206.07.05.02 Rock Embankments

Section 206.07.05.02.01 shall be amended by the addition of the following:

The Contactor shall provide rock fill with maximum particle size of 300 mm in the upper 0.5 m of the temporary detour embankment to facilitate placement of the geotextile and Granular A base material. Rock fill with maximum particle size of 300 mm shall be used in the area of the temporary modular bridge abutments to facilitate pile driving. No particles greater than 1.0 m shall be used in the rock fill embankment (i.e. rock fragments up to 3.0 m diameter are not permitted).

OPERATIONAL CONSTRAINT – Use of Heavy Equipment

Special Provision

The use of heavy construction equipment and in particular, heavy lift cranes, will be required during construction of the highway bridge and temporary modular bridge. The impact of the heavy equipment loads on the underlying very soft to stiff clayey silt to clay soils, river banks and existing foundations must be considered during selection of the methodology and equipment employed for construction.

Prior to commencement of construction, the Contractor shall retain a Geotechnical Consultant to assess the impact of the proposed equipment loads and methodology, and determine requirements and/or restrictions necessary to safely support the loads. All foundation engineering services required for this project shall be performed by a firm listed under MTO's RAQS for providing services under the specialty of Geotechnical (Structures and Embankments) – High Complexity.

The geotechnical assessment carried out by the Contractor's Geotechnical Consultant shall include, but not be limited to, the following:

- Review of available geotechnical information and supplementing with additional subsurface information, as required, in the equipment pad/access road areas;
- Determine appropriate and safe setbacks for heavy equipment from the river banks, from the crest(s) of new and existing fill embankments, and from existing foundations;
- Determine the permissible ground pressure (with due consideration to both bearing capacity and global stability) that may be applied to the foundation soils and/or embankment fills by the equipment;
- Provide recommendations for crane pad design to transfer the crane loads for launching the Temporary Modular Bridge (TMB) to the embankment and ground;
- Provide recommendations for deep (i.e. pile) foundations to support the crane loads or other heavy equipment loads, if required;
- Provide recommendations for the distribution and support of all heavy equipment loads (including crane and pile-driving equipment loads) to prevent foundation failure (either in bearing capacity or in global stability) at any locations along the new and existing fill embankments, access roads and at equipment pads based on the proposed methodology of the Contractor.
- Provide recommendations for temporary support of the TMB during the over-winter condition.

The Contractor shall submit the findings of the geotechnical assessment and details of the proposed equipment and construction methodology to the Contract Administrator for information purposes a minimum of two weeks prior to the start of construction.

At no time shall any heavy equipment be parked on the portion of the existing highway embankments where the existing expanded polystyrene (EPS) will be left in place. Should this occur, the Contractor shall be responsible for replacement of the EPS at his own cost.

All costs for these Consultant fees, and any resulting measures on site, shall be borne by the Contractor.

OPERATIONAL CONSTRAINT – Stockpiling of Material

Special Provision

Stockpiling of excavated soils and/or construction materials including granular material shall not be permitted on the detour embankment within 100 m of the TMB. Stockpiles shall not be permitted on the portion(s) of the existing embankment where the existing expanded polystyrene (EPS) will be left in place.

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision No. FOUN0003

March 8, 2018

Amendment to OPSS 902, November 2010

OPSS 902, November 2010, Construction Specification for Excavating and Backfilling – Structures, is amended as follows:

902.02 REFERENCES

Section 902.02 of OPSS 902 is amended by the addition of the following:

Ontario Provincial Standard Specifications, Construction

OPSS 517 Dewatering
OPSS 805 Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

Section 903.03 of OPSS 902 is amended by the addition of the following:

Automatic Transfer Switch means as defined in OPSS 517.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means as defined in OPSS 517.

Design Storm Return Period means as defined in OPSS 517.

Dewatering System means as defined in OPSS 517.

Groundwater Control System means as defined in OPSS 517.

Plug means as defined in OPSS 517.

Sediment means as defined in OPSS 517.

Sediment Control Measure means as defined in OPSS 517.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Vegetated Discharge Area means as defined in OPSS 517.

Waterbody means as defined in OPSS 517.

Watercourse means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.01 Design Requirements

902.04.01.01 Dewatering

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a 10 year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

902.04.02 Submission Requirements

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

902.04.02.01 Working Drawings

Working Drawings for the dewatering system shall be according to OPSS 517.

902.04.02.02 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, utilities, and structures, within a distance of 150 metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website <https://www.ontario.ca/environment-and-energy/map-well-records> or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

902.04.02.03 Milestone Inspections

Clause 902.04.02.03 of OPSS 902 is deleted in its entirety.

902.07 CONSTRUCTION

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

902.07.04 Dewatering Structure Excavation

902.07.04.01 General

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

902.07.04.02 Discharge of Water

The discharge of water shall be according to OPSS 517.

902.07.04.03 Monitoring

Monitoring shall be according to OPSS 517.

902.07.04.04 System Amendments

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

902.07.04.05 Removal

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.

EXCAVATION FOR STRUCTURE – Item No.

Non-Standard Special Provision

Amendment to OPSS 902, November 2010

902.07 CONSTRUCTION

Section 902.07 shall be amended by the addition of the following:

The Contactor is hereby notified that at the existing/new highway bridge site and temporary modular bridge site, the embankment fill contains cobbles and boulders and the native soils contain cobbles and boulders overlying bedrock, which could affect excavations and the installation of deep foundations. The fill also contains wood pieces. Consideration of the presence of these obstructions must be made in selection of appropriate equipment and procedures for excavations or temporary works.

NOTICE TO CONTRACTOR - Obstructions

Special Provision

The Contactor is hereby notified that at the existing/new highway bridge site and temporary modular bridge site, the embankment fill contains cobbles and boulders and the native soils contain cobbles and boulders overlying bedrock, which could affect excavations and the installation of deep foundations. The fill also contains pieces of wood. Consideration of the presence of these obstructions must be made in selection of appropriate equipment and procedures for installation of the foundations and for excavation and construction of temporary works.



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