



December 5, 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

**SWAMP CROSSINGS/HIGH FILL AREAS, DEEP CUT AND
EXCESS MATERIAL MANAGEMENT AREA
REALIGNMENT OF HIGHWAY 66 AT VIRGINIATOWN FROM 10.6 KM EAST OF
HIGHWAY 624 EASTERLY 3.4 KM
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5091-07-00**

Submitted to:

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REPORT





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NSSP
NSSP
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OC
OC

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Excess Material Management Area

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Foundation Monitoring Plan



PART A

FOUNDATION INVESTIGATION REPORT
SWAMP CROSSING / HIGH FILL AREAS, DEEP CUT AND EXCESS
MATERIAL MANAGEMENT AREA
REALIGNMENT OF HIGHWAY 66 AT VIRGINIATOWN
FROM 10.6 KM EAST OF HIGHWAY 624 EASTERLY 3.4 KM
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5091-07-00



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin (MRC), a member of MMM Group Limited (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for four swamp crossing/high fill embankments and one deep cut area within the limits of the new Highway 66 realignment to the east of the junction with Highway 624, as well as for the excess material management (EMM) area. The proposed work is part of the overall Highway 66 realignment from 10.6 km east of Highway 624 easterly 3.4 km. The foundation engineering components within the overall project limits include the engineering of: high fill embankments and embankments over swamps; a deep cut section; an excess material management area; as well as a number of culverts. The proposed foundation embankment sections along the Highway 66 realignment extend from approximately 11.0 km east of the junction of Highway 66 and Highway 624 easterly for a total distance of about 2.5 km. The general location of this section of the new Highway 66 alignment is shown on the Index Plan on Drawing 1.

The Terms of Reference (TOR) for the foundation investigation are outlined in MTO's Request for Proposal, dated October 2010. Golder's proposal (Scope of Work) for foundation engineering services is contained in Section 6.8 of MRC's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated February 25, 2011. The Base Plan showing the proposed horizontal alignment of the Highway 66 realignment was provided to Golder by MRC.

This report addresses the investigation carried out for the Highway 66 realignment swamp crossing/high fill areas, deep cut area and EMM area only. A detailed list of the swamp crossing/high fill areas, deep cut area and EMM area is presented in Table 1. Separate reports address the foundation investigations for the culvert crossings.

The purpose of this investigation is to establish the subsurface conditions along the new highway alignment at the proposed swamp crossing/high fill areas, deep cut area and EMM area by methods of borehole drilling, rock coring, in situ testing and laboratory testing on selected samples. The centreline of the proposed highway realignment was staked in the field by MRC and the foundation investigation was carried out as defined in the TOR.

2.0 SITE DESCRIPTION

The new Highway 66 alignment is oriented generally in an east-west direction within the Township of McGarry. The section of the new highway realignment is approximately 2.5 km long and extends easterly from about 11.0 km north of the junction of Highway 66 and Highway 624. The proposed new alignment within this section of the highway is up to about 500 m northwest of the existing Highway 66 alignment. Along the proposed realignment the land use varies from industrial (i.e., formerly mining), recreational (ATV and snowmobile trails) to residential.

In general, the topography of this section of the highway consists of rolling terrain, including moderately to densely populated treed areas with two bedrock outcrops separated by large low-lying swamp areas, occasional ponded/standing water and various types of vegetation and surface organics. The ground surface within the limits of the swamp crossing/high fill/deep cut/EMM areas varies between about Elevation 302.5 m and Elevation 317.5 m, across the site. The large bedrock outcrops at the west and east ends of the site rise up to about Elevation 344 m and 330 m, respectively. A detailed description of each investigated swamp



crossing/high fill/deep cut/EMM area is presented in Section 4.0. The locations of these areas are shown in plan on Drawing 1.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The subsurface investigation for the Highway 66 realignment swamp crossing/high fill areas, deep cut area and EMM area was carried out between July 26 and August 6, 2011, between July 26 and November 15, 2012, and between May 5 and May 25, 2013, during which time a total of one hundred and eleven (111) boreholes, seven (7) Cone Penetration Tests (CPTs) and thirty-three (33) Dynamic Cone Penetration Tests (DCPTs) were advanced at the locations summarized in Table 1 and shown on Drawings A1 to F1 in Appendices A to F. In general, boreholes, CPTs and DCPTs were advanced along the centreline and the toes of the proposed embankment. At the deep cut area, which will partially cut through a rock outcrop and an overburden plateau along the south toe of the roadway, the purpose of the investigation is to assess the subsurface conditions along the south toe of the roadway overburden cut as a supplementary assessment to the pavement investigation.

The field investigation completed in 2011 was carried out using track mounted D50 Gas and D50 Turbo drill rigs supplied and operated by Walker Drilling Ltd. of Barrie, Ontario. The field investigation completed in 2012 was carried out using a track mounted CME 55 and portable drilling equipment supplied and operated by Landcore Drilling Inc. of Sudbury, Ontario. The field investigation completed in 2013 was carried out using a track-mounted CME-55 drill rig supplied and operated by George Downing Estate Drilling Ltd. of Grenville sur la Rouge, Quebec.

The boreholes were advanced through the overburden using 108 mm inner diameter hollow-stem augers, and/or 'NW' casing with wash boring techniques. In general, soil samples were advanced at intervals of depth of about 0.75 m, 1.5 m and 3.0 m, using a 50 mm outer diameter (O.D.) split-spoon sampler driven by automatic or cathead hammers on the track-mounted drill rigs, and carried out in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586, Standard Test Method for Standard Penetration Test). Boreholes advanced by portable equipment employed full-weight hammers, dropped from the standard SPT height. Samples of the cohesive soils were obtained at selected locations using 76 mm O.D. thin-walled 'Shelby' tubes (ASTM D1587, Standard Practice for Thin-Walled Tube Sampling) for relatively undisturbed samples. Field vane shear tests were carried out in cohesive soils for assessment of undrained shear strengths (ASTM D2573, Standard Test Method for Field Vane Strength Shear Test) using MTO Standard 'N' size vanes. Samples of the bedrock were obtained using an 'NQ' size rock core barrel. All boreholes not instrumented with a standpipe piezometer were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The boreholes and DCPTs were advanced to depths up to 29.3 m below existing ground surface, generally penetrating 3 m into competent material, which is defined as material that will provide resistance to settlement or instability of the embankments, or to refusal. At some locations, boreholes and DCPTs were terminated on refusal to further auger, casing and/or split-spoon advancement, or dynamic cone penetration. These depths to refusal do not confirm bedrock surface elevations, but may be inferred to indicate potential proximity to the bedrock surface. In thirteen (13) boreholes, bedrock was cored for core lengths between about 3.0 m and 4.1 m and photographs of the recovered rock core samples are provided in the relevant appendices noted in Section 4.

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 11.5 m to 23.6 m, corresponding to the bottom of



the cohesive deposit as defined by the adjacent boreholes. The CPT consists of a special probe equipped with electronic sensing elements to continuously measure tip resistance, local side friction on a sleeve and porewater 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 strength 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. Cone Penetration Test sheets are included with the Record of Borehole sheets in Appendix C. Profiles of tip resistance, friction and porewater 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).

The groundwater conditions and water levels in the open boreholes were observed during the drilling operations and are described on the Record of Borehole sheets provided in Appendices A to F. Piezometers were installed in Boreholes C1-2, BC1-3, BC1A-1, BC2-2, BC5-2 and BC6-2 to permit monitoring of the groundwater levels at their respective locations. Piezometers C1-2, BC5-2 and BC6-2 consist of 20 mm diameter PVC pipe with a 1.5 m long slotted screen sealed within the silty sand to gravelly sand deposit, bedrock and sand and silt to silty sand deposit, respectively. Piezometers BC1-3, BC1A-1, and BC2-2 consist of 50 mm diameter PVC pipe with a 1.5 m long slotted screen sealed within the sand and gravel deposit, silt/gravel deposit and silty sand to gravel deposit, respectively. The borehole annulus surrounding the piezometer screen was backfilled with sand and the remainder of the borehole was backfilled with a bentonite plug and cuttings. All piezometers were decommissioned in May 2013. Borehole H7-5, wherein artesian groundwater was encountered during drilling, was sealed full column with cement grout upon completion of drilling.

Artesian groundwater conditions were encountered within the sand and gravel deposit underlying the clayey silt deposit in Borehole BC1-3 and within the sand deposit underlying the silt deposit in Borehole H7-5. The artesian groundwater level in Borehole BC1-3 was encountered after piezometer installation and was static above ground surface within the piezometer pipe (non-flowing). The piezometer was sealed full column with cement grout on May 19, 2013. Groundwater levels were recorded above ground surface after piezometer installation in Boreholes BC2-2 and BC6-2 at the same level (elevation) as the surface water ponded around the piezometer pipe. Groundwater levels were also measured above ground surface after piezometer installation and about one year later in Borehole BC5-2; however, the piezometer screen was noted to be plugged at the time of decommissioning, likely resulting in water being trapped within the piezometer.

The fieldwork was observed by members of our engineering and technical staff, who located the boreholes, CPTs and DCPTs, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga or Sudbury geotechnical laboratory where the samples underwent visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples. In addition, one-dimensional consolidation (oedometer) tests (both horizontally and vertically trimmed), consolidated isotropic undrained (CIU) triaxial tests with pore pressure measurements and consolidated drained direct shear test were carried out on select samples of the cohesive deposits. The summary of the consolidation test, triaxial tests and direct shear test results are presented in Table 2. The results of the laboratory classification testing are included in the respective appendices.



Classification of the rock mass quality of the bedrock with respect to the Rock Quality Designation (RQD) is described based on Table 3.10 in the Canadian Foundation Engineering Manual (CFEM, 2006)¹. The degree of weathering of the bedrock samples (i.e., fresh to completely weathered) and the strength classification of the intact rock mass based on field identification (i.e., very poor 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. Classification of the bedrock core samples with respect to strength is based on Table 3.5 in CFEM (2006).

The proposed centreline of the new highway alignment was staked in the field by MRC prior to drilling. The as-drilled borehole locations (except for the EMM area), in stations and offsets, were measured in reference to the centreline alignment and were subsequently converted into MTM NAD 83 coordinates in AutoCAD. The as-drilled borehole locations for the EMM area were measured by a handheld global positioning system (GPS) and were subsequently converted into MTM NAD 83 coordinates in ESRI's ArcGIS software. Borehole elevations (except the EMM area) were surveyed by a member of our technical staff in reference to the ground surface elevations at temporary benchmarks. In areas of open water, the depth to firm bottom was measured at the time of drilling and the elevation of the water level was surveyed by a member of our technical staff in reference to the elevations of temporary benchmarks. All temporary benchmarks were installed by MRC prior to the commencement of fieldwork. Borehole elevations for the EMM area were obtained from referencing the borehole locations to the corresponding ground surface elevation in the Ontario Ministry of Natural Resources, Base data MNR LIO Provincial Digital Elevation Model (20m interval). The borehole locations given in the Record of Borehole sheets and shown on Drawings A1 to F1 in Appendices A to F are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

In the Quaternary Period, the Virginiatown area was encompassed by glacial Lakes Barlow and Ojibway. In areas of more turbulent waters in these lakes, coarse grained sediments of sand and gravel were deposited. In the calmer portions of the glacial lakes fine grained sediments consisting primarily of varved clay, were deposited. After Lakes Barlow and Ojibway receded, organic materials were deposited. In the Kirkland Lake area the organic deposits are usually found as fens, bogs and swamps containing varying thicknesses of organics and are often encountered in glaciolacustrine plains (overlying the sand and gravel or clay), along creeks and streams and in bedrock basins.³

Based on NOEGTS⁴ Mapping, the subsoils in the vicinity of the Highway 66 realignment generally consist of till deposited as a ground moraine. A primarily clay/clayey glaciolacustrine deposit is located further than 1 km north of the realignment. The soils along the Highway 66 realignment consist of variable deposits of organic materials, lacustrine sand, silt and clay and till.

¹Canadian 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.

³ C.L. Baker, 1985. Quaternary Geology of the Kirkland Lake Area, Districts of Cochrane and Timiskaming; Ontario Geological Survey.

⁴ Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Map Reference Number 32DSW.



Published literature indicates that the site is located in the Abitibi Subprovince of the Superior Province (OGS, 1991)⁵. The Abitibi Subprovince contains rocks of up to 2.75 Ga in age, is about 800 km by 300 km in area and lies within the southern portion of the Superior Province. Bedrock in this subprovince consists primarily of zones of mafic to intermediate metavolcanic rocks and metasedimentary rocks.

4.2 General Overview of Local Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes and CPTs advanced during this investigation, together with the results of the laboratory tests carried out on selected soil and bedrock core samples, are presented on the attached Record of Borehole and Cone Penetration sheets and the soil laboratory test sheets provided in Appendices A to F. The results of the in situ field tests (i.e., SPT 'N'-values and undrained shear strengths from the field vanes) as presented on the Record of Borehole sheets and in Section 4 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of in situ testing. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole, DCPT and CPT locations. The thickness of the overburden/depth to refusal in the investigated areas as inferred from the resistance to DCPT advancement are shown on the Record of DCPT sheets in Appendices A to F.

The inferred soil stratigraphy as encountered in the boreholes and DCPTs advanced for the Highway 66 realignment high fill areas are shown in profile and cross-section on Drawings A1 to F1, inclusive. The orientation (i.e., north, south, east, west) stated in the text of the report is typically referenced to project north and/or up-chainage (along the proposed Highway 66 alignment). For purposes of this report, Highway 66 is oriented east-west.

In general, the stratigraphy encountered at the various borehole locations typically consists of layers of peat or topsoil underlain by cohesive soils. The overburden (soil material) thickness is variable, with refusal to further auger/split spoon, casing or DCPT advancement measured at depth, ranging from 0.5 m to about 29.3 m below ground surface. The stratigraphy generally consists of:

- surficial layers of fibrous or amorphous peat or topsoil, or silty sand to sandy gravel fill in some locations near the existing highway;
- cohesionless deposits of silty sand to sandy silt;
- cohesive deposits of clayey silt to silty clay to clay, interbedded with silt seams and organic pockets in some areas;
- cohesionless deposits of silt below the cohesive deposits;
- cohesionless deposits of silty sand to gravel below the silt deposits; and
- bedrock (inferred or cored).

⁵ Ontario Geological Survey, 1991. Geology of Ontario, Special Volume 4, Part 1. Eds P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott, Ministry of Northern Development and Mines, Ontario.



Detailed descriptions of the subsurface conditions at each investigated swamp crossing/high fill/excess material management and deep cut areas are provided in the following sections of this report. Where relatively significant thicknesses of overburden were encountered, the various soil types are described in detail for each main deposit.

4.3 Highway 66 – STA 13+080 to 13+185 (High Fill H4) and Highway 66 Connection – STA 10+000 to 10+125 (High Fill H1)

The plan and profiles along the centreline and toes of the proposed Highway 66 realignment embankment showing the borehole locations and interpreted stratigraphy between about STA 13+080 and 13+185 along the Highway 66 realignment and between about 10+000 and 10+125 along the Highway 66 Connection are shown on Drawings A1 and A2 in Appendix A. The realignment extends across a low-lying area bounded by a large rock outcrop to the north and the existing Highway 66 to the south. The proposed embankment in this section of the highway realignment is up to about 6 m high relative to the existing ground surface.

A total of seventeen (17) boreholes (H1-4 to H1-7, H1-6a, H1-9 and H1-10, H4-1 to H4-6, C1-2 and BC1-1 to BC1-3) and five (5) DCPTs (H1-D1, H1-D3, H1-D4, H4-D1 and H4-D2) were completed to investigate the subsurface conditions within the High Fill H1 and H4 areas. The topography in these sections of the proposed highway realignment and connection is characterized by flat terrain and bedrock ridges, sloping downward from north to south. These sections of the proposed realignment contain moderate to dense tree cover and a wet grassy area adjacent to the existing Highway 66.

The subsurface soils generally consist of surficial layers of fill, peat or topsoil, underlain by a cohesive deposit consisting of an upper zone of clayey silt, a middle zone of silty clay to clay and transitioning to a lower zone of clayey silt. The cohesive deposit is underlain by a deposit of silt to sandy silt, which in turn is underlain by a deposit of silty sand to sand to sand and gravel to sandy gravel, underlain by bedrock. Refusal to further split spoon, auger or casing advancement and dynamic cone penetration was encountered at depths of up to about 20.8 m, being deepest in the vicinity of about STA 10+ 125 along the Highway 66 Connection, at the existing Highway 66 location.

4.3.1 Embankment Fill

Borehole H1-10 was advanced through the existing Highway 66 embankment and encountered a 125 mm thick layer of asphalt, with the road surface at approximately Elevation 303.6 m. A deposit of silty sand and gravel to sandy gravel fill was encountered below the asphalt in Borehole H1-10 and from ground surface in Boreholes H1-6 and H1-9, which were advanced through the shoulder of the existing roadway. The fill deposit was encountered between Elevation 303.5 m and 304.2 m and the thickness of the deposit ranges from 1.4 m to 2.2 m.

The SPT 'N'-values measured within the fill deposit range between 9 blows and 85 blows per 0.3 m of penetration, indicating a loose to very dense relative density.

The natural water content measured on one sample of the fill deposit is about 12 per cent.



4.3.2 Peat / Topsoil

A layer of black topsoil ranging from about 0.1 m to 0.2 m thick was encountered from ground surface between Elevation 306.0 m and Elevation 303.5 m in Boreholes H1-4, H1-5, H1-7, H4-1, H4-2, H4-4 to H4-6 and BC1-1 to BC1-3. In Borehole C1-2 and Borehole H4-3, a deposit of black fibrous peat was encountered from ground surface at Elevation 305.4 m and 304.3 m with thicknesses of 0.3 m and 0.8 m, respectively. In Boreholes H1-9 and H1-10, a deposit of black fibrous or amorphous peat was encountered underlying the existing Highway 66 embankment fill at Elevation 301.7 m and 301.5 m with thicknesses of 0.8 m and 0.5 m, respectively.

SPT 'N'-values measured within the peat deposit range from 3 blows to 8 blows per 0.3 m of penetration, suggesting a soft to firm consistency.

The natural water content measured on one sample of the topsoil is about 26 per cent and on two samples of the peat are about 60 per cent and 72 per cent.

4.3.3 Clayey Silt to Clay

A cohesive deposit consisting of an upper zone of clayey silt, a middle zone of silty clay to clay and transitioning in places to a lower zone of clayey silt was encountered underlying the fill, topsoil or peat in the boreholes. The total thickness of the deposit is between about 2.2 m and 12.5 m and the surface of the deposit was encountered between Elevation 305.9 m and 300.9 m.

The upper zone consists of grey to brown clayey silt to gravelly sandy clayey silt, trace organics and trace rootlets and is between 0.5 m and 2.7 m thick. The middle zone consists of brown to grey silty clay to clay, trace sand and is between 1.3 m and 12.5 m thick. The lower zone consists of clayey silt, some sand and is between 1.9 m and 3.0 m thick.

4.3.3.1 Clayey Silt

SPT 'N'-values measured within the upper clayey silt portion of the deposit range between 3 blows and 14 blows per 0.3 m of penetration. In situ field vane tests carried out within this clayey silt portion of the deposit measured undrained shear strengths greater than 100 kPa. The SPT 'N'-values and in situ field vane test results suggest that the deposit has a soft to very stiff consistency.

The natural water content measured on eleven samples of the upper clayey silt portion of the deposit ranges from about 13 per cent to 32 per cent.

The organic content measured on two samples of the upper clayey silt portion of the deposit are between about 1 per cent and 8 per cent. Where the organic content is 8 per cent, the material is classified as an organic clayey silt, although the high organic content is likely influenced by the peat deposit immediately after the sample.

Grain size distributions of five samples from the upper clayey silt portion of the deposit are shown on Figure A1 in Appendix A.

Atterberg limits tests were carried out on six samples of the upper clayey silt portion of the deposit and measured liquid limits ranging from about 23 per cent to 30 per cent, plastic limits ranging from about 16 per cent to 19 per cent and plasticity indices ranging from about 7 per cent to 11 per cent. The results of the



Atterberg limits testing are shown on the plasticity chart on Figure A2 in Appendix A and indicate that the material is classified as clayey silt of low plasticity.

4.3.3.2 Silty Clay to Clay

SPT 'N'-values measured within the silty clay to clay portion of the deposit range between 0 blows (i.e., weight of hammer) and 8 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 between 15 kPa and 96 kPa, with a calculated sensitivity between 1 and 10. The field vane tests results indicate that the silty clay to clay portion of the deposit has a soft to stiff consistency.

The natural water content measured on forty-three samples of the silty clay to clay portion of the deposit ranges from about 25 per cent to 74 per cent.

Grain size distributions of ten samples from the silty clay to clay portion of the deposit are shown on Figure A3.1 in Appendix A. One test result from a silt seam within the silty clay to clay deposit is shown on Figure A3.2 in Appendix A.

Atterberg limits tests were carried out on thirty-one (31) samples of the silty clay to clay portion of the deposit and measured liquid limits ranging from about 35 per cent to 67 per cent, plastic limits ranging from about 19 per cent to 29 per cent and plasticity indices ranging from about 16 per cent to 39 per cent. The result of the Atterberg limits tests are shown on the plasticity chart on Figure A4.1 and A4.2 in Appendix A and indicate that the material is classified as silty clay of intermediate plasticity to clay of high plasticity.

Laboratory consolidation tests were carried out on two specimens of the silty clay to clay deposit obtained from Shelby tube samples in Boreholes H1-6A and H1-9. Preconsolidation pressures ranging between 62 kPa and 130 kPa were estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. Bulk unit weights ranging of about 15.1 kN/m³ to 15.6 kN/m³ and a specific gravity between about 2.73 and 2.75 were measured on the consolidation test specimens. Details of the test results are shown on Figure A5 and A6 in Appendix A and the test results are summarized below.

Borehole Sample No.	Sample Depth/ Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	C_c	C_r	e_o	c_v^* (cm²/s)
Borehole H1-6A Sample 4	4.9 m/ 299.3 m	87	130	43	1.5	1.20	0.06	2.16	3.9×10^{-3}
Borehole H1-9 Sample 7	6.3 m/ 297.6 m	78	78	0	1.0	0.57	0.04	1.92	8.6×10^{-3}

*For stress range between approximately effective overburden stress and final stress due to 3 m high embankment, that is about $31 \text{ kPa} \leq \sigma_v' \leq 143 \text{ kPa}$

where: σ_{vo}' is the effective overburden stress in kPa
 σ_p' is the preconsolidation stress in kPa
OCR is overconsolidation ratio
 C_c is the compression index
 C_r is the recompression index
 e_o is initial void ratio
 c_v is the coefficient of consolidation in cm²/s



4.3.3.3 Clayey Silt

SPT 'N'-values measured within the lower clayey silt portion of the deposit range between 0 blows (weight of hammer) and 7 blows per 0.3 m of penetration. In situ field vane tests carried out within the lower clayey silt portion of the deposit measured undrained shear strengths between 23 kPa and 90 kPa, and sensitivities between 3 and 5. The results of the in situ field vane tests indicate that the deposit has a soft to stiff consistency.

The natural water content measured on four samples of the clayey silt portion of the deposit ranges from about 27 per cent to 34 per cent.

A grain size distribution on one sample from the lower clayey silt portion of the deposit is shown on Figure A1 in Appendix A.

Atterberg limits tests were carried out on two samples of the lower clayey silt portion of the deposit and measured liquid limits of about 26 per cent and 30 per cent, plastic limits of about 18 per cent and 19 per cent and plasticity indices of about 7 per cent and 11 per cent. The result of the Atterberg limits tests are shown on the plasticity chart on Figure A2 in Appendix A and indicate that the material is classified as clayey silt of low plasticity.

4.3.4 Silt to Sandy Silt

A deposit of grey to brown silt to sandy silt, trace to some clay, trace gravel was encountered underlying the silty clay to clay in Boreholes H1-4, H1-6, H1-9, H4-3 and H4-5. The surface of the silt to sandy silt deposit was encountered between Elevation 302.4 m and 292.2 m and the thickness of the deposit ranges from 0.3 m to 4.6 m.

SPT 'N'-values measured within the silt to sandy silt deposit range between 0 blows (i.e., weight of hammer) and 12 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The natural water content measured on eight samples of the silt to sandy silt deposit ranges from about 10 per cent to 38 per cent.

Grain size distributions of five samples from the silt to sandy silt deposit are shown on Figure A7 in Appendix A.

Atterberg limits testing was carried out on four samples of the silt to sandy silt deposit, of which two were determined to be non-plastic and two measured liquid limits of about 18 per cent and 23 per cent, plastic limits of about 16 per cent and 20 per cent and a corresponding plasticity index of about 2 per cent and 4 per cent, respectively. The result of the Atterberg limits testing is shown on the plasticity chart on Figure A8 in Appendix A and together with the non-plastic test results indicate that the material is classified as silt to silt of slight plasticity.

4.3.5 Silty Sand to Sandy Gravel

A deposit of grey to brown silty sand to sandy gravel, trace clay was encountered underlying the cohesive deposit in Boreholes H1-5, H1-7, H4-1, H4-2, H4-4, H4-6, C1-2 and BC1-1 to BC1-3 and underlying the silt to sandy silt deposit in Boreholes H1-4, H1-6, H1-9, H1-10, H4-3 and H4-5. The surface of the silty sand to sandy gravel deposit was encountered between Elevation 303.4 m and 287.6 m and the thickness of the deposit



ranges from 0.2 m to 4.5 m. A zone of cobbles was encountered in Boreholes BC-1 and BC-2 at Elevation 294.9 m and Elevation 299.7 m, with a thickness of 0.2 m and 0.6 m, respectively.

SPT 'N'-values measured within the silty sand to sandy gravel deposit generally range between 10 blows and 90 blows per 0.3 m of penetration with several higher 'N'-values for which the split spoon did not penetrate the full sampled depth, indicating a compact to very dense relative density.

The natural water content measured on thirteen samples of the silty sand to sandy gravel deposit ranges from about 2 per cent to 24 per cent.

Grain size distributions of ten samples from the silty sand to sandy gravel deposit are shown on Figure A9 in Appendix A. One test result is from a sand seam within the sandy gravel deposit.

4.3.6 Refusal/Bedrock

Refusal to further split spoon, auger and/or casing advancement or dynamic cone penetration was encountered in the boreholes at depths ranging from 3.1 m to 20.8 m below existing ground surface, ranging from Elevations 303.9 m to 286.5 m.

Bedrock was encountered at depths between about 6.9 m and 13.0 m below ground surface, corresponding to between Elevation 299.1 m and Elevation 292.1 m in Boreholes BC1-1 to BC1-3 and C1-2, and the bedrock was cored for lengths between 3.4 m and 4.1 m. The retrieved bedrock core is described as very fine grained, moderately weathered to fresh, green to grey, metasediment with occasional fractured sheared zones. The bedrock core retrieved from Borehole C1-2 is described as fine to medium grained, fresh, foliated, green to grey and white, schist. Photographs of the retrieved bedrock core samples are shown on Figure A10.

The Total Core Recovery (TCR) measured on all core samples ranges from 86 per cent to 100 per cent. The Solid Core Recovery (SCR) of the rock core samples ranges from 17 per cent to 100 per cent. The RQD measured on the core samples ranges from 17 per cent to 100 per cent, indicating a rock mass of very poor to excellent quality.

Laboratory Uniaxial Compression Strength (UCS) tests were carried out on selected bedrock core samples. The UCS values are presented on the Record of Drillhole sheets in Appendix A and are summarized below, and indicate that the bedrock is strong to very strong (Grade R4 to R5).

Borehole	Elevation (m)	UCS (MPa)
BC1-1	293.4	112
BC1-2	296.7	114
BC1-3	290.8	62

4.3.7 Groundwater Conditions

Groundwater levels were measured in the open boreholes during and upon completion of drilling at depths ranging between 1.0 m and 3.4 m depth below ground surface, or between Elevation 304.1 m and 300.8 m. A piezometer was installed in Borehole C1-2 sealed within the gravelly sand deposit and in Borehole BC1-3 sealed within the sand and gravel deposit, to monitor the groundwater levels. The piezometers in Boreholes C1-2 and



BC1-3 were installed in August 2011 and September 2012, respectively. The measured groundwater levels in the piezometers are presented below.

Borehole	Installation	Time and/or Date	Depth to Groundwater (m)	Groundwater Elevation (m)
C1-2	Piezometer ¹	May 19, 2013	0.0 (i.e., at ground surface)	305.4
BC1-3	Piezometer	November 17, 2012	-1.0 (i.e., above ground surface)	306.1 ²
	Piezometer	May 19, 2013	-1.1 (i.e., above ground surface)	306.2 ²

¹ Piezometer lock seized, unable to open and obtain water level during 2012 field investigation.

² Groundwater stabilized at this elevation inside the piezometer tubing as the tubing was extended to greater height above ground surface.

Groundwater elevations as encountered in the boreholes may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized on completion of drilling. Furthermore, groundwater elevations will vary depending on seasonal fluctuations, precipitation and local soil permeability.

4.4 Highway 66 – STA 13+300 to 13+345 (High Fill H5)

The plan and profiles along the centreline and toes of the proposed Highway 66 realignment embankment showing the borehole locations and interpreted stratigraphy between about STA 13+300 and 13+345 are shown on Drawing B1 in Appendix B. This section of the realignment extends across a valley between two rock outcrops, which is crossed by an ATV trail in a northwest-southeast orientation. The proposed embankment in this section of the realignment is up to about 7.5 m relative to existing ground surface.

A total of eight (8) boreholes (H5-1, H5-2 and BC1A-1 to BC1A-6) and two (2) DCPTs (H5-D1 and H5-D2) were completed to investigate the subsurface conditions within the High Fill H5 area. The topography in this section of the proposed highway realignment slopes upward to both the west and east sides of the valley and the terrain has moderate to dense tree cover.

The subsurface soils in High Fill H5 area generally consist of a surface layer of topsoil, underlain by a deposit of sandy silt to sand and gravel to gravel and a cohesive deposit consisting of clayey silt to clay. The cohesive deposit is generally underlain by deposits of silt, and gravelly sand to gravel, underlain by metasediment bedrock. The bedrock surface and refusal to further auger/casing advancement and dynamic cone penetration was encountered at depths between about 17 m and 9.2 m below ground surface, being deepest in the vicinity of about STA 13+310 along the Highway 66 Realignment.



4.4.1 Topsoil

A 30 mm to 300 mm thick layer of black topsoil was encountered from ground surface between Elevation 305.7 m and Elevation 303.0 m in all the boreholes.

4.4.2 Sandy Silt to Gravel

A 0.5 m to 0.7 m thick non-cohesive deposit of brown to grey sandy silt to silty sand to sand and gravel to gravel was encountered below the topsoil in Boreholes H5-1, H5-2, BC1A-1 and BC1A-3 to BC1A-5 between Elevation 305.7 m and Elevation 302.8 m.

The SPT 'N'-values measured within the sandy silt to gravel deposit range between 6 blows and 23 blows per 0.3 m of penetration, indicating a loose to compact relative density.

The natural water content measured on two samples of the sandy silt to gravel deposit are about 7 per cent and 11 per cent.

4.4.3 Clayey Silt to Clay

A deposit of grey to brown clayey silt to clay trace sand was encountered underlying the silty sand to gravel deposit in Boreholes H5-1, H5-2, BC1A-1 and BC1A-3 to BC1A-5, and below the topsoil in Boreholes BC1A-2 and BC1A-6. The surface of the clayey silt to clay deposit was encountered between Elevation 305.0 m and 302.2 m and the thickness of the deposit ranges from 1.0 m to 5.1 m. Generally, the deposit transitioned from an upper zone of silty clay to clay lower zone of clayey silt.

The SPT 'N'-values measured within the clayey silt to clay deposit range between 3 blows and 20 blows per 0.3 m of penetration. In situ field vane tests carried out within the clayey silt to clay deposit measured undrained shear strengths ranging from 34 kPa to greater than 100 kPa, and sensitivities between 3 and 7. The results of the field vane tests, together with the SPT 'N'-values indicate that the deposit generally has a firm to very stiff consistency.

The natural water content measured on fourteen samples of the clayey silt to clay deposit ranges from about 18 per cent to 50 per cent.

Grain size distributions of six samples of the clayey silt to clay deposit are shown on Figure B1 in Appendix B.

Atterberg limits tests were carried out on eleven samples of the cohesive deposit and measured liquid limits ranging from about 31 per cent to 60 per cent, a plastic limit ranging from about 18 per cent to 28 per cent and plasticity indices ranging from about 13 per cent to 34 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B2 in Appendix B and indicate that the material is classified as clayey silt of low plasticity to clay of high plasticity.

4.4.4 Silt

A deposit of grey silt, trace to some clay, trace sand was encountered underlying the cohesive deposit in Boreholes BC1A-1 and BC1A-3 to BC1A-6. The surface of the silt deposit was encountered between Elevation 302.0 m and 299.2 m and the thickness of the deposit ranges from 1.1 m to 3.9 m.



SPT 'N'-values measured within the silt deposit typically range between 3 blows and 11 blows per 0.3 m of penetration, indicating a very loose to compact relative density. One SPT 'N'-value measured within the silt deposit was 47 blows per 0.3 m, likely due to the spoon partially penetrating into the underlying gravelly sand to gravel deposit.

The natural water content measured on four samples of the silt deposit ranges from about 22 per cent to 32 per cent.

Grain size distributions of four samples of the silt deposit are shown on Figure B3 in Appendix B.

4.4.5 Silty Sand to Gravel

A deposit of brown to grey silty sand to gravelly sand to sand and gravel to gravel, trace silt, trace organics was encountered underlying the clay to clayey silt deposit in Borehole H5-2 and underlying the silt deposit in Boreholes BC1A-1, BC1A-2 and BC1A-4 to BC1A-6. The silty sand to gravel deposit was encountered between Elevation 301.4 m and 297.4 m and the thickness of the deposit ranges from 0.6 m to 4.9 m.

The SPT 'N'-values measured within the gravelly sand to gravel deposit range between 13 blows and 80 blows per 0.3 m of penetration, and up to 17 blows per 0.1 m of penetration, indicating a compact to very dense relative density. Boulders 0.5 m thick were encountered in Borehole BC1A-4 at Elevation 297.8 m and 296.4 m, respectively. A 0.3 m thick boulder was encountered in Borehole BC1A-6 at Elevation 294.3 m. Cobbles were encountered in Borehole BC1A-5 at Elevation 297.4 m.

The natural water contents measured on four samples of the gravelly sand to gravel deposit range from about 8 per cent to 12 per cent.

Grain size distributions of three samples of the silty sand to gravelly sand portion of the deposit are shown on Figure B4 in Appendix B.

4.4.6 Refusal/Bedrock

Refusal to further split spoon, auger advancement and dynamic core penetration was encountered at depths of 1.7 m and 7.0 m, below existing ground surface, between Elevation 297.2 m and 303.1 m in Boreholes H5-1 and H5-2 and DCPT H5-D1 and H1-D2. Bedrock was encountered at depths ranging from 5.9 m to 10.9 m below existing ground surface, between Elevation 293.2 and 299.8 m in Boreholes BC1A-1 to BC1A-6 and the bedrock was cored for lengths ranging from 3.1 m to 3.8 m. The retrieved bedrock core is described as fine grained, completely weathered to fresh, grey, metasediment, with highly fractured zones. Photographs of the retrieved bedrock cores are shown on Figure B5 in Appendix B.

The TCR ranges between 85 per cent and 100 per cent and the SCR ranges from 0 per cent to 100 per cent. The RQD measured on the core samples ranges from 21 per cent to 100 per cent, but is generally less than 73 per cent, indicating a rock mass of very poor to fair quality with sections of excellent quality.

Laboratory UCS tests were carried out on selected bedrock core samples. The UCS values are presented on the Record of Drillhole sheets in Appendix B and are summarized below, and indicate that the bedrock is medium strong to strong (Grade R3 to R4).



Borehole	Elevation (m)	UCS (MPa)
BC1A-3	297.2	63
BC1A-4	292.0	25
BC1A-5	292.2	44
BC1A-6	293.6	77

4.4.7 Groundwater Conditions

Groundwater levels were measured in the open boreholes during and upon completion of drilling at between 1.1 m and 4.5 m below ground surface, ranging between Elevation 303.3 m and 299.6 m. Boreholes H5-1 and H5-2 were dry. A piezometer was installed in Borehole BC1A-1, sealed within the silt and gravel deposits, to monitor the groundwater levels. The measured groundwater level in the piezometer is presented below.

Borehole	Installation	Time and/or Date	Groundwater Depth (m)	Groundwater Elevation (m)
BC1A-1	Piezometer	November 17, 2012	0.6	303.7
	Piezometer	May 17, 2013	0.8	303.5

Groundwater elevations as encountered in the boreholes may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized on completion of drilling. Furthermore, groundwater elevations will vary depending on seasonal fluctuations, precipitation and local soil permeability.

4.5 Highway 66 – STA 14+020 to 14+650 (Swamp Crossing H6/H7)

The plan and profiles along the centreline and toes of the proposed Highway 66 realignment embankment showing the borehole and CPT locations and interpreted stratigraphy between about STA 14+020 and 14+650 are shown on Drawings C1 to C4 in Appendix C. The realignment of the proposed Highway 66 embankment limits extends across a low-lying swamp, with ponded water in places due to blockage of a creek by a beaver dam. The proposed embankment along this section of the highway realignment is up to about 3.5 m high relative to the existing ground surface.

A total of forty-nine (49) boreholes (H6-1 to H6-19, H6-7A, H6-S1 to H6-S5, H7-1 to H7-15, BC2-1 to BC2-3, BC3-1 to BC3-3 and BC4-1 to BC4-3), seven (7) CPTs (H6-CPT1 to H6-CPT7) and nineteen (19) DCPTs (H6-D1 to H6-D11 and H7-D1 to H7-D8) were completed to investigate the subsurface conditions within this high fill area. The topography in this section of the proposed highway realignment slopes slightly downward from west to east, and the terrain contains swampy areas, a beaver pond, moderate to dense tree cover and multiple ATV trails.

The subsurface soils along the Highway 66 realignment in High Fill H6 and High Fill H7 areas generally consist of surficial layers of fill, peat or topsoil, underlain by an extensive cohesive deposit consisting of an upper zone of clayey silt to silty clay, a middle zone of silty clay to clay and a lower zone of clayey silt to silty clay. The cohesive deposit is generally underlain by a deposit of silt to sandy silt which in turn is underlain by a deposit of



silty sand to sand and gravel to gravel underlain by inferred bedrock. Refusal to further auger and/or casing advancement and dynamic cone penetration was encountered at depths of up to about 29.3 m, being deepest in the vicinity of about STA 14+125.

4.5.1 Peat / Topsoil

A deposit of black fibrous peat ranging in thickness from 0.2 m to 4.0 m was encountered from ground surface or below ponded water, between Elevation 304.1 m and Elevation 309.7 m in Boreholes H6-1 to H6-19, H7-5, H7-6, BC2-1 to BC2-3, BC3-1 to BC3-3 and BC4-1 to BC4-3. In Boreholes H7-1, H7-3, H7-4 and H7-8 to H7-15, black topsoil was encountered from ground surface between Elevation 307.8 m and 305.1 m with thicknesses ranging from less than 0.1 m to 0.3 m.

SPT 'N'-values measured within the peat deposit range between 0 blows (i.e., weight of hammer) and 4 blows per 0.3 m of penetration, suggesting a very soft to soft consistency. One SPT 'N'-value of 4 blows per 0.15 m of penetration was measured within the topsoil deposit in Borehole H7-11 on refusal to further advancement.

The natural water content measured on twenty-three samples of the peat ranges from about 55 per cent to 991 per cent. The natural water content measured on one sample of the topsoil is about 61 per cent.

4.5.2 Sand and Silt to Silty Sand

A deposit of brown to grey sand and silt to silty sand, trace to some clay, trace organics was encountered underlying the peat deposit in Boreholes H6-1, H6-4 and BC2-1 to BC2-3. The sand and silt to sand and silt deposit was encountered between Elevation 309.0 m and 306.0 m with thickness ranging from 0.4 m to 2.3 m.

SPT 'N'-values measured within the sand and silt to silty sand deposit range between 0 blows (i.e., weight of hammer) and 26 blows per 0.3 m of penetration, indicating a very loose to compact relative density. An SPT 'N'-value of 15 blows per 0.15 m of penetration was recorded on inferred cobbles within this deposit in Borehole BC2-3.

The natural water content measured on five samples of the sand and silt to silty sand deposit ranges from about 13 per cent to 28 per cent.

A grain size distribution of one sample of the sand and silt portion of the deposit is shown on Figure C1 in Appendix C.

4.5.3 Clayey Silt to Clay

A cohesive deposit, generally consisting of an upper zone of clayey silt to silty clay, a middle zone consisting of silty clay to clay and a lower zone transitioning to clayey silt, was encountered underlying the fill, topsoil or peat in the boreholes. The total thickness of the cohesive deposit is between about 0.7 m and 17.2 m and the surface of the deposit was encountered between Elevation 307.8 m and 303.7 m.

The upper zone of the cohesive deposit consists of brown to grey clayey silt to silty clay, trace to some sand, trace organics and is between 0.4 m and 5.6 m thick. Within this upper zone of the cohesive deposit silt seams



were frequently encountered and the deposit is considered to be varved. Sand seams and pockets of organic material were occasionally encountered within this upper zone.

The middle zone of the cohesive deposit consists of brown to grey silty clay to clay, trace sand and is between 0.9 m and 11.8 m thick. Pockets of organic material were also encountered in the middle zone of this deposit on occasion in split spoon samples and in the extruded Shelby tube samples. The pockets of organic material encountered were up to 250 mm thick, and were encountered to depths up to about 5 m below ground surface, corresponding to about Elevation 300 m. The pockets of organic material generally consist of black, fibrous peat mixed with clay. Although this zone of the cohesive deposit is not considered varved, occasional irregular varves were encountered, as noted in the extruded Shelby tube samples, at irregular orientations, varying from horizontal to near vertical, and thicknesses between 1 mm and 25 mm.

The lower zone of the cohesive deposit consists of grey clayey silt to silty clay, some sand and is between 0.3 m and 7.6 m thick. Within the lower zone of the deposit silt seams were frequently encountered and this portion of the deposit is considered varved.

4.5.3.1 Clayey Silt to Silty Clay

SPT 'N'-values measured within the upper clayey silt to silty clay portion of the deposit range between 0 blows (i.e., weight of hammer) and 15 blows per 0.3 m of penetration. In situ field vane tests carried out within the clayey silt to silty clay portion of the deposit measured undrained shear strengths ranging from about 17 kPa to 85 kPa. The field vane test results together with the CPT testing indicate that the deposit has a soft to stiff consistency. The sensitivity is calculated to range between about 2 and 9.

The natural water content measured on fifty-two samples of the clayey silt to silty clay portion of the deposit ranges from about 20 per cent to 51 per cent.

The organic content measured on three samples of the clayey silt to silty clay portion of the deposit is between about 1 per cent and 8 per cent.

Grain size distributions of seventeen samples of the clayey silt to silty clay deposit are shown on Figure C2 in Appendix C.

Atterberg limits tests were carried out on forty (40) samples of the clayey silt to silty clay portion of the deposit and measured liquid limits ranging from about 24 per cent to 49 per cent, plastic limits ranging from about 14 per cent to 22 per cent and plasticity indices ranging from about 8 per cent to 27 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure C3 in Appendix C and indicate that the material is classified as clayey silt of low plasticity to silty clay of intermediate plasticity.

Laboratory consolidation tests were carried out on two specimens of the clayey silt to silty clay deposit obtained from Shelby tube samples in Boreholes H6-7A and H6-S1. A preconsolidation pressure of 62 kPa and 67 kPa were estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. A bulk unit weight of about 16.9 kN/m³ and 18.9 kN/m³ and a specific gravity of about 2.74 and 2.76 were measured on the consolidation test specimens. Details of the test results are shown on Figure C4 and C5 in Appendix C, and the test results are summarized below.



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Borehole Sample No.	Sample Depth/ Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	C_c	C_r	e_o	c_v^* (cm ² /s)
Borehole H6-7A Sample 1	6.4 m / 302.7 m	27	62	35	2.3	0.46	0.03	1.50	1.8×10^{-3}
Borehole H6-S1 Sample 1	4.8 m / 304.4 m	15	67	52	1.8	0.22	0.01	0.89	3.8×10^{-3}

*For stress range between approximately effective overburden stress and final stress due to 3 m high embankment, that is
 $31 \text{ kPa} \leq \sigma_v' \leq 156 \text{ kPa}$

where: σ_{vo}' is the effective overburden stress in kPa
 σ_p' is the preconsolidation stress in kPa
OCR is the overconsolidation ratio
 C_c is the compression index
 C_r is the recompression index
 e_o is the initial void ratio
 c_v is the coefficient of consolidation in cm²/s

A total of seven cone penetration tests (H6-CPT1 to H6-CPT7) were performed in this swamp crossing for determination of the pore pressure dissipation at specific horizons within the clayey silt to silty clay deposit. The range of the coefficient of consolidation in the horizontal direction (c_h) obtained from a total of four CPT pore pressure dissipation tests is summarized below.

C_h CPT-Field (cm²/s)		
Upperbound	Lowerbound	Average
9.9×10^{-2}	1.7×10^{-2}	4.2×10^{-2}

Laboratory consolidated isotropic undrained triaxial CIU triaxial tests with pore pressure measurement were carried out on three samples of the cohesive deposit obtained from Shelby tube samples in Boreholes H6-S2, H6-S4 and H6-S5. In total, one specimen and two sets of two specimens were tested in the clayey silt to silty clay deposit. The details of the test results are shown on Figures C6/C16 and C7 in Appendix C and the results are summarized below.

Borehole Sample No.	Sample Depth/Elevation	Effective Cohesion, c' (kPa)	Effective Angle of Internal Friction, ϕ' (degrees)
Borehole H6-S2 Sample 1	4.6 m / 304.1 m	0	35
Borehole H6-S4 Sample 1	3.0 m / 303.7 m		
Borehole H6-S5 Sample 1	1.8 m / 303.4 m		

Note: Assessed shear strength parameters are only valid over range of stress conditions used in the laboratory test.

The triaxial test samples were consolidated to pressures representative of the estimated in situ effective stresses and maximum effective stresses under the proposed embankment loads, at the respective sample depths. The interpreted effective strength parameters provided above are applicable only to design situations for which the



stress conditions during testing are representative. Reference should be made to individual test results for details of the testing conditions.

4.5.3.2 Silty Clay to Clay

SPT 'N'-values measured within the silty clay to clay portion of the deposit typically range between 0 blows (i.e., weight of rods or hammer) and 7 blows per 0.3 m of penetration, and generally 2 blows or less 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 between about 12 kPa and 68 kPa, and generally less than 35 kPa. The field vane test results together with the CPT testing indicate that the deposit has a very soft to stiff consistency, and generally a very soft to firm consistency. The sensitivity is calculated to range between 1 and 10.

The natural water content measured on eighty samples of the silty clay to clay deposit range from about 30 per cent to 78 per cent.

The organic content measured on one sample of the silty clay to clay portion of the deposit is about 8 per cent. The sample contained a root from surface vegetation, resulting in a high organic content, which is not typical of this zone.

Grain size distributions of seventeen samples from the silty clay to clay portion of the deposit are shown on Figure C8 in Appendix C.

Atterberg limits tests were carried out on seventy-seven (77) samples of the silty clay to clay portion of the deposit and measured liquid limits ranging from about 37 per cent to 69 per cent, plastic limits ranging from about 19 per cent to 27 per cent and plasticity indices ranging from about 17 per cent to 45 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure C9 in Appendix C and indicate that the material is classified as silty clay of intermediate plasticity to clay of high plasticity.

Out of the samples tested, two samples were separated into the clayey silt to silty clay laminae and the silty clay to clay laminae. The test results on the silty clay to clay laminae indicate liquid limits of about 48 per cent and 73 per cent, plastic limits of about 21 per cent and 27 per cent and plasticity indices of about 27 per cent and 46 per cent. For the clayey silt to silty clay laminae, the liquid limits are about 32 per cent and 40 per cent, the plastic limits are about 20 per cent and 22 per cent and the plasticity indices are about 13 per cent and 18 per cent. The test results confirm that the 'silty' varves are classified as clayey silt of low plasticity to silty clay of intermediate plasticity and the 'clayey' varves are classified as silty clay of intermediate plasticity to clay of high plasticity.

Laboratory consolidation tests were carried out on five specimens of the silty clay to clay deposit obtained from Shelby tube samples in Boreholes BC4-1 and H6-S1. Preconsolidation pressures ranging between 60 kPa to 134 kPa were estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. Bulk unit weights ranging from about 16.1 kN/m³ to 18.8 kN/m³ and a specific gravity between about 2.73 and 2.76 were measured on the consolidation test specimens. Details of the test results are shown on Figure C10 to C14 in Appendix C, and the test results are summarized below.



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Borehole Sample No.	Sample Depth/ Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	C_c	C_r	e_o	c_v^* (cm ² /s)
Borehole BC4-1 Sample 4	3.4 m/ 301.8 m	18	125	107	6.9	1.04	0.04	1.87	1.4×10^{-2}
Borehole BC4-1 Sample 5	4.9 m/ 300.3 m	30	92	62	3.1	0.68	0.05	1.74	1.3×10^{-2}
Borehole BC4-1 Sample 6	6.4 m/ 298.8 m	40	134	94	3.4	0.90	0.05	1.62	1.6×10^{-2}
Borehole H6-S1 Sample 2	7.8 m/ 301.4 m	35	60	25	1.7	1.20	0.02	1.80	4.0×10^{-3}
Borehole H6-S1 Sample 4	14.2 m/ 295.0 m	76	80	4	1.1	0.82	0.02	1.61	5.7×10^{-3}

*For stress range between approximately effective overburden stress and final stress due to 3 m high embankment, that is $30 \text{ kPa} \leq \sigma_v' \leq 156 \text{ kPa}$

where: σ_{vo}' is the in situ vertical effective overburden stress in kPa
 σ_p' is the preconsolidation stress in kPa
OCR is the overconsolidation ratio
 C_c is the compression index
 C_r is the recompression index
 e_o is the initial void ratio
 c_v is the coefficient of consolidation in cm²/s

In addition to the tests on the horizontally trimmed specimens noted above, consolidation tests were also carried out on a vertically trimmed orientation (VTO) specimen of the silty clay to clay from Borehole H6-S1 to assess the horizontal coefficient of consolidation, c_h . Details of the test results are shown on Figure C15 in Appendix C. The results of the test indicate a c_h value of $7.0 \times 10^{-3} \text{ cm}^2/\text{s}$ for the stress range of about 35 kPa to 145 kPa.

A total of seven cone penetration tests (H6-CPT1 to H6-CPT7) were performed in this swamp crossing for determination of the pore pressure dissipation at specific horizons within the silty clay to clay deposit. The range of the coefficient of consolidation in the horizontal direction (c_h) obtained from a total of twenty-four CPT pore pressure dissipation tests is summarized below.

C_h CPT-Field (cm²/s)		
Upperbound	Lowerbound	Average
1.3×10^{-1}	2.9×10^{-3}	1.9×10^{-2}

Laboratory CIU triaxial tests with pore pressure measurement were carried out on three samples of the cohesive deposit obtained from Shelby tube samples in Borehole H6-S2. In total, two specimens and one set of two specimens were tested in the clayey silt deposit. The details of the test results are shown on Figure C16 in Appendix C and the results are summarized below.



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Borehole Sample No.	Sample Depth/Elevation	Effective Cohesion, c' (kPa)	Effective Angle of Internal Friction, ϕ' (degrees)
Borehole H6-S2 Sample 2	7.9 m / 300.8 m	3	24
Borehole H6-S2 Sample 3	10.8 m / 297.9 m		
Borehole H6-S2 Sample 4	14.0 m / 294.7 m		

Note: Assessed shear strength parameters are only valid over range of stress conditions used in the laboratory test.

A laboratory consolidated drained direct shear test measurement was carried out on one sample of the silty clay to clay deposit obtained from Shelby tube samples in Borehole H6-S2. In total, one specimen and one set of three specimens were tested in the silty clay to clay deposit. The details of the test results are shown on Figure C17 in Appendix C and the results are summarized below.

Borehole Sample No.	Sample Depth/Elevation	Effective Cohesion, c' (kPa)	Effective Angle of Internal Friction, ϕ' (degrees)
Borehole H6-S5 Sample 3	7.6 m / 297.6 m	0	30

The triaxial test and direct shear test samples were consolidated to pressures representative of the estimated in situ effective stresses and maximum effective stresses under the proposed embankment loads, at the respective sample depths. The interpreted effective strength parameters provided above are applicable only to design situations for which the stress conditions during testing are representative. Reference should be made to individual test results for details of the testing conditions.

4.5.3.3 Clayey Silt

SPT 'N'-values measured within the clayey silt portion of the deposit range between 0 blows (i.e., weight of rods or hammer) and 8 blows per 0.3 m of penetration. In situ field vane tests carried out within the clayey silt deposit measured undrained shear strengths ranging from 19 kPa and 69 kPa. The field vane test results together with the CPT testing indicate that the deposit has a soft to stiff consistency. The sensitivity is calculated to range between 2 and 8.

The natural water content measured on twenty samples of the clayey silt portion of the deposit ranges from about 25 per cent to 64 per cent.

Grain size distributions of six samples from the clayey silt portion of the deposit are shown on Figure C18 in Appendix C.

Atterberg limits tests were carried out on seventeen (17) samples of the clayey silt portion of the deposit and measured liquid limits ranging between about 24 per cent and 35 per cent, plastic limits ranging between about



17 per cent and 20 per cent and plasticity indices ranging between about 6 per cent and 17 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure C19 in Appendix C and indicate that the material is classified as clayey silt of low plasticity.

A Laboratory consolidation test was carried out on one specimen of the clayey silt deposit obtained from a Shelby tube sample in Borehole H6-S1. A preconsolidation pressure of 96 kPa were estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. A bulk unit weight of about 18.8 kN/m³ and a specific gravity of about 2.74 were measured on the consolidation test specimen. Details of the test results are shown on Figure C20 in Appendix C, and the test results are summarized below.

Borehole Sample No.	Sample Depth/Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	C_c	C_r	e_o	c_v^* (cm ² /s)
Borehole H6-S1 Sample 5	17.1 m/ 292.1 m	96	130	34	1.4	0.31	0.01	0.97	5.4×10^{-2}

*For stress range between approximately effective overburden stress and final stress due to 3 m high embankment, that is $30 \text{ kPa} \leq \sigma_v' \leq 156 \text{ kPa}$

where: σ_{vo}' is the in situ vertical effective overburden stress in kPa
 σ_p' is the preconsolidation stress in kPa
 OCR is the overconsolidation ratio
 C_c is the compression index
 C_r is the recompression index
 e_o is the initial void ratio
 c_v is the coefficient of consolidation in cm²/s

A total of seven cone penetration tests (H6-CPT1 to H6-CPT7) were performed in this swamp crossing for determination of the pore pressure dissipation at specific horizons within the clayey silt deposit. The range of the coefficient of consolidation in the horizontal direction (c_h) obtained from a total of five CPT pore pressure dissipation tests is summarized below.

C_h CPT-Field (cm ² /s)		
Upperbound	Lowerbound	Average
6.4×10^{-2}	9.9×10^{-3}	4.0×10^{-2}

4.5.4 Silt to Sand and Silt

A deposit of grey to brown silt to sandy silt to sand and silt, trace to some clay, trace to some gravel, and in one borehole, gravelly silt was encountered underlying the cohesive deposit in Boreholes H6-1, H6-4 to H6-19, H7-1 to H7-10, H7-12, H7-15, BC2-1, BC3-1 to BC3-3 and BC4-1 to BC4-3. The sand and silt to silt deposit was encountered between Elevation 307.0 m and 287.9 m and the thickness of the deposit ranges from 0.2 m to 8.7 m.

SPT 'N'-values measured within the silt to sand and silt deposit range between 0 blows (i.e., weight of hammer) and 36 blows per 0.3 m of penetration, indicating a very loose to dense relative density. Five SPTs did not penetrate the full sampler depth indicative of the underlying gravel deposit or inferred refusal conditions.



The natural water content measured on thirty-four samples of the silt to sand and silt deposit ranges from about 11 per cent to 37 per cent.

Grain size distributions of twenty-three samples from the silt to sand and silt deposit are shown on Figure C21 in Appendix C.

Atterberg limits tests were carried out on ten samples of the silt to sand and silt deposit and all indicate that the material is non-plastic.

4.5.5 Silty Sand to Gravel

A deposit of grey to brown silty sand to sand to gravelly sand to sand and gravel to gravel, trace to some clay was encountered underlying the cohesive deposit in Boreholes H6-2, H6-3, H7-13 and BC2-2 and underlying the silt to sand and silt deposit in Boreholes H6-1, H6-4 to H6-8, H6-12, H6-13, H6-15 to H6-17, H6-19, H7-1 to H7-8, H7-10, BC3-2, BC3-3 and BC4-1. The silty sand to gravel deposit was encountered between Elevation 306.5 m and 281.0 m and the thickness of the deposit ranges from 0.6 m to 8.0 m.

SPT 'N'-values measured within the silty sand to gravel deposit range between 4 blows and 47 blows per 0.3 m of penetration, indicating a loose to dense relative density. Several SPTs did not penetrate the full sample depth on refusal conditions.

The natural water content measured on twenty-two samples of the silty sand to gravel deposit ranges from about 7 per cent to 26 per cent.

Grain size distributions for ten samples from the silty sand to gravel are shown on Figure C22 in Appendix C.

4.5.6 Refusal/Bedrock

Refusal to further split spoon, auger and/or casing advancement and dynamic cone penetration was encountered in all boreholes and DCPTs except Boreholes H6-14 and H6-17 at depths ranging from 0.3 m to 29.3 m below existing ground surface, ranging from Elevation 306.5 m to 279.7 m.

Bedrock was encountered in Boreholes BC2-1 to BC2-3 at depths ranging from 2.1 m to 7.4 m below existing ground surface, respectively, corresponding to Elevations 307.2 m and 302.3 m, and was cored for lengths ranging from 2.9 m to 3.4 m.

The retrieved bedrock core is described as fine grained, moderately weathered to fresh, moderately to highly foliated, green to grey, metasediment. Photographs of the retrieved bedrock cores are shown on Figure C23 in Appendix C.

The TCR ranges between 91 per cent and 100 per cent and the SCR ranges between 47 per cent and 89 per cent. The RQD measured on the core samples ranges from 51 per cent to 100 per cent, indicating a rock mass of fair to excellent quality.

Laboratory UCS tests were carried out on two core samples of the bedrock and the UCS values are presented on the Record of Drillholes sheets in Appendix C and are summarized below. The metasediment bedrock in Boreholes BC2-1 and BC2-3 is classified as strong (Grade RA).



Borehole	Elevation (m)	UCS (MPa)
BC2-1	305.6	72
BC2-3	304.6	58

4.5.7 Groundwater Conditions

Groundwater levels were measured in the open boreholes during and upon completion of drilling and range between ground surface and 5.2 m below ground surface, ranging between Elevations 309.7 m and 302.2 m. Up to 0.7 m of ponded water was encountered above the ground surface in some boreholes. Boreholes H7-11 to H7-15 were dry upon completion of drilling. A piezometer was installed in Borehole BC2-2 sealed within the gravelly sand deposit, to monitor the groundwater level. The measured groundwater level in the piezometer is presented below.

Borehole	Installation	Time and/or Date	Depth to Groundwater Below Ground Surface (m)	Groundwater Elevation (m)
BC2-2	Piezometer	November 15, 2012	-0.3 ¹	310.0
	Piezometer	May 15, 2013	-0.4 ¹	310.1

¹ Groundwater level measured above ground surface at the same level (elevation) as the surface water ponded around the piezometer pipe.

Artesian groundwater conditions were encountered in Borehole H7-5 upon penetrating into the sand deposit and the groundwater level was measured 1.5 m above ground surface, corresponding to Elevation 306.5 m, upon completion of drilling. The borehole was backfilled by tremie grouting from the bottom of the borehole using cement grout.

Groundwater elevations as encountered in the boreholes may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized on completion of drilling. Furthermore, groundwater elevations will vary depending on seasonal fluctuations, precipitation and local soil permeability.

4.6 Highway 66 – STA 14+840 to 15+060 (High Fill H3)

The plan and profiles along the centreline and toes of the proposed Highway 66 realignment embankment showing the borehole locations and interpreted stratigraphy between about STA 14+840 and 14+060 are shown on Drawings D1 and D2 in Appendix D. The realignment extends across undulating terrain with rock outcrops present along the proposed Highway 66 embankment limits at some locations. The proposed embankment in this section of the highway realignment is up to about 6.5 m high relative to the existing ground surface.

A total of twenty-one (23) boreholes (H3-1 to H3-15, BC5-1 to BC5-3 and BC6-1 to BC6-5) and seven (7) DCPTs (H3-D1 to H3-D7) were completed to investigate the subsurface conditions within this high fill area. The



topography in this section of the proposed highway realignment is undulating but generally sloping slightly upward from west to east. This section of the proposed realignment has moderate to dense tree cover and multiple ATV trails.

The subsurface soils along the Highway 66 realignment in the High Fill H3 area generally consist of surficial layers of topsoil, underlain by a cohesive deposit consisting of clayey silt. The cohesive deposit is underlain by a deposit of silt, which in turn is underlain by deposits of sand and silt to gravelly silty sand and/or sand and gravel, underlain by inferred bedrock. Refusal to further split spoon and/or auger advancement and dynamic cone penetration was encountered at depths up to about 9.6 m below ground surface, being deepest in the vicinity of about STA 14+860 along the Highway 66 realignment.

4.6.1 Topsoil

A layer of black topsoil was encountered from ground surface between Elevation 311.5 m and 305.3 m in all boreholes except Borehole BC6-4. Typically, the topsoil layer is between about 30 mm and 150 mm thick and 0.8 m thick in Borehole BC6-2.

One SPT 'N'-value measured within the topsoil layer is 6 blows per 0.3 m of penetration, suggesting a firm consistency.

The natural water content measured on one sample of the topsoil is about 45 per cent.

4.6.2 Clayey Silt

A deposit of brown to grey clayey silt, trace sand, trace rootlets/organics in the upper portion of the deposit was encountered underlying the topsoil deposit in Boreholes H3-2, H3-4, H3-6 to H3-11, BC5-2, BC5-3, BC6-1 and BC6-3. The deposit in Borehole BC5-2 is described as gravelly silty clay. The surface of the clayey silt deposit was encountered between Elevation 309.7 m and 306.0 m and the thickness of the deposit ranges from 0.3 m to 1.8 m.

SPT 'N'-values measured within the clayey silt deposit range between 3 blows and 27 blows per 0.3 m of penetration, suggesting a soft to very stiff consistency. Three SPTs near the base of the deposit did not penetrate the full sampler depth indicative of the lower gravelly deposit or refusal conditions.

The natural water content measured on eleven samples of the clayey silt deposit ranges from about 17 per cent to 37 per cent.

Grain size distributions of seven samples of the clayey silt deposit are shown on Figure D1 in Appendix D.

Atterberg limits tests were carried out on ten samples of the clayey silt deposit and measured liquid limits ranging from about 21 per cent to 35 per cent, plastic limits ranging from about 15 per cent to 25 per cent and plasticity indices ranging from about 5 per cent to 13 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure D2 in Appendix D and indicate that the material is generally classified as clayey silt of low plasticity. The Atterberg limits test results for the sample from Borehole H3-7 and BC5-2 measured liquid limits of about 32 per cent and 47 per cent, plastic limits of about 7 per cent and 22 per cent and corresponding plasticity indices of about 25 per cent, classifying the material as organic silt of low plasticity silty



clay of intermediate plasticity, respectively. It is noted that the sample from Borehole H3-7 was mixed with topsoil as a result of the split spoon sampling operation.

4.6.3 Silt

A deposit of brown to grey silt, trace to some clay, trace to some sand, trace to some gravel was encountered from surface, underlying the topsoil or clayey silt in Boreholes H3-1, H3-2, H3-4 to H3-6, H3-8, H3-10, H3-12 to H3-15, BC5-3 and BC6-2 to BC6-5. The surface of the silt deposit was encountered between Elevation 311.5 m and 304.5 m and the thickness of the deposit ranges from 0.5 m to 4.0 m.

SPT 'N'-values measured within the silt deposit range between 1 blow and 31 blows per 0.3 m of penetration, indicating a very loose to dense relative density.

The natural water content measured on eleven samples of the silt deposit ranges from about 17 per cent to 33 per cent.

Grain size distributions of eleven samples of the silt deposit are shown on Figure D3 in Appendix D.

Atterberg limits testing was carried out on ten samples of the silt deposit, all of which indicate that the material is non-plastic.

4.6.4 Sand and Silt to Gravelly Silty Sand

A deposit of brown to grey sand and silt to gravelly silty sand, trace to some clay was encountered underlying the silt deposit or topsoil in Boreholes H3-1 to H3-5, H3-8, H3-10, BC5-1, BC6-2 and BC6-3. The surface of the deposit was encountered between Elevation 308.5 m and 302.6 m and the thickness of the deposit ranges from 0.4 m to 2.5 m.

SPT 'N'-values measured within the sand and silt to gravelly silty sand deposit range between 5 blows and 65 blows per 0.3 m of penetration, indicating a loose to very dense relative density. Several SPTs did not penetrate the full sampler depth as a result of encountering the underlying gravelly deposit or refusal conditions.

The natural water content measured on eleven samples of the sand and silt to gravelly silty sand deposit ranges from about 7 per cent to 28 per cent.

Grain size distributions of eight samples of the sand and silt to gravelly silty sand are shown on Figure D4 in Appendix D.

4.6.5 Sand and Gravel

A deposit of brown to grey sand and gravel, trace to some silt, trace clay was encountered underlying the clayey silt in BC6-1, underlying the silt deposit in Boreholes H3-12, H3-15, BC5-3, BC6-4 and BC6-5 and underlying the sand and silt to gravelly silty sand deposit in Boreholes H3-10, and BC5-1. Cobbles were inferred present in Borehole BC6-1. The deposit was described as silty sand and gravel in Borehole BC6-5. The sand and gravel deposit was encountered between Elevation 310.2 m and 300.1 m with thickness ranging from 0.2 m to 4.9 m.



SPT 'N'-values measured within the sand and gravel deposit range between 19 blows and 75 blows per 0.3 m of penetration, indicating a compact to very dense relative density. Several SPTs did not penetrate the full sampler depth indicative of refusal conditions or the presence of cobbles

The natural water content measured on eight samples of the sand and gravel deposit ranges from about 1 per cent to 12 per cent.

Grain size distributions of four samples of the sand and gravel deposit are shown on Figure D5 in Appendix D.

4.6.6 Refusal/Bedrock

Refusal to further split spoon, and/or auger advancement and dynamic cone penetration was encountered in 11 boreholes and DCPTs at depths ranging from about 0.5 m to 8.2 m below existing ground surface, ranging from Elevation 310.4 m and 299.1 m. Bedrock was encountered at depths ranging from 0.5 m to 9.6 m below existing ground surface, ranging between Elevation 306.1 m and 299.4 m in Boreholes BC5-1 to BC5-3 and BC6-1 to BC6-5.

Bedrock core samples were obtained for lengths ranging from 3.0 m to 3.6 m. The retrieved bedrock core is described as very fine grained to very coarse grained, completely weathered to fresh, grey to greenish grey, metasediment or metasediment siltstone. Photographs of the retrieved bedrock core samples are shown on Figure D6 in Appendix D.

The TCR from Boreholes BC5-1 to BC5-3 and BC6-1 to BC6-5 ranges between 89 per cent and 100 per cent and the Solid Core Recovery ranges from 0 per cent to 100 per cent. The RQD measured on the core samples ranges from 0 per cent to 100 per cent, indicating a rock mass of very poor to excellent quality. Typically, the RQD is greater than 68 per cent indicating that the rock is of fair to excellent quality. The top run in Borehole BC6-2 has an RQD of 0 per cent due to numerous irregular joints.

Laboratory UCS tests were carried out on five selected bedrock core samples. The UCS values are presented on the Record of Drillholes sheets in Appendix D and are summarized below. The metasediment/metasediment siltstone bedrock in Boreholes BC5-1 to BC5-3 and BC6-1 to BC6-3 is considered strong to very strong (Grade R4 to R5).

Borehole	Elevation (m)	UCS (MPa)
BC5-1	301.9	143
BC5-3	297.3	166
BC6-1	299.1	64
BC6-2	299.8	174
BC6-3	302.2	177
BC6-4	302.3	116
BC6-5	302.5	138



4.6.7 Groundwater Conditions

Groundwater levels were measured in the open boreholes during and upon completion of drilling at depths ranging between 1.8 m and 4.9 m below ground surface, ranging between Elevations 306.5 m and 302.2 m. Boreholes H3-1, H3-3, H3-5 to H3-9 and H3-11 to H3-15 were dry upon completion of drilling. Piezometers were installed in Boreholes BC5-2 and BC6-2 sealed within the bedrock and within the silt/silty sand deposit, respectively. The measured groundwater levels in the piezometers are presented below.

Borehole	Installation	Date	Depth to Groundwater Below Ground Surface (m)	Groundwater Elevation (m)
BC5-2	Piezometer	November 15, 2012	-0.5 ¹ (i.e., above ground surface)	307.1
	Piezometer	May 15, 2013	-0.6 ¹ (i.e., above ground surface)	307.2
BC6-2	Piezometer	November 15, 2012	-0.2 ² (i.e., above ground surface)	305.5
	Piezometer	May 15, 2013	-0.2 ² (i.e., above ground surface)	305.5

¹ Piezometer screen noted to be plugged upon removal of well casing.

² Groundwater level measured above ground surface at the same level (elevation) as the surface water ponded around the piezometer pipe.

Groundwater elevations as encountered in the boreholes may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized on completion of drilling. Furthermore, groundwater elevations will vary depending on seasonal fluctuations, precipitation and local soil permeability.

4.7 Highway 66 – STA 15+590 to 15+610 (Deep Cut)

The plan, profile along the north toe of the proposed deep cut (offset 20 m from the cut centreline) and cross-sections through the deep cut area of the Highway 66 realignment, showing the borehole locations and interpreted stratigraphy between about STA 15+590 and 15+610 are shown on Drawings E1 and E2 in Appendix E. The realigned section of the highway reconnects with the existing Highway 66 at the west end of the realignment, extending through a cut up to about 8.5 m deep relative to the existing ground surface. The north side of the alignment will cut through a bedrock outcrop; whereas, the south side of the alignment (towards the existing highway) will be in an earth cut. In this area, both permanent and temporary cut slopes are required to accommodate detours and staging.

A total of five (5) boreholes (C1 to C5) were completed to investigate the subsurface conditions within the earth cut section of the realignment, along the south toe of the bedrock outcrop. The topography in this section of the



proposed cut in general slopes downward from west to east. This section of the proposed realignment contains moderate to dense tree cover, and is crossed by various walking/ATV trails and the existing Highway 66.

The subsurface soils along the south toe of the Highway 66 realignment in this Deep Cut section generally consist of granular fill at the ground surface underlain by a cohesive deposit of clayey silt to silty clay. The cohesive deposit is underlain by a deposit of silt, which in turn is underlain by a deposit of silty gravelly sand to sand and gravel to sandy gravel, underlain by inferred bedrock. Refusal to split spoon and further auger advancement and dynamic cone penetration was encountered at depths of up to about 10.8 m below ground surface, being deepest in the vicinity of about STA 15+610 along the Highway 66 realignment.

4.7.1 Embankment Fill

Borehole C5 was advanced through the existing Highway 66 embankment and encountered a 200 mm thick layer of asphalt, with the road surface at Elevation 316.8 m. A deposit of silty sand to gravelly sand fill was encountered below the asphalt in Borehole C5 and from ground surface in Boreholes C1 to C3, which were advanced adjacent to the existing roadway. The fill deposit was encountered between Elevation 316.6 m and 317.5 m and the thickness of the deposit ranges from 0.7 m to 1.8 m.

The SPT 'N'-values measured within the fill deposit are 18 blows and 22 blows per 0.3 m of penetration, indicating a compact relative density.

The natural water content measured on two samples of the fill deposit is about 5 per cent.

A grain size distribution of one sample of the fill deposit is shown on Figure E1 in Appendix E.

4.7.2 Clayey Silt to Silty Clay

A deposit of brown to grey clayey silt to silty clay, trace to some sand, trace organics was encountered underlying the fill in Boreholes C1 to C3 and C5 and from ground surface in Borehole C4. The surface of the clayey silt to silty clay deposit was encountered between Elevation 316.4 m and 315.7 m and the thickness of the deposit ranges from 1.0 m to 2.1 m.

The SPT 'N'-values measured within the clayey silt to silty clay deposit range between 3 blows and 12 blows per 0.3 m of penetration. One in situ field vane test carried out within the clayey silt to silty clay deposit near the bottom of the deposit measured an undrained shear strength greater than 100 kPa. The SPT 'N'-values together with the field vane shear strength suggest that the deposit has a soft to very stiff consistency.

The natural water content measured on five samples of the clayey silt to silty clay deposit ranges from about 18 per cent to 40 per cent.

Grain size distributions of two samples of the clayey silt to silty clay deposit are shown on Figure E2 in Appendix E.

Atterberg limits tests were carried out on four samples of the cohesive deposit and measured liquid limits ranging from about 32 per cent to 49 per cent, plastic limits ranging from about 20 per cent to 24 per cent and plasticity indices ranging from about 12 per cent to 25 per cent. The result of the Atterberg limits tests are shown on the plasticity chart on Figure E3 in Appendix E and indicate that the material is classified as clayey silt of low plasticity to silty clay of intermediate plasticity.



4.7.3 Silt

A deposit of grey silt, trace to some clay, trace to some sand was encountered underlying the clayey silt to silty clay deposit in all boreholes. The surface of the silt deposit was encountered between Elevation 314.6 m and 313.8 m and the thickness of the deposit ranges from 4.0 m to 5.0 m.

The SPT 'N'-values measured within the silt deposit range between 0 blows (i.e., weight of hammer) and 17 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The natural water content measured on eleven samples of the silt deposit ranges from about 19 per cent to 28 per cent.

Grain size distributions of six samples of the silt deposit are shown on Figure E4 in Appendix E.

An Atterberg limits test was carried out on one sample of the silt deposit and indicates that the material is non-plastic.

4.7.4 Silty Gravelly Sand to Sand and Gravel

A deposit of grey to brown silty gravelly sand to sand and gravel, trace to some clay was encountered underlying the silt in all the boreholes. The surface of the deposit was encountered between Elevation 310.6 m and 308.8 m and the thickness of the deposit ranges from 0.6 m to 4.6 m.

The SPT 'N'-values measured within the silty gravelly sand to sand and gravel deposit range between 8 blows and 60 blows per 0.3 m of penetration, indicating a loose to very dense relative density.

The natural water content measured on four samples of the silty gravelly sand to sand and gravel deposit ranges from about 6 per cent to 10 per cent.

Grain size distributions of three samples of the silty gravelly sand to sand and gravel deposit are shown on Figure E5 in Appendix E.

4.7.5 Refusal/Bedrock

Refusal to further split spoon/auger advancement and dynamic cone penetration was encountered in Boreholes C1 to C3 and C5 and Borehole C4, respectively, at depths ranging from about 7.8 m to 10.8 m below existing ground surface, ranging from Elevation 309.1 m to 306.0 m.

4.7.6 Groundwater Conditions

Groundwater levels were measured in open Boreholes C1, C3 and C5 during and upon completion of drilling at depths ranging between 2.2 m and 3.0 m below ground surface, corresponding to between Elevation 314.6 m and 314.1 m. Boreholes C2 and C4 caved at 0.6 m and 1.8 m below ground surface, respectively.

Groundwater elevations as encountered in the boreholes may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized on completion of drilling. Furthermore, groundwater elevations will vary depending on seasonal fluctuations, precipitation and local soil permeability.



4.8 Highway 66 - Excess Material Management Area

The plan and profile in the proposed excess material management (EMM) area showing the borehole locations and interpreted stratigraphy are shown on Drawing F1 in Appendix F. The proposed Highway 66 EMM area extends across a low-lying swamp, adjacent to swamp crossing H6/H7.

A total of nine (9) boreholes (EMM1 to EMM9) were completed to investigate the subsurface conditions within the EMM area. The topography in this section of the proposed highway realignment slopes slightly downward from west to east and from south to north and the terrain contains swampy areas, moderate to dense tree cover and multiple ATV trails.

The subsurface soils within the EMM area generally consist of peat underlain by a cohesive deposit consisting of an upper zone of clayey silt, a middle zone of silty clay to clay and a lower zone of clayey silt. The cohesive deposit is generally underlain by a deposit of silt which in turn is underlain by a deposit of silty sand to sand and gravel. Refusal to further casing advancement and/or split spoon penetration was encountered at depths of up to about 23.3 m, being deepest in the vicinity of Borehole EMM1.

4.8.1 Peat

A deposit of black fibrous peat ranging in thickness from 0.5 m to 4.3 m was encountered from ground surface, from Elevation 311.3 m to Elevation 308.0 m.

SPT 'N'-values measured within the peat deposit generally ranged between 0 blows (i.e., weight of hammer) and 5 blows per 0.3 m of penetration, suggesting a very soft to firm consistency. Two SPT 'N'-values at 2.4 m depth of 11 and 38 blows per 0.3 m of penetration were measured within the peat deposit in Boreholes EMM1 and EMM2, however these 'N' values are inferred to be due to the presence of frozen material and thus not representative. In situ field vane tests carried out within the peat deposit measured undrained shear strengths ranging from 10 kPa to 96 kPa. The SPT 'N'-values together with the field vane shear strength suggest that the deposit has a very soft to stiff consistency.

The natural water content measured on twelve samples of the peat ranges from about 13 per cent to 1,124 per cent. The organic content measured on one sample of the peat is about 59 per cent.

A laboratory consolidation test was carried out on one specimen of the peat deposit obtained from a Shelby tube sample in Boreholes EMM5. A preconsolidation pressure of 48 kPa was estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. A bulk unit weight of 10.2 kN/m³ and a specific gravity of 1.58 was measured on the consolidation test specimen. Details of the test results are shown on Figure F1 in Appendix F, and the test results are summarized below.



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Borehole Sample No.	Sample Depth/ Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	C_c	C_r	e_o	c_v^* (cm ² /s)
Borehole EMM5 Sample 2	1.8 m/ 308.8 m	4.0	48	44.0	12.0	2.4	0.5	5.8	7.7×10^{-3}

*For stress range between approximately effective overburden stress and approximate final stress, that is $6 \text{ kPa} \leq \sigma_v' \leq 20 \text{ kPa}$

where:

- σ_{vo}' is the effective overburden stress in kPa
- σ_p' is the preconsolidation stress in kPa
- OCR is the overconsolidation ratio
- C_c is the compression index
- C_r is the recompression index
- e_o is the initial void ratio
- c_v is the coefficient of consolidation in cm²/s

4.8.2 Sandy Silt

A deposit of brown sandy silt was encountered underlying the peat deposit in Borehole EMM9. The sandy silt deposit is of 1.0 m thick and was encountered at Elevation 307.5 m.

4.8.3 Clayey Silt to Clay

A cohesive deposit generally consisting of an upper zone of clayey silt, a middle zone consisting of silty clay to clay and a lower zone transitioning to clayey silt was encountered underlying the peat. The total thickness of the cohesive deposit is between 6.8 m and 15.4 m and the surface of the deposit was encountered between Elevation 309.7 m and 305.0 m.

The upper zone of the cohesive deposit consists of grey clayey silt, trace to some sand, trace organics and is between 1.7 m and 3.8 m thick. Within this upper zone of the cohesive deposit silt seams were frequently encountered and the deposit is considered to be varved.

The middle zone of the cohesive deposit consists of grey silty clay to clay, trace sand and is between 4.5 m and 9.5 m thick. Pockets of organic material were also encountered in the middle zone of this deposit on occasion in split spoon samples. Although this zone of the cohesive deposit is not considered varved, irregular varves were occasionally encountered.

The lower zone of the cohesive deposit consists of grey clayey silt, trace sand and is between 1.3 m and 4.8 m thick.

4.8.3.1 Clayey Silt

SPT 'N'-values measured within the clayey silt upper zone of the cohesive deposit range between 0 blows (i.e., weight of hammer) and 6 blows per 0.3 m of penetration. In-situ field vane tests carried out within the clayey silt portion of the deposit measured undrained shear strengths ranging from about 17 kPa to 77 kPa, indicating that the deposit has a soft to stiff consistency. The sensitivity is calculated to range between about 2 and 14.

The natural water content measured on ten samples of the clayey silt portion of the deposit ranges from about 28 per cent to 35 per cent.



A grain size distribution of one sample of the clayey silt upper zone of the cohesive deposit are shown on Figure F2 in Appendix F.

Atterberg limits tests were carried out on seven samples of the clayey silt portion of the deposit and measured liquid limits ranging from about 24 per cent to 31 per cent, plastic limits ranging from about 15 per cent to 18 per cent and plasticity indices ranging from about 9 per cent to 14 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure F3 in Appendix F and indicate that the material is classified as clayey silt of low plasticity.

A laboratory consolidation test was carried out on a specimen of the clayey silt deposit obtained from a Shelby tube in Borehole EMM1. A preconsolidation pressure of 110 kPa was estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. A bulk unit weight and a specific gravity of 18.9 kN/m³ and 2.76, respectively, were measured on the consolidation test specimen, respectively. Details of the test results are shown on Figure F4 in Appendix F, and the test results are summarized below.

Borehole Sample No.	Sample Depth/ Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	C_c	C_r	e_o	c_v^* (cm ² /s)
Borehole EMM1 Sample 5	6.4 m/ 304.2 m	20	110	90	5.5	0.235	0.007	0.89	1.4×10^{-2}

*For stress range between approximately effective overburden stress and final stress due to 3 m high embankment, that is 40 kPa $\leq \sigma_v' \leq 156$ kPa

where: σ_{vo}' is the in situ vertical effective overburden stress in kPa
 σ_p' is the preconsolidation stress in kPa
OCR is overconsolidation ratio
 e_o is 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

4.8.3.2 Silty Clay to Clay

SPT 'N'-values of 0 blows (i.e., weight of rods or hammer) per 0.3 m of penetration were measured within the silty clay to clay portion of the deposit. In situ field vane tests carried out within the silty clay to clay portion of the deposit measured undrained shear strengths ranging between about 12 kPa and 45 kPa, indicating that the deposit has a very soft to firm consistency. The sensitivity is calculated to range between about 2 and 12. One SPT 'N'-value of 51 blows per 0.3 m of penetration was encountered at the base of the cohesive deposit in Borehole EMM5, likely influenced by the underlying silty sand deposit.

The natural water content measured on twenty-one samples of the silty clay to clay portion of the deposit range from about 32 per cent to 85 per cent.

Grain size distributions of three samples from the silty clay to clay portion of the deposit are shown on Figure F5 in Appendix F.

Atterberg limits tests were carried out on thirteen samples of the silty clay to clay portion of the deposit and measured liquid limits ranging from about 35 per cent to 59 per cent, plastic limits ranging from about 19 per cent to 26 per cent and corresponding plasticity indices ranging from about 16 per cent to 33 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure F6 in Appendix F and indicate that the material is classified as silty clay of intermediate plasticity to clay of high plasticity.



4.8.3.3 Clayey Silt

SPT 'N'-values measured within the clayey silt lower zone of the cohesive deposit range between 0 blows (i.e., weight of rods or hammer) and 8 blows per 0.3 m of penetration. In situ field vane tests carried out within the clayey silt deposit measured undrained shear strengths ranging from 25 kPa and 34 kPa, indicating that the deposit has a soft to firm consistency. The sensitivity is calculated to range between 2 and 4.

The natural water content measured on seven samples of the clayey silt portion of the deposit ranges from about 33 per cent to 62 per cent.

Grain size distributions of two samples from the clayey silt lower zone of the cohesive deposit are shown on Figure F7 in Appendix F.

Atterberg limits tests were carried out on three samples of the clayey silt portion of the deposit and measured liquid limits ranging between of about 23 per cent and 31 per cent, plastic limits ranging between about 16 per cent and 20 per cent and corresponding plasticity indices ranging between about 7 per cent and 12 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure F8 in Appendix F and indicate that the material is classified as clayey silt of low plasticity.

4.8.4 Silt

A deposit of grey silt, trace to some clay, trace sand was encountered underlying the cohesive deposit in Boreholes EMM1, EMM2, and EMM6 to EMM9. The surface of the silt deposit was encountered between Elevation 300.1 m and 292.8 m and the thickness of the deposit ranges from 1.5 m to 4.6 m.

SPT 'N'-values measured within the silt deposit range between 2 blows and 10 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The natural water content measured on eight samples of the silt deposit ranges from about 23 per cent to 31 per cent.

Grain size distributions of three samples from the silt deposit are shown on Figure F9 in Appendix F.

Atterberg limits tests were carried out on three samples of the silt deposit. One of the samples was non-plastic and the other two samples measured liquid limit of about 24 per cent, plastic limits of about 20 per cent and 21 per cent, and corresponding plasticity indices of about 3 per cent and 4 per cent, indicating silt of slight plasticity as shown in Figure F10 in Appendix F.

4.8.5 Silty Sand to Sand and Gravel

A deposit of grey silty sand to sand and gravel, trace clay was encountered underlying the cohesive deposit in Boreholes EMM3 and EMM4 and underlying the silt deposit in Boreholes EMM1, EMM2 and EMM5 to EMM9. In Borehole EMM3, only a small amount of material was recovered from the one sample in this deposit indicative of the presence of gravel. The surface of the silty sand to sand and gravel deposit was encountered between Elevation 300.2 m and 289.5 m and the thickness of the deposit ranges from 0.3 m to 5.4 m.



SPT 'N'-values measured within the silty sand to sand and gravel deposit range between 4 blows and 48 blows per 0.3 m of penetration, indicating a very loose to dense relative density. Four SPTs did not penetrate the full sample depth on refusal conditions.

The natural water content measured on six samples of the silty sand to sand and gravel deposit ranges from about 7 per cent to 42 per cent.

Grain size distributions for three samples from the silty sand to sand and gravel deposit are shown on Figure F11 in Appendix F.

4.8.6 Refusal

Refusal to further split spoon and/or casing advancement was encountered in all boreholes except Borehole EMM6 at depths ranging from 10.7 m to 23.3 m below existing ground surface, ranging from Elevation 299.9 m to 286.1 m.

4.8.7 Groundwater Conditions

Groundwater levels were measured in the open boreholes upon completion of drilling at the ground surface in all boreholes except EMM9, in which the groundwater level was at a depth of 2.3 m below ground surface, ranging between Elevation 311.3 m and 305.7 m. Groundwater elevations as encountered in the boreholes may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized on completion of drilling. Furthermore, groundwater elevations will vary depending on seasonal fluctuations, precipitation and local soil permeability.

5.0 CLOSURE

The field drilling program was supervised by Mr. Gabriel Mathieu, Mr. Alex Mayot, Mr. Billy Murphy, Mr. Mat Riopelle, Mr. Ed Savard, Mr. Matt Soderman, and Mr. Matthew Thibeault over the three years that the investigation took place. This report was prepared by Mr. Matthew Thibeault, a geotechnical engineering intern and reviewed by Ms. Sarah Coyne, P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Jorge Costa, P.Eng., Principal and Golder's Designated MTO Foundations Contact, conducted an independent review of this report.



Report Signature Page

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PART B

FOUNDATION DESIGN REPORT

SWAMP CROSSING / HIGH FILL AREAS, DEEP CUT AND EXCESS

MATERIAL MANAGEMENT AREA

REALIGNMENT OF HIGHWAY 66 AT VIRGINIATOWN

FROM 10.6 KM EAST OF HIGHWAY 624 EASTERLY 3.4 KM

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5091-07-00



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides an interpretation of the geotechnical data obtained during the investigation and recommendations on the foundation aspects of design of the proposed works. The recommendations provided are intended for the guidance of the design engineer. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the design of the project. Those requiring information on aspects of construction must make their own interpretation of the subsurface information provided as it affects their proposed construction methods, costs, equipment selection, scheduling and the like.

6.1 General

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation, Ontario (MTO) to provide recommendations on foundation aspects for the detail design of the high fill embankments at various locations within the proposed Highway 66 realignment, in the Township of McGarry. The proposed foundation embankments along the Highway 66 realignment extend from approximately 11.0 km east of the junction of Highway 66 and Highway 624 easterly for a total distance of about 2.5 km. As part of this work, foundation recommendations are required for four (4) proposed high fill embankments (about 975 m in total length) and one (1) deep cut section. Recommendations have also been given for the excess material management area. Table 1 summarizes the locations of the areas investigated within the Highway 66 realignment project limits that require foundation design.

This report presents the results of embankment stability and settlement analyses and provides recommendations for stable embankment geometry, embankment fill materials and implementation of mitigation alternatives that may be required as a means to improve stability (if necessary) and reduce post-construction settlements. The report also addresses potential construction concerns and geotechnical problems associated with embankment construction, sub-excavating soft/organic materials and placement of fill materials. In addition, this report provides recommendations regarding management of excess material at the site.

6.2 High Fill Embankments, Embankments Over Swamps and Deep Cut

Based on the vertical profiles of the proposed Highway 66 realignment provided by MRC, the realigned highway swamp crossing/high fill sections will require fill embankments up to about 7.5 m high and a deep cut section through a bedrock outcrop and earthen plateau up to about 11 m deep.

Sections 6.2.2 and 6.2.3 of this report summarize the methods used to analyze the stability and settlement for critical sections of the swamp crossing/high fill embankments and the deep cut excavation required for the new Highway 66 realignment. Sections 6.3 and 6.4 present discussions related to settlement performance requirements and recommendations on potential alternatives for mitigating embankment in stability and settlement related design and construction issues. The results of the analyses and recommendations for mitigating in stability and time-dependent settlements for each individual swamp crossing/high fill area and deep cut section, where applicable, are presented in Section 6.5. Recommendations for excess material management are presented in Section 6.6. General aspects of subgrade preparation and embankment construction are presented in Section 6.7.

At all swamp crossing/high fill areas, the analyses assume of stability and settlement that the peat and near surface organic soils (i.e., peat and/or topsoil) will be removed prior to constructing the new embankments (as



discussed in Section 6.7.1). At the excess material management area, the analyses of stability and settlement assume that the rock fill containment berms will be constructed on the peat. For details on the thickness of organic deposits at each swamp crossing/high fill areas and the excess material management area, refer to Section 4. The piezometric conditions required in the analyses are based on the groundwater levels noted during drilling which were generally located at about or slightly below the level of the natural ground surface at most borehole locations, with artesian conditions noted in a few boreholes and piezometers (see Section 4).

6.2.1 Embankment Fill Types, Berm Requirements and Earth Cut Slopes

Different fill materials (i.e., rock fill and granular fill) used for embankment construction provide relative advantages and disadvantages in terms of availability, weight (i.e., driving force and applied load to the founding deposit), construction cost and time, ease of construction and post-construction performance.

It is understood that rock fill is the preferred embankment fill material for this project as it is available from rock blasting for road cuts required elsewhere on the project. In this regard, the majority of the stability and settlement analyses discussed in Section 6.5 have been carried out on the basis that the highway embankment will be constructed of rock fill. As required for different stability and settlement mitigation options, the use of granular fill, either for backfill of sub-excavated areas or for embankment construction, has also been considered in the analysis.

Rock Fill

The main advantages of constructing embankments using rock fill are: the ability to achieve steeper side slopes of 1.25 Horizontal to 1 Vertical (1.25H:1V), which is required in areas with limited right-of-way; reducing the overall quantity of fill material required for the project; and for placement of material in sub-excavated areas under water. Rock fill will also be available locally from excavations in deep cuts through bedrock outcrops within this project alignment. The disadvantage of using rock fill for the construction of embankments is that some post-construction settlement of the embankment fill itself will occur. Settlement of the rock fill is discussed further in Section 6.2.3.3.

In accordance with MTO Northern Region Pavement Practices and Guidelines (1997) as amended by MTO Memorandum “Use of Mid-Slope Berms for Rockfill Embankments” dated February 8, 2005, 2 m wide berms should be incorporated into the rock fill embankment side slope profile for uninterrupted slopes greater than 10 m high. Given that the proposed new embankments along the realignment are less than 10 m high, 2 m wide mid-slope berms are not required.

Granular Fill

The main advantages of using granular fill for embankment construction are: the ease of construction; and negligible post-construction settlement within the embankment fill itself. However, this fill option requires a larger volume of material and potentially wider right-of-way because the side slopes of granular fill embankments (2H:1V) are flatter than those of rock fill. For this project, acceptable granular fill is considered to be well graded, locally available and/or imported granular material.



For granular fill embankments, 2 m wide berms should be incorporated into the side slope profiles for uninterrupted slopes greater than 8 m high. Given that the proposed new embankments along the realignment are less than 8 m high, 2 m wide mid-slope berms are not required.

Granular fill should also be used as backfill below the ground surface after removal of the organic deposits and near surface cohesive soils in areas proposed for wick drains, should they be considered the preferred settlement mitigation alternative, to allow for installation through the backfill.

Earth Cut Slopes

For the earth cut extending between STA 15+590 and STA 15+610 a 2 m wide bench should be incorporated into the cut slope profile for uninterrupted slopes greater than 8 m high/deep. For this site, a mid-slope bench should be incorporated into the cut slope in the vicinity of STA 15+610.

6.2.2 Stability

The following sections outline the methodology used to evaluate embankment stability at the various swamp crossing/high fill areas and deep cut section and also present the parameters used in the analyses for each of the critical section(s). The results of the stability analyses for each swamp crossing/high fill area and the deep cut section, the results of the settlement analyses, and recommendations regarding possible design and construction alternatives to mitigate stability issues and/or post-construction settlement are presented in Section 6.5.

6.2.2.1 Methodology

Stability analyses were carried out for the critical sections of the proposed fill embankments in each swamp crossing/high fill area and the excess material management area and for a critical cut slope in the deep cut area. Critical sections correspond to the greatest new embankment height or deep cut, and/or the maximum thickness of soft, compressible cohesive soils. Generally, one critical section was identified for each swamp crossing/high fill area and the deep cut section. In all areas where cohesive deposits were encountered, the stability of the proposed new embankment section(s) was analyzed using limit equilibrium methods. In areas where cohesive deposits were not encountered, the stability of the proposed embankment section was considered adequate based on engineering judgement and precedent experience in similar soil conditions. For the swamp crossing/high fill areas the stability analysis assumes that the organic deposits and near surface cohesive soils have been removed and replaced in accordance with OPSD 203.010 (Embankments Over Swamp) prior to construction of the new embankment (see Table 3 for estimated thickness of these deposits).

All limit equilibrium slope stability analyses were carried out using the commercially available program GeoStudio 2007 (Version 7.19), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis or Slide (Version 6.0), produced by Rocscience Inc., 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 FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static total stress (undrained) conditions at the end of construction and for long-term effective stress (drained) conditions for MTO embankments. This FoS is considered adequate for the



embankments at these sites considering the design requirements and the field data available and is based on deep-seated, global failure surfaces that would affect the operation of the highway. In order to obtain the critical failure surface, circular or “rotational” and/or block or “wedge” failure surfaces were modeled in the analysis. The stability analyses were carried out to check that the target minimum FoS was achieved for the various embankment heights and geometries.

For the excess material management sections bearing capacity and lateral spreading analyses were performed, assuming geosynthetics were used to establish a rigid footing. A target minimum FoS of 2.0 is normally adopted for the bearing capacity and lateral spreading design of embankment slopes under static total stress (undrained) conditions. This FoS is considered adequate for the containment berms in this area. The stability analyses were carried out to check that the target minimum FoS was achieved for the various embankment heights and geometries.

6.2.2.2 Parameter Selection

The simplified stratigraphy together with the associated strength(s) and unit weight(s) employed for the different native soil types at the critical sections in each swamp crossing/high fill areas and the excess material management area and deep cut are summarized in Table 3. Additional details of foundation engineering parameters employed for some cohesive deposits (i.e., clayey silt/silty clay/clay/peat) encountered in high fills H1/H4, H6/H7 and the EMM areas are provided on Figures A11, C24, F12 and F13 in Appendices A, C and F, respectively. The rock fill modelled in the analyses is assumed to have a unit weight of 19 kN/m^3 and an effective friction angle of 40° . The stability of the proposed Highway 66 embankments was analyzed for a side slope geometry of 1.25H:1V assuming rock fill construction and backfill. If granular fill is required to backfill sub-excavated areas to allow for wick drain installation, then a unit weight of 20 kN/m^3 and an effective friction angle of 35° would be applicable, assuming the granular fill is placed uncompacted below the water level.

The overburden encountered in the various swamp crossing areas and deep cut areas is composed of either granular soils only (silts, sands, sandy silt/silty sand, and/or sand and gravel) or a combination of cohesive deposits (clayey silt, silty clay, clay and/or peat) and granular soils. For granular soils, effective stress parameters were employed in the analyses assuming drained conditions. The effective stress parameters (effective friction angle and effective cohesion) for the peat/root mat and granular soils were estimated from empirical correlations using the results of in situ 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 tests, inferred from the laboratory consolidation tests results, and estimated from correlations with the SPT results and other laboratory test data (i.e., natural water content), where appropriate. For 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:

$$\begin{aligned} s_u &= \text{average mobilized undrained shear strength (kPa)} \\ \sigma'_p &= \text{preconsolidation pressure (kPa)} \end{aligned}$$



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:

$S_{u(mob)}$	=	average mobilized undrained shear strength (kPa)
$S_{u(FV)}$	=	undrained shear strength from field vane test (kPa)
μ	=	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 of minimum shearing resistance (Milligan and Lo, 1967).

For the peat, a reduction factor of 50 per cent was employed to the undrained shear strength obtained from the field vanes to account for mode and rate of shear (Mesri and Ajlouni, 2007). In addition, the correlation between undrained shear strength and water content from the Muskeg Engineering Handbook (1969) was also used.

6.2.3 Settlement

The following sections outline the methods used to carry out the settlement analyses at the various swamp crossing/high fill areas and the excess material management area and also present the parameters used in the analyses for each of the embankment critical section(s). The results of the analyses are presented in Section 6.5 for each swamp crossing/high fill areas and the excess material management area where they are discussed together with the results of the stability analyses and recommendations regarding possible design and construction alternatives to mitigate stability issues and/or post-construction settlement.

6.2.3.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out at the critical sections of the proposed fill embankments using the commercially available program *Settle*^{3D} (Version 2.0) produced by Rocscience Inc. and/or hand/spreadsheet calculations. Critical sections correspond to the greatest new embankment height and/or the maximum thickness of soft, compressible cohesive soils. The settlement analysis assumes that the organic deposits and near surface cohesive soils have been removed and replaced in accordance with OPSD 203.010 (Embankments Over Swamp) prior to construction of the new embankments.

The sources of settlement are 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) consolidation of the cohesive deposits (long-term); and
- self-weight compression of the embankment fill materials (long-term).

The thickness of the compressible foundation soils and the height of the embankments vary along the proposed highway alignment within each swamp crossing/high fill/excess material management areas, and as such the settlements along the length of a given alignment will similarly vary. Given that the analyses were carried out at



the critical sections of each swamp crossing/high fill areas and the excess material management area, the settlements estimated will generally represent the maximum value along a given section of the alignment.

6.2.3.2 Parameter Selection

The simplified stratigraphy together with the associated deformation and time-rate consolidation parameters employed for the different native soil types for the critical sections in each swamp crossings/high fill area/excess material management area are given in Table 3. Additional details of foundation engineering parameters employed for some cohesive deposits (i.e., clayey silt/silty clay/clay) encountered in high fill areas H1/H4, H6/H7 and EMM area are provided on Figures A11, C24, F12 and F13 in Appendices A, C and F, respectively.

The immediate compression of the cohesionless deposits (i.e., silt, sandy silt to silty sand, sand, sand and gravel fill) were modelled by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). These estimated values were compared with the typical range of expected values for similar soil types, as outlined in Canadian Highway Bridge Design Code and its Commentary (CHBDC, 2006) and adjusted, if necessary.

The consolidation settlement of the cohesive deposits was assessed using the results of the laboratory consolidation tests and, where appropriate, in situ field vane tests to estimate the deformation parameters. In addition, for the clayey soils the results of the laboratory index tests were also employed to further assess deformation parameters (i.e., compression and recompression indices) using empirical correlations proposed in literature by Koppula (1986), Terzaghi and Peck (1967), Kulhawy and Mayne (1990) and Azzouz et al. (1976). The correlation by Terzaghi and Peck (1967) and Koppula (1986) relating the natural water content and liquid limit to the compression index was found to be the most consistent with the results of laboratory consolidation tests for the clayey soils at this site.

For clayey soils, the following correlation relating in situ undrained shear strength to preconsolidation pressure (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 established for the swamp crossings/high fill areas using the combined results of the laboratory consolidation tests and the estimated c_v values based on the Unified Facilities Criteria (U.S. Navy, NAVFAC 1986) correlation with liquid limit assuming normally consolidated soils.

For the peat, the following correlation relating in situ undrained shear strength to preconsolidation pressure (Mesri and Ajlouni, 2007) was employed:



$$S_{u(FV)} = \sigma'_p$$

where:

$S_{u(FV)}$	=	undrained shear strength from field vane test (kPa)
σ'_p	=	preconsolidation pressure (kPa)

In addition to primary consolidation within the cohesive deposits (i.e., clayey silt 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 stress. The following relationship has 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
H	=	initial thickness of 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 consolidation tests, the following empirical correlation by Mesri (1973) was also utilized to estimate $C_{\alpha\epsilon}$ from water content:

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

where:

w_n	=	natural water content (%)
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For peat, in addition to estimating the modified secondary compression index from consolidation tests, the following empirical correlation by Mesri and Ajlouni (2007) was also utilized to estimate $C_{\alpha\epsilon}$ from water content:

$$C_{\alpha} / C_c = 0.06 \pm 0.01$$

where:

C_{α}	=	secondary compression index as estimated from laboratory consolidation tests
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$$C_{\alpha} = \text{Compression index as estimated from laboratory consolidation tests}$$

6.2.3.3 Settlement of Embankment Fill

Where rock fill is used for the construction of the proposed 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); and



- 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 MTO Foundations Guideline, “MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates”, dated September 2010.

Rock fill should be placed, whenever possible, in a controlled manner (i.e., not end-dumped) in accordance with Special Provision (SP) 206S03 (Rock Excavation, 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.

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 Foundations Guideline (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 (6) months following construction of the embankment to full height. The short-term settlement is expected to be fully completed within one (1) year following the completion of embankment construction to full height.

Long-Term Rock Fill Settlement

The magnitude of long-term post-construction settlement for compacted and end-dumped rock fill may be estimated in accordance with the MTO Foundations Guideline (September 2010), as follows:

Total Height of Rock Fill, H	Long-Term Rock Fill Settlement	
	Compacted Rock Fill	Dumped Rock Fill
Up to 15 m	0.1% H	0.2% H

The long-term rock fill settlement is expected to occur from one (1) year following the completion of construction over the life of the embankment.



6.3 Settlement Performance Requirements

The settlement performance criteria for design of high fill embankments and embankments over swamp crossings is in accordance with Section 1.1 of MTO Foundation Guideline, “Embankment Settlement Criteria for Design”, dated March 2010. In general, new embankments not approaching a structural element are to be designed as follows:

- Total settlements and differential settlement rates are to be less than 200 mm and 100:1, respectively, over a 20-year period following completion of construction for a “non-freeway” highway.

Where new embankments approach structural elements, more stringent settlement criterion will apply in accordance with Section 1.2 of the MTO Foundation Guideline.

These performance criteria form part of the overall design performance for each high fill area.

6.4 Stability and Settlement Mitigation Options

At each high fill embankment, embankment over swamp crossing location and deep cut section, as applicable, stability and settlement have been assessed based on existing subsurface conditions and proposed embankment fill heights. The presence of weak/soft, compressible soils underlying a proposed embankment can lead to the potential for instability or unacceptably large settlements with the placement of fills. There are a number of options for mitigating the potential for instability and/or settlements. A brief discussion on these alternatives is given below.

Details of the mitigation options for the swamp crossing/high fill areas requiring measures to mitigate stability/settlement issues of the foundation soils are provided in Section 6.5. These measures include: full or partial sub-excavation of unsuitable (organic/soft) soils; preloading with or without surcharging; wick drains; lightweight fill [expanded polystyrene (EPS)]; and combinations of these measures. Ground improvement measures such as rammed aggregate piers, deep soil mixing and dynamic compaction are considered mitigation alternatives for high fill areas where preloading is not possible due to construction staging. In other areas, such ground improvement measures are not considered suitable or cost effective due to the composition (i.e., cohesive soils) and/or thickness of the deposit and/or high groundwater conditions, and are not further discussed herein. The advantages, disadvantages, relative costs and risks/consequences of mitigation alternatives for the swamp crossing/high fill areas and deep cut area, where required, are summarized in Tables A1 to A3 in Appendix A. In addition, a comparison of the estimated post-construction settlement over a 20-year period between the base case (i.e., no foundation mitigation carried out) and the various mitigation alternatives considered was carried out for each of the swamp crossing/high fill areas. The results of the settlement analyses are summarized in Table 4.

Depending on the area, one alternative or a combination of alternatives to mitigate stability and/or settlement issues may be more advantageous than others. A summary of the preferred foundation mitigation option for each swamp crossing/high fill area and deep cut section, including the recommended embankment fill type and embankment side slope, maximum depth of organics encountered, stability/settlement mitigation, estimated settlement (during construction and post-construction), recommended width of platform widening and recommended excavation guideline is provided in Table 5.

In areas where the foundation soils consist primarily or only of granular deposits, it is not anticipated that there will be embankment stability issues or significant settlement problems, provided all organic deposits and near



surface cohesive soils are removed prior to construction and the requirements for mid-height berms are incorporated into the embankment design, where necessary. As such, in these areas, generally there is no need to implement any special construction procedures to maintain stability or to minimize long-term foundation settlements or to adjust the construction schedule.

6.4.1 Full Sub-Excavation

Sub-excavation of the weak/soft and compressible (i.e., clayey) deposits underlying the footprint of a proposed embankment in advance of the placement of rock fill is a potential option for improving the stability and controlling long-term settlement in some areas of this site. The removal of the soft, compressible cohesive soils would result in improved stability and significantly reduce settlement within the areas underlain by relatively thinner cohesive deposits and/or where high embankment fills are proposed. It should be noted that despite the reduction in settlement, the post-construction settlement of rock fill itself may still exceed the settlement performance criterion. As such, the embankments may need to be preloaded for a period of time to be able to attain adequate construction settlement and subsequently meet the post-construction settlement criterion associated with long-term performance of the roadway. The additional below grade rock fill embankment should be constructed with the same side slope profile as that of the above grade embankment (i.e., 1.25H:1V for rock fill) since the natural slope of the rock fill should not be affected by placement under water. This option has the advantage that construction of the above grade embankment could proceed upon completion of sub-excavation and replacement without concerns of instability. However, full sub-excavation will produce a large volume of excess material for disposal and will require a large volume of rock fill or granular fill replacement. The necessity to develop stable side slopes or back slopes within the excavation may result in cut slope geometries ranging from 1H:1V to as flat as 3H:1V. Flatter slopes would increase the lateral extent of the excavation and may require a wider right-of-way.

Based on the results of the subsurface investigation in the swamp crossing/high fill areas, the depth to the bottom of the cohesive deposit is up to about 21 m below existing ground surface. In general, groundwater was encountered at/near the existing ground surface. We understand that based on MTO field experience on similar embankment construction projects, the practical maximum depths that can be reached with conventional and long stick excavator equipment is about 6 m and 12 m, respectively. Below a depth of 12 m, specialized drag-line equipment would be required. As such, in the absence of unforeseen conditions which would prohibit its application, sub-excavation of organic and soft compressible soils up to a practical depth of about 12 m and replacement with rock fill is considered a generally feasible option for construction of the highway embankments at this site and would result in enhanced stability and reduced settlement of the embankments.

This option is most suited for areas where there is a limited thickness of organic deposits and near surface weak/soft compressible soils underlying the proposed embankment (i.e., less than 12 m), making their removal practical where there are no requirements for setbacks, where adequate right-of-way is available, and where there are no conflicts with encroachment on existing adjacent features.

The advantages of this option are:

- improved stability;
- reduced post-construction settlements of the foundation;
- potential reduced delay in construction; and



- no requirement for stabilizing toe berms.

The disadvantages of this option are:

- generation of large volume of excess material requiring disposal/management;
- increased post-construction settlement of rock fill, typically requiring a preload period, in addition to sub-excavation, to satisfy the settlement performance criterion;
- potential for requiring a larger right-of-way corridor due to backslope requirements; and
- requires greater quantities of rock fill.

6.4.2 Preloading (with Stability [Toe] Berms and/or Staged Construction)

As an alternative to sub-excavation and replacement of the weak/soft, compressible foundation soils, preloading may be considered for improving the stability and reducing post-construction settlements of the proposed embankments. Preloading refers to the placement of fill either up to the proposed profile grade of the highway/ramp or a portion thereof (i.e., partial preload), in one or more stages, in advance of embankment completion and final pavement construction, in order to preconsolidate the underlying compressible soils. Preloading reduces the magnitude of long-term, post-construction settlements by promoting such settlements to occur under fill loads in advance of final grading of the embankment. It also may increase the strength of cohesive soils underlying the embankment footprint, thereby improving stability.

In general, preloading requires placement of embankment fill (either in whole or part), and in some cases, monitoring of settlements, and possibly pore pressures, for a period of time corresponding to approximately the 'End of Primary' (EoP) consolidation of cohesive soils. EoP consolidation times will vary depending on the properties and thicknesses of the cohesive deposits, and the height of the fill. Where secondary consolidation (creep) settlements are expected to be small over the design life of the embankment, final grading for construction can proceed once the estimated EoP consolidation has occurred. Where creep settlements are considered to be large enough to affect the long-term performance of the highway, these settlements can be reduced by surcharging the embankment with an additional thickness of fill or by constructing a portion of the final embankment with lightweight fill (i.e., EPS) upon the completion of the design preload period.

In areas where cohesive deposits are thick and/or weak/soft, and where such conditions coincide with proposed high embankment fills, it may be necessary to construct stability berms along the embankment toes and/or place the embankment fill in stages in layers of limited thickness to ensure that the stability of the embankment is maintained. Stability (toe) berms consist of rock fill buttresses placed against the toe of the proposed embankment fill, producing a stepped embankment cross-section geometry. This stepped configuration produces a similar effect (i.e., increased stability) as using flatter embankment slopes, but often requires less fill material. Depending on the subsurface conditions and the proposed embankment height, toe berms will typically be on the order of about one third to one half of the height of the final embankment. The lateral extent (width) of toe berms will vary depending on the results of the stability analyses, but could range from half to one times the highway embankment height, or greater. Where staged construction is required, the individual layers of fill would have limited thickness and each construction stage would be separated by a suitable time interval to allow pore pressures to dissipate and strength gain to occur in the underlying cohesive soils while limiting the potential for instability of the embankment.



It should also be noted that with preloading, it is still typically required that all existing organic deposits and near surface cohesive soils be sub-excavated prior to placement of any fill, because these soils are highly compressible and experience significant secondary consolidation (creep) settlement.

This option is most suited for areas where removal of cohesive soils and their replacement with rock fill is not considered practical (i.e., the depth of cohesive deposits is greater than 12 m) and where a delay in the construction schedule is acceptable or can be accommodated.

The advantages of this option are:

- reduced generation of excess excavation spoil compared with full sub-excavation;
- will not require a larger right-of-way corridor, unless large toe berms are required; and
- the quantity of rock fill is limited to that required for sub-excavation and replacement of the organic deposits and near surface cohesive soils (if toe berms are not required), and to compensate for consolidation and foundation soil settlements.

The disadvantages of this option are:

- construction is delayed to allow for all or a portion of primary consolidation to be completed, and possibly for staged construction (if required);
- increased quantity of rock fill if toe berms are required for stability;
- may require lightweight fill for a portion of the construction of the final embankment to reduce long-term post-construction settlements if creep settlements are expected to be large;
- requires an instrumentation and monitoring program to assess when the settlement performance criterion has been achieved; and
- requires re-grading to account for settlement prior to construction of the final pavement structure.

6.4.3 Surcharging (with Stability [Toe] Berms and/or Staged Construction)

Similar to preloading, surcharging refers to the placement of embankment fill in advance of final pavement construction to reduce long-term, post-construction settlements (including creep settlement). The difference between preloading and surcharging is the amount of fill placed and the time required for consolidation to be achieved. With surcharging, the fill is placed to the full embankment height, followed by an additional lift of fill (the surcharge) above that required to construct the final embankment geometry. This additional lift of fill applies greater stress to the underlying cohesive deposits and reduces the time to achieve primary over-consolidation that achieved by preloading only, resulting in over consolidation of the underlying compressible foundations soils. At the EoP consolidation, the portion of the surcharge fill remaining above the required embankment height (sub-base level) is removed. The surcharge fill can also be left in place for a longer duration to reduce the magnitude of long-term, creep settlement.

As with preloading, it may be necessary to construct toe berms or stage the placement of embankment fill and surcharge to limit the potential for instability. Depending on the stability conditions, toe berms required during the surcharge period may be temporary and could be fully or partially removed upon completion of the surcharge



period. Upon the completion of the design surcharge period, the removed surcharge fill may be re-used on other parts of the site.

Surcharging is most suited to those areas considered appropriate for preloading, where the stability of the higher surcharged embankment can be practically maintained by reasonably sized excavations, toe berms or staged construction, but where sufficient time for primary consolidation settlements to occur under preload fill loads alone is not available. Surcharging can also be considered for areas where large creep settlement is expected.

The advantages of this option are:

- reduced generation of excess excavation spoil over full sub-excavation;
- reduced secondary consolidation (creep) settlement;
- will not require a larger right-of-way corridor, unless large toe berms are required;
- the quantity of rock fill is limited to that required for sub-excavation and replacement organic deposits and near surface cohesive soils, and to compensate for consolidation and foundation soil settlement (if toe berms are not required); and
- decreased delay time for construction over preloading alone.

The disadvantages of this option are:

- construction is delayed, albeit less than for preloading, to allow for primary consolidation to occur;
- longer construction time if staged construction is required;
- larger quantity of rock fill if toe berms are required for stability as compared to preloading alone;
- requires an instrumentation and monitoring program to assess when EoP consolidation is reached; and
- increased handling of the surcharge fill.

6.4.4 Wick Drains

Where sub-excavation is not practical (i.e., due to the thickness of or depth to the compressible soil deposits), and where the time required to reach the settlement performance criterion is considered too long, even with surcharging, consideration may be given to installing wick drains in conjunction with surcharging to accelerate the rate of primary consolidation. Wick drains are prefabricated geotextile drains installed vertically from ground surface into or through the soft, compressible foundation soils in order to increase the rate of excess pore pressure dissipation. Typically, wick drains are installed on a 1 m to 3 m triangular grid spacing over the embankment footprint.

The use of wick drains is most suited to areas with thick (i.e., greater than about 5 m) deposits of soft, compressible foundation soils and proposed high embankment fills where primary consolidation times are large even under surcharge conditions.

It would still be necessary to sub-excavate and remove organic deposits and near surface cohesive soils and place a granular drainage blanket at ground surface prior to the installation of the wick drains.



If the thickness of sub-excavation of the organic deposits and near surface cohesive soils is greater than about 4 m, then pre-drilling to install wick drains may be required to advance the drains through the backfill material, which must consist of granular material rather than rock fill. Pre-drilling may also be required if there is a substantial thickness of a non-cohesive deposit (i.e., sand) present above the surface of the main cohesive deposit.

The advantages of this option are:

- decreased consolidation time under surcharging; and
- decreased rate of staged construction, if required to maintain stability during construction.

The disadvantages of this option are:

- additional time and expense to install wick drains prior to embankment construction;
- may require pre-drilling at wick drain locations if a substantial thickness of backfill or non-cohesive deposit is present above the clay, incurring additional time and expense;
- additional long-term settlements due to creep settlement of the cohesive deposit (if not compensated for by surcharging or lightweight fill);
- requires an instrumentation and monitoring program to assess when the settlement performance criterion has been achieved; and
- requires re-grading to account for settlement prior to construction of the final pavement structure.

6.4.5 Lightweight Fill

Another alternative for reducing the magnitude of long-term settlement and improving stability in areas of weak/soft, compressible foundation soils is to use lightweight fill, such as EPS, for embankment construction. The use of lightweight fill reduces the load applied to the foundation soils due to the low density of the fill material. This in turn reduces the magnitude of post-construction settlement and reduces the potential for instability.

Lightweight fill is not considered a practical option for general use due to the expense and/or shipping costs for the supply of these types of fills. Rather, lightweight fill is most suited for areas underlain by deep compressible subsurface deposits, where sub-excavation is not practical or feasible, where long-term post-construction creep settlements affect the performance of the highway and where there is no available time in the construction schedule for a sufficient preload or surcharge period. In addition, lightweight fill can be used in conjunction with preloading, surcharging and wick drain designs in order to optimize the design.

Where a stability and/or settlement mitigation option requires the use of lightweight fill (i.e., EPS) as part of the construction of the embankments, rock fill cannot be used as the levelling pad or protective cover for the EPS due to the size of rock fill particles. As such, for these embankments, granular fill is to be used for levelling pad construction and for the side slope protective cover.

The advantages of this option are:

- improved stability;



- reduced long-term post-construction settlements; and
- shortened construction schedule.

The disadvantages of this option are:

- requires embankments to be constructed with 2H:1V side slopes given the need for granular fill for levelling pad and conventional soil cover on the side slopes (i.e., cannot use rock fill);
- significant additional expense of lightweight fill (depending on the volume required); and
- not feasible to install below the groundwater table (due to buoyancy forces) and in low height embankments (due to minimum conventional soil cover requirements on top of the EPS).

6.4.6 Ground Improvement

Where conventional stability or settlement mitigation techniques are not viable due to space or property restrictions, traffic staging, timing restrictions or other considerations, then another mitigation alternative that can be considered is ground improvement. Typically, ground improvement consists of physically altering the properties of the soft compressible soils to enhance stability and to reduce the post-construction settlement. Ground improvement options that can be considered include but are not limited to: rammed aggregate piers, soil grouting, dry or wet in situ soil mixing and dynamic compaction. The specific ground improvement option that may be feasible/utilized will depend on the soil properties, construction restrictions and embankment height.

The advantages of this option are:

- improved stability;
- reduced long-term post-construction settlements;
- can be used in areas of restricted space; and
- shortened construction schedule.

The disadvantages of this option are:

- potential embankment fill type restrictions; and
- significant additional expense of ground improvement technique - cost of specialized equipment and design.

6.4.7 Instrumentation and Monitoring

For some areas where the preloading and surcharging options are adopted and in all areas where staged construction and/or wick drains foundation options are adopted, the magnitude and time-rate of settlement as well as the dissipation of pore pressures during and after construction of embankments should be assessed with monitoring instrumentation. Such monitoring would consist of installing settlement pins/stakes, settlement plates (SPs) and vibrating wire piezometers (VWPs) below the embankments and taking regular measurements/readings at given intervals of time during and after construction of the embankments for the duration of the preloading/surcharging period. In addition, standpipe piezometers (SPPs) may be required and



are usually installed to provide background pore pressure readings for the vibrating wire piezometers. This monitoring instrumentation is particularly important where it is considered necessary to carefully monitor the stability of the foundation soils during staged placement of embankment fill.

The extent of instrumentation and the frequency of monitoring required will depend on the foundation treatment alternative chosen for a given site and the height of the proposed embankment fill. Specifications for the type, number and layout of the instrumentation, together with the supply, installation, protection and monitoring frequency should be included as a Non-Standard Special Provision (NSSP) in the Contract Documents.

6.5 Results of Analysis

The results of the stability and settlement analyses for each swamp crossing/high fill areas and the deep cut section are provided in the following sections. In addition, the options and recommendations for achieving the target FoS for the required embankment geometry and for minimizing the time-dependent, post-construction settlements are also discussed. For swamp crossing/high fill areas and deep cut section that require stability and/or settlement mitigation, the advantages, disadvantages, relative costs, and risks/consequences of various alternatives for these areas and ranking of the alternatives are summarized in the Evaluation of Stability/Settlement Mitigation Options tables presented in the Appendices.

In areas where the foundation soils consist of cohesionless deposits, it is anticipated that there will not be any significant risk of instability of the embankments. Similarly, the settlement of the foundation soils in these areas is expected to occur during or shortly after construction, as a result of the estimated relatively high permeability of these soils. In these areas there is typically no need to implement any special construction procedures or to extend the schedule to maintain stability or to mitigate settlement of the foundation soils.

In areas where the foundation soils are comprised of cohesive deposits, soft layers constitute zones of potential instability for the proposed embankments and time-dependent settlements of the new embankments are expected. In these areas, consideration must be given to an enhanced design and/or a construction sequence that will achieve the minimum target FoS of 1.3 for the proposed new embankment height and geometry and limit the post-construction settlements and subsequent maintenance of the new highway pavement structure.

For new embankments constructed with rock fill or where sub-excavation and backfilling with rock fill is carried out, settlement of the embankment rock fill is also expected due to compression of the rock fill itself (see Section 6.2.3.3). In these areas, the post-construction settlement of rock fill may exceed the settlement performance criterion. As such, the embankment would need to be preloaded to promote settlement prior to roadway construction such that post-construction settlements associated with long-term performance are minimized to an acceptable magnitude.

6.5.1 Highway 66 – STA 13+080 to 13+185 (High Fill H4) Existing Highway 66 Connection – STA 10+000 to 10+125 (High Fill H1)

Due to the localized subsurface conditions and varying embankment heights within High Fill H1/H4, this high fill section has been divided into two separate areas which have similar stability and/or settlement issues and associated mitigation requirements as follows:



FOUNDATION REPORT – SWAMP CROSSING/HIGH FILL AREAS REALIGNMENT OF HIGHWAY 66, AT VIRGINIATOWN, GWP 5091-07-00

Highway	Stations
Proposed Highway 66 Alignment High Fill H4	STA 13+080 to 13+185
Proposed Highway 66 Connection High Fill H1	STA 10+000 to 10+050 STA 10+050 to 10+125

In the vicinity of High Fill H1/H4, there are large rock outcrops to the north of the proposed realignment centreline at High Fill H4 and to the east of the connector roadway at High Fill H1. As a result of the rock outcrops, the topography along the proposed Highway 66 Realignment slopes upward from west to east and south to north. The topography along the Existing Highway 66 Connection slopes upward from south to north. A wet grassy area is present at the west end of the section adjacent to the existing Highway 66. The subsurface soils along the high fill section are comprised generally of surficial organic deposits underlain by a clayey silt to clay deposit up to about 10 m thick, which in turn is underlain by cohesionless soils consisting of silt to sand and gravel which extend up to about 17 m below ground surface.

6.5.1.1 Highway 66 – STA 13+080 to 13+185 (High Fill H4)

Between STA 13+080 and STA 13+185, the north (left) side of the embankment is within a rock cut which will be up to about 2 m deep, and the south (right) side of the embankment will require a fill embankment up to about 4.3 m high above the existing ground surface at the right shoulder of the embankment (or about 3.5 m high at the proposed centreline) to achieve the proposed vertical highway profile. The topography in this area consists of bedrock outcrops with moderate to dense tree cover and the ground surface is sloping downwards towards the south.

The subsurface conditions in this area generally consist of a 0.1 m to 0.8 m thick deposit of peat and/or topsoil, underlain by an up to 10.3 m thick deposit of clayey silt to clay extending to a depth of up to about 11 m below the existing ground surface. The clayey silt to clay deposit is underlain by deposits of sandy silt to silt, silty sand to sandy gravel and/or bedrock at depths up to 17 m below ground surface. The upper about 2 m to 3 m portion of the cohesive deposit is of stiff consistency and is considered to be the weathered crust. Details of the subsurface conditions for High Fill H4 are presented in Section 4.3 and shown on Drawing A1 in Appendix A.

The simplified stratigraphy and the associated unit weight, strength, deformation and time-rate consolidation parameters employed for the different soil types encountered in this area are summarized in Table 3 and shown on Figure A9 in Appendix A. The piezometric condition used in the analyses is based on the groundwater level noted during drilling and installed piezometer readings, as summarized in Section 4.3.

6.5.1.1.1 Stability

The critical section (i.e., the greatest embankment height and maximum thickness of soft, compressible foundation soils at the right toe of slope) for this area is located at about STA 13+150. The proposed embankment height is about 4.3 m at the right crest of the embankment at this location and the cohesive deposit is up to 10.3 m thick and extends to a depth of about 11 m below ground surface. The stability analysis carried out at the critical section indicates that after completion of construction (including removal and replacement of the organic deposits); the embankment will have a FoS greater than 1.3 for a deep-seated, global failure surface



that would impact the operation of the highway as shown on Figure A10 in Appendix A. Therefore, stability mitigation is not required in High Fill H4.

6.5.1.1.2 Settlement

To estimate the magnitude of the expected settlements due to new construction, an analysis was carried out at a different critical section (i.e., the maximum thickness of soft, compressible foundation soils), at approximately STA 13+080, since at this section, the full thickness of the deposit of compressible soils are encountered directly beneath the embankment rather than only at the right toe of slope, as is the case at STA 13+150. The proposed embankment is about 3.8 m high at the right shoulder of the embankment at STA 13+080 and the cohesive deposit is up to 8.9 m thick.

Based on the results of the settlement analysis for the condition of removal of the organic deposits, the total settlement of the foundation soils is estimated to be about 230 mm. The estimated total settlement is comprised of about 10 mm of immediate settlement due to compression of the cohesionless deposits and about 220 mm of primary consolidation of the 8.9 m thick cohesive deposit.

Based on an average coefficient of consolidation (c_v) of about $1.4 \times 10^{-3} \text{ cm}^2/\text{s}$ estimated for the cohesive deposit, the imposed loading conditions for the 3.8 m high embankment after removal of the organic deposits at the critical section, and assuming two-way drainage of the 8.9 m thick cohesive deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in 3.8 years.

The magnitude of secondary consolidation (creep) settlement for the cohesive deposit is expected to be about 70 mm per log-cycle of time for this area corresponding to about 50 mm over a 20-year period following completion of construction.

In addition, the total settlement of the rock fill embankment itself (based on a 3.8 m high embankment) is estimated to be about 25 mm, with about 20 mm expected to occur within six months of construction of the embankment and 5 mm expected to occur over the remaining design life of the embankment.

Since the total post-construction settlement of 295 mm (comprised of 220 mm primary, 50 mm creep and 25 mm long-term rock fill settlement) exceeds the settlement criterion of 200 mm, settlement mitigation measures are required in this high fill area.

6.5.1.1.3 Mitigation of Time Dependent Settlements

The presence of the up to 8.9 m thick compressible clayey silt to clay deposit influences the settlement for the up to about 3.8 m high embankment at STA 13+080. In order to construct the embankment in this section to minimize schedule delays due to settlement, the alternatives presented below can be considered. The alternatives described have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risk/consequences and are summarised in Table A1 in Appendix A. A summary of the settlement results for each alternative is provided in Table 4.

Given the 8.9 m thick cohesive deposits (the bottom of which is up to about 9 m below ground surface) and the associated magnitude of primary and secondary consolidation settlement (about 245 mm) of the foundation soils, it is recommended that preloading for six months be instituted as the preferred settlement mitigation alternative in this area (Refer to Table 5).



Preloading

It is recommended that preloading the high fill area H4 embankment for six months be undertaken as it is the most technically feasible mitigation alternative to minimize the post-construction settlements and subsequent maintenance on the new roadway surface. The main advantage of preloading the embankment is that it reduces the post-construction settlement of the cohesive soils as well as that of the rock fill. The main disadvantage of this alternative is that it requires six months in the schedule as well as an instrumentation and monitoring program.

Although the time period to complete 90 per cent of primary consolidation is 3.8 years, in order to satisfy the settlement performance criterion of 200 mm of post-construction settlement over a 20-year period following completion of construction, a preload period of 6 months is recommended, or approximately until about 40 per cent of the consolidation settlement is complete. After preloading for 6 months, the total post-construction settlement is estimated to be about 200 mm (comprised of 145 mm remaining primary, 50 mm creep settlement and 5 mm long-term rock fill settlement). Since preloading alone will meet the settlement criterion within a reasonable amount of time (6 months), consideration has not been given to surcharging or wick drains which could accelerate the time rate of settlement or reduce the preload. Since 12 months of preloading has been recommended for the culvert to reduce differential settlement (reported under separate covert), we recommend that consideration be given to preloading the entire area 12 months.

Lightweight Fill

An alternative to reduce post-construction settlement of the embankment is to incorporate lightweight fill into the embankment fill mass. The advantage of this alternative is that preloading would not be required. The main disadvantage is the cost of the EPS, which is typically an order of magnitude higher than standard fill materials.

An approximately 1.0 m thick zone of EPS would be required within the embankment mass to reduce the post-construction settlement to an estimated 185 mm (comprised of 110 mm remaining primary, 50 mm creep settlement and 25 mm long-term rock fill settlement). This thickness of EPS would be required for about the 105 m length of High Fill H4 tapering upwards to 0 m west of STA 13+080 and east of STA 13+185 at 5H:1V to prevent differential icing. Specifications to supply and install the EPS should be incorporated into an NSSP in the Contract (an example is included in Appendix G).

Full Sub-Excavation

Full or partial sub-excavation of the clayey silt to clay deposit below the embankment footprint can also be considered to mitigate post-construction settlement at this site. However, given the proximity of the existing Highway 66 (near the critical section STA 13+080), full sub-excavation may not be possible without the need for roadway protection, and in this area, consideration would have to be given to only partially sub-excavating the cohesive deposit. The main advantage of full sub-excavation is the removal/reduction of consolidation settlement of the clay deposit. The main disadvantages are the depth of sub-excavation (up to about 9 m below the original ground or up to about 11 m below the existing highway grade) and possible need for specialized long-stick or drag-line excavating equipment, the possible need for temporary shoring adjacent to the existing highway, the increase in the amount of post-construction settlement of the additional thickness of rock fill and the risk of not removing a sufficient thickness of cohesive soils within the zone of influence of the new embankment.



Although the maximum depth of sub-excavation in this area is up to about 9 m at the right (i.e., south) toe of slope (i.e., closest to the existing embankment) at the critical section, the depth of sub-excavation along the entire high fill section is variable and is typically less than 5 m under the majority of the embankment.

For the case of full sub-excavation, the total settlement of the rock fill embankment itself (based on an approximately 4 m high embankment and 9 m of below grade backfill is estimated to be about 175 mm, with about 140 mm expected to occur within six months of construction of the embankment, 15 mm occurring during the next six months and about 20 mm expected to occur over the remaining design life of the embankment.

In order to eliminate the need for roadway protection adjacent to the existing highway, consideration could be given to a partial sub-excavation utilizing OPSD 203.020 geometry for excavating adjacent to an embankment at 1H:1V from the crest. A 6 m partial sub-excavation would be possible using this technique, however, the removal of the upper 3 m crust and only a small (i.e., 3 m) portion of the soft clayey silt to clay deposit, results in post-construction settlement of 275 mm (140 mm remaining primary, 25 mm creep and 110 mm rock fill) which is more than 200 mm and therefore, a partial sub-excavation is not feasible.

6.5.1.2 Highway 66 Connection – STA 10+000 to 10+125 (High Fill H1)

Between STA 10+000 and STA 10+125, a new embankment up to about 6 m high is required to achieve the proposed vertical highway profile. The topography in this area is relatively flat and sloping down to the south with moderate to dense tree cover transitioning into low-lying wet grassy area adjacent to the existing Highway 66 embankment. Where the connection ties into the existing Highway 66, the embankment is approximately up to 4 m high relative to the existing highway grade, but is up to 5.7 m high above the original ground in the ditch near STA 10+075.

The subsurface conditions in this area generally consist of a 0.1 m to 0.8 m thick layer of topsoil/peat underlain by an up to 12.5 m thick deposit of clayey silt to clay extending to a depth of up to about 12 m below the existing ground surface. The upper about 3 m portion of the cohesive deposit is considered to be the weathered crust. The clayey silt to clay deposit is underlain by deposits of sandy silt to silt, silty sand to sandy gravel and/or bedrock at depths up to 17 m below ground surface. Details of the subsurface conditions for High Fill H1 are presented in Section 4.3 and shown on DrawingsA1 in Appendix A.

The simplified stratigraphy and the associated unit weight, strength, deformation and time-rate consolidation parameters employed for the different soil types encountered in this area are summarized in Table 3 and shown on Figure A9 in Appendix A. The piezometric condition used in the analyses is based on the groundwater level noted during drilling and measured in a piezometer installed in one of the boreholes in this area, as summarized in Section 4.3.

6.5.1.2.1 Stability

The critical section (i.e., the greatest embankment height and maximum thickness of soft, compressible foundation soils) for this area is located at about STA 10+075 along the proposed Highway 66 Connection. The proposed embankment height is about 6 m and the cohesive deposit is up to about 10.3 m thick extending to about 12 m depth below ground surface. The stability analysis for the up to 5.7 m high rock fill embankment carried out at the critical section indicates that after completion of construction (including removal and replacement of the organic deposits), the embankment will have a FoS of less than 1.3 for a deep-seated, global



failure surface that would impact the operation of the highway. Therefore, stability mitigation is required for portions of High Fill H1.

6.5.1.2.2 Settlement

To estimate the magnitude of the expected settlement due to new construction, analysis was carried out at the critical section representative of the subsurface conditions within the high fill area at approximately STA 10+075.

Based on the results of the settlement analysis for the condition of the removal of the organic deposits, the total settlement of the foundation soils is estimated to be about 620 mm. The estimated total settlement is comprised of about 60 mm of immediate settlement due to compression of the cohesionless deposits and about 560 mm of primary consolidation of the 10.3 m thick cohesive deposit.

Based on an average coefficient of consolidation (c_v) of about $1.4 \times 10^{-3} \text{ cm}^2/\text{s}$ estimated for the cohesive deposit, the imposed loading conditions for the approximately 5.7 m high embankment and assuming two-way drainage of the 10.3 m thick cohesive deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 5.1 years.

The magnitude of secondary consolidation (creep) settlement for the cohesive deposit is expected to be about 85 mm per log-cycle of time for this area corresponding to about 50 mm over a 20-year period following completion of construction.

In addition, the total settlement of the rock fill embankment itself (based on a 5.7 m high embankment at the critical section) is estimated to be about 55 mm, with about 45 mm expected to occur within six months of construction of the embankment, 5 mm occurring during the next six months and about 5 mm expected to occur over the remaining design life of the embankment.

Since the total post-construction settlement of 665 mm (comprised of 560 mm primary, 50 mm creep and 55 mm long-term rock fill settlement) exceeds the settlement criterion of 200 mm, settlement mitigation measures are required in this high fill area, including in the area where it ties into the existing highway embankment east of STA 10+075.

6.5.1.2.3 Mitigation of Stability Issues and/or Time Dependent Settlements

The presence of the up to 12.5 m thick compressible clayey silt to clay deposit influences both the stability and the settlement of the embankment. The embankment is up to 5.7 m high at the north ditch location at STA 10+075 and up to about 4 m above the existing highway fill between STA 10+075 and 10+125. In order to construct the embankment to minimize schedule delays due to settlement, the alternatives presented below can be considered. The alternatives described have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risk/consequences and are summarised in Table A2 for the area between STA 10+000 and STA 10+050 and in Table A3 in Appendix A for the area between STA 10+050 and STA 10+125. A summary of the settlement results for each alternative is provided in Table 4.

Between STA 10+000 and about STA 10+050, it is recommended that toe berms and preloading for nine months be instituted as the preferred stability and settlement mitigation alternative in this area. Between about STA 10+050 and 10+125, it is recommended that lightweight fill be used as the preferred stability and settlement mitigation alternative in this area (Refer to Table 5).



6.5.1.2.3.1 Between STA 10+000 and STA 10+050

Preloading with Toe Berms

It is recommended that preloading the High Fill H1 embankment be undertaken, incorporating toe berms into the construction to enhance stability as the preferred and most technically feasible mitigation alternative to achieve stable embankments and to minimize the post-construction settlement and subsequent maintenance on the new roadway surface. The main advantage of preloading the embankment is to reduce the post-construction settlement of the cohesive soils as well as that of the rock fill. The main disadvantage of this alternative is that permanent toe berms are required for stability in a limited area, as well as an instrumentation and monitoring program.

For the 5.7 m high rock fill embankment at this location, 2 m high by 6.5 m wide toe berms will be required along both sides of the embankment to achieve a FoS equal to 1.3, as shown on Figure A11 in Appendix A. Between STA 10+000 and about STA 10+025, the new realigned Highway 66 embankment will be present and therefore the toe berms for the Connection embankment are essentially the realigned embankment itself. The toe berms can taper into the existing highway embankment beyond about STA 10+050. Given that the roadway Connection alignment is on a curve, the length of the toe berms will vary slightly on either side of the embankment being up to about 25 m long on the right (south) side and about 50 m long on the left (north) side (i.e., almost to STA 10+075).

Based on the settlement analysis for the section at STA 10+050 where the clay is 7.6 m thick and the embankment is 5.7 m high above the original ground, the time period to complete 90 per cent of primary consolidation is 2.7 years. In order to satisfy the settlement performance criterion of 200 mm of post-construction settlement over a 20-year period following completion of construction, a preload period of 9 months is recommended, or until approximately 50 per cent of the consolidation settlement is complete. After preloading for 9 months, the total post-construction settlement is estimated to be 200 mm (comprised of 140 mm remaining primary settlement, 50 mm creep settlement and 10 mm long-term rock fill settlement). Since preloading alone will meet the settlement criteria within a reasonable amount of time (10 months), consideration has not been given to other measures that could accelerate the time rate of settlement or reduce the preload period. Also, since 12 months of preloading has been recommended for the culvert and adjacent section H4, we recommend that consideration be given to preloading the entire area 12 months.

Lightweight Fill

An alternative to reduce post-construction settlement of the embankment is to incorporate lightweight fill into the embankment fill mass. The advantages of this alternative are that the embankment would be stable without the need for toe berms and that preloading would not be required. The main disadvantage is the cost of the EPS, which is typically an order of magnitude higher than standard fill materials.

An approximately 2.0 m thick zone of EPS would be required within the embankment mass to reduce the post-construction settlement to an estimated 220 mm (comprised of 150 mm remaining primary, 50 mm creep settlement and 20 mm long-term rock fill settlement) which is just slightly above the settlement criterion. This thickness of EPS would be required throughout this portion of the high fill area.

Full Sub-Excavation

Full sub-excavation of the clayey silt to clay deposit below the embankment footprint can also be considered to mitigate post-construction settlement at this site. However, given the proximity of the existing Highway 66 (near



the critical section at STA 10+075), full sub-excavation may not be possible without roadway protection. The main disadvantages of full sub-excavation are the depth of sub-excavation (up to about 12 m below the ground surface) and possible need for specialized long-stick or drag-line equipment, the possible need for shoring adjacent to the existing highway, the increase in the amount of post-construction settlement of the rock fill and the risk of not removing a sufficient thickness of cohesive soils within the zone of influence of the new embankment.

For the case of full sub-excavation to a depth of 12 m, the total settlement of the rock fill embankment itself (based on an approximately 6 m high embankment) is estimated to be about 315 mm, with about 255 mm expected to occur within six months of construction of the embankment, 30 mm occurring during the next 6 months and about 30 mm expected to occur over the remaining design life of the embankment.

6.5.1.2.3.2 *Between STA 10+050 and STA 10+125*

We understand that due to the requirement to maintain the existing Highway 66 open to traffic during construction of the new realignment embankment, including the Highway 66 Connection, preloading beyond about STA 10+050 will not be possible. Given that the clay deposit is up to about 12 m thick below the existing embankment beyond STA 10+050 and that about 1 m of peat was also encountered below the existing highway, other mitigation alternatives have to be considered to mitigate both stability and settlement for this section of the Connection roadway.

Lightweight Fill

An alternative to reduce post-construction settlement of the embankment is to incorporate lightweight fill into the embankment fill mass. The advantages of this alternative are that the embankment would be stable without the need for toe berms and that preloading would not be required. The main disadvantage is the cost of the EPS, which is typically an order of magnitude higher than standard fill.

An approximately 3.0 m thick zone of EPS would be required within the embankment mass to reduce the post-construction settlement to an estimated 200 mm (comprised of 130 mm remaining primary settlement, 50 mm creep settlement and 20 mm long-term rock fill settlement). This thickness of EPS would be required between about STA 10+075 and STA 10+100. Between STA 10+100 and 10+125, the EPS can be stepped to 2 m. The EPS should taper to 0 m down chainage from STA 10+075 and up chainage from STA 10+125 at 5H:1V.

Figure A14 shows the recommended configuration of the EPS at STA 10+075. EPS is typically provided in blocks 0.5 m or 1 m thick, but other thickness can also be obtained, to accommodate the taper requirements. A minimum of 1 m of conventional granular cover should be provided over the EPS on the side slopes and the EPS should also taper to a thickness of 0.5 m in these areas, as applicable. Appropriate staggered layout, ties and spacers should be used to ensure the EPS block mass acts as a single unit. The entire top and sides of the EPS mass should be covered with a 6 mil (0.15 mm) thick polyethylene sheet for protection from hydrocarbon-based products that may infiltrate the pavement/shoulder structure. It is estimated that a volume of EPS of about 2,200 m³ will be required to satisfy the recommendations for this site. An NSSP should be included in the Contract for Rigid Expanded Polystyrene Embankment Fill; an example is included in Appendix G.



Preloading and Maintenance

We understand that once traffic has been re-routed to the new highway and that a temporary connection at the east end of the alignment may be utilized during construction of the west connection. Therefore, we understand that the west connection must be open to traffic as soon as possible. Given the secondary nature of this roadway, it may be possible to construct the roadway and then allow settlement to occur while traffic is using the roadway, followed by maintenance of the roadway in the future. The estimated settlement of the peat and clay deposits at STA 10+125 expected to occur after construction is up to 595 mm (comprised of 510 mm primary, 60 mm creep and 25 mm short and long term rock fill settlement) which exceeds the settlement criterion of 200 mm for this roadway and hence, maintenance will be required. After the embankment has been in place for about 3.5 years (i.e., “preloaded”), approximately 395 mm of settlement (comprised of 370 mm primary and 25 mm creep settlement) will have occurred.

After this time period, consideration could be given to re-constructing the roadway to the final grade by removing the asphalt (or gravel surface if desired) and adding more base/subbase materials (approximately 0.4 m) and re-instating the pavement. The total post-construction settlement after re-construction of the roadway, including the settlement due to the addition fill is approximately 200 mm (comprised of 140 mm primary and 60 mm creep) which will meet the settlement criterion of 200 mm. It is assumed that this re-construction could be carried out in stages.

It should be noted that differential settlement will also occur both longitudinally along the length of the Hwy 66 Connection and laterally across the embankment. Where the new embankment is constructed partially over the existing highway and partially over the original ground at about STA 10+075, up to about 200 mm of differential settlement will occur across laterally across the embankment. This value of differential settlement exceeds the criterion of 200:1 during the “preload period”. Beyond STA 10+075 to STA 10+125, as well as longitudinally along the length of roadway in this area, the lateral differential settlement meets the differential settlement criterion. After the roadway is re-constructed, the differential settlement values will meet the criteria.

Surcharging and Maintenance

If road re-construction after a “preload period” is not desirable, consideration could be given to overbuilding or “surcharging” the embankment such that road re-construction is not required after the “preload period”. The disadvantage with this method is that it may not be physically possible to surcharge in the mitigation area without toe berms and to maintain a suitable grade in the Connection roadway. Further, the differential settlement or uneven settlement based on the variability in the thickness and/or presence of the peat and clay deposits below the existing highway could mean that some level of maintenance would still be required such as re-paving and/or grade lowering (if it doesn’t settle to final grade) and/or grade raising (if settles below final grade). Approximately 0.4 m of fill above the final grade would be required for a surcharge such that re-grading should not be required. As a minimum surface settlement points should be monitored.

Ground Improvement

Another potential alternative to mitigate stability and settlement issues is to employ specialized ground improvement techniques. Typically, ground improvement consists of physically altering the properties of the soft compressible soils to enhance stability and to reduce post-construction settlement. Ground improvement options that can be considered include but are not limited to: rammed aggregate piers, soil grouting, dry or wet in



situ soil mixing, dynamic compaction and/or other methods. The specific ground improvement option that may be feasible/utilized will depend on the soil properties, construction restrictions and embankment height.

Given the small section of roadway (about 75 m) that would require mitigation, it may not be feasible from a cost perspective to mobilize the required equipment and perform the work. In addition, the proprietary contractor may need additional samples/investigation to allow for design of their works.

The main advantage of using ground improvement to strengthen the cohesive deposit is that it can be used as a stand-alone option for reducing the post-construction settlement of the roadway. The main disadvantages of using a ground improvement technique is the cost of the proprietary design and installation (estimated to be of the same order of magnitude or greater than EPS). The most feasible alternatives for ground improvement at this site are:

- Dry Soil Mixing (DSM) columns; and
- Rammed Aggregate Piers.

Dry Soil Mixing

DSM is typically used in high groundwater conditions and can be carried out in sub-aqueous conditions. The process involves mixing between about 5 per cent and 15 per cent of cement or lime, by mass, into a clayey soil to reinforce the soils a typical approach used to improve the geotechnical characteristics of the clay stratum. The DSM process involves mixing individual “soilcrete” columns or rows of columns and does not produce any cuttings/spoil.

The use of this specialized technique could improve the stiffness of the clay deposits (and peat where present) thereby achieving overall stabilization of the foundation soil mass below the embankment as well as reducing post-construction settlement. The DSM “soilcrete” column type, size and spacing would have to be designed by a specialist ground improvement company. The amount of lime or cementitious material(s) required in each DSM column to provide a sufficient increase in the stiffness of the soil mass is not possible to estimate without bench scale testing and possibly a field test program. However, in general the higher the water content and lower the strength of the existing soils, the larger the amount of mixing additive required and the closer the required spacing of columns to afford a sufficient level of improvement. In addition to the DSM columns, a geogrid/geotextile reinforcement may be required to distribute the embankment loading over the columns. The geogrid reinforcement would have to extend the entire width of the new embankment and could be installed from the existing embankment or native ground, as applicable.

Rammed Aggregate Piers

This method of ground improvement relies on the drilling of large (approx. 0.8 m) diameter auger holes followed by ramming lifts of granular material (such as crushed concrete or crushed stone) progressively up the auger hole to create a pattern of buried “stone columns” which enhance the strength and stiffness of the treated soil mass. We understand that the equipment readily available to perform this type of work is limited to creating pier depths of up to about 7 m. Given this limitation and the depth of clay up to about 15 m below the existing highway grade, this option may not provide enough stabilization of the soil mass to reduce the post-construction settlement of the new embankment.



Full Sub-Excavation

Full sub-excavation of the clayey silt to clay deposit below the existing highway below the entire embankment footprint can also be considered to mitigate post-construction settlement at this site. However, it is likely that roadway protection would be required due to the proximity of private property and due to the depth of the bottom of the clay (up to 15 m below the existing highway grade). The main disadvantages of full sub-excavation are the depth of sub-excavation and possible need for specialized long-stick or drag-line equipment (typically no longer used), the possible need for shoring adjacent to the existing highway, the increase in the amount of post-construction settlement of the rock fill and the risk of not removing a sufficient thickness of cohesive soils within the zone of influence of the new embankment.

For the case of full sub-excavation to a depth of 15 m, the total settlement of the rock fill embankment itself (based on approximately 5.5 m of rock fill above the water level and 13 m below grade rock fill) is estimated to be about 360 mm, with about 300 mm expected to occur within six months of construction of the embankment, 30 mm occurring during the next six months and about 30 mm expected to occur over the remaining design life of the embankment. Therefore, if traffic must travel on the roadway prior to the rock fill settlement, future roadway maintenance may be required).

6.5.2 Highway 66 – STA 13+300 to 13+345 (High Fill H5)

The proposed embankment within this section of the highway will be up to about 7.1 m high relative to the existing ground surface. The topography in this area consists of large rock outcrops to the east and west of the proposed high fill section and the terrain contains moderate tree cover and is crossed by an ATV trail.

The subsurface conditions in this area generally consist of a 0.2 m thick layer of topsoil underlain by a 0.1 m thick deposit of silty sand to gravel in turn underlain by a 5.1 m thick deposit of clayey silt to clay. The clayey silt to clay deposit is underlain by deposits of silt, gravelly sand to gravel and/or bedrock at depths up to about 9 m below ground surface. Details of the subsurface conditions for High Fill H5 are presented in Section 4.4 and shown on Drawing B1 in Appendix B.

The simplified stratigraphy and the associated unit weight, strength, deformation and time-rate consolidation parameters employed for the different soil types encountered in this area are summarized in Table 3. The piezometric condition used in the analyses is based on the groundwater level noted in the piezometer installed into one of the boreholes in this area, as summarized in Section 4.4.

6.5.2.1 Stability

The critical section (i.e., the greatest embankment height and maximum thickness of soft, compressible foundation soils) for this area is located at about STA 13+320. The proposed embankment height is 7.1 m and the cohesive deposit is up to 5.1 m thick. The stability analysis for the up to 7.1 m high rock fill embankment carried out at the critical section indicates that after completion of construction (including removal and replacement of the organic deposits), the embankment will have a FoS greater than 1.3 for a deep-seated, global failure surface that would impact the operation of the highway as shown on Figure B6 in Appendix B. Therefore, stability mitigation is not required for High Fill H5.



6.5.2.2 Settlement

To estimate the magnitude of the expected settlements due to new construction, an analysis was carried out at the critical section representative of the subsurface conditions within the High Fill H5, at approximately STA 13+320.

Based on the results of the settlement analysis assuming that the 0.2 m thick layer of topsoil has been removed, the total settlement of the foundation soils is estimated to be about 105 mm. The estimated total settlement is comprised of about 15 mm of immediate settlement due to compression of the cohesionless deposits and about 90 mm of primary consolidation of the 5.1 m thick cohesive deposit.

Based on an average coefficient of consolidation (c_v) of about $4.3 \times 10^{-3} \text{ cm}^2/\text{s}$ estimated for the cohesive deposit in the overconsolidated range, the imposed loading conditions for the approximately 7.1 m high embankment at the critical section, and assuming two-way drainage of the 5.1 m thick cohesive deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 5 months. Since the estimated preconsolidation pressure will not be exceeded for the given embankment loading, creep settlement will not occur.

In addition, the total settlement of the rock fill embankment itself is estimated to be about 65 mm, with about 50 mm expected to occur within six months of construction of the embankment, 5 mm occurring during the next six months and about 10 mm expected to occur over the remaining design life of the embankment.

Since the total post-construction settlement is estimated to be about 155 mm (comprised of 90 mm primary consolidation and 65 mm rock fill settlement), settlement mitigation measures will not be required to meet the settlement criterion of 200 mm post-construction settlement.

6.5.3 Highway 66 – STA 14+020 to 14+650 (Swamp Crossing H6/H7)

The proposed embankment within this section of the highway realignment will be up to about 3.5 m high relative to the existing ground surface. The topography in this section of roadway realignment consists of a low-lying swampy area with moderate to dense tree cover and open water in places due to the presence of a beaver dam/pond. Multiple recreational trails traverse this area.

The subsurface conditions in this area generally consist of a peat deposit which is up to 3.8 m thick at the west end of the swamp and about 0.2 m at the east of the swamp at about STA 14+400. Beneath the peat, a deposit of very soft clayey silt to clay was encountered extending to a depth of about 21 m below ground surface (up to about 17 m thick) towards the west end of the swamp and to a depth of about 10 m below ground surface (up to 10 m thick) east of about STA 14+300. The cohesive deposit is considered to be layered, or varved, in the upper and lower clayey silt to silty clay sections of the deposit and of higher plasticity in the middle portion of the deposit. The cohesive deposit is underlain by deposits of silt to sand and silt and silty sand to gravel to the refusal depth, encountered up to 29 m below ground surface. East of about STA 14+575, bedrock was encountered at shallow depth. Details of the subsurface conditions for Swamp Crossing H6/H7 are presented in Section 4.4 and shown on Drawings C2 to C4 in Appendix C.

The simplified stratigraphy and associated unit weight, strength, deformation and time-rate consolidation parameters employed for the different soil types encountered in this area are summarized in Table 3. A



summary plot of the pertinent engineering parameters for the cohesive deposits is shown on Figure C24 in Appendix C. The piezometric condition used in the analyses is based on the groundwater level noted during drilling and measured in the piezometer installed in a selected borehole in this area, as summarized in Section 4.5.

6.5.3.1 Stability

The critical section (i.e., the greatest embankment height and maximum thickness of soft, compressible foundation soils) for this area is located at about STA 14+085 along the proposed Highway 66 Connection. The proposed embankment is about 3 m high and the cohesive deposit is up to about 17.1 m thick extending to a depth of up to about 21 m below ground surface. The stability analysis for the 3.0 m high rock fill embankment carried out at the critical section indicates that after completion of construction (including removal and replacement of the organic deposits), the embankment will have a FoS less than 1.3 for a deep-seated, global failure surface that would impact the operation of the highway. Therefore, stability mitigation is required for Swamp Crossing H6/H7 between STA 14+040 and STA 14+570.

Between STA 14+020 and STA 14+040 and between STA 14+570 and STA 14+650, the up to 3.5 m high embankments have a FoS greater than 1.3 and stability mitigation is not required.

6.5.3.2 Settlement

To estimate the magnitude of the expected settlement due to new construction, analysis was carried out at the critical section representative of the subsurface conditions within the swamp crossing area at approximately STA 14+085.

Based on the results of the settlement analysis for the condition of the removal of the organic deposits, the total settlement of the foundation soils is estimated to be about 1,345 mm. The estimated total settlement is comprised of about 65 mm of immediate settlement due to compression of the cohesionless deposits and about 1,280 mm of primary consolidation of the 17.1 m thick cohesive deposit.

Based on an average coefficient of consolidation (c_v) of about $1.4 \times 10^{-3} \text{ cm}^2/\text{s}$ estimated for the cohesive deposit, the imposed loading conditions for the approximately 3.0 m high embankment plus about 3.8 m backfill for replacement of organic deposits, and assuming two-way drainage of the 17.1 m thick cohesive deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 14 years.

The magnitude of secondary consolidation (creep) settlement for the cohesive deposit is expected to be about 170 mm per log-cycle of time for this area corresponding to about 25 mm over a 20-year period following completion of construction.

In addition, the total settlement of the rock fill embankment itself (based on a 3.0 m high embankment plus about 3.8 m of additional rock fill required after removal of the organic deposit) is estimated to be about 65 mm, with about 50 mm expected to occur within six months of construction of the embankment, 5 mm occurring during the next six months and about 10 mm expected to occur over the remaining design life of the embankment.

Since the total post-construction settlement of 1,370 mm (comprised of 1,280 mm primary consolidation, 25 mm creep and 65 mm rock fill settlement) exceeds the settlement criterion of 200 mm, settlement mitigation measures are required in this swamp crossing.



Between STA 14+020 and STA 14+040 and between STA 14+570 and STA 14+650, the post-construction settlement is less than 200 mm and therefore settlement mitigation is not required.

6.5.3.3 Mitigation of Stability Issues and/or Time Dependent Settlements

The presence of the up to 17.1 m thick compressible clayey silt to clay deposit influences both the stability and the settlement of the up to about 3.5 m high embankment. In order to construct the embankment to minimize schedule delays due to settlement, the alternatives presented below can be considered. The alternatives described have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risk/consequences and are summarised in Table C1 in Appendix C for the area between STA 14+040 and STA 14+570. A summary of the settlement results for each alternative is provided in Table 4.

Due to the presence of relatively thin cohesive deposits between about STA 14+020 and STA 14+040 and between about STA 14+570 and 14+650, no foundation mitigation options are required. However, between about STA 14+040 and 14+570, in order to maintain stability and minimize post-construction settlements, wick drains in combination with surcharging and stability berms is ranked as the preferred alternative for this area (Refer to Table 5).

Full Sub-Excavation

Taking into consideration the depth to the bottom of the clay deposits (i.e., up to about 21 m below the existing ground surface), full sub-excavation of the cohesive deposit is not feasible for this area.

Preloading with Toe Berms

If preloading is adopted as the foundation mitigation option for the up to 3.0 m high rock fill embankment at this location, 1.5 m high by 17 m wide toe berms at the level of the existing ground surface along the embankment toes would be required in order to maintain a FoS greater than or equal to 1.3. It should be noted that as a result of the extremely large toe berms, it will not be possible to construct these toe berms within the allowable right-of-way.

Furthermore, in order to meet the settlement performance criterion, it is estimated that a preload period of about 14 years would be required. Considering the duration associated with the preload period at this location, the magnitude and time-rate settlement as well as dissipation of excess pore pressures during and after construction of the preload embankment should be assessed by monitoring this area to confirm the end of the preload period. Monitoring instrumentation would consist of SPs, VWP and SPPs.

Given the extremely long duration of the preload period and the requirement for extremely large toe berms (which may not be constructed due to property restrictions), preloading is not feasible for this area.

Wick Drains

Due to the large settlements expected and long duration to complete primary consolidation associated with the up to about 3.5 m high rock fill embankment, wick drains in combination with a 2 m granular surcharge applied on top of the rock fill embankment is considered the preferred mitigation option at this location.



The complete wick drain design report for the wick drain foundation treatment system between STA 14+040 and 14+570 is provided in Appendix C.

Lightweight Fill

Consideration could be given to constructing the embankment with lightweight fill (i.e., EPS) to mitigate against stability and settlement issues. For an up to about 3.0 m high embankment with 2H:1V side slopes (consisting of a 0.5 m thick granular base, including 300 mm thick levelling pad, a 1.5 m thick EPS core and 1 m of granular protective cover/pavement structure), stability analysis indicates that the embankment will have a FoS greater than 1.3 for deep-seated, global failure surface.

The results of settlement analysis indicate that the total settlement of the foundation soils under the loading imposed by the combined EPS and granular fill is estimated to be about 780 mm. The estimated total settlement is comprised of about 40 mm of immediate settlement due to compression of the non-cohesive deposits and about 740 mm of primary consolidation settlement of the cohesive soils. Secondary consolidation settlement is estimated to be about 25 mm over a 20-year period following completion of construction.

In order to meet the settlement performance criterion of 200 mm over a 20-year period following completion of construction, it is estimated that a minimum preload period of about 10 years would be required. Considering the duration associated with the preload period at this location, the magnitude and time-rate settlement as well as dissipation of excess pore pressures during and after construction of the preload embankment should be assessed by monitoring this area to confirm the end of the preload period. Monitoring instrumentation would consist of settlement plates SPs, VWPs and SPPs.

Considering the very long duration of the preload and high cost associated with EPS installation, this option is not considered cost effective or practical.

6.5.4 Highway 66 – STA 14+840 to 15+060 (High Fill H3)

The proposed embankment within this section of the highway will be up to about 6.5 m high relative to the existing ground surface. The topography in this section of the highway realignment consists of various exposed rock outcrops separated by low-lying areas. The terrain contains moderate tree cover and is crossed by an ATV trail paralleling south of the proposed realignment. The topography in this area is undulating but gradually sloping upwards to the south and east.

The subsurface conditions in this area generally consist of an up to 0.8 m thick layer of topsoil underlain by a deposit of clayey silt up to 1.8 m thick and/or a deposit of silt up to 4.7 m thick. The clayey silt or silt deposits are underlain by deposits of sand and silt to gravelly silty sand to sand and gravel up to about 4.9 m thick, extending up to about 10 m below the existing ground surface, underlain by bedrock. Details of the subsurface conditions for High Fill H3 are presented in Section 4.6 and shown on Drawings D1 and D2 in Appendix D.

The simplified stratigraphy and the associated unit weight, strength, deformation and time-rate consolidation parameters employed for the different soil types encountered in this area are summarized in Table 3. The piezometric condition used in the analyses is based on the groundwater level noted during drilling operations and as measured in the piezometer installed in two of the boreholes in this area, as summarized in Section 4.6.



6.5.4.1 Stability

The critical section (i.e., the maximum thickness of compressible foundation soils) for this area is located at about STA 14+975, although the maximum embankment height is at about STA 15+010. The proposed embankment is about 4.5 m high at STA 14+975 and the cohesive deposit is up to about 1.8 m thick, extending to a depth of about 2 m below ground surface. Where the embankment is highest (i.e., 6.5 m), the cohesive deposit is of negligible thickness. The stability analysis for the up to 4.5 m high rock fill embankment, carried out at the critical section indicates that after completion of construction (including removal and replacement of the organic deposit), the embankment will have a FoS greater than 1.3 for a deep-seated, global failure surface that would impact the operation of the highway as shown on Figure D7 in Appendix D. Therefore, stability mitigation is not required for High Fill H3.

6.5.4.2 Settlement

To estimate the magnitude of the expected settlement due to new construction, an analysis was carried out at the critical section representative of the subsurface conditions within the high fill area, at approximately STA 14+975.

Based on the results of the settlement analysis assuming that the 0.2 m thick layer of topsoil is removed, the total settlement of the foundation soils is estimated to be about 30 mm of primary consolidation settlement of the 1.8 m thick cohesive deposit. Although at this critical section there are no cohesionless deposits, it is estimated that up to about 50 mm of immediate settlement could occur along the alignment due to compression of cohesionless deposits.

Based on an average coefficient of consolidation (c_v) of about $2.8 \times 10^{-2} \text{ cm}^2/\text{s}$ estimated for the cohesive deposit in the overconsolidated range, the imposed loading conditions for the approximately 4.5 m high embankment at the critical section, and assuming two-way drainage of the 1.8 m thick cohesive deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed essentially during construction. Since the estimated preconsolidation pressure will not be exceeded for the given embankment loading, creep settlement will not occur.

In addition, the total settlement of the rock fill embankment itself is estimated to be about 30 mm, with about 20 mm expected to occur within six months of construction of the embankment, 5 mm occurring during the next six months and about 5 mm expected to occur over the remaining design life of the embankment.

Since the cohesive deposit consolidation settlement of 30 mm will essentially occur during construction, the total post-construction settlement is estimated to be 30 mm due to the rock fill. As such, settlement mitigation measures are not required to meet the settlement criterion of 200 mm of post-construction settlement criteria.

6.5.5 Highway 66– STA 15+590 to 15+610 (Deep Cut)

A cut extending up to 8.5 m below ground surface is proposed along the Highway 66 realignment section from about STA 15+590 to 15+610 to achieve the vertical highway profile and connect to the existing Highway 66. The north side of the cut is fully within the bedrock while the south side of the cut is within earth. The current topography in the area of this cut section of the proposed realignment is sloping downward to the east. The existing Highway 66 is bordered by tree covered valley slopes to the east; and the ground cover is generally



comprised of exposed bedrock outcrops to the north and tree cover terrain crossed by various walking/ATV trails.

The subsurface conditions in the deep cut area generally consist of a 0.7 m to 1.8 m thick layer of fill constituting the north portion of the existing embankment, underlain by an up to about 2.1 m thick deposit of clayey silt to silty clay. The cohesive deposit is underlain by an up to 5 m thick deposit of very loose to compact silt and an up to about 5 m thick deposit of silty gravelly sand to sand and gravel which extends to the inferred bedrock surface at a depth of about 11 m. In addition, the cut in this area is below groundwater level which was encountered within 3.2 m below the ground (earth) surface. Pavement boreholes, provided under separate cover, indicate the north side of the proposed cut is within the bedrock. Details of the subsurface conditions for this Deep Cut section are presented in Section 4.7 and shown on Drawing E1 in Appendix E.

6.5.5.1 Stability

The critical section (i.e., the deepest earth cut) for this section of the realignment is located at about STA 15+590. We recommend that the up to 8.5 m deep permanent cut slopes through the overburden be made 2.5H:1V to achieve a FoS greater than 1.3 for a deep-seated, global failure surface that would impact the operation of the highway as shown on Figure E6 in Appendix E. Cut slopes steeper than 2.5H:1V will not be stable. The stability of the permanent cut slopes assume that proper drainage is achieved, consistent with the recommendations for drainage/groundwater control and protection of cut slopes provided in Section 6.7.3. Recommendations for the excavation and stability for the portion of the cut through the bedrock is provided in Section 6.7.6.

We understand that during the Stage 1 detour period, it will not be possible to excavate the earth cut slopes to a configuration of 2.5H:1V because of the need to allow for both the new highway cut to be constructed and for traffic to be maintained along the existing highway. Therefore, either a temporary retaining wall will be required along the approximately 20 m to 30 m length of the earth cut or temporary earth or rock cuts and traffic staging will be required to maintain controlled one-way traffic between Stage 1 and Stage 2 construction until the existing roadway area can be fully excavated for the final 2.5H:1V slope configuration. We recommend that all temporary and final earth cuts including those along the upper portion of the slope be constructed at 2.5H:1V. A temporary retaining wall/excavation support should be constructed in the approximate location of the new highway ditch, to allow the wall to be as low as possible. For this configuration, it is estimated that the approximate maximum height of such a wall is about 5.0 m.

Consideration could be given to construction of a soldier pile and lagging wall provided with adequate drainage to allow for groundwater dissipation and prevention of migration of fines through the wall. For such a retaining wall supporting sloping ground above/behind it, the use of tie-backs or anchors will be required. The global stability of a 5.0 m high wall with a 2.5H:1V slope behind the wall will have a FoS greater than 1.3. For the design of such a temporary tied back retaining wall the following soil parameters may be used to calculate the resulting earth pressures acting on the wall.



Fill Type	Soil Unit Weight	ϕ' ($^{\circ}$)	S_u (kPa)	Coefficients of Static Lateral Earth Pressure	
				At-Rest, K_o	Active, K_a
Granular 'A'	22 kN/m ³	35	-	0.43	0.27
Granular 'B' Type II	21 kN/m ³	35	-	0.43	0.27
Existing Granular Fill	21 kN/m ³	35	-	0.43	0.27
Clayey Silt to Silty Clay	17.5 kN/m ³	30	50	0.50	0.33
Silt	18 kN/m ³	28	-	0.36	0.53
Silty Gravelly Sand to Sand and Gravel	20 kN/m ³	32	-	0.31	0.47

It should be noted that these design recommendations and parameters are for 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. The design must also include water pressure (using a unit weight of water = 9.8 kN/m³) with groundwater levels based on those measured in the boreholes and assuming the wall will be designed such that there will be positive drainage from behind the wall and that the water will drain into the cut. In addition, surcharge pressures should be included in the design as applicable.

6.5.6 Excess Material Management Area

We understand that due to environmental constraints at the site that it is desirable to dispose of excess material from the sub-excavation operations from High Fill/Swamp Crossing H6/H7 in the area immediately to the south of the swamp area. This excess material management (EMM) area extends from approximately STA 14+050 to STA 14+675 and extends from the south edge of the right-of-way (ROW) for an additional 20 m to 70 m south of the ROW, as shown on Drawing F1 in Appendix F.

The topography in this area is a relatively flat, sloping slightly down to the north with moderate to dense tree cover over low-lying swampy terrain. The southern extent of the excess material management (EMM) area is bounded by a snowmobile trail and residential developments to the east and a rock outcrop to the west.

The subsurface conditions in this area generally consist of a peat deposit which is up to 4.3 m thick adjacent to the proposed highway right-of-way at STA 14+135 and generally becomes thinner towards the south and east extents of the excess material management area. Beneath the peat, a deposit of very soft clayey silt to clay was encountered extending to a depth of about 20 m below ground surface (up to about 16 m thick) towards the west end of the swamp and to a depth of about 12 m below ground surface (about 10 m thick) at the east end of the investigated area at about STA 14+550. The cohesive deposit is considered to be layered, or varved, in the upper and lower clayey silt to silty clay portions of the deposit and exhibits higher plasticity in the middle portion of the deposit. The cohesive deposit is generally underlain by deposits of silt to sand and silt, which in turn are underlain by a deposit of silty sand to gravel to the refusal depth, encountered up to about 23 m below ground surface. Details of the subsurface conditions for the EMM area are presented in Section 4.8 and shown on Drawing F1 in Appendix F.

The simplified stratigraphy and the associated unit weight, strength, deformation and time-rate consolidation parameters employed for the peat and clay deposits are summarized in Table 3 and shown on Figures F12 and



F13 in Appendix F. The piezometric condition used in the analyses is based on the groundwater level at ground surface as summarized in Section 4.8.

It is not known what method will be used to dispose of the material. Typically, a rock fill containment berm is constructed surrounding the perimeter of the disposal area and the material is dumped within the interior by constructing access roads as appropriate. At this site, conventional construction techniques may not be appropriate due to the presence of the up to 4.3 m thick deposit of peat and 16 m thick deposit of soft clay across the disposal area. The following sections outline the analysis conducted to determine the restrictions on construction of berms, the sequencing of the placement and thicknesses of excess material, offset distances from the new highway alignment and existing structures (i.e., nearby houses) and other issues.

6.5.6.1 Stability

The critical section (i.e., the maximum thicknesses of peat and clay) for the proposed EMM area is located in the vicinity of Borehole EMM1. The maximum stable height of a rock fill berm between STA 14+050 and STA 14+375 is 1.5 m above the original ground surface (i.e., above the 4.3 m thick peat layer). Between STA 14+375 and STA 14+550, where there is less than 0.5 m of peat present at the ground surface, the berm can be constructed to 2.5 m above the original ground (i.e., above the clay deposit). East of STA 14+550 (i.e., between STA 14+550 and STA 14+675), the berms can be constructed to 4 m above ground surface. In these areas (i.e., between STA 14+050 and STA 14+550), the analysis assumes that a geogrid reinforcement layer will be placed between the subgrade and the rock fill berm as discussed below. The location and height of the berms across the EMM area are shown on Figure F14. The berm side slopes should be constructed at 1.5H:1V. The geogrid is required to achieve a FoS of 2.0 against bearing failure of the underlying peat/clay deposits. The geogrid should extend a minimum of 2 m beyond the base of the rock fill containment berms.

The reinforcement layer should consist of an extruded polypropylene uniaxial geogrid with a tensile stiffness of 1,000 kN/m. The geogrid should be placed such that the strong direction is oriented parallel to the cross section of the berm. A 300 mm thick bedding layer of granular material should be placed over the prepared surface of the peat or clay (i.e., cleared and grubbed) and the geogrid should be placed on the bedding layer, to prevent the rock fill of the berm from immediately migrating into the peat and thus creating interlock between the rock fill and the geogrid to support the stability assumptions. A 0.5 m overlap of adjacent geogrid sections is appropriate. At locations where the berm changes direction (i.e., corners), a 100 mm vertical separation should be maintained between adjacent geogrid sections and filled with bedding material.

Alternatively, sub-excavation of the peat below the rock fill berm and replacement with rock fill could be considered instead of the use of geogrid. In this case, the rock fill berm could still only extend to about 1.5 m (or 2.5 m in the areas noted above) above the original ground to meet a Factor of Safety of 1.3 for stability. Partial peat removal, such as by the “displacement” method, will also result in a berm no higher than 1.5 m above the original ground surface (or 2.5 m in the areas noted above) plus the displaced thickness. However, without a geogrid layer, embankments or berms built directly over the peat may experience bearing failure.

6.5.6.2 Settlement

Settlement of the rock fill containment berms placed on the peat deposit will cause time dependent settlement of both the peat deposit and the underlying clay deposits.



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Based on the results of the settlement analysis, the total settlement of the rock fill, peat and clay soils between STA 14+050 and STA 14+550 is as follows:

Approx. Location	Borehole	Deposit Thickness (m)		Settlement of Rock Fill Berm (mm)*				
		Peat	Clay	Peat		Clay		Rock Fill***
				Primary	Secondary**	Primary	Secondary**	
STA 14+150 (1.5 m high berm + 1 m top up)	EMM1	4.3	13.5	280	90	90	135	15
STA 14+500 (2.5 m high berm + 0.5 m top up)	EMM9	0.5	10.2	75	10	60	100	20

*These values do not include additional settlement from topping up of the berms/fill material which will result in higher values. These values exclude immediate settlement of the peat.

**Secondary settlement per log cycle. It is estimated that about one log cycle will occur from completion of primary settlement and 5 years after the material is first dumped.

*** Settlement in first 6 months after placement.

The settlement values given above are for the areas under the rock fill berm. The material that is to be dumped behind the rock fill berm is anticipated to be mainly the sub-excavated peat from the adjacent area which is fibrous peat, typically very wet. In the areas behind the berm where this material is dumped, the settlement will be less and there will be differential settlement between the berms and the dumped material. The amount of differential settlement will vary depending on the timing of placement, the consistency of the dumped material and the presence of other fill types (i.e., rock fill for access roads, etc.).

It is anticipated that some immediate (i.e., days or weeks) settlement of the peat under the rock fill berms or under the dumped material will take place, likely on the order of about 0.5 m (with or without geogrid present). Once the excess material is placed to the full height (i.e., 1.5 m or 2.5 m), both the peat and clay settlement will occur over a period of months to years.

For the peat deposit, based on an estimated average coefficient of consolidation (c_v) of about $7.0 \times 10^{-3} \text{ cm}^2/\text{s}$, assuming one-way drainage of the 4.3 m thick peat deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 7 months. For the clayey silt to clay deposit, based on an estimated average c_v of about $1.4 \times 10^{-3} \text{ cm}^2/\text{s}$, assuming two-way drainage of the 13.5 m thick cohesive deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 5 years.

East of STA 14+550, the settlement of the underlying soils is estimated to be less than 200 mm and will likely occur as material is placed, and that of the rock fill placement is about 35 mm in the first six months.

6.5.6.3 Construction Constraints for EMM Area

The constraints for construction of berms or placement of excess material in the disposal area are as follows:



- The Contractor shall be responsible for ensuring that the EMM operations do not negatively impact the new construction (i.e., of the realigned highway).
- A minimum buffer zone of 5 m between the outside edge of the rock fill containment berms and any excavation, structures or roadways (this should be shown in the contract drawings as a typical section, see Figure F14 for example).
- The area under the entire EMM footprint should be prepared such that any trees, brush or surface irregularities be removed, especially under the perimeter berms, to provide an approximately clean and even subgrade for placement of the geogrid and berm construction. Preparation of the surface is only required where geogrid will be placed.
- Rock fill containment berms between STA 14+050 and 14+375 may be constructed to a maximum of 1.5 m high above original ground surface. Between STA 14+375 and STA 14+550, rock fill containment berm may be constructed to a maximum of 2.5 m above original ground surface. The berms may be “topped up” provided that they never exceed the maximum indicated height above the original ground surface. Geogrid shall be placed below all containment berms between STA 14+050 and STA 14+675.
- Prior to placing geogrid, a 300 mm thick granular bedding layer should be placed on the prepared surface. The bedding should be nominally compacted by the construction equipment prior to placement of the geogrid. The geogrid shall be placed such that the strong direction is oriented parallel to the cross section of the berm. Adjacent sections of geogrid shall be overlapped a minimum of 0.5 m except where major changes in direction occur (i.e., corners). Where major changes in the berm direction occur, a minimum 100 mm vertical separation shall be maintained between adjacent geogrids and filled with bedding material, or as specified by the manufacturer.
- All materials shall be placed in a manner such that any mud waves generated move away from the new embankment, structures and roadways (i.e., they should move from the sides of the disposal area to then centre).
- The original ground surface shall be determined by contractor survey. The height that the berms can be topped up shall be relative to the survey of the original ground.
- After construction of the new highway embankment commences above ground surface (i.e., after installation of wick drains and monitoring instrumentation), no more material may be placed in the EMM Area within a distance of 25 m from the nearest wick drain on the main alignment or nearest structure or roadway.

An operational constraint should be included in the Contract for this purpose and an example is included in Appendix G.

For purposes of estimating quantities for disposal, it should be assumed that at least an additional 1 m of material in addition to the recommended maximum berm height will be able to be placed across the EMM area. For example, in the area where the berms are recommended to be a maximum of 1.5 m height, it is estimated that 0.5 m of settlement will occur almost immediately and that about another 0.5 m will occur in the next few months while the dumping operation is taking place. In the area where the maximum berm height is 2.5 m, an additional approximately 0.5 m of settlement should be anticipated for estimating quantities. In the area where the maximum berm height is 4 m, no additional settlement should be anticipated when estimating quantities.



6.6 Subgrade Preparation and Embankment Construction

The following sections discuss general aspects of subgrade preparation and embankment construction for the swamp crossing/high fill areas and deep cut section, including: removal of surficial and near surface organic materials; excavation and replacement of soft, cohesive deposits; groundwater control; temporary and permanent earth/rock cut construction; and placement of embankment fills.

A summary of the recommended/preferred foundation mitigation option for each swamp crossing/high fill area and deep cut is presented in Table 5 and includes: recommendations on embankment fill types and side slope profiles; estimated maximum depth of organic deposits encountered; the magnitude of estimated settlement (during and post-construction) for the embankment materials and the foundation soils; recommended width of platform widening as may be required to accommodate future raising of the embankment; temporary and permanent cut slope configuration; and the recommended Ontario Provincial Standard Drawings (OPSD) excavation guideline.

6.6.1 Removal/Preparation of Organic Deposits

Based on the substantial information gathered from the boreholes advanced during the field investigation, the thickness of organic deposits (i.e., peat and topsoil) along the proposed Highway 66 realignment generally ranges from about 0.1 m to 4.0 m as presented in Table 3. After clearing and grubbing the swamp crossing/high fill areas and prior to the placement of any fill for new construction, all organic deposits within the swamp crossing/high fill areas should be stripped from the plan limits of the proposed works, including toe berms, if applicable. The organic materials should be removed using construction procedures in accordance with OPSS 209 (Embankments Over Swamps and Compressible Soils).

In areas where the new embankment is being constructed away from existing roadway embankment, the excavation limits should be consistent with OPSD 203.010 Embankments Over Swamp for New Construction, modified to remove the restrictions on the height of the embankment and the depth of excavation (i.e., Note A).

All excavations must be carried out in accordance with Ontario Regulation 213 Ontario Occupational Health and Safety Act for Construction Projects (as amended). In addition, provisions for traffic control measures should be included in the Contract Documents to maintain the safe operation of Highway 66 and any associated side roads or detours that are in close proximity to the excavation operations.

It is understood that the containment berm within the EMM area will be constructed on surficial organic deposits (i.e., peat). Based on the information gathered from the boreholes advanced during the field investigation for the EMM area, the thickness of organic deposits within the proposed area generally ranges from about 0.5 m to 4.3 m as presented in Table 3. In order to construct the berm, it is first recommended that any trees, brush or surface irregularities be removed to provide an approximately clean and even subgrade for placement of the geogrid and berm construction.

6.6.2 Excavation of Soft Soils

For areas where excavation and replacement of soft deposits is recommended to enhance stability and/or reduce the magnitude of post-construction settlement, conventional excavating equipment (backhoe) will likely be adequate; however, long-stick or drag-line type equipment may be required in areas where the depth of the sub-excavation is greater than about 6 m. The soft deposits should be removed using construction procedures



in accordance with OPSS 209 (Embankments Over Swamps and Compressible Soils) and the excavations carried out in accordance with Ontario Regulation 213 with traffic control measures, if applicable.

6.6.3 Groundwater and Surface Water Control and Protection of Cut Slopes

Excavation within the plan limits of the proposed works will be required to remove organic and/or soft deposits prior to embankment fill placement, which will extend below the water table. Groundwater flow into the excavations will occur due to the presence of relatively permeable deposits and relatively high groundwater levels observed in the low-lying swamp crossing/high fill areas. Unwatering is not required for the excavation and backfilling in the swamp crossing/high fill areas, however, surface water should be directed away from the excavations at all times.

For the new Highway 66 earth cut extending between about STA 15+590 and STA 15+610 and most likely about 10 m beyond these stations (see Section 6.5.5), the up to 8.5 m deep cut will extend through the overburden soils (consisting mainly of fill, clay and silt) and intercept the groundwater table. The stability analyses assume that the entire highway section through this earth cut, will generally be formed initially within the bedrock, which will allow for drainage to occur away from the earth cut section as the earth cut is made progressively deeper. Once overall drainage is provided to the cut slope, long-term drainage of the slope and slope protection will be required to maintain stability. The cut slope should be provided with a 2 m wide bench for the extent of the cut where the slope height/depth is greater than 8 m. In this case, the 2 m wide bench should be incorporated with the cut slope in the vicinity of STA 15+610, such that the uninterrupted slope is less than 8 m high/deep.

In order to achieve adequate long-term drainage of the earth cut slopes and to minimize surficial sloughing, a granular blanket sheeting should be provided on the slope as per OPSS 511 (Rip-Rap, Rock Protection and Granular Sheeting). The granular sheeting material should be as per OPSS PROV. 1004 (Aggregates - Miscellaneous). The granular sheeting should be used on the permanent cut slopes formed within the native soils below the existing ground surface and should be connected to toe drains/interceptor ditches that are adequately drained. The granular sheeting should be a minimum of 600 mm thick. We recommend that a non-woven geotextile (i.e., Terrafix 270R or equivalent) be placed on the native soil slope prior to the granular sheeting being placed to prevent migration of the fines into the gravel sheeting and out through the slope.

6.6.4 Backfilling

In general, it is recommended that rock fill be used for replacement of the sub-excavated material. However, in areas where wick drains are required to mitigate stability and/or post-construction settlements (such as in swamp crossing area H6/H7), it is recommended that granular fill such as OPSS PROV. 1010 (Aggregates) Granular 'B' Type II with 100 per cent passing the 26.5 mm sieve size and not more than 5 per cent passing the 0.075 mm sieve size be used for the replacement of the sub-excavated material. Where sub-excavation of organic deposits and/or soft deposits is being carried out as a foundation mitigation option, it will not likely be possible to place rock fill or granular fill in accordance with SP 206S03 (Earth Excavation, Grading), as discussed in Section 6.7.5. Rock fill and granular fill placed below the water table, should be end dumped as the excavation advances.



6.6.5 Embankment Fill Placement

Placement of rock fill and granular fill above the water table for construction of new embankments should be carried out in accordance with the requirements as outlined in SP 206S03 (Earth Excavation, Rock Excavation Grading). The rock fill should not be dumped in final position, but 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. Side slopes for rock fill embankments should be no steeper than 1.25H:1V. Where surcharge fills or expanded polystyrene (EPS) levelling pads are required, granular fill should be placed in regular lifts with loose thickness not exceeding 300 mm and compacted to at least 95 per cent of the standard Proctor maximum dry density. Side slopes for granular fill should be no steeper than 2H:1V.

The EPS fill should be installed in accordance with the manufacturer’s requirements. It is recommended that a minimum 300 mm thick levelling pad comprised of OPSS PROV. 1010 Granular ‘A’ be placed prior to the installation of the EPS. The EPS should be covered on top and sides by a 0.25 mm (10 mil) thick polyethylene, followed by the placement of a protective cover/pavement structure over the EPS (for a minimum thickness of 1 m including compacted granular materials and asphalt). The EPS on the side slopes of the embankment should be covered with a 1 m thick layer of conventional soil.

6.6.6 Rock Excavation and Blasting

For the deep cut section through the bedrock, the overall slope of the cut face may be formed vertically or near vertically (i.e., about 0.25H:1V). The use of carefully controlled excavation techniques will be required to ensure a neat excavation line and minimize face instabilities and long-term maintenance problems resulting from damage to the rock mass.

The use of controlled blasting techniques is recommended for all bedrock excavation for the final rock faces, and the use of explosives should be in accordance with OPSS 120 (Use of Explosives). It is recommended that all blasting operations be carried out in accordance with SP 299F06 [Rock Excavation (Controlled Blasting)]. The Special Provision should include, but not limited to, the following:

- An outline of the requirements, procedure and extent of a pre-blast survey, including all structures within a radius of about 100 m of the blasting operations, as well as notification to all persons working or living within 500 m of the blasting area.
- A blast proposal by the blasting contractor or their blast consultant detailing the blast methodology, including drill hole patterns, hole size and depths, size of blasts, explosive and initiation product details, as well as all blast control procedures. Blast control procedures would include details on controlling flyrock, temporary road closures, blast signalling and site clearing, as well as procedures to deal with debris clean-up. This submission would be required prior to the commencement of any blasting operations.
- The requirement for trial blasts for all proposed production and wall control blast procedures.
- The requirements for ground and air vibration monitoring during the blasting operations. This would include details on instrumentation, number and location of monitoring sites, blast recording and reporting procedures, and procedures to be followed in the event of excessive vibration readings.



It is recommended that ground vibration levels be limited to 50 mm/s for adjacent services and structures. Continuous monitoring of all blasting operations would dictate when changes to the blast procedures become necessary to meet this limit and how close the blasting can be carried out adjacent to any existing services and structures.

Inspection of all new rock cut faces by a Quality Verification Engineer (QVE) should be carried out soon after blasting in order to assess where scaling/loosened rock removal should be carried out. All loose, unstable rock should be removed from the cut faces in accordance with SP 299F03 [Rock Excavation (Machine Scaling)] before access to the toe area of the slope is permitted.

After rock blasting, where potentially unstable rock blocks or wedges cannot be removed safely or where the removal of such blocks/wedges could undermine the rock mass above, rock bolting may be required. Where rock bolting is necessary, it should be carried out in accordance with SP 299S07 (Rock Bolting).

6.7 Slope Flattening

It is understood that consideration is being given to flattening the proposed embankment rock fill slopes using surplus excavated materials, which is typically considered for all embankments under 4.5 m high as per OPSD 203.010 (Embankments Over Swamp). However, depending on the type of material used, and the timing of placement of the surplus material, slope flattening may adversely affect the long-term performance of the roadway by inducing further post-construction settlement. Considerations with respect to settlement and stability are discussed below. It is assumed that the rock fill embankment side slopes will be constructed at an inclination of 1.25H:1V and that the flattened side slopes will be constructed at 3H:1V. It is also understood that the material used for the slope flattening will consist of granular fill, rock fill or excess earth material, excavated elsewhere or locally.

6.7.1.1 Stability

In general, global stability is enhanced when side slopes are flattened, hence instability of the final embankment slopes is not an issue.

6.7.1.2 Settlement

Post-construction settlement of the embankments will occur as a result of placement of the excess material in the slope flattening areas of the embankments. Therefore, the timing of placement of the additional/excess material load should be considered in determining whether slope flattening should be implemented. Three scenarios are presented below for different stages of placement of the additional slope flattening material as well as the corresponding settlement implications.

- 1) Concurrently with construction of the embankment (in stages where required). This construction method would produce the least amount of post-construction settlement of the roadway embankment.
- 2) After construction of the preload embankment and prior to placement of the final surcharge and/or prior to the full preload period. Any settlement induced prior to construction of the final roadway can be accommodated by grading operations.



- 3) After the preload/surcharge period is complete. This construction method imposes additional loads from the slope flattening material, which will cause immediate and long-term settlement beneath both the embankment side slopes and the roadway and is the least preferred construction method. The magnitude of the settlement could be significant, depending on the embankment geometry and subsoil conditions in the area.

Given the construction scenarios described above it is recommended that slope flattening in the swamp crossing/high fill areas be carried out as follows:

- High Fill H1/H4 (preloading areas): Slope flattening should be carried out concurrent with embankment construction to avoid exceeding post-construction settlement criteria;
- High Fill H5: Slope flattening should be carried out concurrent with embankment construction to avoid exceeding post-construction settlement criteria;
- Swamp Crossing H6/H7: Slope flattening can be carried out after the surcharge period is complete. The amount of slope flattening material above the toe berm to flatten the embankment side slopes will be relatively small resulting in small additional settlements; and
- High Fill H3: Slope flattening can be carried out at any time during embankment construction.

6.8 Embankment Platform Widening

In accordance with the requirements of MTO Northern Region Engineering Directive NRE 98-200, “Northern Region Embankment Design Guidelines”, the construction of the embankments should include an allowance for platform widening (in 0.5 m increments) to accommodate settlements during construction as well as post-construction settlements, so that the minimum standard shoulder widths are maintained if future grade raises on the embankments are required. According to NRE 98-200, the need for future raises in road grade could occur due to settlement/compression of the embankment fill, settlement of the foundation soils and to accommodate future pavement overlays up to 200 mm thick. It is understood that this directive applies to all rock fill embankments as well as for granular fill embankments where widening restrictions are present (such as the presence of a sensitive body of water or due to space/property issues). It is further understood that the minimum required platform widening on major highways (i.e., including Highway 66) over swamp crossings is 2 m per side, unless the preferred mitigation option eliminates uncertainty regarding embankment settlement/performance (i.e., full sub-excavation to bedrock and backfilling with granular material). For non-major highways and roadways (i.e., ramps and side roads) over swamp crossings, the minimum required platform widening is 1 m per side.

The minimum required embankment platform widening (per embankment side) is calculated based on the estimated consolidation settlement of the foundation soils (including creep) and the settlement/compression of the embankment fill plus an additional 200 mm for the future pavement overlay, multiplied by the horizontal component of the side slope of the pavement structure (3H:1V), but cannot be less than the minimum platform widening requirements as described above.

For the proposed embankments in these swamp crossing/high fill areas, the minimum platform widening values are summarized in Table 5. The initial platform widening is required to account for settlement during and post construction. The final platform widening is required to account for post-construction settlement and future overlay.



7.0 CLOSURE

This report was prepared by Mr. Matthew Thibeault, a geotechnical engineering intern and the technical aspects were reviewed by Ms. Sarah E. M. Coyne, P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and a Principal with Golder, conducted an independent quality control review of the report.



Report Signature Page

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ASTM International

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D1587	Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes
ASTM D2573	Standard Test Method for Field Vane Shear Test in Cohesive Soil
ASTM D5778	Standard Test Method for Piezocone Penetration

Commercial Software:

GeoStudio (Version 7.19) by Geo-Slope International Ltd.

Settle^{3D} (Version 2.0) by Rocscience Inc.

Contract Design Estimating and Documentation (CDED):

Special Provision 206S03	Amendment to OPSS 206 – Earth Excavation, Rock Excavation, Grading
Special Provision 299F03	Rock Excavation (Machine Scaling)
Special Provision 299F06	Rock Excavation (Controlled Blasting)
Special Provision 299S07	Rock Bolting

Ministry of Transportation Ontario:

Embankment Settlement Criteria for Design. March 2010.

MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates. September 2010.

Northern Region Engineering Directive NRE 98-200. Northern Region Embankment Design Guidelines. October 1998.

Northeastern Region Geotechnical Section Memorandum. "Use of Mid-Slope Berms for Rockfill Embankments, Northeastern Region" dated February 8, 2005.

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91 Construction Projects

Ontario Regulation 443/09 Amendment to Ontario Regulation 213

Ontario Provincial Standard Drawings:

OPSD 203.010	Embankments Over Swamp – New Construction
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Ontario Provincial Standard Specifications:

OPSS 120	General Specification for Use of Explosives
OPSS 209	Construction Specification for Embankments Over Swamps and Compressible Soils
OPSS 511	Construction Specification for Rip-Rap, Rock Protection and Granular Sheetting
OPSS PROV. 1004	Material Specification for Aggregates – Miscellaneous



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OPSS PROV. 1010

Material Specification for Aggregates – Base, Subbase, Select Subgrade and
Backfill Material

Ontario Water Resources Act:

Ontario Regulation 903/90 Wells



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

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

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



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 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

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

w	water content
w _p	plastic limit
w _l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test 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



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

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



**FOUNDATION REPORT – SWAMP CROSSING/HIGH FILL AREAS
REALIGNMENT OF HIGHWAY 66, AT VIRGINIATOWN, GWP 5091-07-00**

**Table 1: Summary of Swamp Crossings/High Fill Areas, Deep Cut and Excess Material Management Area
Highway 66 Realignment, Virginiatown**

Foundation Investigation Area	Foundation Investigation Area Designation	Maximum Proposed Embankment Height ¹	Boreholes/DCPT	Appendix
Highway 66 STA 13+080 to 13+185 Highway 66 Connection STA 10+000 to 10+100	High Fill H4 High Fill H1	5.7 m	17 Boreholes (H1-4 to H1-7, H1-6a, H1-9, H1-10, H4-1 to H4-6, C1-2, BC1-1 to BC1-3) 5 DCPTs (H1-D1, H1-D3, H1-D4, H4-D1 and H4-D2)	A
Highway 66 STA 13+300 to 13+345	High Fill H5	7.1 m	8 Boreholes (H5-1, H5-2, BC1A-1 to BC1A-6) 2 DCPTs (H5-D1 and H5-D2)	B
Highway 66 STA 14+020 to 14+650	Swamp Crossing H6/H7	3.0 m	49 Boreholes (H6-1 to H6-19, H6-7A, H6-S1 to H6-S5, H7-1 to H7-15, BC2-1 to BC2-3, BC3-1 to BC3-3, BC4-1 to BC4-3) 19 DCPTs (H6-D1 to H6-D11, H7-D1 to H7-D8) 7 CPTs (CPT1 to CPT7)	C
Highway 66 STA 14+840 to 15+060	High Fill H3	6.5 m	23 Boreholes (H3-1 to H3-15, BC5-1 to BC5-3, BC6-1 to BC6-5) 7 DCPTs (H3-D1 to H3-D7)	D
Highway 66 STA 15+590 to 15+610	Deep Cut	8.5 m Cut	5 Boreholes (C1 to C5)	E
Highway 66 STA 14+050 to 14+675 (o/s right)	Excess Material Management Area	n/a	9 Boreholes (EMM1 to EMM9)	F

Prepared By: MT

Reviewed By: SEMC/JMAC

Note: 1. Based on centreline profile of highway alignment and existing ground surface profiles provided by MRC. Embankment height is approximate and is relative to top of peat/original ground surface at the borehole locations.



**FOUNDATION REPORT – SWAMP CROSSING/HIGH FILL AREAS
REALIGNMENT OF HIGHWAY 66, AT VIRGINIATOWN, GWP 5091-07-00**

**Table 2: Summary of Consolidation Test Parameters and Triaxial Test Results
Highway 66 Realignment**

Foundation Investigation Area	Borehole and Sample No.	Depth/Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_{vo}' - \sigma_p'$ (kPa)	OCR	C_c	C_r	e_o	c_v^* (cm ² /s)	c' (cm ² /s)	ϕ' (cm ² /s)
High Fill H1/H4	Borehole H1-6A Sample 4	4.9/299.3	87	130	43	1.5	1.20	0.06	2.16	3.9×10^{-3}	-	-
High Fill H1/H4	Borehole H1-9 Sample 7	6.3/297.6	78	78	0	1.0	0.57	0.04	1.92	8.6×10^{-3}	-	-
Swamp Crossing H6/H7	Borehole H6-7A Sample 1	6.4/302.7	27	62	35	2.3	0.46	0.03	1.50	1.8×10^{-3}	-	-
Swamp Crossing H6/H7	Borehole H6-S1 Sample 1	4.8/304.4	15	67	52	1.8	0.22	0.01	0.89	3.8×10^{-3}	-	-
Swamp Crossing H6/H7	Borehole BC4-1 Sample 4	3.4/301.8	18	125	107	6.9	1.04	0.04	1.87	1.4×10^{-2}	-	-
Swamp Crossing H6/H7	Borehole BC4-1 Sample 5	4.9/300.3	30	92	62	3.1	0.68	0.05	1.74	1.3×10^{-2}	-	-
Swamp Crossing H6/H7	Borehole BC4-1 Sample 6	6.4/298.8	40	134	94	3.4	0.90	0.05	1.62	1.6×10^{-2}	-	-
Swamp Crossing H6/H7	Borehole H6-S1 Sample 2	7.8/301.4	35	60	25	1.7	1.20	0.02	1.80	4.0×10^{-3}	-	-
Swamp Crossing H6/H7	Borehole H6-S1 Sample 4	14.2/ 295.0	76	80	4	1.1	0.82	0.02	1.61	5.7×10^{-3}	-	-
Swamp Crossing H6/H7	Borehole H6-S1 Sample 5	17.1/ 292.1	96	130	34	1.4	0.31	0.01	0.97	5.4×10^{-2}	-	-
Swamp Crossing H6/H7	Borehole H6-S2 Samples 1	4.6 /304.1	-	-	-	-	-	-	-	-	0	35
	Borehole H6-S2 Sample 2	7.9/300.8	-	-	-	-	-	-	-	-	3	24



**FOUNDATION REPORT – SWAMP CROSSING/HIGH FILL AREAS
REALIGNMENT OF HIGHWAY 66, AT VIRGINIATOWN, GWP 5091-07-00**

**Table 2: Summary of Consolidation Test Parameters and Triaxial Test Results
Highway 66 Realignment**

Foundation Investigation Area	Borehole and Sample No.	Depth/Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_{vo}' - \sigma_p'$ (kPa)	OCR	C_c	C_r	e_o	c_v^* (cm ² /s)	c' (cm ² /s)	ϕ' (cm ² /s)
Swamp Crossing H6/H7	Borehole H6-S4 Sample 1	3.0/ 303.7	-	-	-	-	-	-	-	-	0	35
	Borehole H6-S5 Sample 1	1.8/303.4	-	-	-	-	-	-	-	-	0	35
Swamp Crossing H6/H7	Borehole H6-S2 Sample 3	10.8/297.9	-	-	-	-	-	-	-	-	3	24
	Borehole H6-S2 Sample 4	14.0/294.7	-	-	-	-	-	-	-	-	3	24
EMM Area	Borehole EMM5 Sample 2 (peat)	1.8/308.8	4	48	44	12	2.4	0.5	5.8	7.0×10^{-3}	-	-
EMM Area	Borehole EMM1 Sample 5	6.4/ 304.3	20	110	90	5.5	0.24	0.007	0.89	1.4×10^{-2}	-	-

Note: *For stress range between approximately in situ effective overburden stress and final stress as indicated in Section 4 for the individual tests.

where: σ_{vo}' is the effective overburden stress in kPa
 σ_p' is the preconsolidation stress in kPa
OCR is the overconsolidation ratio
 C_c is the compression index
 C_r is the recompression index
 e_o is the initial void ratio
 c_v is the coefficient of consolidation in cm²/s
 c' is the effective cohesion in kPa
 ϕ' is the effective friction angle (degrees)

Prepared By: MT

Reviewed By: SEMC



Table 3: Summary of Foundation Engineering Parameters
Highway 66 Realignment

Foundation Investigation Area	Stratigraphic Unit	Top Elevation (m)	Thickness (m)	γ' (kN/m ³)	ϕ' (°)	c' (kPa)	S_u (kPa)	$\sigma_{p'}$ (kPa)	e_o	C_c	C_r	E' (MPa)	c_v (cm ² /s)
Highway 66 STA 13+080 to 13+185 and Connection STA 10+000 to 10+125 (High Fill H1/H4)	Sandy Gravel (Fill)	304.2	1.4	21	35	-	-	-	-	-	-	-	-
	Peat/Topsoil	305.9 to 303.5	0.1 to 0.8	12	27	1	-	-	-	-	-	-	-
	Clayey Silt (Crust)	305.9 to 302.8	2.0 to 3.0	17.5	-	-	65-20	296-91	1.2-1.8	0.6-1.0	0.03-0.05	-	1.4×10^{-3}
	Clayey Silt to Clay (Lower)	303.9 to 299.8	0.1 to 9.5	16.5	-	-	20	91	1.8-0.8	1.0-0.6	0.05-0.03	-	1.4×10^{-3}
	Silt to Sandy Silt	302.4 to 292.5	0.3 to 4.6	18	28	-	-	-	-	-	-	6	-
	Silty Sand to Sandy Gravel	303.4 to 287.9	0.2 to 4.0	20	32	-	-	-	-	-	-	35	-
Highway 66 STA 13+300 to 13+345 (High Fill H5)	Topsoil	305.7 to 303.1	0.1 to 0.2	12	27	1	-	-	-	-	-	-	-
	Sandy Silt to Gravel	305.1 to 304.2	0.6 to 0.7	20	30	-	-	-	-	-	-	12	-
	Clay to Clayey Silt	305.0 to 302.9	1.0 to 5.1	17.5	-	-	45	200	0.8	0.5	0.03	-	4.3×10^{-3}
	Silt	302.0 to 300.6	1.9 to 2.2	18	28	-	-	-	-	-	-	6	-
	Gravelly Sand to Gravel	301.4 to 297.8	0.6 to 3.9	20	32	-	-	-	-	-	-	35	-
Highway 66 STA 14+020 to 14+650 (Swamp Crossing H6/H7)	Peat/Topsoil	307.8 to 305.1	0.2 to 4.0	12	27	1	-	-	-	-	-	-	-
	Silty Sand to Sand and Silt	309.0 to 306.0	0.4 to 2.3	20	30	-	-	-	-	-	-	12	-
	Clayey Silt to Silty Clay (Upper)	307.8 to 303.7	0.9 to 8.7	17	-	-	14	64	0.8 - 1.7	0.6 - 1.0	0.03 - 0.05	-	1.4×10^{-3}
	Clayey Silt to Clay (Lower)	299.0	0.7 to 11.1	16.5	-	-	14-35	64 to 159	1.7 - 1.0	1.0 - 0.6	0.05 - 0.03	-	1.4×10^{-3}
	Silt to Sand and Silt	307.0 to 287.9	0.2 to 8.7	18	28	-	-	-	-	-	-	3	-
	Silty Sand to Gravel	306.5 to 281.0	0.6 to 8.0	20	32	-	-	-	-	-	-	35	-
Highway 66 STA 14+840 to 15+060 (High Fill H3)	Topsoil	311.5 to 305.3	0.1 to 0.8	12	27	1	-	-	-	-	-	-	-
	Clayey Silt	309.7 to 306.0	0.3 to 1.8	17.5	-	-	100	450	1.0	0.5	0.03	-	2.8×10^{-2}
	Silt	311.5 to 304.5	0.5 to 4.0	18	28	-	-	-	-	-	-	6	-
	Sand and Silt to Silty Sand	308.5 to 302.6	0.4 to 2.5	20	30	-	-	-	-	-	-	12	-
	Sand and Gravel to Gravel	310.2 to 300.1	0.2 to 4.9	20	32	-	-	-	-	-	-	35	-
Highway 66 STA 15+590 to 15+610 (Deep Cut)	Silty Sand to Gravelly Sand (Fill)	317.5 to 316.6	0.7 to 1.8	21	35	-	-	-	-	-	-	-	-
	Clayey Silt to Silty Clay	316.4 to 315.7	1.0 to 2.1	17.5	30	-	50	-	-	-	-	-	-
	Silt	314.6 to 313.6	4.0 to 5.0	18	28	-	-	-	-	-	-	-	-
	Silty Gravelly Sand to Sand and Gravel	310.6 to 308.8	0.6 to 4.6	20	32	-	-	-	-	-	-	-	-
Highway 66 (Excess Material Management Area)	Peat	311.3 to 308.0	0.5 to 4.3	12	27	1	10	48	5.8	2.4	0.5	-	7.0×10^{-3}
	Sandy Silt	307.5	1.0	18	28	-	-	-	-	-	-	6	-
	Clayey Silt to Silty Clay (Upper)	309.7 to 305.0	3.0 to 7.7	17	-	-	14	64	0.8 - 1.7	0.6 - 1.0	0.03 - 0.05	-	1.4×10^{-3}
	Clayey Silt to Clay (Lower)	302.0	1.0 to 12.4	16.5	-	-	14-35	64 to 159	1.7 - 1.0	1.0 - 0.6	0.05 - 0.03	-	1.4×10^{-3}
	Silt	300.1 to 292.8	1.5 to 4.6	18	28	-	-	-	-	-	-	3	-
	Silty Sand to Sand and Gravel	300.2 to 289.5	0.3 to 5.4	20	32	-	-	-	-	-	-	35	-

Note: Additional details of Foundation Engineering Parameters for cohesive deposits (i.e., clayey silt / silty clay / clay) in High Fill H1 / H4 and Swamp Crossing H6 / H7 are provided in Figures A9 and C15 in Appendices A and C, respectively.

Prepared By: MT

Reviewed By: SEMC



**FOUNDATION REPORT – SWAMP CROSSING/HIGH FILL AREAS
REALIGNMENT OF HIGHWAY 66, AT VIRGINIATOWN, GWP 5091-07-00**

**Table 4: Summary of Settlement Analyses
Highway 66 Realignment**

Foundation Investigation Area	Settlement (mm) / Delay Time ^{1,2} (days)	Estimated Post-Construction Settlement Over 20-Year Period at the Critical Section (mm)					Preferred Foundation Mitigation Option
		No Foundation Mitigation	Preloading	Lightweight Fill	Staged Construction with Wick Drains	Full Sub-Excavation ³	
Highway 66 STA 13+080 to 13+185 (High Fill H4)	δ_{primary}	220	145	110	n/a	0	Preloading for 6 months.
	$\delta_{\text{secondary}}$	50	50	50		0	
	$\delta_{\text{rock fill}}$	25	5	25		175	
	δ_{total}	295	200	185		175	
	t_{delay}	3.8 years	6 months	No delay		May not be constructable	
Highway 66 Connector STA 10+000 to 10+050 (High Fill H1)	δ_{primary}	335	140	150	n/a	0	Preloading for 10 months with 6.5 m wide toe berms.
	$\delta_{\text{secondary}}$	50	50	50		0	
	$\delta_{\text{rock fill}}$	50	10	20		315	
	δ_{total}	435	200	220		315	
	t_{delay}	2.7 years	10 months	No delay		May not be constructable	
Highway 66 Connector STA 10+050 to 10+125 (High Fill H1)	δ_{primary}	560	n/f	130	n/f	n/f	Lightweight fill (2 m to 3 m EPS).
	$\delta_{\text{secondary}}$	50		50			
	$\delta_{\text{rock fill}}$	55		20			
	δ_{total}	665		200			
	t_{delay}	5.1 years		No delay			
Highway 66 STA 13+300 to 13+345 (High Fill H5)	δ_{primary}	90	-	-	-	-	No mitigation required.
	$\delta_{\text{secondary}}$	0	-	-	-	-	
	$\delta_{\text{rock fill}}$	65	-	-	-	-	
	δ_{total}	155	-	-	-	-	
	t_{delay}	No delay	-	-	-	-	
Highway 66 STA 14+020 to 14+650 (Swamp Crossing H6 / H7)	δ_{primary}	1,280	n/f	n/f	170	n/f	Staged construction and preloading with wick drains and 5 m wide toe berms.
	$\delta_{\text{secondary}}$	25			~ 0		
	$\delta_{\text{rock fill}}$	65			5		
	δ_{total}	1,370			175		
	t_{delay}	14 years			220 days		
Highway 66 STA 14+840 to 15+060 (High Fill H3)	δ_{primary}	0	-	-	-	-	No mitigation required.
	$\delta_{\text{secondary}}$	0	-	-	-	-	
	$\delta_{\text{rock fill}}$	30	-	-	-	-	
	δ_{total}	30	-	-	-	-	
	t_{delay}	No delay	-	-	-	-	

- Notes: 1. Depths do not include any ponded water that may be present over the peat.
2. In all swamp crossing/high fill areas, removal of organic deposits is required prior to embankment construction.
3. Full sub-excavation implies complete removal of soft, compressible cohesive deposits.

Prepared By: MT

Reviewed By: SEMC



**FOUNDATION REPORT – SWAMP CROSSING/HIGH FILL AREAS
REALIGNMENT OF HIGHWAY 66, AT VIRGINIATOWN, GWP 5091-07-00**

**Table 5: Summary of Preferred Foundation Mitigation Options
Highway 66 Realignment**

Foundation Investigation Area	Foundation Design Issue (Maximum Height of Fill or Depth of Cut)	Topography and Surface Conditions	Recommended Embankment Side Slope and Platform Widening Per Side	Maximum Thickness of Organics Encountered Along Alignment ¹	Preferred Stability/Settlement Mitigation Option ^{2,3}	Estimated Settlement (δ) During Construction at the Critical Section	Estimated Post-Construction Settlement (δ) Over 20-Year Period at the Critical Section	Swamp Excavation / Organic Removal Specification
Highway 66 STA 13+080 to 13+185 (High Fill H4)	High Fill shoulder (4.3 m at right crest of embankment)	Wet low-lying grassy terrain transitions to tree covered terrain as it slopes upward from west to east and south to north.	1.25H:1V (Rock Fill) 2.0 m initial platform 1.5 m final platform	Peat/topsoil up to about 0.8 m (typically less than 0.3 m).	Preloading for 6 to 12 months.	$\delta_{\text{Immediate}} = 10 \text{ mm}$ $\delta_{\text{Primary}} = 75 \text{ mm}$ $\delta_{\text{Rock Fill}} = 20 \text{ mm}$	$\delta_{\text{Primary}} = 145 \text{ mm}$ $\delta_{\text{Secondary}} = 55 \text{ mm}$ $\delta_{\text{Rock Fill}} = 5 \text{ mm}$	OPSD 203.010
Highway 66 Connection STA 10+000 to 10+050 (High Fill H1 adjacent to new alignment)	High Fill (5.7 m)	Tree covered terrain transitions to wet low-lying grassy terrain as it slopes downward from north to south.	1.25H:1V (Rock Fill) 2.5 m initial platform 1.5 m final platform	Topsoil up to about 0.2 m.	Preloading for 10 to 12 months to end of preload section (approx. STA 10+050) and 6.5 m wide toe berms.	$\delta_{\text{Immediate}} = 60 \text{ mm}$ $\delta_{\text{Primary}} = 195 \text{ mm}$ $\delta_{\text{Rock Fill}} = 40 \text{ mm}$	$\delta_{\text{Primary}} = 140 \text{ mm}$ $\delta_{\text{Secondary}} = 50 \text{ mm}$ $\delta_{\text{Rock Fill}} = 10 \text{ mm}$	OPSD 203.010
Highway 66 Connection STA 10+050 to 10+125 (High Fill H1 adjacent to existing highway)	High Fill (5.7 m adjacent to existing highway, 4 m over existing highway)	Tree covered terrain transitions to wet low-lying grassy terrain as it slopes downward from north to south and over existing highway.	1.25H:1V (Rock Fill) 1.5 m initial platform 1.5 m final platform	Topsoil up to about 0.2 m.	Lightweight fill from STA 10+050 to STA 10+125 (2 m to 3 m thick).	$\delta_{\text{Immediate}} = 60 \text{ mm}$ $\delta_{\text{Primary}} = 0 \text{ mm}$ $\delta_{\text{Rock Fill}} = 0 \text{ mm}$	$\delta_{\text{Primary}} = 130 \text{ mm}$ $\delta_{\text{Secondary}} = 50 \text{ mm}$ $\delta_{\text{Rock Fill}} = 20 \text{ mm}$	OPSD 203.010
Highway 66 STA 13+300 to 13+345 (High Fill H5)	High Fill (7.1 m)	Rock outcrops are present to the east and west of the proposed high fill section. The terrain along the proposed realignment contains moderate tree cover and is crossed by an ATV trail.	1.25H:1V (Rock Fill) 1.0 m initial platform 1.0 m final platform	Topsoil up to about 0.2 m.	No mitigation required.	$\delta_{\text{Immediate}} = 15 \text{ mm}$ $\delta_{\text{Primary}} = 0 \text{ mm}$ $\delta_{\text{Rock Fill}} = 0 \text{ mm}$	$\delta_{\text{Primary}} = 90 \text{ mm}$ $\delta_{\text{Secondary}} = 0 \text{ mm}$ $\delta_{\text{Rock Fill}} = 65 \text{ mm}$	OPSD 203.010



**FOUNDATION REPORT – SWAMP CROSSING/HIGH FILL AREAS
REALIGNMENT OF HIGHWAY 66, AT VIRGINIATOWN, GWP 5091-07-00**

**Table 5: Summary of Preferred Foundation Mitigation Options
Highway 66 Realignment**

Foundation Investigation Area	Foundation Design Issue (Maximum Height of Fill or Depth of Cut)	Topography and Surface Conditions	Recommended Embankment Side Slope and Platform Widening Per Side	Maximum Thickness of Organics Encountered Along Alignment ¹	Preferred Stability/Settlement Mitigation Option ^{2,3}	Estimated Settlement (δ) During Construction at the Critical Section	Estimated Post-Construction Settlement (δ) Over 20-Year Period at the Critical Section	Swamp Excavation / Organic Removal Specification
Highway 66 STA 14+020 to 14+650 (Swamp Crossing H6/H7)	Swamp Crossing (3.0 m)	Relatively flat swamp area located with moderate tree cover and open water in places. Multiple ATV trails are present in this area.	1.25H:1V (Rock Fill embankment) 2H:1V (for Granular 'B' Type II backfill of sub-excavated organics for wick drains) 3.0 m initial platform 1.5 m final platform	Peat up to about 4 m west of STA 14+400. Topsoil up to 0.2 m east of STA 14+400.	Staged construction and surcharging, wick drains and toe berms.	$\delta_{\text{Immediate}} = 85 \text{ mm}$ $\delta_{\text{Primary}} = 1,490 \text{ mm}$ $\delta_{\text{Rock Fill}} = 20 \text{ mm}$	$\delta_{\text{Primary}} = 170 \text{ mm}$ $\delta_{\text{Secondary}} = 0 \text{ mm}$ $\delta_{\text{Rock Fill}} = 5 \text{ mm}$	OPSD 203.010
Highway 66 STA 14+840 to 15+060 (High Fill H3)	High Fill (6.5 m)	Rock outcrops separated by low-lying areas throughout this high fill section. Moderate tree cover present with an existing ATV paralleling to the south of the proposed realignment. In general the topography in this area is sloping upwards towards the south and east.	1.25H:1V (Rock Fill) 1.0 m initial platform 1.0 m final platform	Topsoil up to about 0.8 m.	No mitigation required.	$\delta_{\text{Immediate}} = 50 \text{ mm}$ $\delta_{\text{Primary}} = 30 \text{ mm}$ $\delta_{\text{Rock Fill}} = 0 \text{ mm}$	$\delta_{\text{Primary}} = 0 \text{ mm}$ $\delta_{\text{Secondary}} = 0 \text{ mm}$ $\delta_{\text{Rock Fill}} = 30 \text{ mm}$	OPSD 203.010



**FOUNDATION REPORT – SWAMP CROSSING/HIGH FILL AREAS
REALIGNMENT OF HIGHWAY 66, AT VIRGINIATOWN, GWP 5091-07-00**

**Table 5: Summary of Preferred Foundation Mitigation Options
Highway 66 Realignment**

Foundation Investigation Area	Foundation Design Issue (Maximum Height of Fill or Depth of Cut)	Topography and Surface Conditions	Recommended Embankment Side Slope <i>and</i> Platform Widening Per Side	Maximum Thickness of Organics Encountered Along Alignment ¹	Preferred Stability/Settlement Mitigation Option ^{2,3}	Estimated Settlement (δ) During Construction at the Critical Section	Estimated Post-Construction Settlement (δ) Over 20-Year Period at the Critical Section	Swamp Excavation / Organic Removal Specification
Highway 66 STA 15+590 to 15+610 (Deep Cut)	Deep Cut (8.5 m)	Sloping downward to the east. The existing Highway 66 is bordered by tree covered valley slopes to the east; and the ground cover is generally comprised of exposed bedrock outcrops with tree cover to the north and south.	2.5H:1V (earth cut)	Peat / topsoil up to about 0.4 m.	n/a	n/a	n/a	n/a

Notes:

1. Depths do not include any ponded water that may be present over the peat.
2. In all swamp crossing/high fill areas, removal of organic deposits is required prior to embankment construction.
3. Full sub excavation implies complete removal of soft, compressible cohesive deposits.

Prepared By: MT

Reviewed By: SEMC

N 5335000

N 5334500

N 5334000

N 5333500

E 409500

E 410000

E 410500

E 411000

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

CONT No.
GWP No. 5091-07-00

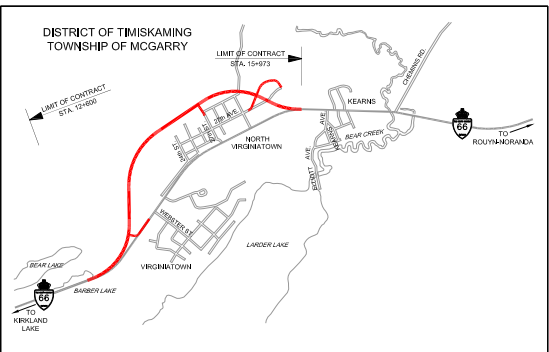


HIGHWAY 66
Swamp Crossings / High Fill /
Excess Material Management Areas,
Culverts and Foundation Deep Cut
INDEX PLAN

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



KEY PLAN
SCALE
700 0 700 m



LEGEND

- H1** High Fill/Swamp Crossing Area
- BC1** Culvert
- C** Deep Cut
- EMM** Excess Material Management Area

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.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by MMM, drawing file nos. H3211009D16 ROLL PLAN-ULTIMATE and PDR.dwg, received DEC 3, 2012. Keyplan drawing file nos. H3211009G02 received JAN 24, 2013.

SCALE
80 0 80 160 m

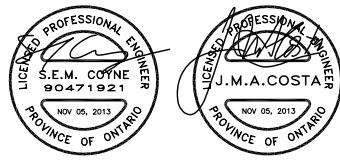
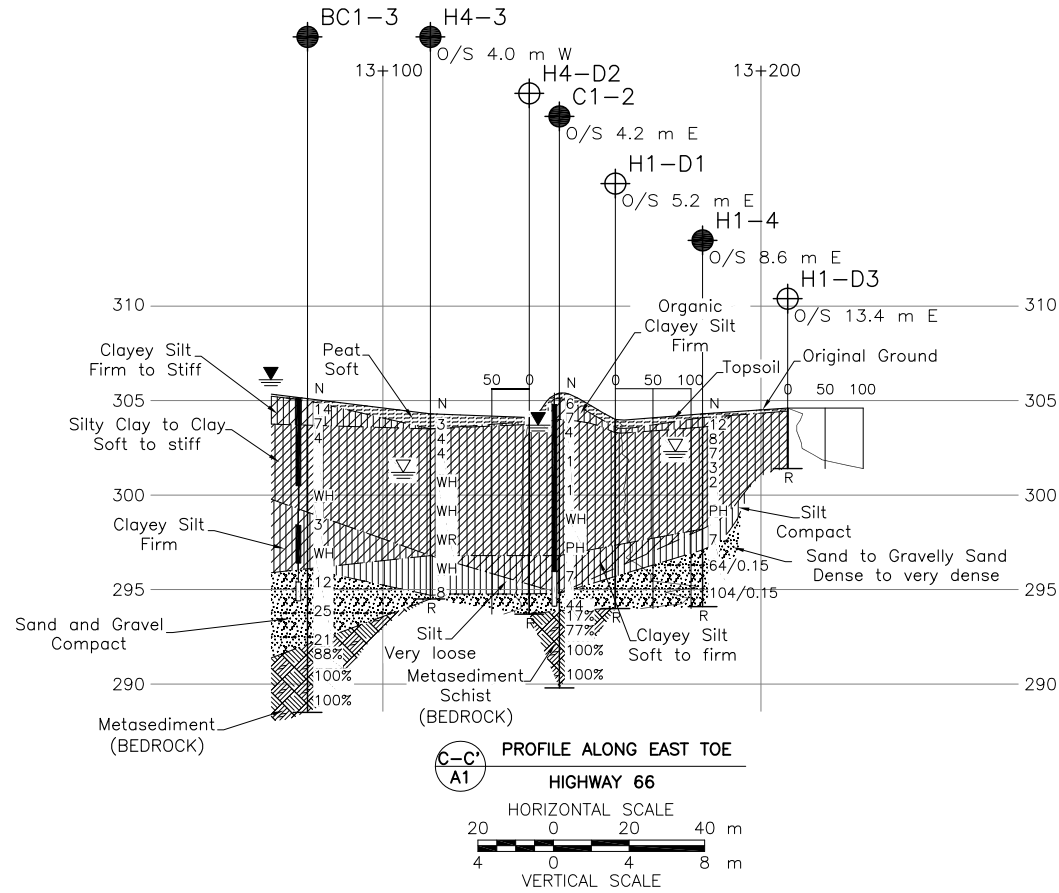
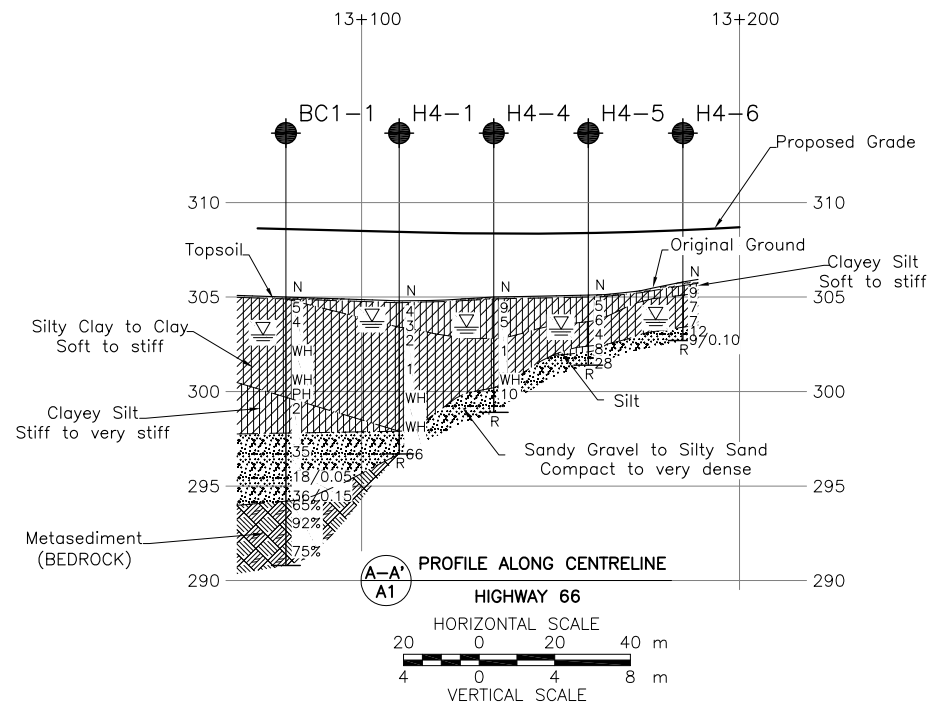
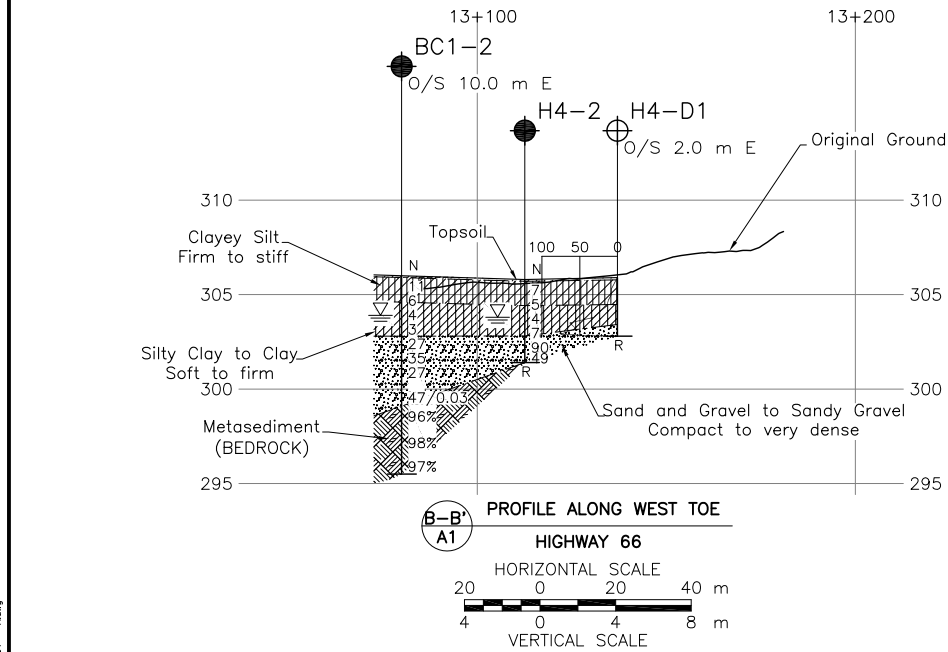
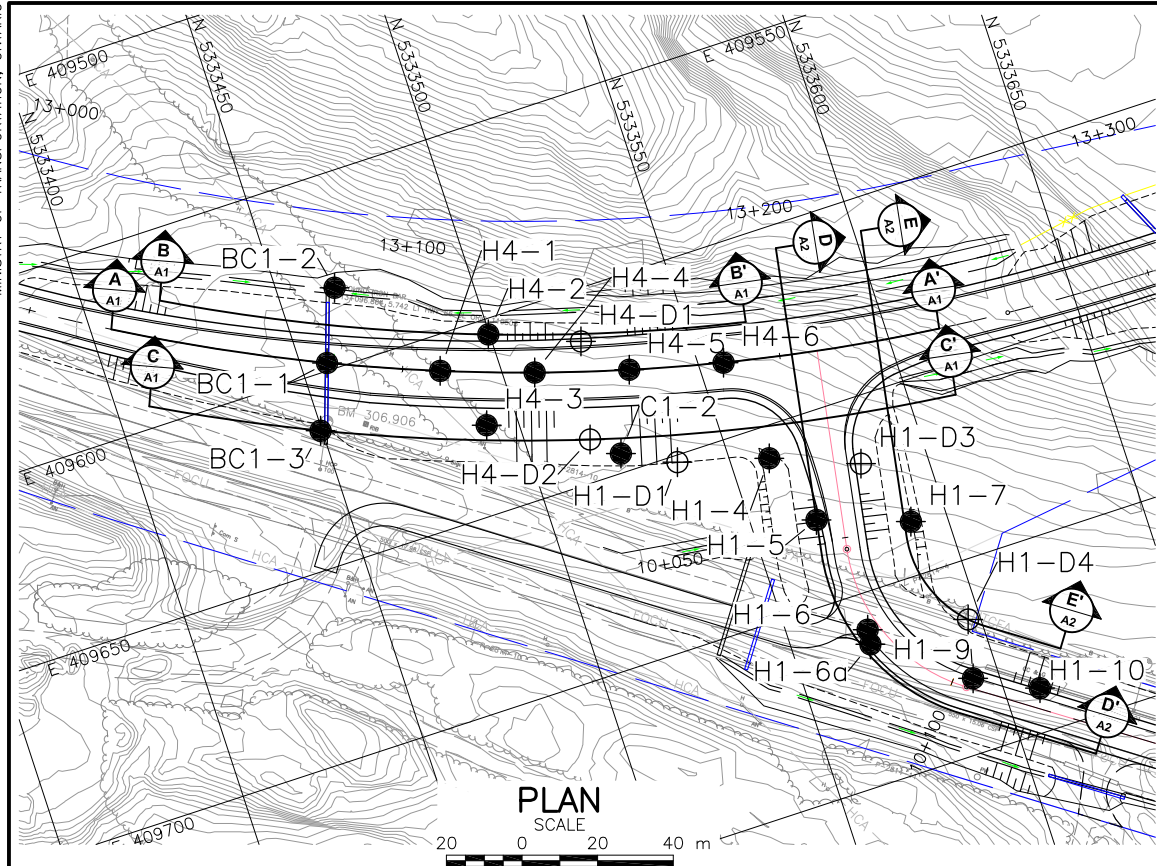
NO.	DATE	BY	REVISION
Geocres No.			
HWY. 66		PROJECT NO. 10-1191-0044	DIST.
SUBM'D. MT	CHKD.	DATE: DEC 2013	SITE:
DRAWN: JJJ	CHKD. SEMC	APPD. JMAC	DWG. 1



APPENDIX A

Highway 66 – STA 13+080 to 13+185 (High Fill H4)

Highway 66 Connection – STA 10+000 to 10+125 (High Fill H1)



REFERENCE
 Base plans provided in digital format by MMM, drawing file nos. H3211009D16 ROLL PLAN-ULTIMATE and PDR.dwg, received DEC 3, 2012. Keyplan drawing file nos. H3211009G02 received JAN 24, 2013.

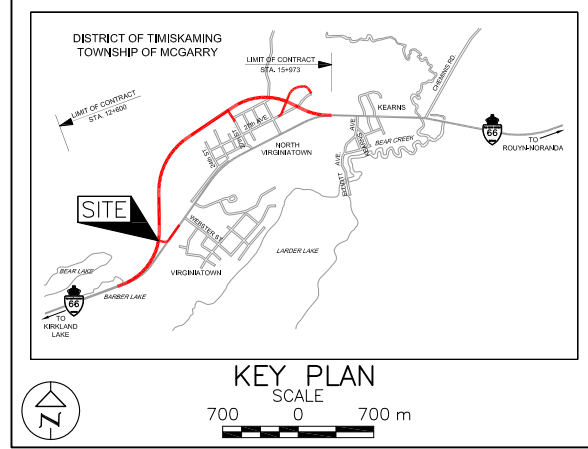
NOTES
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METRIC
 DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 5091-07-00

HIGHWAY 66
 HWY 66 - STA 13+080 TO 13+185 AND
 HWY 66 CONNECTION - STA 10+100 TO 10+125
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



- LEGEND**
- Borehole
 - ⊕ Dynamic Cone Penetration Test
 - 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
 - ▽ WL in piezometer, measured on NOV 17, 2012

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
BC1-1	305.0	5333456.8	409598.0
BC1-2	306.0	5333464.9	409579.7
BC1-3	305.1	5333449.6	409614.5
C1-2	305.4	5333523.2	409645.0
H1-4	304.3	5333560.1	409658.4
H1-5	303.5	5333566.9	409677.8
H1-6	304.2	5333570.7	409709.6
H1-6a	304.2	5333570.2	409713.5
H1-7	303.8	5333590.5	409686.0
H1-9	303.9	5333593.2	409730.5
H1-10	303.6	5333609.2	409738.5
H1-D1	304.0	5333536.7	409651.8
H1-D3	304.6	5333582.7	409667.3
H1-D4	302.5	5333596.8	409715.3
H4-1	304.8	5333484.6	409609.3
H4-2	305.8	5333499.7	409604.1
H4-3	304.3	5333491.8	409626.8
H4-4	305.0	5333508.2	409617.5
H4-5	305.1	5333532.2	409624.6
H4-6	305.8	5333556.5	409630.6
H4-D1	305.9	5333522.4	409613.5
H4-D2	304.1	5333516.6	409638.9

NO.	DATE	BY	REVISION
Geocres No. 32D-17			
HWY. 66	PROJECT NO. 10-1191-0044		DIST.
SUBM'D. MT	CHKD.	DATE: DEC 2013	SITE:
DRAWN: JJJ	CHKD. SEMC	APPD. JMAC	DWG. A1

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

CONT No.
GWP No. 5091-07-00

HIGHWAY 66
HWY 66 - STA 13+080 TO 13+185 AND
HWY 66 CONNECTION - STA 10+100 TO 10+125
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

LEGEND

- Borehole
- ⊕ Dynamic Cone Penetration Test
- 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
H1-4	304.3	5333560.1	409658.4
H1-5	303.5	5333566.9	409677.8
H1-6	304.2	5333570.7	409709.6
H1-7	303.8	5333590.5	409686.0
H1-9	303.9	5333593.2	409730.5
H1-10	303.6	5333609.2	409738.5
H1-D3	304.6	5333582.7	409667.3
H1-D4	302.5	5333596.8	409715.3

NOTES

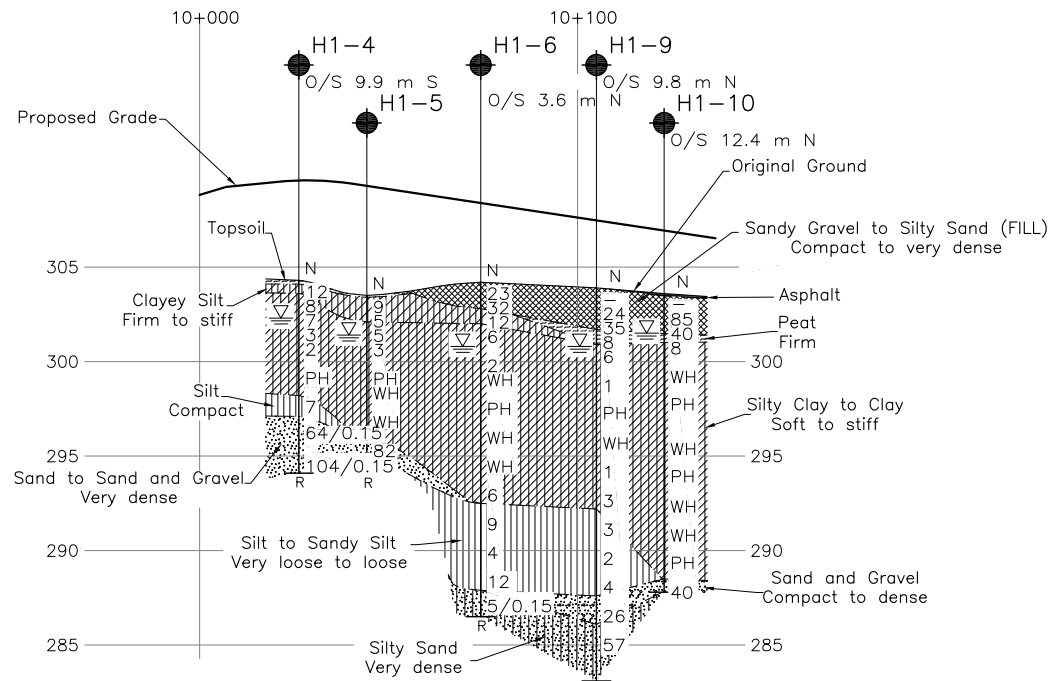
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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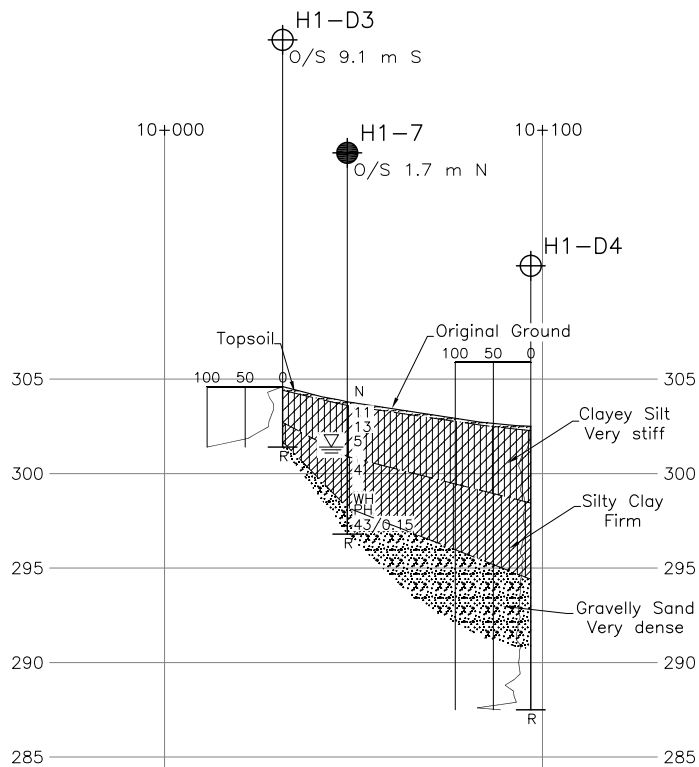
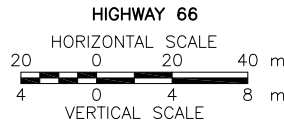
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

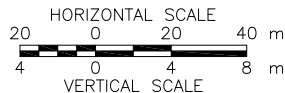
Base plans provided in digital format by MMM, drawing file nos. H3211009D16 ROLL PLAN-ULTIMATE and PDR.dwg, received DEC 3, 2012. Keyplan drawing file nos. H3211009G02 received JAN 24, 2013.



D-D'
A1 PROFILE ALONG HIGHWAY 66 CONNECTOR SOUTH TOE



E-E'
A1 PROFILE ALONG HIGHWAY 66 CONNECTOR NORTH TOE



NO.	DATE	BY	REVISION
Geocres No. 32D-17			
HWY. 66	PROJECT NO. 10-1191-0044		DIST.
SUBM'D. MT	CHKD.	DATE: DEC 2013	SITE:
DRAWN: JJL	CHKD. SEMC	APPD. JMAC	DWG. A2

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

PROJECT 10-1191-0044			RECORD OF BOREHOLE No H1-5			1 OF 1 METRIC										
G.W.P. 5091-07-00			LOCATION N 5333566.9; E 409677.8			ORIGINATED BY MT										
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY MT										
DATUM GEODETIC			DATE July 28, 2011			CHECKED BY SEMC										
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								
303.5	GROUND SURFACE															
0.0	TOPSOIL		1	SS	9											
	CLAYEY SILT, trace to some sand, trace rootlets Firm to stiff Grey and brown Moist		2	SS	5											
302.1	SILTY CLAY, trace sand		3	SS	5											
1.4	Soft to stiff Grey Wet		4	SS	3											
			5	TO	PH											
			6	SS	WH											
			7	SS	WH											
295.9	SAND and GRAVEL, some silt Very dense Grey Wet Spoon attempted at 8.7 m depth, bouncing.		8	SS	82											
294.8	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING)															
8.7	Note: 1. Water level at a depth of 2.1 m below ground surface (Elev. 301.4 m) upon completion of drilling.															

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044				RECORD OF BOREHOLE No H1-6				1 OF 2 METRIC						
G.W.P. 5091-07-00				LOCATION N 5333570.7; E 409709.6				ORIGINATED BY MT						
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT						
DATUM GEODETIC				DATE July 26, 2012				CHECKED BY SEMC						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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 (%)				
304.2	GROUND SURFACE													
0.0	Sandy gravel, trace silt (FILL) Compact to dense Brown Moist		1	SS	23									
			2	SS	32									
302.8														
1.4	Sandy CLAYEY SILT, trace to some gravel Stiff Brown Moist		3	SS	12									
302.0														
2.2	SILTY CLAY, trace sand Soft to stiff Grey Moist to wet		4	SS	6									
			5	SS	2									
			6	SS	WH									
			7	TO	PH									
			8	SS	WH									
			9	SS	WH									
			10	SS	6									
	Silt seams encountered between 10.7 m and 11.3 m depth.													
292.5														
11.7	Sandy SILT, trace to some gravel, trace to some clay Loose to compact Grey Wet		11	SS	9									
			12	SS	4									

Continued Next Page

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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT		RECORD OF BOREHOLE				No H1-6		2 OF 2		METRIC							
G.W.P. 5091-07-00		LOCATION				N 5333570.7; E 409709.6				ORIGINATED BY MT							
DIST _____ HWY 66		BOREHOLE TYPE				108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT							
DATUM GEODETIC		DATE				July 26, 2012				CHECKED BY SEMC							
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									
	--- CONTINUED FROM PREVIOUS PAGE ---																
287.9	Sandy SILT, trace to some gravel, trace to some clay Loose to compact Grey Wet		13	SS	12		289										11 27 45 7
16.3	Silty SAND, some gravel Loose Grey Wet		14	SS	5/0.15		288										
286.5							287										
17.7	END OF BOREHOLE AUGER REFUSAL Note: 1. Water level at a depth of 3.4 m below ground surface (Elev. 300.8 m) upon completion of drilling.																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044				RECORD OF BOREHOLE No H1-6a				1 OF 1 METRIC					
G.W.P. 5091-07-00				LOCATION N 5333570.2; E 409713.5				ORIGINATED BY MR					
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY AC					
DATUM GEODETIC				DATE May 21, 2013				CHECKED BY SEMC					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)		
304.2	GROUND SURFACE												
0.0	Hollow stem augers advanced from ground surface to 1.5 m depth without sampling (See Borehole H1-6).						304						
302.7							303						
1.5	Silty sand and gravel (FILL), trace organics, trace to some clay		1	SS	9								
302.0	Loose Brown Moist						302						
2.2	SILTY CLAY to CLAY Soft to stiff Grey Wet		2	TO	PH								
			3	TO	PH		301						
			4	TO	PH		300						
							299						
			5	TO	PH		298						
							297						
			6	TO	PH		296						
							295						
			7	TO	PH		294						
			8	TO	PH		293						
292.3	END OF BOREHOLE												
11.9	Note: 1. Borehole dry upon completion of drilling.												


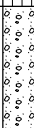


SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

PROJECT 10-1191-0044			RECORD OF BOREHOLE No H1-7			1 OF 1 METRIC								
G.W.P. 5091-07-00			LOCATION N 5333590.5; E 409686.0			ORIGINATED BY AM								
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY MT								
DATUM GEODETIC			DATE July 28, 2011			CHECKED BY SEMC								
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						
303.8	GROUND SURFACE							20 40 60 80 100	20 40 60					
0.0	TOPSOIL													
0.2	CLAYEY SILT, trace to some sand, trace gravel Very stiff Brown Moist to wet		1	SS	11									
			2	SS	13									
			3	SS	5									
300.9	SILTY CLAY, trace sand													
2.9	Firm Grey Wet		4	SS	4									
			5	SS	WH									
			6	TO	PH									
298.2	Gravelly SAND, some silt, trace clay Very dense Grey Wet													
5.6			7	SS	43/0.15									
	Spoon attempted at 7.0 m depth, bouncing.													
296.8	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING)													
7.0	1. Water level at a depth of 2.3 m below ground surface (Elev. 301.5) upon completion of drilling.													

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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>				RECORD OF BOREHOLE No H1-9				2 OF 2 METRIC									
G.W.P. <u>5091-07-00</u>				LOCATION <u>N 5333593.2; E 409730.5</u>				ORIGINATED BY <u>MR</u>									
DIST <u> </u> HWY <u>66</u>				BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>AC</u>									
DATUM <u>GEODETIC</u>				DATE <u>May 21, 2013</u>				CHECKED BY <u>SEMC</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									
	--- CONTINUED FROM PREVIOUS PAGE ---																
287.6	SILT, trace to some clay, trace to some sand Very loose Grey Wet		13	SS	4												
16.3	SAND and GRAVEL, some silt Compact Grey Wet		14	SS	26												
286.1	Silty SAND Very dense Grey Wet		15	SS	57												
283.1	Spoon attempted at 20.8 m depth; spoon bouncing.																
20.8	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING) Note: 1. Water level at a depth of 3.1 m below ground surface (Elev. 300.8 m) upon completion of drilling.																

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

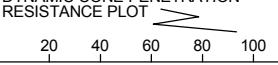


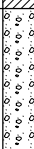
SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>			RECORD OF BOREHOLE No H1-10				2 OF 2 METRIC											
G.W.P. <u>5091-07-00</u>			LOCATION <u>N 5333609.2; E 409738.5</u>				ORIGINATED BY <u>MR</u>											
DIST <u> </u> HWY <u>66</u>			BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>AC</u>											
DATUM <u>GEODETIC</u>			DATE <u>May 22, 2013</u>				CHECKED BY <u>SEMC</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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 ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> W_p W W_L </div>						
288.5 15.1	SAND and GRAVEL, trace to some silt, trace clay Dense Grey Wet	12	SS	40		288												30 58 10 2
287.8 15.8	END OF BOREHOLE AUGER REFUSAL Note: 1. Water level at a depth of 2.1 m below ground surface (Elev. 301.5 m) upon completion of drilling.																	

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044			RECORD OF BOREHOLE No H4-1			1 OF 1 METRIC							
G.W.P. 5091-07-00			LOCATION N 5333484.6; E 409609.3			ORIGINATED BY MT							
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY MT							
DATUM GEODETIC			DATE August 28, 2012			CHECKED BY SEMC							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT <div style="display: flex; justify-content: space-around; font-size: small;"> 20 40 60 80 100 </div>	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								
304.8	GROUND SURFACE												
0.0	TOPSOIL		1	SS	4								
	CLAYEY SILT, trace gravel, trace sand, trace organics		2	SS	3								
303.4	SILTY CLAY to CLAY		3	SS	2								
1.4	Firm Grey Moist to wet		4	SS	1								
	Silt seams encountered between 3.0 m and 6.9 m depth.		5	SS	WH								
297.9	SAND and GRAVEL, trace silt, trace clay		6	SS	WH								
6.9	Very dense Grey Wet		7	SS	66								
296.7	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING)												
8.1	Note: 1. Water level at a depth of 1.1 m below ground surface (Elev. 303.7 m) upon completion of drilling.												

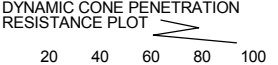
SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

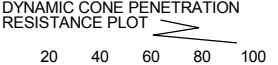
PROJECT 10-1191-0044			RECORD OF BOREHOLE No H4-2			1 OF 1 METRIC										
G.W.P. 5091-07-00			LOCATION N 5333499.7; E 409604.1			ORIGINATED BY MT										
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY MT										
DATUM GEODETIC			DATE August 28, 2012			CHECKED BY SEMC										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES											
305.8	GROUND SURFACE															
0.0	TOPSOIL		1	SS	7		305			OC=1.1	14 22 46 18					
	Sandy CLAYEY SILT, some gravel Firm Brown Moist		2	SS	5											
304.4	CLAY				304											
1.4	Soft to firm Brown to grey Moist Silt seams encountered between 1.5 m and 2.1 m depth.	3	SS	4												
		4	SS	7	303											
302.8																
3.0	SAND and GRAVEL, some silt, trace clay Dense to very dense Grey Wet		5	SS	90	302					34 49 14 3					
			6	SS	49											
301.4	END OF BOREHOLE AUGER REFUSAL															
4.4	Note: 1. Water level at a depth of 2.0 m below ground surface (Elev. 303.8 m) upon completion of drilling.															

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044			RECORD OF BOREHOLE No H4-3			1 OF 1 METRIC								
G.W.P. 5091-07-00			LOCATION N 5333491.8; E 409626.8			ORIGINATED BY MT								
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY MT								
DATUM GEODETIC			DATE August 28, 2012			CHECKED BY SEMC								
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						
304.3	GROUND SURFACE							20 40 60 80 100	20 40 60					
0.0	PEAT (Fibrous) Soft Black Moist		1	SS	3		304							
303.5			2	SS	4		303							
0.8	CLAY Soft to firm Grey Moist to wet		3	SS	4		302							
							301							
			4	SS	WH		300							
			5	SS	WH		299							
							298							
	Silt seams encountered between 6.1 m and 6.7 m depth.		6	SS	WR		297							
296.8							296							
7.5	SILT Very loose Grey Wet		7	SS	WH		295							
			8	SS	8									
294.7	SAND and GRAVEL, trace to some silt Grey Wet END OF BOREHOLE AUGER REFUSAL													
9.8	Note: 1. Water level at a depth of 3.1 m below ground surface (Elev. 301.2 m) upon completion of drilling.													

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044			RECORD OF BOREHOLE No H4-4			1 OF 1 METRIC							
G.W.P. 5091-07-00			LOCATION N 5333508.2; E 409617.5			ORIGINATED BY MT							
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY MT							
DATUM GEODETIC			DATE August 28, 2012			CHECKED BY SEMC							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT <div style="text-align: center;">  </div>	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								
305.0	GROUND SURFACE												
0.1	TOPSOIL		1a	SS	9								
	CLAYEY SILT, trace to some sand, trace gravel, trace organics Very stiff Brown Moist		1b										1 7 65 27
			2	SS	5								
302.8	CLAY												
2.2	Firm Brown to grey Moist to wet		3	SS	1								
	Silt seams encountered between 3.8 m and 4.8 m depth.		4	SS	WH								
300.2			5a										
4.8	Spoon attempted at 6.1 m depth, bouncing. Silty SAND Compact Grey Wet		5b	SS	10								1 2 85 12
298.9	Silt seam encountered between 4.8 m and 4.9 m depth.												
6.1	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING)												
	Note: 1. Water level at a depth of 1.6 m below ground surface (Elev. 303.4 m) upon completion of drilling.												


PROJECT 10-1191-0044			RECORD OF BOREHOLE No H4-5			1 OF 1 METRIC					
G.W.P. 5091-07-00			LOCATION N 5333532.2; E 409624.6			ORIGINATED BY MT					
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY MT					
DATUM GEODETIC			DATE August 29, 2012			CHECKED BY SEMC					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
305.1	GROUND SURFACE										
0.0	TOPSOIL		1a		5						
	CLAYEY SILT, some sand, trace organics		1b	SS							
	Firm										
	Brown		2	SS	6						
	Moist										
303.7	CLAY, trace organics		3	SS	4						
1.4	Soft to firm										
	Grey to brown										
	Moist to wet		4a	SS	8						
302.4			4b								
	SILT, trace to some clay										
302.1	Grey to brown										
3.0	Wet										
	SAND and GRAVEL, trace silt		5	SS	28						
301.4	Compact										
3.7	Brown										
	Wet										
	Spoon bouncing at 3.6 m depth.										
	END OF BOREHOLE										
	AUGER REFUSAL										
	Note:										
	1. Water level at a depth of 1.8 m below ground surface (Elev. 303.3 m) upon completion of drilling.										

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT		10-1191-0044		RECORD OF BOREHOLE No H4-6				1 OF 1 METRIC						
G.W.P.		5091-07-00		LOCATION		N 5333556.5; E 409630.6		ORIGINATED BY						
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY						
DATUM		GEODETIC		DATE		September 4, 2012		CHECKED BY						
								SEMC						
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						
305.8	GROUND SURFACE							20 40 60 80 100	20 40 60					
0.0	TOPSOIL													
0.2	CLAYEY SILT, some sand, trace gravel, trace organics		1	SS	9									
305.1	Stiff													
0.7	Brown Moist CLAY, trace sand		2	SS	7									
	Firm Grey Moist		3	SS	7									
303.4														
2.4	Silty SAND, trace to some gravel, trace clay		4	SS	12									
	Compact Brown													
302.7	Moist to wet		5	SS	9/0.10									
3.1	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING)													
Note: 1. Water level at a depth of 2.0 m below ground surface (Elev. 303.8 m) upon completion of drilling.														

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

PROJECT		RECORD OF BOREHOLE				No C1-2		2 OF 2		METRIC							
G.W.P. 5091-07-00		LOCATION				N 5333523.2; E 409645.0				ORIGINATED BY MS							
DIST _____ HWY 66		BOREHOLE TYPE				108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring				COMPILED BY MT							
DATUM GEODETIC		DATE				August 6, 2011				CHECKED BY SEMC							
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									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
289.8 15.6	END OF BOREHOLE Note: 1. Water level at a depth of 1.7 m below ground surface (Elev. 303.7 m) upon completion of drilling. 2. Water level in piezometer measured at ground surface (Elev. 305.4 m) on May 19, 2013.		4	RC	REC 100%		290										RQD = 100%

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: C1-2

SHEET 1 OF 1

LOCATION: N 5333523.2 ; E 409645.0

DRILLING DATE: August 6, 2011

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D50

DRILLING CONTRACTOR: Walker Drilling Ltd.

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.																NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
							FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA						HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)		RMC -Q' AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
								TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION		Jr	Ja	Jn	k, cm/s	10 ⁻⁶	10 ⁻⁵				10 ⁻⁴																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
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DEPTH SCALE

1 : 50



LOGGED: MS

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:

PROJECT 10-1191-0044			RECORD OF BOREHOLE No BC1-1			1 OF 2 METRIC						
G.W.P. 5091-07-00			LOCATION N 5333456.8; E 409598.0			ORIGINATED BY MT						
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring			COMPILED BY MT						
DATUM GEODETIC			DATE September 8 and 9, 2012			CHECKED BY SEMC						
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	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					
305.0	GROUND SURFACE											
0.0	TOPSOIL		1	SS	5							
	SILTY CLAY to CLAY Firm to stiff Grey Moist to wet		2	SS	4							
			3	SS	WH							
	Silt seams encountered between 0.1 m and 5.3 m depth.		4	SS	WH							
	Roots and silt pockets encountered at 5.2 m depth in Shelby tube Sample 5.		5	TO	PH							
299.7												
5.3	CLAYEY SILT Stiff Grey Wet		6	SS	2							
297.8												
7.2	Sandy GRAVEL Dense to very dense Grey Wet		7	SS	35							
			8	SS	18/0.05							
	Cobbles encountered between 10.1 m and 10.3 m depth.		9	SS	36/0.15							
294.2												
10.8	METASEDIMENT (BEDROCK)		1	RC	REC 100%							
	Bedrock cored from 10.8 m depth to 14.2 m depth.											
	For coring details see Record of Drillhole BC1-1.		2	RC	REC 100%							
			3	RC	REC 98%							
290.8												
14.2												

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

Continued Next Page

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

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No BC1-1				2 OF 2 METRIC											
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5333456.8; E 409598.0</u>				ORIGINATED BY <u>MT</u>											
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u>				COMPILED BY <u>MT</u>											
DATUM <u>GEODETIC</u>		DATE <u>September 8 and 9, 2012</u>				CHECKED BY <u>SEMC</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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. Water level at a depth of 2.0 m below ground surface (Elev. 303.0 m) upon completion of drilling.																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC1-1

SHEET 1 OF 1

LOCATION: N 5333456.8 ; E 409598.0

DRILLING DATE: September 9, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling Inc.

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										DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s	Diametral Point Load Index (MPa)	RMC -Q AVG.	NOTES WATER LEVELS INSTRUMENTATION					
							RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA															HYDRAULIC CONDUCTIVITY k, cm/s			Diametral Point Load Index (MPa)	RMC -Q AVG.
							TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION															HYDRAULIC CONDUCTIVITY k, cm/s				
							80 90 95 100	80 90 95 100			B Angle 0° 10° 20° 30° 40° 50° 60° 70° 80° 90°	DIP w.r.t. CORE AXIS 0° 10° 20° 30° 40° 50° 60° 70° 80° 90°	Ur	Ja	Jn	10° 10° 10° 10° 10° 10° 10° 10° 10° 10°										10° 10° 10° 10° 10° 10° 10° 10° 10° 10°	10° 10° 10° 10° 10° 10° 10° 10° 10° 10°			
REFER TO PREVIOUS PAGE																														
11	NW September 9, 2012 NQ Coring	METASEDIMENT Very strong Fine grained Moderately weathered to fresh Greenish grey Sheared zone encountered between 12.7 m and 14.2 m depth.		294.2 10.8	1	WHITE 100%																	112 MPa							
12					2	GREY/WHITE 100%																								
13					3	GREY/WHITE 100%																								
14	END OF DRILLHOLE			290.8 14.2																										
15																														
16																														
17																														
18																														
19																														
20																														

DEPTH SCALE

1 : 50



LOGGED: MT

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:

PROJECT 10-1191-0044			RECORD OF BOREHOLE No BC1-2			1 OF 1 METRIC																
G.W.P. 5091-07-00			LOCATION N 5333464.9; E 409579.7			ORIGINATED BY MT																
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring			COMPILED BY MT																
DATUM GEODETIC			DATE September 9 and 10, 2012			CHECKED BY SEMC																
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 (%) W _p — W — W _L			γ			GR SA SI CL			
306.0	GROUND SURFACE							20 40 60 80 100														
0.9	TOPSOIL		1a	SS	11																	
	Gravelly Sandy CLAYEY SILT		1b																			
	Firm to stiff																					
	Brown		2	SS	6																	
	Moist																					
304.6	SILTY CLAY, trace gravel, trace sand																					
1.4	Soft		3	SS	4																	
	Moist																					
	Approximately 25 mm thick sand seam encountered at 2.1 m depth.		4	SS	3																	
302.8	Sandy GRAVEL, trace silt, trace clay		5a																			
3.2	Compact to dense		5b	SS	27																	
	Grey to brown																					
	Wet		6	SS	35																	
	Approximately 0.4 m thick sand seam encountered at 4.6 m depth.		7a																			
			7b	SS	27																	
			8	SS	47/0.03																	
	Cobbles encountered between 6.3 m and 6.9 m depth.																					
299.1	METASEDIMENT (BEDROCK)																					
6.9	Bedrock cored from 6.9 m depth to 10.5 m depth.		1	RC	REC 100%																	
	For coring details see Record of Drillhole BC1-2.																					
			2	RC	REC 100%																	
			3	RC	REC 100%																	
295.5	END OF BOREHOLE																					
10.5	Note: 1. Water level at a depth of 2.1 m below ground surface (Elev. 303.9 m) upon completion of drilling.																					

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

SHEET 1 OF 1

DATUM: GEODETIC

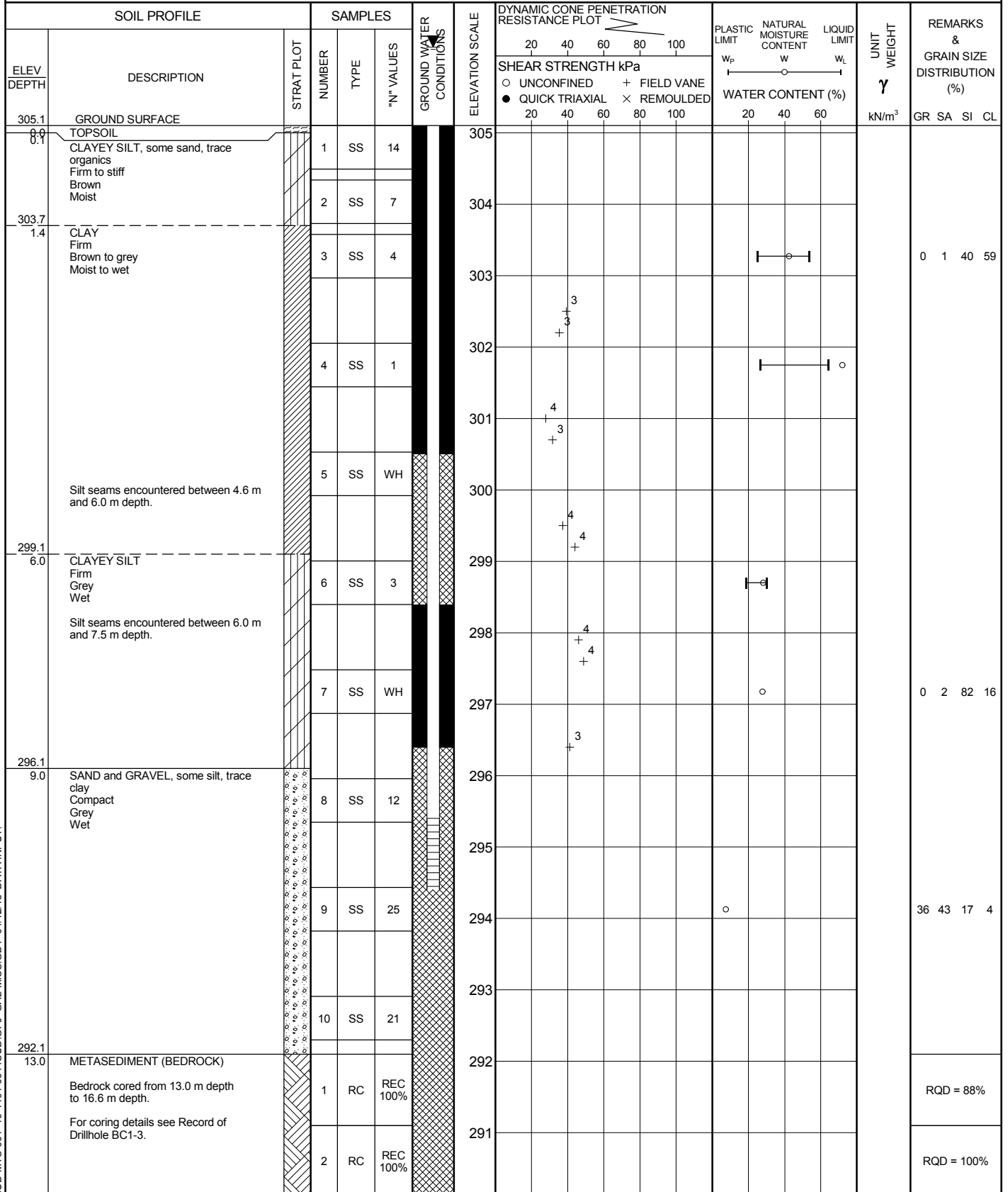
DRILLING CONTRACTOR: Landcore Drilling Inc.

CHECKED: SEMC

RECORD OF BOREHOLE No BC1-3

1 OF 2 **METRIC**

PROJECT 10-1191-0044
 G.W.P. 5091-07-00 LOCATION N 5333449.6; E 409614.5 ORIGINATED BY MT
 DIST HWY 66 BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring COMPILED BY MT
 DATUM GEODETIC DATE September 10, 2012 CHECKED BY SEMC



Continued Next Page

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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044				RECORD OF BOREHOLE No BC1-3				2 OF 2 METRIC									
G.W.P. 5091-07-00				LOCATION N 5333449.6; E 409614.5				ORIGINATED BY MT									
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring				COMPILED BY MT									
DATUM GEODETIC				DATE September 10, 2012				CHECKED BY SEMC									
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									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
	METASEDIMENT (BEDROCK)		2	RC			290										RQD = 100%
	Bedrock cored from 13.0 m depth to 16.6 m depth. For coring details see Record of Drillhole BC1-3.		3	RC	REC 100%		289										RQD = 100%
288.5 16.6	END OF BOREHOLE Note: 1. Water level at a depth of 1.0 m below ground surface (Elev. 304.1 m) upon completion of drilling. 2. Water level in piezometer measured at 1.0 m above ground surface (Elev. 306.1 m) on November 17, 2012 and at 1.1 m above ground surface (Elev. 306.2 m) on May 19, 2013. 3. Piezometer installed within heaving sand and gravel.																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC1-3

SHEET 1 OF 1

LOCATION: N 5333449.6 ; E 409614.5

DRILLING DATE: September 10, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular PO- Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break BR - Broken Rock										NOTES WATER LEVELS INSTRUMENTATION				
							FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA						HYDRAULIC CONDUCTIVITY k, cm/s	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
13	NQ Coring September 10, 2012	REFER TO PREVIOUS PAGE		292.1																	
		METASEDIMENT Strong Fine grained Fresh Greenish Grey		13.0	1	GREY 100%															
14		Sheared zone between 13.0 m and 16.6 m depth.			2	GREY 100%													62 MPa		
15					3	GREY 100%															
16		END OF DRILLHOLE		288.5																	
17				16.6																	
18																					
19																					
20																					
21																					
22																					
23																					

DEPTH SCALE

1 : 50



LOGGED: MT

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST No H1-D3				1 OF 1 METRIC							
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5333582.7; E 409667.3</u>				ORIGINATED BY <u>MT</u>							
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>MT</u>							
DATUM <u>GEODETIC</u>		DATE <u>July 28, 2011</u>				CHECKED BY <u>SEMC</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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
304.6 0.0	GROUND SURFACE						<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>						
301.4 3.2	END OF DCPT REFUSAL TO FURTHER PENETRATION 100 BLOWS / 0.15 m (HAMMER BOUNCING)												

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST No H1-D4		1 OF 2 METRIC	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5333596.8; E 409715.3</u>		ORIGINATED BY <u>MT</u>	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>		COMPILED BY <u>MT</u>	
DATUM <u>GEODETIC</u>		DATE <u>August 5, 2011</u>		CHECKED BY <u>SEMC</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	20 40 60 80 100	20 40 60 80 100	W _p W W _L	20 40 60		
302.5 0.0	GROUND SURFACE						<div style="display: flex; justify-content: space-between;"> <div> ○ UNCONFINED ● QUICK TRIAXIAL </div> <div> + FIELD VANE × REMOULDED </div> </div>						
302													
301													
300													
299													
298													
297													
296													
295													
294													
293													
292													
291													
290													
289													
288													
287.5													


SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

Continued Next Page

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

PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST No H1-D4				2 OF 2 METRIC											
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5333596.8; E 409715.3</u>				ORIGINATED BY <u>MT</u>											
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>MT</u>											
DATUM <u>GEODETIC</u>		DATE <u>August 5, 2011</u>				CHECKED BY <u>SEMC</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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 ---																
15.0	END OF DCPT REFUSAL TO FURTHER PENETRATION 40 BLOWS / 0.10 m (HAMMER BOUNCING)																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST No H4-D1				1 OF 1 METRIC				
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5333522.4; E 409613.5</u>				ORIGINATED BY <u>MT</u>				
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>TR</u>				
DATUM <u>GEODETIC</u>		DATE <u>August 29, 2012</u>				CHECKED BY <u>SEMC</u>				
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
305.9 0.0	GROUND SURFACE									
302.8 3.1	END OF DCPT REFUSAL TO FURTHER PENETRATION 22 BLOWS / 0.03 m (HAMMER BOUNCING)									

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



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



**FOUNDATION REPORT – SWAMP CROSSINGS/HIGH FILL AREAS AND DEEP CUT
HIGHWAY 66 AT VIRGINIATOWN, GWP 5091-07-00**

**Table A1: Evaluation of Settlement Mitigation Options (High Fill H4)
Highway 66 – STA 13+080 to 13+185**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Preloading ■ 6 month preload period ■ Total post-construction settlement = 200 mm (145 mm primary + 50 mm creep + 5 mm rock fill)	1	■ Standard construction operation. ■ Reduces post-construction settlement. ■ Makes use only of fill material that is required for embankment construction.	■ Delay in construction schedule to allow for sufficient settlement to meet post-construction settlement criterion. ■ Instrumentation and monitoring program required to assess end of preload period. ■ Re-grading is required prior to final pavement structure construction.	■ Additional cost for instrumentation and associated monitoring program = ~\$150,000.	■ Low risk of not achieving/maintaining stability of embankments on weak/soft foundation soils. ■ Low risk that additional time may be required for continued preloading but delay period is subject to the monitoring data. ■ Low risk of experiencing unexpected post-construction settlement (i.e. creep).
Lightweight Fill (1.0 m EPS) ■ Total post-construction settlement = 185 mm (110 mm primary + 50 mm creep + 25 mm rock fill)	2	■ Reduces total load on subsoils thereby reducing total settlement of foundations soils. ■ No delay time in construction.	■ Expensive material compared to conventional embankment fill. ■ Restricted thickness that can be used dependent on overall thickness of embankment and groundwater/water level.	■ Relative cost of EPS fill is about an order of magnitude higher than fill required for the other options. ■ $2100 \text{ m}^3 \times \$200/\text{m}^3 = \$420,000$.	■ Very low risk of not achieving stability of preload embankments and of final EPS embankments on weak/soft foundation soils. ■ Low risk of experiencing unexpected post-construction settlements (i.e. creep).
Full Sub-Excavation (up to 9 m deep at new alignment or 11 m below existing highway) ■ Total post-construction settlement = 110 mm (rock fill only)	NF	■ Reduces magnitude of total settlement of foundations soils as soft compressible material has been removed.	■ Generation of very large volume of excess excavation spoil. ■ Very large quantity of rock fill required. ■ Longer construction period required to sub-excavate to 9 m depth and replacement with rock fill. ■ Specialized equipment and additional effort required for deep sub-excavation and replacement. ■ Additional post-construction settlement of rock fill itself. ■ May require protection system/shoring adjacent to the existing highway to allow for removal of cohesive deposits within the embankment zone of influence.	■ Additional cost for sub-excavation (long-stick) equipment, disposal and replacement of weak/soft, compressible deposits. ■ $35,000 \text{ m}^3 \times \$20/\text{m}^3$ (sub-excavation and replacement with rock fill) = \$700,000. ■ Additional cost for shoring system = ~\$500,000.	■ Higher risk of not achieving/maintaining stability of excavation slopes. ■ Very low of achieving/maintaining stability of proposed embankments. ■ Very low risk of experiencing unexpected post-construction settlements associated with long term rock fill settlement. ■ High risk that not all compressible soils are removed during the sub-aqueous operations which could lead to unexpected settlement.

NF: Not Feasible

Prepared By: SEMC Reviewed By: JMAC



**FOUNDATION REPORT – SWAMP CROSSINGS/HIGH FILL AREAS AND DEEP CUT
HIGHWAY 66 AT VIRGINIATOWN GWP 5091-07-00**

**Table A2: Evaluation of Settlement Mitigation Options (High Fill H1)
Highway 66 Connection – STA 10+000 to 10+050 (adjacent to new alignment)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Preloading and Toe Berms (2 m high by 6.5 m wide) ■ 10 month preload period ■ Total post-construction settlement = 200 mm (145 mm primary + 50 mm creep + 10 mm rock fill)	1	■ Standard construction operation. ■ Reduces post-construction settlement.	■ Toe berms are required to maintain stability, requiring additional fill. ■ Delay in construction schedule to allow for sufficient settlement to meet post-construction settlement criterion. ■ Instrumentation and monitoring program required to assess end of preload period. ■ Re-grading is required prior to final pavement structure construction.	■ Additional cost of rock fill for toe berms. ■ Additional cost for instrumentation and associated monitoring program = ~\$150,000.	■ Low risk of not achieving/maintaining stability of embankments on weak/soft foundation soils. ■ Low risk that additional time may be required for continued preloading, but delay period is subject to the monitoring data. ■ Low risk of experiencing unexpected post-construction settlements (i.e. creep).
Lightweight Fill (2.0 m EPS) ■ Total post-construction settlement = 200 mm (130 mm primary + 50 mm creep + 20 mm rock fill)	2	■ Reduces total load on subsoils thereby reducing total settlement of foundations soils. ■ No delay time in construction.	■ Expensive material compared to conventional embankment fill. ■ Restricted thickness that can be used dependent on overall thickness of embankment and groundwater/water level.	■ Relative cost of EPS fill is about an order of magnitude higher than fill required for the other options. ■ $2,000 \text{ m}^3 \times \$200/\text{m}^3 = \$400,000$.	■ Very low risk of not achieving stability of preload embankments and final EPS embankments on weak/soft foundation soils. ■ Low risk of experiencing unexpected post-construction settlements (i.e. creep).
Full Sub-Excavation (up to 7.3 m deep) ■ Total post-construction settlement = 175 mm (rock fill only)	3	■ Reduces total settlement of foundations soils as soft compressible material has been removed.	■ Generation of very large volume of excess excavation spoil. ■ Very large quantity of rock fill required. ■ Longer construction period required to sub-excavate to about 7 m depth and replacement with rock fill. ■ Additional post-construction settlement of rock fill itself.	■ Additional cost for sub-excavation, disposal and replacement of weak/soft, compressible deposits. ■ $22,500 \text{ m}^3 \times \$20/\text{m}^3$ (sub-excavation and replacement with rock fill) = \$450,000.	■ Higher risk of not achieving/maintaining stability of excavation slopes. ■ Very low risk of achieving/maintaining stability of proposed embankments. ■ Very low risk of experiencing unexpected post-construction settlements associated with long term rock fill settlement. ■ High risk that not all compressible soils are removed during the sub-aqueous operations which could lead to unexpected settlement.

NF: Not Feasible

Prepared By: SEMC Reviewed By: JMAG



FOUNDATION REPORT – SWAMP CROSSINGS/HIGH FILL AREAS AND DEEP CUT HIGHWAY 66 AT VIRGINIATOWN GWP 5091-07-00

Table A3: Evaluation of Settlement Mitigation Options (High Fill H1)
Highway 66 Connection – STA 10+050 to 10+125 (adjacent/over existing highway)

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Lightweight Fill (2 m to 3 m EPS) ■ Total post-construction settlement = 200 mm (130 mm primary +50 mm creep + 20 mm rock fill)	1	■ Reduces total load on subsoils thereby reducing total settlement of foundations soils. ■ No delay time in construction.	■ Expensive material compared to conventional embankment fill.	■ Relative cost of EPS fill is about an order of magnitude higher than fill required for the other options. ■ $2,500 \text{ m}^3 \times \$200/\text{m}^3 = \$500,000$.	■ Very low risk of not achieving stability of preload embankments and final EPS embankments on weak/soft foundation soils. ■ Low risk of experiencing unexpected post-construction settlements (i.e. creep).
Ground Improvement	2	■ Improves engineering parameters of soft cohesive soils thereby improving stability and reducing post-construction settlement.	■ Since existing highway is to remain in operation, the ground improvement would have to be installed in sections/stages, unless other staging/detours are implemented. ■ Type of embankment fill may be determined by the type of ground improvement option. ■ Bench scale testing may be required by proprietary contractor.	■ Relative cost of ground improvement techniques about an order of magnitude higher than other options but similar to EPS option. ■ $(75 \text{ m}/1.5 \text{ m}) \times (35 \text{ m}/1.5 \text{ m}) = 1150 \text{ columns} \times 12 \text{ m} = 14,000 \text{ m} \times \$180/\text{m} = \$2,500,000$.	■ Very low risk of not achieving stability of preload embankments. ■ There is a risk that unexpected post-construction settlements (i.e. creep) will be experienced depending on the efficacy of improving/strengthening the soft/weak soil deposit.
Preloading and Maintenance ■ Total post-construction settlement = 595 mm (510 mm primary + 60 mm creep + 25 mm rock fill) ■ Exceeds criterion, maintenance required.	3	■ Standard construction operation.	■ All settlement occurs post-construction; road re-construction involving removing asphalt, adding material to final grade and re-paving will be required after 3.5 years. Settlement will continue to occur after the re-construction but will meet the settlement criterion.	■ Cost of future road re-paving.	■ Low risk with maintaining stability of embankments on weak/soft foundation soils. ■ Settlement will occur after construction.
Surcharging and Maintenance	NF	■ Standard construction operation.	■ All settlement occurs post-construction; road re-construction should only involve re-paving provided that roadway has settled to the level of the final grade after 3.5 years. Settlement will continue to occur after the re-paving but will meet the settlement criterion.	■ Cost of future road re-paving.	■ Low risk with maintaining stability of embankments on weak/soft foundation soils. ■ Settlement will occur after construction. ■ Moderate risk that settlement magnitude may be less/more than anticipated to meet final grade.



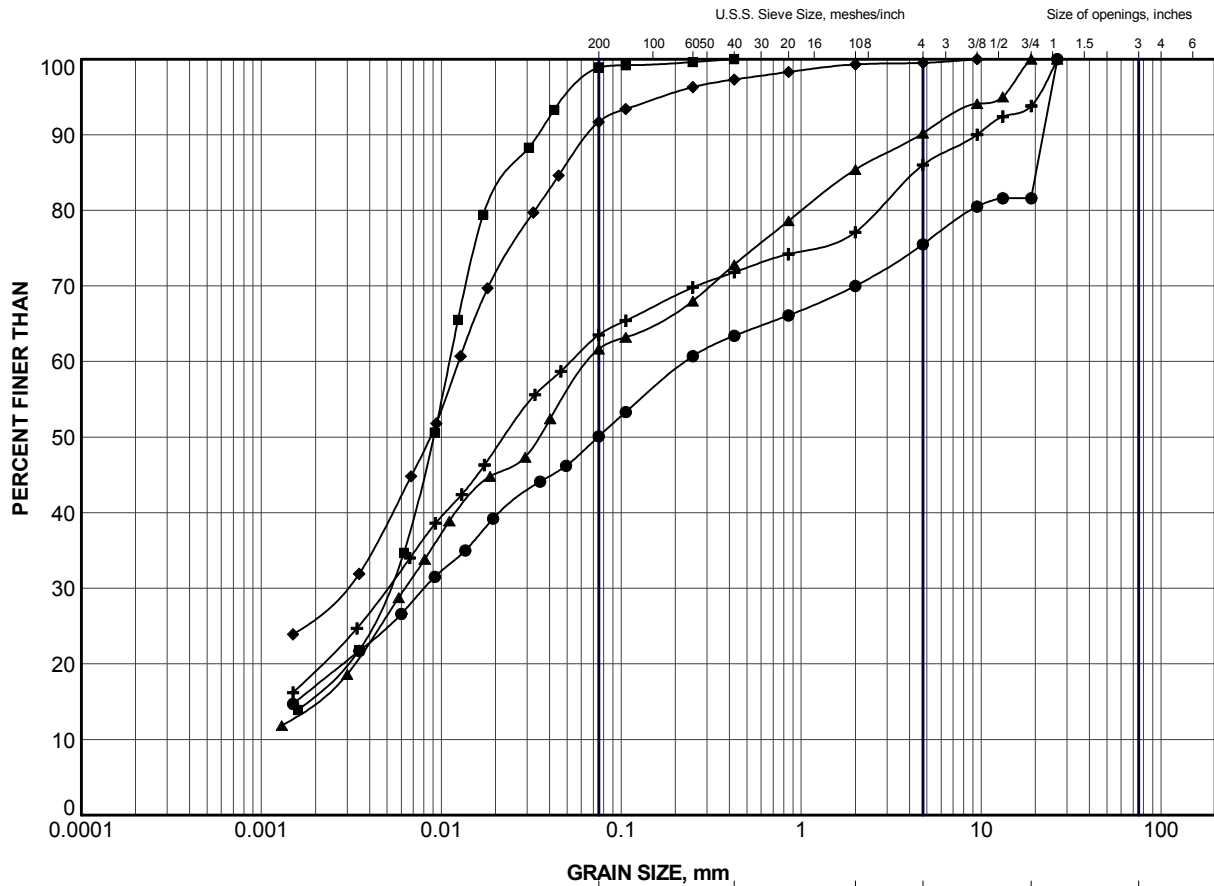
FOUNDATION REPORT – SWAMP CROSSINGS/HIGH FILL AREAS AND DEEP CUT
HIGHWAY 66 AT VIRGINIATOWN GWP 5091-07-00

Table A3: Evaluation of Settlement Mitigation Options (High Fill H1)
Highway 66 Connection – STA 10+050 to 10+125 (adjacent/over existing highway)

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Full Sub-Excavation (up to 15 m)	NF	<ul style="list-style-type: none">■ Reduces total settlement of foundations soils as soft compressible material has been removed.	<ul style="list-style-type: none">■ Not practical to remove entire existing embankment to remove underlying cohesive soil.■ Since highway to remain in operation, extensive shoring system would be required to carry out the sub-excavation and maintain traffic on existing highway, unless other staging/detours methods are implemented.■ Generation of very large volume of excess excavation spoil.■ Very large quantity of rock fill required.■ Longer construction period required to sub-excavate and replace with rock fill.■ Additional post-construction settlement of rock fill itself.	<ul style="list-style-type: none">■ Additional cost for sub-excavation, disposal and replacement of weak/soft, compressible deposits.■ 34,900 m³ x \$20/m³ (sub-excavation and replacement with rock fill) = \$700,000.■ Large cost of shoring = ~\$500,000 to \$1,000,000.	<ul style="list-style-type: none">■ Higher risk of not achieving/maintaining stability of excavation slopes.■ Very low risk of achieving/maintaining stability of proposed embankments.■ Very low risk of experiencing unexpected post-construction settlements (i.e. long term rock fill settlement).■ High risk that not all compressible soils are removed during the sub-aqueous operations which could lead to unexpected settlement.

NF: Not Feasible

Prepared By: SEMC Reviewed By: JMAC



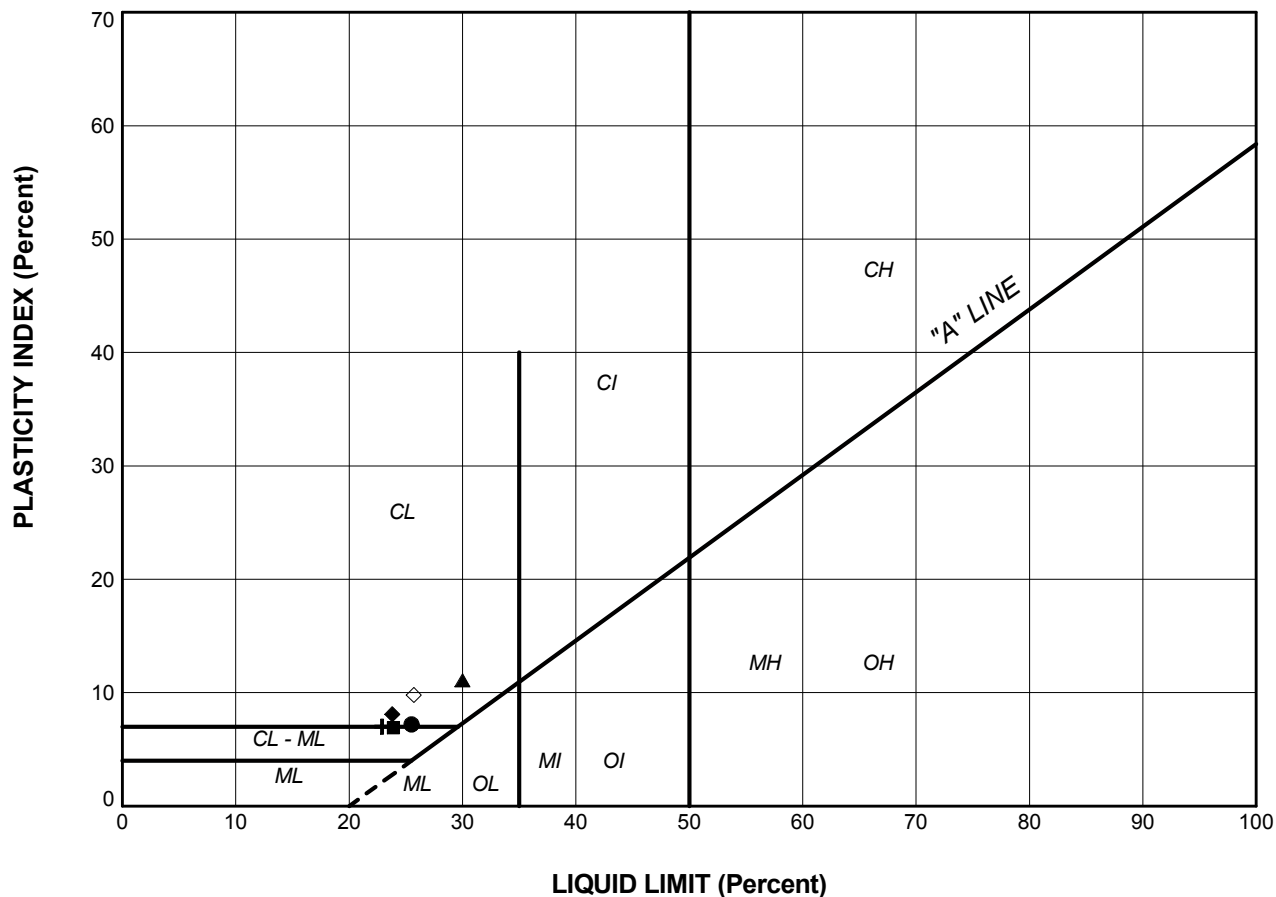
CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC1-2	1b	305.7
■	BC1-3	7	297.2
▲	H1-6	3	302.4
+	H4-2	1	305.5
◆	H4-4	1b	304.6

PROJECT					HIGHWAY 66/ HWY 66 CONNECTION HIGH FILL H1 / H4 STA 13+110 TO 13+185 / STA 10+000 TO 10+125				
TITLE					GRAIN SIZE DISTRIBUTION CLAYEY SILT				
PROJECT No.		10-1191-0044		FILE No.		10-1191-0044SUD.GPJ			
DRAWN	JJL	Aug 2013		SCALE	N/A	REV.			
CHECK	SEMC	Aug 2013		FIGURE A1					
APPR	JMAC	Aug 2013							





SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

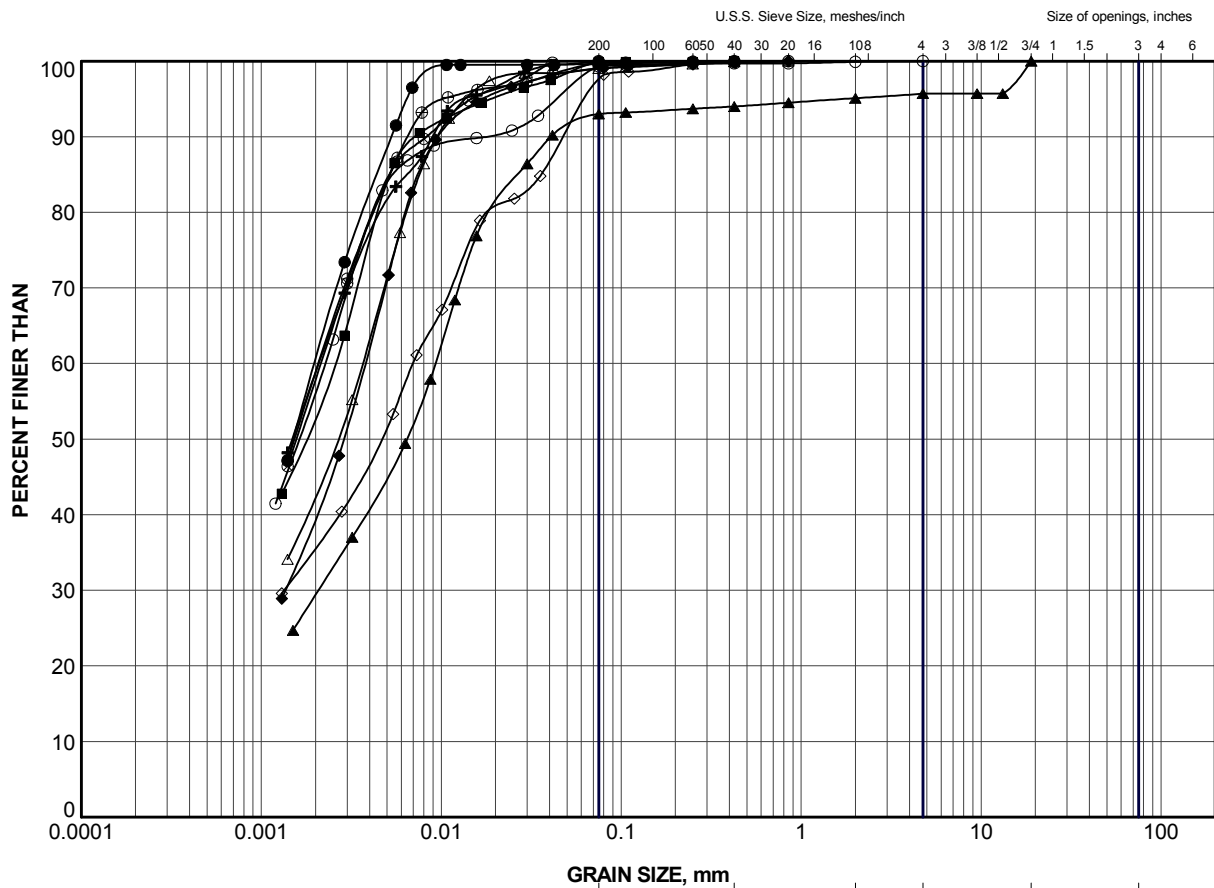
PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BC1-1	6	25.5	18.3	7.2
■	BC1-2	1b	23.9	17.0	6.9
▲	BC1-3	6	30.0	18.9	11.1
+	H4-2	1	22.9	15.9	7.0
◆	H4-4	1b	23.8	15.7	8.1
◇	H4-5	1b	25.7	15.9	9.8

PROJECT					HIGHWAY 66/ HWY 66 CONNECTION HIGH FILL H1 / H4 STA 13+110 TO 13+185 / STA 10+000 TO 10+125				
TITLE					PLASTICITY CHART CLAYEY SILT				
PROJECT No.			10-1191-0044		FILE No.			10-1191-0044SUD.GPJ	
DRAWN	JJL	Aug 2013	CHECK	SEMC	Aug 2013	SCALE	N/A	REV.	
APPR	JMAC	Aug 2013				FIGURE A2			




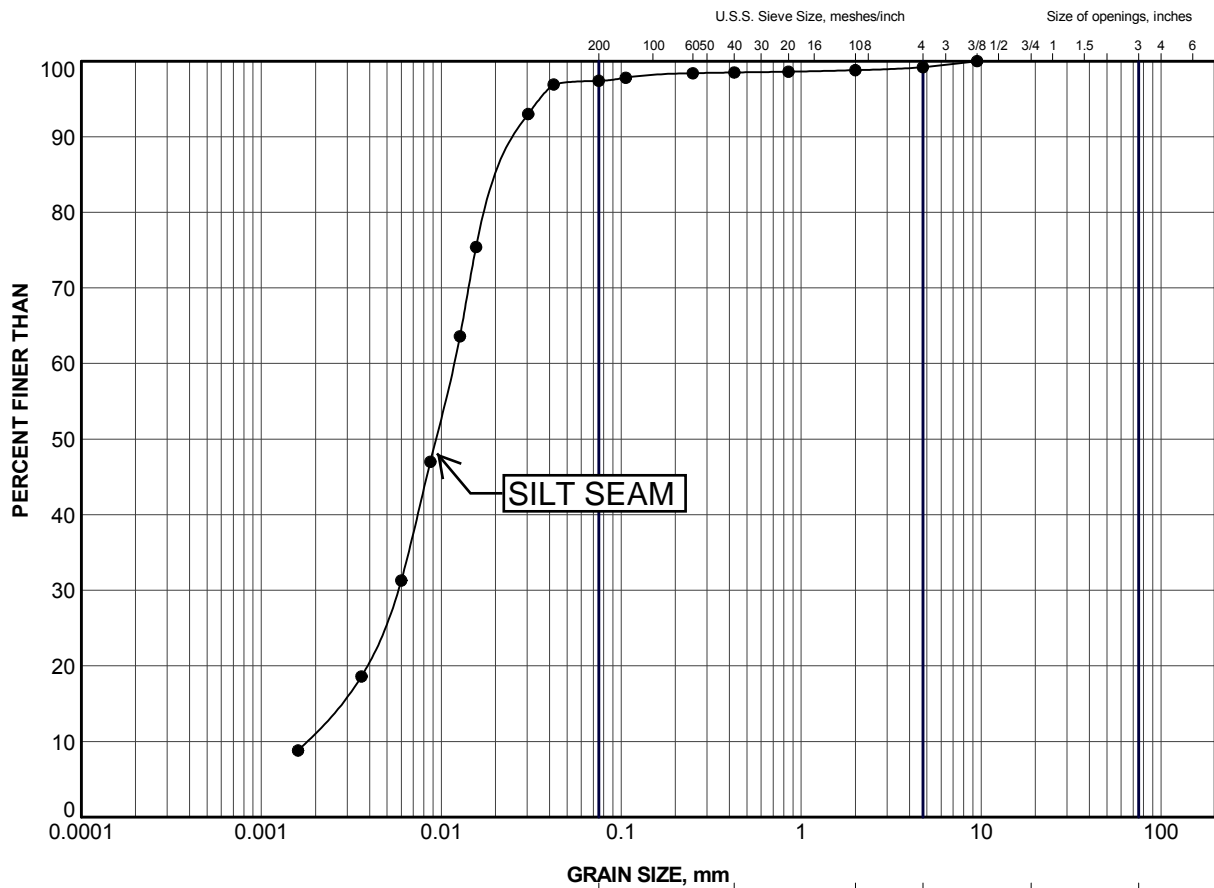


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC1-1	4	300.9
■	BC1-1	5	300.2
▲	BC1-2	5a	302.9
+	BC1-3	3	303.3
◆	C1-2	6	299.0
◇	H1-4	2	303.2
○	H1-5	4	300.9
△	H1-10	7	295.7
⊗	H4-1	3	303.0
⊕	H4-3	4	300.9


PROJECT						HIGHWAY 66/ HWY 66 CONNECTION HIGH FILL H1 / H4 STA 13+110 TO 13+185 / STA 10+000 TO 10+125					
TITLE						GRAIN SIZE DISTRIBUTION SILTY CLAY to CLAY					
PROJECT No.			10-1191-0044			FILE No.			10-1191-0044SUD.GPJ		
DRAWN		JJL		Aug 2013		SCALE		N/A		REV.	
CHECK		SEMC		Aug 2013							
APPR		JMAC		Aug 2013							
 Golder Associates SUDBURY, ONTARIO						FIGURE A3.1					

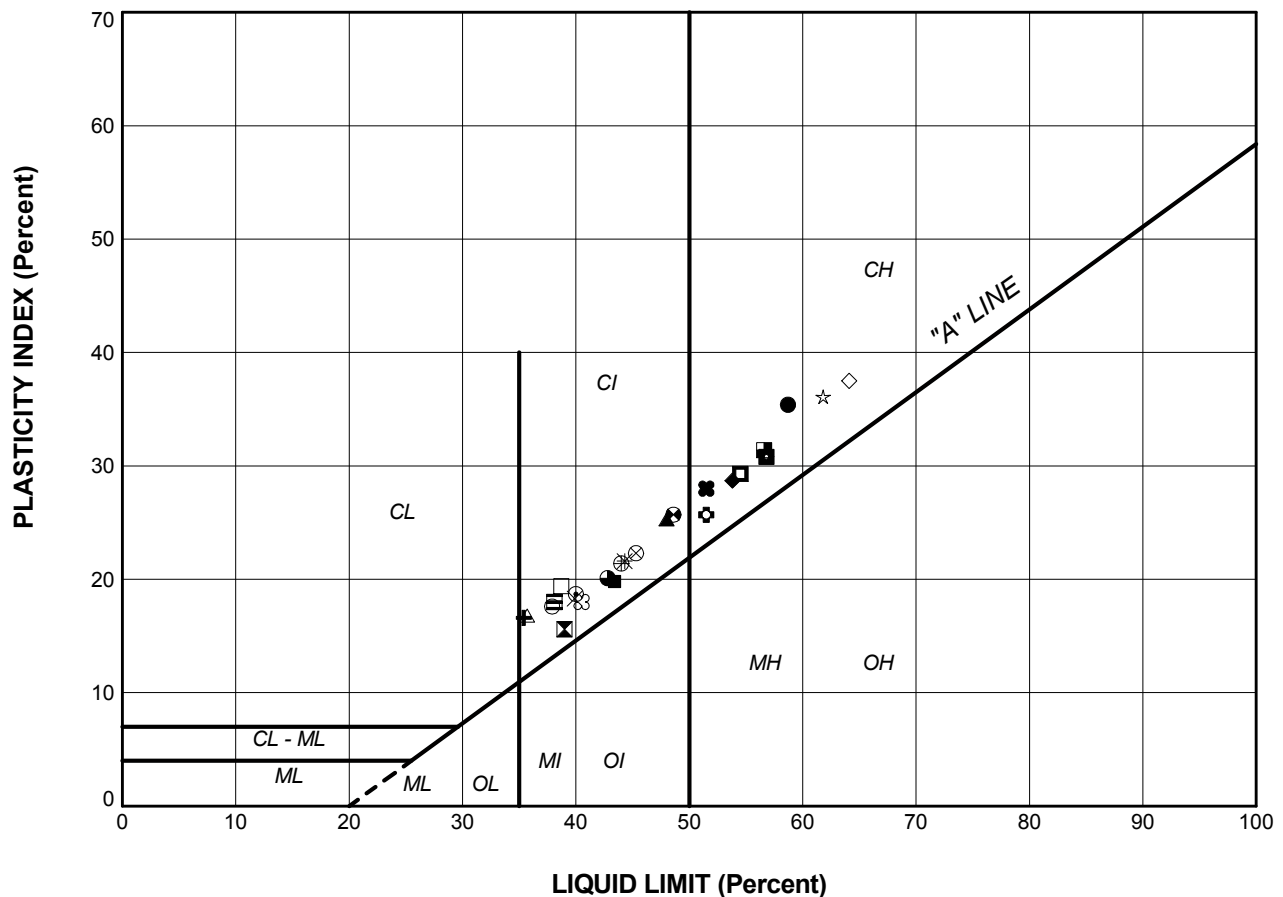


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	H4-4	5	300.3

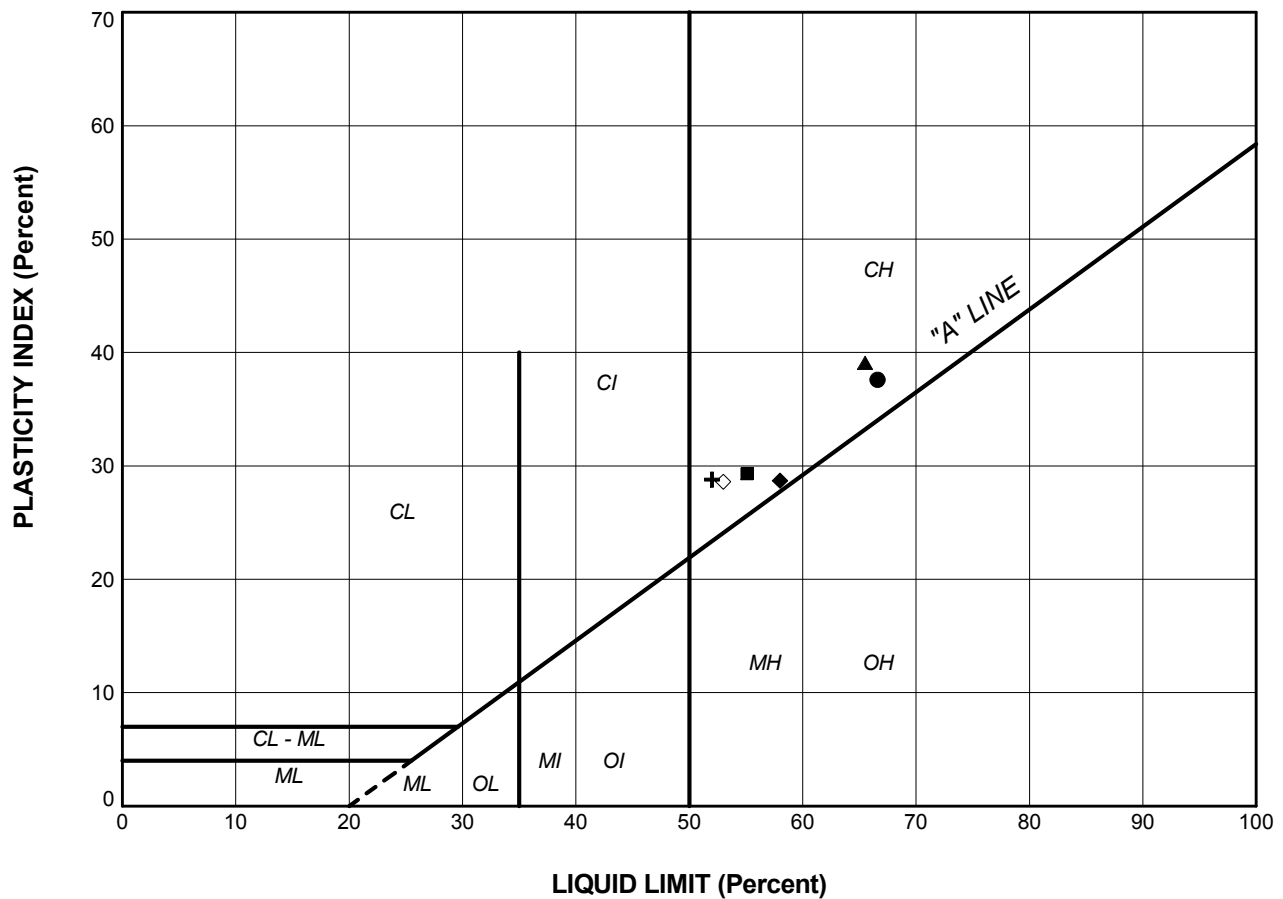
PROJECT						HIGHWAY 66/ HWY 66 CONNECTION HIGH FILL H1 / H4 STA 13+110 TO 13+185 / STA 10+000 TO 10+125					
TITLE						GRAIN SIZE DISTRIBUTION SILTY CLAY to CLAY					
 Golder Associates SUDBURY, ONTARIO			PROJECT No. 10-1191-0044			FILE No. 10-1191-0044SUD.GPJ					
			DRAWN	JJL	Aug 2013	SCALE	N/A	REV.			
			CHECK	SEMC	Aug 2013	FIGURE A3.2					
			APPR	JMAC	Aug 2013						



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BC1-1	4	58.7	23.3	35.4
■	BC1-1	5	43.4	23.6	19.8
▲	BC1-2	3	48.0	22.7	25.3
+	BC1-2	5a	35.4	18.8	16.6
◆	BC1-3	3	53.8	25.1	28.7
◇	BC1-3	4	64.1	26.6	37.5
○	C1-2	6	37.9	20.3	17.6
△	H1-4	2	35.7	18.9	16.8
⊗	H1-5	4	45.3	23.0	22.3
⊕	H1-5	6	44.0	22.6	21.4
□	H1-6	4	38.7	19.3	19.4
⊙	H1-6	6	48.6	22.9	25.7
⊗	H1-6a	3	42.8	22.7	20.1
☆	H1-6a	4	61.8	25.7	36.1
⊗	H1-7	5	40.5	22.5	18.0
⊗	H1-9	5	39.0	23.4	15.6
⊗	H1-9	6	40.0	21.3	18.7
⊗	H1-9	7	51.5	25.8	25.7
×	H1-9	8	39.9	21.6	18.3
⊗	H1-10	4b	51.5	23.5	28.0
■	H1-10	5	56.8	26.0	30.8
*	H1-10	7	44.3	22.7	21.6
□	H4-1	3	54.5	25.2	29.3
■	H4-1	4	56.6	25.2	31.4
■	H4-1	6	38.1	20.1	18.0

PROJECT					
HIGHWAY 66/ HWY 66 CONNECTION HIGH FILL H1 / H4 STA 13+110 TO 13+185 / STA 10+000 TO 10+125					
TITLE					
PLASTICITY CHART SILTY CLAY to CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Aug 2013	SCALE	N/A	REV.
CHECK	SEMC	Aug 2013			
APPR	JMAC	Aug 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE A4.1		



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	H4-2	3	66.6	29.0	37.6
■	H4-3	2	55.1	25.8	29.3
▲	H4-3	4	65.5	26.4	39.1
+	H4-4	4	52.0	23.2	28.8
◆	H4-5	3	58.0	29.3	28.7
◇	H4-6	3	53.0	24.4	28.6

PROJECT					HIGHWAY 66/ HWY 66 CONNECTION HIGH FILL H1 / H4 STA 13+110 TO 13+185 / STA 10+000 TO 10+125				
TITLE					PLASTICITY CHART SILTY CLAY to CLAY				
PROJECT No.			10-1191-0044		FILE No.			10-1191-0044SUD.GPJ	
DRAWN	JJL	Aug 2013	CHECK	SEMC	Aug 2013	SCALE	N/A	REV.	
APPR	JMAC	Aug 2013				FIGURE A4.2			



CONSOLIDATION TEST SUMMARY**FIGURE A5**

Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Number	10-1191-0044	Sample Number	4
Borehole Number	H1-6a	Sample Depth, m	4.88

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	1		
Date Started	May 30, 2013		
Date Completed	June 13, 2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.544	Unit Weight, kN/m ³	15.12
Sample Diameter, cm	6.349	Dry Unit Weight, kN/m ³	8.49
Area, cm ²	31.66	Specific Gravity, measure	2.733
Volume, cm ³	80.53	Solids Height, cm	0.805
Water Content, %	78.14	Volume of Solids, cm ³	25.50
Wet Mass, g	124.13	Volume of Voids, cm ³	55.03
Dry Mass, g	69.68		

TEST COMPUTATIONS

Pressure kPa	Primary Consolidation	Corr. Height cm	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	0.00	2.544	2.158	2.544					
9	0.02	2.542	2.156	2.543	194	0.00705	9.24E-05	6.39E-08	0.004
18	0.03	2.538	2.152	2.540	194	0.00704	1.50E-04	8.36E-08	0.019
35	0.11	2.527	2.138	2.533	317	0.00428	2.46E-04	1.03E-07	0.132
69	0.19	2.509	2.115	2.518	290	0.00463	2.16E-04	9.79E-08	0.517
143	0.52	2.457	2.051	2.483	470	0.00278	2.77E-04	7.53E-08	2.700
285	2.76	2.181	1.708	2.319	2089	0.00055	7.63E-04	4.08E-08	26.720
570	1.54	2.027	1.517	2.104	960	0.00098	2.12E-04	2.03E-08	56.920
1140	1.05	1.922	1.387	1.975	866	0.00095	7.22E-05	6.75E-09	101.003
570	-0.09	1.932	1.399	1.927					
143	-0.26	1.958	1.431	1.945					
35	-0.36	1.994	1.476	1.976					
9	-0.27	2.022	1.510	2.008					

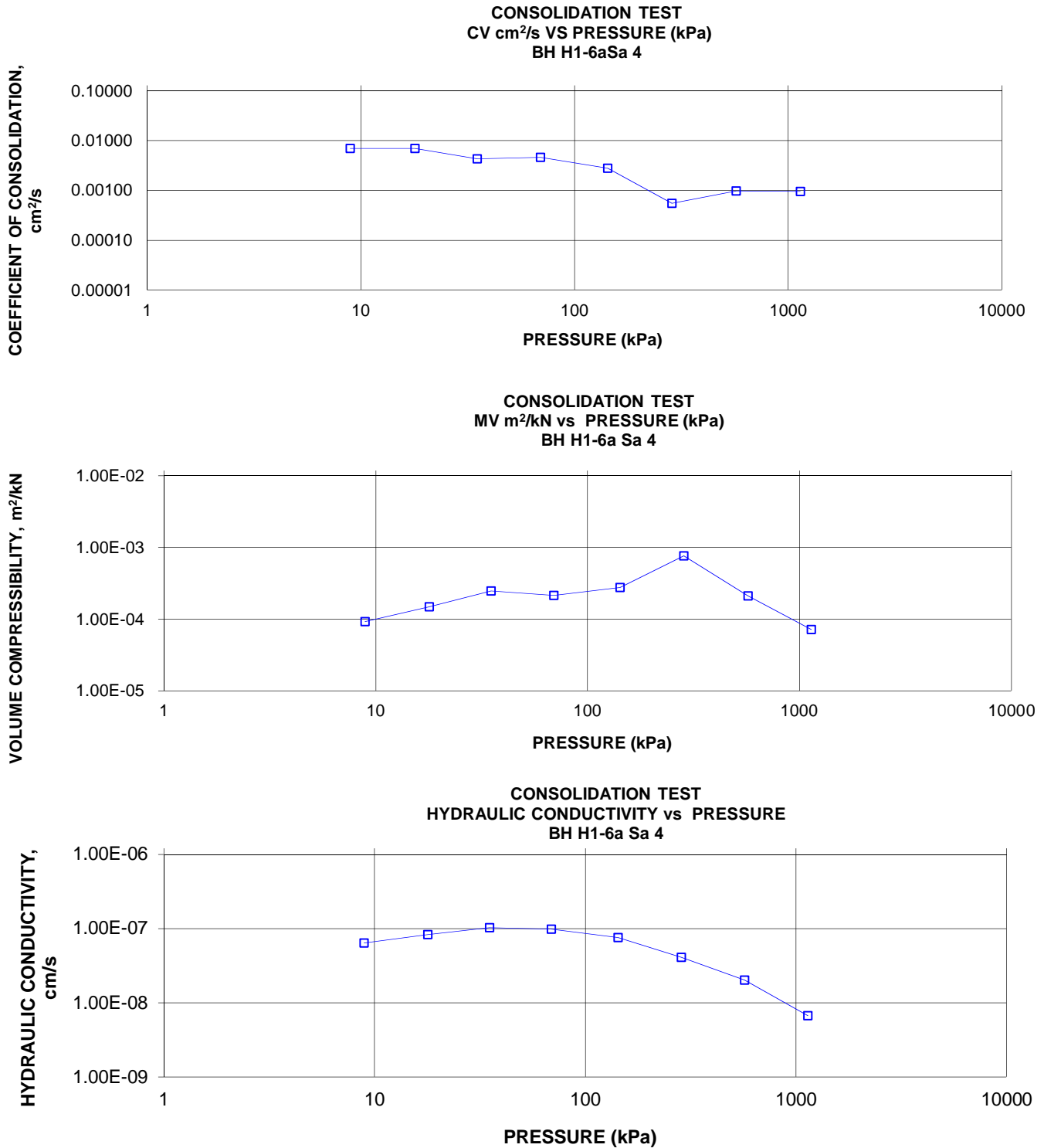
Note:

k calculated using α based on t_{90} values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	2.022	Unit Weight, kN/m ³	15.38
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	10.68
Area, cm ²	31.66	Specific Gravity, measure	2.733
Volume, cm ³	64.00	Solids Height, cm	0.805
Water Content, %	44.04	Volume of Solids, cm ³	25.50
Wet Mass, g	100.37	Volume of Voids, cm ³	38.50
Dry Mass, g	69.68		

CONSOLIDATION TEST SUMMARY

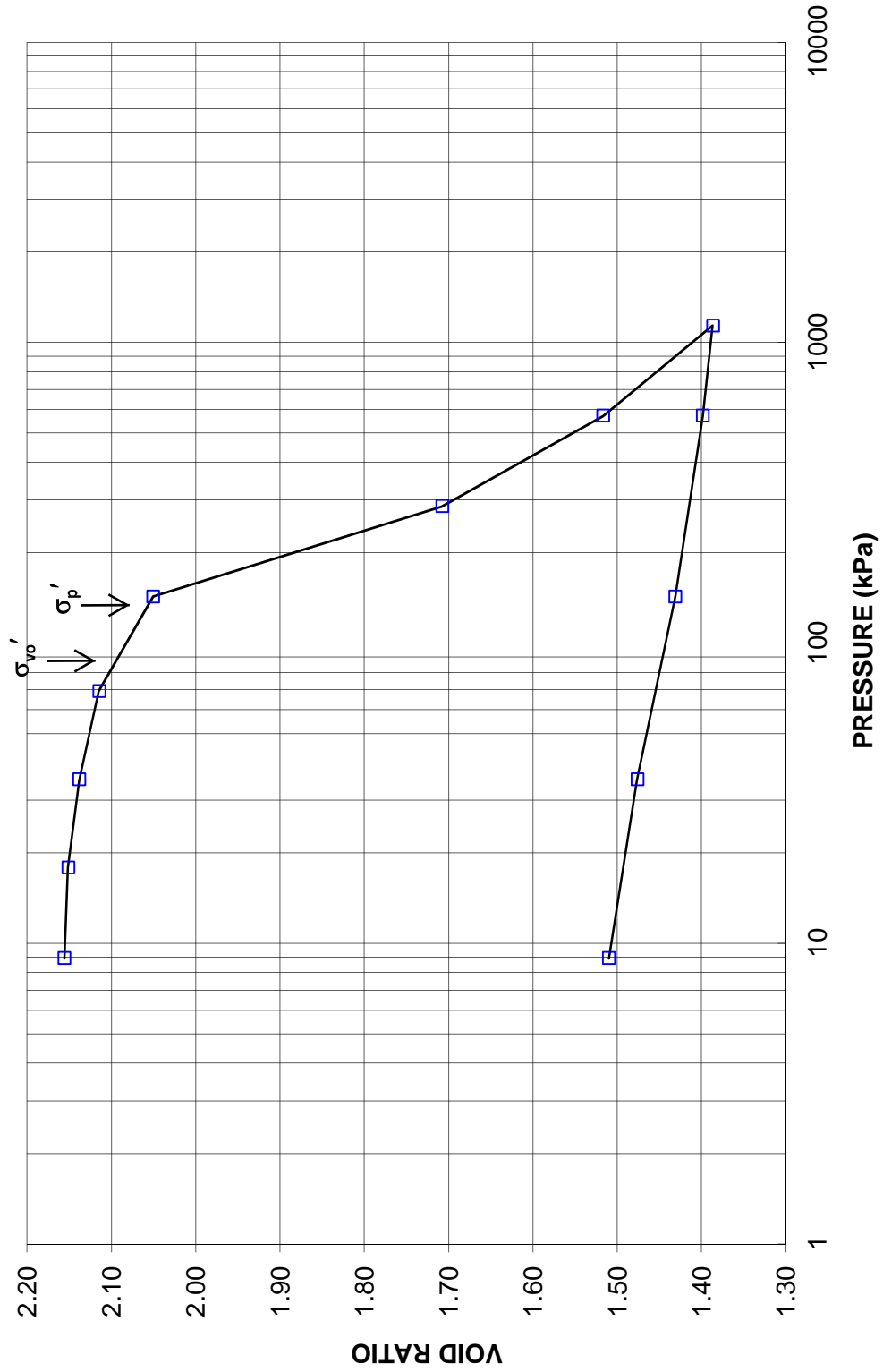
FIGURE A5
Pg. 2 of 4



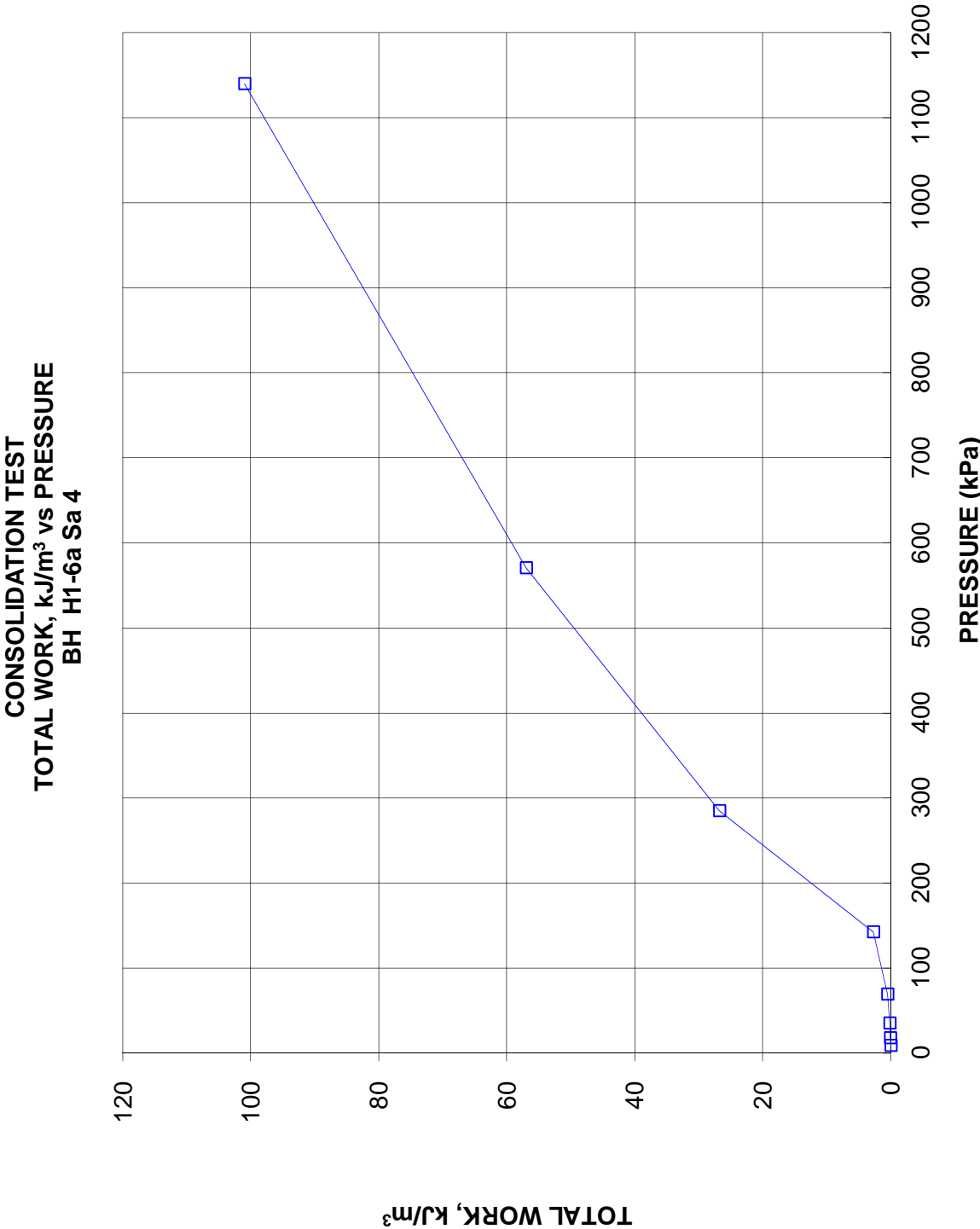
CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE

FIGURE A5
Pg. 3 of 4

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH H1-6a Sa 4



CONSOLIDATION TEST
TOTAL WORK VS PRESSURE



CONSOLIDATION TEST SUMMARY**FIGURE A6****Pg. 1 of 4****SAMPLE IDENTIFICATION**

Project Number:	10-1191-0044	Sample Number:	7
Borehole Number:	H1-9	Sample Depth, m:	6.25

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	May 30, 2013		
Date Completed	June 13, 2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.517	Unit Weight, kN/m ³	15.64
Sample Diameter, cm	6.347	Dry Unit Weight, kN/m ³	9.24
Area, cm ²	31.64	Specific Gravity, Measured	2.748
Volume, cm ³	79.64	Solids Height, cm	0.863
Water Content, %	69.30	Volume of Solids, cm ³	27.29
Wet Mass, g	126.97	Volume of Voids, cm ³	52.35
Dry Mass, g	75.00		

TEST COMPUTATIONS

Pressure kPa	Primary Consolidation	Corr. Height cm	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	2.517	1.918	2.517					
4	0.03	2.514	1.914	2.516	38	0.0349	3.19E-04	1.09E-06	0.003
13	0.04	2.510	1.910	2.512	73	0.0184	1.73E-04	3.13E-07	0.016
31	0.08	2.502	1.900	2.506	66	0.0201	1.84E-04	3.63E-07	0.088
66	0.24	2.478	1.872	2.490	265	0.0050	2.72E-04	1.32E-07	0.558
137	1.60	2.318	1.687	2.398	1500	0.0008	9.00E-04	7.17E-08	7.088
277	1.46	2.172	1.517	2.245	735	0.0015	4.14E-04	5.89E-08	20.146
558	0.81	2.090	1.423	2.131	240	0.0040	1.15E-04	4.53E-08	35.795
1117	0.63	2.028	1.350	2.059	101	0.0089	4.45E-05	3.87E-08	60.919
558	-0.07	2.035	1.359	2.031					
137	-0.20	2.054	1.381	2.045					
31	-0.24	2.079	1.409	2.067					
4	-0.21	2.100	1.434	2.089					

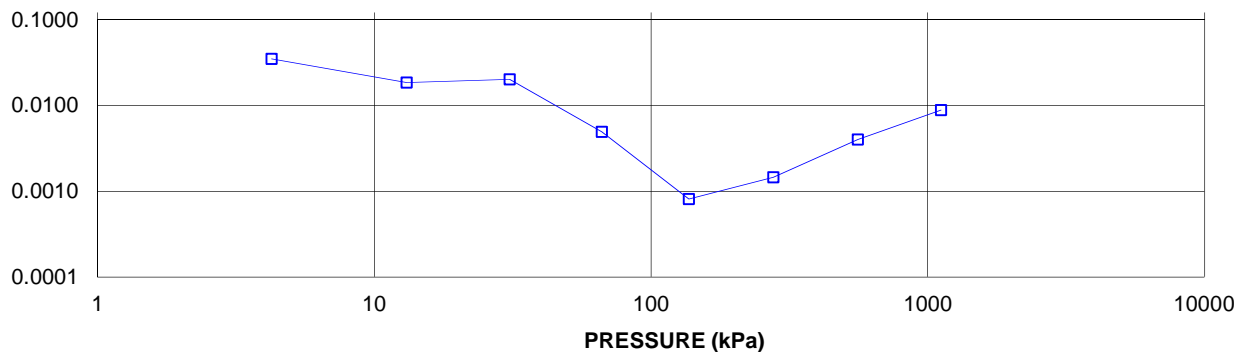
Note:

k calculated using α based on t₉₀ values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	2.100	Unit Weight, kN/m ³	15.53
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	11.07
Area, cm ²	31.64	Specific Gravity, Measured	2.748
Volume, cm ³	66.43	Solids Height, cm	0.863
Water Content, %	40.26	Volume of Solids, cm ³	27.29
Wet Mass, g	105.19	Volume of Voids, cm ³	39.14
Dry Mass, g	75.00		

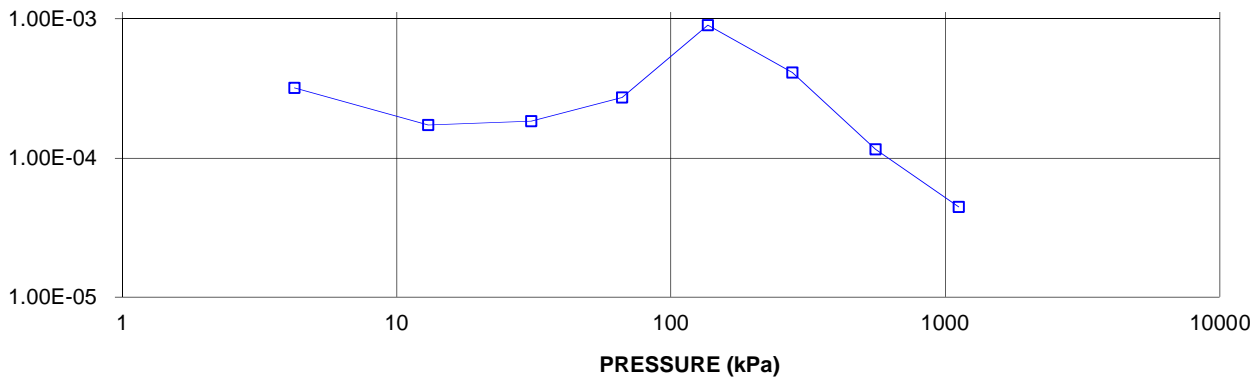
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH H1-9 Sa 7



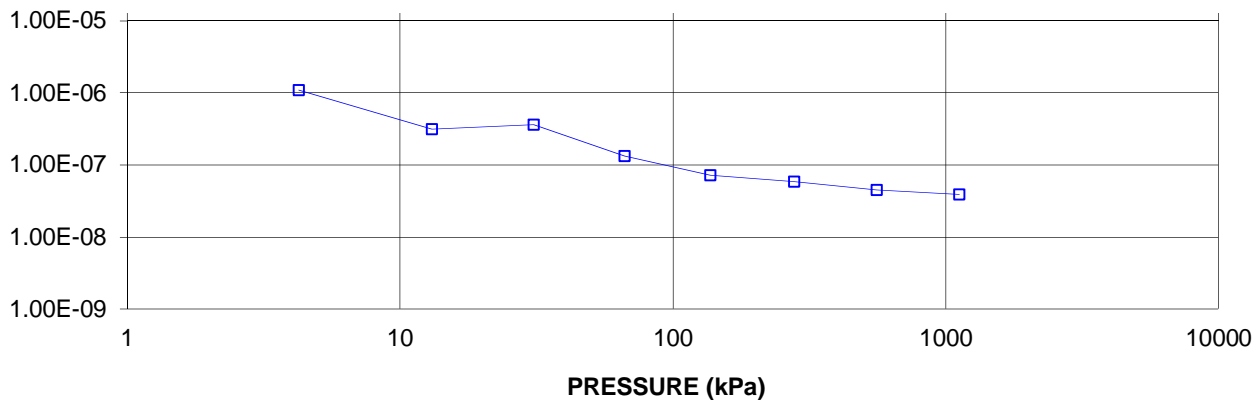
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH H1-9 Sa 7



HYDRAULIC CONDUCTIVITY,
cm/s

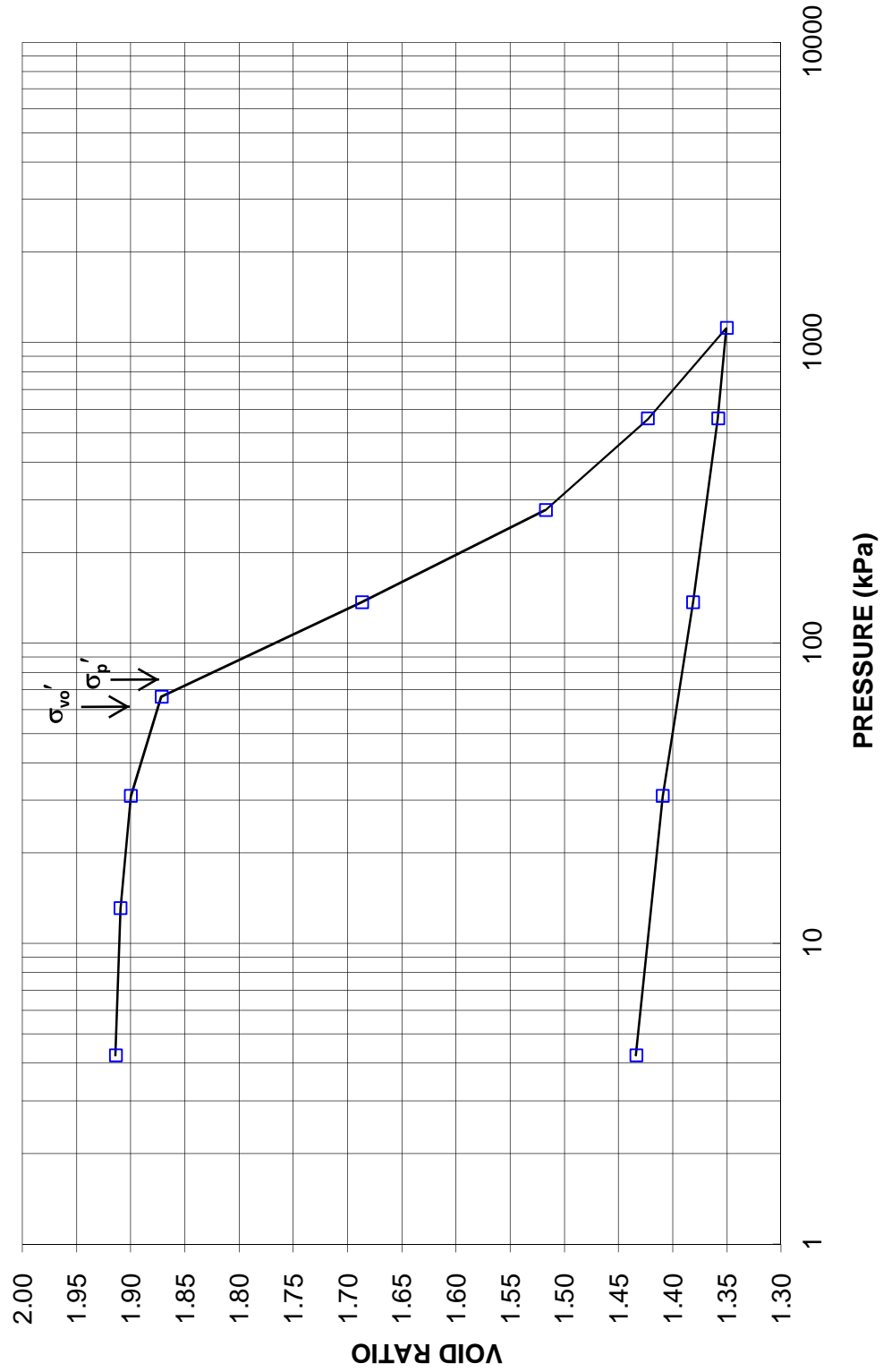
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH H1-9 Sa 7



CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE

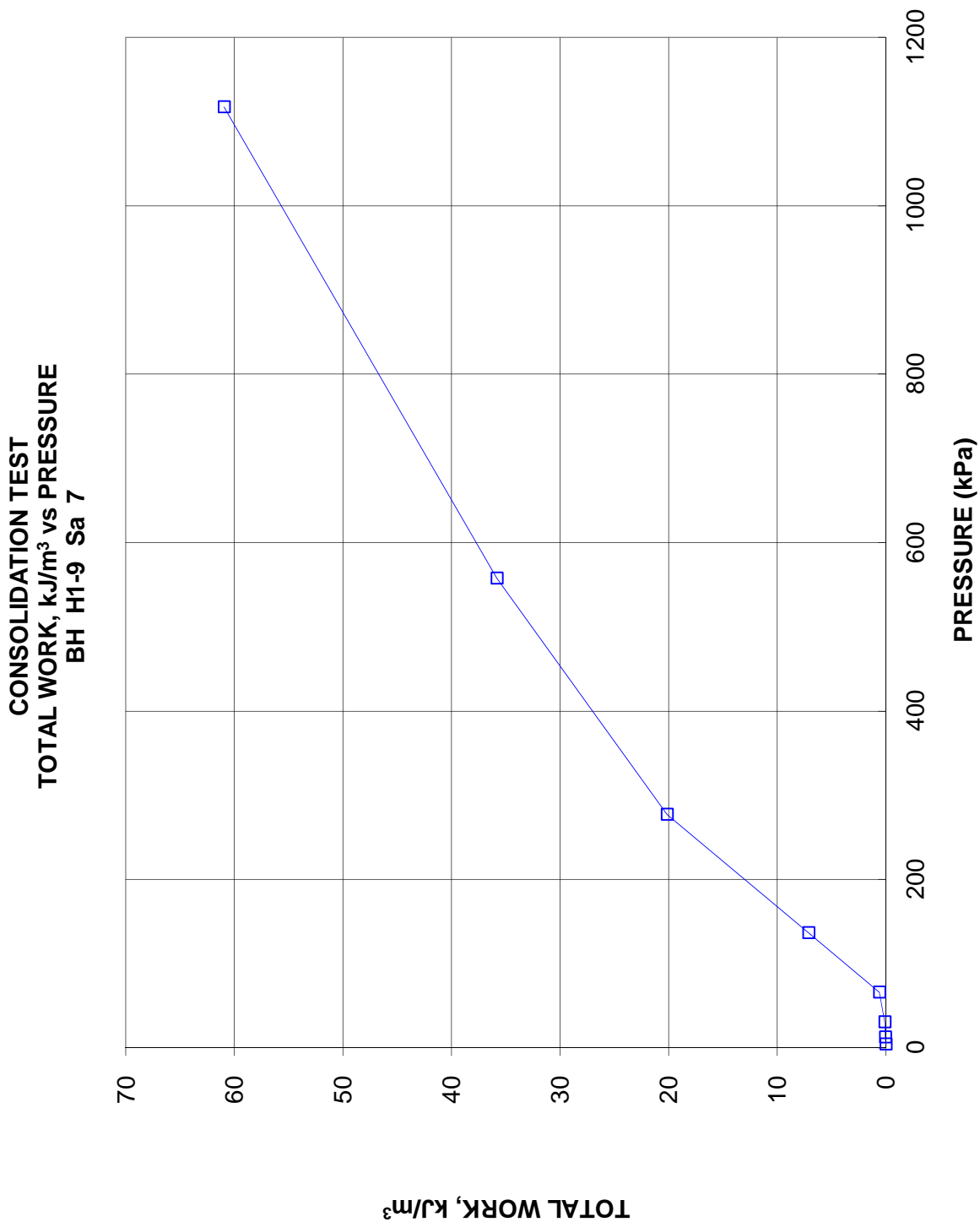
FIGURE A6
Pg. 3 of 4

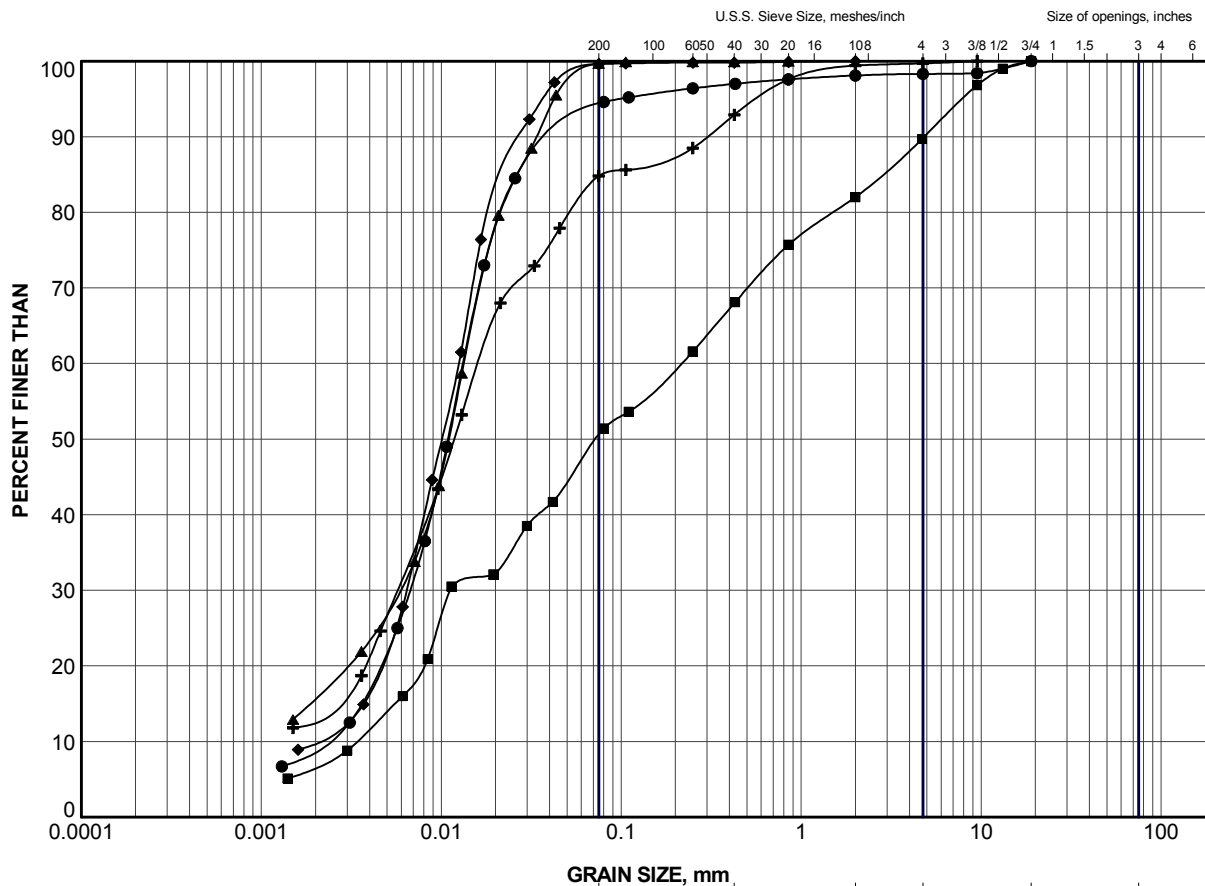
CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH H1-9 Sa 7



CONSOLIDATION TEST TOTAL WORK VS PRESSURE

FIGURE A6
Pg. 4 of 4




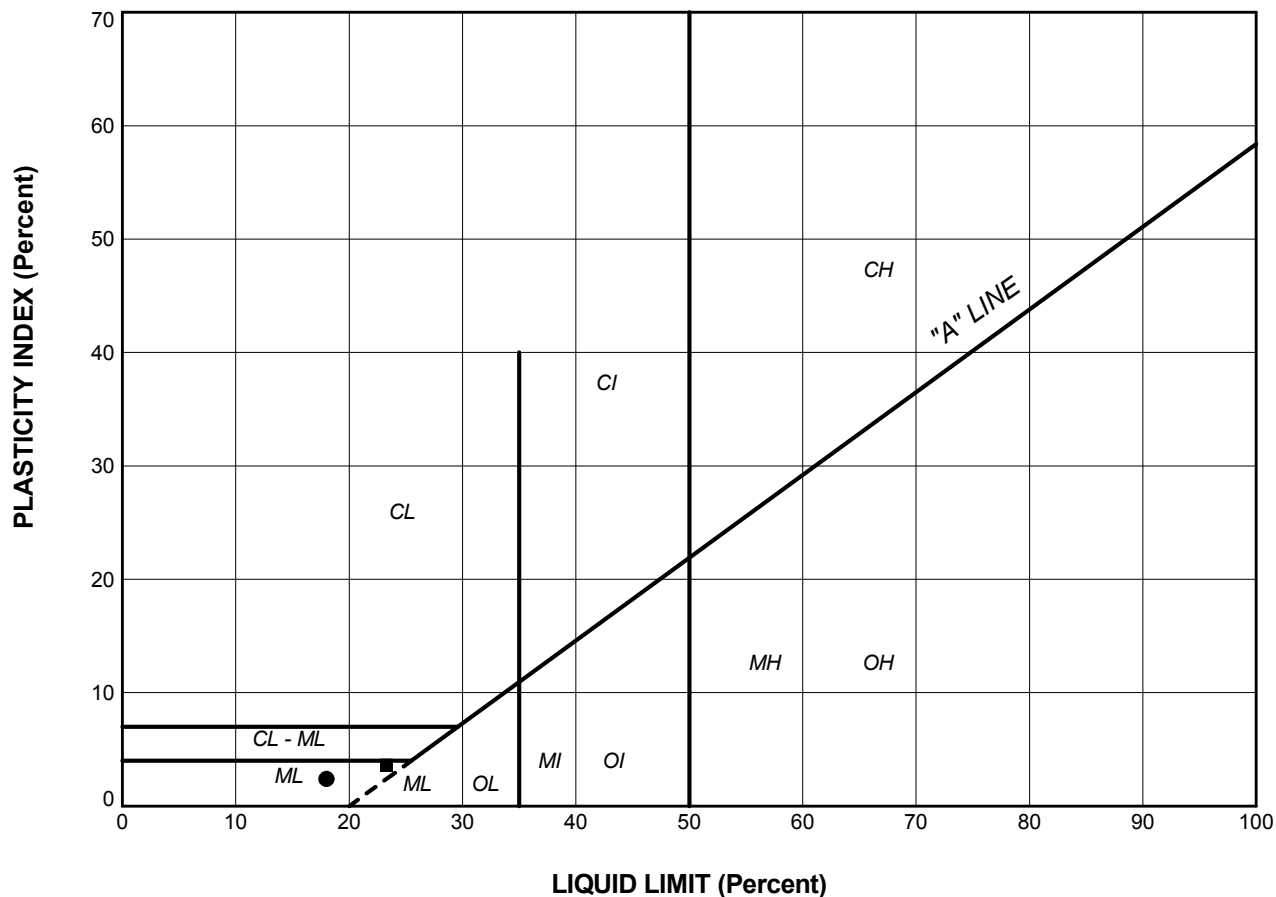


GRAVEL SIZE, mm						Cobble Size
CLAY AND SILT	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

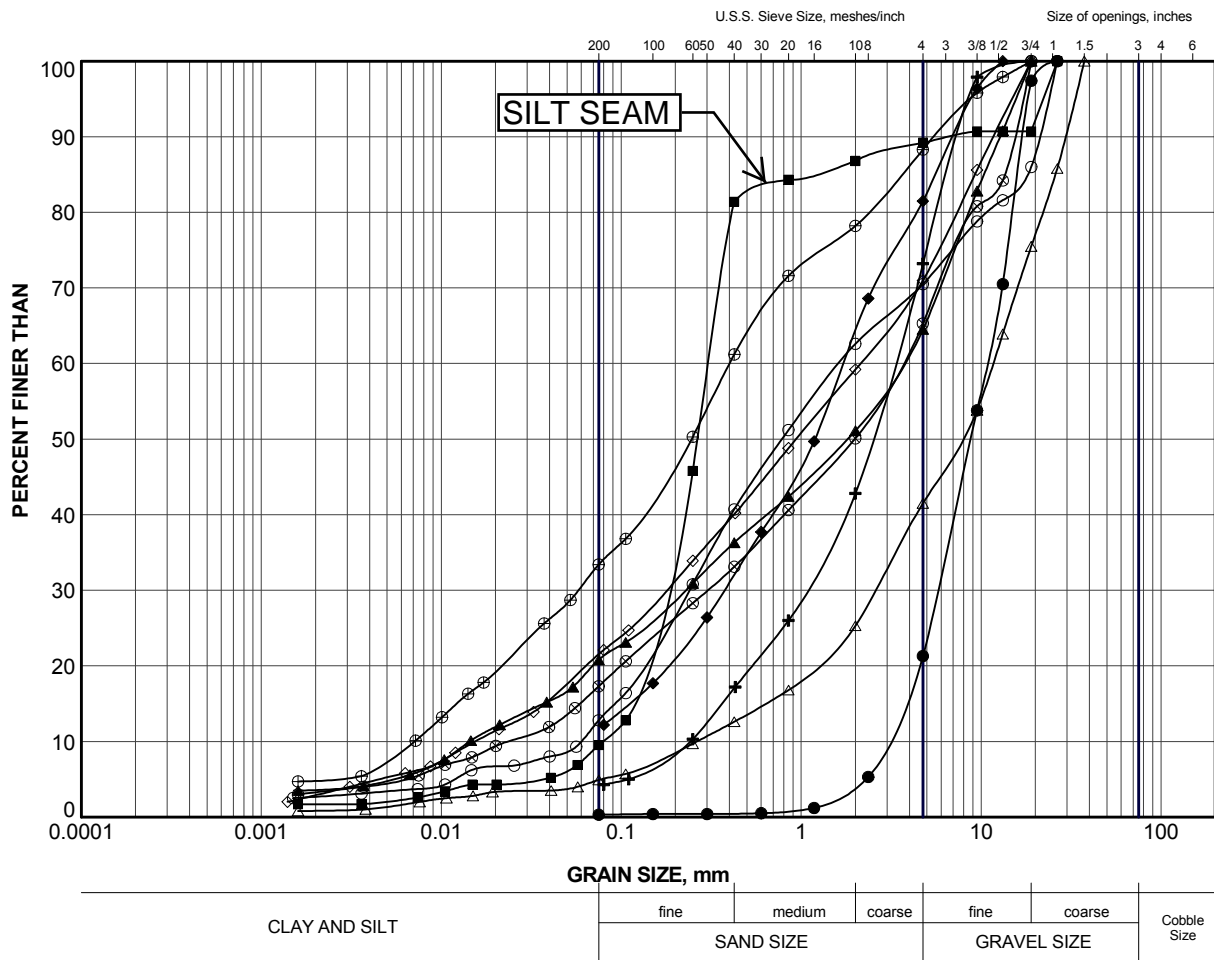
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	H1-4	7	297.9
■	H1-6	13	288.7
▲	H1-9	11	291.4
+	H1-9	13	288.4
◆	H4-5	4b	302.3

PROJECT						HIGHWAY 66/ HWY 66 CONNECTION HIGH FILL H1 / H4 STA 13+110 TO 13+185 / STA 10+000 TO 10+125					
TITLE						GRAIN SIZE DISTRIBUTION SILT to SANDY SILT					
PROJECT No.			10-1191-0044			FILE No.			10-1191-0044SUD.GPJ		
DRAWN	JJL	Aug 2013	SCALE	N/A	REV.						
CHECK	SEMC	Aug 2013									
APPR	JMAC	Aug 2013									
 Golder Associates SUDBURY, ONTARIO			FIGURE A7								



PROJECT			HIGHWAY 66/ HWY 66 CONNECTION HIGH FILL H1 / H4 STA 13+110 TO 13+185 / STA 10+000 TO 10+125		
TITLE			PLASTICITY CHART SILT to SANDY SILT		
PROJECT No.		10-1191-0044	FILE No.		10-1191-0044SUD.GPJ
DRAWN	JJL	Aug 2013	SCALE	N/A	REV.
CHECK	SEMC	Aug 2013	FIGURE A8		
APPR	JMAC	Aug 2013			





LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC1-1	7	297.1
■	BC1-2	7a	301.2
▲	BC1-3	9	294.1
+	C1-2	9	294.4
◆	H1-4	8	296.4
◇	H1-7	7	297.4
○	H1-10	12	288.1
△	H4-1	7	296.9
⊗	H4-2	5	302.5
⊕	H4-6	4	303.2

PROJECT
 HIGHWAY 66/ HWY 66 CONNECTION
 HIGH FILL H1 / H4
 STA 13+110 TO 13+185 / STA 10+000 TO 10+125

TITLE
GRAIN SIZE DISTRIBUTION
 SILTY SAND to SANDY GRAVEL



PROJECT No.	10-1191-0044	FILE No.	10-1191-0044SUD.GPJ
DRAWN	JJL	Aug 2013	SCALE N/A
CHECK	SEMC	Aug 2013	REV.
APPR	JMAC	Aug 2013	

FIGURE A9



Borehole C1-2
Elevation 293.9 m to 289.8 m



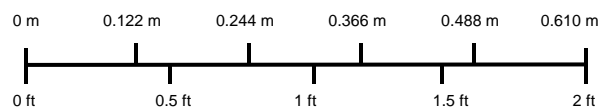
Borehole BC1-1
Elevation 294.2 m to 290.8 m



Borehole BC1-2
Elevation 299.1 m to 295.5 m



Borehole BC1-3
Elevation 292.1 m to 288.5 m



PROJECT HIGHWAY 66 / HIGHWAY 66 CONNECTION HIGH FILL H1 & HIGH FILL H4 STA 13+080 to 13+185 / STA 10+000 to 10+100			
TITLE BEDROCK CORE PHOTOGRAPHS			
PROJECT No. 10-1191-0044		FILE No. ----	
DESIGN	MT	APR 2013	SCALE AS SHOWN REV.
CADD	--		
CHECK	SEMC	APR 2013	
REVIEW	JMAC	APR 2013	

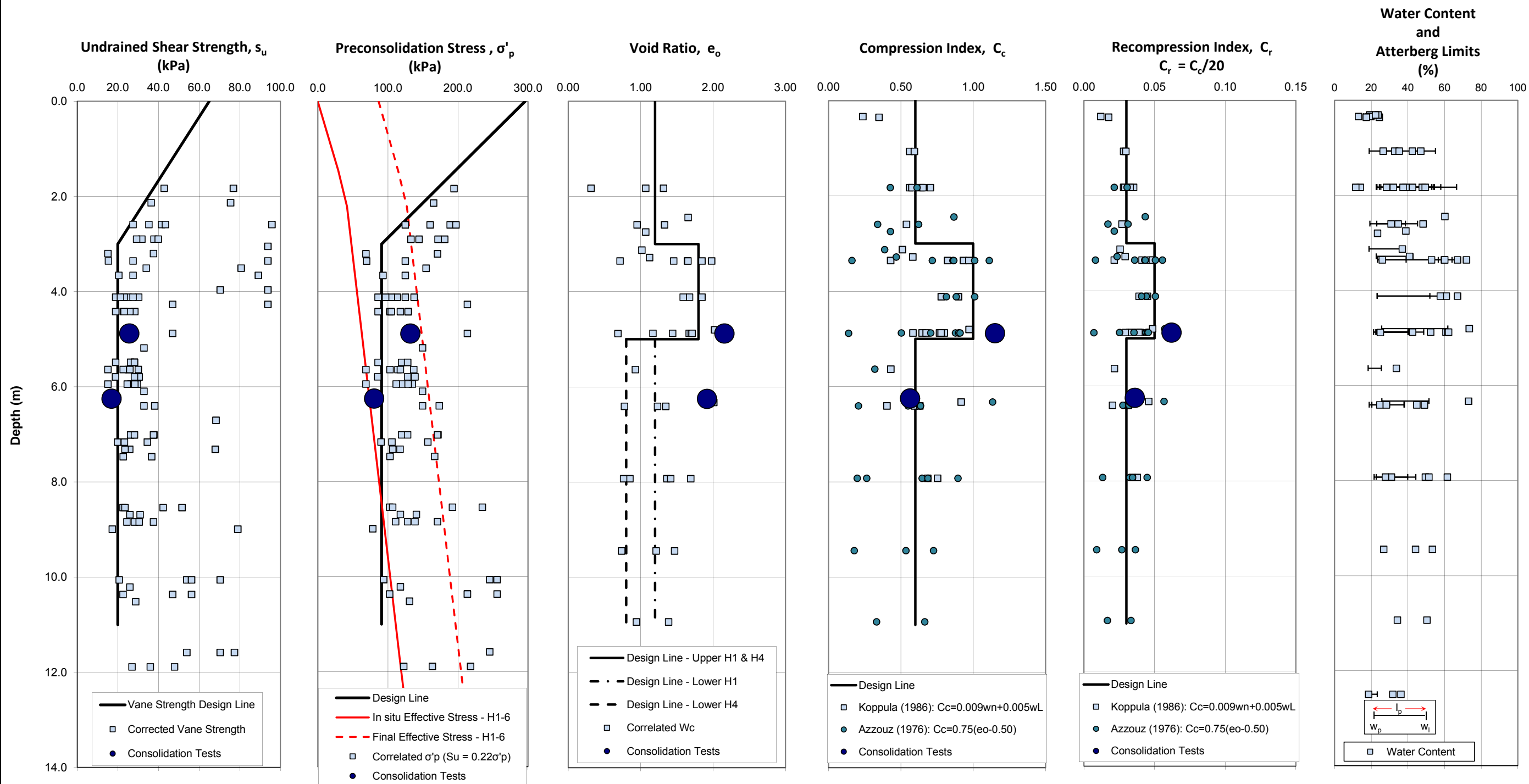


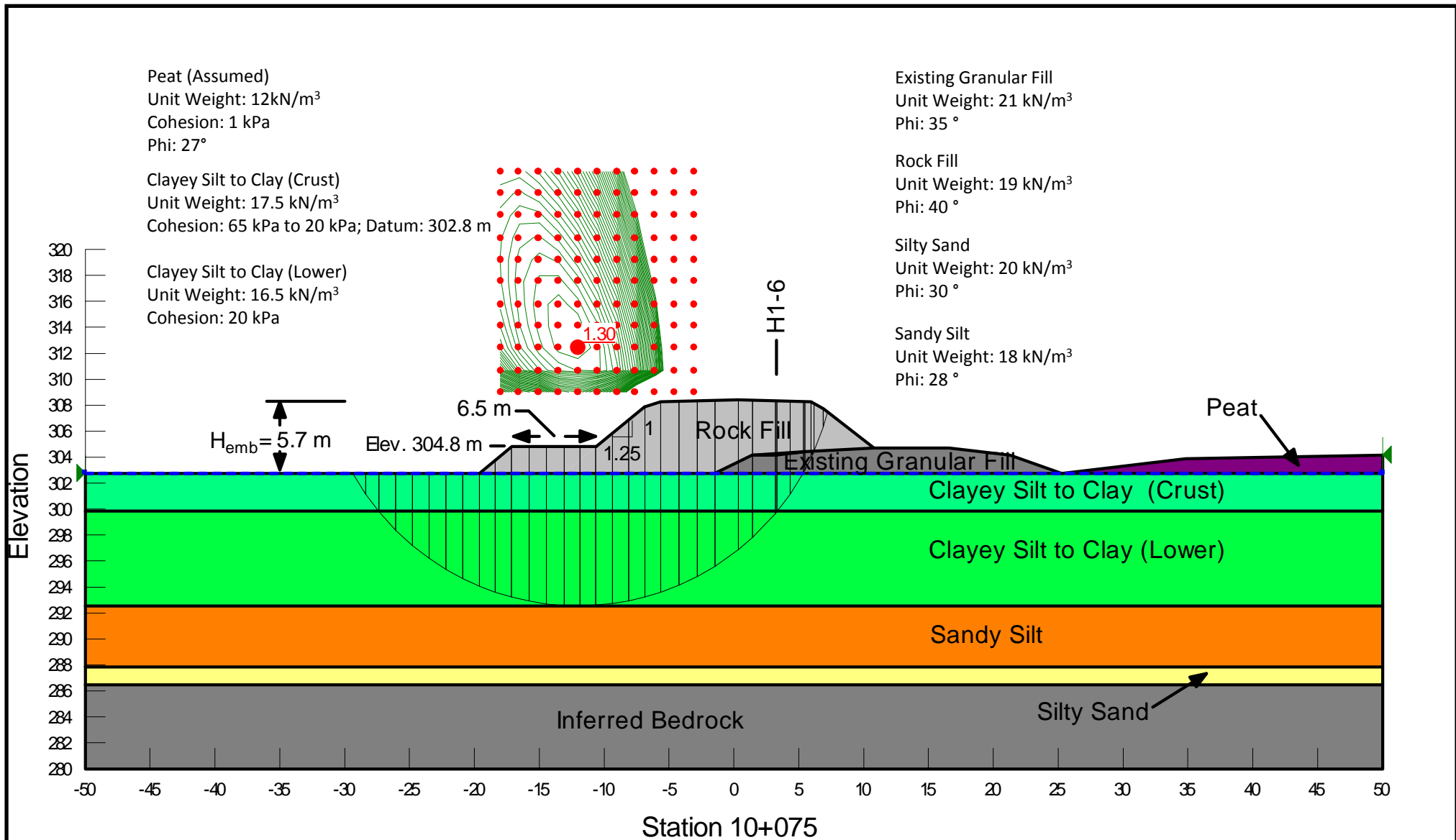
FIGURE A10

N:\Active\2010\1190 Sudbury\1191\10-1191-0044 MRC Hwy 66 Virginiatown\Analyses\High Fill & Swamp Crossing\Parameters\Design Lines and CV\10-1191-0044-Parameters and Design Lines.xls\H1&H4 Plots - Final

**SUMMARY PLOT OF ENGINEERING PARAMETERS FOR
COHESIVE DEPOSITS**
Highway 66 Realignment, Virginiatown - STA 13+080 to 13+185 (H4)
Highway 66 Connection, Virginiatown - 10+000 to 10+125 (H1)

FIGURE A11





PROJECT		HIGHWAY 66 CONNECTION HIGH FILL H1 & H4			
TITLE		STABILITY ANALYSIS NORTH SIDE SLOPE AT STA 10+075			
		PROJECT No. 10-1191-0044	FILE No. ----		
DESIGN	MT	MAY 2013	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	SEMC	MAY 2013	FIGURE A13		
REVIEW	JMAC	MAY 2013			



Peat (Assumed)
Unit Weight: 12 kN/m³
Cohesion: 1 kPa
Phi: 27°

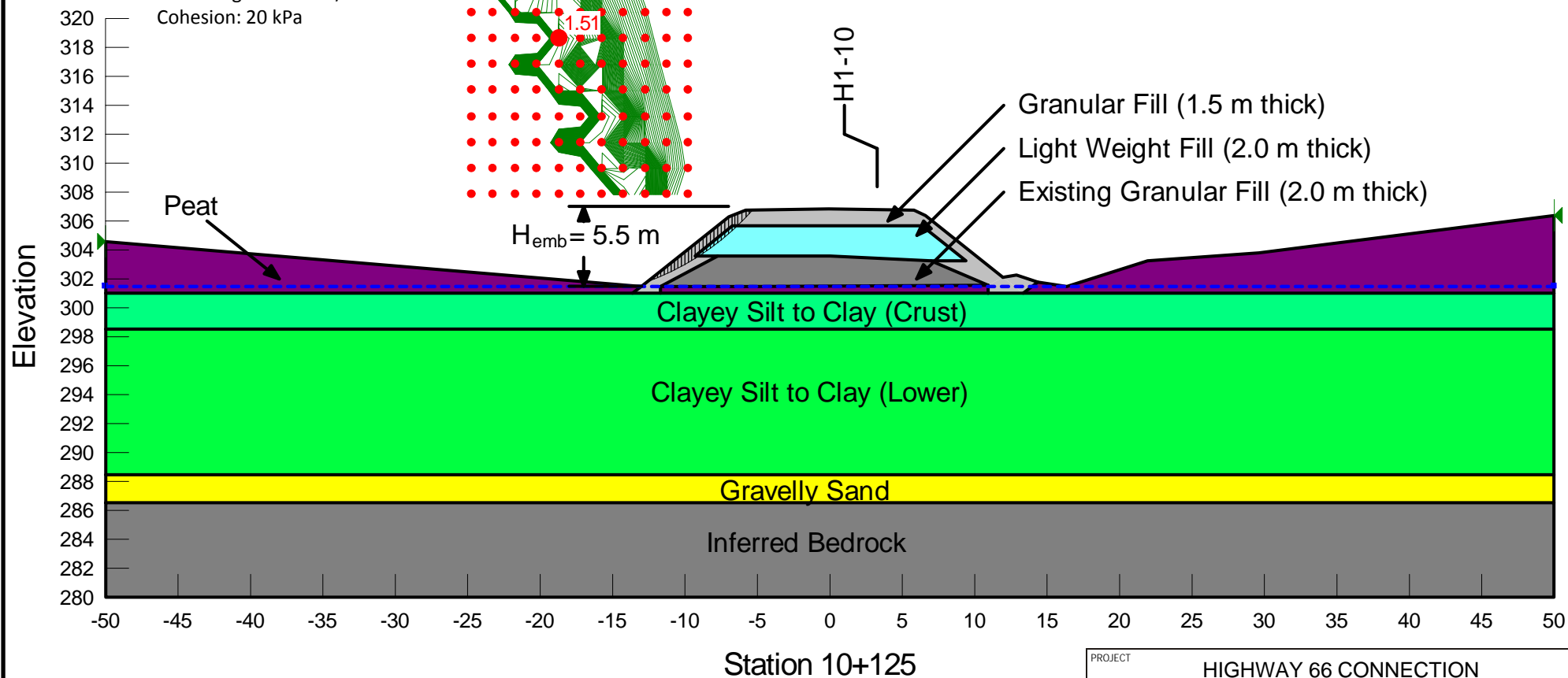
Clayey Silt to Clay (Crust)
Unit Weight: 17.5 kN/m³
Cohesion: 65 kPa to 20 kPa; Datum: 301.5 m

Clayey Silt to Clay (Lower)
Unit Weight: 16.5 kN/m³
Cohesion: 20 kPa

Light Weight Fill
Unit Weight: 0.5 kN/m³
Cohesion: 15 kPa

Granular Fill
Unit Weight: 21 kN/m³
Phi: 35°

Gravelly Sand
Unit Weight: 20 kN/m³
Phi: 30°



PROJECT				HIGHWAY 66 CONNECTION HIGH FILL H1 & H4			
TITLE				STABILITY ANALYSIS SOUTH SIDE SLOPE AT STA 10+125 2.0 m EPS Fill			
PROJECT No. 10-1191-0044		FILE No. ----		SCALE AS SHOWN		REV.	
DESIGN	MT	AUG 2013					
CADD	--						
CHECK	SEMC	AUG 2013					
REVIEW	JMAC	AUG 2013					

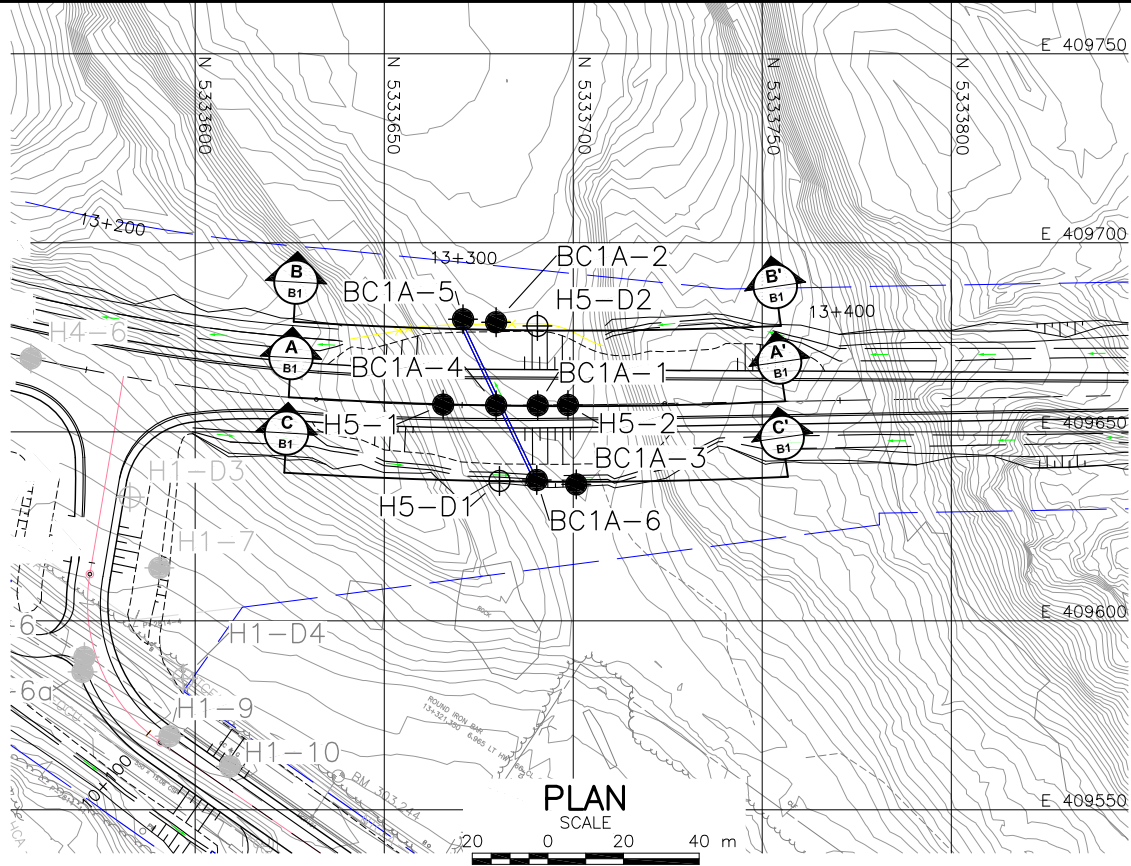


FIGURE A14



APPENDIX B

Highway 66 – STA 13+300 to 13+345 (High Fill H5)



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

CONT No.
GWP No. 5091-07-00

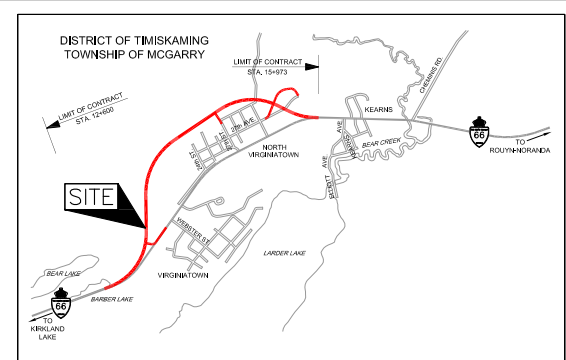


HIGHWAY 66
HWY 66 - STA 13+300 TO 13+345
BOREHOLE LOCATIONS AND
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



KEY PLAN

SCALE
0 700 m

LEGEND

- Borehole
- Dynamic Cone Penetration Test
- 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 upon completion of drilling
- WL in piezometer, measured on NOV 17, 2012

BOREHOLE CO-ORDINATES

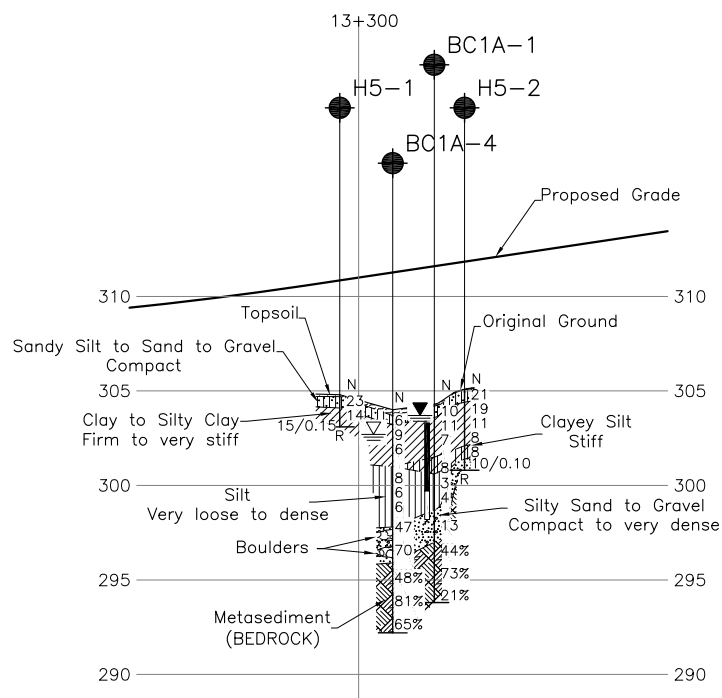
No.	ELEVATION	NORTHING	EASTING
BC1A-1	304.3	5333690.6	409643.0
BC1A-2	303.1	5333679.6	409621.0
BC1A-3	305.7	5333700.8	409663.9
BC1A-4	304.0	5333679.6	409643.0
BC1A-5	303.0	5333670.9	409620.2
BC1A-6	304.9	5333690.4	409662.8
H5-1	304.8	5333665.6	409642.8
H5-2	305.1	5333698.6	409642.9
H5-D1	305.1	5333680.5	409663.0
H5-D2	303.5	5333690.5	409622.0

NOTES

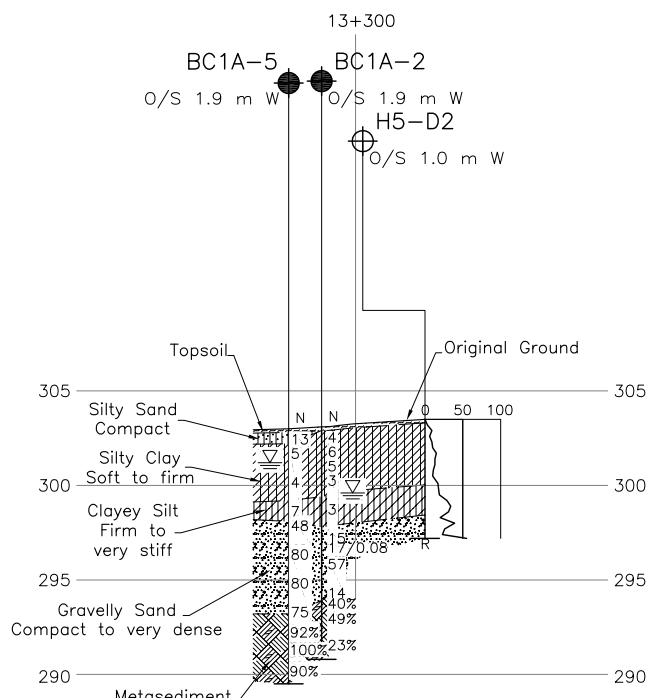
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

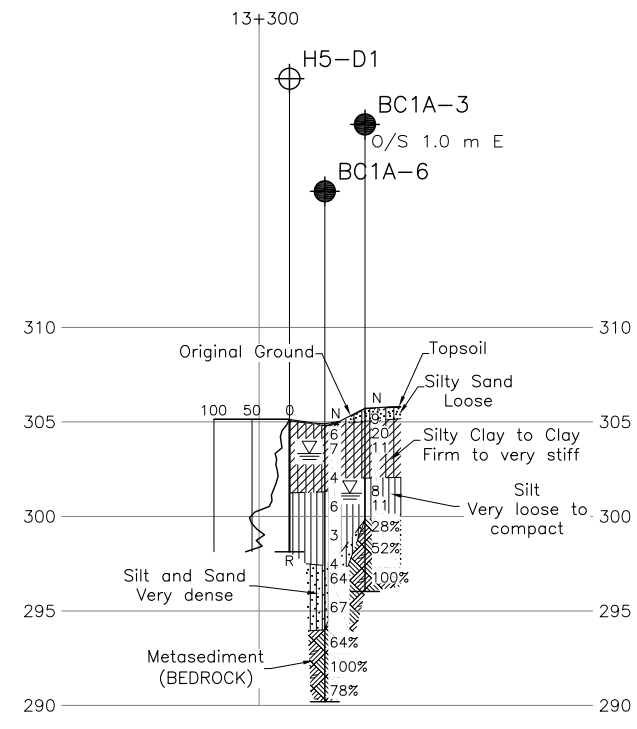
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of GPS General Conditions.



A-A'
B1
PROFILE ALONG CENTRELINE
HIGHWAY 66
HORIZONTAL SCALE
0 20 40 m
VERTICAL SCALE
0 4 8 m



B-B'
B1
PROFILE ALONG NORTH TOE
HIGHWAY 66
HORIZONTAL SCALE
0 20 40 m
VERTICAL SCALE
0 4 8 m



C-C'
B1
PROFILE ALONG SOUTH TOE
HIGHWAY 66
HORIZONTAL SCALE
0 20 40 m
VERTICAL SCALE
0 4 8 m

REFERENCE

Base plans provided in digital format by MMM, drawing file nos. H3211009D16 ROLL PLAN-ULTIMATE and PDR.dwg, received DEC 3, 2012. Keyplan drawing file nos. H3211009G02 received JAN 24, 2013.








NO.	DATE	BY	REVISION
1	DEC 3, 2012	JUL	ISSUED FOR PERMIT
2	JAN 24, 2013	JUL	KEYPLAN
3	DEC 3, 2013	JUL	FINAL

Geocres No. 32D-17

HWY. 66	PROJECT NO. 10-1191-0044	DIST.
SUBM'D. MT	CHKD.	DATE: DEC 2013
DRAWN: JUL	CHKD. SEMC	APPD. JMAC
SITE:		DWG. B1

PROJECT		10-1191-0044				RECORD OF BOREHOLE No H5-1				1 OF 1 METRIC								
G.W.P.		5091-07-00		LOCATION		N 5333665.6; E 409642.8				ORIGINATED BY								
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY								
DATUM		GEODETIC		DATE		September 5, 2012				CHECKED BY								
										SEMC								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)					
304.8	GROUND SURFACE							20	40	60	80	100						
0.0	TOPSOIL		1	SS	23													
304.1	Sandy SILT, trace to some gravel Compact Grey to brown Moist																	
0.7	CLAY, trace sand Stiff Brown Moist		2	SS	14													
303.1			3	SS	15/0.15													
1.7	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING)																	
Note: 1. Borehole dry upon completion of drilling.																		

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044			RECORD OF BOREHOLE No H5-2			1 OF 1 METRIC								
G.W.P. 5091-07-00			LOCATION N 5333698.6; E 409642.9			ORIGINATED BY MT								
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY MT								
DATUM GEODETIC			DATE September 5, 2012			CHECKED BY SEMC								
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						
305.1	GROUND SURFACE							20 40 60 80 100	20 40 60					
0.0	TOPSOIL		1	SS	21		305							
304.4	GRAVEL, some sand, trace silt, trace organics Compact Grey to brown Moist		2	SS	19		304							
0.7	CLAY, trace sand Stiff to very stiff Grey to brown Moist		3	SS	11		303							
			4	SS	8		302							
302.1	CLAYEY SILT, trace sand Stiff Grey Moist		5	SS	8		301							
301.4	SAND and GRAVEL, some silt Compact Brown Moist		6	SS	10/0.10									
300.8														
4.3	Spoon bouncing at 4.1 m depth. END OF BOREHOLE AUGER REFUSAL													
Note: 1. Borehole dry upon completion of drilling.														

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044		RECORD OF BOREHOLE No BC1A-1				1 OF 1 METRIC						
G.W.P. 5091-07-00		LOCATION N 5333690.6; E 409643.0				ORIGINATED BY MT						
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring				COMPILED BY MT						
DATUM GEODETIC		DATE September 6 and 7, 2012				CHECKED BY SEMC						
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL 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	20 40 60 80 100	W _p W W _L			WATER CONTENT (%)
304.3	GROUND SURFACE											
0.0	TOPSOIL		1	SS	10		304					
303.6	SAND, some silt, some gravel, trace clay, trace organics Compact Brown Moist		2	SS	11		303					
0.7	CLAY, trace sand Firm to stiff Grey to brown Moist		3	SS	7		302					
301.5	CLAYEY SILT, trace sand Stiff Grey to brown Moist to wet		4	SS	8		301					
300.6	SILT, some clay, trace sand Very loose Grey Wet		5	SS	3		300					
			6	SS	4		299					
298.7	GRAVEL, some sand, some silt Compact Grey Wet		7	SS	13		298					
	Spoon bouncing at 6.6 m depth. Broken rock encountered between 6.6 m and 7.3 m depth.						297					
297.0	METASEDIMENT (BEDROCK)		1	RC	REC 100%		296					
	Bedrock cored from 7.3 m depth to 10.5 m depth. For coring details see Record of Drillhole BC1A-1.		2	RC	REC 100%		295					
			3	RC	REC 85%		294					
293.8	END OF BOREHOLE											
10.5	Note: 1. Water level at a depth of 1.1 m below ground surface (Elev. 303.2 m) upon completion of drilling. 2. Water level in piezometer at a depth of 1.1 m below ground surface (Elev. 303.2 m) after installation of piezometer, at 0.6 m below ground surface (Elev. 303.7 m) on November 17, 2012 and at 0.8 m below ground surface (Elev. 303.5 m) on May 17, 2013.											

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC1A-1

SHEET 1 OF 1

LOCATION: N 5333690.6 ; E 409643.0

DRILLING DATE: September 7, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling Inc.

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										Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION								
							FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA								HYDRAULIC CONDUCTIVITY k, cm/s							
								TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION						Jr	Ja	Jn	10 ⁰	10 ¹	10 ²	10 ³	
		REFER TO PREVIOUS PAGE		297.0																							
	NW	METASEDIMENT		7.3																							
8		Fine grained			1	GREY	100%																				
		Slightly to moderately weathered																									
		Grey																									
		Highly fractured (sheared) zones																									
		between 7.3 m and 8.2 m depth.																									
9	September 7, 2012				2	GREY	100%																				
	NQ Coring																										
10					3	GREY	100%																				
		END OF DRILLHOLE		293.8																							
11				10.5																							
12																											
13																											
14																											
15																											
16																											
17																											

DEPTH SCALE

1 : 50



LOGGED: MT

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:

PROJECT		10-1191-0044		RECORD OF BOREHOLE No BC1A-2				1 OF 1 METRIC						
G.W.P.		5091-07-00		LOCATION		N 5333679.6; E 409621.0		ORIGINATED BY MT						
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring		COMPILED BY MT						
DATUM		GEODETIC		DATE		September 5 and 6, 2012		CHECKED BY SEMC						
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						
303.1	GROUND SURFACE													
0.0	TOPSOIL													
0.2	SILTY CLAY, trace sand Soft to firm Brown Moist		1	SS	4									
			2	SS	6									
			3	SS	5									
			4	SS	3									
299.4	CLAYEY SILT, trace sand Firm Grey Wet		5	SS	3									
3.7														
297.8	Gravelly SAND, some silt, trace clay Compact to very dense Grey Wet Spoon bouncing at 6.2 m depth. Auger refusal at 6.2 m depth. Casing grinding between 5.9 m and 6.7 m depth.		6	SS	15									
5.3			7	SS	17/0.08									
			8	SS	57									
			9	SS	14									
293.9	METASEDIMENT (BEDROCK)		1	RC	REC 100%									RQD = 40%
9.2	Bedrock cored from 9.2 m depth to 12.3 m depth. For coring details see Record of Drillhole BC1A-2.		2	RC	REC 100%									RQD = 49%
			3	RC	REC 100%									RQD = 23%
290.8	END OF BOREHOLE													
12.3	Note: 1. Water level at a depth of 3.5 m below ground surface (Elev. 299.6 m) upon completion of drilling.													

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC1A-2

SHEET 1 OF 1

LOCATION: N 5333679.6 ; E 409621.0

DRILLING DATE: September 6, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	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.	WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
				DEPTH (m)									FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY			Diametral 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	10 ⁰			10 ¹	10 ²	10 ³																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
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DEPTH SCALE

1 : 50



LOGGED: MT

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:

PROJECT		10-1191-0044		RECORD OF BOREHOLE No BC1A-3		1 OF 1 METRIC								
G.W.P.		5091-07-00		LOCATION		N 5333700.8; E 409663.9								
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring								
DATUM		GEODETIC		DATE		September 8, 2012								
				ORIGINATED BY		MT								
				COMPILED BY		MT								
				CHECKED BY		SEMC								
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						
305.7	GROUND SURFACE							20 40 60 80 100	20 40 60					
0.0	TOPSOIL		1	SS	9									
305.0	Silty SAND, some gravel, trace organics Loose Brown to black Moist		2	SS	20									0 2 24 74
0.7	SILTY CLAY to CLAY, trace sand Firm to very stiff Brown Moist		3	SS	11									
			4	SS	5									
302.0														
3.7	SILT, trace to some clay, trace sand Loose to compact Grey Wet		5	SS	8									
			6	SS	11									0 1 90 9
299.8	METASEDIMENT (BEDROCK)													
5.9	Bedrock cored from 5.9 m depth to 9.7 m depth. For coring details see Record of Drillhole BC1A-3.		1	RC	REC 100%									RQD = 28%
			2	RC	REC 91%									RQD = 52%
			3	RC	REC 100%									RQD = 100%
296.0	END OF BOREHOLE													
9.7	Note: 1. Water level at a depth of 4.5 m below ground surface (Elev. 301.2 m) upon completion of drilling.													

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC1A-3

SHEET 1 OF 1

LOCATION: N 5333700.8 ; E 409663.9

DRILLING DATE: September 8, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling Inc.

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.	NOTES WATER LEVELS INSTRUMENTATION
							RECOVERY				FRACT. INDEX METRES				DISCONTINUITY DATA								HYDRAULIC CONDUCTIVITY				Diametral Point Load																													
							TOTAL CORE %		SOLID CORE %		R.Q.D. %		INDEX METRES		TYPE AND SURFACE DESCRIPTION				k, cm/s				Index (MPa)																																	
							FLUSH								B Angle	DIP w.r.t. CORE AXIS					Jr	Ja	Jn	10 ⁰	10 ¹	10 ²	10 ³	10 ⁰	10 ¹	10 ²	10 ³	RMC -Q' AVG.																								
6	NW	REFER TO PREVIOUS PAGE METASEDIMENT Strong Weak to moderately foliated Fine grained Moderately weathered to fresh Grey Highly fractured between 5.9 m and 6.4 m depth.		299.8 5.9		GREY - LIGHT GREY 100%																																																		
7					1	GREY - LIGHT GREY 100%																																																		
8					2	GREY 91%																																																		
9					3	GREY 100%																						63 MPa																												
10		END OF DRILLHOLE		296.0 9.7																																																				
11																																																								
12																																																								
13																																																								
14																																																								
15																																																								

DEPTH SCALE

1 : 50



LOGGED: MT

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:

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

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC1A-4

SHEET 1 OF 1

LOCATION: N 5333679.6 ; E 409643.0

DRILLING DATE: May 17, 2013

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR FLUSH % 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.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
							RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA										HYDRAULIC CONDUCTIVITY		Diameter 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																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
							888888	888888			0°-1°	0°-1°	0°-1°	0°-1°	0°-1°	0°-1°	0°-1°	0°-1°	0°-1°	0°-1°	10 ⁻⁶	10 ⁻⁶																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
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DEPTH SCALE

1 : 50



LOGGED: MR

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:

PROJECT 10-1191-0044		RECORD OF BOREHOLE No BC1A-5				1 OF 1 METRIC							
G.W.P. 5091-07-00		LOCATION N 5333670.9; E 409620.2				ORIGINATED BY MR							
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring				COMPILED BY MT							
DATUM GEODETIC		DATE May 18, 2013				CHECKED BY SEMC							
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					
303.0	GROUND SURFACE						20 40 60 80 100	20 40 60					
0.0	TOPSOIL		1a	SS	13								
0.2	Silty SAND		1b										
302.2	Compact Brown/grey Moist												
0.8	SILTY CLAY to CLAY Firm to very stiff Grey/brown Wet		2	SS	5								
			3	SS	4								
299.2	SILT, trace to some clay Loose Grey Wet		4	SS	7								
298.1	Gravelly Silty SAND, trace clay Dense to very dense Brown/grey Wet		5a	SS	48								
4.9			5b										
	Cobbles encountered from 5.6 m to 5.8 m depth.												
			6	SS	80								
			7	SS	80								
			8	SS	75								
293.2	METASEDIMENT (BEDROCK)												
9.8	Bedrock cored from 9.8 m depth to 13.5 m depth.		1	RC	REC 100%								
	For coring details see record of borehole BC1A-5.												
			2	RC	REC 100%								
			3	RC	REC 100%								
289.5	END OF BOREHOLE												
13.5	Note: 1. Water level at a depth of 1.8 m below ground surface (Elev. 301.2 m) upon completion of drilling.												

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

SHEET 1 OF 1

DATUM: GEODETIC

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:

[illegible]

Continued Next Page

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

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC1A-6

SHEET 1 OF 1

LOCATION: N 5333690.4 ;E 409662.8

DRILLING DATE: May 18, 2013

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

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.	NOTES WATER LEVELS INSTRUMENTATION
							RECOVERY				R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA												HYDRAULIC CONDUCTIVITY				Diameter Point Load Index (MPa)	RMC -Q' AVG.																											
							FLUSH	TOTAL CORE %	SOLID CORE %	B Angle			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION												Jr	Ja	Jn			k, cm/s																										
														0 0																																											

DEPTH SCALE

1 : 50



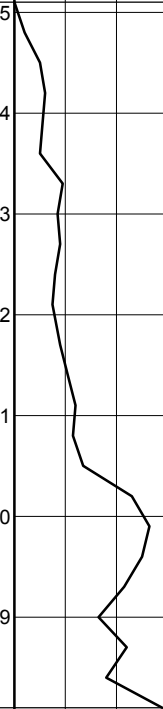
LOGGED: MR

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No BC1A-6				2 OF 2 METRIC																
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5333690.4; E 409662.8</u>				ORIGINATED BY <u>MR</u>																
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u>				COMPILED BY <u>MT</u>																
DATUM <u>GEODETIC</u>		DATE <u>May 18, 2013</u>				CHECKED BY <u>SEMC</u>																
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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; font-size: small;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between; font-size: x-small;"> ○ UNCONFINED ○ FIELD VANE </div> <div style="display: flex; justify-content: space-between; font-size: x-small;"> ● QUICK TRIAXIAL × REMOULDED </div>															
	END OF BOREHOLE Note: 1. Water level at a depth of 1.6 m below ground surface (Elev. 303.3 m) upon completion of drilling.																					

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

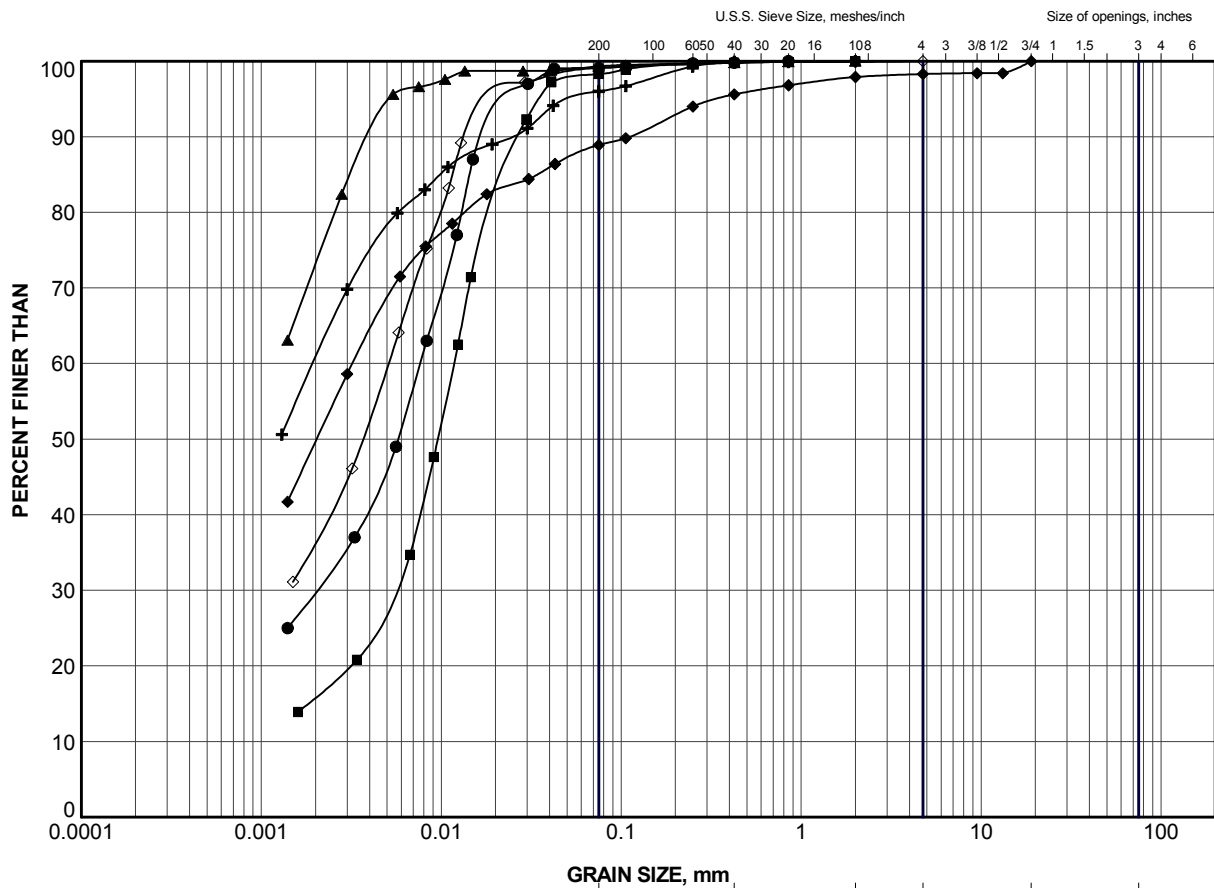
PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST No H5-D1				1 OF 1 METRIC							
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5333680.5; E 409663.0</u>				ORIGINATED BY <u>MT</u>							
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>TR</u>							
DATUM <u>GEODETIC</u>		DATE <u>September 5, 2012</u>				CHECKED BY <u>SEMC</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					
305.1 0.0	GROUND SURFACE						<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>						
						305							
						304							
						303							
						302							
						301							
						300							
298.1 7.0	END OF DCPT REFUSAL TO FURTHER PENETRATION 58 BLOWS / 0.28 m (HAMMER BOUNCING)					299							

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



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


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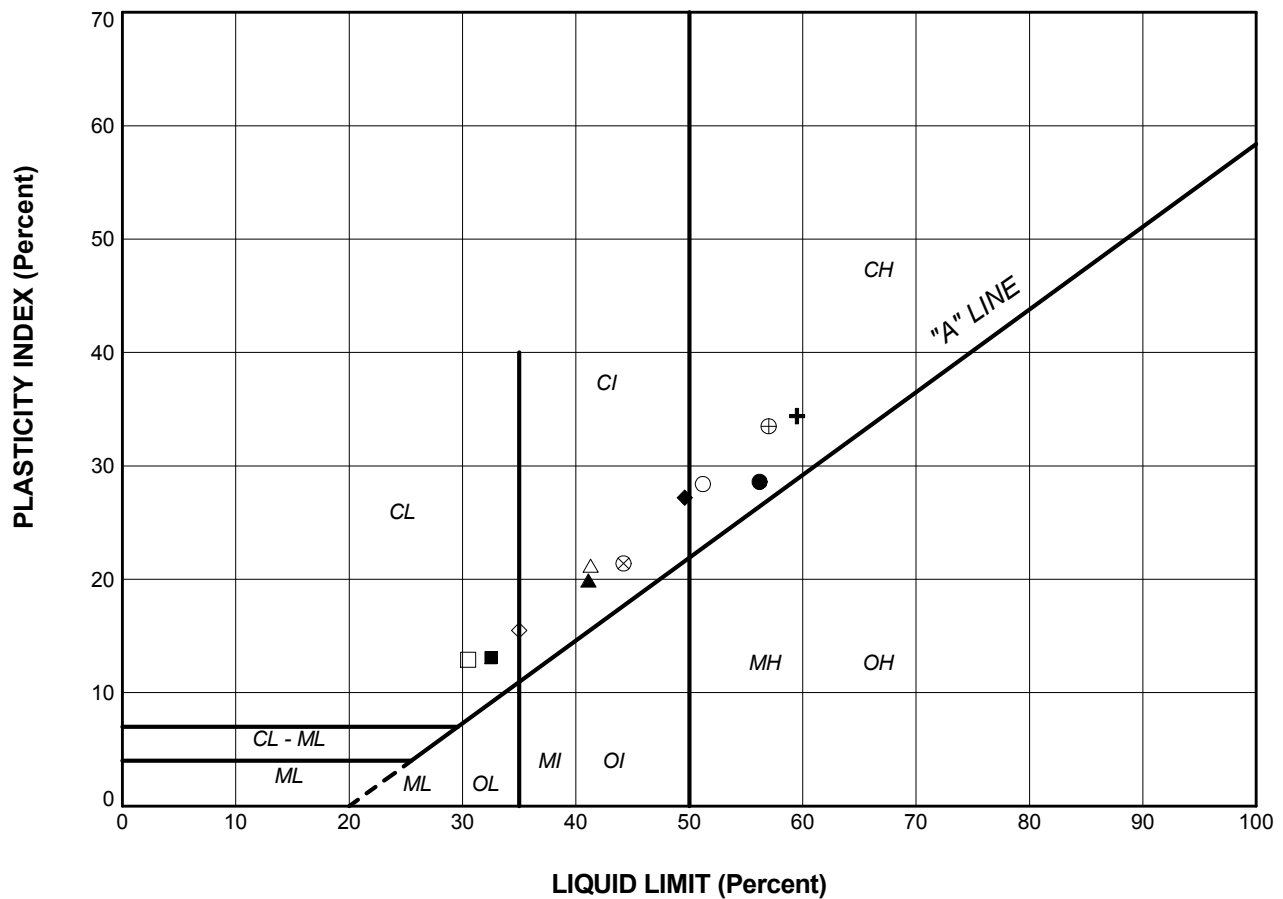


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC1A-1	4	300.9
■	BC1A-2	5	299.0
▲	BC1A-3	2	304.7
+	BC1A-4	2	302.9
◆	BC1A-6	1	304.4
◇	H5-1	2	303.7

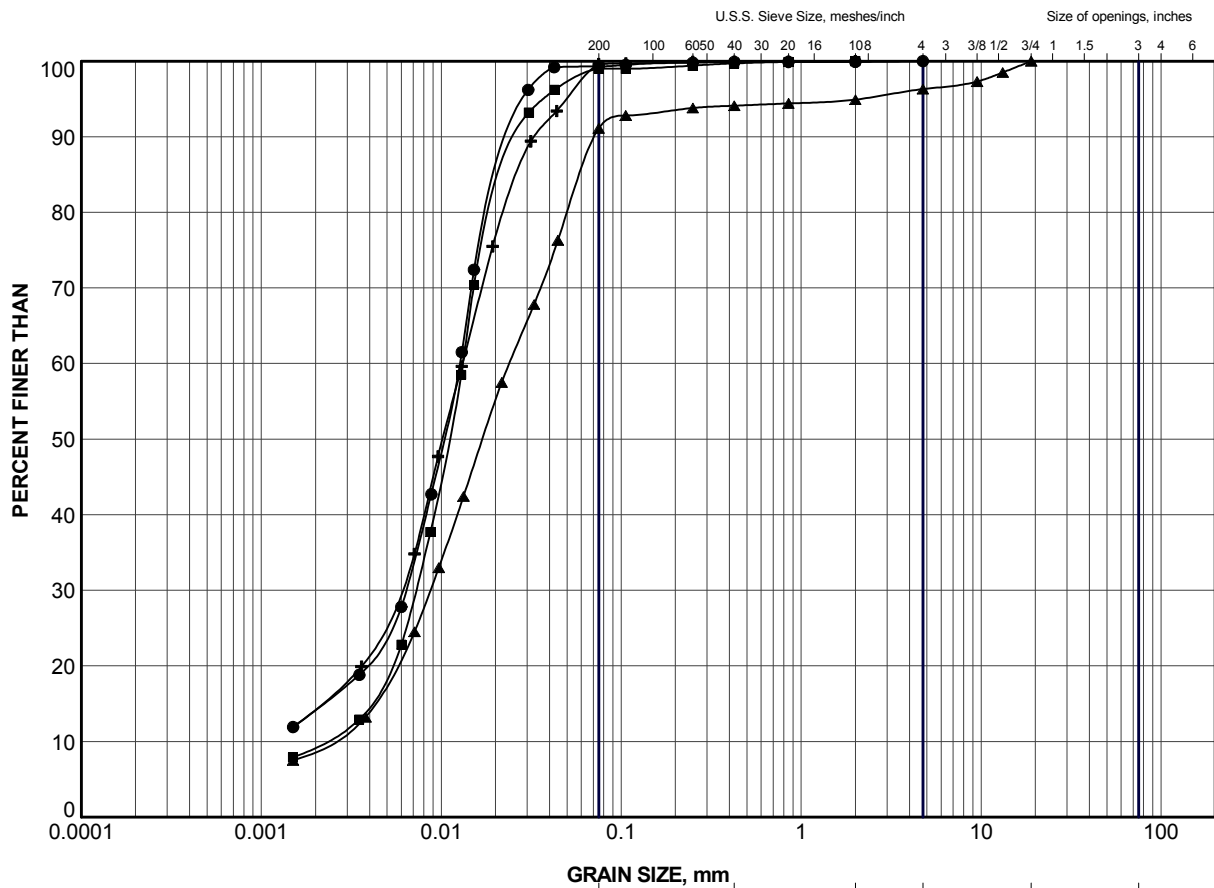
PROJECT					
HIGHWAY 66 - HIGH FILL H5 STA 13+300 TO 13+345					
TITLE					
GRAIN SIZE DISTRIBUTION CLAYEY SILT to CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013			
APPR	JMAC	Jul 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE B1		



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BC1A-1	2	56.2	27.6	28.6
■	BC1A-1	4	32.5	19.4	13.1
▲	BC1A-2	3	41.1	21.2	19.9
+	BC1A-3	2	59.5	25.1	34.4
◆	BC1A-3	4	49.6	22.4	27.2
◇	BC1A-4	2	35.0	19.5	15.5
○	BC1A-5	3	51.2	22.8	28.4
△	BC1A-6	1b	41.3	20.1	21.2
⊗	BC1A-6	3	44.2	22.8	21.4
⊕	H5-2	3	57.0	23.5	33.5
□	H5-2	5	30.5	17.6	12.9

PROJECT					
HIGHWAY 66 - HIGH FILL H5 STA 13+300 TO 13+345					
TITLE					
PLASTICITY CHART CLAYEY SILT to CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013			
APPR	JMAC	Jul 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE B2		



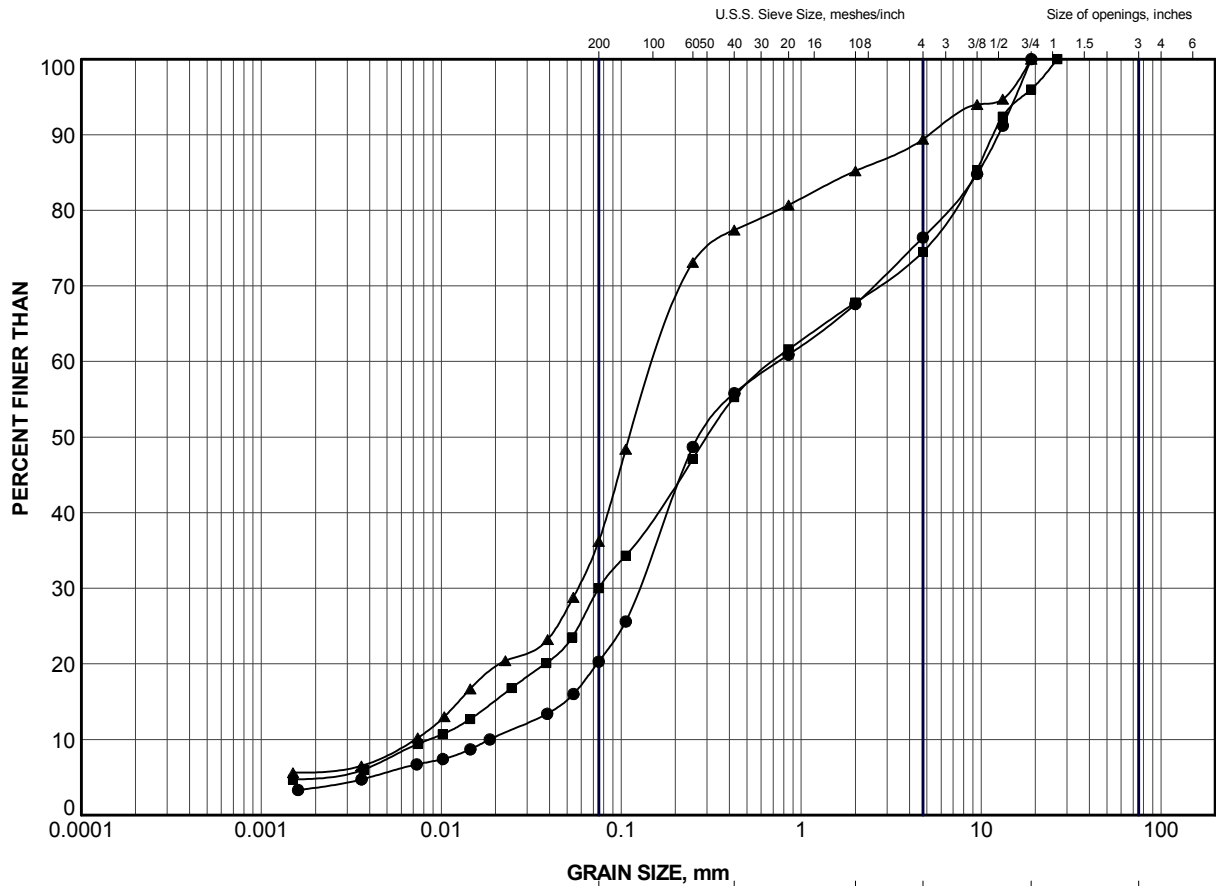
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC1A-1	6	299.4
■	BC1A-3	6	300.8
▲	BC1A-4	7	298.1
+	BC1A-6	5	299.3

PROJECT					
HIGHWAY 66 - HIGH FILL H5 STA 13+300 TO 13+345					
TITLE					
GRAIN SIZE DISTRIBUTION SILT					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013			
APPR	JMAC	Jul 2013			
			FIGURE B3		






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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC1A-2	8	296.1
■	BC1A-5	7	295.1
▲	BC1A-6	7	297.0

PROJECT					
HIGHWAY 66 - HIGH FILL H5 STA 13+300 TO 13+345					
TITLE					
GRAIN SIZE DISTRIBUTION SILTY SAND to GRAVELLY SAND					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013			
APPR	JMAC	Jul 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE B4		



Borehole BC1A-1
Elevation 297.0 m to 293.8 m




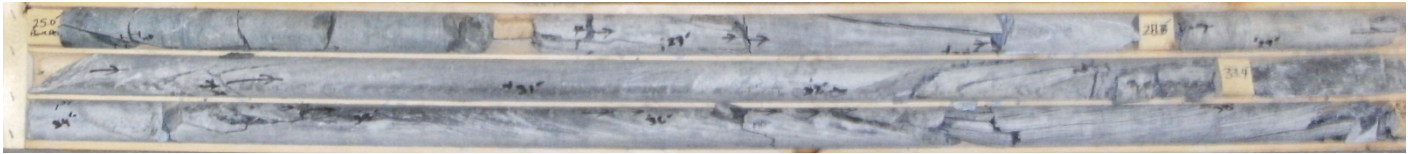
Borehole BC1A-2
Elevation 293.9 m to 290.8 m



Borehole BC1A-3
Elevation 299.8 m to 296.0 m



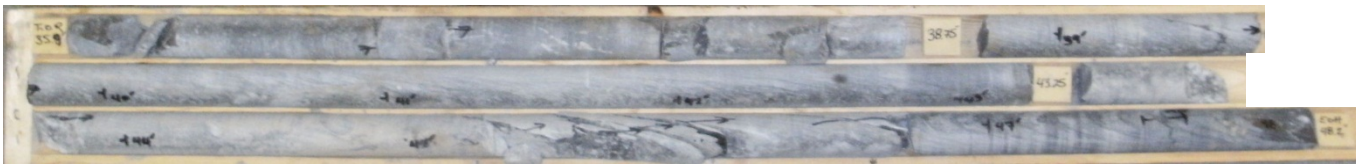
PROJECT		HIGHWAY 66 HIGH FILL H5 STA 13+300 to 13+345			
TITLE		BEDROCK CORE PHOTOGRAPHS			
	PROJECT No.	10-1191-0044		FILE No. ----	
	DESIGN	MT	AUG 2013	SCALE	AS SHOWN
	CADD	--		REV.	
	CHECK	SEMC	AUG 2013	FIGURE B5.1	
	REVIEW	JMAC	AUG 2013		



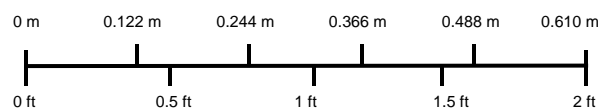
Borehole BC1A-4
Elevation 295.9 m to 292.2 m



Borehole BC1A-5
Elevation 293.2 m to 289.5 m



Borehole BC1A-6
Elevation 294.0 m to 290.2 m




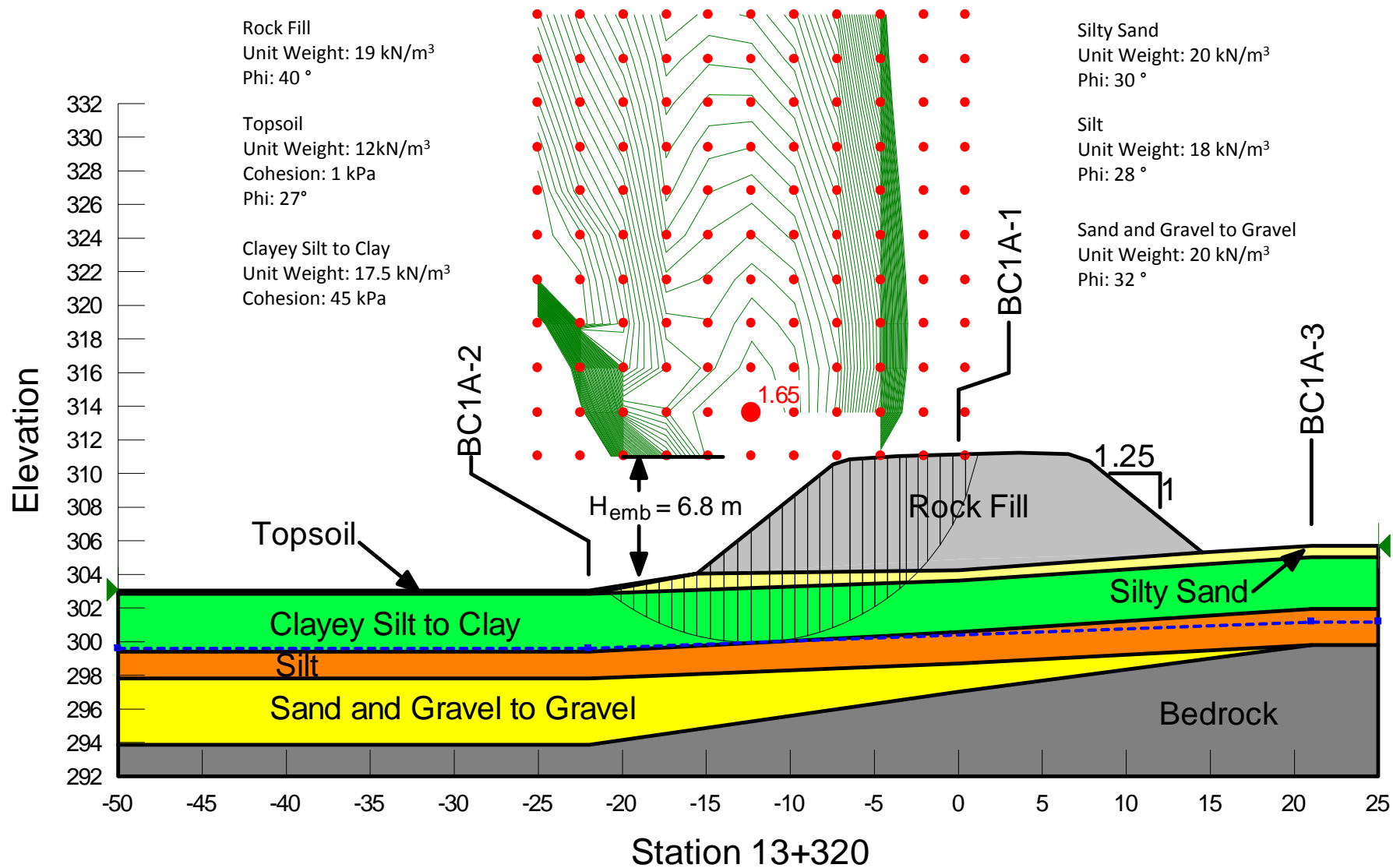
PROJECT		HIGHWAY 66 HIGH FILL H5 STA 13+300 to 13+345			
TITLE		BEDROCK CORE PHOTOGRAPHS			
	PROJECT No. 10-1191-0044			FILE No. ----	
	DESIGN	MT	Aug 2013	SCALE AS SHOWN REV.	
	CADD	--			
	CHECK	SEMC	Aug 2013		
	REVIEW	JMAC	Aug 2013		

FIGURE B5.2



PROJECT		HIGHWAY 66 HIGH FILL H5			
TITLE		STABILITY ANALYSIS NORTH SIDE SLOPE AT STA 13+320			
		PROJECT No. 10-1191-0044	FILE No. ----		
DESIGN	MT	MAY 2013	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	SEMC	MAY 2013			
REVIEW	JMAC	MAY 2013			



FIGURE B6



APPENDIX C

Highway 66 – STA 14+020 to 14+650 (Swamp Crossing H6/H7)



December 5, 2013

FOUNDATION REPORT

**WICK DRAIN TREATMENT AREAS
REALIGNMENT OF HIGHWAY 66 AT VIRGINIATOWN
FROM 10.6 KM EAST OF HIGHWAY 624 EASTERLY 3.4 KM
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5091-07-00**

Submitted to:
McCormick Rankin Corporation
A Member of MMM Group Limited
2655 North Sheridan Way
Mississauga, Ontario
L5K 2P8

REPORT



Report Number: 10-1191-0044-R1 (Appendix C)





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FOUNDATION REPORT - WICK DRAIN TREATMENT AREAS - REALIGNMENT OF HIGHWAY 66, AT VIRGINIATOWN, GWP 5091-07-00

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C1.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides an interpretation of the geotechnical data obtained during the foundation investigation and presents recommendations on the wick drain foundation treatment aspects of design for the proposed works. The recommendations provided are intended for the guidance of the design engineer. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the design of the project. Those requiring information on aspects of construction must make their own interpretation of the subsurface information provided as it affects their proposed construction methods, costs, equipment selection, scheduling and the like.

C1.1 General

Golder has been retained by McCormick Rankin Corporation (MRC), a member of MMM Group Limited (MMM), on behalf of Ministry of Transportation, Ontario (MTO) to carry out detail design for wick drain foundation treatment of one swamp crossing along the proposed Highway 66 alignment as part of the Detail Foundation Investigation and Design for the re-alignment of Highway 66 at Virginiatown, from 10.6 km east of Highway 624 easterly 3.4 km, under G.W.P. 5091-07-00.

The foundation engineering services associated with the wick drain foundation treatment area have been carried out in accordance with the Scope of Work outlined in our change request letter dated February 19, 2013.

The following sections address the design of a proposed wick drain foundation treatment system to be installed at the swamp crossing below:

■ Highway 66 – STA 14+020 to 14+650 (Swamp Crossing H6/H7)

As noted in Section 6.5.3.3, a wick drain system was identified as being the preferred foundation treatment option at this swamp crossing, in comparison to other alternatives of foundation improvement as presented in Table C1, to allow the embankment construction to be completed within an accelerated schedule and to mitigate long-term, post-construction settlement. The design of the wick drain system addresses the stability and settlement of the proposed embankment during and following construction.

The results of the additional field investigation and laboratory testing that was carried out as part of this assignment have been included in Part A (Foundation Investigation) of this report and it is comprised of seven Cone Penetration Tests (CPTs) with pore pressure dissipation tests, five boreholes to obtain 'Shelby' tube samples of the cohesive deposits as well as five consolidation tests (four horizontally trimmed orientation (HTO) and one vertically trimmed orientation (VTO), three sets of consolidated isotropic undrained (CIU) triaxial tests, and one set of drained direct shear tests on samples of clayey silt to silty clay deposit.

C2.0 EMBANKMENTS OVER SWAMPS

Based on the profile drawing provided by MRC on April 21, 2013, the proposed highway will require fill embankments ranging in height from about 1.5 m up to about 3.5 m in the wick drain swamp area.

Sections C2.1 and C2.2 of this report summarize the methodology used for the analysis of stability and settlement at critical sections of embankment construction through the wick drain swamp crossing. The results



of the analyses and recommendations on mitigating stability and time-dependent settlements in conjunction with the wick drain treatment are presented in Section C3.0.

C2.1 Stability

The following sections outline the methodology used to evaluate embankment stability at the wick drain swamp crossing. In addition, the parameters used in the analyses are also presented. The results of the total stress and effective stress stability analyses are presented in Section C3.1 and C3.2, respectively.

C2.1.1 Methodology

Stability analyses, in terms of total stress and effective stress conditions, were performed for the critical sections of the proposed embankment. The critical sections were selected based on the following criteria:

- where the height of embankment fill is greatest;
- where the depth of sub-excavation of peat/organic deposits is greatest and smallest;
- where the total thickness of cohesive deposits is greatest; and/or
- where the thickness of the upper portions of the cohesive deposit is thinnest.

The stability of the proposed new embankment sections was analyzed using the limit equilibrium method.

All limit equilibrium slope stability analyses were performed using the commercially available program Slide (Version 6.0), produced by Rocscience Inc., 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 FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static conditions. This FoS is considered adequate for the embankments at these sites considering the design requirements and the field data available and is based on deep-seated, global failure surfaces that would affect the operation of the roadway. The stability analyses were performed to check that the geometry and proposed rate of construction satisfied the target minimum FoS of 1.3 at each stage of the embankment construction.

The stability analyses assume that all peat/organic soils will be removed prior to construction of the new embankments and that granular fill (i.e. Granular 'B' Type I) will be used for replacement of sub-excavated material (as discussed in Sections C5.1 and C5.3). The piezometric conditions required in the analyses were based on the groundwater levels observed during drilling, which were generally located at about the level of the natural ground surface at most locations. The impact of the excess pore pressure development on the stability of the embankments at each stage of construction was assessed as part of the effective stress analysis. The stability analysis was carried out assuming a 1.25H:1V side slope profile for the rock fill embankment and granular backfill below the ground surface and assuming a 2H:1V side slope profile for the granular surcharge load placed on top of the rock fill embankment.

Given the presence of thick and soft cohesive deposits and stability issues associated with the proposed embankment geometry, the use of stability berms at the toe of the embankments was included in the design to enhance stability. Taking into consideration the limited width of the right-of-way, the proposed height of the



embankment and the depth of sub-excavation of peat/organic deposits, 1.5 m high by 5 m wide stability berms were included in the overall embankment geometry for the purpose of stability analyses. In general, the addition of stability berms allows for higher excess pore pressures to develop and therefore can accommodate faster rates of construction, while maintaining the target FoS of 1.3.

The total stress analyses using the undrained shear strength parameters were carried out as an additional verification on the maximum height of embankment that could be constructed instantaneously while still maintaining a FoS of 1.3. The results of the total stress analysis were compared to and used as an indicator of the suitability of the strength parameters selected for the effective stress analysis. Both the total stress and effective stress strength parameters are summarized in Table C2.

For the effective stress analysis, the excess pore pressure (Δu) response within the cohesive deposits as a result of the embankment and stability berm construction, including the sub-excavation and replacement operation, was estimated as follows:

$$\Delta u = \bar{B} \cdot \Delta \sigma_1 = \bar{B} \cdot \gamma \cdot \Delta H \text{ (in kPa)}$$

where:

\bar{B} = overall (or combined) pore pressure coefficient (*see below*)

γ = bulk unit weight of embankment fill (19 kN/m³ for rock fill and 21 kN/m³ for granular surcharge and drainage blanket and 8 kN/m³ for replacement fill (due to the difference between the unit weight of granular fill (20 kN/m³) and the existing peat/organic deposits to be excavated (12 kN/m³))

ΔH = change in height of embankment fill (m)

The overall pore pressure coefficient (\bar{B}) was calculated at regular intervals of depth and lateral distance below and beyond the critical embankment section to create a two-dimensional field of values that were subsequently employed to develop a two-dimensional contour plot of total pore pressures (i.e. initial/hydrostatic pore pressures plus excess pore pressures) throughout the foundation soil deposits at each stage of the embankment construction. The overall pore pressure coefficient (\bar{B}) was calculated using the following equation proposed by Skempton (1954):

$$\bar{B} = B \left(1 - (1 - A) \left(1 - \frac{\Delta \sigma_3}{\Delta \sigma_1} \right) \right)$$

where:

A and B = Skempton pore pressure parameters estimated from the results of the CIU triaxial tests (as summarized in Section C2.1.3).

$\Delta \sigma_3 / \Delta \sigma_1$ = principal stress ratio at point of interest within the foundation soil below the embankment and toe berms (after Poulos and Davis, 1974)

The change in excess pore pressure with time ($\Delta u(t)$, kPa) was calculated based on the average degree of consolidation (U) completed at the time of interest (as described above) using the following formula:



$$\Delta u(t) = \Delta u(1 - U)$$

where: Δu = excess pore pressure due to embankment/toe berm loading (kPa)
 U = average degree of consolidation at time (t)

As noted above, a two-dimensional field of total pore pressures throughout the foundation soil deposits at critical time periods was developed to assess the stability of the embankment at the different stages of construction. In addition, different rates of construction (i.e. time periods between construction stages) were also considered in the analyses. The rate of construction influences the excess pore pressure calculated at any time (t) due to the fact that the slower the rate of construction, the larger the degree of consolidation that will be completed within any one construction stage.

For the effective stress analysis, the total pore pressure ($u_T(t)$) within the cohesive deposits at the appropriate time (t) was calculated as follows:

$$u_T(t) = u_o + \Delta u(t) \text{ (in kPa)}$$

where: u_o = initial pore pressure within the cohesive deposit (assuming groundwater table to be at ground surface) (kPa)
 $\Delta u(t)$ = excess pore pressure at time (t)

C2.1.2 Strength Parameters

The simplified stratigraphy together with the associated unit weight(s) and foundation engineering parameters employed for the different native soil types at the critical sections are summarized in Table C2. Additional details of foundation engineering parameters employed for the cohesive deposits encountered in swamp crossing are provided on Figure C24.

The overburden encountered in the swamp crossing is composed of a combination of non-cohesive (granular) deposits (sandy silt to sand and gravel) and cohesive deposits (clayey silt to clay). For non-cohesive soils, effective stress parameters were employed in the analysis assuming drained conditions. The effective stress parameters (i.e. effective friction angle (ϕ')) for the granular soils were estimated from empirical correlations using the results of in situ Standard Penetration Tests (SPT) using the method proposed by Schmertmann (1975) and Meyerhof (1956), in conjunction with engineering judgement based on experience on similar soil conditions.

For cohesive deposits, both total stress and effective stress parameters were employed in the analyses assuming undrained and drained conditions, respectively.

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 tests and Cone Penetration Tests (CPTs), inferred from the laboratory consolidation tests results, and estimated from correlations with the SPT results and other laboratory test data (i.e. natural water content). Based on the results of the consolidation tests and estimates of the preconsolidation stress (σ_p'), from the in situ CPT tests (as described in Section C2.2.2), the following correlation proposed by Mesri (1975) was employed to estimate the undrained shear strength:



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$$s_u = 0.22 \cdot \sigma_p' \text{ (in kPa)}$$

where:

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

$$\sigma_p' = \text{preconsolidation stress (kPa)}$$

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

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

where:

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

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

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

The effective stress parameters (effective friction angle (ϕ') and effective cohesion (c')) for the cohesive deposits were assessed based on the results of three sets of consolidated isotropic undrained (CIU) triaxial tests with pore pressure measurements and one set of drained direct shear tests. The effective friction angle and effective cohesion estimated directly from the results of the triaxial and direct shear tests are presented below.

Soil Unit	Borehole / Sample Number	Laboratory Test Type	Effective Cohesion, c' (kPa)	Effective Friction Angle, ϕ' (degrees)
Upper portion of cohesive deposit above Elevation 303 m	Borehole H6-S2 Sample 1 Borehole H6-S4 Sample 1 Borehole H6-S5 Sample 1	CIU Triaxial	0	35
Lower portion of cohesive deposit below Elevation 303 m	Borehole H6-S2 Sample 2 Borehole H6-S2 Sample 3 Borehole H6-S2 Sample 4	CIU Triaxial	3	24
	Borehole H6-S5 Sample 3	Direct Shear	0	30

These results were compared with estimates of effective friction angle from the empirical correlations with plasticity index (PI) proposed by Mitchell (1993), Ladd et al. (1977) and Kulhawy and Mayne (1990) to establish the appropriate design values for the cohesive deposits. The range of effective friction angles based on the various empirical correlations is summarized below.

Empirical Correlation	Effective Friction Angle of Cohesive Soils, ϕ' (degrees)		
	Lowerbound	Upperbound	Average
Mitchell (1993)	25	45	31
Ladd et al. (1977)	26	36	31
Kulhawy and Mayne (1990)	26	45	32

It can be seen that the results of the triaxial tests and direct shear tests associated with the cohesive deposits are generally near the average of the range or near the lower bound end of the range of the effective friction



angles estimated from the empirical correlations. When selecting the design strength parameters, the results of the laboratory triaxial and direct shear tests were considered in combination with the lower bound and average estimates (as appropriate) from the empirical corrections. In the stability analysis, the effective strength parameters for the cohesive soils were defined using a shear-normal strength envelope based on the results of the CIU triaxial test data at stress levels between about 0 kPa and 200 kPa, which is the operative range of stresses for the proposed embankment height along the swamp crossing. The inset plots in Table C2 show Mohr's circles based on the triaxial testing with the fully defined Mohr-Coulomb failure envelope employed in the stability analysis.

C2.1.3 Pore Pressure Parameters

The average pore pressure coefficients (A and B) used in the analyses of the excess pore pressure response in the foundation soil deposits due to embankment construction at the critical sections are given below.

Skempton's Pore Pressure Coefficients		
$A_{Elastic}$	A_{Yield}	B
0.33	1.00	0.98

The pore pressure coefficients were assessed from the results of the CIU triaxial tests with pore pressure measurement performed on specimens of the cohesive deposits. The values of $A_{Elastic}$ were estimated from the initial straight line portions of the deviator stress and excess pore pressure versus strain curves at low strain levels. The values of A_{Yield} were estimated at higher strain levels on the non-linear portions of the deviator stress versus strain curves, but at strain levels less than failure.

C2.2 Settlement

The following sections outline the methods used to conduct the settlement analyses in the wick drain swamp area. In addition, the parameters used in the analyses for the critical sections are also presented. The results of the analyses are summarized in Section C3.0.

C2.2.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed fill embankments using the commercially available program Settle3D (Version 2.0) produced by Rocscience Inc. The critical sections were selected based on the criteria as presented in Section C2.2.1.

The sources of settlement were considered to include:

- immediate settlement of the drainage blanket and of the replacement fill materials in sub-excavation areas;
- immediate settlement of the native non-cohesive (granular) soils;
- primary time-dependent consolidation of the cohesive deposits; and



- secondary time-dependent (creep) consolidation of the cohesive deposits.

The self-weight compression of the embankment rock fill materials was also considered (as described in Section C2.2.5) but not included in the criteria used to assess the required duration of embankment surcharging.

The thickness of the compressible drainage blanket fill, foundation soil deposits and the height of the embankments vary along the proposed alignment within the swamp crossing and as such the settlements along the length of the alignment will similarly vary. Furthermore, where settlements of the foundation soils are anticipated to be greater than 1 m, the analyses assume that an additional 0.5 m thick rock fill top-up will be placed prior to placement of the surcharge load to compensate for these large settlements. Given that the analyses were carried out at the critical sections, the settlements estimated will generally represent the maximum value along the alignment.

The settlement analyses assume that all organic soils within the wick drain limits will be sub-excavated and replaced with OPSS PROV. 1010 (Aggregates) Granular 'B' Type I (as discussed in Section C5.1 and C5.3) prior to construction of the proposed embankment. The piezometric conditions required in the analyses were based on the groundwater levels noted during drilling and were essentially located at about the level of the natural ground surface at most locations.

To estimate the rate of excess pore pressure dissipation and consolidation, analyses were carried out to assess the effect of different wick drain spacings on the response of the foundation soil deposits to the proposed embankment fills. The analyses employed the analytical solutions for assessing the degree of consolidation by radial (or horizontal) drainage (U_h) proposed by Barron (1948), including the extended solutions of Hansbo (1979) developed specifically to assess the use of prefabricated geosynthetic (wick) drains for the consolidation of compressible cohesive deposits. The extended solutions by Hansbo (1979) permit including the effects of the wick drain well resistance/discharge capacity and the effects of smear of the soil along the wick drain (due to installation) on the rate of excess pore pressure dissipation/consolidation.

The average degree of horizontal consolidation (\bar{U}_h) within the cohesive soils was calculated using the following formula (after Hansbo, 1979):



$$\bar{U}_h = 1 - e^{-\frac{8C_h t}{\mu D^2}}$$

where:

$$\mu = \ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k_s} \ln(s) - 0.75\right) + \pi z(2L - z) \frac{k_h}{q_w}$$

$$n = D/d$$

$$s = d_s/d$$

and

$$t = \text{time of consolidation (s)}$$

$$c_h = \text{horizontal coefficient of consolidation (cm}^2/\text{s)}$$

$$D = \text{diameter of zone of influence of vertical drain (m)}$$

$$d = \text{equivalent diameter of vertical drain (m)}$$

$$d_s = \text{diameter of disturbed zone (m)}$$

$$k_h = \text{horizontal permeability (m/s)}$$

$$k_s = \text{permeability of disturbed zone (m/s)}$$

$$L = \text{initial effective length of drain (m)}$$

$$z = \text{distance from open end of drain to worst location of well resistance (m)}$$

$$q_w = \text{factored discharge capacity of well (m}^3/\text{s)}$$

It should be noted that the general equation for the consolidation of cohesive foundation soil deposits includes both the horizontal consolidation (U_h) and vertical consolidation (U_v) component as follows:

$$U = 1 - (1 - U_v)(1 - U_h)$$

where:

$$U_v = \text{average degree of consolidation from vertical drainage only (from Terzaghi's one-dimensional consolidation theory)}$$

$$U_h = \text{average degree of consolidation from radial drainage only (to the wick drains).}$$

However, considering the thickness of the cohesive deposits and potential drain spacings that are normally adopted for similar MTO wick drain foundation projects, the vertical consolidation component is expected to be relatively small in comparison to the horizontal component and in this case, the general consolidation equation will reduce to $U = U_h$ (i.e. the majority of the consolidation will occur horizontally to the wick drains).

The primary consolidation settlement with time ($s(t)$) was calculated based on the average degree of consolidation (U) completed at the time of interest (as described above) using the following formula:

$$s(t) = s_c \cdot U(t) \text{ (in m)}$$

where:

$$s_c = \text{total primary consolidation settlement (m)}$$

$$U(t) = \text{average degree of consolidation at time (t)}$$

The secondary consolidation settlements (s_s) of the cohesive deposits were calculated using the appropriate values of $C_{\alpha(\epsilon)}$ for both the over-consolidated (O/C) and normally-consolidated (N/C) portions of the deposits in conjunction with the following formula:



$$s_s = C_{\alpha(\epsilon)} \cdot H \cdot \Delta \log \left(\frac{t}{t_{EOP}} \right) \text{ (in m)}$$

where:	$C_{\alpha(\epsilon)}$	=	modified secondary compression index as estimated from laboratory consolidation tests
	H	=	initial thickness of compressible clay deposit (mm)
	t	=	post-construction period of interest (20 years)
	t_{EOP}	=	time to reach end of primary consolidation (years)

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

$$C_{\alpha(\epsilon)} = w_n / 10,000$$

where:	w_n	=	natural water content (per cent)
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C2.2.2 Deformation Parameters

The simplified stratigraphy together with the associated unit weight(s) and foundation engineering parameters employed for the different native soil types at the critical sections are summarized in Table C2. Additional details of foundation engineering parameters employed for the cohesive deposits encountered in swamp crossing are provided on Figure C24.

The immediate compression of the drainage blanket/replacement fills and the native non-cohesive deposits were modeled by estimating an elastic modulus of deformation based on the SPT 'N'-values (where applicable) and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). These estimated values were compared with the typical range of expected values for similar soil types, as outlined in Canadian Highway Bridge Design Code (CHBDC, 2006) and adjusted based on precedent experience, if necessary.

The deformation parameters and stress history of the cohesive deposits were assessed based on the results of the laboratory consolidation tests, in situ field vane tests and CPTs. In addition, the results of consolidation tests were supplemented with estimates of deformation parameters (i.e. recompression and compression indices) using empirical correlations proposed in literature. The correlation by Koppula (1986) relating the natural water content and liquid limit to the compression index was found to be the most consistent with the results of laboratory consolidation tests for the cohesive deposits at these sites and is reflected in the summary plots shown on Figure C24.

The preconsolidation stress (σ_p') was evaluated from the results of the consolidation tests using the methods proposed by Casagrande (1936) and Becker et al. (1987).

The following correlation relating undrained shear strength (as measured in situ by the field vane) to preconsolidation stress (Mesri, 1975) was also employed:

$$\sigma_p' = s_{u(mob)} / 0.22 \text{ (in kPa)}$$

where	$s_{u(mob)}$	=	$\mu \cdot s_{u(FV)}$
	σ_p'	=	preconsolidation stress (kPa)
	$s_{u(mob)}$	=	average mobilized undrained shear strength (kPa)
	$s_{u(FV)}$	=	undrained shear strength from field vane test (kPa)



μ = Bjerrum's correction factor based on Plasticity Index

The preconsolidation stress was also estimated from the results of the CPTs (Demer and Leroueil, 2002):

$$\sigma_p' = \frac{q_t - \sigma_{vo}}{3.4}$$

where:

$$q_t = q_c - u_2(1 - A_n) \text{ (kPa)}$$

$$q_c = \text{tip stress measured by the CPT (kPa)}$$

$$u_2 = \text{pore pressure measured at cone 'shoulder' (kPa)}$$

$$A_n = \text{cone constant}$$

$$\sigma_{vo} = \text{total vertical stress (kPa)}$$

The initial void ratio (e_o) within the cohesive deposits was evaluated based on measurements from the trimmed specimens used for the laboratory consolidation and triaxial tests and based on the water contents measured on specimens from the SPT and 'Shelby' tube samples obtained during the field investigation using the following correlation:

$$e_o = w_n \cdot G_s \text{ (assuming 100 per cent saturation)}$$

where

$$w_n = \text{natural water content (per cent)}$$

$$G_s = \text{specific gravity (2.74 based on the average of 9 consolidation test results)}$$

The results of the laboratory consolidation tests are presented in Figures C4, C5, C10 to C14 and C20. The void ratio versus log stress curves from the consolidation test results generally show a curvilinear relation at stresses beyond the preconsolidation stress (i.e. the slope of the curve is steep immediately after the preconsolidation stress and then becomes flatter at higher stress levels). This phenomenon can be attributed to a partial collapse of the soil structure (and therefore large changes in void ratio) immediately after the effective stresses in the sample exceed the maximum past pressure experienced by the soil. At higher cumulative stresses, relatively smaller changes in void ratio occur because the soil structure cannot experience any further collapse. The values of the compression index (C_c) employed in the analysis have been assessed based on the steeper portions of the void ratio versus log-pressure curves since this represents the range of operational stresses expected to occur in the field under the proposed embankment loading.

The recompression index (C_r) and compression index (C_c) for the cohesive deposits was evaluated based on the results of the laboratory consolidation tests. The results from the consolidation tests were supplemented with estimates of C_c based on the Atterberg limits and water content testing using the following empirical correlations:

$$C_c = 0.009 \cdot w_n + 0.005 \cdot w_L \text{ (Koppula, 1986)}$$

where:

$$w_n = \text{natural water content (per cent)}$$

$$w_L = \text{liquid limit (per cent)}$$

$$C_c = 0.75 \cdot (e_o - 0.5) \text{ (Azzouz, 1976)}$$

where:

$$e_o = \text{void ratio}$$

Based on the laboratory consolidation tests carried out on samples obtained from this swamp crossing, the results indicate an approximate ratio between the compression index and the recompression index of about 20.



As such, the above noted correlations between the index properties and the compression index were utilized to obtain an estimate of the recompression index as follows:

$$C_r = C_c / 20$$

The secondary consolidation compression index ($C_{\alpha(\varepsilon)}$) for the cohesive deposits was assessed from the results of the 24 hour load increment, consolidation tests (considering the appropriate stress level anticipated in the field under embankment loading as well as the stress history of the cohesive deposits) and from estimates based on the results of the index testing (i.e. water contents) using the empirical correlation proposed by Mesri (1973). The range of values of $C_{\alpha(\varepsilon) (N/C)}$ for the normally consolidated portions of the cohesive deposits estimated using these methods are summarized below.

Method of Estimating	Secondary Consolidation Compression Index – Normally Consolidated Range, $C_{\alpha(\varepsilon) (N/C)}$ (%)		
	Lowerbound	Upperbound	Average
Laboratory Consolidation Test	0.17	1.84	0.65
Empirical Correlation (Mesri, 1973)	0.12	0.93	0.47

It can be seen that the values of $C_{\alpha(\varepsilon) (N/C)}$ from the laboratory test results are generally higher than the estimated values from the empirical correlation. Values obtained from the upperbound of the laboratory tests were given more weight when selecting the design values based on precedent experience on other MTO wick drain design and monitoring projects in Northern Ontario. Values of $C_{\alpha(\varepsilon) (O/C)}$ for the over-consolidated portions of the cohesive deposits were estimated as being equal to one-fifth of the normally consolidated values (i.e. $C_{\alpha(\varepsilon) (O/C)} = 1/5 \cdot C_{\alpha(\varepsilon) (N/C)}$).

C2.2.3 Rate of Consolidation Parameters

The values of the coefficient of consolidation in the horizontal direction (c_h) were assessed primarily from the results of the pore pressure dissipation tests carried out as part of the CPT testing performed in the swamp crossing. A total of 33 pore pressure dissipation tests were carried out and the results are shown on Figure C25. Based on this data, the c_h values were estimated using two methods as presented by Robertson et al. (1992):

Method No. 1:

$$c_h = \left(\frac{m}{M}\right)^2 \sqrt{I_r \cdot r^2} \quad (\text{in cm}^2/\text{s})$$

where:

m = gradient of the initial linear portion of the CPT pore pressure dissipation curve ($\sqrt{\text{time}}$ units)

M = 1.15 (for CPT pore pressure sensor at position u_2)

I_r = rigidity index, G/s_u (ranges from about 55 to 75 for this site)

r = radius of CPT probe (17.8 mm)



The gradients of the initial portion of several dissipation curves (m) selected to represent a range of horizontal coefficient of consolidation was based on the average value of the gradient of the initial linear portion of the CPT pore pressure dissipation (m) from all the CPT dissipation tests carried out in the cohesive deposits. The gradients of the initial linear portion used to estimate a range of horizontal coefficients of consolidation for various cohesive deposits is shown on Figure C25.

Method No. 2:

$$c_h = \frac{T^* \cdot r^2 \cdot \sqrt{I_r}}{t}$$

where:

T^* = dimensionless time factor (0.245 for 50 per cent of pore pressure dissipation and for a CPT pore pressure sensor at position u_2)

t = time required to reach a specific degree of consolidation

I_r = rigidity index, G/s_u (ranges from about 55 to 75 for this site)

r = radius of CPT probe (17.8 mm)

The time (t) required to reach 50 per cent completion of the full pore water pressure dissipation (i.e. t_{50}) was utilized in the method outlined above to estimate horizontal coefficients of consolidation for all the CPT dissipation tests.

The rigidity index is defined as the ratio of the shear modulus to the undrained shear strength of the cohesive deposit (i.e. G/s_u). The shear modulus (G) is calculated as a function of the elastic modulus (E) and the Poisson's ration (ν) and is defined as follows:

$$G = \frac{E}{2(1+\nu)} \text{ (in MPa)}$$

The elastic moduli for the cohesive deposits, based on the results of the laboratory consolidation tests (i.e. estimated from the constrained modulus, $D = 1/m_v$), Poisson's ratio, and the average design undrained shear strength (s_u) for calculating the coefficient of consolidation in the horizontal direction are presented below:

Elevation (m)	Elastic Modulus, E (MPa)	Poisson's Ration, ν	Undrained Shear Strength, s_u (kPa)
306 – 295	2	0.3	14 – 20
< 295	2 – 7	0.3	20 – 34

A value of c_h was also assessed from the results of the laboratory consolidation tests performed on a VTO of the cohesive deposit.

The coefficient of consolidation in the vertical direction (c_v) was assessed from the results of the laboratory consolidation tests performed on the HTO of the cohesive soils and were also compared with estimates based on the results of the Atterberg limits testing (i.e. liquid limit) and the empirical correlation proposed by U.S. Navy (1986).



The results of the HTO and VTO laboratory consolidation tests were used to assess the ratio between the horizontal and vertical coefficient of consolidation (c_h/c_v) employed in the analyses. The coefficients of consolidation (c_v and c_h) from the CPT dissipation tests as well as the laboratory tests are summarized below.

c_h – Field Data (cm ² /s) ¹			c_h – Laboratory Data (cm ² /s) ²			c_v – Laboratory Data (cm ² /s) ³		
Lower - bound	Upper - bound	Average	Lower - bound	Upper - bound	Average	Lower - bound	Upper - bound	Average
2.9×10^{-3}	1.3×10^{-1}	2.4×10^{-2}	5.9×10^{-4}	2.2×10^{-2}	9.7×10^{-3}	9.5×10^{-4}	5.4×10^{-2}	9.2×10^{-3}

Notes:

1. Coefficient of consolidation values based on 33 pore pressure dissipation tests carried out during CPT investigation.
2. Coefficient of consolidation values based on one consolidation test carried out on a vertically trimmed sample.
3. Coefficient of consolidation values based on eight consolidation tests carried out on horizontally trimmed samples.

The coefficients of horizontal consolidation (c_h) used in the analysis for estimating the rate of excess pore pressure dissipation and consolidation settlement of the various cohesive deposits are presented on Figure C26.

C2.2.4 Smear Ratio

The horizontal permeability of the cohesive deposits immediately adjacent to the wick drain is generally less than the permeability measured or estimated for the overall cohesive deposits as a result of localized disturbance/smearing of the soil caused by insertion of the steel mandrel into the subsurface during installation of the wick drains. The ratio of the horizontal permeability of the undisturbed soil (k_h) to the permeability of the soil in the smear zone (k_s) adjacent to the wick is termed the smear ratio (k_h/k_s). This parameter is difficult to select for analysis as it is site specific, cannot be evaluated directly from any field or laboratory testing and there are no empirical correlations available for its estimation.

Based on published information in literature, the smear ratio (k_h/k_s) can reportedly vary from 3 to 10 as suggested by Rankine et al. (2008), Xiao (2002), Crawford et al. (1992), Bergado et al. (1990, 1993) and Rixner et al. (1986). Many researchers recommend that the smear ratio is best evaluated by carrying out back-analysis of the rate of consolidation below an embankment on a wick drain foundation based on a comparison of the estimated and actual field measured excess pore pressure dissipation and/or settlement data. By fixing the other known variables in the analysis, the smear ratio can be varied until a reasonable correlation is obtained. Using field foundation monitoring data collected at several wick drain project sites in Northern Ontario, this approach has been followed revealing that the smear ratio can vary from as low as about 3 to as high as about 15, even within a single swamp crossing/wick drain treatment area. Given this, it appears that there can be significant localized variability that will affect the actual value of smear ratio. As such, although a low value of smear ratio (such as 3) may be valid at some locations, it may be conservatively low at other locations. Conversely, a high value (such as 10 or even 15) may be valid at some locations, but conservatively high at others. Considering the above, a smear ratio (k_h/k_s) of 5 has been employed for the design of the wick drain foundation systems for the current sites. However, it should be noted that because of the likely variability in the subsurface conditions, when using a smear ratio of 5, it is possible that at some locations the actual rate of consolidation in the field may be slower than that estimated by the analyses, therefore requiring a slower rate of construction and/or longer surcharge period.



C2.2.5 Settlement of Embankment Rock Fill

Where rock fill is to be used for the construction of the proposed 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 soil deposits 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); and
- 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 MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates, dated September 2010.

Rock fill should be placed, whenever possible, in a controlled manner (i.e. not end-dumped) in accordance with Special Provision (SP) 206S03 (Rock Excavating, 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.

C2.2.5.1 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 Foundations Guideline (2010), as follows:

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

It should be noted that approximately 90 per cent of the short-term rock fill 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.



C2.2.5.2 Long-Term Rock Fill Settlement

The magnitude of long-term post-construction settlement for compacted and end-dumped rock fill may be estimated in accordance with the MTO Foundations Guideline (2010), as follows:

Total Height of Rock Fill, H (m)	Long-Term Rock Fill Settlement (m)	
	Compacted Rock Fill	Dumped Rock Fill
Up to 15 m	$0.1\% \cdot H$	$0.2\% \cdot H$

The long-term rock fill settlement is expected to occur from one year following the completion of construction to over the life of the embankment.

C2.3 Settlement Performance Requirements

The following criterion was developed, in consultation with MRC, for the long-term performance of the embankment at this site and considered in the selection of the wick drain spacing and duration of the embankment surcharge period as part of the design of the wick drain foundation systems:

- Post-construction settlements of the foundation soil deposits should be 200 mm or less over a 20-year period following completion of construction, consistent with MTO's Embankment Settlement Criteria (March 2010) for Non-freeways on Compressible Soils.

C3.0 RESULTS OF ANALYSES

The following sections present the results of the analyses assessing the effect of wick drains (at different spacings) on the stability of the embankments plus surcharge and stability toe berms, and the development of excess pore pressure and the rate of settlement in the cohesive foundation soil deposits. Wick drain spacings of 1.5 m and 2.5 m were analysed to assess the effect that varying the spacing has on the embankment construction. A rate of construction equivalent to either 1.5 m of embankment fill (including the stability berms) placed per 7-day period, and placement of up to 2 m of Granular 'A' or Granular 'B' surcharge per 7-day period, has been assumed for the construction stages. The following lists the critical sections within the swamp crossing employed in the analysis. Although the critical sections were assessed based on the criteria identified in Section C2.1.1, the main factors associated with the selection of these critical sections have also been provided.

- STA 14+060 (where the height of embankment fill is greatest);
- STA 14+085 (where the depth of sub-excavation of peat/organic deposits and total thickness of cohesive deposits is greatest); and
- STA 14+530 (where the depth of sub-excavation of peat/organic deposits is smallest).



C3.1 Total Stress Stability Analysis

Total stress analysis, using the undrained parameters summarized in Table C2, were carried out to assess the maximum height of embankment that could be constructed instantaneously (i.e. without any delays between lift placement) while still maintaining a FoS of 1.3. The results of the total stress analyses for the maximum embankment heights under undrained loading conditions for the critical sections are presented below.

Critical Section	Maximum Height ¹ of Embankment for a FoS Greater Than 1.3 (m)
STA 14+060	2.0
STA 14+085	1.5
STA 14+530	3.0 ²

Note:

1. Maximum height refers to the embankment height above original ground surface and includes toe berms (1.5 m high by 5 m wide).
2. Proposed height of embankment has FoS > 1.3.

The results of the total stress stability analyses for the maximum stable embankment height with a FoS greater than or equal to 1.3 are shown on Figures C27 to C29. The total stress analyses indicates that after the embankment heights have been reached, the FoS will decrease with each subsequent lift of fill placed when excess pore pressure dissipation and strength gain of the underlying foundation soils is not considered.

C3.2 Effective Stress Stability Analysis

The impact of staged construction on excess pore pressure development within the cohesive deposits on the stability of the proposed embankment at the critical sections at each stage of the construction was assessed as part of the analysis. The overall pore pressure coefficients (\bar{B}) and pore pressure change were calculated at closely spaced intervals (i.e. a grid of points) covering the extent of the foundation soil deposits in order to create a two-dimensional field of total pore pressures (i.e. initial/hydrostatic pore pressure plus excess pore pressure) below and beyond the footprint of the embankment. This analysis was carried out for each critical time interval to examine the stability of the embankment at the different stages of construction.

In addition, effective stress analyses were also carried out for the conditions in which construction of the embankment to its full (maximum) height allowed in the total stress analyses was completed instantaneously, as opposed to construction of 1.5 m thick lifts of fill in a 7-day period, to simulate the effect that would be comparable to the rapid loading, undrained condition in the total stress analyses. This allows an assessment of the suitability of the strength parameters used for the construction sequence for the various wick drain spacings at the swamp crossing. The results of the effective stress analyses for the maximum embankment heights under instant loading conditions are presented below (together with the results of the total stress analyses for comparison):



Critical Section	Embankment Height (m)	Factor of Safety	
		Total Stress Analysis	Effective Stress Analysis
STA 14+060	2.0	1.4	1.6
STA 14+085	1.5	1.4	1.3
STA 14+530	3.0	1.4	0.7

The results of the effective stress stability analyses for the embankment heights as presented in the table above are shown on Figures C30 to C32.

The results of the effective stress analyses are in general agreement with the total stress analyses, implying that the effective stress strength parameters (i.e. fully defined shear-normal function) combined with the estimated excess pore pressure development used in the analysis is comparable to the undrained shear strength parameters (s_u). Based on the results summarized above, it is evident that the governing critical section is at STA 14+530. To maintain a minimum FoS of 1.3, the results of the effective stress stability analysis indicates that the embankment can only be constructed to a height of 0.5 m without any delays. Attributing factors for the difference in FoS between the total and effective stress analysis at STA 14+530 include the combination of the depth to the top of the cohesive deposit (i.e. about 0.5 m below ground surface as compared to about 4 m below ground surface at the other critical sections) and the development of excess pore pressures near the toes of the embankment, generating surficial slip surfaces in the effective stress analysis as well as the potential over-estimation of undrained shear strength at low confining stresses in the total stress analysis for similar surficial slip surfaces.

The effective stress analysis at various stages of construction, which considers the effect of dissipation of excess pore pressure with time, is utilized to estimate the time delay required between subsequent lift placements to maintain a minimum FoS of 1.3. The results of the effective stress analysis on the time required for construction of the embankment to the top of the surcharge level for two different wick drain spacing are presented below:

Maximum Thickness of Embankment Fill (including Top-Up and Surcharge) (m)	Estimated Time from Start of Embankment Construction to Completion of Surcharge Fill (days) ¹	
	1.5 m Wick Drain Spacing (with 2 m Surcharge)	2.5 m Wick Drain Spacing (with 2 m Surcharge)
6.0 ²	176	235

Note:

1. The estimated time for construction includes all hold times between different stages of construction.
2. The maximum thickness of embankment fill includes an additional 0.5 m of rock fill top-up placed prior to the granular surcharge load to compensate for the settlement of the fill during the construction operation. The rock fill top-up should be placed between STA 14+040 and STA 14+300 where the embankment is expected to settle the most.

C3.3 Surcharging and Removal of Surcharge

An embankment surcharge was considered as part of the wick drain/staged construction design to accelerate the rate of consolidation and satisfy the post-construction settlement performance criterion. The estimated time



for surcharging and surcharge removal for a 1.5 m and 2.5 m wick drain spacing is detailed in Table C3 and summarized below.

Maximum Thickness of Embankment Fill (including Top-Up and Surcharge) (m)	Estimated Time from Start of Construction to Removal of Surcharge (and Surcharge Period ¹) (days)	
	1.5 m Wick Drain Spacing (with 2 m Surcharge)	2.5 m Wick Drain Spacing (with 2 m Surcharge)
6.0 ²	220 (44)	410 (175)

Note:

1. The estimated surcharge period shown in the parentheses is the time between the completion of surcharge fill placement and the removal of surcharge.
2. The maximum thickness of embankment fill includes an additional 0.5 m of rock fill top-up placed prior to the granular surcharge load to compensate for the settlement of the fill during the construction operation. The rock fill top-up should be placed between STA 14+040 and STA 14+300 where the embankment is expected to settle the most.

C3.4 Recommended Wick Drain Spacing and Surcharge Thickness

Based on the results of the effective stress stability analyses, time-rate consolidation and excess pore pressure response analysis and in consideration of the preferred duration of about twelve months for construction of the embankment including the surcharge period (i.e. one construction season), the recommended design layout of the wick drain foundation system and surcharge for swamp crossing is presented below and is detailed in Table C4.

Longitudinal Extent	Recommended Wick Drain Foundation System	Estimated Time from Start of Construction to the Removal of Surcharge (days)
STA 14+040 to STA 14+570	1.5 m triangular wick drain spacing with 2 m surcharge and toe berms (5 m wide and 1.5 m high)	220

The results of the effective stress stability analyses and the estimated excess pore pressure below the embankment centerline versus time for the critical sections are shown on Figures C33 to C35 and Figures C36 to C38, respectively.

C3.5 Settlement of Foundation Soils

The thickness of the compressible cohesive deposits used in the settlement analyses, together with the estimated magnitude of elastic settlement, primary consolidation, secondary consolidation, total settlement during construction and the post-construction settlement for the recommended wick drain spacing and surcharge thickness at the critical sections of alignment are summarized below. Plots of settlement of the cohesive deposits at the embankment centreline versus time for each critical section are shown on Figures C39 to C41.



FOUNDATION REPORT - WICK DRAIN TREATMENT AREAS - REALIGNMENT OF HIGHWAY 66, AT VIRGINIATOWN, GWP 5091-07-00

Critical Section	Height of Embankment Fill (including Top-Up and Surcharge) (m)	Thickness of Cohesive Deposits (m)	Elastic Settlement (mm)	Primary Consolidation Settlement (mm)	Secondary Consolidation Settlement (mm)	Total Settlement of Foundation Soils During Construction (mm)	Post – Construction Settlement of Foundation Soils (mm)
STA 14+060	6.0	12.7	70	1,210	~ 0	1,280	140
STA 14+085	5.5	17.1	85	1,490	~ 0	1,575	170
STA 14+530	5.0	9.5	80	670	~ 0	750	<5

C3.6 Settlement of Embankment Rock Fill

In addition to the settlement of the foundation soils, settlement of the embankment rock fill will occur over the life of the highway embankment. The estimated settlement of the rock fill during and following completion of construction at the critical sections is summarized below.

Critical Section	Height of Embankment Fill ¹ (including Top-Up and Surcharge) (m)	Thickness of Rock Fill Embankment (m)	Estimated Settlement of Rock Fill		
			Total (mm)	During Construction ³ (mm)	Following Completion of Construction (mm)
STA 14+060	$3.5 + 0.5^2 + 2.0$	3.5	~ 35	30	~ 5
STA 14+085	$3.0 + 0.5^2 + 2.0$	3.0	~ 25	20	~ 5
STA 14+530	$3.0 + 2.0$	2.5	~ 20	15	~ 5

Note:

1. The embankment height includes the 0.5 m thick drainage blanket constructed at the existing ground surface.
2. The embankment height includes an additional 0.5 m of rock fill top-up placed prior to granular surcharge load to compensate for the settlement of the fill during the construction operation.
3. "During construction" refers to construction and surcharge time period.

C4.0 DESIGN RECOMMENDATIONS FOR EMBANKMENT ON WICK DRAIN FOUNDATIONS

Based on the results of the time rate consolidation analyses and in consideration of the preferred target duration of about twelve months for construction, the recommendations for the design layout of the wick drain foundation system for the swamp crossing embankment are provided in Table C5.

C4.1 Construction / Installation Requirements

The following recommendations are provided as guidelines for good construction practice and to ensure that drainage from the wick drains is facilitated away from the embankment area so that the wick drain foundation functions efficiently during the construction and surcharge period.

- All topsoil / organic soils to be stripped from within the area of the embankment and stability berm footprint prior to placement of the drainage blanket and embankment fill.



- Where the subgrade extends above the groundwater level and therefore can be shaped, the subgrade should be shaped to 3 per cent minimum crossfall to be carried out before placing the drainage blanket to provide for drainage efficiency, where possible.
- Shallow ditching to be constructed around the perimeter of the stability berms to facilitate drainage away from the embankment area to provide for drainage efficiency.
- A minimum 0.5 m thick drainage blanket composed of OPSS PROV. 1010 (Aggregates) Granular 'B' Type I fill to be placed on the ground surface following stripping and sub-excavation and in accordance with the NSSP provided in Appendix G. A thicker drainage blanket/sub-excavation and replacement will be required at locations where thick organic deposits are present. Pit run Granular 'B' Type I with 100 per cent passing the 26.5 mm sieve and no more than 5 per cent passing the 0.075 mm sieve should be used to minimize difficulties with obstructions that otherwise could occur during wick drain installation through areas where the drainage blanket is thicker.
- The plan area of the wick drain installation should extend at least two rows beyond the toes of all stability berms.
- The wick drains should fully penetrate the cohesive deposits and be installed to 'refusal' within the non-cohesive deposits below the cohesive deposits to the depths (and elevations) listed in Table C4.
- Wick drain installation and fill placement should not be performed during frozen ground conditions to prevent damage to the wick drains and delays in the dissipation of excess pore pressures.

It is noted that grading the subgrade to achieve a 3 per cent crossfall and providing perimeter ditching may not be possible at this site given the low-lying swamp conditions. The wick drains will still function without implementing these construction practices, but they may not perform at their maximum efficiency in these areas. The reduction in wick drain efficiency due to the lack of crossfall is difficult to estimate and depends on a number of factors including the depth of ponded water that may form at the top of the wick drains (i.e. within the drainage blanket) in the absence of proper drainage, as well as on the length of time that this ponded water may be present. The presence of ponded water at the top of the system could result in a build-up of 'back pressure' on the wick drain system which may have a temporary effect on the development (or continuation) of additional time-dependent settlements. In essence, the presence of ponded water would reduce the hydraulic head and hence the hydraulic gradient that causes flow (dissipation) of excess pore pressure within the cohesive deposits. In turn, this would result in a decrease in the flow rate (dissipation). If this condition occurs, the settlement monitoring may show a delayed response (or premature flattening) of the time-rate consolidation settlement plot. This delay could continue until the 'back pressure' dissipates or until the water level within the swamp crossing decreases.

It should be noted that after removal/stripping of the peat and organic soils, the exposed subgrade may (in places) be soft and unable to support the equipment required for construction. Where soft conditions exist, the following options could be considered to facilitate construction on the subgrade:

- Carry out stripping and grading operations in the summer in a staged manner whereby some drainage is achieved and the work is done by alternatively stripping and grading with excavators over a small area, followed by placement of the drainage blanket (by pushing out onto exposed area as a working mat) and then subsequently advancing the excavators out onto the completed portions of the drainage blanket/working mat.



- Carry out stripping and grading operations in the winter and rely on frozen ground conditions to minimize the seepage of water through the surficial peat deposits and to provide additional support of the subgrade. Note however that the wick drains should not be installed in frozen ground conditions.

The wick drains must penetrate through the cohesive deposits to practical 'refusal' in order to perform at maximum efficiency. Since the base of the cohesive deposits is variable along the length of the swamp crossings and also varies in cross section, the design tip elevations are to be provided in a table in the specifications.

C4.2 Construction Staging Requirements and Surcharge Removal

In order to maintain a FoS greater than or equal to 1.3 throughout the construction period, it is essential to allow excess pore pressures to dissipate during and between construction stages (as necessary) such that the total excess pore pressure developed does not exceed the maximum allowable excess pore pressure limit for each stage of construction. In addition, the Granular 'A' or Granular 'B' surcharge must remain in place for the minimum duration as presented in Table C5 in order to satisfy the post-construction settlement performance criterion.

An operational constraint should to be included in the Contract Documents that details the requirement for each embankment lift placement such that construction will be carried out in accordance with the recommended schedule rate and thickness of fill specified in the design (refer to Appendix G). It should be noted that the actual pore pressure dissipation timing may differ from the estimated rates and pore pressures will be measured during the recommended monitoring program. The construction schedule and required delays between fill placements/removal of surcharge should be controlled by the actual pore pressures and dissipation rates measured during construction and the Contract Documents should reflect this.

C5.0 SUBGRADE PREPARATION AND EMBANKMENT CONSTRUCTION

The following sections discuss general aspects of subgrade preparation and embankment construction for the swamp crossing/wick drain treatment areas, including: removal of surficial and near surface organic materials; excavation and replacement of soft cohesive deposits; staged excavation; groundwater control, where required; and embankment fill placement.

C5.1 Removal of Organic Materials

Based on the information from the boreholes advanced during the field investigation, the thickness of the deposits of organic materials (i.e. topsoil and peat) in the swamp crossing generally ranges from about 0.1 m to 4.0 m. After clearing and grubbing of the swamp crossing and prior to the placement of any fill for new construction, all surficial and near surface layers of topsoil and organic materials should be stripped and/or sub-excavated from the plan limits of the proposed works. The depths of required sub-excavation at the boreholes advanced within the swamp crossing are provided in Table C6. The organic materials and localized compressible soils should be removed using construction procedures in accordance with OPSS 209 (Embankments over Swamps and Compressible Soils) and in an NSSP included in Appendix G.



All excavations must be carried out in accordance with Ontario Regulation 213 Ontario Occupational Health and Safety Act for Construction Projects (as amended by Ontario Regulation 443).

C5.2 Control of Groundwater and Surface Water

Excavation within the plan limits of the proposed works will be required to remove organic deposits prior to embankment fill placement. Groundwater flow into the excavations will occur due to the relatively permeable near surface soils, high groundwater levels observed in the swamp crossing and because the excavation for the removal of organic deposits will extend below the groundwater table. Unwatering is not required for the excavations and backfilling in the swamp crossing, however, surface water should be directed away from the excavations at all times.

C5.3 Backfilling

For replacement of the sub-excavated materials, it is assumed that granular fill (i.e. SP 110S13 (Aggregates) Granular 'B' Type I, as discussed in Section C4.1) will be used. Where sub-excavation of organic materials is to be carried out, it will not likely be possible to place granular fill in accordance with SP 206S03 (Rock Excavation, Grading), as discussed in Section C5.4. The granular fill is anticipated to be end dumped (typically below the water table) as the excavation advances.

C5.4 Embankment Fill Placement

Placement of rock fill material above the water table for construction of new embankments should be carried out in accordance with the requirements as outlined in SP 206S03 (Rock Excavation, Grading). The rock fill should not be dumped in final position, but 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. Side slopes for rock fill embankments should be no steeper than 1.25H:1V.

C5.5 Embankment Platform Widening

In accordance with the requirements of "MTO Northern Region Engineering Directive NRE 98-200, Northern Region Embankment Design Guidelines", the construction of the embankments should include an allowance for platform widening (in 0.5 m increments) to accommodate settlements during construction as well as post-construction settlements, so that the minimum standard shoulder widths are maintained if future grade raises on the embankments are required. According to NRE 98-200, the need for future raises in road grade could occur due to settlement/compression of the embankment fill, settlement of the foundation soils and to accommodate future pavement overlays up to 200 mm thick. It is understood that this directive applies to all rock fill embankments as well as for granular fill embankments where widening restrictions are present (i.e. due to space/property issues, presence of a sensitive body of water and so on) and that the minimum required platform widening on non-major highways over swamp crossings is 1 m per side, unless the preferred mitigation option eliminates uncertainty regarding embankment settlement/performance (i.e. full sub-excavation to bedrock and backfilling with granular material).



The minimum required embankment platform widening (per embankment side) is calculated based on the estimated consolidation settlement of the foundation soils (including creep) and long-term settlement/compression of the embankment fill plus an additional 200 mm for the future pavement overlay, multiplied by the horizontal component of the side slope profile of the pavement structure (3H:1V), but cannot be less than the minimum platform widening requirement as described above.

For the proposed embankment along swamp crossing, the minimum required platform widening is 3 m to account for settlement during and post construction. The final platform to account for post-construction settlement and future overlay is 1.5 m. The platform widening details are presented in Table C7.

C5.6 Transitions between Wick Drain and No Foundation Mitigation Areas

Where no foundation mitigation measures are required immediately adjacent to a wick drain area (i.e. between STA 14+020 and STA 14+040 and between STA 14+560 and STA 15+650), it is recommended that the crest of the 2 m granular surcharge on top of the embankment should be located approximately 5 m beyond the limits of the wick drains, extending over the areas that do not require foundation mitigation.

C6.0 WICK DRAIN SPECIFICATIONS

The specification describing the requirements for the supply and installation of the wick drains to be included in the Contract Documents is included in Appendix G for reference.

C6.1 Monitoring Instrumentation Specifications

The specifications describing the type, location, supply and installation of the instrumentation required for the monitoring of the embankment to be included in the Contract Drawings and Specifications is included in Appendix G for reference.

C6.2 Monitoring Program

The foundation specialty plan describing the frequency of monitoring readings, review and alert levels and reporting required for the foundation monitoring program to be included as part of the Contract Administration Assignment is provided in Appendix H.

C7.0 CLOSURE

This report was prepared by Mr. Tomasz Zalucki, P. Eng., a geotechnical engineer. The technical aspects were reviewed by Messrs. Christopher Ng, P. Eng., a senior geotechnical engineer and Associate with Golder and J. Paul Dittrich, Ph.D., P. Eng., a senior geotechnical engineer and Principal with Golder. Mr. Jorge M. A. Costa, P. Eng., Golder's Designated MTO Contact for this project and a Principal with Golder, conducted an independent quality control review of the report.



Report Signature Page

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STANDARDS:

Commercial Software:

Slide (Version 6.0) by Roscience Inc.

Settle 3D (Version 2.0) by Rocscience Inc.

Contract Design Estimating and Documentation (CDED):

Special Provision 206S03 Amendment to OPSS 206 – Earth Excavation, Grading.

Ministry of Transportation Ontario:

Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates. September 2010.

Embankment Settlement Criteria for Design, March 2010.



FOUNDATION REPORT - WICK DRAIN TREATMENT AREAS - REALIGNMENT OF HIGHWAY 66, AT VIRGINIATOWN, GWP 5091-07-00

Northern Region Engineering Directive NRE 98-200. Northern Region Embankment Design Guidelines.
October 1998.

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91 Construction Projects

Ontario Regulation 443/09 Amendment to Ontario Regulation 213

Ontario Provincial Standard Specification:

OPSS 209 Construction Specification For Embankments Over Swamps and Compressible Soils.
April 2009.

OPSS PROV. 1010 Material Specification for Aggregates – Base, Subbase, Select Subgrade and
Backfill Material

Ontario Water Resources Act:

Ontario Regulation 903/90 Wells



FOUNDATION REPORT – SWAMP CROSSINGS/HIGH FILL/EXCESS MATERIAL MANAGEMENT AREAS AND DEEP CUT HIGHWAY 66 AT VIRGINIATOWN, GWP 5091-07-00

Table C1: Evaluation of Settlement Mitigation Options (High Fill/Swamp Crossing H6/H7)
Highway 66 – STA 14+020 to 14+650

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Staged Construction with Surcharge, Wick Drains and Toe Berms (5 m wide) <ul style="list-style-type: none">■ 220 day period preferred■ Staged construction with 1.5 m wick drain spacing■ Total post-construction settlement = 175 mm (170 mm primary + 5 mm rock fill)	1	<ul style="list-style-type: none">■ Reduces post-construction settlement.■ Reduces delay in construction schedule to allow for sufficient settlement to meet post-construction settlement criterion.■ Reduces delay between construction stages to allow for sufficient pore water dissipation to meet stability requirements.	<ul style="list-style-type: none">■ Delay in construction schedule to allow for sufficient settlement to meet post-construction settlement criterion.■ Instrumentation and monitoring program required to assess end of surcharge period.■ Re-grading is required prior to final pavement structure construction.■ Time and cost of wick drain and instrumentation installation.■ Granular backfill and drainage blanket required for wick drain installation.	<ul style="list-style-type: none">■ Additional cost for instrumentation and associated monitoring program = ~\$400,000.■ Additional cost for wick drains installation.■ 126,000 linear metres of wick drains x \$5/m = \$630,000.	<ul style="list-style-type: none">■ Low risk of not achieving/maintaining stability of embankments on weak/soft foundation soils.■ Low risk that additional time may be required for continued preloading but delay period is subject to the monitoring data.■ Low risk of experiencing unexpected post-construction settlement (i.e. creep).
Lightweight Fill with Preloading (1.5 m EPS) <ul style="list-style-type: none">■ 10 year preload period	NP	<ul style="list-style-type: none">■ Reduces total load on subsoils thereby reducing total settlement of foundations soils.	<ul style="list-style-type: none">■ Expensive material compared to conventional embankment fill.■ Restricted thickness that can be used dependent on overall thickness of embankment and groundwater/water level.■ Very long delay in construction schedule to allow for sufficient settlement to meet post-construction settlement criterion even with lightweight fill.■ Instrumentation and monitoring program required to assess end of preload period.■ Re-grading is required prior to final pavement structure construction.	<ul style="list-style-type: none">■ Relative cost of EPS fill is about an order of magnitude higher than fill required for the other options.■ $18,900 \text{ m}^3 \times \\$200/\text{m}^3 = \\$3,780,000$.■ Additional cost for instrumentation and associated monitoring program.	<ul style="list-style-type: none">■ Very low risk of not achieving stability of preload embankments and of final EPS embankments on weak/soft foundation soils.■ Low risk that additional time may be required for continued preloading but delay period is subject to the monitoring data.■ Low risk of experiencing unexpected post-construction settlements (i.e. creep).
Preloading <ul style="list-style-type: none">■ 14 year preload period	NP	<ul style="list-style-type: none">■ Standard construction operation.■ Reduces post-construction settlement.■ Makes use only of fill material that is required for embankment construction.	<ul style="list-style-type: none">■ Very long delay in construction schedule to allow for sufficient settlement to meet post-construction settlement criterion.■ Instrumentation and monitoring program required to assess end of preload period.■ Re-grading is required prior to final pavement structure construction.	<ul style="list-style-type: none">■ Additional cost for instrumentation and associated monitoring program = ~\$400,000.	<ul style="list-style-type: none">■ Low risk of not achieving/maintaining stability of embankments on weak/soft foundation soils.■ Low risk that additional time may be required for continued preloading but delay period is subject to the monitoring data.■ Low risk of experiencing unexpected post-construction settlement (i.e. creep).
Full Sub-Excavation (up to 21 m deep)	NF	<ul style="list-style-type: none">■ Reduces magnitude of total settlement of foundations soils as soft compressible material has been removed.	<ul style="list-style-type: none">■ Generation of very large volume of excess excavation spoil.■ Very large quantity of rock fill required.■ Longer construction period required to sub-excavate to 21 m depth and replacement with rock fill.■ Specialized equipment and additional effort required for deep sub-excavation and replacement.■ Additional post-construction settlement of rock fill itself.	<ul style="list-style-type: none">■ Additional cost for sub-excavation (long-stick or drag-line) equipment, disposal and replacement of weak/soft, compressible deposits.■ $709,000 \text{ m}^3 \times \\$20/\text{m}^3$ (sub-excavation and replacement with rock fill) = \$14,000,000.	<ul style="list-style-type: none">■ Higher risk of not achieving/maintaining stability of excavation slopes.■ Very low of achieving/maintaining stability of proposed embankments.■ Very low risk of experiencing unexpected post-construction settlements associated with long term rock fill settlement.■ High risk that not all compressible soils are removed during the sub-aqueous operations which could lead to unexpected settlement.

NP: Not Practical, NF: Not Feasible

Prepared By: SEMC Reviewed By: JMAC



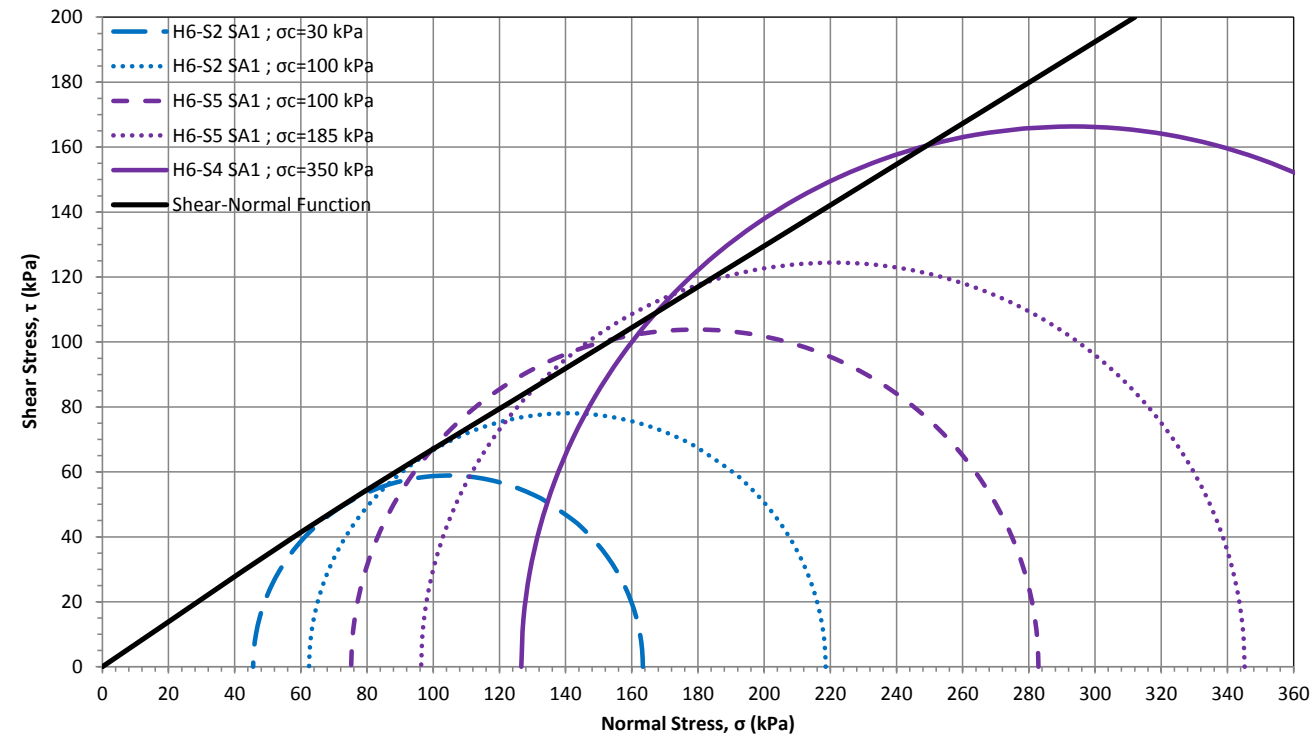
TABLE C2 – SUMMARY OF FOUNDATION ENGINEERING PARAMETERS

Swamp Crossing	Stratigraphic Unit	Top Elevation (m)	Thickness (m)	γ (kN/m ³)	ϕ' (°)	c' (kPa)	s_u (kPa)	$\sigma_{p'}$ (kPa)	e_o	C_c	C_r	E' (MPa)	$C_{\alpha(\varepsilon)}$ (%)		c_h (cm ² /s)
													N/C	O/C	
Highway 66 STA 14+020 to STA 14+650 (Swamp Crossing H6/H7)	Drainage Blanket	308.2 – 305.6	~ 0.5	21	35	0	-	-	-	-	-	25	-	-	-
	Peat / Topsoil	307.8 – 305.1	0.1 – 4.0	12	27	1	-	-	-	-	-	-	-	-	-
	Silty Sand to Sand and Silt	309.0 – 306.0	0.4 – 2.3	20	30	0	-	-	-	-	-	12	-	-	-
	Generally Clayey Silt ¹	307.8 – 303.7	0.7 – 4.8	17	35 ²	0 ²	14	64	0.8	0.5	0.025	2	1.0	0.2	2.0 x 10 ⁻²
	Generally Silty Clay to Clay ¹	~ 303.0	~ 8.0	16.5	24 ³	3 ³	14 – 21	64 – 93	1.7	1.0	0.050	2	1.0	0.2	5.0 x 10 ⁻³
	Generally Clayey Silt ¹	~ 295.0	0.2 – 7.1	16.5	24 ³	3 ³	21 – 35	93 – 160	1.0	0.5	0.025	2 – 7	1.0	0.2	2.0 x 10 ⁻²
	Silt to Sand and Silt	307.0 – 287.9	0.2 – 8.7	18	28	0	-	-	-	-	-	3	-	-	-
	Silty Sand to Gravel	306.5 – 281.0	0.6 – 8.0	20	32	0	-	-	-	-	-	35	-	-	-

Note:

1. Additional details of foundation engineering parameters for the cohesive deposits encountered in Swamp Crossing H6/H7 are provided on Figures C24 and C26.
2. Cohesive deposits above Elevation 303 m use a fully defined shear-normal function based on results of triaxial test (equivalent to $c' = 0$ kPa and $\phi' = 35^\circ$ over the stress range of $0 \text{ kPa} \leq \sigma'_n \leq 150 \text{ kPa}$). See inset below.
3. Cohesive deposits below Elevation 303 m use a fully defined shear-normal function based on results of triaxial test (approximately equivalent to $c' = 3$ kPa and $\phi' = 24^\circ$ over the stress range of $0 \text{ kPa} \leq \sigma'_n \leq 150 \text{ kPa}$). See inset below.

Shear Normal Function for Cohesive Deposits above Elevation 303 m



Shear Normal Function for Cohesive Deposits below Elevation 303 m

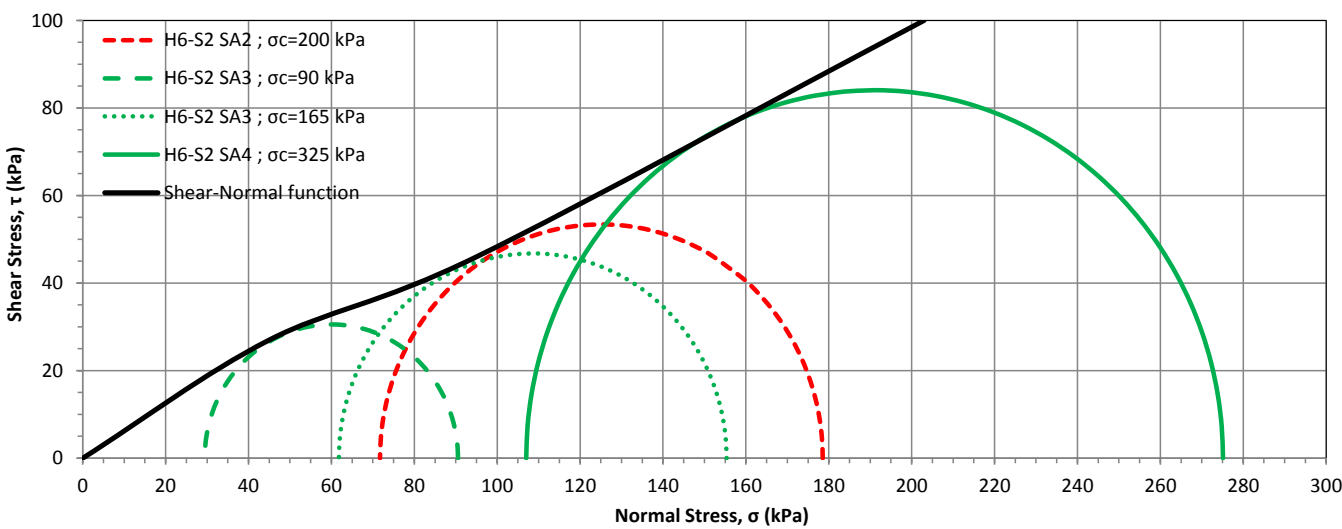




TABLE C3 – SUMMARY OF WICK DRAIN ANALYSIS AND CONSTRUCTION DURATIONS

Proposed Height of Rock Fill Embankment (m)	Rock Fill Top-Up Thickness (m)	Granular Surcharge Height (m)	Total Thickness of Embankment Fill (m)	Wick Drain Spacing (m)	Estimated Time from Start of Construction to Completion of Surcharge Embankment (days)	Surcharge Period (days)	Estimated Time from Start of Construction to Surcharge Removal (days)
3.5 ¹	0.5 ²	2.0	6.0	1.5	176	44	220
				2.5	235	175	410

Note:

1. Includes 1.5 m high by 5 m wide stability berms at the toes of embankment.
2. An additional 0.5 m of rock fill top-up should be placed prior to the granular surcharge load to compensate for the settlement of the fill during the construction operation. The rock fill top-up should be placed between STA 14+040 and STA 14+300 where the embankment is expected to settle the most.



TABLE C4 – SUMMARY OF RECOMMENDED WICK DRAIN LAYOUT REQUIREMENTS

Wick Drain Details				
Spacing	Grid Pattern	Lateral Extent	Longitudinal Extent	Vertical Extent
1.5 m	Triangular	Between the toes of the stability berms plus two rows (i.e. 3.0 m) beyond the toes of the stability berms	Approximately STA 14+040 to STA 14+560	From a minimum depth of 6.0 m (Elevation 297.5 m) to a maximum depth of 21 m (Elevation 288 m)



TABLE C5 – SUMMARY OF STAGED CONSTRUCTION

Wick Drain Spacing	Stage No.	Fill Height (m)	Thickness of Embankment Fill (m)	Fill Type	Estimated Time to Start of Filling (Day)	Estimated Time to End of Filling (Day)	Hold Time Before Next Stage (days)
1.5 m	1 ¹	0.5	0.5	Granular Fill (Drainage Blanket)	1	3.5	60
	2 ²	1.0	1.5	Rock Fill	63.5	68	60
	3 ³	2.5	4.0 ⁵	Rock Fill	128	140	30
	4 ⁴	2.0	6.0	Granular Fill (Surcharge)	170	177	44
	Estimated Time to Surcharge Removal				221		

Notes:

1. The first stage represents the construction of the 0.5 m thick drainage blanket immediately after the sub-excavation of the peat / organic deposits and replacement with granular fill operation.
2. The second stage includes the construction of the stability toe berms.
3. The third stage represents the construction of the embankment up to the proposed grade and an additional 0.5 m of rock fill placed between STA 14+040 and STA 14+300 to compensate for the settlement of the fill during the construction operation.
4. The fourth stage represents the construction of the 2 m high granular surcharge.
5. The thickness of embankment fill includes an additional 0.5 m of rock fill top-up placed prior to the granular surcharge load to compensate for the settlement of the fill during the construction operation. The rock fill top-up should be placed between STA 14+040 and STA 14+300 where the embankment is expected to settle the most.



**FOUNDATION REPORT - WICK DRAIN TREATMENT AREAS - REALIGNMENT OF HIGHWAY 66, AT VIRGINIATOWN,
GWP 5091-07-00**

TABLE C6 – SUMMARY OF EXCAVATION OF ORGANICS

Borehole / DCPT No.	Station and Offset from Centreline	Ground Surface Elevation¹ (m)	Bottom of Excavation Elevation (m)	Depth of Excavation (m)
BC2-2	STA 14+030 o/s 21.0 m Lt	309.7	309.0	0.7
BC2-1	STA 14+030 o/s 0.3 m Rt	309.4	308.4	1.0
BC2-3	STA 14+030 o/s 21.0 m Rt	309.3	308.6	0.7
H6-D1	STA 14+045 o/s 20.0 m Rt	309.6	-	-
H6-1	STA 14+057.6 CL	309.2	306.8	2.4
H6-D2	STA 14+071.5 o/s 16.0 m Rt	309.2	-	-
H6-2	STA 14+071.5 CL	309.0	305.3	3.7
H6-3	STA 14+085 CL	309.2	305.4	3.8
H6-4	STA 14+096.5 o/s 18.0 m Lt	309.1	306.0	3.1
H6-D3	STA 14+096.5 o/s 17.0 m Rt	309.0	-	-
H6-5	STA 14+108 CL	309.0	305.3	3.7
H6-D4	STA 14+115.5 o/s 16.0 m Lt	308.9	-	-
H6-6	STA 14+115.5 o/s 17.0 m Rt	308.9	305.0	3.9
H6-7	STA 14+122.5 CL	309.0	305.2	3.8
H6-8	STA 14+136.5 o/s 16.0 m Lt	308.9	304.9	4.0
H6-D5	STA 14+136.5 o/s 16.5 m Rt	308.9	-	-



TABLE C6 – SUMMARY OF EXCAVATION OF ORGANICS

Borehole / DCPT No.	Station and Offset from Centreline	Ground Surface Elevation¹ (m)	Bottom of Excavation Elevation (m)	Depth of Excavation (m)
H6-9	STA 14+150 o/s 1.5 m Rt	308.9	305.2	3.7
H6-D6	STA 14+174.7 o/s 14.0 m Lt	308.8	-	-
H6-10	STA 14+175 o/s 15.0 m Rt	308.8	305.1	3.7
BC3-2	STA 14+200 o/s 16.0 m Lt	308.8	306.0	2.8
BC3-1	STA 14+200 CL	308.7	305.6	3.1
BC3-3	STA 14+200 o/s 16.0 m Rt	308.6	304.9	3.7
H6-11	STA 14+225 o/s 15.0 m Lt	308.4	306.1	2.3
H6-D7	STA 14+225 o/s 14.0 m Rt	308.3	-	-
H6-12	STA 14+250 CL	308.3	306.0	2.3
H6-D8	STA 14+275 o/s 15.0 m Lt	307.7	-	-
H6-13	STA 14+275 o/s 14.0 m Rt	307.9	305.6	2.3
H6-14	STA 14+300 CL	307.4	305.6	1.8
H6-15	STA 14+325 o/s 15.0 m Lt	306.8	304.6	2.2
H6-D9	STA 14+325.6 o/s 15.6 m Rt	307.0	-	-
H6-16	STA 14+349 CL	306.7	305.0	1.7
H6-D10	STA 14+375 o/s 15.0 m Lt	305.9	-	-



TABLE C6 – SUMMARY OF EXCAVATION OF ORGANICS

Borehole / DCPT No.	Station and Offset from Centreline	Ground Surface Elevation¹ (m)	Bottom of Excavation Elevation (m)	Depth of Excavation (m)
H6-17	STA 14+374.6 o/s 15.0 m Rt	306.2	305.4	0.8
H6-18	STA 14+400 o/s 1.0 m Rt	305.7	305.2	0.5
H6-19	STA 14+425 o/s 15.0 m Lt	305.3	304.7	0.6
H6-D11	STA 14+425 o/s 15.0 m Rt	305.4	-	-
H7-1	STA 14+450 CL	305.2	305.1	0.1
H7-2	STA 14+465 o/s 17.0 m Lt	304.9	-	-
H7-D1	STA 14+465 o/s 16.0 m Rt	305.5	-	-
H7-3	STA 14+480 CL	305.1	305.0	0.1
H7-D2	STA 14+495 o/s 16.0 m Lt	304.9	-	-
BC4-3	STA 14+505 o/s 18.0 m Rt	304.9	304.6	0.3
BC4-1	STA 14+510 CL	305.0	304.3	0.7
BC4-2	STA 14+515 o/s 18.0 m Lt	304.1	303.9	0.2
H7-D3	STA 14+520 o/s 17.0 m Lt	304.4	-	-
H7-4	STA 14+520 o/s 18.0 m Rt	305.3	305.2	0.1
H7-5	STA 14+530 CL	305.0	304.6	0.4
H7-6	STA 14+542.5 o/s 17.0 m Lt	304.4	303.7	0.7



TABLE C6 – SUMMARY OF EXCAVATION OF ORGANICS

Borehole / DCPT No.	Station and Offset from Centreline	Ground Surface Elevation¹ (m)	Bottom of Excavation Elevation (m)	Depth of Excavation (m)
H7-D4	STA 14+542.5 o/s 16.0 m Rt	305.7	-	-
H7-7	STA 14+555 CL	305.4	-	-
H7-D5	STA 14+567.5 o/s 16.0 m Lt	304.5	-	-
H7-8	STA 14+567.5 o/s 15.0 m Rt	306.2	306.0	0.2
H7-9	STA 14+580 CL	306.0	305.9	0.1
H7-10	STA 14+592.5 o/s 13.0 m Lt	305.5	305.4	0.1
H7-D6	STA 14+592.5 o/s 15.0 m Rt	307.0	-	-
H7-11	STA 14+604 CL	306.6	306.3	0.3
H7-D7	STA 14+615 o/s 15.0 m Lt	306.4	-	-
H7-12	STA 14+615 o/s 12.0 m Rt	307.8	307.7	0.1
H7-13	STA 14+620 CL	307.5	307.4	0.1
H7-14	STA 14+635 o/s 15.0 m Lt	306.9	306.8	0.1
H7-D8	STA 14+635 o/s 12.0 m Rt	307.8	-	-
H7-15	STA 14+650 CL	307.5	307.4	0.1

Note:

1. Ground surface elevation excludes any ponded water encountered at the borehole / DCPT locations during the field investigation.

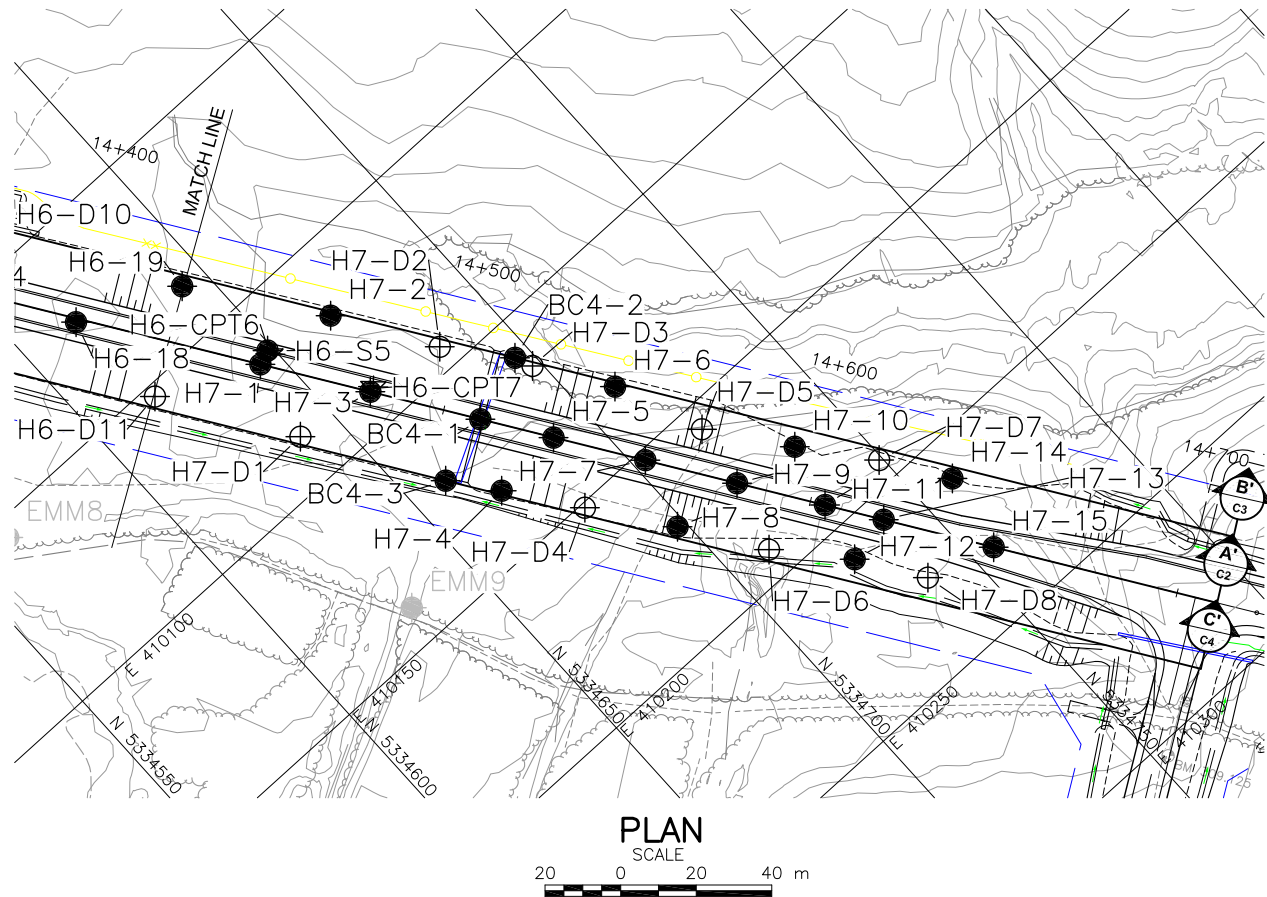
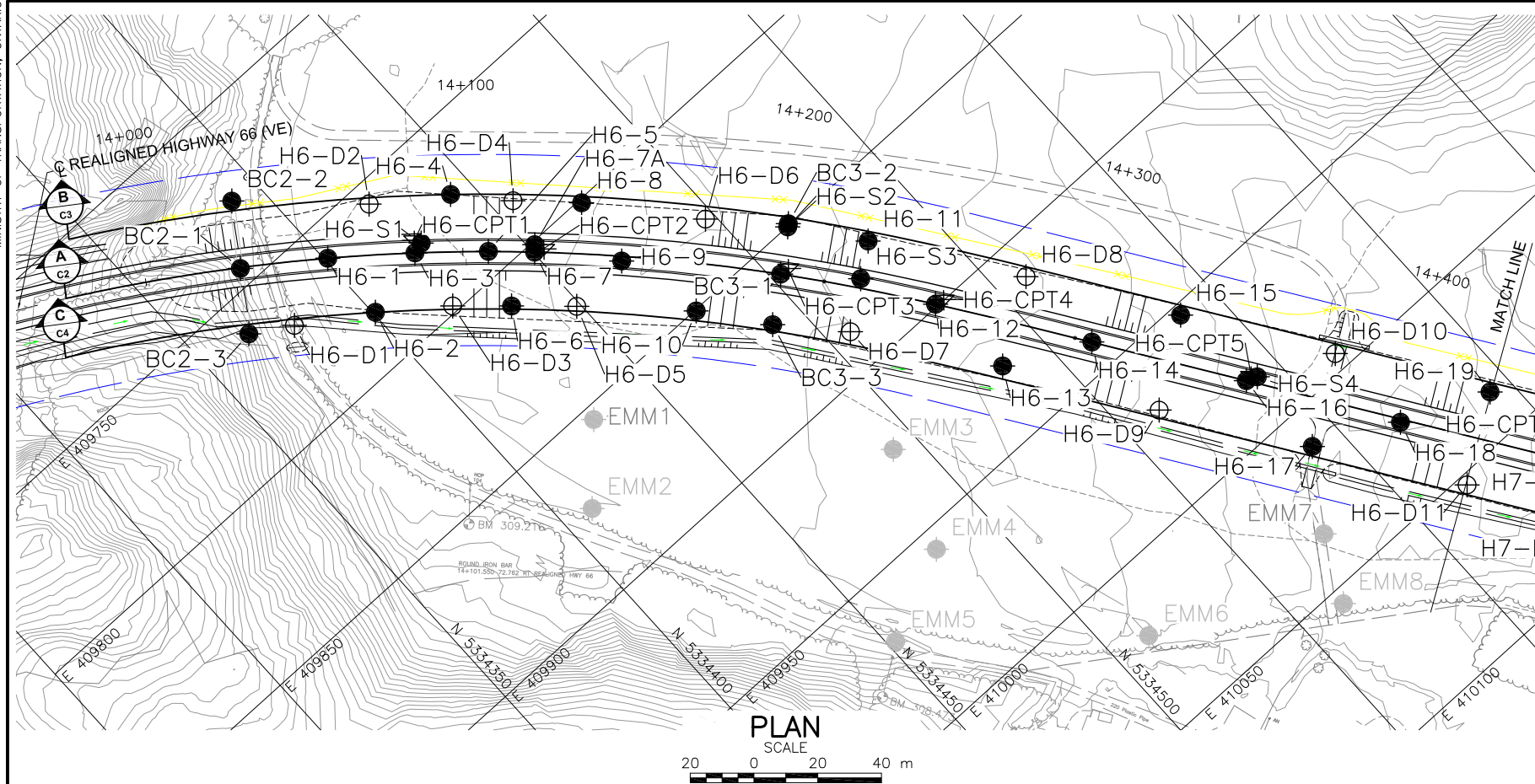


TABLE C7 – SUMMARY OF EMBANKMENT PLATFORM WIDENING REQUIREMENTS

Wick Drain Spacing and Grid Pattern	Time from Start of Construction to Removal of Surcharge (days)	Estimated Maximum Settlement from Start of Construction to 20 Years Following Removal of Surcharge ¹ (mm)	Assumed Side Slope of Pavement Structure	Minimum Embankment Platform Widening Per Side (m)
1.5 m Triangular	220	1,510 + 175	3H:1V	3.0 initial platform 1.5 final platform

Note:

1. Estimated settlement is comprised of settlement during construction plus (+) post-construction settlement and includes primary and secondary (creep) consolidation of the cohesive deposits as well as rock fill settlement. The settlements do not include the immediate settlement of the granular fills and foundation soils, where applicable.



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

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
H6-18	305.7	5334616.0	410024.5
H6-19	305.3	5334643.2	410036.3
H6-CPT1	309.2	5334425.0	409774.7
H6-CPT2	309.0	5334452.9	409803.7
H6-CPT3	308.7	5334505.8	409859.9
H6-CPT4	307.8	5334535.5	409899.8
H6-CPT5	306.7	5334591.1	409982.2
H6-CPT6	305.2	5334648.3	410065.5
H6-CPT7	305.1	5334662.5	410090.2
H6-D1	309.6	5334379.1	409769.2
H6-D2	309.2	5334422.0	409756.7
H6-D3	309.0	5334420.0	409798.0
H6-D4	308.9	5334456.2	409786.2
H6-D5	308.9	5334448.8	409824.2
H6-D6	308.8	5334497.1	409831.0
H6-D7	308.3	5334507.1	409887.7
H6-D8	307.7	5334559.6	409911.8
H6-D9	307.0	5334562.4	409970.9
H6-D10	305.9	5334615.3	409994.8
H6-D11	305.4	5334618.3	410053.1
H6-S1	309.2	5334425.9	409776.7
H6-S2	308.7	5334514.6	409850.1
H6-S3	308.3	5334520.6	409877.5
H6-S4	306.7	5334592.2	409983.8
H6-S5	305.2	5334648.5	410064.1
H7-1	305.2	5334644.7	410065.5
H7-2	305.1	5334667.1	410068.4
H7-3	305.1	5334661.4	410090.4
H7-4	305.3	5334669.6	410133.1
H7-5	305.2	5334689.2	410131.9
H7-6	305.0	5334710.3	410132.8
H7-7	305.4	5334703.2	410152.6
H7-8	306.2	5334697.7	410171.4
H7-9	306.0	5334717.1	410173.4
H7-10	305.5	5334734.9	410176.5
H7-11	306.6	5334730.5	410193.3
H7-12	307.8	5334726.7	410209.2
H7-13	307.5	5334739.4	410206.6
H7-14	306.9	5334760.2	410210.7
H7-15	307.5	5334756.1	410231.5
H7-D1	305.5	5334639.7	410086.8
H7-D2	304.9	5334683.0	410093.9
H7-D3	304.4	5334697.8	410114.1
H7-D4	305.7	5334682.9	410151.2
H7-D5	304.5	5334719.8	410156.6
H7-D6	307.0	5334711.6	410192.1
H7-D7	306.4	5334749.1	410194.1
H7-D8	307.8	5334737.8	410225.8

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.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by MMM, drawing file nos. H3211009D16 ROLL PLAN-ULTIMATE and PDR.dwg, received DEC 3, 2012. Keyplan drawing file nos. H3211009G02 received JAN 24, 2013.

CONT No.
GWP No. 5091-07-00

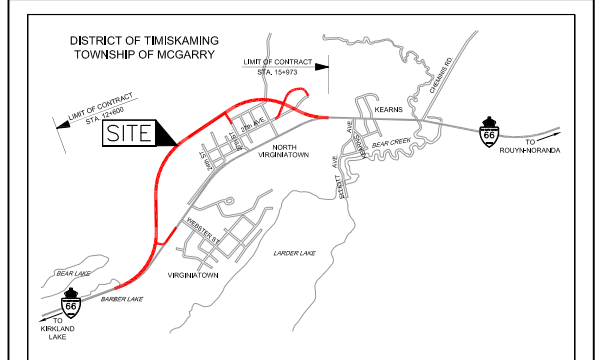


HIGHWAY 66
HWY 66 - STA 14+020 TO 14+650
BOREHOLE LOCATIONS

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



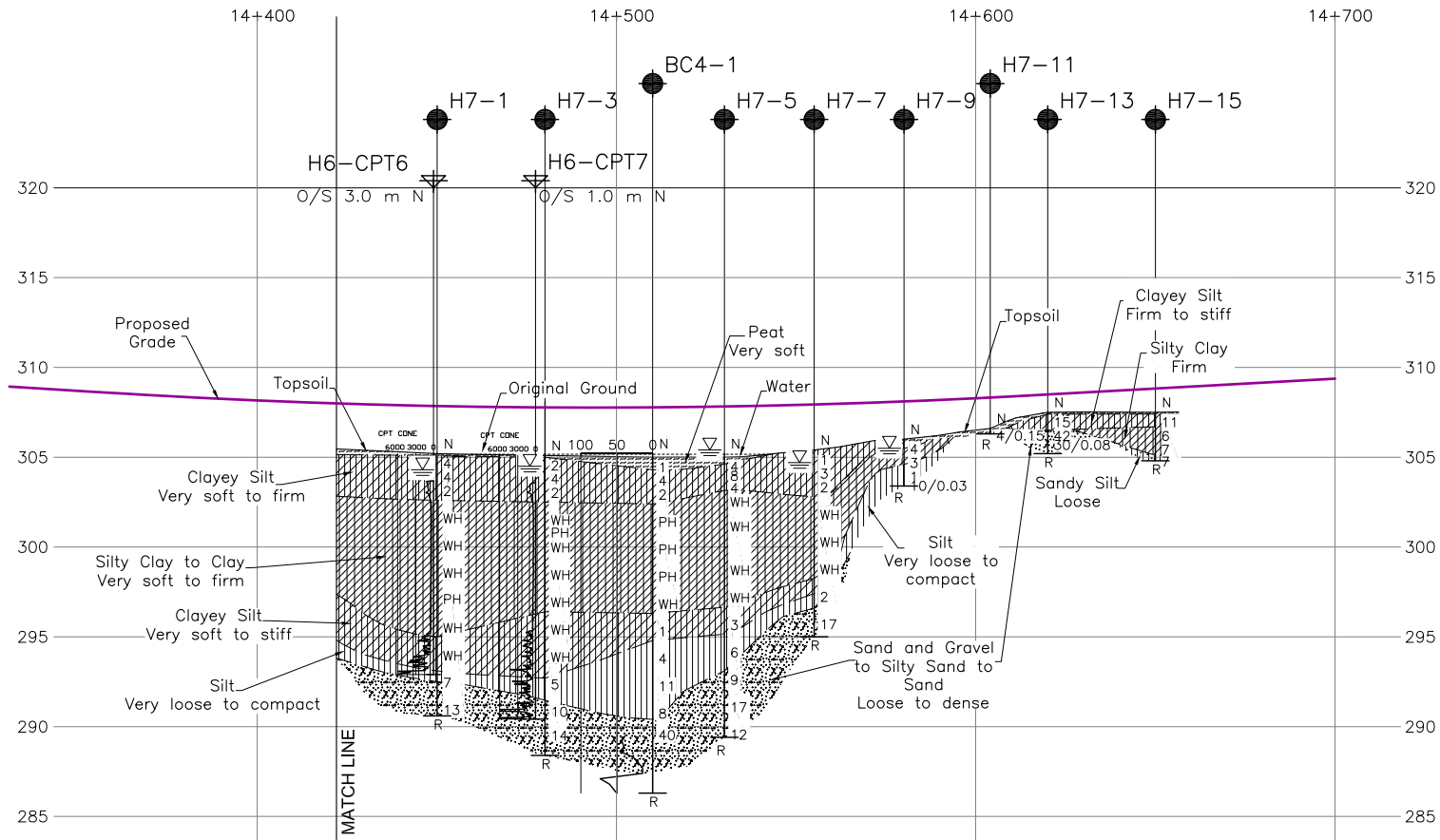
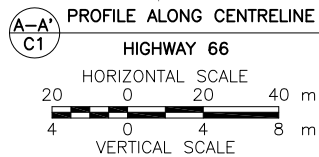
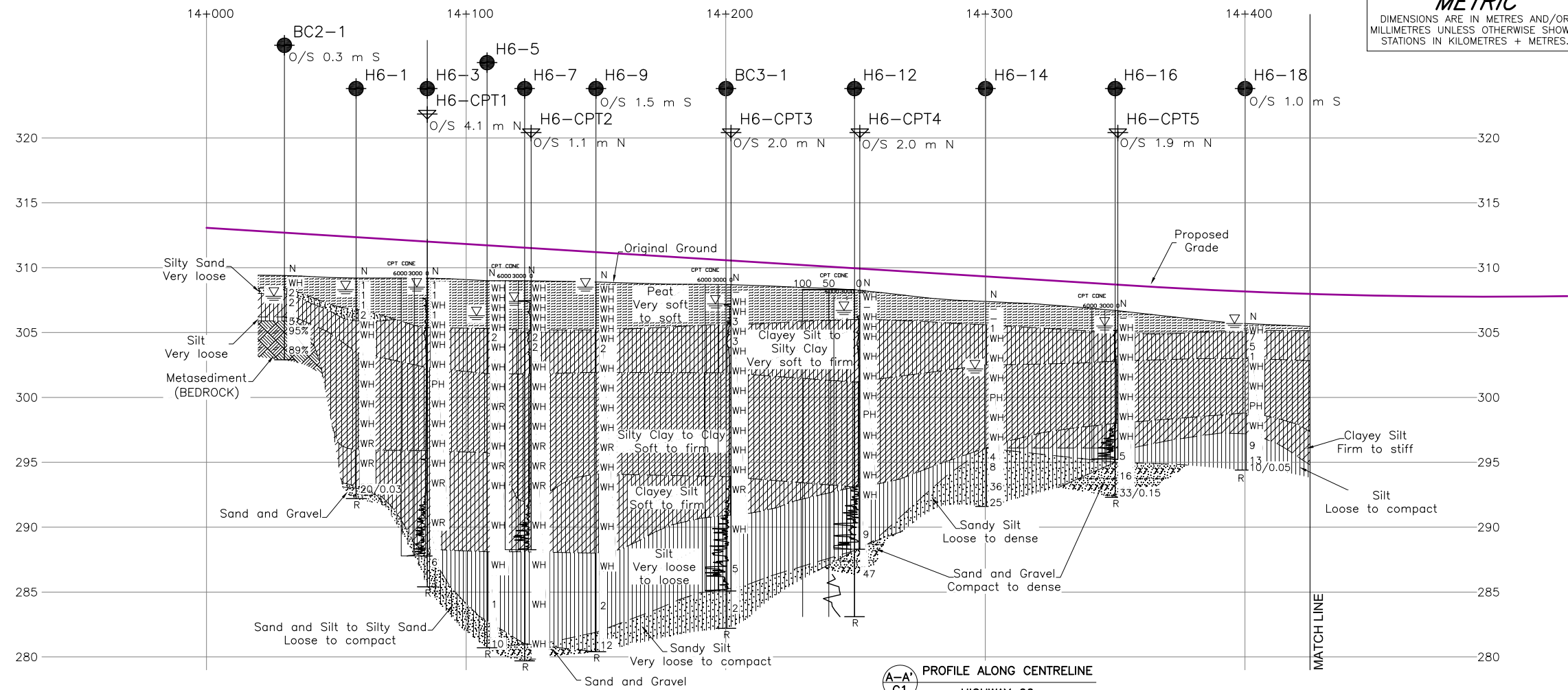
KEY PLAN
SCALE
700 0 700 m

LEGEND

- Borehole
- Dynamic Cone Penetration Test
- CPT

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
BC2-1	309.4	5334378.5	409744.6
BC2-2	309.7	5334390.7	409727.1
BC2-3	309.3	5334366.7	409761.6
BC3-1	308.7	5334503.0	409859.6
BC3-2	308.8	5334515.4	409849.5
BC3-3	308.6	5334490.5	409869.7
BC4-1	305.2	5334678.1	410115.3
BC4-2	304.8	5334695.8	410109.4
BC4-3	305.2	5334660.4	410121.2
H6-1	309.2	5334401.0	409760.6
H6-2	309.0	5334400.7	409783.2
H6-3	309.2	5334422.4	409777.7
H6-4	309.1	5334443.0	409771.6
H6-5	309.0	5334439.8	409792.8
H6-6	308.9	5334433.7	409810.4
H6-7	309.0	5334450.4	409802.7
H6-7A	309.1	5334452.0	409801.0
H6-8	308.9	5334471.7	409801.1
H6-9	308.9	5334468.8	409823.1
H6-10	308.8	5334475.6	409850.4
H6-11	308.4	5334530.3	409870.3
H6-12	308.3	5334532.7	409899.2
H6-13	307.9	5334535.3	409927.7
H6-14	307.4	5334561.1	409940.9
H6-15	306.8	5334587.4	409953.3
H6-16	306.7	5334588.9	409982.4
H6-17	306.2	5334590.4	410011.6

NO.	DATE	BY	REVISION
Geocres No. 32D-17			
HWY. 66		PROJECT No. 10-1191-0044	
SUBM'D. MT	CHKD.	DATE: DEC 2013	SITE:
DRAWN: JJJ	CHKD. SEMC	APPD. JMAC	DWG. C1



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

CONT No.
GWP No. 5091-07-00

HIGHWAY 66
HWY 66 - STA 14+020 TO 14+650
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

LEGEND

- Borehole
- CPT
- 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
BC2-1	309.4	5334378.5	409744.6
BC3-1	308.7	5334503.0	409859.6
BC4-1	305.2	5334678.1	410115.3
H6-1	309.2	5334401.0	409760.6
H6-3	309.2	5334422.4	409777.7
H6-5	309.0	5334439.8	409792.8
H6-7	309.0	5334450.4	409802.7
H6-9	308.9	5334468.8	409823.1
H6-12	308.3	5334532.7	409899.2
H6-14	307.4	5334561.1	409940.9
H6-16	306.7	5334588.9	409982.4
H6-18	305.7	5334616.0	410024.5
H6-CPT1	309.2	5334425.0	409774.7
H6-CPT2	309.0	5334452.9	409803.7
H6-CPT3	308.7	5334505.8	409859.9
H6-CPT4	307.8	5334535.5	409899.8
H6-CPT5	306.7	5334591.1	409982.2
H6-CPT6	305.2	5334648.3	410065.5
H6-CPT7	305.1	5334662.5	410090.2
H7-1	305.2	5334644.7	410065.5
H7-3	305.1	5334661.4	410090.4
H7-5	305.2	5334689.2	410131.9
H7-7	305.4	5334703.2	410152.6
H7-9	306.0	5334717.1	410173.4
H7-11	306.6	5334730.5	410193.3
H7-13	307.5	5334739.4	410206.6
H7-15	307.5	5334756.1	410231.5

NOTES

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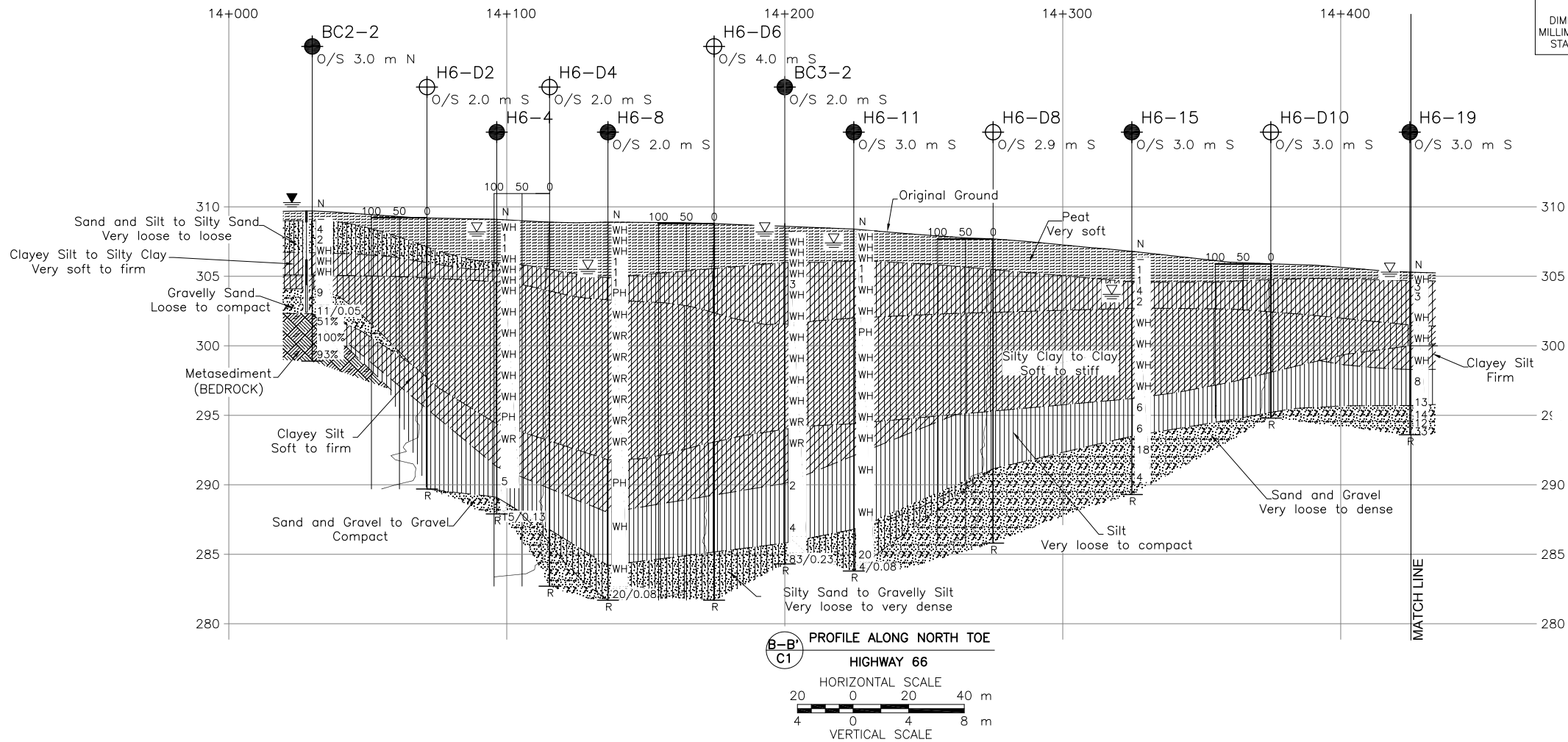
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.



REFERENCE

Base plans provided in digital format by MMM, drawing file nos. H3211009D16 ROLL PLAN-ULTIMATE and PDR.dwg, received DEC 3, 2012. Keyplan drawing file nos. H3211009G02 received JAN 24, 2013.

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

CONT No.
GWP No. 5091-07-00

HIGHWAY 66
HWY 66 - STA 14+020 TO 14+650
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

LEGEND

- Borehole
- Dynamic Cone Penetration Test
- 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 upon completion of drilling
- WL in piezometer, measured on Nov. 15, 2012

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
BC2-2	309.7	5334390.7	409727.1
BC3-2	308.8	5334515.4	409849.5
BC4-2	304.8	5334469.8	410109.4
H6-4	309.1	5334443.0	409771.6
H6-8	308.9	5334471.7	409801.1
H6-11	308.4	5334530.3	409870.3
H6-15	306.8	5334587.4	409953.3
H6-19	305.3	5334643.2	410036.3
H6-D2	309.2	5334422.0	409756.7
H6-D4	308.9	5334456.2	409786.2
H6-D6	308.8	5334497.1	409831.0
H6-D8	307.7	5334559.6	409911.8
H6-D10	305.9	5334615.3	409994.8
H7-2	305.1	5334667.1	410068.4
H7-6	305.0	5334710.3	410132.8
H7-10	305.5	5334734.9	410176.5
H7-14	306.9	5334760.2	410210.7
H7-D2	304.9	5334683.0	410093.9
H7-D3	304.4	5334697.8	410114.1
H7-D5	304.5	5334719.8	410156.6
H7-D7	306.4	5334749.1	410194.1

NOTES

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REFERENCE

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NO.	DATE	BY	REVISION
Geocres No. 32D-17			
HWY. 66	PROJECT No. 10-1191-0044		DIST.
SUBM'D. MT	CHKD.	DATE: DEC 2013	SITE:
DRAWN: JJJ	CHKD. SEMC	APPD. JMAC	DWG. C3

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

CONT No.
GWP No. 5091-07-00

HIGHWAY 66
HWY 66 - STA 14+020 TO 14+650
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

LEGEND

- Borehole - Current Investigation
- ⊕ Dynamic Cone Penetration Test
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- R Refusal
- ▽ WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
BC2-3	309.3	5334366.7	409761.6
BC3-3	308.6	5334490.5	409869.7
BC4-3	305.2	5334660.4	410121.2
H6-2	309.0	5334400.7	409783.2
H6-6	308.9	5334433.7	409810.4
H6-10	308.8	5334475.6	409850.4
H6-13	307.9	5334535.3	409927.7
H6-17	306.2	5334590.4	410011.6
H6-D1	309.6	5334379.1	409769.2
H6-D3	309.0	5334420.0	409798.0
H6-D5	308.9	5334448.8	409824.2
H6-D7	308.3	5334507.1	409887.7
H6-D9	307.0	5334562.4	409970.9
H6-D11	305.4	5334618.3	410053.1
H7-4	305.3	5334669.6	410133.1
H7-8	306.2	5334697.7	410171.4
H7-12	307.8	5334726.7	410209.2
H7-D1	305.5	5334639.7	410086.8
H7-D4	305.7	5334682.9	410151.2
H7-D6	307.0	5334711.6	410192.1
H7-D8	307.8	5334737.8	410225.8

NOTES

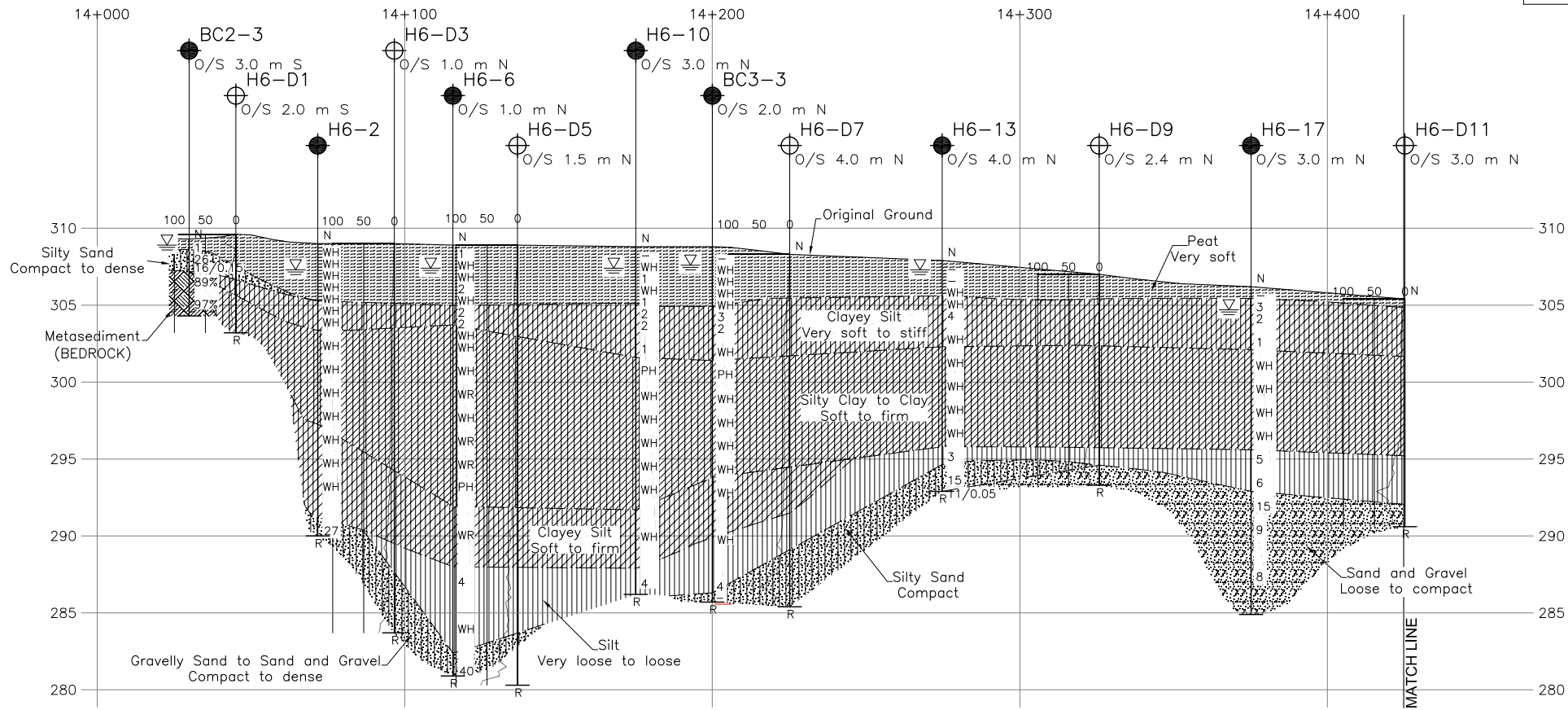
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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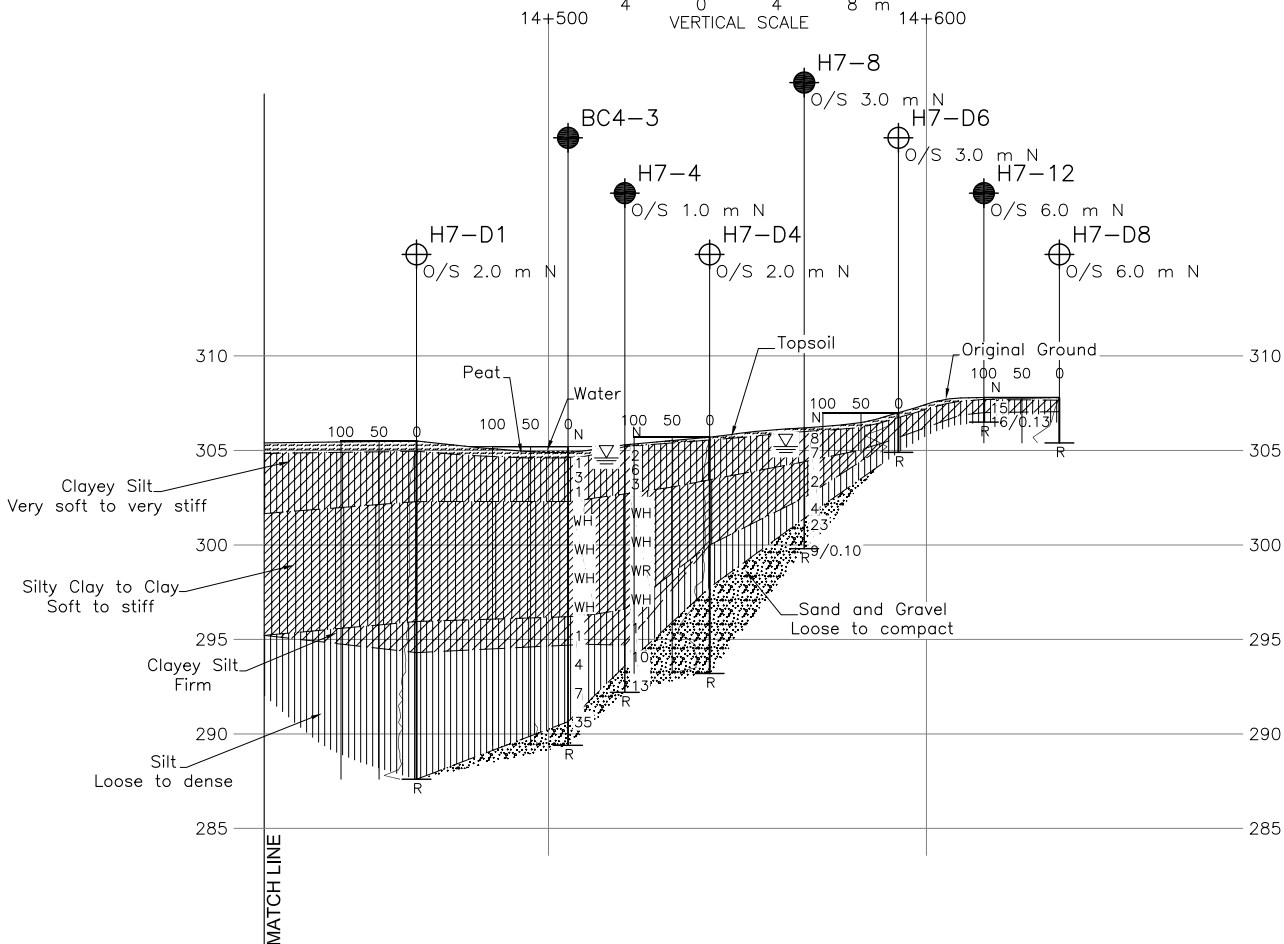
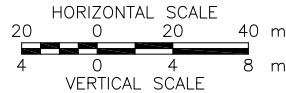
REFERENCE

Base plans provided in digital format by MMM, drawing file nos. H3211009D16 ROLL PLAN-ULTIMATE and PDR.dwg, received DEC 3, 2012. Keyplan drawing file nos. H3211009G02 received JAN 24, 2013.

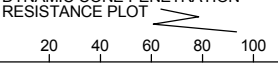








PROFILE ALONG SOUTH TOE

HIGHWAY 66




NO.	DATE	BY	REVISION
Geocres No. 32D-17			
HWY. 66	PROJECT NO. 10-1191-0044		DIST.
SUBM'D. MT	CHKD.	DATE: DEC 2013	SITE:
DRAWN: JJJ	CHKD. SEMC	APPD. JMAC	DWG. C4

PROJECT 10-1191-0044			RECORD OF BOREHOLE No H6-1			1 OF 2 METRIC					
G.W.P. 5091-07-00			LOCATION N 5334401.0; E 409760.6			ORIGINATED BY MT					
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY MT					
DATUM GEODETIC			DATE July 26, 2012			CHECKED BY SEMC					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
309.2	GROUND SURFACE										
0.0	PEAT (Fibrous) Very soft Black Moist		1	SS	1		309				
			2	SS	1		308				
			3	SS	1		307				51.9
306.8	Silty SAND, some clay Very loose Grey Moist to wet		4	SS	2		306				
305.9	CLAYEY SILT Very soft Grey Wet		5a	SS	WH		305				
3.3	Silt seams encountered between 3.3 m and 4.6 m depth.		5b	SS	WH		304				
			6	SS	WH		303				
303.5	SILTY CLAY to CLAY Soft to firm Grey Wet		7	SS	WH		302				
5.7			8	SS	WH		301				
			9	SS	WH		300				
	Silt seams encountered between 9.1 m and 13.3 m depth.		10	SS	WH		299				
			11	SS	WR		298				
			12	SS	WR		297				
295.9	CLAYEY SILT, trace sand Firm Grey Wet						296				
13.3	Silt seams encountered between 13.3 m and 16.0 m depth.						295				

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

Continued Next Page

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

PROJECT 10-1191-0044				RECORD OF BOREHOLE No H6-1				2 OF 2 METRIC									
G.W.P. 5091-07-00				LOCATION N 5334401.0; E 409760.6				ORIGINATED BY MT									
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT									
DATUM GEODETIC				DATE July 26, 2012				CHECKED BY SEMC									
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									
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100					WATER CONTENT (%) 20 40 60					
293.2	SAND and SILT, some clay, trace to some gravel Compact Grey Wet		13	SS	20.0.03		294										8 35 37 20
16.2								293									
292.2	SAND and GRAVEL, trace clay Grey Wet		14	AS	-												
17.0	END OF BOREHOLE AUGER REFUSAL																
Note: 1. Water level at a depth of 0.9 m below ground surface (Elev. 308.3 m) upon completion of drilling.																	

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



1 OF 2 **METRIC**

ORIGINATED BY MT

COMPILED BY MT

CHECKED BY SEMC

Continued Next Page

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

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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No H6-3		1 OF 2 METRIC	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334422.4; E 409777.7</u>		ORIGINATED BY <u>MT</u>	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>MT</u>	
DATUM <u>GEODETIC</u>		DATE <u>July 30, 2012</u>		CHECKED BY <u>SEMC</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L	20 40 60		
309.2 0.0	GROUND SURFACE PEAT (Fibrous) Very soft Black Moist to wet		1	SS	1	∇	309						
			2	SS	1		308						
			3	SS	WH		307						
			4	SS	1		306						
			5	SS	WH		305						
305.4 3.8	CLAYEY SILT Soft to firm Grey Wet Silt and sand seams encountered between 3.8 m and 5.3 m depth. Silt seams encountered between 5.3 and 6.9 m depth.		6	SS	WH		305						
			7	SS	WH		304						
							303						
			8	SS	WH		302						
302.3 6.9	SILTY CLAY to CLAY Soft Grey Wet						301						
			9	TO	PH		300						
							299						
			10	SS	WH		298						
						297							
			11	SS	WH	296							
						295							
295.9 13.3	CLAYEY SILT Soft to firm Grey Wet Silt seams encountered between 12.2 m and 13.3 m depth. Silt seams encountered between 13.3 m and 20.9 m depth. Sand seam at 13.8 m depth.		12	SS	WH								
			13	SS	WH								

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

Continued Next Page

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

PROJECT 10-1191-0044		RECORD OF BOREHOLE No H6-3				2 OF 2 METRIC								
G.W.P. 5091-07-00		LOCATION N 5334422.4; E 409777.7				ORIGINATED BY MT								
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT								
DATUM GEODETIC		DATE July 30, 2012				CHECKED BY SEMC								
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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 ---														
288.3 20.9	CLAYEY SILT Soft to firm Grey Wet Silt seams encountered from 13.3 m and 20.9 m depth.		14	SS	WR	294	+							
						293								
						292	+	4						
						291								
						290								
288.4 23.8	Silty SAND, some gravel, trace to some clay Loose Grey Wet		15	SS	WR	289	+	5						
						288								
						287								
						286								
285.4 23.8	END OF BOREHOLE AUGER REFUSAL Note: 1. Water level at a depth of 0.7 m below ground surface (Elev. 308.5 m) upon completion of drilling.													

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT		10-1191-0044		RECORD OF BOREHOLE No H6-4		1 OF 2 METRIC								
G.W.P.		5091-07-00		LOCATION		N 5334443.0; E 409771.6								
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers								
DATUM		GEODETIC		DATE		July 31, 2012								
						ORIGINATED BY MT								
						COMPILED BY MT								
						CHECKED BY SEMC								
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						
309.1	GROUND SURFACE							20 40 60 80 100	20 40 60					
0.0	PEAT (Fibrous) Very soft Black Wet		1	SS	WH									
			2	SS	1									
			3	SS	1									
			4	SS	WH									
306.0														
3.1	Silty SAND, trace to some clay Very loose Grey Wet		5	SS	WH									
305.4														
3.7	CLAYEY SILT Very soft Grey Wet		6	SS	WH									
304.6														
4.5	Silt seams encountered between 3.7 m and 4.5 m depth. SILTY CLAY to CLAY Soft Grey Wet Root encountered at 4.8 m depth.		7	SS	WH									
			8	SS	WH									
			9	SS	WH									
			10	SS	WH									
			11	SS	WH									
			12	SS	WH									
			13	TO	PH									
294.4														
14.7														

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

Continued Next Page

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

PROJECT		10-1191-0044		RECORD OF BOREHOLE No H6-4				2 OF 2 METRIC								
G.W.P.		5091-07-00		LOCATION		N 5334443.0; E 409771.6		ORIGINATED BY MT								
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY MT								
DATUM		GEODETIC		DATE		July 31, 2012		CHECKED BY SEMC								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
--- CONTINUED FROM PREVIOUS PAGE ---																
291.3	CLAYEY SILT Soft to firm Grey Wet Silt seams encountered between 14.7 m and 17.8 m depth.		14	SS	WR		294									
293								293								
292							292									
291							291									
290							290									
289.1	SILT, trace to some clay Loose Grey Wet		15	SS	5		289									
289							289									
287.9	GRAVEL, some sand, trace to some silt Compact Grey Wet		16	SS	15/0 13		288									
288							288									
21.2	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING) Note: 1. Water level at a depth of 0.9 m below ground surface (Elev. 308.2 m) upon completion of drilling.															

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No H6-5		1 OF 2 METRIC	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334439.8; E 409792.8</u>		ORIGINATED BY <u>MT</u>	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>MT</u>	
DATUM <u>GEODETIC</u>		DATE <u>August 7 and 8, 2012</u>		CHECKED BY <u>SEMC</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 (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED						
309.0	GROUND SURFACE														
0.0	PEAT (Fibrous) Very soft Black Wet		1	SS	WH										
			2	SS	WH										
			3	SS	WH										
			4	SS	WH										
			5	SS	WH										
305.3															
3.7	CLAYEY SILT, trace organics Firm Grey Wet Silt seams encountered between 3.7 m and 7.2 m depth.		6	SS	2										
			7	SS	WH										
			8	SS	WH										
301.8															
7.2	CLAY Soft Grey Wet		9	SS	WH										
			10	SS	WR										
			11	SS	WH										
			12	SS	WH										
295.7															
13.3	CLAYEY SILT Firm Grey Wet Silt seams encountered from 13.3 m to 20.9 m depth.		13	SS	WR										

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

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

PROJECT		10-1191-0044		RECORD OF BOREHOLE No H6-6		1 OF 2 METRIC											
G.W.P.		5091-07-00		LOCATION		N 5334433.7; E 409810.4											
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers											
DATUM		GEODETIC		DATE		August 9 and 10, 2012											
						ORIGINATED BY MT											
						COMPILED BY MT											
						CHECKED BY SEMC											
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	W _p	W	W _L	γ	GR	SA	SI	CL	
308.9	GROUND SURFACE																
0.0	PEAT (Fibrous) Very soft Black Wet		1	SS	1		308										
			2	SS	WH		307										
			3	SS	WH		306										
			4	SS	2		305										
			5	SS	WH		304										
305.0	CLAYEY SILT, trace organics Very soft Grey Wet		6	SS	2		303										
303.7	Silt seams encountered between 3.9 m and 5.2 m depth.		7	SS	2		302										
5.2	SILTY CLAY TO CLAY, trace sand Soft to firm Grey Wet		8	SS	WH		301										
	Sand seams encountered between 5.3 m and 6.2 m and at 7.5 m depth.		9	SS	WH		300										
			10	SS	WH		299										
			11	SS	WR		298										
			12	SS	WH		297										
			13	SS	WR		296										
			14	SS	WR		295										
							294										

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

Continued Next Page

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

PROJECT		10-1191-0044		RECORD OF BOREHOLE No H6-6		2 OF 2 METRIC											
G.W.P.		5091-07-00		LOCATION		N 5334433.7; E 409810.4											
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers											
DATUM		GEODETIC		DATE		August 9 and 10, 2012											
				ORIGINATED BY		MT											
				COMPILED BY		MT											
				CHECKED BY		SEMC											
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 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL				
--- CONTINUED FROM PREVIOUS PAGE ---																	
	SILTY CLAY TO CLAY, trace sand Soft to firm Grey Wet		15	TO	PH		293										
291.9	CLAYEY SILT Firm Grey Wet Silt seams encountered between 17.0 m and 20.9 m depth.						292										
17.0							291										
								290									
			16	SS	WR		289										
288.0	SILT, trace to some clay, trace sand Very loose Grey Wet						288										
20.9							287										
							286										
							285										
			17	SS	4		284										
			18	SS	WH		283										
282.4	SAND and GRAVEL, some silt, trace clay Dense Grey to brown Wet						282										
26.5							281										
280.9	END OF BOREHOLE AUGER REFUSAL		19	SS	40												
28.0	Note: 1. Water level at a depth of 1.5 m below ground surface (Elev. 307.4 m) upon completion of drilling.																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

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

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No H6-7		3 OF 3 METRIC	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334450.4; E 409802.7</u>		ORIGINATED BY <u>MT</u>	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>MT</u>	
DATUM <u>GEODETIC</u>		DATE <u>August 12 and 13, 2012</u>		CHECKED BY <u>SEMC</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		GR	SA	SI	CL	
								○ UNCONFINED	+	FIELD VANE	● QUICK TRIAXIAL	×	REMOULDED	WATER CONTENT (%)							
	--- CONTINUED FROM PREVIOUS PAGE --- END OF BOREHOLE AUGER REFUSAL Note: 1. Water level at a depth of 1.6 m below ground surface (Elev. 307.4 m) upon completion of drilling.																				

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044		RECORD OF BOREHOLE No H6-7A				1 OF 1 METRIC							
G.W.P. 5091-07-00		LOCATION N 5334452.0; E 409801.0				ORIGINATED BY MT							
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT							
DATUM GEODETIC		DATE August 23, 2012				CHECKED BY SEMC							
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					
309.1	GROUND SURFACE						20 40 60 80 100						
0.0	For stratigraphy see Record of Borehole H6-7.						20 40 60 80 100						
303.0													
6.1	CLAYEY SILT		1	TO	PH							16.9	
301.9	Faint irregular layering noted throughout Shelby tube sample.												
7.2	CLAY		2	TO	PH								
	Pockets of organics noted in Shelby tube samples 2 and 4.												
			3	TO	PH								
			4	TO	PH								
			5	TO	PH								
296.5	END OF BOREHOLE												
12.6	Note: 1. Water level at a depth of 1.2 m below ground surface (Elev. 307.9 m) upon completion of drilling.												

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

PROJECT		10-1191-0044		RECORD OF BOREHOLE No H6-8		2 OF 2 METRIC													
G.W.P.		5091-07-00		LOCATION		N 5334471.7; E 409801.1													
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers													
DATUM		GEODETIC		DATE		August 13 and 14, 2012													
				ORIGINATED BY		MT													
				COMPILED BY		MT													
				CHECKED BY		SEMC													
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					
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100	○ UNCONFINED + FIELD VANE	W _p	W	W _L							
								20 40 60 80 100	● QUICK TRIAXIAL × REMOULDED										
291.8	SILTY CLAY to CLAY Soft to firm Grey Wet Silt seams encountered between 15.2 m and 17.1 m depth.		14	SS	WR		293												
291.8							292												
17.1	CLAYEY SILT Firm Grey Wet						291												
			15	TO	PH		290												
							289												
288.0	SILT Very loose Grey Wet						288												
20.9			16	SS	WH		287												
							286												
							285												
284.2	Silty SAND, trace to some clay Very loose Grey Wet		17a	SS	WH		284												
24.7			17b				283												
							282												
281.8	SAND and GRAVEL Compact Grey to brown Wet END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING) Note: 1. Water level at a depth of 3.4 m below ground surface (Elev. 305.5 m) upon completion of drilling.		18	SS	20/0.08														
27.2																			

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No H6-9		1 OF 2 METRIC	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334468.8; E 409823.1</u>		ORIGINATED BY <u>MT</u>	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>MT</u>	
DATUM <u>GEODETIC</u>		DATE <u>September 23 and 24, 2012</u>		CHECKED BY <u>SEMC</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			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 80 100	W _p	W	W _L		
308.9	GROUND SURFACE													
0.0	PEAT (Fibrous) Very soft Black Wet		1	SS	WH									
			2	SS	WH									
			3	SS	WH									
			4	SS	WH									
			5	SS	WH									
305.2														
3.7	CLAYEY SILT, trace sand, trace organics Firm Grey Wet Silt seams encountered between 3.7 m and 5.6 m depth.		6	SS	WH									
			7	SS	2									
			8	SS	WH									
302.0														
6.9	SILTY CLAY Soft to firm Grey Wet Silt seams encountered between 8.7 m and 9.9 m depth.		9	SS	WH									
				10	SS	WH								
				11	SS	WH								
				12	SS	WR								
			13	SS	WH									
294.1														
14.8														







SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

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

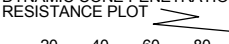

PROJECT 10-1191-0044		RECORD OF BOREHOLE No H6-10		1 OF 2 METRIC	
G.W.P. 5091-07-00		LOCATION N 5334475.6; E 409850.4		ORIGINATED BY MT	
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY MT	
DATUM GEODETIC		DATE September 25, 2012		CHECKED BY SEMC	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			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 (%)		
308.8	GROUND SURFACE																
0.0	PEAT (Fibrous) Very soft Black Wet		1	AS	-		308										
			2	SS	WH												
			3	SS	1			307									
			4	SS	WH			306									
			5	SS	1			305									
305.1	CLAYEY SILT Very soft Grey Wet		6	SS	2			304								0 0 61 39	
3.7				7	SS		2										
				8	SS		1		303								
								302									
	SILT SEAMS ENCOUNTERED BETWEEN 4.5 m AND 7.2 m DEPTH.							301									
								300									
								299									
								298									
							297										
							296										
							295										
							294										
301.6	SILTY CLAY TO CLAY Soft to firm Grey Wet		9	TO	PH												
7.2																	
	ROOTS ENCOUNTERED AT 9.7 m, 11.3 m AND 14.3 m DEPTH.		10	SS	WH												
			11	SS	WH												
			12	SS	WH												
			13	SS	WH												

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

PROJECT 10-1191-0044			RECORD OF BOREHOLE No H6-10				2 OF 2 METRIC				
G.W.P. 5091-07-00			LOCATION N 5334475.6; E 409850.4				ORIGINATED BY MT				
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT				
DATUM GEODETIC			DATE September 25, 2012				CHECKED BY SEMC				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
291.7	SILTY CLAY to CLAY Soft to firm Grey Wet		14	SS	WH		293				
17.1	CLAYEY SILT Soft to firm Grey Wet Silt seams encountered between 17.1 m and 20.9 m depth.						292				
							291				
							290				
287.9	SILT, trace to some clay Loose Grey Wet Wood fibre in tip of spoon. Spoon attempted at 22.6 m depth, bouncing.		15	SS	WH		289				
20.9							288				
							287				
286.2	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING) Note: 1. Water level at a depth of 1.4 m below ground surface (Elev. 307.4 m) upon completion of drilling.										
22.6											

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

PROJECT 10-1191-0044		RECORD OF BOREHOLE No H6-11				2 OF 2 METRIC												
G.W.P. 5091-07-00		LOCATION N 5334530.3; E 409870.3				ORIGINATED BY MT												
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT												
DATUM GEODETIC		DATE October 16, 2012				CHECKED BY SEMC												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m³	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 80 100	20 40 60	20 40 60	20 40 60						
--- CONTINUED FROM PREVIOUS PAGE ---																		
292.1	CLAYEY SILT Firm Grey Wet Silt seams encountered between 14.0 m and 16.3 m.						293										NP	
16.3	SILT, trace clay, some sand Very loose Grey Wet		14	SS	WH		292											
							291											
							290											
							289											
			15	SS	WH		288											
286.8	Gravelly SILT, trace to some sand, trace to some clay Compact Grey Wet						287											
21.6			16	SS	20		286											
							285										27 10 51 12	
283.8			17	SS	4/0.08		284											
24.6	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING) Note: 1. Water level at a depth of 1.0 m below ground surface (Elev. 307.4 m) upon completion of drilling.																	

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

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

PROJECT 10-1191-0044				RECORD OF BOREHOLE No H6-12				2 OF 2 METRIC						
G.W.P. 5091-07-00				LOCATION N 5334532.7; E 409899.2				ORIGINATED BY MT						
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT						
DATUM GEODETIC				DATE October 17 and 18, 2012				CHECKED BY SEMC						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		
293.1	---							20 40 60 80 100						
15.2	SILT, some clay Very loose to loose Grey Wet		13	SS	WH		293	20 40 60 80 100					NP	0 0 84 16
							292							
							291							
							290							
			14	SS	9		289							
							288							
288.2	SAND and GRAVEL, some silt, trace clay Dense Grey Wet						287							
20.1			15	SS	47		286							
286.4	END OF BOREHOLE START OF DCPT						285							
21.9							284							
283.1														
25.2	END OF DCPT REFUSAL TO FURTHER PENETRATION 28 BLOWS / 0.24 M (HAMMER BOUNCING) Note: 1. Water level at a depth of 1.6 m below ground surface (Elev. 306.7 m) upon completion of drilling.													

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044		RECORD OF BOREHOLE No H6-13		1 OF 2 METRIC	
G.W.P. 5091-07-00		LOCATION N 5334535.3; E 409927.7		ORIGINATED BY MT	
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY MT	
DATUM GEODETIC		DATE October 18 and 19, 2012		CHECKED BY SEMC	

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 (%)				
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
307.9	GROUND SURFACE													
0.0	PEAT (Fibrous) Very soft Black Wet		1	AS	-									
			2	AS	-									
			3	SS	WH									
305.6														
2.3	CLAYEY SILT, trace organics Firm Grey Wet		4	SS	WH									
	Trace organics above 4.1 m depth.		5	SS	4									
	Silt seams encountered between 4.1 m and 5.6 m depth.		6	SS	WH									
302.3														
5.6	CLAY Soft to firm Grey Wet		7	SS	WH									
			8	SS	WH									
	Silt seams encountered between 8.7 m and 12.1 m depth.		9	SS	WH									
295.8														
12.1	SILT Very loose Grey Wet		11	SS	3									
294.6														
13.3	Silty SAND, trace to some gravel, trace clay Compact Grey Wet		12	SS	15									
292.9			13	SS	11/0.05									

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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No H6-13		2 OF 2 METRIC	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334535.3; E 409927.7</u>		ORIGINATED BY <u>MT</u>	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>MT</u>	
DATUM <u>GEODETIC</u>		DATE <u>October 18 and 19, 2012</u>		CHECKED BY <u>SEMC</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	W _p	W	W _L					
15.0	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING) Note: 1. Water level at a depth of 0.6 m below ground surface (Elev. 307.3 m) upon completion of drilling.																			

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044			RECORD OF BOREHOLE No H6-14			1 OF 2 METRIC																
G.W.P. 5091-07-00			LOCATION N 5334561.1; E 409940.9			ORIGINATED BY MT																
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY MT																
DATUM GEODETIC			DATE October 19 and 22 and November 3, 2012			CHECKED BY SEMC																
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						
								20 40 60 80 100	20 40 60 80 100	20 40 60	W _p W W _L	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	
307.4	GROUND SURFACE																					
0.0	PEAT (Fibrous) Very soft Black Wet		1	AS	-		307															
			2	AS	-																	
305.6							306															
1.8	CLAYEY SILT, trace sand Stiff Grey Wet Trace organics above 3.8 m depth.		3	SS	1																	
			4	SS	WH		305															
	Silt seams encountered between 3.8 m and 4.9 m depth.																					
			5	SS	WH		304															
302.5							303															
4.9	SILTY CLAY to CLAY Soft to firm Grey Wet		6	SS	WH		302															
			7	TO	PH		301															
	Silt seams encountered at 7.5 m depth.						300															
			8	SS	WH		299															
							298															
	Silt seams encountered between 9.9 m and 11.3 m depth.		9	SS	WH		297															
296.1							296															
11.3	Sandy SILT, some gravel, trace to some clay Loose to dense Grey Wet No recovery in Sample 10.		10	SS	4		295															
			11	SS	8		294															
							293															
			12	SS	36																	

Continued Next Page

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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>				RECORD OF BOREHOLE No H6-14				2 OF 2 METRIC										
G.W.P. <u>5091-07-00</u>				LOCATION <u>N 5334561.1; E 409940.9</u>				ORIGINATED BY <u>MT</u>										
DIST <u> </u> HWY <u>66</u>				BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>MT</u>										
DATUM <u>GEODETIC</u>				DATE <u>October 19 and 22 and November 3, 2012</u>				CHECKED BY <u>SEMC</u>										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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 ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> W_p W W_L </div>						
291.6			13	SS	25	292									○			
15.8	END OF BOREHOLE Note: 1. Water level at a depth of 5.2 m below ground surface (Elev. 302.2 m) upon completion of drilling.																	

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No H6-15				2 OF 2 METRIC											
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334587.4; E 409953.3</u>				ORIGINATED BY <u>MT</u>											
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>MT</u>											
DATUM <u>GEODETIC</u>		DATE <u>November 3 and 4, 2012</u>				CHECKED BY <u>SEMC</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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 ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> W_p W W_L </div>					
289.3	SAND and GRAVEL, some silt, trace clay Very loose to compact Grey Wet	13	SS	4		291											
						290											
17.5	END OF BOREHOLE AUGER REFUSAL Note: 1. Water level at a depth of 3.1 m below ground surface (Elev. 303.7 m) upon completion of drilling.																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044		RECORD OF BOREHOLE No H6-16				1 OF 2 METRIC								
G.W.P. 5091-07-00		LOCATION N 5334588.9; E 409982.4				ORIGINATED BY MT								
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT								
DATUM GEODETIC		DATE November 4 and 5, 2012				CHECKED BY SEMC								
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									
306.7	GROUND SURFACE													
0.0	PEAT (Fibrous) Very soft Black Wet		1	AS	-	▽	306							518
			2	SS	WH									
305.0							305							
1.7	CLAYEY SILT, trace organics Stiff Grey Wet		3	SS	WH									
	Silt seams encountered between 3.0 m and 3.9 m depth.						304							
			4	SS	WH									
302.8							303							
3.9	CLAY Soft to firm Grey Wet													
			5	SS	WH									
							302							
							301							
			6	SS	WH									
						300								
						299								
			7	SS	WH									
						298								
298.0														
8.7	CLAYEY SILT Firm to stiff Grey Wet													
	Silt seams encountered between 9.1 m and 10.6 m depth.		8	SS	WH									
						297								
296.1														
10.6	SILT, trace to some clay Loose Grey Wet		9	SS	5									
						296								
						295								
295.0														
11.7	Silty SAND and GRAVEL, trace clay Compact to dense Grey Wet		10	SS	16									
						294								
						293								
			11	SS	33/0.15									
	Spoon bouncing at 13.9 m depth.													
292.3														
14.4														

Continued Next Page

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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No H6-16		2 OF 2 METRIC	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334588.9; E 409982.4</u>		ORIGINATED BY <u>MT</u>	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>MT</u>	
DATUM <u>GEODETIC</u>		DATE <u>November 4 and 5, 2012</u>		CHECKED BY <u>SEMC</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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								○ UNCONFINED	● QUICK TRIAXIAL	+	×	FIELD VANE	REMOULDED	W _p	W		W _L				
	END OF BOREHOLE AUGER REFUSAL Note: 1. Water level at a depth of 1.2 m below ground surface (Elev. 305.5 m) upon the completion of drilling.																				

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

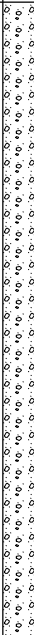
PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE		No H6-17		1 OF 2		METRIC	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334590.4; E 410011.6</u>				ORIGINATED BY		<u>MT</u>	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY		<u>MT</u>	
DATUM <u>GEODETIC</u>		DATE <u>November 5 and 6, 2012</u>				CHECKED BY		<u>SEMC</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 (%)							
								○ UNCONFINED		+ FIELD VANE						● QUICK TRIAXIAL		× REMOULDED					
								20	40	60						80	100	20	40	60	20	40	60
306.2	GROUND SURFACE																						
0.0	PEAT (Fibrous) Black Grey		1	AS	-		306																
305.4																							
0.8	CLAYEY SILT, trace sand, trace organics Stiff Grey Wet		2	SS	3		305																
			3	SS	2		304																
	Silt seams encountered between 3.0 m and 4.1 m depth.		4	SS	1		303																
302.1																							
4.1	CLAY Soft Grey Wet		5	SS	WH		302																
	Silt seams encountered between 4.6 m and 5.2 m depth.						301																
			6	SS	WH		300																
							299																
	Silt seams encountered between 7.6 m and 10.6 m depth.		7	SS	WH		298																
							297																
			8	SS	WH																		
							296																
295.6																							
10.6	SILT, some clay Loose Grey Wet		9	SS	5		295																
							294																
			10	SS	6																		
							293																
292.9																							
13.3	SAND and GRAVEL, trace to some silt, trace clay Loose to compact Grey Wet		11	SS	15		292																

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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>			RECORD OF BOREHOLE No H6-17			2 OF 2 METRIC		
G.W.P. <u>5091-07-00</u>			LOCATION <u>N 5334590.4; E 410011.6</u>			ORIGINATED BY <u>MT</u>		
DIST <u> </u> HWY <u>66</u>			BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>			COMPILED BY <u>MT</u>		
DATUM <u>GEODETIC</u>			DATE <u>November 5 and 6, 2012</u>			CHECKED BY <u>SEMC</u>		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L WATER CONTENT (%) 20 40 60
--- CONTINUED FROM PREVIOUS PAGE ---								
	SAND and GRAVEL, trace to some silt, trace clay Loose to compact Grey Wet		12	SS	9		291	
							290	
							289	
							288	
			13	SS	8		287	
							286	
284.9	Approximately 2.4 m of heave inside augers at 21.3 m depth.						285	
21.3	END OF BOREHOLE Note: 1. Water level at a depth of 1.5 m below ground surface (Elev. 304.7 m) upon completion of drilling.							

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



PROJECT 10-1191-0044			RECORD OF BOREHOLE No H6-18			1 OF 1 METRIC								
G.W.P. 5091-07-00			LOCATION N 5334616.0; E 410024.5			ORIGINATED BY GM								
DIST _____ HWY 66			BOREHOLE TYPE Portable Equipment, NW Casing, Wash Boring			COMPILED BY MT								
DATUM GEODETIC			DATE November 6, 2012			CHECKED BY SEMC								
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						
305.7	GROUND SURFACE							20 40 60 80 100	20 40 60					
0.0	PEAT (Fibrous) Very soft Black Wet		1	SS	WH									
305.2														
0.5	CLAYEY SILT Very soft to firm Grey Wet		2	SS	7									
	Silt seams encountered between 2.0 m and 2.7 m depth.		3	SS	5									
			4	SS	1									
303.0														
2.7	CLAY Soft to firm Grey Wet		5	SS	WH									
			6	SS	WH									
			7	TO	PH									
298.8														
6.9	CLAYEY SILT Firm Grey Wet		8	SS	WH									
	Silt seams encountered between 6.9 m and 8.5 m depth.													
297.2														
8.5	SILT, some clay Loose to compact Grey Wet		9	SS	9									
			10	SS	13									
294.4														
11.3	END OF BOREHOLE SPOON REFUSAL AND REFUSAL TO FURTHER CASING PENETRATION (HAMMER BOUNCING)		11	SS	10/0.05									
	Note: 1. Water level at ground surface upon completion of drilling.													

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

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

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

PROJECT <u>10-1191-0044</u>			RECORD OF BOREHOLE No H6-S1			2 OF 2 METRIC											
G.W.P. <u>5091-07-00</u>			LOCATION <u>N 5334425.9; E 409776.7</u>			ORIGINATED BY <u>BM</u>											
DIST <u> </u> HWY <u>66</u>			BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing and Wash Boring</u>			COMPILED BY <u>AC</u>											
DATUM <u>GEODETIC</u>			DATE <u>May 1, 2013</u>			CHECKED BY <u>SEMC</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 (%) W _p — W — W _L			γ kN/m ³	GR SA SI CL
							20 40 60 80 100										
293.6 15.6	CLAYEY SILT Soft to stiff Grey Wet						294										
							293										
			6	TO	PH		292										
							291										
			7	SS	1		290										
288.5 20.7	Silty SAND, trace clay Loose Grey Wet						289										
			8	SS	7		288										
287.3 21.9	END OF BOREHOLE																
	Note: 1. Water level at ground surface upon completion of drilling.																

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

PROJECT 10-1191-0044				RECORD OF BOREHOLE No H6-S2				2 OF 2 METRIC									
G.W.P. 5091-07-00				LOCATION N 5334514.6; E 409850.1				ORIGINATED BY BM									
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY AC									
DATUM GEODETIC				DATE May 2, 2013				CHECKED BY SEMC									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT W _p W 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 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100			
--- CONTINUED FROM PREVIOUS PAGE ---																	
	SILTY CLAY Soft to firm Grey Wet						293										
							292										
							291										
			5	TO	PM		290										
289.3	CLAYEY SILT, trace sand Firm Grey Wet						289										
19.4			2	SS	WH		288										
							287										
286.3	SILT, trace clay, trace sand Grey Wet						286										
22.4			3	SS													
285.2	END OF BOREHOLE																
23.5	Note: 1. Water level at ground surface upon completion of drilling.																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044				RECORD OF BOREHOLE No H6-S3				1 OF 2 METRIC				
G.W.P. 5091-07-00				LOCATION N 5334520.6; E 409877.5				ORIGINATED BY BM				
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing and Wash Boring				COMPILED BY AC				
DATUM GEODETIC				DATE May 3, 2013				CHECKED BY SEMC				
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT		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 (%)		
308.3 0.0	GROUND SURFACE Hollow stem augers advanced from ground surface to 2.7 m depth without sampling.						20 40 60 80 100	20 40 60				
305.6 2.7	CLAYEY SILT, trace sand Soft Grey Wet		1	SS	3							
304.5 3.8	SILTY CLAY Soft to firm Grey Wet		2	TO	PM							
			3	TO	PM							
			4	TO	PM							
	No recovery in Shelby tube Sample 4.		5	TO	PM							
	No recovery in Shelby tube Sample 5.											

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

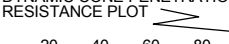

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+ 3, x 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE



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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT		10-1191-0044		RECORD OF BOREHOLE No H6-S4		1 OF 1 METRIC				
G.W.P.		5091-07-00		LOCATION		N 5334592.2; E 409983.8				
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers				
DATUM		GEODETIC		DATE		May 3, 2013				
ORIGINATED BY		BM		COMPILED BY		AC				
CHECKED BY		SEMC								
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
306.7 0.0	GROUND SURFACE Hollow stem augers advanced from ground surface to 2.7 m depth without sampling.									
304.0 2.7	CLAYEY SILT Firm to stiff Grey Wet Silt seams encountered between 2.7 m and 4.3 m depth.		1	TO	PH				19.5	
302.4 4.3	SILTY CLAY Firm Grey Wet									
298.2 8.5	CLAYEY SILT Firm to stiff Grey Wet		3	TO	PM					
296.8 9.9	SILT, trace to some clay, trace to some sand Loose to compact Grey Wet		4	SS	6					
294.2 12.5	END OF BOREHOLE Note: 1. Water level at ground surface upon completion of drilling.		5	SS	11					

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No H7-1		1 OF 2 METRIC	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334644.7; E 410065.5</u>		ORIGINATED BY <u>MT</u>	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>MT</u>	
DATUM <u>GEODETIC</u>		DATE <u>August 27, 2012</u>		CHECKED BY <u>SEMC</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			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		
								20 40 60 80 100	20 40 60 80 100	20 40 60				
305.2	GROUND SURFACE													
0.9	TOPSOIL		1	SS	4									
	CLAYEY SILT, trace to some sand, trace organics, layered Soft to firm Grey to brown Moist		2	SS	4									0 0 60 40
	Silt seams encountered between 1.5 m and 2.6 m depth.		3	SS	2									
302.6														
2.6	CLAY Soft to firm Grey Wet		4	SS	WH									
	Silt seams encountered between 2.6 m and 10.2 m depth.													
			5	SS	WH									
			6	SS	WH									
			7	TO	PH									
			8	SS	WH									
295.0														
10.2	CLAYEY SILT Firm to stiff Grey Wet		9	SS	WH									
	Silt seams encountered between 10.2 m and 12.1 m depth.													
293.1														
12.1	SILT Loose Grey Wet		10	SS	7									
292.5														
12.7	SAND and GRAVEL, trace to some silt Compact Grey Wet		11	SS	13									
290.6														
14.6														

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

Continued Next Page

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

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No H7-1		2 OF 2 METRIC	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334644.7; E 410065.5</u>		ORIGINATED BY <u>MT</u>	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>MT</u>	
DATUM <u>GEODETIC</u>		DATE <u>August 27, 2012</u>		CHECKED BY <u>SEMC</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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								○ UNCONFINED	● QUICK TRIAXIAL	+	×	FIELD VANE	REMOULDED	W _p	W		W _L				
	END OF BOREHOLE AUGER REFUSAL																				
	Note: 1. Water level at a depth of 0.9 m below ground surface (Elev. 304.3 m) upon completion of drilling.																				

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT		10-1191-0044		RECORD OF BOREHOLE No H7-2				1 OF 1 METRIC					
G.W.P.		5091-07-00		LOCATION		N 5334667.1; E 410068.4		ORIGINATED BY		GM			
DIST		HWY 66		BOREHOLE TYPE		Portable Equipment, NW Casing, Wash Boring		COMPILED BY		MT			
DATUM		GEODETIC		DATE		November 13, 2012		CHECKED BY		SEMC			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60	W _p W W _L			
305.1	WATER SURFACE												
0.0	WATER												
0.2	CLAYEY SILT, trace organics Soft Brown Wet		1	SS	3								
			2	SS	4								
	Silt seams encountered between 1.4 m and 2.6 m depth.		3	SS	1								
302.3													
2.8	CLAY Soft to firm Grey Wet		4	SS	WH								
			5	TO	PH								
	Silt seams encountered between 5.6 m and 8.7 m depth.		6	SS	WH								
296.2													
8.9	CLAYEY SILT Soft Grey Wet		8	SS	4								
294.7													
10.4	SILT Loose Grey Wet		9	SS	7								
293.2													
11.9	SAND and GRAVEL, some silt, tace clay Compact to very dense Grey Wet		10	SS	28								
292.0													
13.1	END OF BOREHOLE SPOON REFUSAL AND REFUSAL TO FURTHER CASING PENETRATION (HAMMER BOUNCING)		11	SS	50/0.08								

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

PROJECT 10-1191-0044			RECORD OF BOREHOLE No H7-3			1 OF 2 METRIC															
G.W.P. 5091-07-00			LOCATION N 5334661.4; E 410090.4			ORIGINATED BY MT															
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY MT															
DATUM GEODETIC			DATE August 26, 2012			CHECKED BY SEMC															
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		
305.1	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	20 40 60											
0.9	TOPSOIL		1	SS	2		305														
	CLAYEY SILT, trace to some sand Very soft to soft Brown Moist		2	SS	4		304														
	Trace organics above 0.6 m depth.		3	SS	2		303														
	Silt seams encountered between 0.6 m and 2.6 m depth.																				
302.5	CLAY		4	SS	WH		302														
2.6	Firm Grey Wet		5	TO	PH		301														
	Silt seams encountered between 4.6 m and 5.6 m depth.		6	SS	WH		300														
							299														
			7	SS	WH		298														
	Silt seams encountered between 7.6 m and 8.7 m depth.		8	SS	WH		297														
296.4	CLAYEY SILT		9	SS	WH		296														
8.7	Firm Grey Wet						295														
	Silt seams encountered between 8.7 m to 12.4 m depth.		10	SS	WH		294														
							293														
292.7	SILT, trace to some clay		11	SS	5		292														
12.4	Loose to compact Grey Wet		12	SS	10		291														
290.3																					
14.8																					

Continued Next Page

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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



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

PROJECT 10-1191-0044			RECORD OF BOREHOLE No H7-4			1 OF 1 METRIC								
G.W.P. 5091-07-00			LOCATION N 5334669.6; E 410133.1			ORIGINATED BY MT								
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY MT								
DATUM GEODETIC			DATE August 25, 2012			CHECKED BY SEMC								
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						
305.3	GROUND SURFACE							20 40 60 80 100	20 40 60					
0.0	TOPSOIL		1	SS	2	▽	305							
	CLAYEY SILT to SILTY CLAY, trace organics, layered Firm Brown Moist		2	SS	6		304							
			3	SS	3		303							
302.7							302							
2.6	CLAY Firm Grey Wet		4	SS	WH		301							
			5	SS	WH		300							
	Silt seams encountered between 6.1 m and 8.5 m depth.		6	SS	WR		299							
			7	SS	WH		298							
							297							
296.6							296							
8.7	CLAYEY SILT Firm Grey Wet		8	SS	1									
	Silt seams encountered between 8.7 m to 10.6 m depth.					295								
294.7						294								
10.6	SILT, trace to some clay Loose Grey Wet		9	SS	10									
293.6						293								
11.7	SAND and GRAVEL, trace to some silt Compact Grey Wet		10	SS	13									
292.2														
13.1	END OF BOREHOLE AUGER REFUSAL													
	Note: 1. Water level at a depth of 0.7 m below ground surface (Elev. 304.6 m) upon completion of drilling.													


SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044			RECORD OF BOREHOLE No H7-5			1 OF 2 METRIC						
G.W.P. 5091-07-00			LOCATION N 5334689.2; E 410131.9			ORIGINATED BY GM						
DIST _____ HWY 66			BOREHOLE TYPE Portable Equipment, NW Casing, Wash Boring			COMPILED BY MT						
DATUM GEODETIC			DATE October 31, 2012			CHECKED BY SEMC						
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT REMARKS			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL
305.2	WATER SURFACE											
0.0	WATER											
304.6	PEAT Very soft to soft Black Wet		1	SS	4		305					
0.6	CLAYEY SILT, trace to some sand, layered Soft to firm Grey to brown Wet		2	SS	8		304					
			3	SS	4							
303.1	CLAY Very soft to firm Grey Wet		4	SS	WH		303					
2.1							302					
			5	SS	WH							
							301					
							300					
			6	SS	WH		299					
							298					
			7	SS	WH		297					
296.6	CLAYEY SILT Soft Grey Wet		8	SS	3		296					
8.6							295					
295.1	SILT, trace to some clay Loose Grey Wet		9	SS	6		294					
10.1							293					
293.1	SAND, some gravel, trace to some clay Loose to compact Grey to brown Wet		10	SS	9		292					
12.1							291					
			11	SS	17							

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

Continued Next Page

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

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No H7-5				2 OF 2 METRIC												
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334689.2; E 410131.9</u>				ORIGINATED BY <u>GM</u>												
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Portable Equipment, NW Casing, Wash Boring</u>				COMPILED BY <u>MT</u>												
DATUM <u>GEODETIC</u>		DATE <u>October 31, 2012</u>				CHECKED BY <u>SEMC</u>												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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 ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> W_p W W_L </div>						
289.4 15.8	END OF BOREHOLE REFUSAL TO FURTHER CASING ADVANCEMENT Note: 1. Water level at 1.5 m above ground surface (Elev. 306.5 m) upon completion of drilling.		12	SS	12	290												

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044			RECORD OF BOREHOLE No H7-6			1 OF 1 METRIC															
G.W.P. 5091-07-00			LOCATION N 5334710.3; E 410132.8			ORIGINATED BY GM															
DIST _____ HWY 66			BOREHOLE TYPE Portable Equipment, NW Casing, Wash Boring			COMPILED BY MT															
DATUM GEODETIC			DATE November 3, 2012			CHECKED BY SEMC															
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		
305.0	WATER SURFACE							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p — W — W _L 20 40 60			kN/m ³					
0.0	WATER																				
304.4	0.6		1	SS	WH		304														
303.7	1.3		2	SS	2		303														
	CLAYEY SILT to SILTY CLAY		3	SS	3		302														
	CLAYEY SILT to SILTY CLAY						301														
301.8	3.2		4	SS	WH		300														
	SILTY CLAY to CLAY						299														
	Silt seams encountered between 4.1 m and 9.3 m depth.		5	SS	WH		298														
			6	SS	1		297														
			7	SS	WH		296														
295.8	9.2		8	SS	7		295														
	CLAYEY SILT						294														
294.3	10.7		9	SS	10		293														
	SILT						292														
292.7	12.3		10	SS	40																
	SAND and GRAVEL, trace to some silt, trace clay																				
291.5	13.5																				
	Spoon attempted at 13.5 m depth, bouncing.																				
	END OF BOREHOLE SPOON REFUSAL AND REFUSAL TO FURTHER CASING PENETRATION (HAMMER BOUNCING)																				

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

PROJECT		10-1191-0044		RECORD OF BOREHOLE No H7-7				1 OF 1 METRIC						
G.W.P.		5091-07-00		LOCATION		N 5334703.2; E 410152.6		ORIGINATED BY MT						
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY MT						
DATUM		GEODETIC		DATE		August 24 and 25, 2012		CHECKED BY SEMC						
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						
305.4	GROUND SURFACE							20 40 60 80 100	20 40 60					
0.0	CLAYEY SILT, trace to some organics, trace sand, layered Very soft to soft Black to brown Moist		1	SS	1	▽	305							OC=8.1%
			2	SS	3		304							
			3	SS	2									
302.8							303							
2.6	CLAY Firm Grey Moist to wet		4	SS	WH		302							
	Silt seams encountered between 4.1 m and 7.2 m depth.						301							
			5	SS	WH		300							
			6	SS	WH		299							
298.2														
7.2	CLAYEY SILT, silt seams Firm Grey Wet		7a	SS	2	298								
297.4			7b			297								
8.0	SILT, trace to some clay Very loose Grey Wet													
296.6														
8.8	SAND and GRAVEL, trace to some silt Compact Grey Wet		8	SS	17	296								
295.0														
10.4	END OF BOREHOLE AUGER REFUSAL					295								
	Note: 1. Water level at a depth of 0.7 m below ground surface (Elev. 304.7 m) upon completion of drilling.													

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:


PROJECT 10-1191-0044			RECORD OF BOREHOLE No H7-8			1 OF 1 METRIC								
G.W.P. 5091-07-00			LOCATION N 5334697.7; E 410171.4			ORIGINATED BY MT								
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY MT								
DATUM GEODETIC			DATE August 24, 2012			CHECKED BY SEMC								
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						
306.2	GROUND SURFACE													
0.0	TOPSOIL		1a	SS	8									
0.2	CLAYEY SILT, layered Stiff Brown Moist		1b											
			2	SS	7									
304.4														
1.8	SILTY CLAY Soft to stiff Grey Wet													
			3	SS	2									
302.5														
3.7	SILT, trace to some clay, trace sand Loose Grey Wet		4	SS	4									
301.4			5a											
4.8	SAND and GRAVEL, some silt, trace clay Loose to compact Grey Wet		5b	SS	23									
299.8			6	SS	9/0.10									
6.4	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING) Note: 1. Water level at a depth of 1.0 m below ground surface (Elev. 305.2 m) upon completion of drilling.													

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT		10-1191-0044				RECORD OF BOREHOLE No H7-9				1 OF 1 METRIC							
G.W.P.		5091-07-00		LOCATION		N 5334717.1; E 410173.4				ORIGINATED BY							
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY							
DATUM		GEODETIC		DATE		August 24, 2012				CHECKED BY							
SEM		SEM		SEM		SEM				SEM							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
306.0	GROUND SURFACE																
0.0	TOPSOIL																
305.5	CLAYEY SILT, layers Soft Brown Moist		1	SS	4	▽											
0.5	SILTY CLAY, silt seams Soft Grey to brown Moist		2	SS	3												
304.6	SILT Very loose to compact Grey Moist		3	SS	1												
1.4			4	SS	10/0.03												
303.4	END OF BOREHOLE AUGER REFUSAL																
2.6	Note: 1. Water level at a depth of 0.5 m below ground surface (Elev. 305.5 m) upon completion of drilling.																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

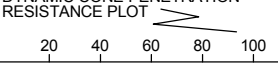



PROJECT 10-1191-0044				RECORD OF BOREHOLE No H7-10				1 OF 1 METRIC									
G.W.P. 5091-07-00				LOCATION N 5334734.9; E 410176.5				ORIGINATED BY MT									
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT									
DATUM GEODETIC				DATE August 24, 2012				CHECKED BY SEMC									
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									
305.5	GROUND SURFACE																
0.0	TOPSOIL		1a														
0.1	CLAYEY SILT, trace sand, trace organics		1b	SS	3												
0.7	Very soft Brown Moist CLAY		2	SS	3												
	Firm to stiff Grey Moist to wet																
	Silt seams encountered between 1.8 m and 3.7 m depth.																
			3	SS	WH												
301.8																	
3.7	SILT, trace to some clay		4	SS	10												
	Loose Grey Wet																
300.7																	
4.8	Silty SAND, some gravel		5	SS	8												
	Loose to compact Grey Wet																
			6	SS	11												
298.8																	
6.7	END OF BOREHOLE AUGER REFUSAL																
	Note: 1. Water level at a depth of 1.2 m below ground surface (Elev. 304.3 m) upon completion of drilling.																

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No H7-11				1 OF 1 METRIC						
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334730.5; E 410193.3</u>				ORIGINATED BY <u>MT</u>						
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>MT</u>						
DATUM <u>GEODETIC</u>		DATE <u>August 24, 2012</u>				CHECKED BY <u>SEMC</u>						
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L			20 40 60
306.6	GROUND SURFACE											
0.0 306.3 0.3	TOPSOIL		1	SS	4/0.15							
	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING)											
	Note: 1. Borehole dry upon completion of drilling.											

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>				RECORD OF BOREHOLE No H7-12				1 OF 1 METRIC									
G.W.P. <u>5091-07-00</u>				LOCATION <u>N 5334726.7; E 410209.2</u>				ORIGINATED BY <u>MT</u>									
DIST <u> </u> HWY <u>66</u>				BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>MT</u>									
DATUM <u>GEODETIC</u>				DATE <u>August 24, 2012</u>				CHECKED BY <u>SEMC</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									
307.8	GROUND SURFACE							20	40	60	80	100					
0.0	TOPSOIL		1	SS	15		307										
307.0	CLAYEY SILT Stiff Brown Moist																
0.8	SILT, some sand, trace to some clay, trace gravel		2	SS	16/0.13												5 18 66 11
306.5	Compact Grey to brown Moist																
1.3	END OF BOREHOLE AUGER REFUSAL																
Note: 1. Borehole dry upon completion of drilling.																	

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>			RECORD OF BOREHOLE No H7-13				1 OF 1 METRIC				
G.W.P. <u>5091-07-00</u>			LOCATION <u>N 5334739.4; E 410206.6</u>				ORIGINATED BY <u>MT</u>				
DIST <u> </u> HWY <u>66</u>			BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>MT</u>				
DATUM <u>GEODETIC</u>			DATE <u>August 23, 2012</u>				CHECKED BY <u>SEMC</u>				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
307.5	GROUND SURFACE										
0.0	TOPSOIL		1	SS	15		307				0 2 72 26
	CLAYEY SILT Stiff Brown Dry to moist										
306.5			2	SS	42						
1.0	Silty SAND to SAND and GRAVEL Dense to very dense Brown Moist		3	SS	30/0.08		306				
305.2											
2.3	END OF BOREHOLE AUGER REFUSAL Note: 1. Borehole dry upon completion of drilling.										

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044				RECORD OF BOREHOLE No H7-14				1 OF 1 METRIC									
G.W.P. 5091-07-00				LOCATION N 5334760.2; E 410210.7				ORIGINATED BY MT									
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT									
DATUM GEODETIC				DATE August 23, 2012				CHECKED BY SEMC									
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									
306.9	GROUND SURFACE							20	40	60	80	100					
0.0	TOPSOIL		1	SS	8												
306.2	CLAYEY SILT Firm Brown Dry to moist		2	SS	6												
0.7	CLAY, layered Soft to firm Brown Moist		3	SS	4												
304.8	CLAYEY SILT, silt seams Soft Brown Moist		4	SS	3/0.08												
304.5	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING)																
2.4	Note: 1. Borehole dry upon completion of drilling.																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044				RECORD OF BOREHOLE No H7-15				1 OF 1 METRIC									
G.W.P. 5091-07-00				LOCATION N 5334756.1; E 410231.5				ORIGINATED BY MT									
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT									
DATUM GEODETIC				DATE August 23, 2012				CHECKED BY SEMC									
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									
307.5	GROUND SURFACE																
0.0	TOPSOIL		1	SS	11												
306.7	CLAYEY SILT, trace organics Stiff Brown Dry to moist		2	SS	6												
0.8	SILTY CLAY, silt seams Firm Brown Moist		3	SS	7												
305.1																	
304.8	Sandy SILT, trace to some clay Loose Brown Moist		4	SS	7												
2.7	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING)																
Note: 1. Borehole dry upon completion of drilling.																	

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT		10-1191-0044		RECORD OF BOREHOLE No BC2-1		1 OF 1 METRIC	
G.W.P.		5091-07-00		LOCATION		N 5334378.5; E 409744.6	
DIST		HWY 66 - HIGH FILL H6 /		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring	
DATE		GEODETIC		DATE		September 12, 2012	
						CHECKED BY SEMC	
SOIL PROFILE				SAMPLES		DYNAMIC CONE PENETRATION RESISTANCE PLOT	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
							20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L WATER CONTENT (%)
309.4	GROUND SURFACE						
0.0	PEAT (Fibrous) Very soft Black Moist		1	SS	WH		309
308.4							
308.0	Silty SAND, some clay, trace organics Very loose Grey Wet		2	SS	2		308
1.4	SILTY CLAY, trace sand Firm Grey Wet		3	SS	2		307
306.2			4a				
305.9	SILT, some clay, trace sand Very loose Grey to brown Wet		4b	SS	3		306
3.5	METASEDIMENT (BEDROCK)						
	Bedrock cored from 3.5 m depth to 6.5 m depth. For coring details see Record of Drillhole BC2-1.		1	RC	REC 100%		305
			2	RC	REC 91%		304
302.9							303
6.5	END OF BOREHOLE						
Note: 1. Water level at a depth of 1.6 m below ground surface (Elev. 307.8 m) upon completion of drilling.							

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

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC2-1

SHEET 1 OF 1

LOCATION: N 5334378.5 ; E 409744.6

DRILLING DATE: September 12, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE		305.9									
4	NW	METASEDIMENT Strong Fine grained Moderately weathered to fresh Grey to green		3.5	1	GREY	100%						72 MPa
5	September 12, 2012 NQ Coring												
6		Mechanical breaks below 5.7 m depth.			2	GREY	100%						
		END OF DRILLHOLE		302.9									
7				6.5									
8													
9													
10													
11													
12													
13													

DEPTH SCALE

1 : 50



LOGGED: MT

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 02/12/13 DATA INPUT:

PROJECT		10-1191-0044		RECORD OF BOREHOLE No BC2-2				1 OF 1		METRIC							
G.W.P.		5091-07-00		LOCATION		N 5334390.7; E 409727.1		ORIGINATED BY		MT							
DIST		HWY 66 - HIGH FILL H6 /		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring		COMPILED BY		MT							
DATUM		GEODETIC		DATE		September 17, 2012		CHECKED BY		SEMC							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	GR	SA	SI	CL
309.7	0.0	GROUND SURFACE		1	AS	-											
309.0	0.7	PEAT (Fibrous) Black Wet		2	SS	4		309						2	40	42	16
		SAND and SILT, some clay, trace gravel Very loose to loose Grey to brown Wet		3	SS	2		308									
				4	SS	WH		307									
306.7	3.0	CLAYEY SILT, trace sand, silt seams Very soft Grey Wet		5	SS	WH		306						0	18	54	28
				6	SS	WH											
305.1	4.6	CLAY Firm Grey Wet						305									
304.1	5.6	Gravelly SAND, some silt Loose to compact Grey to brown Wet		7a	SS	9		304									
		Boulder encountered between 7.0 m and 7.4 m depth.		7b													
				8	SS	11/0.05		303									
302.3	7.4	METASEDIMENT (BEDROCK)						302									
		Bedrock cored from 7.4 m depth to 10.8 m depth.		1	RC	REC 100%		301									RQD = 51%
		For coring details see Record of Drillhole BC2-2.		2	RC	REC 100%		300									RQD = 100%
				3	RC	REC 100%											RQD = 93%
298.9	10.8	END OF BOREHOLE						299									
		Note: 1. Water level at ground surface (Elev. 309.7 m) upon completion of drilling. 2. Water level in piezometer at 0.3 m above ground surface (Elev. 310.0 m) on November 15, 2012 and at 0.4 m above ground surface (Elev. 310.1 m) on May 15, 2013.															

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC2-2

SHEET 1 OF 1

LOCATION: N 5334390.7 ; E 409727.1

DRILLING DATE: September 17, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR FLUSH % 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.	NOTES WATER LEVELS INSTRUMENTATION
DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR FLUSH % 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.	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE		302.3								
8	NW September 11, 2012 NQ Coring	METASEDIMENT Highly foliated Fine grained Slightly weathered Grey		7.4	1	WHITE - GREY 100%						
9					2	WHITE - GREY 100%						
10					3	GREY 100%						
11		END OF DRILLHOLE		298.9 10.8								
12												
13												
14												
15												
16												
17												

DEPTH SCALE






1 : 50



LOGGED: MT

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 02/12/13 DATA INPUT:

PROJECT		10-1191-0044				RECORD OF BOREHOLE No BC2-3				1 OF 1 METRIC								
G.W.P.		5091-07-00		LOCATION		N 5334366.7; E 409761.6				ORIGINATED BY		MT						
DIST		HWY 66 - HIGH FILL H6 /		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring				COMPILED BY		MT						
DATUM		GEODETIC		DATE		September 11, 2012				CHECKED BY		SEMC						
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 (%)
309.3	GROUND SURFACE							20	40	60	80	100						
0.0	PEAT (Fibrous) Very soft Black Wet		1	SS	1		309											
308.6																		
0.7	Silty SAND Compact to dense Grey to brown Wet		2	SS	26		308											
			3	SS	16/0.15													
307.2	METASEDIMENT (BEDROCK)																	
2.1	Bedrock cored from 2.1 m depth to 5.0 m depth. For coring details see Record of Drillhole BC2-3.		1	RC	REC 100%		307										RQD = 89%	
			2	RC	REC 100%		306											
							305										RQD = 97%	
304.3	END OF BOREHOLE																	
5.0	Note: 1. Water level at a depth of 0.4 m below ground surface (Elev. 308.9 m) upon completion of drilling.																	

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC2-3

SHEET 1 OF 1

LOCATION: N 5334366.7 ; E 409761.6

DRILLING DATE: September 11, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling Inc.

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.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
							RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA										HYDRAULIC CONDUCTIVITY		Diametral Point Load Index (MPa)	RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
							TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	k, cm/s	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
							888888	888888													888888	888888			888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888				888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888	888888

DEPTH SCALE

1 : 50



LOGGED: MT

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 02/12/13 DATA INPUT:

PROJECT 10-1191-0044		RECORD OF BOREHOLE No BC3-1		1 OF 2 METRIC	
G.W.P. 5091-07-00		LOCATION N 5334503.0; E 409859.6		ORIGINATED BY MT	
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY MT	
DATUM GEODETIC		DATE August 15 and 20, 2012		CHECKED BY SEMC	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			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		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED					
308.7 0.0	GROUND SURFACE		1	AS	-									
	PEAT (Fibrous) Very soft to soft Black Wet		2	SS	WH									
			3	SS	WH									
			4	SS	3									
305.6 3.1	CLAYEY SILT Soft to firm Grey Wet		5	SS	WH									
	Some sand above 3.7 m depth.		6	SS	3									
	Trace organics above 4.5 m depth.		7	SS	WH									
	Silt seams encountered between 4.5 m and 8.7 m depth.		8	SS	WH									
301.9 6.8	CLAY Soft to firm Grey Wet		9	SS	WH									
			10	SS	WH									
			11	SS	WH									
			12	SS	WH									
			13	SS	WH									
293.9 14.8	Silt seams encountered between 13.3 m and 14.8 m depth.													

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

Continued Next Page

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

PROJECT 10-1191-0044			RECORD OF BOREHOLE No BC3-1			2 OF 2 METRIC																		
G.W.P. 5091-07-00			LOCATION N 5334503.0; E 409859.6			ORIGINATED BY MT																		
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY MT																		
DATUM GEODETIC			DATE August 15 and 20, 2012			CHECKED BY SEMC																		
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 W _p — W — W _L			20 40 60 W _p — W — W _L			20 40 60 W _p — W — W _L			20 40 60 W _p — W — W _L		
290.9	CLAYEY SILT Soft to firm Grey Wet Silt seams encountered between 14.8 m and 17.8 depth.		14	SS	WR		293	+					○			NP			0 0 75 25					
17.8	SILT, trace to some clay Very loose to loose Grey Wet		15	SS	WH		292	3 + 2 +																
285.2	Sandy SILT, trace to some clay Very loose Grey Wet		16	SS	5		291																	
23.5			17	SS	2		290						○			NP								
282.2							289																	
26.5	END OF BOREHOLE AUGER REFUSAL Note: 1. Water level at a depth of 1.5 m below ground surface (Elev. 307.2 m) upon completion of drilling.						288																	
							287																	
							286																	
							285																	
							284						○											
							283																	

[illegible]

Continued Next Page

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

PROJECT <u>10-1191-0044</u>			RECORD OF BOREHOLE No BC3-2			2 OF 2 METRIC								
G.W.P. <u>5091-07-00</u>			LOCATION <u>N 5334515.4; E 409849.5</u>			ORIGINATED BY <u>MT</u>								
DIST <u> </u> HWY <u>66</u>			BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>			COMPILED BY <u>MT</u>								
DATUM <u>GEODETIC</u>			DATE <u>August 22, 2012</u>			CHECKED BY <u>SEMC</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						
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100						
	CLAYEY SILT Firm Grey Wet		14	SS	WR			20 40 60 80 100						
290.1							293							
							292							
							291							
							290							
18.7	SILT, trace to some clay Very loose to loose Grey Wet		15	SS	2		289							
							288							
							287							
285.8			16	SS	4		286							
23.0	Silty SAND, trace to some gravel, trace clay Very dense Grey Wet						285							
284.3			17	SS	83/0.23									
24.5	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING) Note: 1. Water level at a depth of 0.6 m below ground surface (Elev. 308.2 m) upon completion of drilling.													

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:





PROJECT 10-1191-0044		RECORD OF BOREHOLE No BC3-3		1 OF 2 METRIC	
G.W.P. 5091-07-00		LOCATION N 5334490.5; E 409869.7		ORIGINATED BY MT	
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY MT	
DATUM GEODETIC		DATE August 21, 2012		CHECKED BY SEMC	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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		
								20 40 60 80 100	20 40 60 80 100					
308.6	GROUND SURFACE													
0.0	PEAT (Fibrous) Very soft Black Wet		1	AS	-									
			2	SS	WH									
			3	SS	WH									
			4	SS	WH									
			5	SS	WH									
304.9														
3.7	CLAYEY SILT Soft to firm Grey Wet Silt seams encountered between 4.5 m and 7.2 m depth.		6	SS	3									
			7	SS	2									
			8	SS	WH									
301.4														
7.2	CLAY Soft to firm Grey Wet		9	TO	PH									
			10	SS	WH									
			11	SS	WH									
			12	SS	WH									
			13	SS	WH									
293.8														
14.8														

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

Continued Next Page

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

PROJECT 10-1191-0044		RECORD OF BOREHOLE No BC3-3				2 OF 2 METRIC									
G.W.P. 5091-07-00		LOCATION N 5334490.5; E 409869.7				ORIGINATED BY MT									
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT									
DATUM GEODETIC		DATE August 21, 2012				CHECKED BY SEMC									
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 (%) W _p — W — W _L		γ	GR SA SI CL		
--- CONTINUED FROM PREVIOUS PAGE ---															
289.9	CLAYEY SILT Firm Grey Wet		14	SS	WH		293							NP	0 0 95 5
							292								
							291								
							290								
							289								
18.7	SILT, trace clay Very loose to loose Grey Wet		15a	SS	WH		290								
15b															
						289									
						288									
						287									
286.3	Silty SAND, trace gravel, trace clay Grey Wet		16	SS	4		287								
22.3															
285.7	END OF BOREHOLE		17	AS	-		286								
22.9															
Note: 1. Water level at a depth of 1.2 m below ground surface (Elev. 307.4 m) upon completion of drilling.															

1 OF 2 **METRIC**


ORIGINATED BY GM

COMPILED BY MT

CHECKED BY SEMC

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

PROJECT 10-1191-0044				RECORD OF BOREHOLE No BC4-1				2 OF 2 METRIC								
G.W.P. 5091-07-00				LOCATION N 5334678.1; E 410115.3				ORIGINATED BY GM								
DIST _____ HWY 66				BOREHOLE TYPE Portable Equipment, NW Casing, Wash Boring				COMPILED BY MT								
DATUM GEODETIC				DATE November 16, 2012				CHECKED BY SEMC								
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								
	--- CONTINUED FROM PREVIOUS PAGE ---						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED				WATER CONTENT (%)					
289.5	Gravelly SAND, some silt, trace clay Dense Grey Wet		12	SS	40		290									26 59 13 2
15.7	END OF BOREHOLE START OF DCPT						289									
							288									
							287									
286.3	END OF DCPT REFUSAL TO FURTHER PENETRATION 50 BLOWS / 0.08 m (HAMMER BOUNCING)															
18.9																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:


SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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



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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044			RECORD OF BOREHOLE No BC4-3			1 OF 2 METRIC					
G.W.P. 5091-07-00			LOCATION N 5334660.4; E 410121.2			ORIGINATED BY GM					
DIST _____ HWY 66			BOREHOLE TYPE Portable Equipment, NW Casing, Wash Boring			COMPILED BY MT					
DATUM GEODETIC			DATE November 17 and 18, 2012			CHECKED BY SEMC					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
305.2	WATER SURFACE										
0.0	WATER										
304.9	PEAT (Fibrous)		1	SS	1						
0.6	CLAYEY SILT, layered Very soft to firm Grey to brown Wet Trace organics above 0.9 m depth.		2	SS	3						
			3	SS	1						
302.3	SILTY CLAY Soft to firm Grey Wet		4	SS	WH						
2.9			5	SS	WH						
			6	SS	WH						
			7	SS	WH						
			8	SS	1						
296.2	CLAYEY SILT Firm Grey Wet		9	SS	4						
9.0			10	SS	7						
294.7	SILT, trace to some clay Loose to dense Grey Wet		11	SS	35						
10.5											
290.6	END OF BOREHOLE START OF DCPT										
14.6											

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

Continued Next Page

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



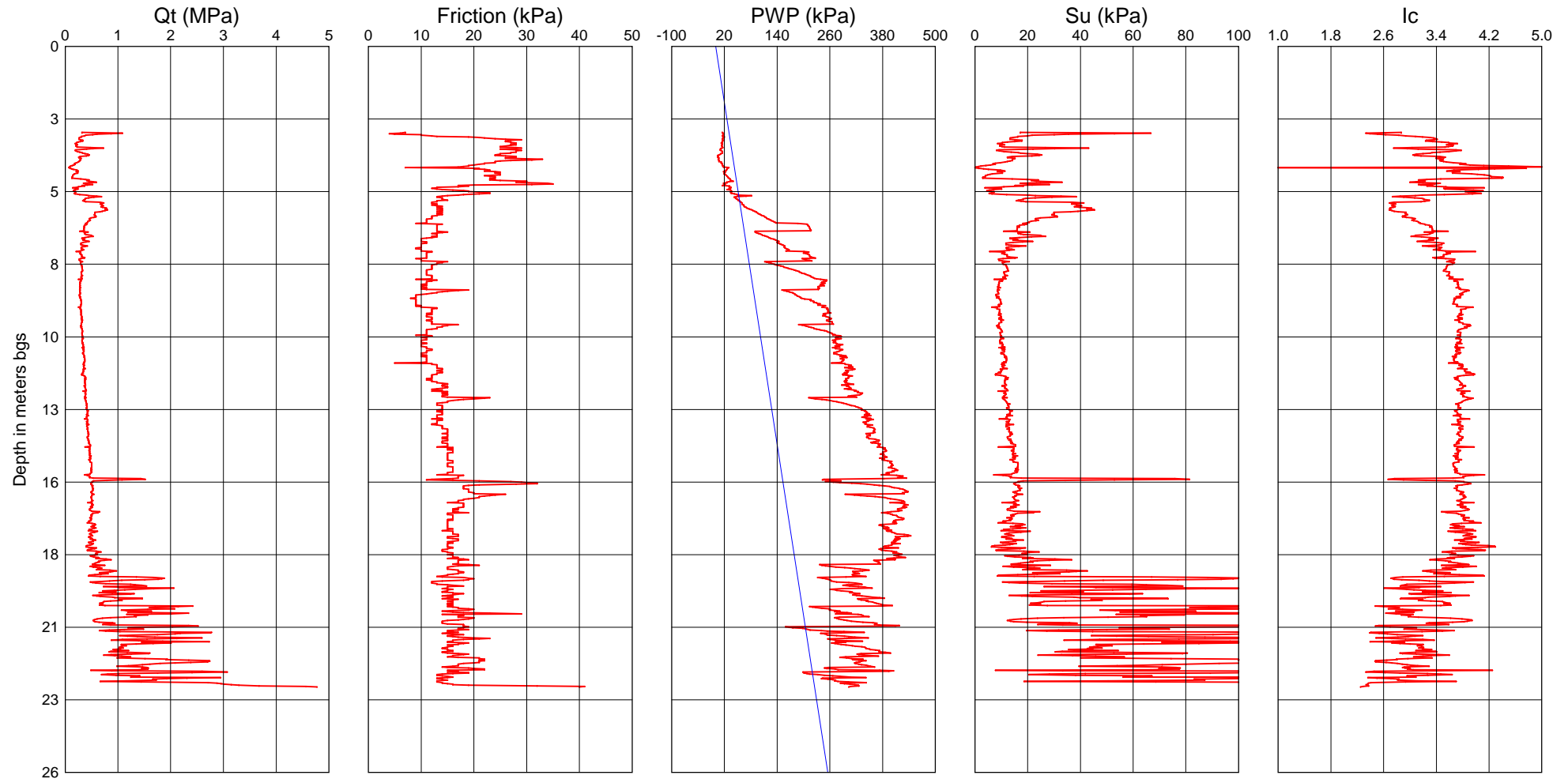
+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Cone Penetration Test - H6-CPT1

Test Date : April 30, 2013
Location : Highway 66 - STA 14+085 o/s 4.0m Left of CL

Operator : Golder Associates Ltd.

Ground Surf. Elev. : 309.20
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

$S_u = (Q_t - \sigma_{av}) / N_k$
 $N_k = 15.5$
 $\gamma = 16.5 \text{ kN/m}^3$

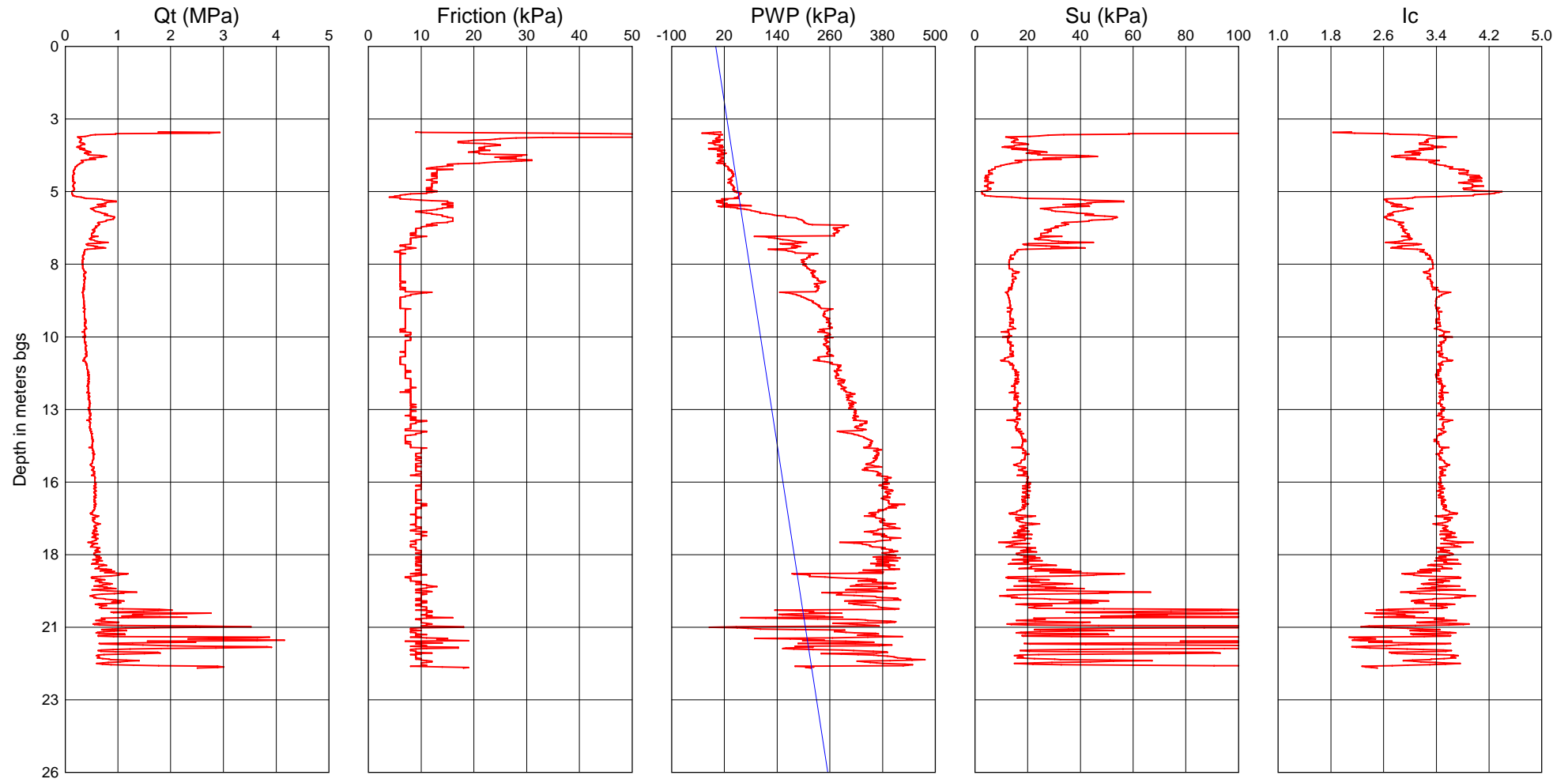
After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 < Ic < 2.05 - Clean to silty sand
2.05 < Ic < 2.60 - Silty sand to sandy silt
2.60 < Ic < 2.95 - Clayey silt to silty clay
2.95 < Ic < 3.60 - Clays

Cone Penetration Test - H6-CPT2

Test Date : May 23, 2013
Location : Highway 66 - STA 14+125 o/s 1 m Left of CL

Operator : Golder Associates Ltd.

Ground Surf. Elev. : 309.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$
 $N_k = 15.5$
 $\gamma = 16.5 \text{ kN/m}^3$

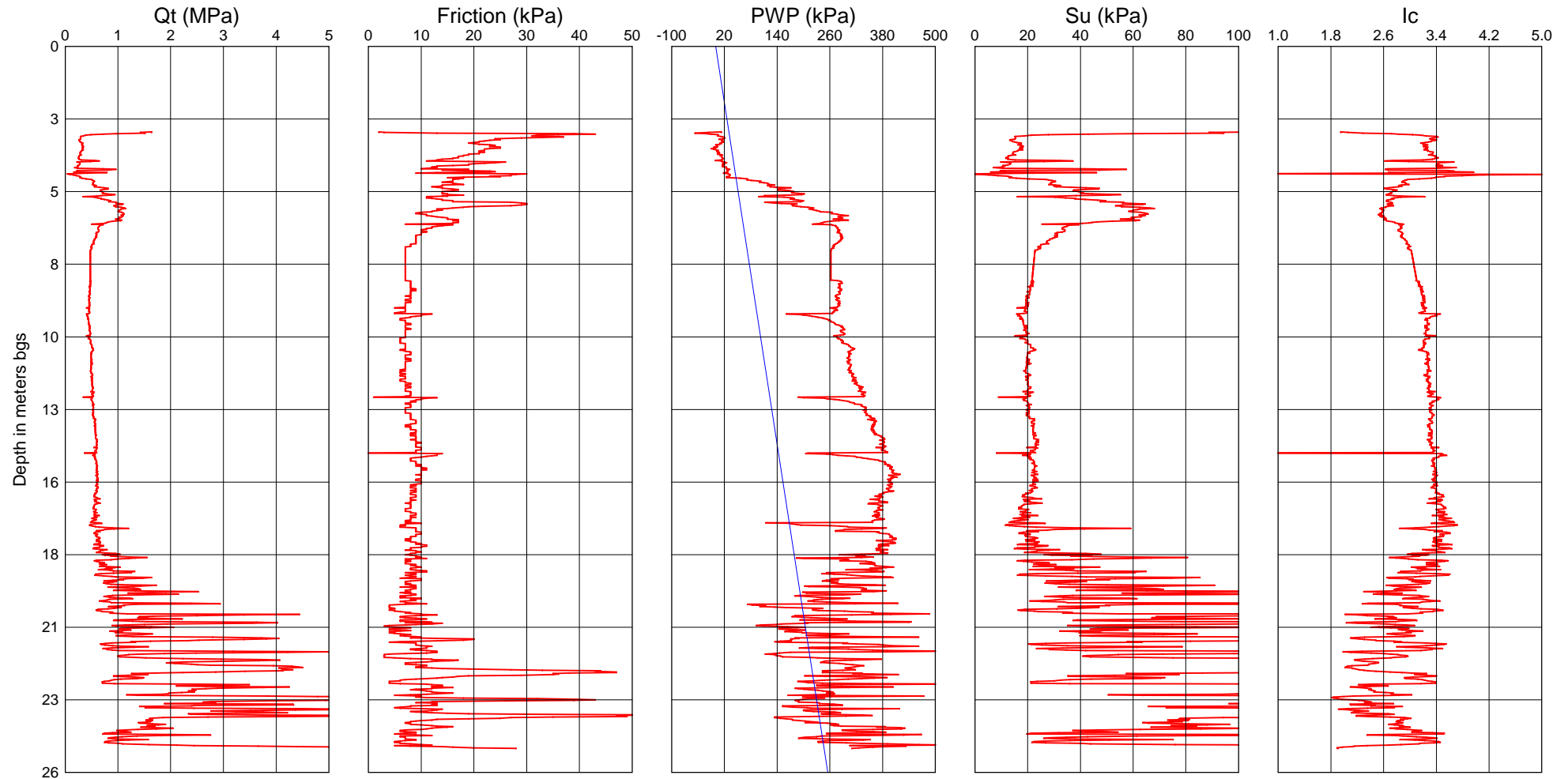
After Robertson and (Fear) Wride (1998)
 $I_c < 1.31$ - Gravelly sands
 $1.31 < I_c < 2.05$ - Clean to silty sand
 $2.05 < I_c < 2.60$ - Silty sand to sandy silt
 $2.60 < I_c < 2.95$ - Clayey silt to silty clay
 $2.95 < I_c < 3.60$ - Clays

Cone Penetration Test - H6-CPT3

Test Date : May 23, 2013
Location : Highway 66 - STA 14+202 o/s 2 m Left of CL

Operator : Golder Associates Ltd.

Ground Surf. Elev. : 308.70
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

$S_u = (Q_t - \sigma_{av}) / N_k$
 $N_k = 15.5$
 $\gamma = 16.5 \text{ kN/m}^3$

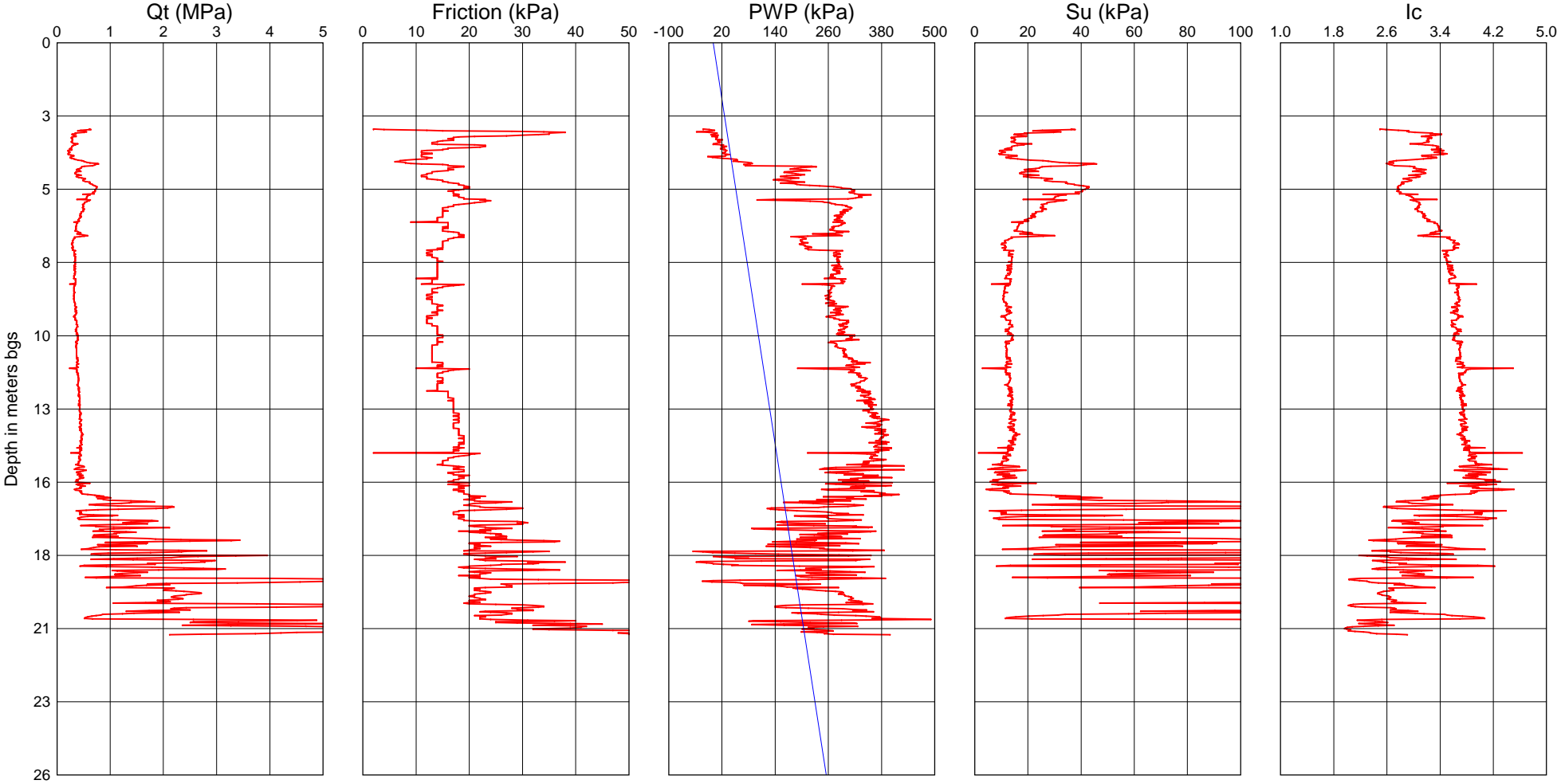
After Robertson and (Fear) Wride (1998)
 $I_c < 1.31$ - Gravely sands
 $1.31 < I_c < 2.05$ - Clean to silty sand
 $2.05 < I_c < 2.60$ - Silty sand to sandy silt
 $2.60 < I_c < 2.95$ - Clayey silt to silty clay
 $2.95 < I_c < 3.60$ - Clays

Cone Penetration Test - H6-CPT4

Test Date : May 24, 2013
Location : Highway 66 - STA 14+251.5 o/s 2 m Left of CL

Operator : Golder Associates Ltd.

Ground Surf. Elev. : 307.80
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$
 $N_k = 15.5$
 $\gamma = 16.5 \text{ kN/m}^3$

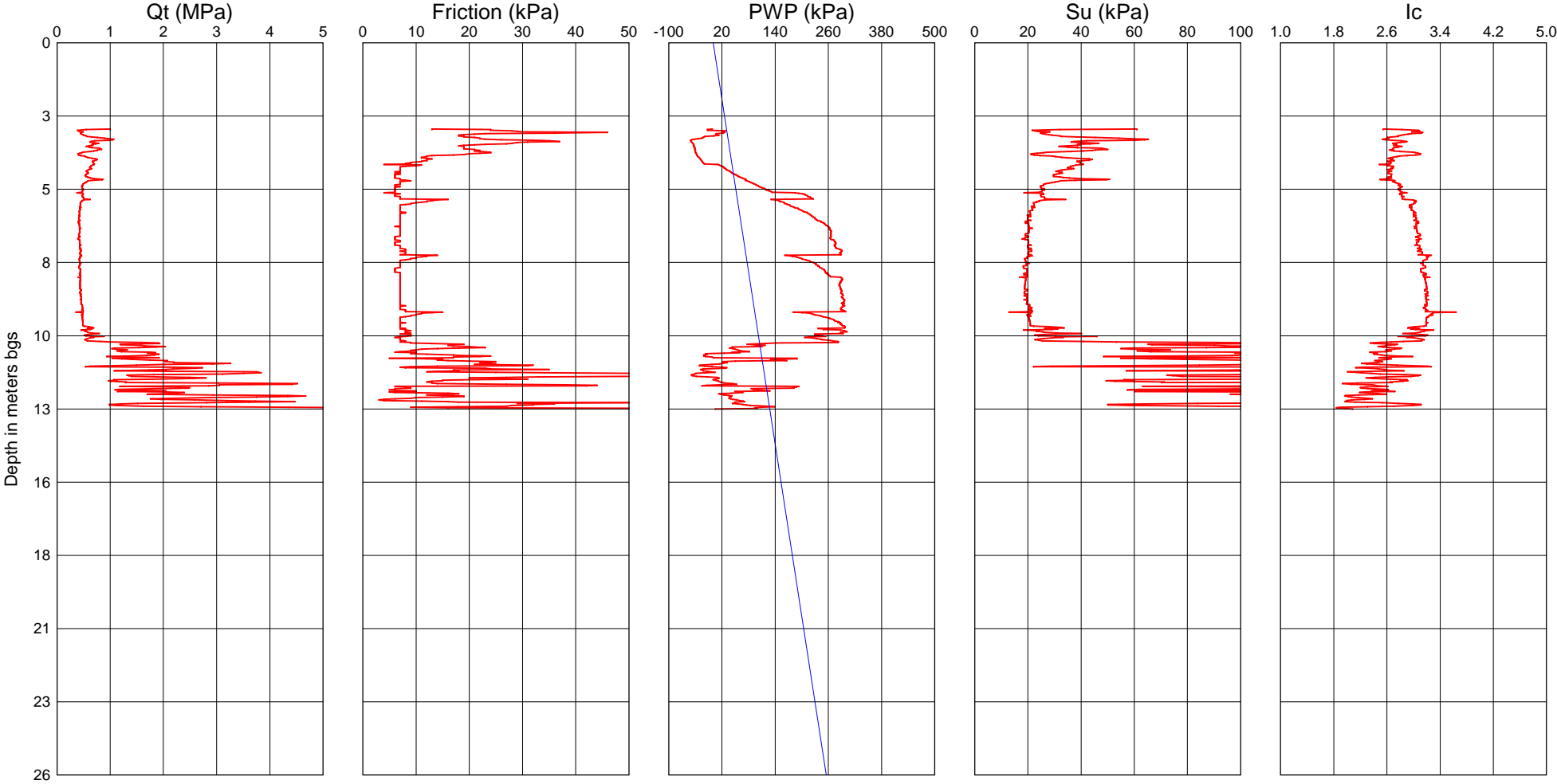
After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 < Ic < 2.05 - Clean to silty sand
2.05 < Ic < 2.60 - Silty sand to sandy silt
2.60 < Ic < 2.95 - Clayey silt to silty clay
2.95 < Ic < 3.60 - Clays

Cone Penetration Test - H6-CPT5

Test Date : May 24, 2013
Location : Highway 66 - STA 14+651 o/s 2 m Left of CL

Operator : Golder Associates Ltd.

Ground Surf. Elev. : 306.70
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$
 $N_k = 15.5$
 $\gamma = 16.5 \text{ kN/m}^3$

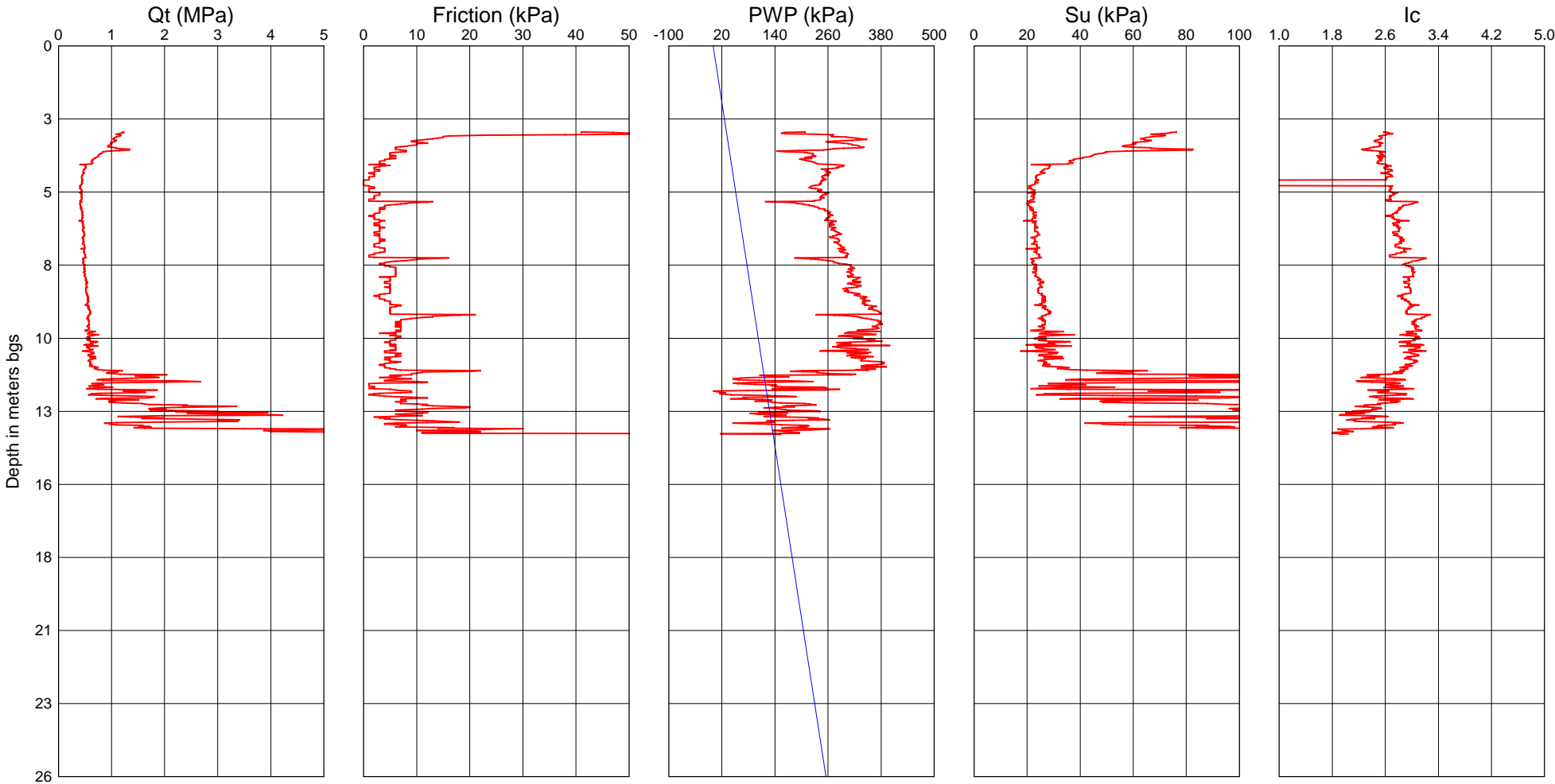
After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 < Ic < 2.05 - Clean to silty sand
2.05 < Ic < 2.60 - Silty sand to sandy silt
2.60 < Ic < 2.95 - Clayey silt to silty clay
2.95 < Ic < 3.60 - Clays

Cone Penetration Test - H6-CPT6

Test Date : May 25, 2013
Location : Highway 66 - STA 14+452 o/s 3 m Left of CL

Operator : Golder Associates Ltd.

Ground Surf. Elev. : 305.20
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

$Su = (Qt - \sigma_v) / Nk$
 $Nk = 15.5$
 $\gamma = 16.5 \text{ kN/m}^3$

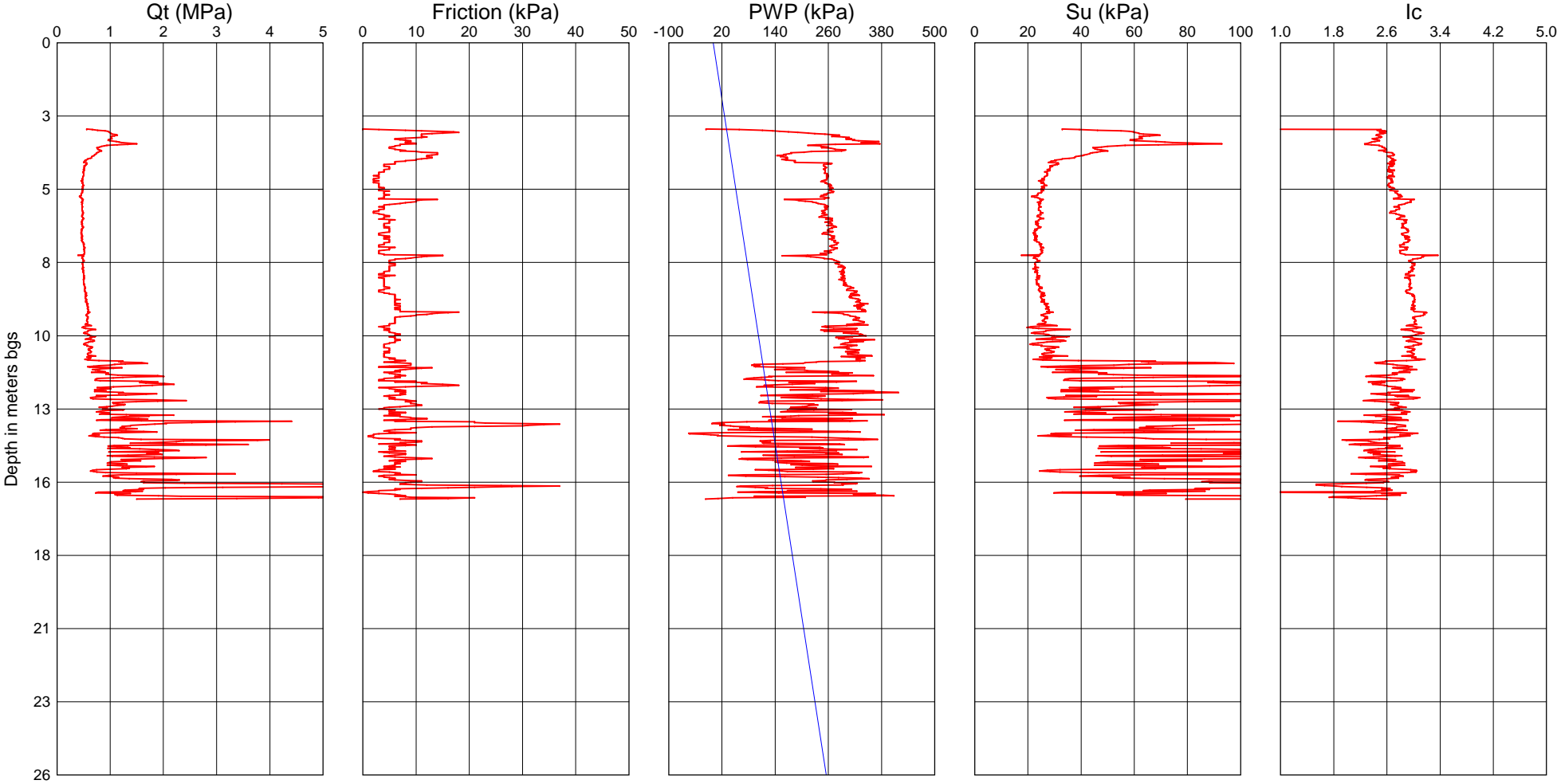
After Robertson and (Fear) Wride (1998)
 $Ic < 1.31$ - Gravelly sands
 $1.31 < Ic < 2.05$ - Clean to silty sand
 $2.05 < Ic < 2.60$ - Silty sand to sandy silt
 $2.60 < Ic < 2.95$ - Clayey silt to silty clay
 $2.95 < Ic < 3.60$ - Clays

Cone Penetration Test - H6-CPT7

Test Date : May 25, 2013
Location : Highway 66 - STA 14+480.5 o/s 1 m Left of CL

Operator : Golder Associates Ltd.

Ground Surf. Elev. : 305.10
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$
 $N_k = 15.5$
 $\gamma = 16.5 \text{ kN/m}^3$

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

PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST No H6-D1				1 OF 1 METRIC								
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334379.1; E 409769.2</u>				ORIGINATED BY <u>MT</u>								
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>TR</u>								
DATUM <u>GEODETIC</u>		DATE <u>July 26, 2012</u>				CHECKED BY <u>SEMC</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	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa						WATER CONTENT (%)
309.6 0.0	GROUND SURFACE						<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>							
309														
308														
307														
306														
305														
304														
303.2 6.4	END OF DCPT REFUSAL TO FURTHER PENETRATION 11 BLOWS / 0.25 m (HAMMER BOUNCING)													

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:


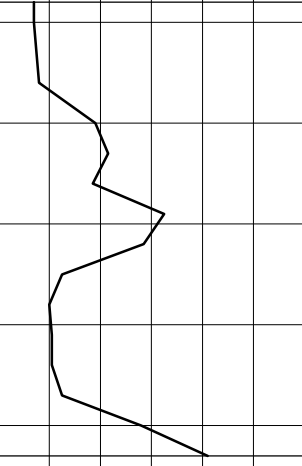


PROJECT		RECORD OF PENETRATION TEST				1 OF 2		METRIC							
G.W.P.		LOCATION				ORIGINATED BY		MT							
DIST		BOREHOLE TYPE				COMPILED BY		TR							
DATUM		DATE				CHECKED BY		SEMC							
SOIL PROFILE		SAMPLES				DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT		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	
309.2 0.0	GROUND SURFACE							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED		Wp W WL		kN/m³			
							309								
							308								
							307								
							306								
							305								
							304								
							303								
							302								
							301								
							300								
							299								
							298								
							297								
							296								
							295								

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

Continued Next Page

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

PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST No H6-D2				2 OF 2 METRIC				
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334422.0; E 409756.7</u>				ORIGINATED BY <u>MT</u>				
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>TR</u>				
DATUM <u>GEODETIC</u>		DATE <u>July 28, 2012</u>				CHECKED BY <u>SEMC</u>				
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
	--- CONTINUED FROM PREVIOUS PAGE ---									
289.7						294				
						293				
						292				
						291				
						290				
19.5	END OF DCPT REFUSAL TO FURTHER PENETRATION 82 BLOWS / 0.30 m (HAMMER BOUNCING)									

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

PROJECT <u>10-1191-0044</u>	RECORD OF PENETRATION TEST No H6-D3	2 OF 2	METRIC
G.W.P. <u>5091-07-00</u>	LOCATION <u>N 5334420.0; E 409798.0</u>	ORIGINATED BY <u>MT</u>	
DIST <u> </u> HWY <u>66</u>	BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>	COMPILED BY <u>TR</u>	
DATUM <u>GEODETIC</u>	DATE <u>August 1, 2012</u>	CHECKED BY <u>SEMC</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT			UNIT WEIGHT γ kN/m³	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						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	WATER CONTENT (%)										
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100				20 40 60							
								<div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div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SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST No H6-D4		2 OF 2 METRIC	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334456.2; E 409786.2</u>		ORIGINATED BY <u>MT</u>	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>		COMPILED BY <u>TR</u>	
DATUM <u>GEODETIC</u>		DATE <u>August 10, 2012</u>		CHECKED BY <u>SEMC</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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		WATER CONTENT (%)			
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED								
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100				20 40 60		GR SA SI CL			
								<div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div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SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

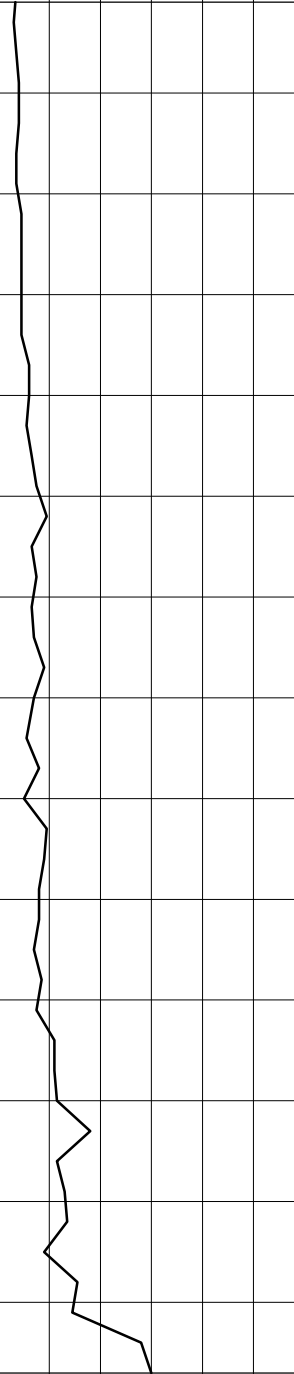


PROJECT		RECORD OF PENETRATION TEST				No H6-D5		1 OF 2		METRIC							
G.W.P.		LOCATION				ORIGINATED BY		MT									
DIST		BOREHOLE TYPE				COMPILED BY		TR									
DATUM		DATE				CHECKED BY		SEMC									
SOIL PROFILE		SAMPLES				DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE 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		
308.9	0.0	GROUND SURFACE							20 40 60 80 100	20 40 60							
								308									
								307									
								306									
								305									
								304									
								303									
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								298									
								297									
								296									
								295									
								294									

Continued Next Page

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

SUD-MTO 001 10-1191-004SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST No H6-D5				2 OF 2 METRIC							
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334448.8; E 409824.2</u>				ORIGINATED BY <u>MT</u>							
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>TR</u>							
DATUM <u>GEODETIC</u>		DATE <u>August 10, 2012</u>				CHECKED BY <u>SEMC</u>							
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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		
--- 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 ● QUICK TRIAXIAL × REMOULDED </div>						
293													
292													
291													
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288													
287													
286													
285													
284													
283													
282													
281													
280.3 28.6	END OF DCPT REFUSAL TO FURTHER PENETRATION 60 BLOWS / 0.25 m (HAMMER BOUNCING)												

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST No H6-D6				2 OF 2 METRIC	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334497.1; E 409831.0</u>				ORIGINATED BY <u>MT</u>	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>TR</u>	
DATUM <u>GEODETIC</u>		DATE <u>September 26, 2012</u>				CHECKED BY <u>SEMC</u>	

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60	W _p W W _L			
--- CONTINUED FROM PREVIOUS PAGE ---							<div style="display: flex; justify-content: space-between;"> <div> SHEAR STRENGTH kPa ○ UNCONFINED ● QUICK TRIAXIAL </div> <div> + FIELD VANE × REMOULDED </div> </div>						
293													
292													
291													
290													
289													
288													
287													
286													
285													
284													
283													
282													
281.7 27.1	END OF DCPT REFUSAL TO FURTHER PENETRATION (HAMMER BOUNCING)												

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



PROJECT		RECORD OF PENETRATION TEST				No H6-D7		1 OF 2		METRIC										
G.W.P.		LOCATION				ORIGINATED BY		MT												
DIST		BOREHOLE TYPE				COMPILED BY		TR												
DATUM		DATE				CHECKED BY		SEMC												
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		W _p		W		W _L		γ		GR SA SI CL	
308.3	0.0	GROUND SURFACE							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED		20 40 60		20 40 60		20 40 60		kN/m ³			
								308												
								307												
								306												
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								294												

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

PROJECT <u>10-1191-0044</u>	RECORD OF PENETRATION TEST No H6-D7	2 OF 2	METRIC
G.W.P. <u>5091-07-00</u>	LOCATION <u>N 5334507.1; E 409887.7</u>	ORIGINATED BY <u>MT</u>	
DIST <u> </u> HWY <u>66</u>	BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>	COMPILED BY <u>TR</u>	
DATUM <u>GEODETIC</u>	DATE <u>September 26, 2012</u>	CHECKED BY <u>SEMC</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			W _p	W	W _L					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	WATER CONTENT (%)									
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100				20 40 60						
							293											
							292											
							291											
							290											
							289											
							288											
							287											
							286											
285.4 22.9	END OF DCPT REFUSAL TO FURTHER PENETRATION 18 BLOWS / 0.08 m (HAMMER BOUNCING)																	

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST No H6-D8		1 OF 2 METRIC	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334559.5; E 409911.8</u>		ORIGINATED BY <u>MT</u>	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>		COMPILED BY <u>TR</u>	
DATUM <u>GEODETIC</u>		DATE <u>October 18, 2012</u>		CHECKED BY <u>SEMC</u>	

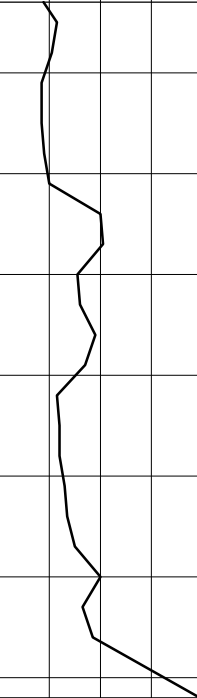
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT W P W L			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED					
307.7 0.0	GROUND SURFACE							20 40 60 80 100		20 40 60				

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

Continued Next Page

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

PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST No H6-D8				2 OF 2 METRIC	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334559.5; E 409911.8</u>				ORIGINATED BY <u>MT</u>	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>TR</u>	
DATUM <u>GEODETIC</u>		DATE <u>October 18, 2012</u>				CHECKED BY <u>SEMC</u>	

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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		
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between; border-bottom: 1px solid black; margin-bottom: 5px;"> 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between; border-bottom: 1px solid black; margin-bottom: 5px;"> 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between; border-bottom: 1px solid black; margin-bottom: 5px;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between; border-bottom: 1px solid black; margin-bottom: 5px;"> ● QUICK TRIAXIAL × REMOULDED </div>	<div style="display: flex; justify-content: space-between; border-bottom: 1px solid black; margin-bottom: 5px;"> 20 40 60 </div>					
													
285.8 21.9	END OF DCPT REFUSAL TO FURTHER PENETRATION 79 BLOWS / 0.28 m (HAMMER BOUNCING)												

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



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



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



SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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



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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST No H7-D1				2 OF 2 METRIC														
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334639.7; E 410086.8</u>				ORIGINATED BY <u>MT</u>														
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>TR</u>														
DATUM <u>GEODETIC</u>		DATE <u>August 26, 2012</u>				CHECKED BY <u>SEMC</u>														
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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>													
287.6							290													
							289													
							288													
17.9	END OF DCPT REFUSAL TO FURTHER PENETRATION 22 BLOWS / 0.22 m (HAMMER BOUNCING)																			

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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



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



PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST		No H7-D3	1 OF 2	METRIC
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334697.8; E 410114.1</u>		ORIGINATED BY <u>GM</u>		
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>		COMPILED BY <u>MT</u>		
DATUM <u>GEODETIC</u>		DATE <u>November 2, 2012</u>		CHECKED BY <u>SEMC</u>		

SOIL PROFILE					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES	GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT
304.4 0.0	GROUND SURFACE				<div><div>20406080100</div><div>○ UNCONFINED + FIELD VANE</div><div>● QUICK TRIAXIAL × REMOULDED</div></div> <div><div>W_p W W_L</div><div>PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT</div><div>WATER CONTENT (%)</div><div>204060</div></div> <div><div>UNIT WEIGHT</div><div>γ</div><div>kN/m³</div></div> <div><div>REMARKS & GRAIN SIZE DISTRIBUTION (%)</div><div>GR SA SI CL</div></div>
290.2 14.2					

Continued Next Page

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



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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



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



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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT		10-1191-0044				RECORD OF PENETRATION TEST No H7-D6				1 OF 1 METRIC									
G.W.P.		5091-07-00		LOCATION		N 5334711.6; E 410192.1		ORIGINATED BY		MT									
DIST		HWY 66		BOREHOLE TYPE		Dynamic Cone Penetration Test		COMPILED BY		TR									
DATUM		GEODETIC		DATE		August 24, 2012		CHECKED BY		SEMC									
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				W _p W W _L		WATER CONTENT (%)		γ		GR SA SI CL	
307.0 0.0	GROUND SURFACE						306	20	40	60	80	100	20	40	60	kN/m ³			
304.9 2.1	END OF DCPT REFUSAL TO FURTHER PENETRATION 24 BLOWS / 0.28 m (HAMMER BOUNCING)						305												

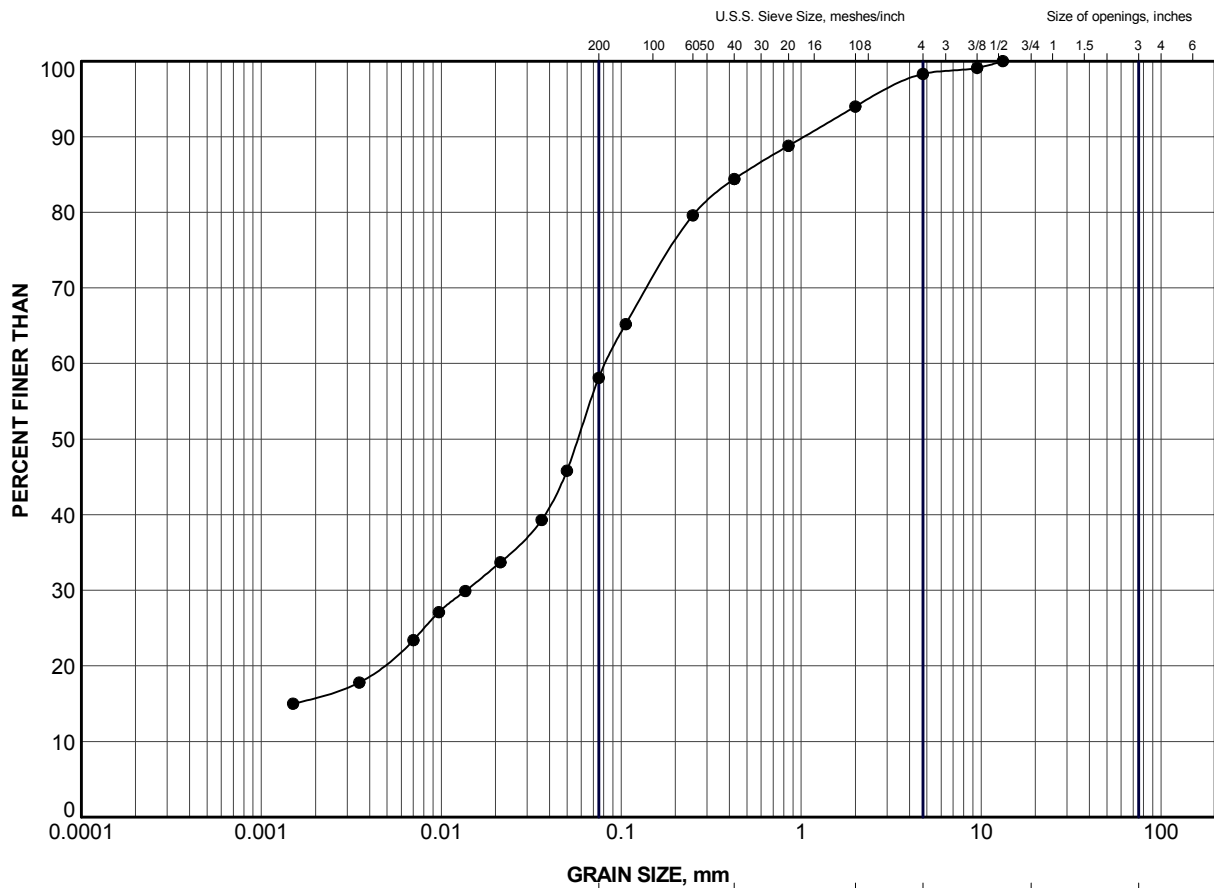
SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT		10-1191-0044				RECORD OF PENETRATION TEST No H7-D7				1 OF 1		METRIC					
G.W.P.		5091-07-00		LOCATION		N 5334749.1; E 410194.1				ORIGINATED BY		MT					
DIST		HWY 66		BOREHOLE TYPE		Dynamic Cone Penetration Test				COMPILED BY		TR					
DATUM		GEODETIC		DATE		August 24, 2012				CHECKED BY		SEMC					
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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
306.4 0.0	GROUND SURFACE							20	40	60	80	100					
							306										
							305										
304.3 2.1	END OF DCPT REFUSAL TO FURTHER PENETRATION 11 BLOWS / 0.25 m (HAMMER BOUNCING)																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:




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

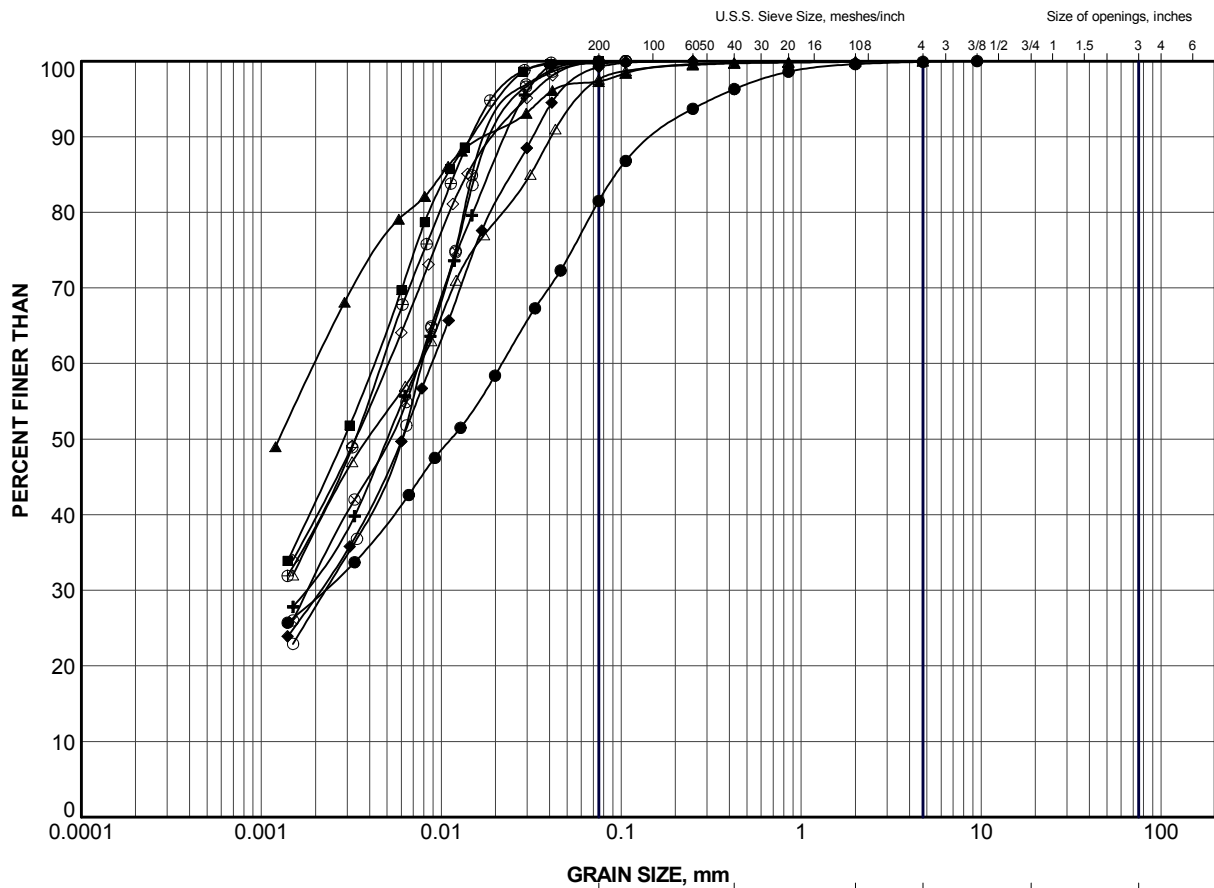


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC2-2	2	308.6

PROJECT					
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650					
TITLE					
GRAIN SIZE DISTRIBUTION SAND AND SILT					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	May 2013	SCALE	N/A	REV.
CHECK	SEMC	May 2013			
APPR	JMAC	May 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE C1		



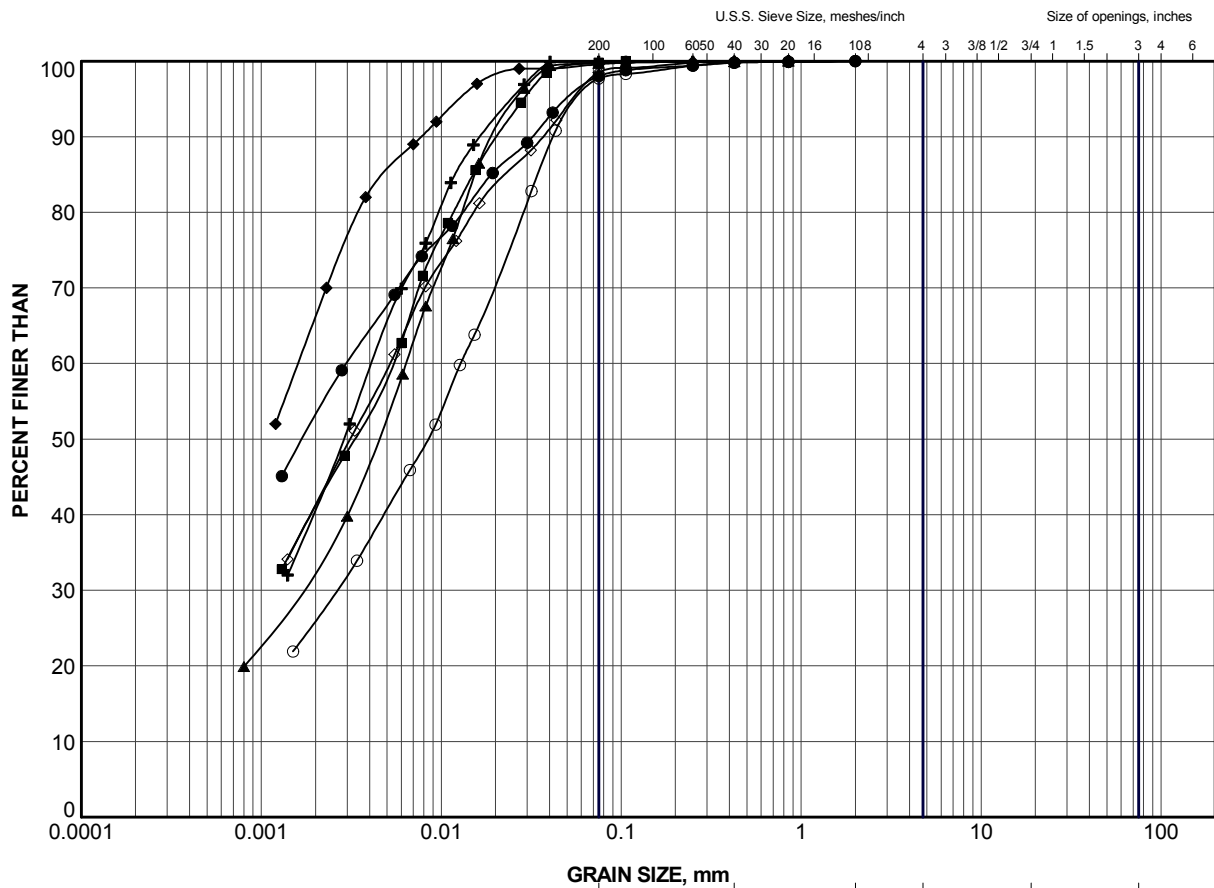
CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC2-2	5	306.3
■	BC3-1	6	304.6
▲	BC3-2	5	305.4
+	BC3-3	8	302.2
◆	BC4-1	3	303.2
◇	H6-2	6	304.9
○	H6-4	6	305.0
△	H6-7	8	302.6
⊗	H6-8	6b	304.7
⊕	H6-10	7	303.9

PROJECT					
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650					
TITLE					
GRAIN SIZE DISTRIBUTION CLAYEY SILT to SILTY CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	May 2013	SCALE	N/A	REV.
CHECK	SEMC	May 2013			
APPR	JMAC	May 2013			
			FIGURE C2.1		





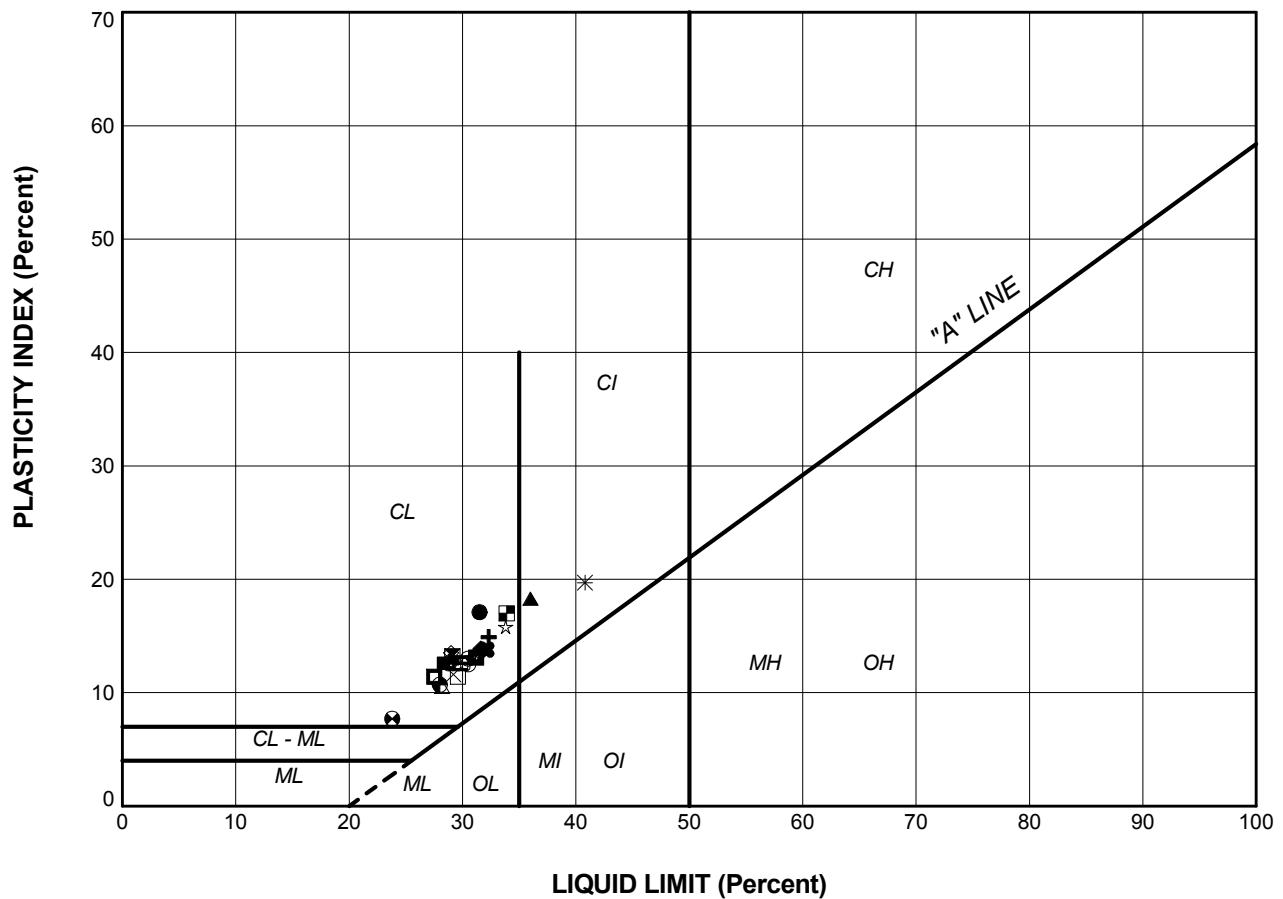
CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	H6-12	4	305.7
■	H6-16	4	303.3
▲	H6-18	4	303.4
+	H7-1	2	304.1
◆	H7-6	3	302.6
◇	H7-10	1b	305.2
○	H7-13	1b	307.2


PROJECT					
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650					
TITLE					
GRAIN SIZE DISTRIBUTION CLAYEY SILT to SILTY CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	May 2013	SCALE	N/A	REV.
CHECK	SEMC	May 2013			
APPR	JMAC	May 2013			
			FIGURE C2.2		

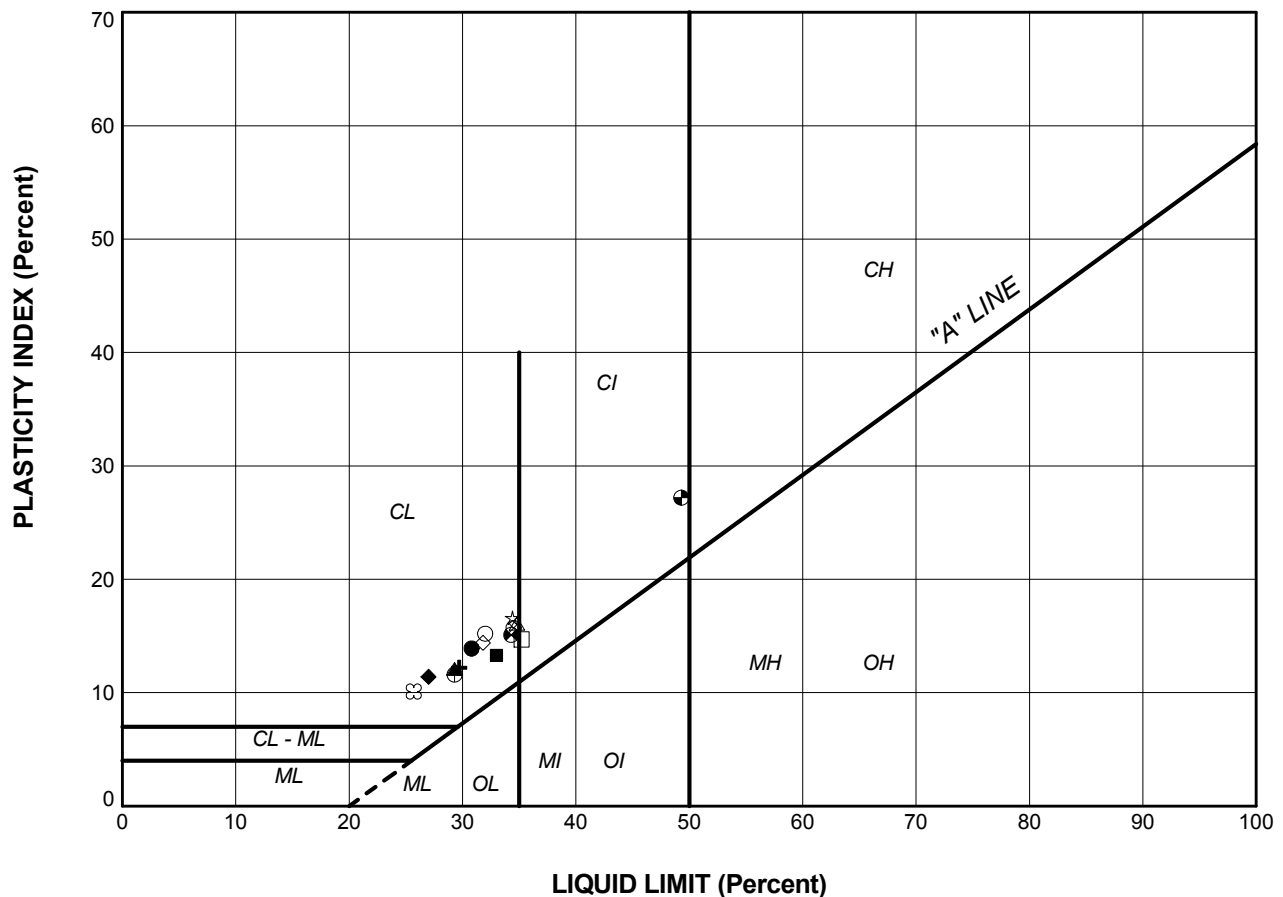




LEGEND


SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BC3-1	6	31.5	14.4	17.1
■	BC3-1	8	28.3	15.7	12.6
▲	BC3-2	5	36.0	17.7	18.3
+	BC3-2	7	32.3	17.4	14.9
◆	BC3-3	6	31.6	17.7	13.9
◇	BC3-3	8	29.0	15.5	13.5
○	BC4-1	3	30.5	18.0	12.5
△	BC4-2	1b	28.2	17.7	10.5
⊗	BC4-3	3	31.2	18.0	13.2
⊕	H6-1	5b	28.8	16.2	12.6
□	H6-2	6	29.6	18.2	11.4
⊗	H6-2	7	23.8	16.1	7.7
⊕	H6-3	6	28.0	17.3	10.7
☆	H6-5	8	33.8	18.0	15.8
⊗	H6-6	7	30.0	17.4	12.6
⊗	H6-7	8	29.1	15.9	13.2
⊗	H6-7A	1	30.5	17.5	13.0
⊕	H6-8	6b	29.0	16.3	12.7
×	H6-9	7	29.2	17.6	11.6
⊗	H6-10	7	32.1	18.3	13.8
■	H6-11	7	31.2	18.1	13.1
*	H6-12	4	40.8	21.1	19.7
□	H6-13	6	27.5	16.1	11.4
⊗	H6-15	4	33.9	16.9	17.0
⊗	H6-16	4	29.8	17.2	12.6

PROJECT				
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650				
TITLE				
PLASTICITY CHART CLAYEY SILT to SILTY CLAY				
PROJECT No.		10-1191-0044		FILE No.
DRAWN		JJL	May 2013	SCALE
CHECK		SEMC	May 2013	N/A
APPR		JMAC	May 2013	REV.
 Golder Associates SUDBURY, ONTARIO		FIGURE C3.1		



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	H6-17	4	30.8	16.9	13.9
■	H6-18	2	33.0	19.7	13.3
▲	H6-18	4	29.3	17.2	12.1
+	H6-19	3	29.7	17.5	12.2
◆	H6-S1	1	27.0	15.6	11.4
◇	H6-S4	1	31.8	17.4	14.4
○	H6-S5	1	32.0	16.8	15.2
△	H7-1	2	34.8	18.7	16.1
⊗	H7-2	1	34.5	18.8	15.7
⊕	H7-3	3	29.3	17.7	11.6
□	H7-4	2	35.2	20.5	14.7
⊙	H7-5	2	34.3	19.2	15.1
⊛	H7-6	3	49.3	22.1	27.2
☆	H7-8	2	34.4	17.8	16.6
⊗	H7-10	1b	25.7	15.6	10.1

PROJECT					
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650					
TITLE					
PLASTICITY CHART CLAYEY SILT to SILTY CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013			
APPR	JMAC	Jul 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE C3.2		

CONSOLIDATION TEST SUMMARY**FIGURE C4****Pg. 1 of 4****SAMPLE IDENTIFICATION**

Project Number:	10-1191-0044	Sample Number:	1
Borehole Number:	H6-7A	Sample Depth, m:	6.4

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	9/12/12		
Date Completed	9/24/10		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.526	Unit Weight, kN/m ³	16.88
Sample Diameter, cm	6.351	Dry Unit Weight, kN/m ³	10.75
Area, cm ²	31.68	Specific Gravity, Measured	2.74
Volume, cm ³	80.02	Solids Height, cm	1.010
Water Content, %	56.91	Volume of Solids, cm ³	32.01
Wet Mass, g	137.70	Volume of Voids, cm ³	48.02
Dry Mass, g	87.76	Degree of Saturation, %	104.0

TEST COMPUTATIONS

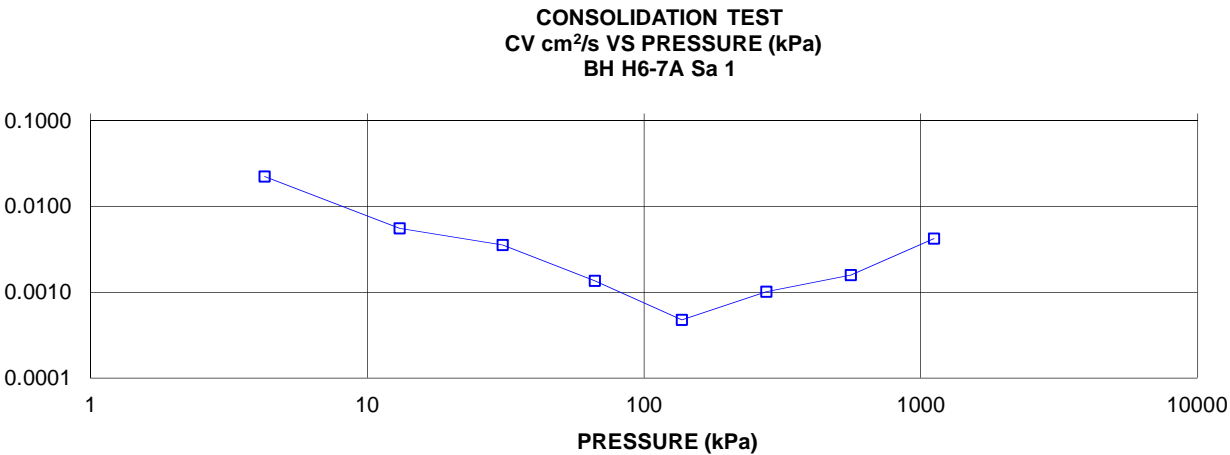
Pressure kPa	Primary Consolidation	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s	Total Work kJ/m ³
0	0	2.526	1.500	2.526					
4	0.09	2.517	1.492	2.522	60	0.0225	8.22E-04	1.81E-06	0.007
13	0.06	2.512	1.486	2.514	240	0.0056	2.47E-04	1.35E-07	0.026
31	0.17	2.495	1.470	2.503	375	0.0035	3.67E-04	1.27E-07	0.171
66	0.35	2.460	1.435	2.478	960	0.0014	3.94E-04	5.23E-08	0.856
137	1.38	2.322	1.298	2.391	2535	0.0005	7.76E-04	3.64E-08	6.546
277	1.25	2.197	1.175	2.260	1058	0.0010	3.52E-04	3.53E-08	17.683
558	0.88	2.109	1.087	2.153	614	0.0016	1.24E-04	1.95E-08	34.407
1117	0.61	2.048	1.027	2.079	217	0.0042	4.32E-05	1.79E-08	58.634
558	-0.05	2.053	1.032	2.051					
137	-0.21	2.074	1.052	2.063					
31	-0.25	2.099	1.077	2.086					
4	-0.27	2.126	1.104	2.112					

Note:

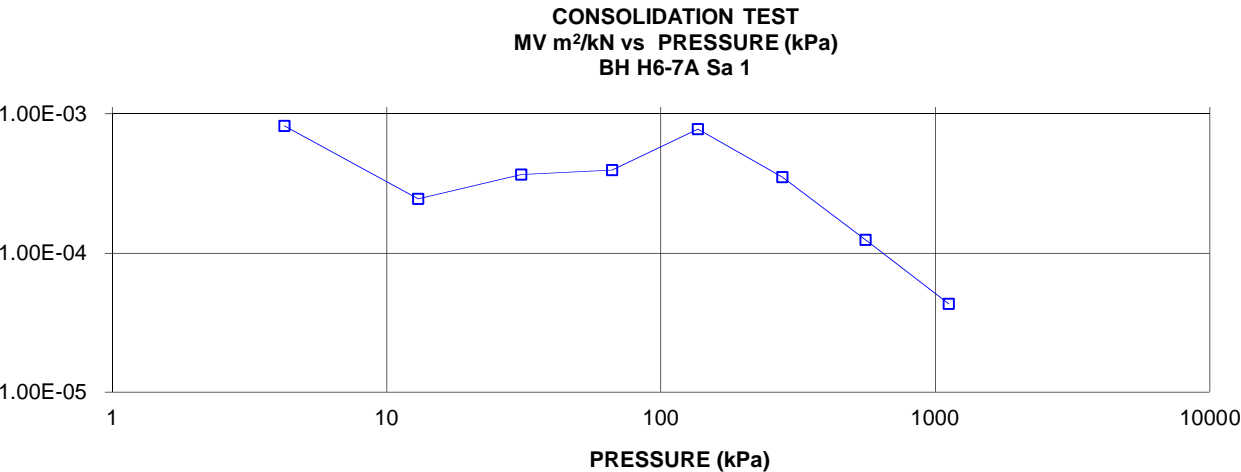
k calculated using α based on t₉₀ values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	2.126	Unit Weight, kN/m ³	17.19
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	12.78
Area, cm ²	31.68	Specific Gravity, Measured	2.74
Volume, cm ³	67.33	Solids Height, cm	1.010
Water Content, %	34.48	Volume of Solids, cm ³	32.01
Wet Mass, g	118.02	Volume of Voids, cm ³	35.33
Dry Mass, g	87.76		

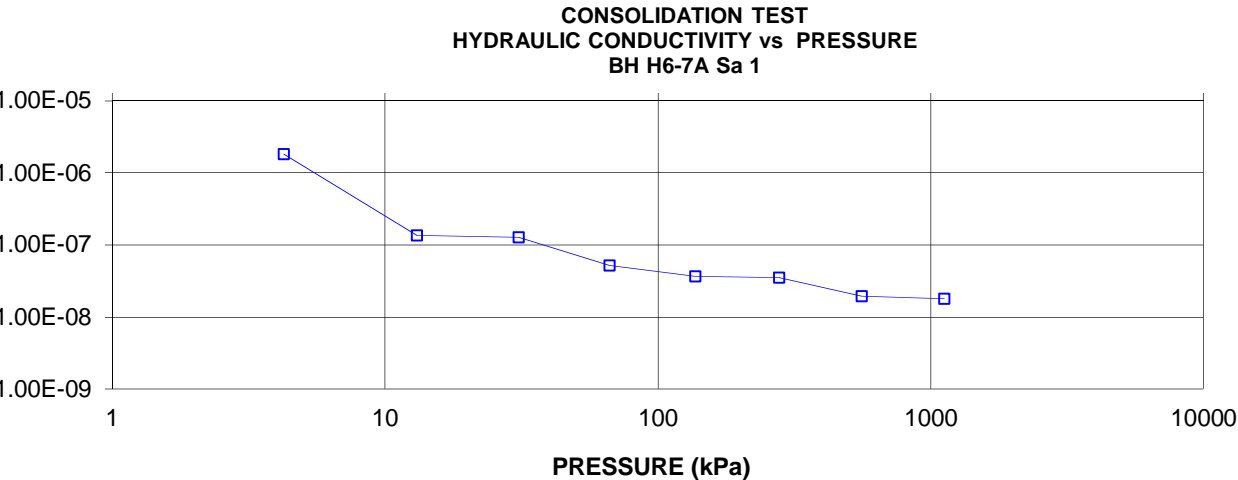
COEFFICIENT OF CONSOLIDATION,
cm²/s



VOLUME COMPRESSIBILITY, m²/kN



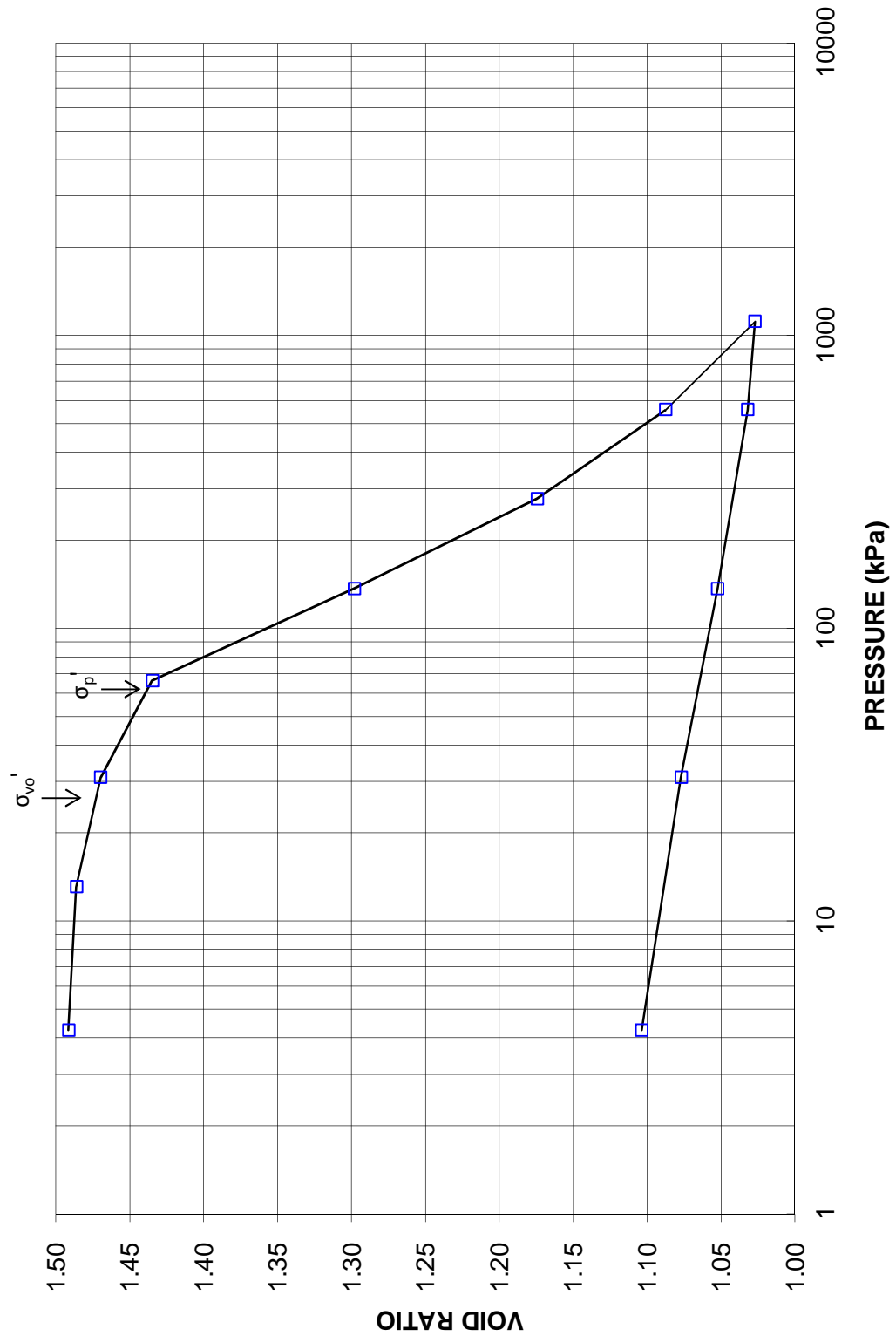
HYDRAULIC CONDUCTIVITY,
cm/s



**CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE**

FIGURE C4
Pg. 3 of 4

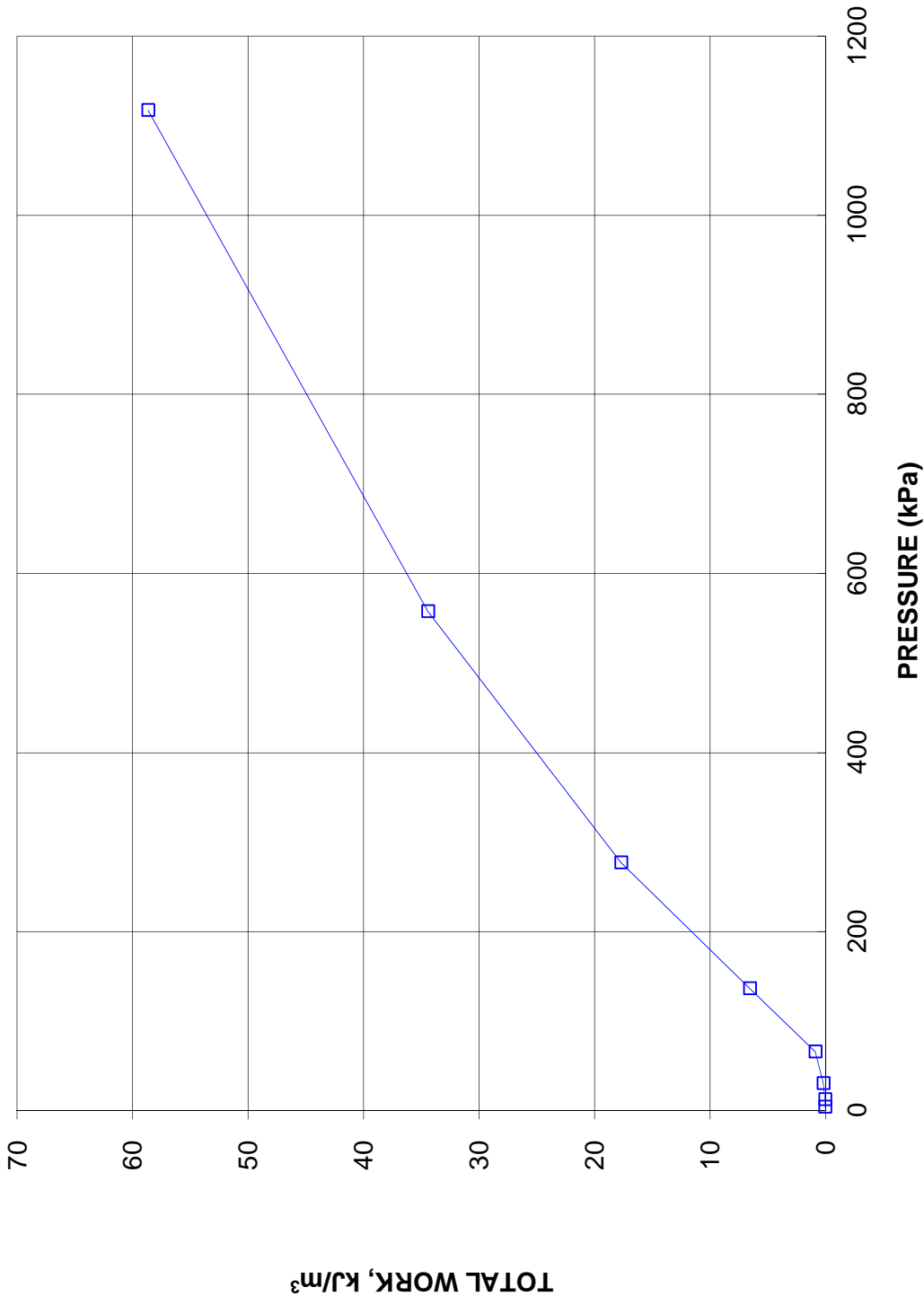
**CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH H6-7A SA 1**



CONSOLIDATION TEST
TOTAL WORK VS PRESSURE

FIGURE C4
Pg. 4 of 4

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH H6-7A Sa 1



CONSOLIDATION TEST SUMMARY**FIGURE C5**

Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Number	10-1191-0044	Sample Number	1
Borehole Number	H6-S1	Sample Depth, m	4.6-5.0

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	5/30/2013		
Date Completed	6/17/2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.53	Unit Weight, kN/m ³	18.92
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	14.32
Area, cm ²	31.71	Specific Gravity, measured	2.76
Volume, cm ³	80.26	Solids Height, cm	1.339
Water Content, %	32.17	Volume of Solids, cm ³	42.45
Wet Mass, g	154.86	Volume of Voids, cm ³	37.80
Dry Mass, g	117.17	Degree of Saturation, %	99.7

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	2.531	0.890	2.531				
6.03	2.529	0.889	2.530	1	1.36E+00	1.31E-04	1.74E-05
10.63	2.527	0.888	2.528	73	1.86E-02	1.37E-04	2.50E-07
20.64	2.519	0.882	2.523	94	1.44E-02	3.32E-04	4.67E-07
10.63	2.523	0.884	2.521				
6.03	2.525	0.886	2.524				
20.80	2.518	0.881	2.522	60	2.25E-02	1.93E-04	4.24E-07
40.01	2.504	0.870	2.511	305	4.38E-03	2.90E-04	1.25E-07
78.58	2.470	0.845	2.487	296	4.43E-03	3.51E-04	1.53E-07
155.74	2.386	0.782	2.428	505	2.47E-03	4.30E-04	1.04E-07
308.37	2.298	0.716	2.342	265	4.39E-03	2.27E-04	9.77E-08
616.10	2.219	0.658	2.259	135	8.01E-03	1.01E-04	7.94E-08
1233.99	2.149	0.605	2.184	167	6.05E-03	4.50E-05	2.67E-08
2469.94	2.080	0.553	2.114	109	8.69E-03	2.21E-05	1.88E-08
1233.99	2.081	0.555	2.080				
308.37	2.102	0.570	2.092				
78.58	2.126	0.588	2.114				
20.64	2.149	0.605	2.137				
6.03	2.165	0.617	2.157				

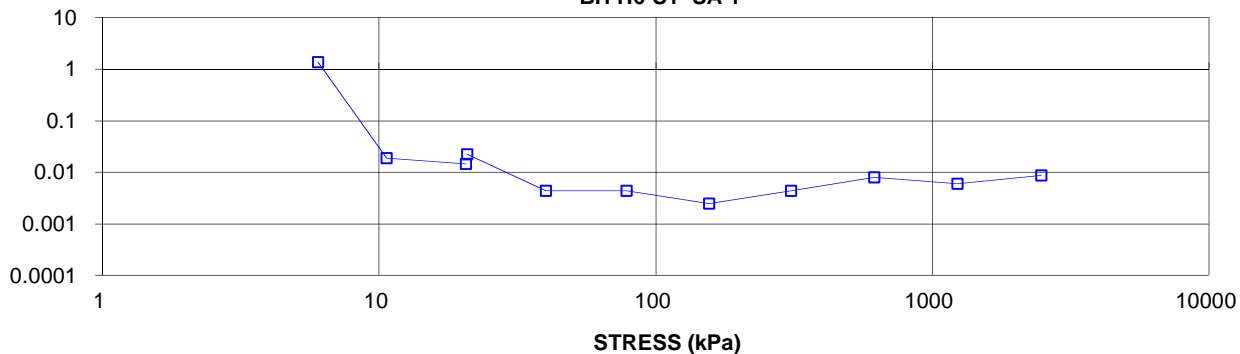
Specimen taken 28 to 36cm from top of the tube.
k calculated using cv based on t₉₀ values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	2.17	Unit Weight, kN/m ³	20.48
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	16.74
Area, cm ²	31.71	Specific Gravity, measured	2.76
Volume, cm ³	68.66	Solids Height, cm	1.339
Water Content, %	22.39	Volume of Solids, cm ³	42.45
Wet Mass, g	143.40	Volume of Voids, cm ³	26.20
Dry Mass, g	117.17		

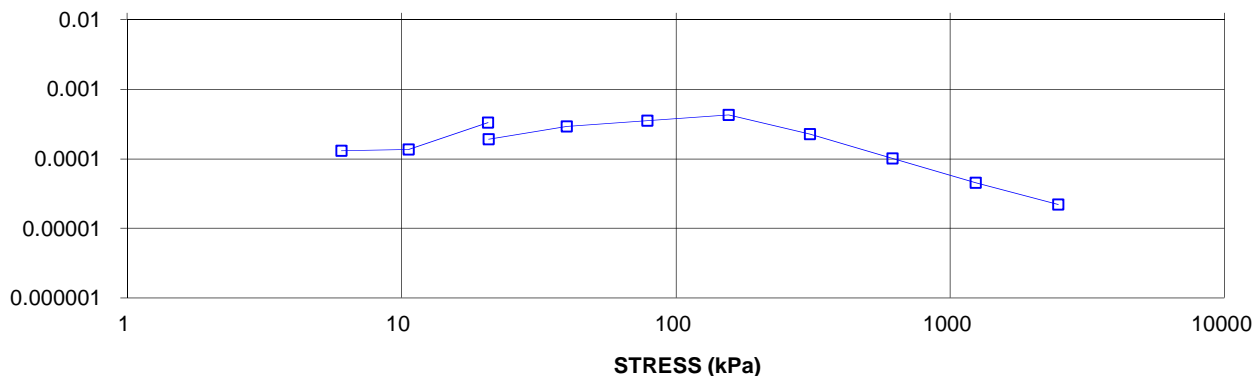
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS STRESS (kPa)
BH H6-S1 SA 1



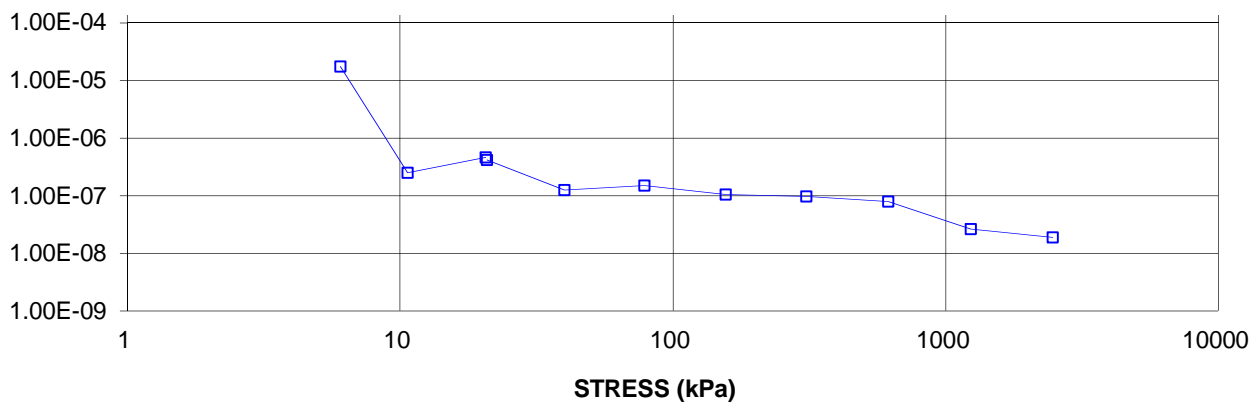
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs STRESS (kPa)
BH H6-S1 SA 1



HYDRAULIC CONDUCTIVITY,
cm/s

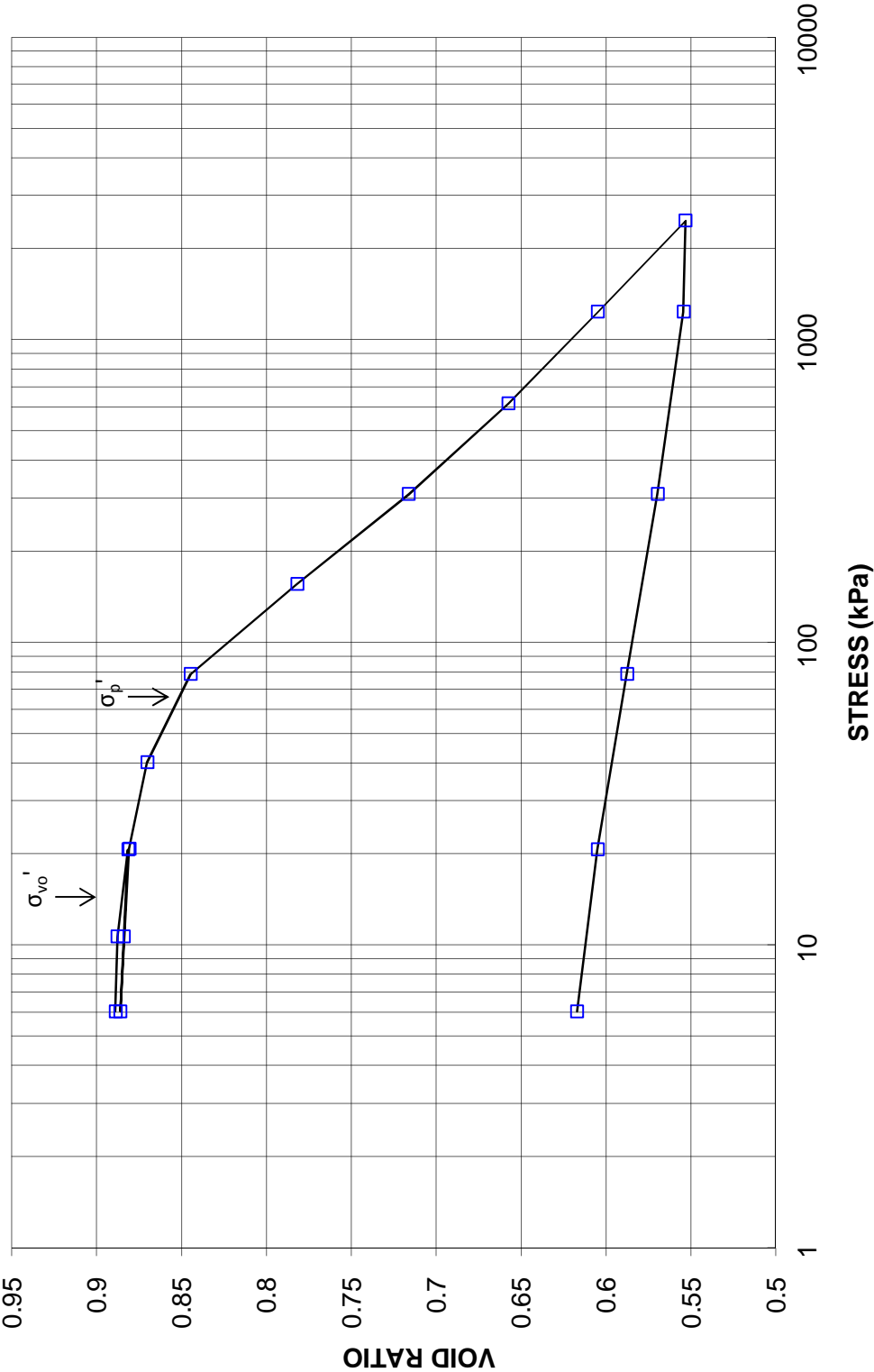
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs STRESS
BH H6-S1 SA 1



CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE C5
Pg. 3 of 4

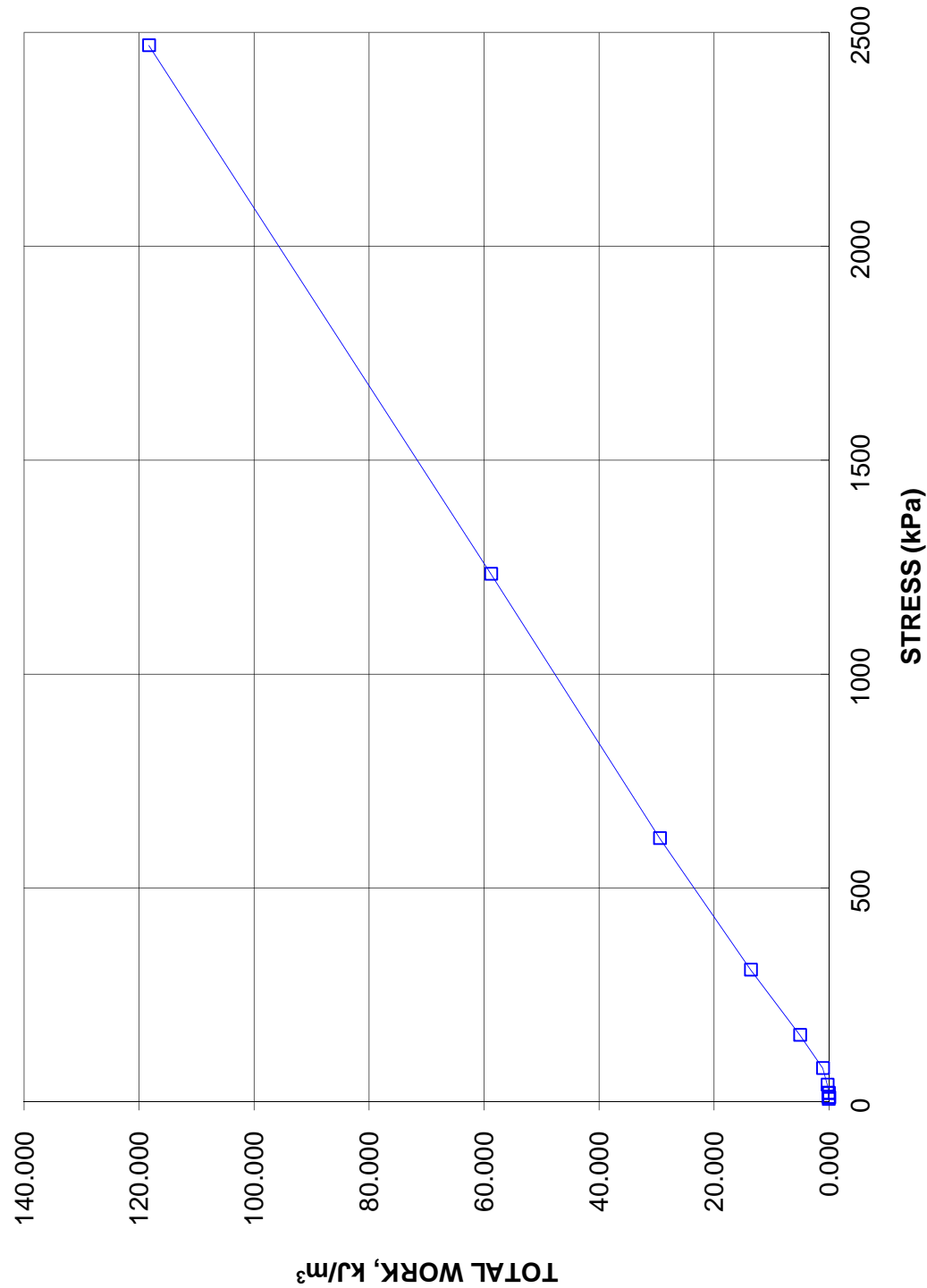
CONSOLIDATION TEST
VOID RATIO vs STRESS
BH H6-S1 SA 1



CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE C5
Pg. 4 of 4

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs STRESS
BH H6-S1 SA 1



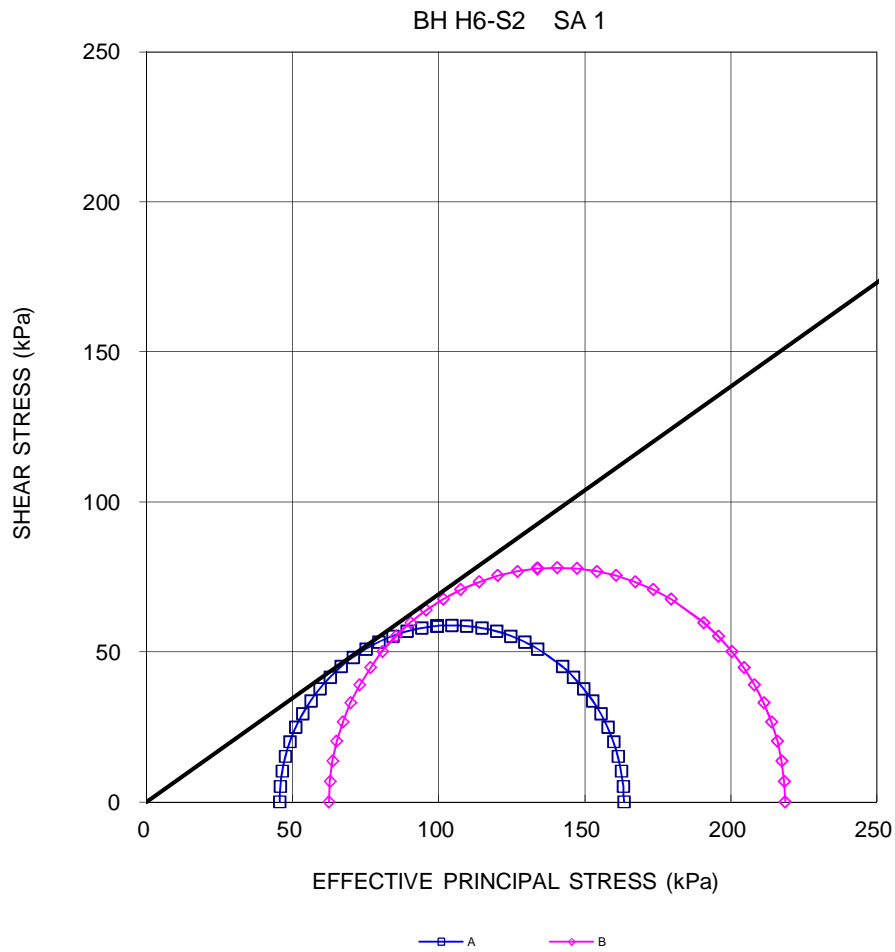
**CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 1 OF 4**

FIGURE C6

TEST STAGE	A	B
BOREHOLE NUMBER	H6-S2	H6-S2
SAMPLE	1	1
DEPTH, m	4.57-5.03	4.57-5.03
SPECIMEN DIAMETER, cm	5.00	5.00
SPECIMEN HEIGHT, cm	10.11	10.08
NATURAL WATER CONTENT, %	29.3	31.8
DRY DENSITY, Mg/m ³	1.54	1.48
WATER CONTENT AFTER SATURATION, %	30.6	33.2
CELL PRESSURE, σ_3 , kPa	165.0	235.0
BACK PRESSURE, kPa	135.0	135.0
PORE PRESSURE PARAMETER "B"	0.99	0.97
CONSOLIDATION PRESSURE, σ_c , kPa	30.0	100.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	3.8	4.3
WATER CONTENT AFTER CONSOLIDATION, %	28.2	30.3
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5
TIME TO FAILURE, HOURS	27.1	22.2
WATER CONTENT AFTER TEST, %	27.5	28.4
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$, kPa	117.8	156.1
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ maximum, %	13.5	11.1
MAX EFFECTIVE PRINCIPAL STRESS RATIO, (σ'_1 / σ'_3) maximum	3.8	3.5
DEVIATOR STRESS AT (σ'_1 / σ'_3) maximum, kPa	78.0	148.4
AXIAL STRAIN AT (σ'_1 / σ'_3) maximum, %	2.8	7.6
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1 - \sigma_3)$ maximum	-0.13	0.24
PORE PRESSURE PARAMETER, Af, AT (σ'_1 / σ'_3) maximum	0.03	0.28
FILTER DRAINS USED, y/n	y	y
TEST NOTES:		
<p style="text-align: center;">Specimen A taken 10-23 cm from top of tube. Specimen B taken 23-36 cm from top of tube.</p>		
FAILURE PLANE NUMBER	1.0	1.0
ANGLE OF FAILURE, DEGREES	70.0	70.0
<div style="display: flex; justify-content: space-between; align-items: flex-end; padding-top: 20px;"> <div> <p>Date: 6/3/2013</p> <p>Project No. 10-1191-0044</p> </div> <div style="text-align: center;"> <p>Golder Associates</p> </div> <div> <p>Prepared By: LH</p> <p>Checked By: MT</p> </div> </div>		

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 2 OF 4

FIGURE C6



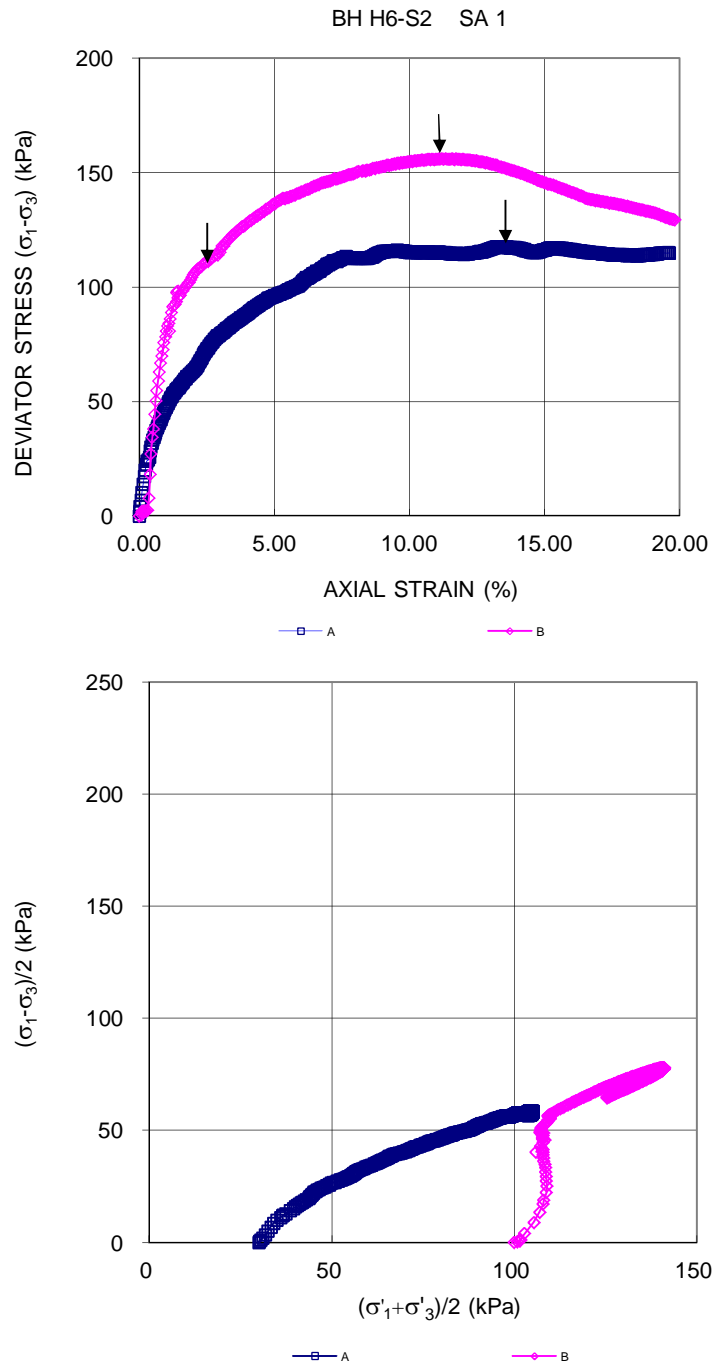
Date: 6/3/2013
Project No. 10-1191-0044

Golder Associates

Prepared By: LH
Checked By: MT

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 3 OF 4

FIGURE C6



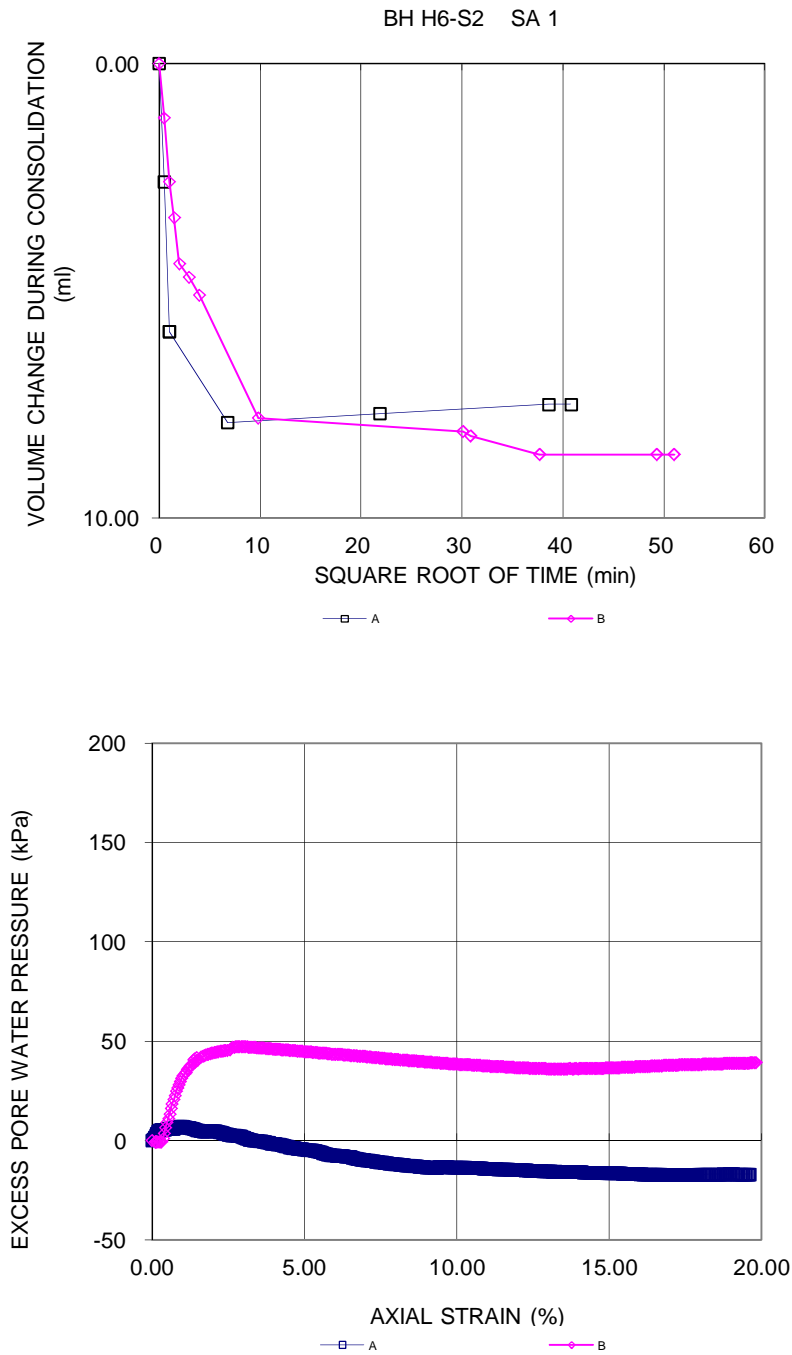
Date: 6/3/2013
Project No. 10-1191-0044

Golder Associates

Prepared By: LH
Checked By: MT

**CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 4 OF 4**

FIGURE C6



Date: 6/3/2013
Project No. 10-1191-0044

Golder Associates

Prepared By: LH
Checked By: MT

**CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 1 OF 4**

FIGURE C7

TEST STAGE	A	B	C
BOREHOLE NUMBER	H6-S5	H6-S5	H6-S4
SAMPLE	1	1	1
DEPTH, m	1.52-1.98	1.52-1.98	2.74-3.35
SPECIMEN DIAMETER, cm	5.03	4.98	4.96
SPECIMEN HEIGHT, cm	10.09	10.11	10.09
NATURAL WATER CONTENT, %	29.3	28.7	29.2
DRY DENSITY, Mg/m ³	1.53	1.54	1.54
WATER CONTENT AFTER SATURATION, %	32.0	30.3	29.3
CELL PRESSURE, σ_3 , kPa	375.0	460.0	415.0
BACK PRESSURE, kPa	275.0	275.0	65.0
PORE PRESSURE PARAMETER "B"	0.96	0.98	0.96
CONSOLIDATION PRESSURE, σ_c , kPa	100.0	185.0	350.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	2.8	3.2	7.0
WATER CONTENT AFTER CONSOLIDATION, %	32.2	28.2	24.8
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, HOURS	16.1	17.3	20.3
WATER CONTENT AFTER TEST, %	28.4	26.2	25.2
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$, kPa	207.7	248.9	332.7
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ maximum, %	8.0	8.6	10.1
MAX EFFECTIVE PRINCIPAL STRESS RATIO, (σ'_1 / σ'_3) maximum	3.5	3.6	3.7
DEVIATOR STRESS AT (σ'_1 / σ'_3) maximum, kPa	183.8	246.1	324.5
AXIAL STRAIN AT (σ'_1 / σ'_3) maximum, %	3.4	6.9	7.2
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1 - \sigma_3)$ maximum	0.12	0.36	0.67
PORE PRESSURE PARAMETER, Af, AT (σ'_1 / σ'_3) maximum	0.21	0.37	0.70
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:			
<p style="margin-left: 40px;">Specimen A taken 14-27 cm from top of tube.</p> <p style="margin-left: 40px;">Specimen B taken 27-40 cm from top of tube.</p> <p style="margin-left: 40px;">Specimen C taken 21-34 cm from top of tube.</p>			
FAILURE PLANE NUMBER	1.0	1.0	-
ANGLE OF FAILURE, DEGREES	70.0	65.0	bulged

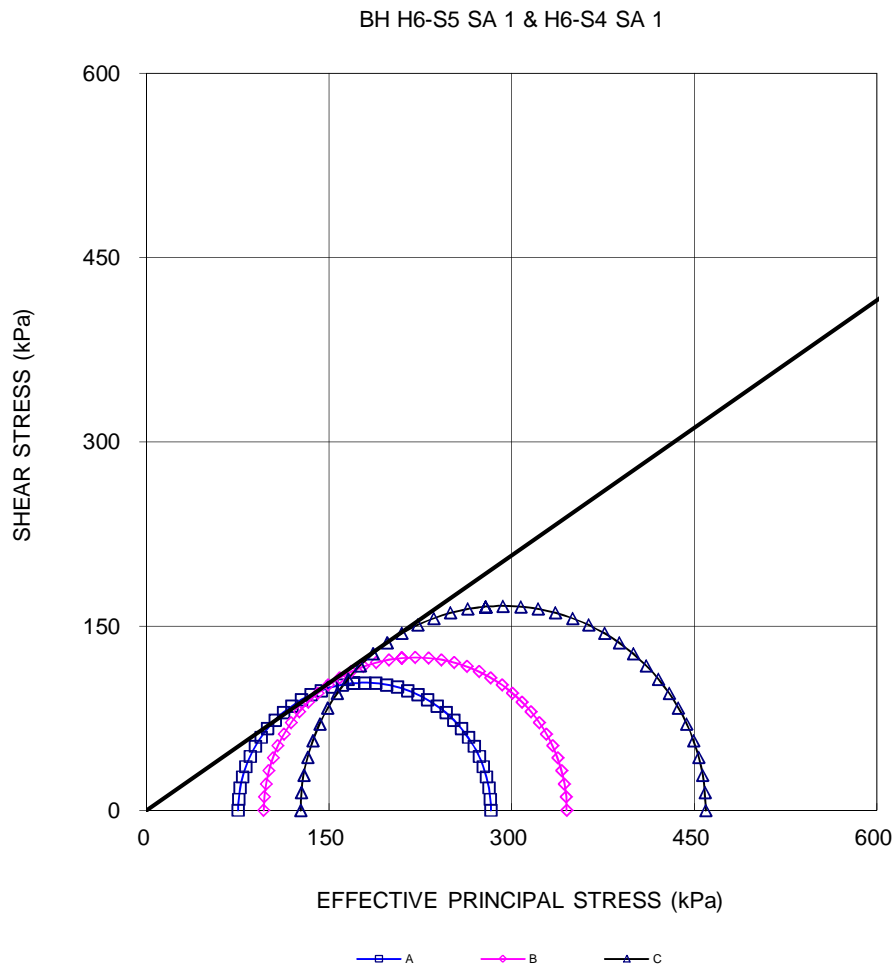
Date: 6/10/2013
Project No. 10-1191-0044

Golder Associates

Prepared By: LH
Checked By: MT

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 2 OF 4

FIGURE C7



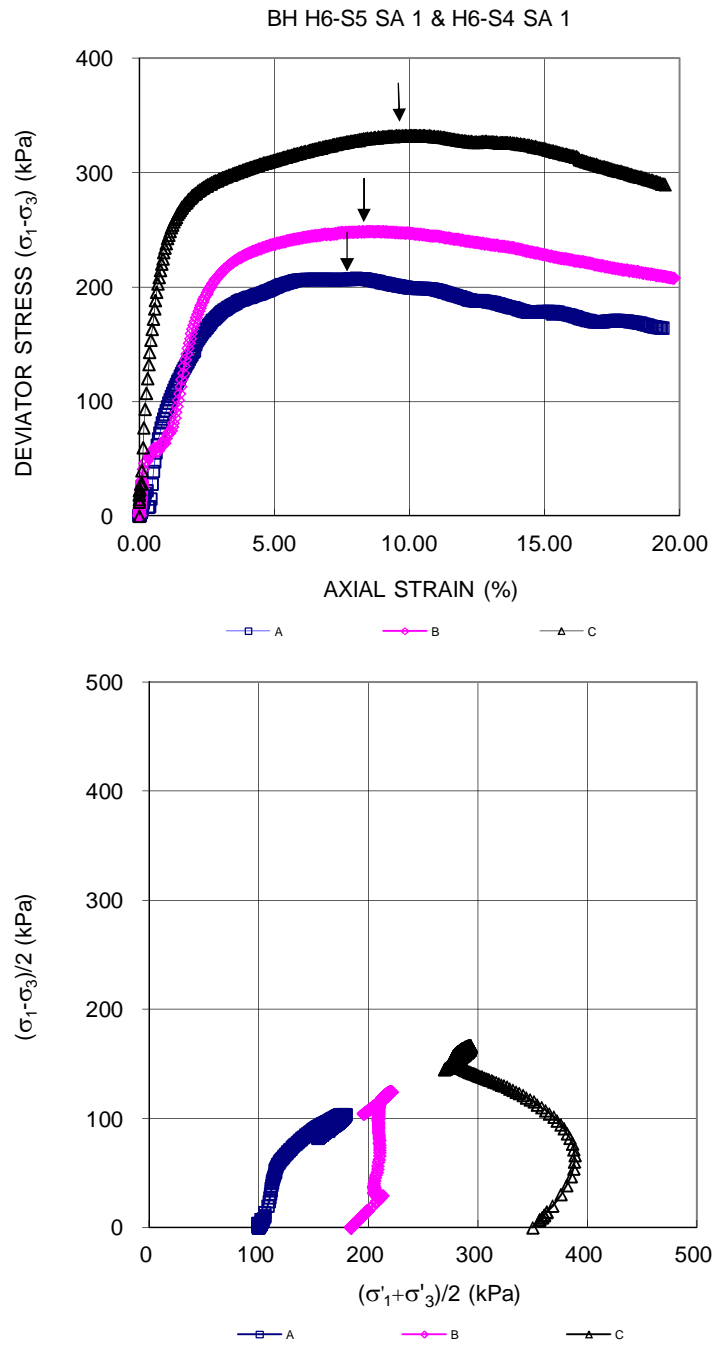
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Project No. 10-1191-0044

Golder Associates

Prepared By: LH
Checked By: MT

**CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 3 OF 4**

FIGURE C7



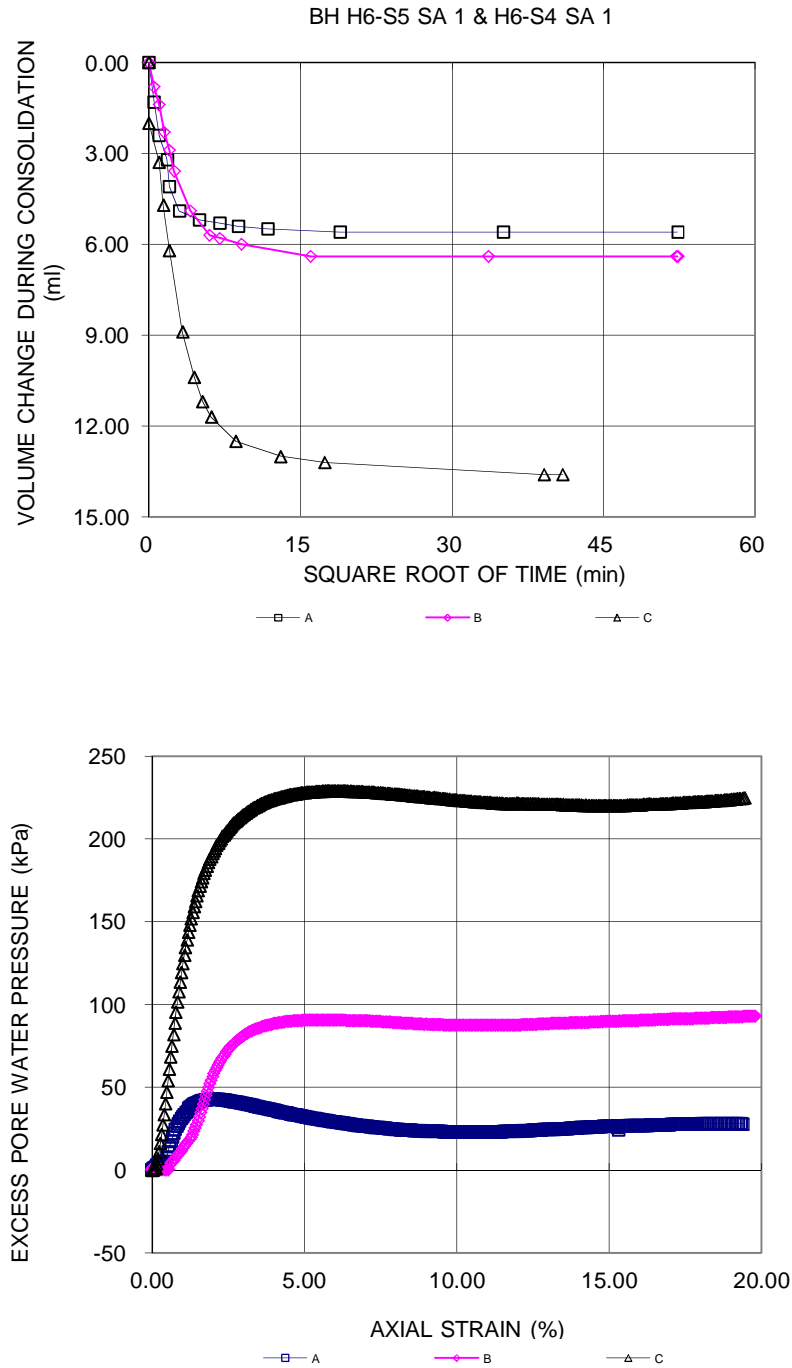
Date: 6/10/2013
Project No. 10-1191-0044

Golder Associates

Prepared By: LH
Checked By: MT

**CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 4 OF 4**

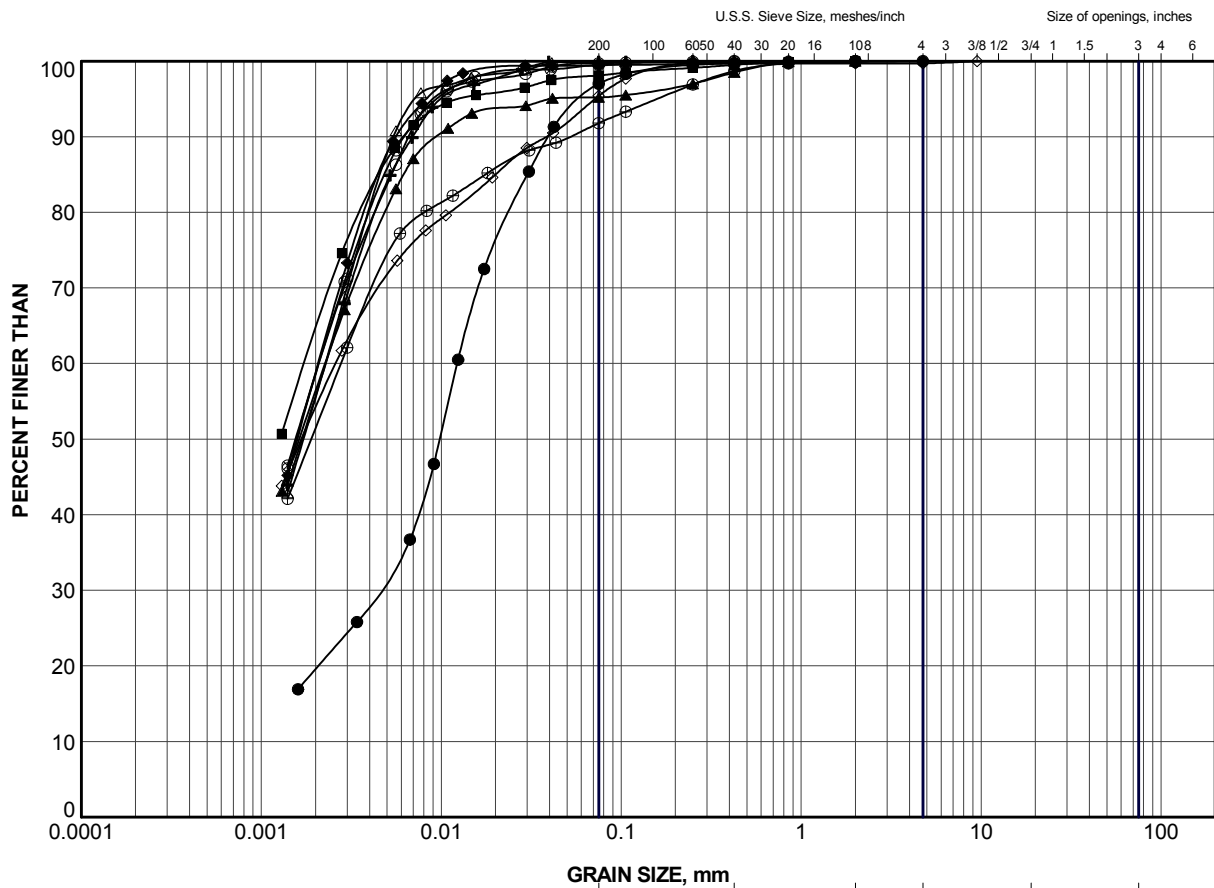
FIGURE C7



Date: 6/10/2013
Project No. 10-1191-0044

Golder Associates

Prepared By: LH
Checked By: MT



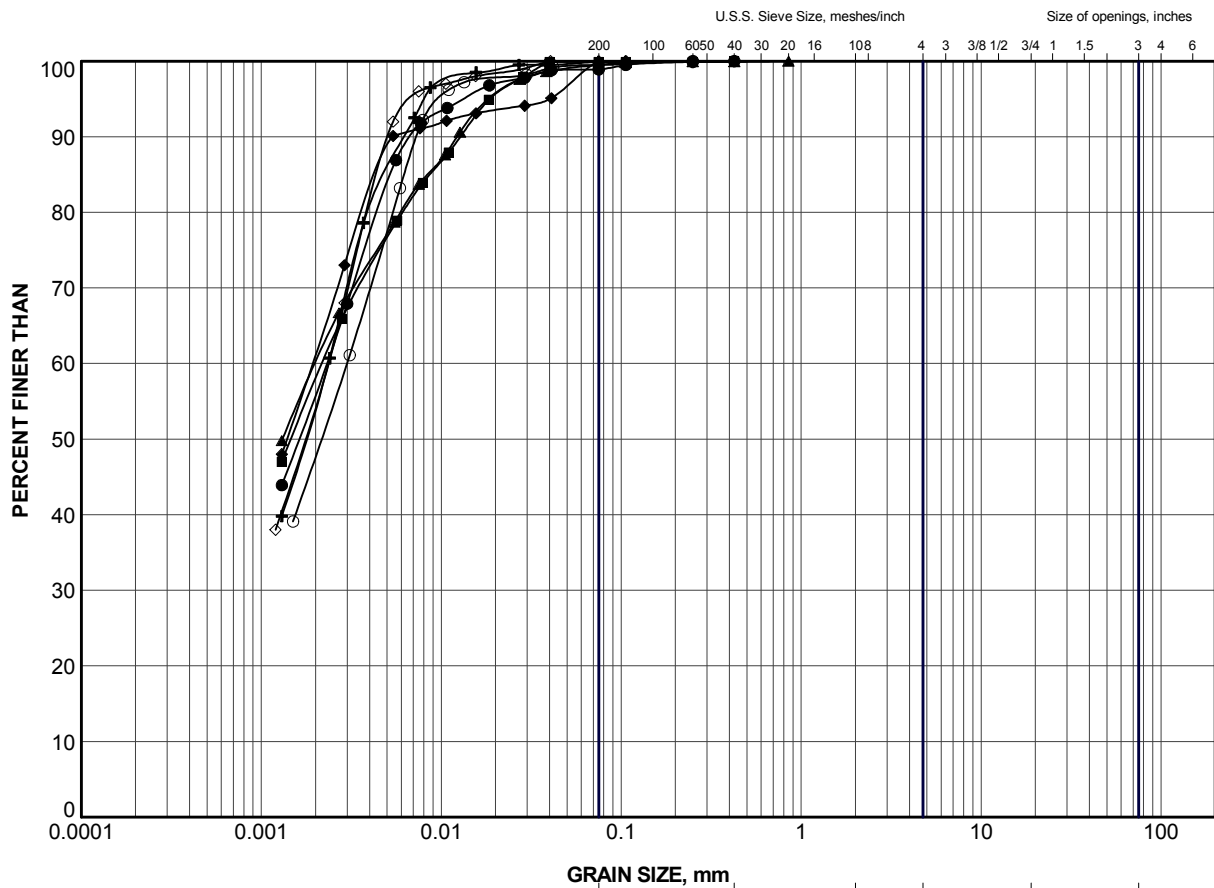
CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC2-1	4a	306.3
■	BC4-1	4	301.8
▲	BC4-1	6	298.8
+	BC4-2	5	299.2
◆	H6-2	9	301.1
◇	H6-3	9	301.3
○	H6-4	9	301.2
△	H6-5	12	296.5
⊗	H6-6	12	297.9
⊕	H6-8	10	299.5

PROJECT					
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650					
TITLE					
GRAIN SIZE DISTRIBUTION SILTY CLAY to CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013	FIGURE C8.1		
APPR	JMAC	Jul 2013			





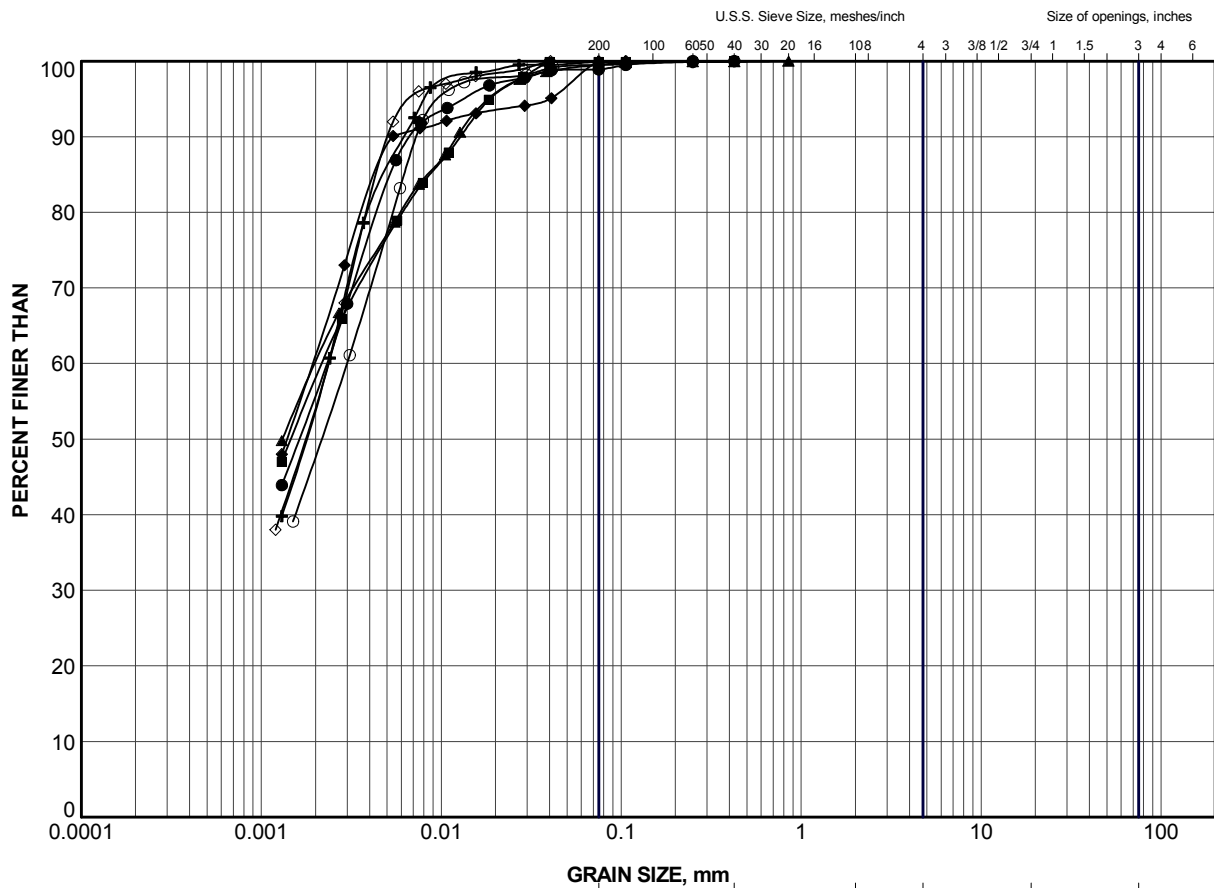
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	H6-9	9	301.0
■	H6-13	8	300.0
▲	H6-15	6	301.9
+	H6-19	5	300.8
◆	H7-3	6	300.2
◇	H7-7	5	300.5
○	H7-15	2	306.4

PROJECT					
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650					
TITLE					
GRAIN SIZE DISTRIBUTION SILTY CLAY to CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013			
APPR	JMAC	Jul 2013			
			FIGURE C8.2		




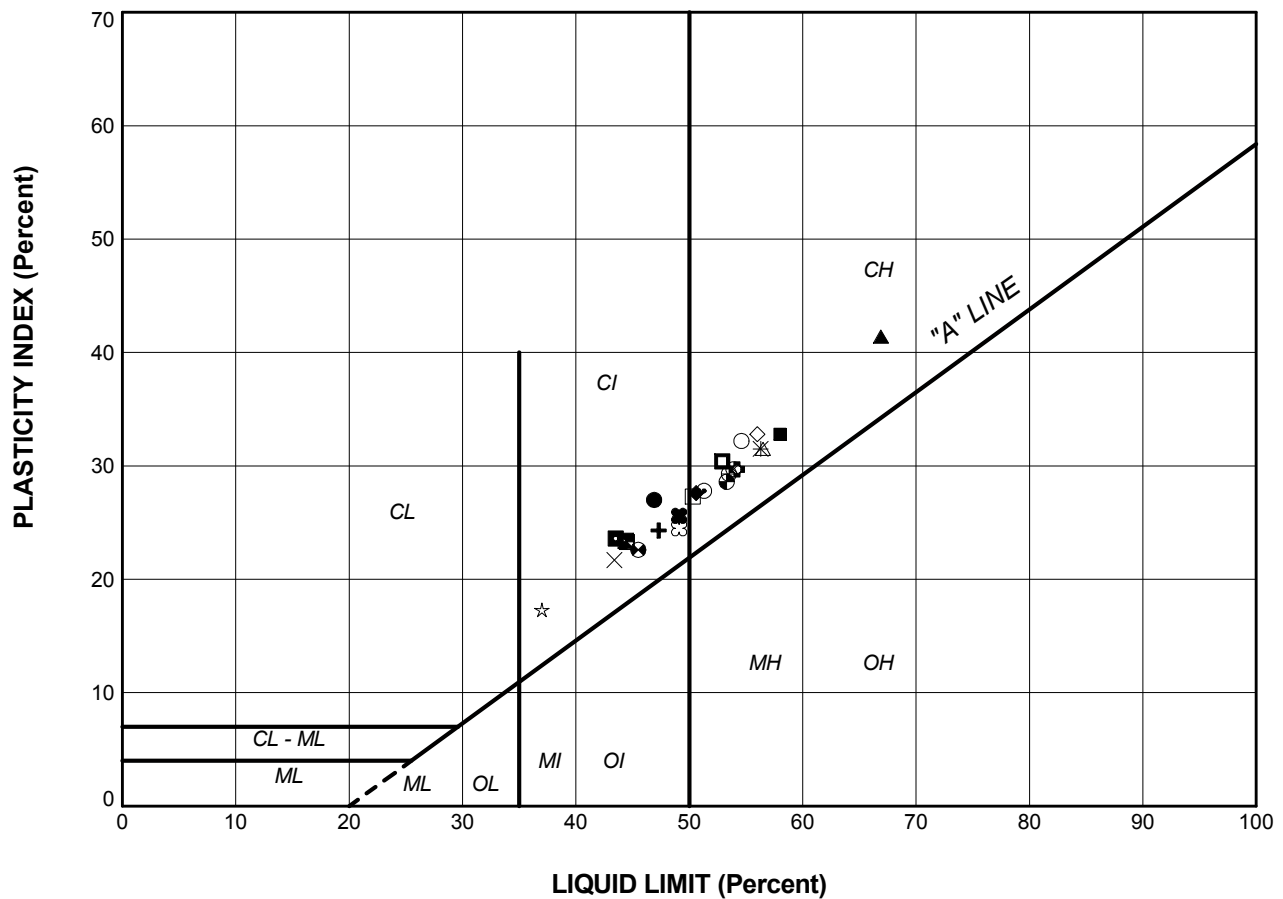


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	H6-9	9	301.0
■	H6-13	8	300.0
▲	H6-15	6	301.9
+	H6-19	5	300.8
◆	H7-3	6	300.2
◇	H7-7	5	300.5
○	H7-15	2	306.4

PROJECT					
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650					
TITLE					
GRAIN SIZE DISTRIBUTION SILTY CLAY to CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013			
APPR	JMAC	Jul 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE C8.2		

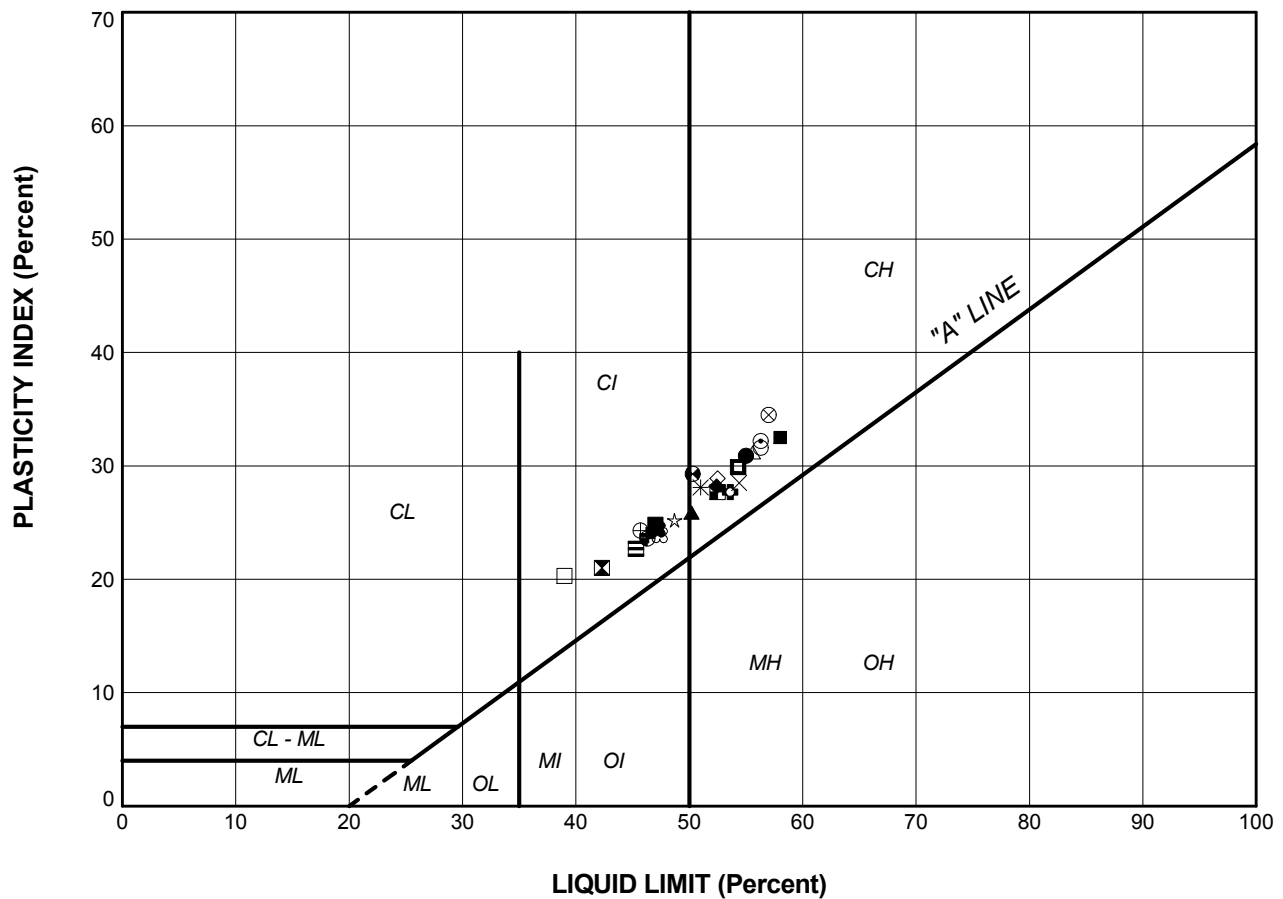


LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BC2-1	3	46.9	19.9	27.0
■	BC2-2	7a	58.0	25.2	32.8
▲	BC3-1	10	66.9	25.5	41.4
+	BC3-2	9	47.3	23.0	24.3
◆	BC3-2	11	50.6	23.0	27.6
◇	BC3-3	10	56.0	23.2	32.8
○	BC3-3	12	54.6	22.4	32.2
△	BC4-1	4	56.5	25.0	31.5
⊗	BC4-1	5	53.9	24.2	29.7
⊕	BC4-1	6	53.5	24.2	29.3
□	BC4-1	7	50.3	23.0	27.3
⊙	BC4-2	3	45.5	22.9	22.6
⊛	BC4-2	5	53.3	24.7	28.6
☆	BC4-2	7	37.0	19.7	17.3
⊘	BC4-3	5	49.1	24.6	24.5
⊗	H6-1	7	44.4	21.0	23.4
⊕	H6-1	9	51.3	23.5	27.8
⊛	H6-1	11	54.2	24.5	29.7
×	H6-2	9	43.4	21.7	21.7
⊗	H6-2	11	49.1	23.5	25.6
■	H6-3	9	43.5	19.9	23.6
*	H6-3	10	56.3	24.8	31.5
□	H6-3	12	52.9	22.5	30.4
■	H6-4	9	44.5	21.2	23.3


PROJECT					
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650					
TITLE					
PLASTICITY CHART SILTY CLAY to CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013	FIGURE C9.1		
APPR	JMAC	Jul 2013			

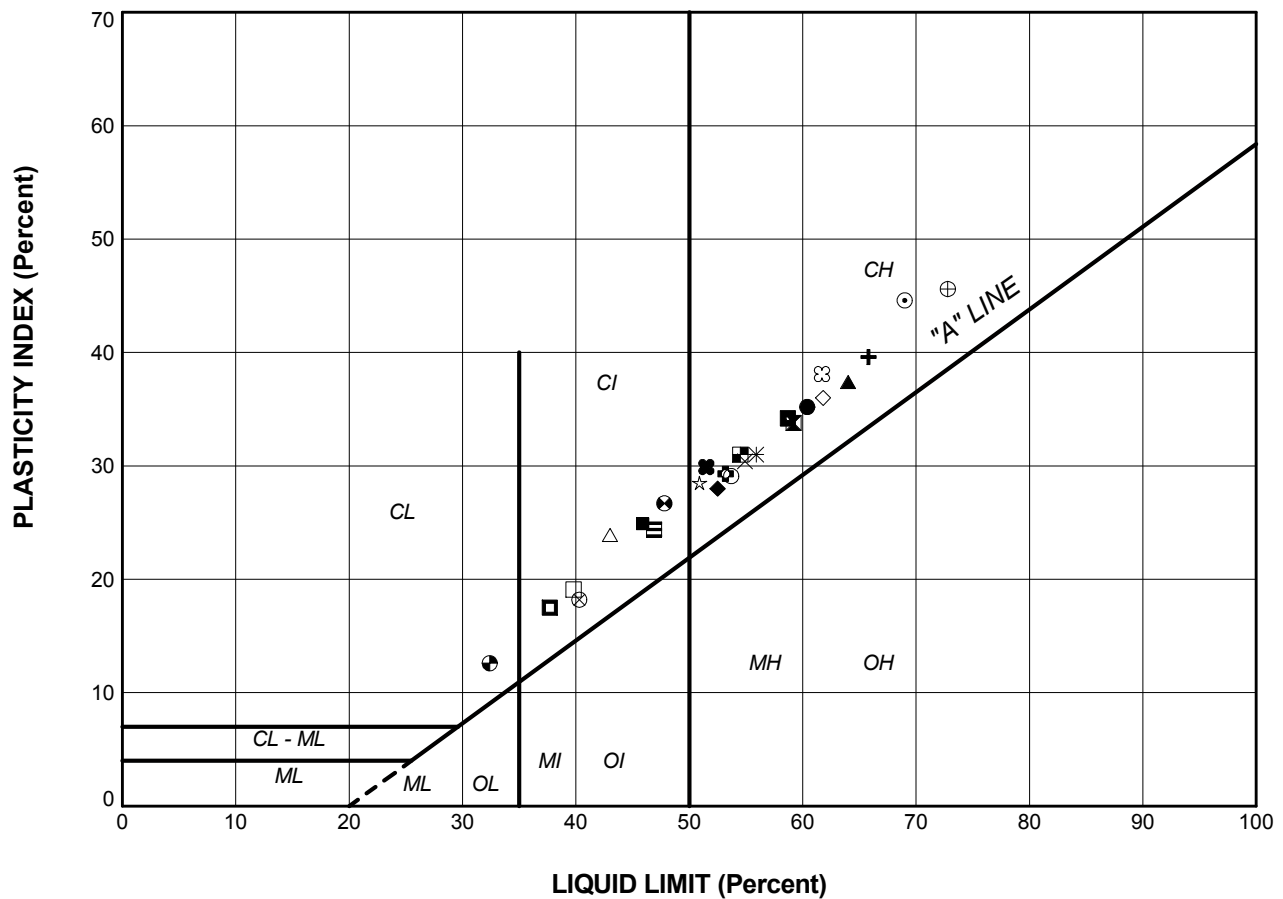




LEGEND


SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	H6-4	12	55.0	24.1	30.9
■	H6-5	10	58.0	25.5	32.5
▲	H6-5	12	50.2	24.3	25.9
+	H6-6	9	46.3	22.4	23.9
◆	H6-6	12	52.4	24.2	28.2
◇	H6-6	14	52.5	23.6	28.9
○	H6-7	10	56.3	24.7	31.6
△	H6-7	12	55.6	24.4	31.2
⊗	H6-7A	4	57.0	22.5	34.5
⊕	H6-8	8	45.7	21.4	24.3
□	H6-8	10	39.0	18.7	20.3
⊗	H6-8	13	50.3	21.0	29.3
⊕	H6-9	9	46.3	22.7	23.6
☆	H6-9	11	48.7	23.5	25.2
⊗	H6-9	13	47.4	23.5	23.9
⊗	H6-10	9	42.3	21.3	21.0
⊕	H6-10	10	56.3	24.1	32.2
⊕	H6-10	12	53.6	25.9	27.7
×	H6-11	10	54.4	25.9	28.5
⊗	H6-11	12	47.2	22.8	24.4
■	H6-12	7	47.0	22.2	24.8
*	H6-12	11	51.0	22.9	28.1
□	H6-13	8	54.3	24.4	29.9
⊗	H6-13	10	52.5	24.8	27.7
⊗	H6-14	6	45.3	22.6	22.7

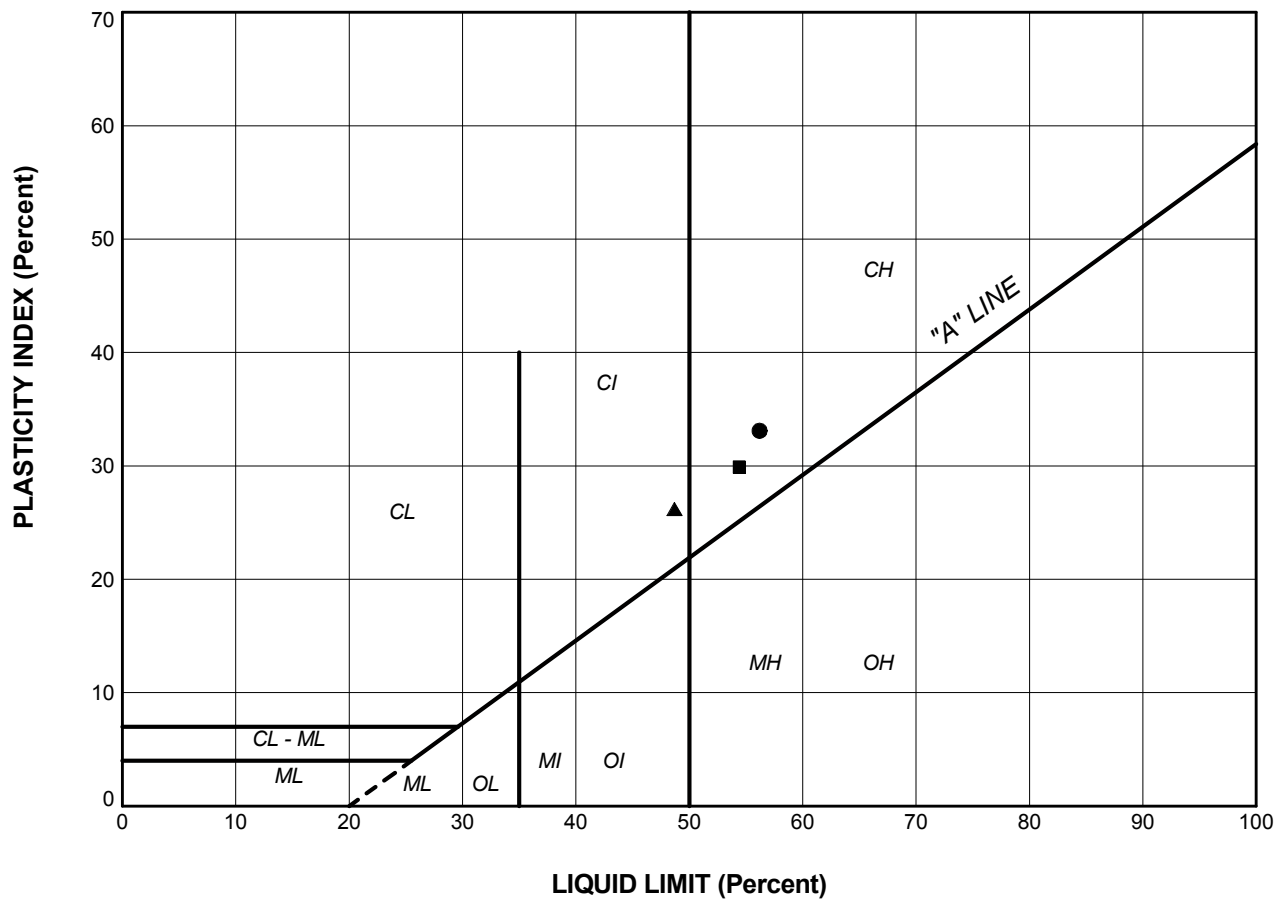
PROJECT					
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650					
TITLE					
PLASTICITY CHART SILTY CLAY to CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013			
APPR	JMAC	Jul 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE C9.2		



LEGEND


SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	H6-14	8	60.4	25.2	35.2
■	H6-15	6	45.9	21.0	24.9
▲	H6-16	6	64.0	26.6	37.4
+	H6-17	6	65.8	26.2	39.6
◆	H6-17	7	52.5	24.5	28.0
◇	H6-18	6	61.8	25.8	36.0
○	H6-19	5	53.7	24.6	29.1
△	H6-S1	2	43.0	19.1	23.9
⊗	H6-S1	4a	40.3	22.1	18.2
⊕	H6-S1	4b	72.8	27.2	45.6
□	H6-S2	2	39.8	20.7	19.1
⊗	H6-S2	4a	47.8	21.1	26.7
⊕	H6-S2	4b	32.4	19.8	12.6
☆	H7-1	4	50.9	22.4	28.5
⊗	H7-1	6	61.7	23.6	38.1
⊗	H7-2	4	59.2	25.4	33.8
⊕	H7-3	6	69.0	24.4	44.6
⊕	H7-3	8	53.2	23.9	29.3
×	H7-4	4	54.9	24.5	30.4
⊗	H7-5	4	51.5	21.6	29.9
■	H7-5	6	58.7	24.5	34.2
*	H7-6	5	55.9	24.9	31.0
□	H7-6	7	37.7	20.2	17.5
■	H7-7	5	54.5	23.5	31.0
■	H7-8	3	46.9	22.5	24.4

PROJECT					
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650					
TITLE					
PLASTICITY CHART SILTY CLAY to CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013			
APPR	JMAC	Jul 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE C9.3		



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	H7-10	3	56.2	23.1	33.1
■	H7-14	2	54.4	24.5	29.9
▲	H7-15	2	48.7	22.5	26.2

PROJECT						
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650						
TITLE						
PLASTICITY CHART SILTY CLAY to CLAY						
 Golder Associates SUDBURY, ONTARIO		PROJECT No. 10-1191-0044		FILE No. 10-1191-0044SUD.GPJ		
		DRAWN	JJL	Jul 2013	SCALE	N/A
		CHECK	SEMC	Jul 2013	REV.	
		APPR	JMAC	Jul 2013		
FIGURE C9.4						

CONSOLIDATION TEST SUMMARY**FIGURE C10****Pg. 1 of 4****SAMPLE IDENTIFICATION**

Project Number: 10-1191-0044

Sample Number: 4

Borehole Number: BC4-1

Sample Depth, m: 3.4

TEST CONDITIONS

Test Type Standard

Load Duration, hr 24

Oedometer Number 2

Date Started Dec. 12/12

Date Completed Dec. 26/12

SAMPLE DIMENSIONS AND PROPERTIES - INITIALSample Height, cm 2.526 Unit Weight, kN/m³ 16.11Sample Diameter, cm 6.351 Dry Unit Weight, kN/m³ 9.37Area, cm² 31.68 Specific Gravity, Measured 2.74Volume, cm³ 80.02 Solids Height, cm 0.880Water Content, % 72.04 Volume of Solids, cm³ 27.87Wet Mass, g 131.47 Volume of Voids, cm³ 52.15

Dry Mass, g 76.42

TEST COMPUTATIONS

Pressure	Primary	Corr.		Average					Total
kPa	Consolidation	Height	Void	Height	t ₉₀	cv.	mv	k	Work
		cm	Ratio	cm	sec	cm ² /s	m ² /kN	cm/s	kJ/m3
0	0	2.526	1.871	2.526					
4	0.02	2.524	1.869	2.525	60	0.0225	1.96E-04	4.33E-07	0.002
13	0.03	2.521	1.865	2.522	73	0.0186	1.38E-04	2.51E-07	0.012
31	0.04	2.516	1.860	2.519	60	0.0224	9.78E-05	2.15E-07	0.051
66	0.09	2.508	1.850	2.512	86	0.0155	9.85E-05	1.49E-07	0.221
137	0.41	2.467	1.804	2.487	505	0.0026	2.29E-04	5.83E-08	1.867
277	2.64	2.203	1.504	2.335	1500	0.0008	7.44E-04	5.62E-08	24.007
558	1.13	2.090	1.375	2.146	694	0.0014	1.60E-04	2.20E-08	45.480
1117	0.88	2.002	1.275	2.046	406	0.0022	6.23E-05	1.33E-08	80.756
558	-0.08	2.010	1.285	2.006					
137	-0.29	2.039	1.318	2.024					
31	-0.36	2.075	1.358	2.057					
4	-0.31	2.105	1.393	2.090					

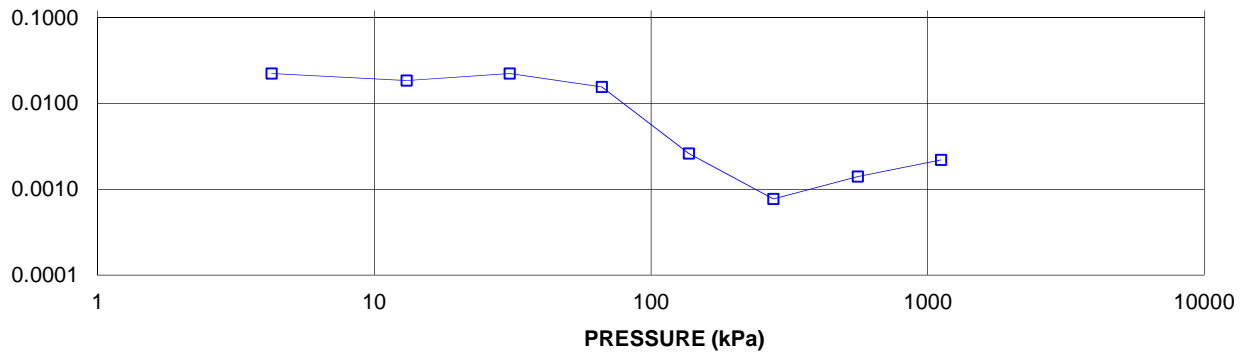
Note:

k calculated using α based on t₉₀ values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**Sample Height, cm 2.105 Unit Weight, kN/m³ 15.89Sample Diameter, cm 6.35 Dry Unit Weight, kN/m³ 11.24Area, cm² 31.68 Specific Gravity, Measured 2.74Volume, cm³ 66.70 Solids Height, cm 0.880Water Content, % 41.39 Volume of Solids, cm³ 27.87Wet Mass, g 108.05 Volume of Voids, cm³ 38.83

Dry Mass, g 76.42

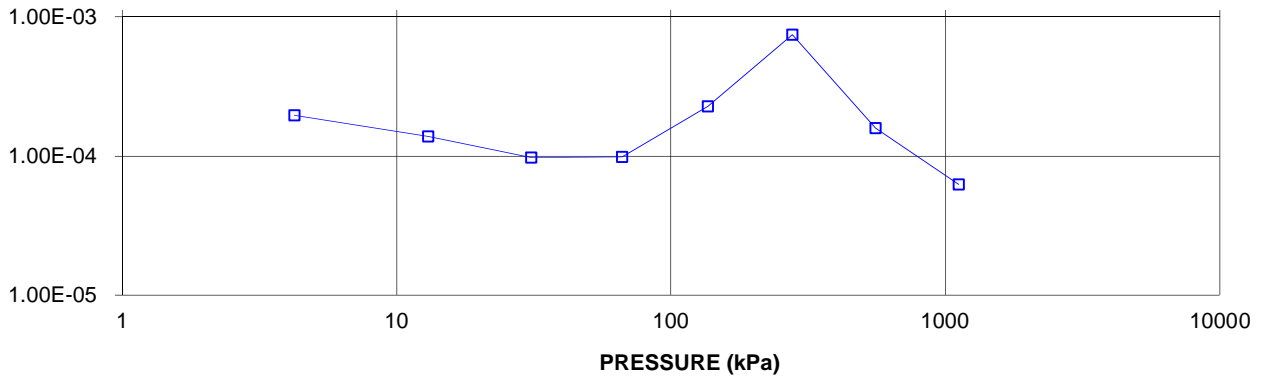
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH BC4-1 Sa 4



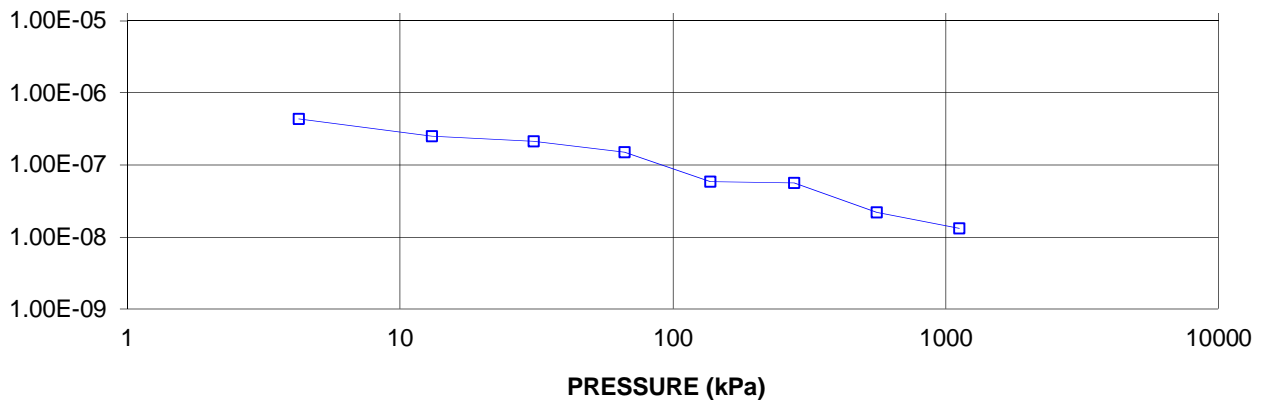
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH BC4-1 Sa 4



HYDRAULIC CONDUCTIVITY,
cm/s

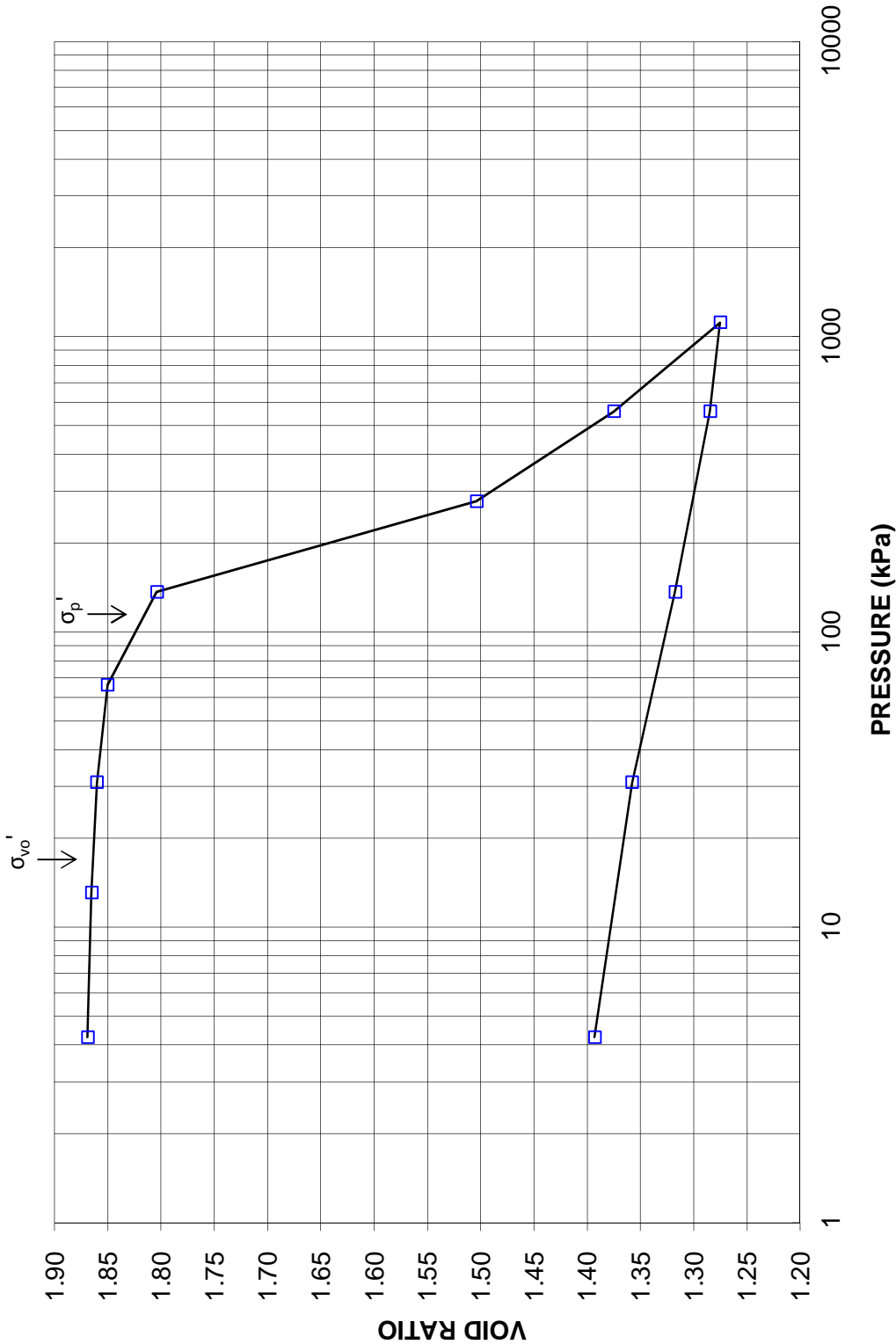
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH BC4-1 Sa 4



CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE

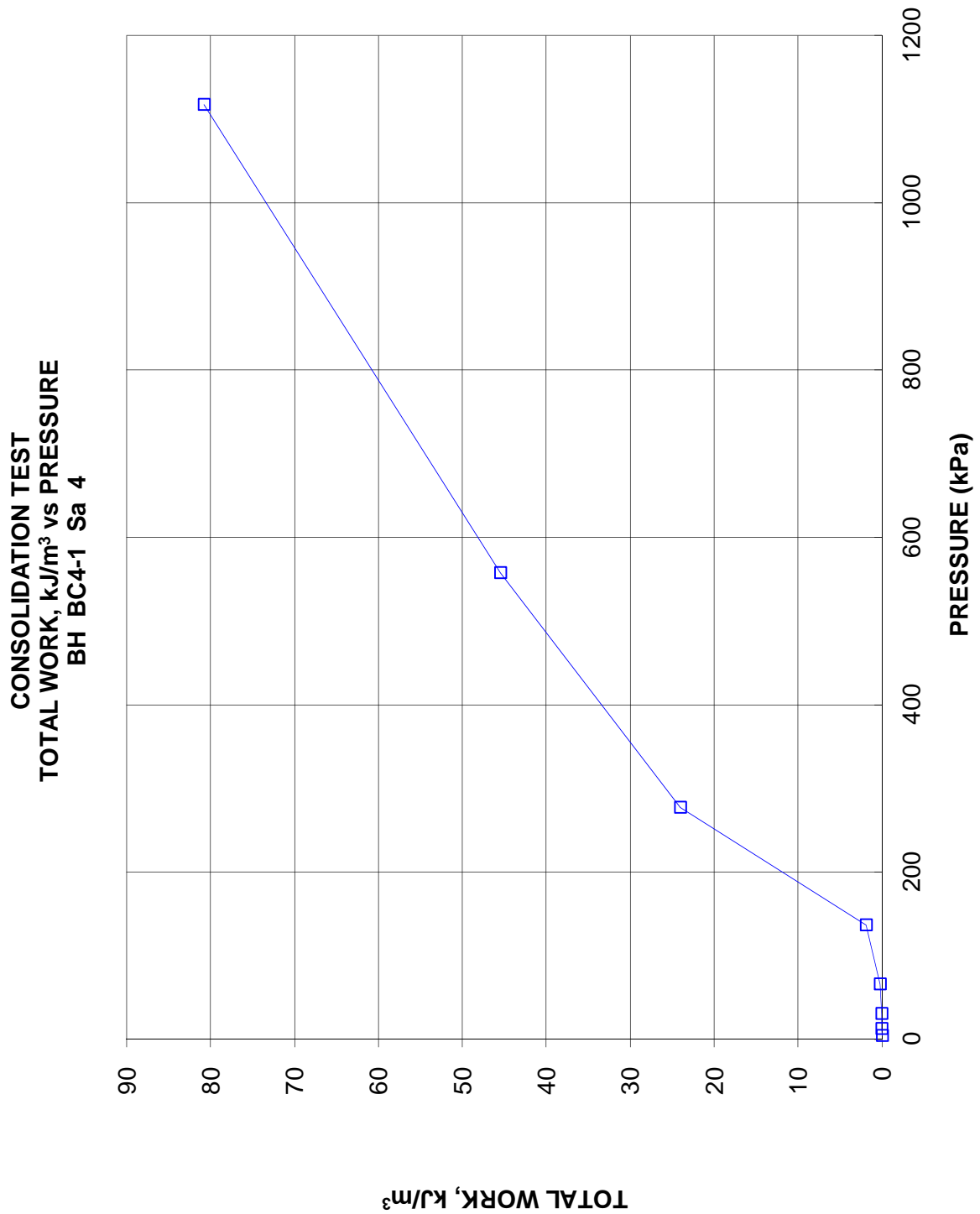
FIGURE C10
Pg. 3 of 4

CONSOLIDATION TEST
VOID RATIO VS PRESSURE
BH BC4-1 Sa 4



**CONSOLIDATION TEST
TOTAL WORK VS PRESSURE**

FIGURE C10
Pg. 4 of 4



CONSOLIDATION TEST SUMMARY**FIGURE C11****Pg. 1 of 4****SAMPLE IDENTIFICATION**

Project Number:	10-1191-0044	Sample Number:	5
Borehole Number:	BC4-1	Sample Depth, m:	4.9

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	Nov. 28/12		
Date Completed	Dec. 11/12		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.526	Unit Weight, kN/m ³	16.24
Sample Diameter, cm	6.351	Dry Unit Weight, kN/m ³	9.81
Area, cm ²	31.68	Specific Gravity, Measured	2.74
Volume, cm ³	80.02	Solids Height, cm	0.923
Water Content, %	65.50	Volume of Solids, cm ³	29.25
Wet Mass, g	132.48	Volume of Voids, cm ³	50.77
Dry Mass, g	80.05	Degree of Saturation, %	103.3

TEST COMPUTATIONS

Pressure kPa	Primary Consolidation	Corr. Height cm	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	2.526	1.736	2.526					
4	0.02	2.524	1.734	2.525	86	0.0156	2.05E-04	3.15E-07	0.002
13	0.02	2.521	1.731	2.523	60	0.0225	1.08E-04	2.39E-07	0.010
31	0.05	2.516	1.725	2.519	60	0.0224	1.17E-04	2.58E-07	0.056
66	0.11	2.505	1.714	2.511	86	0.0155	1.19E-04	1.81E-07	0.262
137	1.20	2.386	1.584	2.445	1622	0.0008	6.75E-04	5.17E-08	5.116
277	1.80	2.205	1.388	2.295	1109	0.0010	5.08E-04	5.01E-08	20.762
558	0.89	2.116	1.292	2.161	375	0.0026	1.26E-04	3.25E-08	37.632
1117	1.41	1.975	1.139	2.046	194	0.0046	9.96E-05	4.46E-08	93.368
558	-0.15	1.991	1.156	1.983					
137	-0.29	2.020	1.188	2.005					
31	-0.31	2.050	1.221	2.035					
4	-0.27	2.078	1.250	2.064					

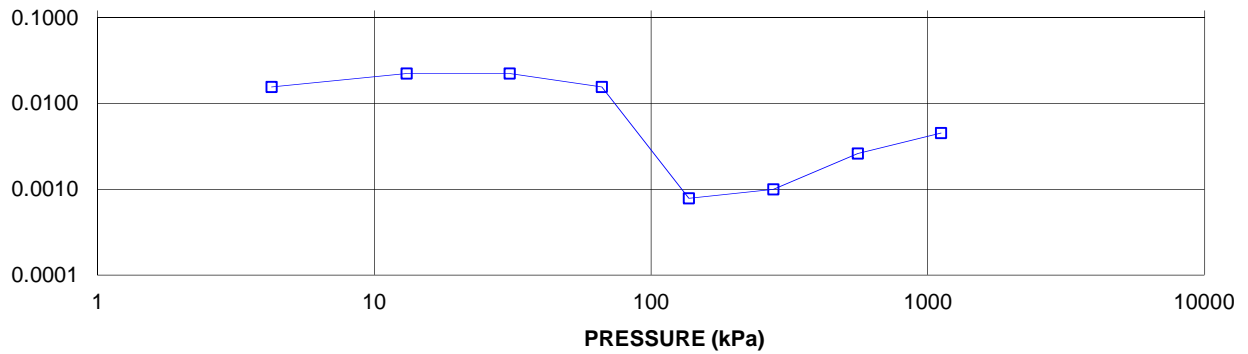
Note:

k calculated using α based on t₉₀ values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	2.078	Unit Weight, kN/m ³	16.38
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	11.93
Area, cm ²	31.68	Specific Gravity, Measured	2.74
Volume, cm ³	65.82	Solids Height, cm	0.923
Water Content, %	37.31	Volume of Solids, cm ³	29.25
Wet Mass, g	109.92	Volume of Voids, cm ³	36.57
Dry Mass, g	80.05		

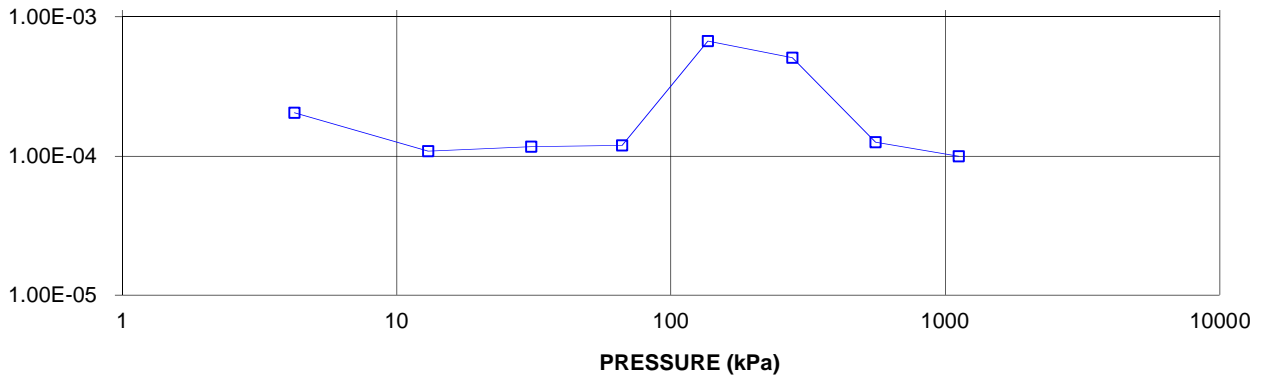
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH BC4-1 Sa 5



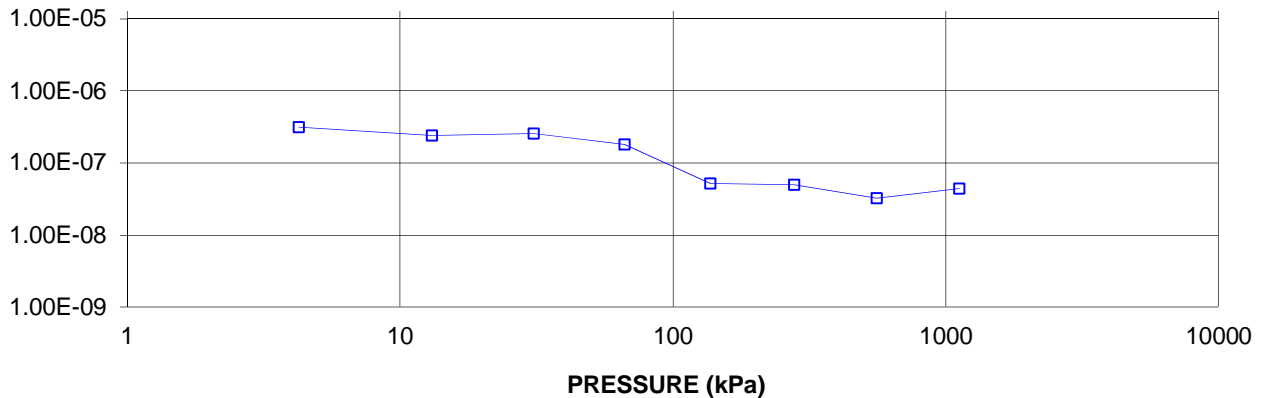
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH BC4-1 Sa 5



HYDRAULIC CONDUCTIVITY,
cm/s

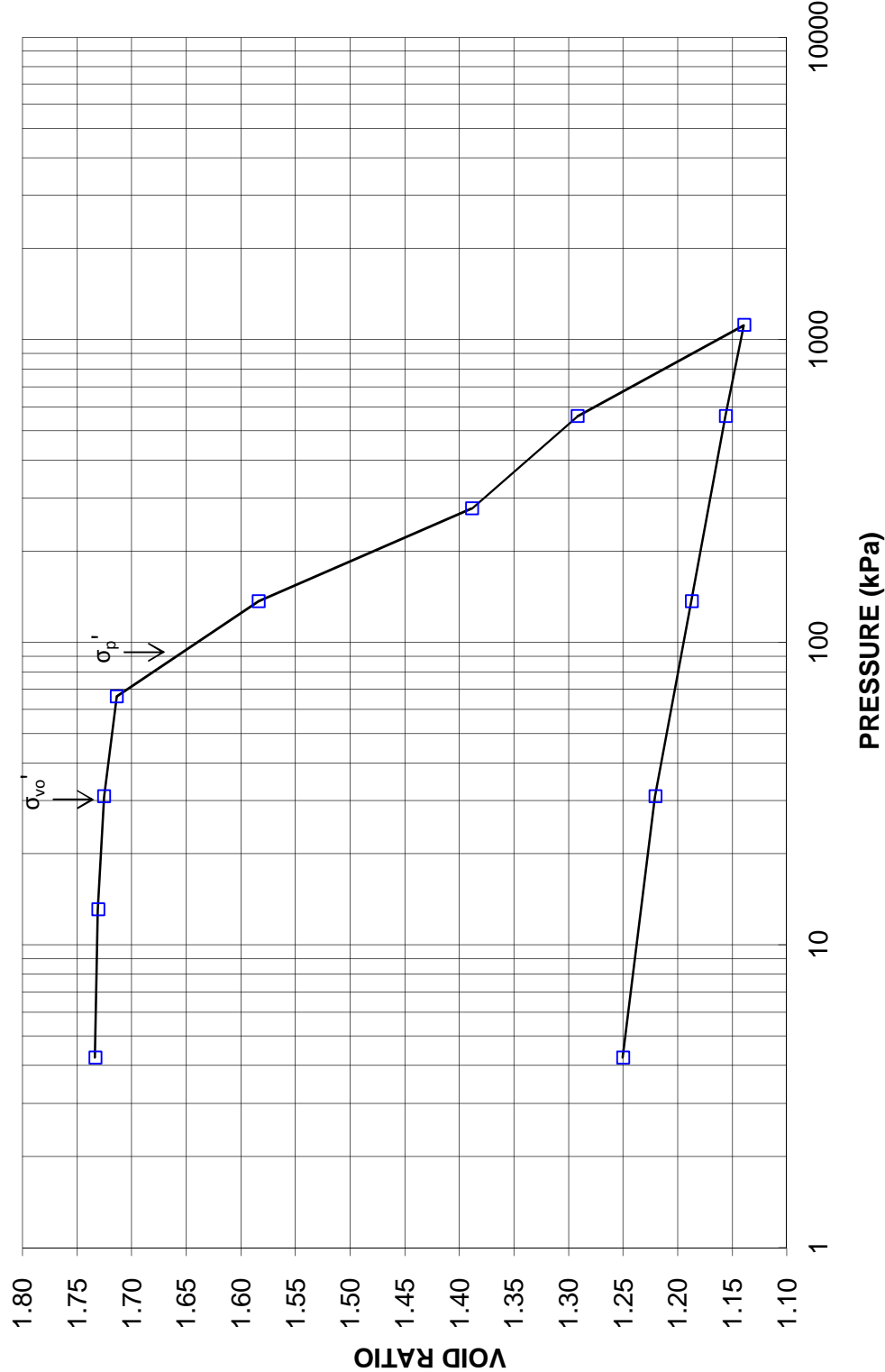
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH BC4-1 Sa 5



CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE

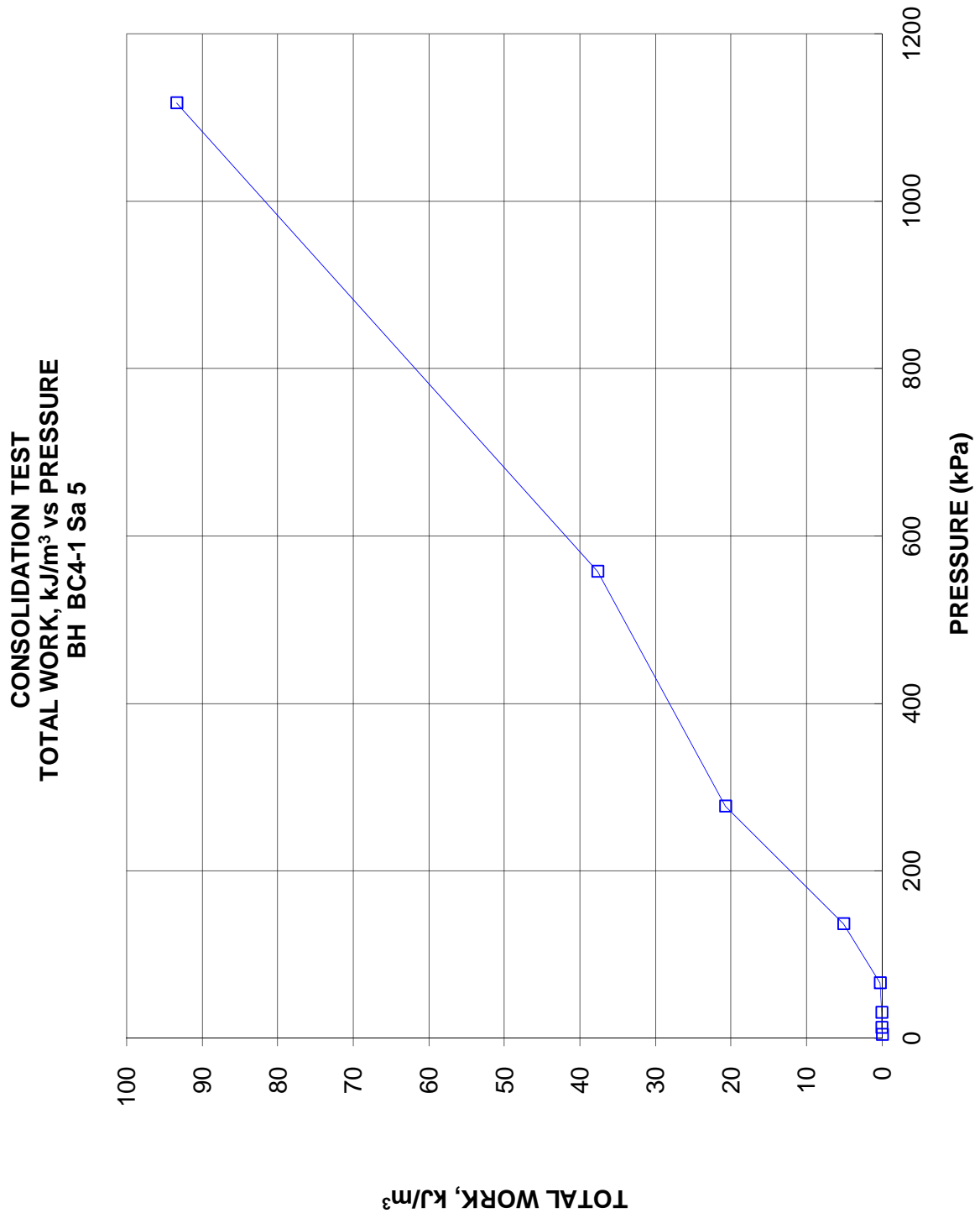
FIGURE C11
Pg. 3 of 4

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH BC4-1Sa 5



**CONSOLIDATION TEST
TOTAL WORK VS PRESSURE**

FIGURE C11
Pg. 4 of 4



CONSOLIDATION TEST SUMMARY**FIGURE C12****Pg. 1 of 4****SAMPLE IDENTIFICATION**

Project Number	10-1191-0044	Sample Number	6
Borehole Number	BC4-1	Sample Depth, m	6.4

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	1		
Date Started	Dec. 12/12		
Date Completed	Dec. 26/12		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.544	Unit Weight, kN/m ³	16.63
Sample Diameter, cm	6.353	Dry Unit Weight, kN/m ³	10.32
Area, cm ²	31.70	Specific Gravity, measure	2.76
Volume, cm ³	80.64	Solids Height, cm	0.971
Water Content, %	61.04	Volume of Solids, cm ³	30.78
Wet Mass, g	136.71	Volume of Voids, cm ³	49.86
Dry Mass, g	84.89		

TEST COMPUTATIONS

Pressure kPa	Primary Consolidation	Corr. Height cm	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	0.00	2.544	1.620	2.544					
9	0.06	2.538	1.614	2.541	118	0.01164	2.42E-04	2.76E-07	0.010
18	0.04	2.535	1.611	2.536	101	0.01345	1.55E-04	2.62E-07	0.032
35	0.06	2.528	1.604	2.532	60	0.02265	1.43E-04	3.18E-07	0.097
69	0.10	2.519	1.594	2.524	60	0.02250	1.14E-04	2.52E-07	0.301
143	0.35	2.483	1.558	2.501	317	0.00418	1.88E-04	7.71E-08	1.781
285	2.53	2.230	1.297	2.357	960	0.00123	6.99E-04	8.40E-08	23.559
570	1.10	2.120	1.184	2.175	614	0.00163	1.51E-04	2.42E-08	44.653
1140	0.72	2.049	1.110	2.085	290	0.00317	4.94E-05	1.54E-08	73.486
570	-0.06	2.055	1.116	2.052					
143	-0.24	2.079	1.141	2.067					
35	-0.30	2.109	1.172	2.094					
9	-0.26	2.135	1.199	2.122					

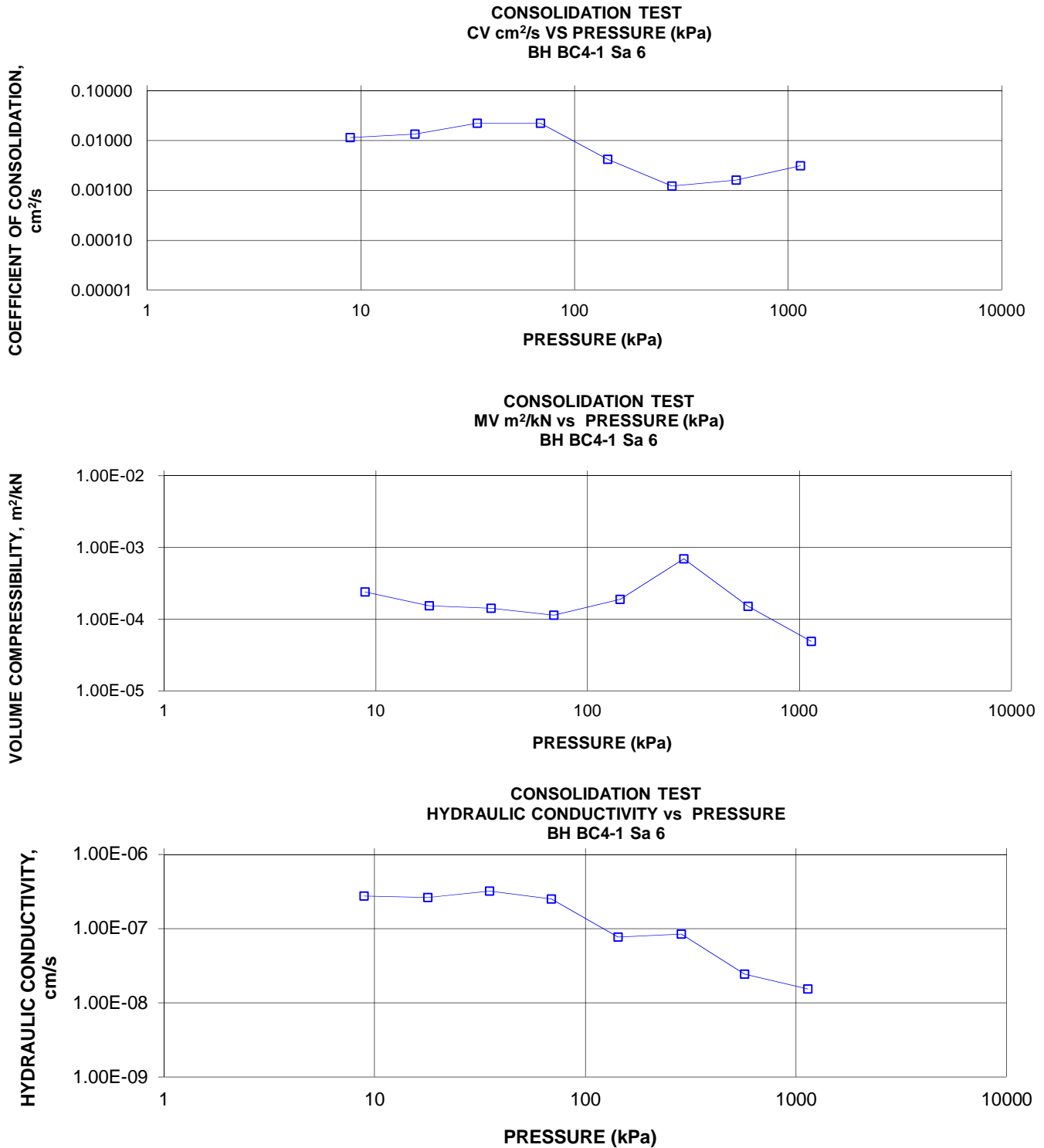
Note:

k calculated using α based on t_{90} values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	2.135	Unit Weight, kN/m ³	16.96
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	12.30
Area, cm ²	31.70	Specific Gravity, measure	2.76
Volume, cm ³	67.69	Solids Height, cm	0.971
Water Content, %	37.94	Volume of Solids, cm ³	30.78
Wet Mass, g	117.10	Volume of Voids, cm ³	36.91
Dry Mass, g	84.89		

CONSOLIDATION TEST SUMMARY

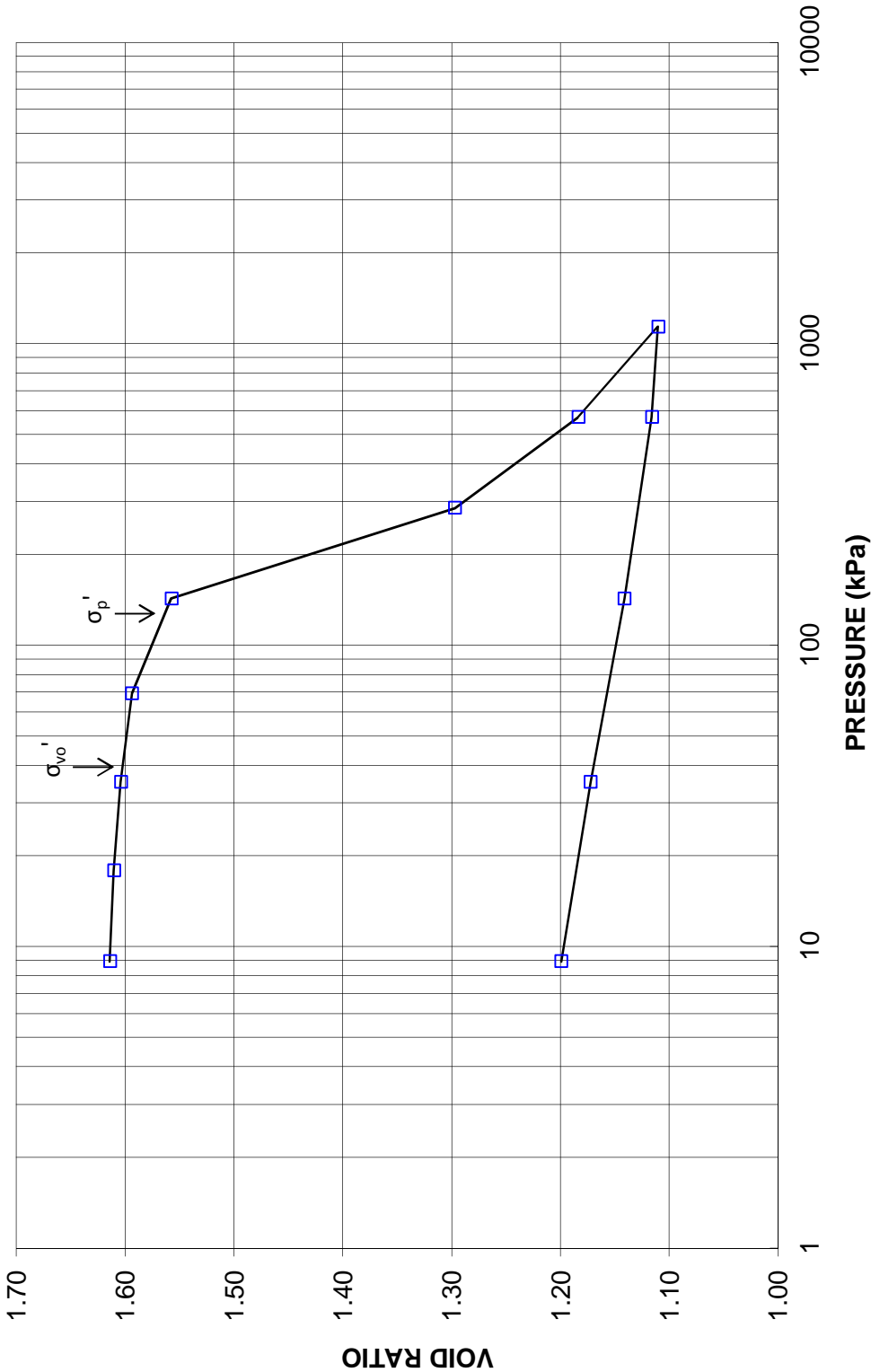
FIGURE C12
Pg. 2 of 4



CONSOLIDATION TEST VOID RATIO VS LOG PRESSURE

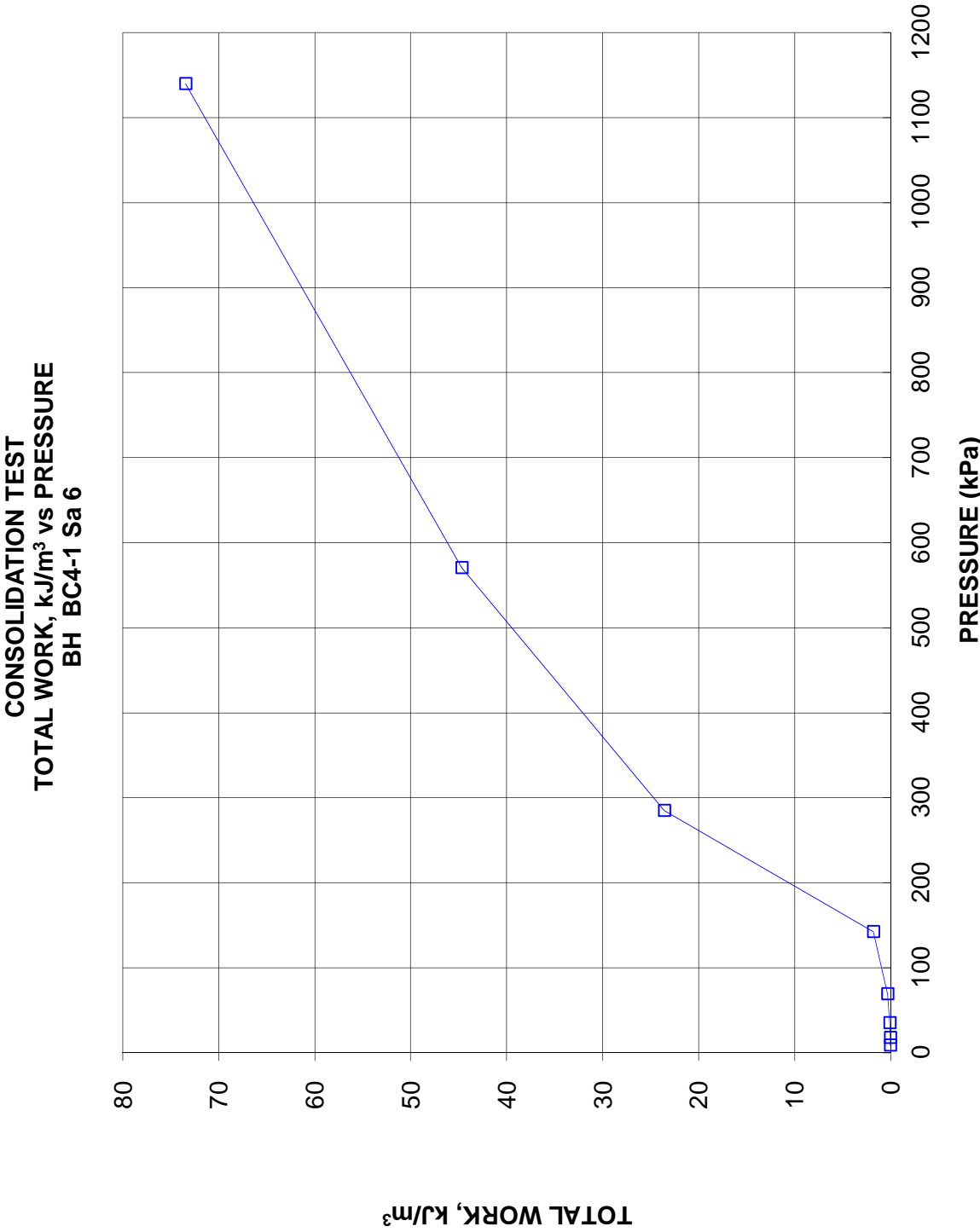
FIGURE C12
Pg. 3 of 4

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH BC4-1 Sa 6



**CONSOLIDATION TEST
TOTAL WORK VS PRESSURE**

FIGURE C12
Pg. 4 of 4



CONSOLIDATION TEST SUMMARY**FIGURE C13**

Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Number	10-1191-0044	Sample Number	2
Borehole Number	H6-S1 Horizontally Trimmed	Sample Depth, m	7.80-7.87

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	4		
Date Started	5/30/2013		
Date Completed	6/19/2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.53	Unit Weight, kN/m ³	15.96
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	9.58
Area, cm ²	31.71	Specific Gravity, measured	2.73
Volume, cm ³	80.22	Solids Height, cm	0.905
Water Content, %	66.59	Volume of Solids, cm ³	28.71
Wet Mass, g	130.56	Volume of Voids, cm ³	51.52
Dry Mass, g	78.37	Degree of Saturation, %	101.3

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	2.530	1.795	2.530				
6.24	2.528	1.793	2.529	2	6.78E-01	1.14E-04	7.58E-06
10.88	2.522	1.785	2.525	173	7.81E-03	5.71E-04	4.37E-07
20.77	2.510	1.772	2.516	154	8.71E-03	4.68E-04	3.99E-07
35.10	2.483	1.743	2.496	577	2.29E-03	7.42E-04	1.66E-07
20.77	2.486	1.746	2.484				
6.24	2.494	1.755	2.490				
20.77	2.487	1.747	2.491	36	3.65E-02	2.04E-04	7.30E-07
35.10	2.479	1.738	2.483	135	9.68E-03	2.18E-04	2.07E-07
78.79	2.318	1.561	2.399	694	1.76E-03	1.45E-03	2.51E-07
155.94	2.025	1.236	2.171	1500	6.66E-04	1.50E-03	9.82E-08
310.41	1.863	1.057	1.944	735	1.09E-03	4.15E-04	4.43E-08
619.20	1.738	0.920	1.800	437	1.57E-03	1.59E-04	2.46E-08
1237.22	1.631	0.801	1.684	173	3.48E-03	6.88E-05	2.34E-08
2472.94	1.537	0.698	1.584	194	2.74E-03		8.00E-09
619.20	1.556	0.719	1.547				
155.94	1.588	0.754	1.572				
35.10	1.624	0.794	1.606				
20.77	1.634	0.805	1.629				
6.24	1.652	0.825	1.643				

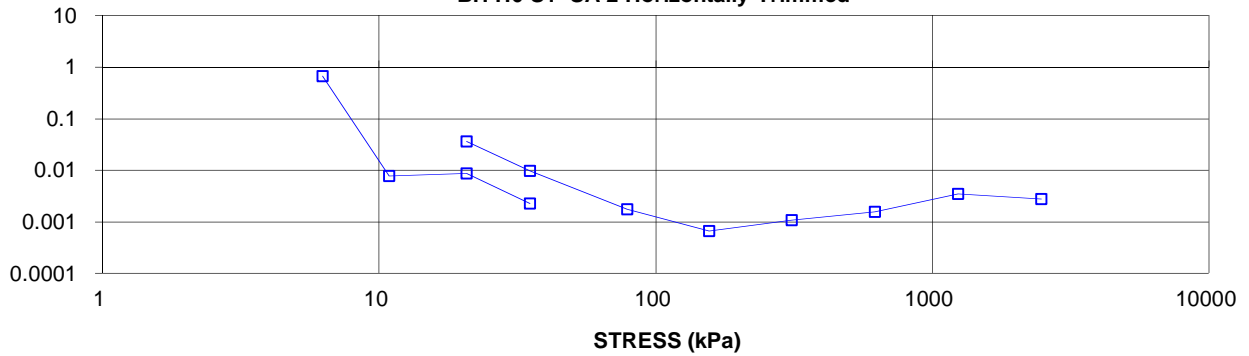
Specimen taken 21 to 28cm from bottom of the tube.
k calculated using cv based on $\bar{\epsilon}_0$ values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.65	Unit Weight, kN/m ³	19.42
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	14.67
Area, cm ²	31.71	Specific Gravity, measured	2.73
Volume, cm ³	52.39	Solids Height, cm	0.905
Water Content, %	32.37	Volume of Solids, cm ³	28.71
Wet Mass, g	103.74	Volume of Voids, cm ³	23.69
Dry Mass, g	78.37		

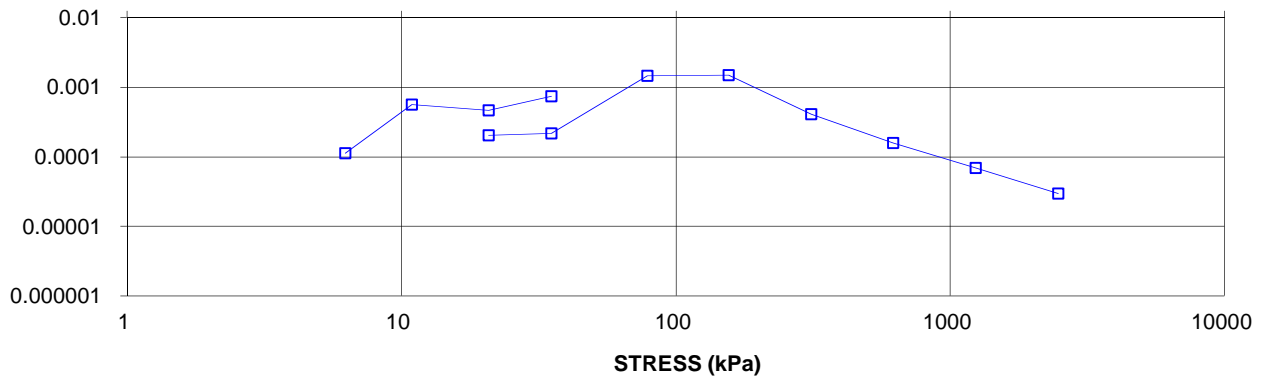
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS STRESS (kPa)
BH H6-S1 SA 2 Horizontally Trimmed



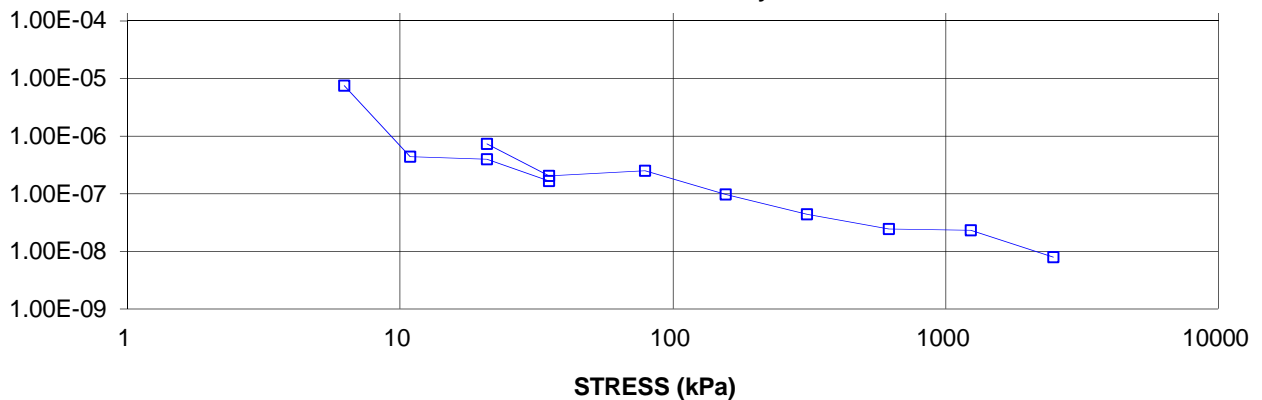
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs STRESS (kPa)
BH H6-S1 SA 2 Horizontally Trimmed



HYDRAULIC CONDUCTIVITY,
cm/s

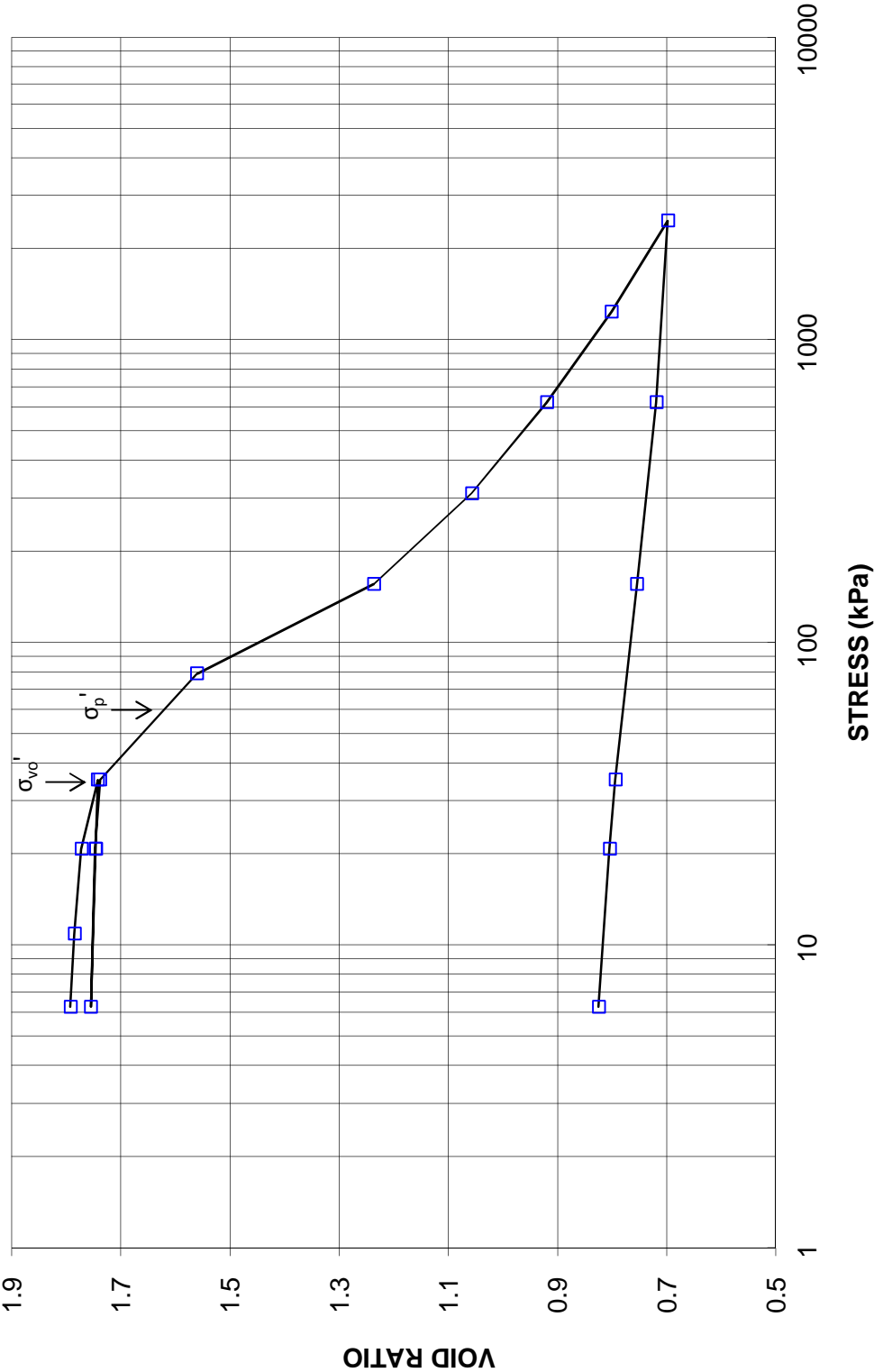
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs STRESS
BH H6-S1 SA 2 Horizontally Trimmed



CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE C13
Pg. 3 of 4

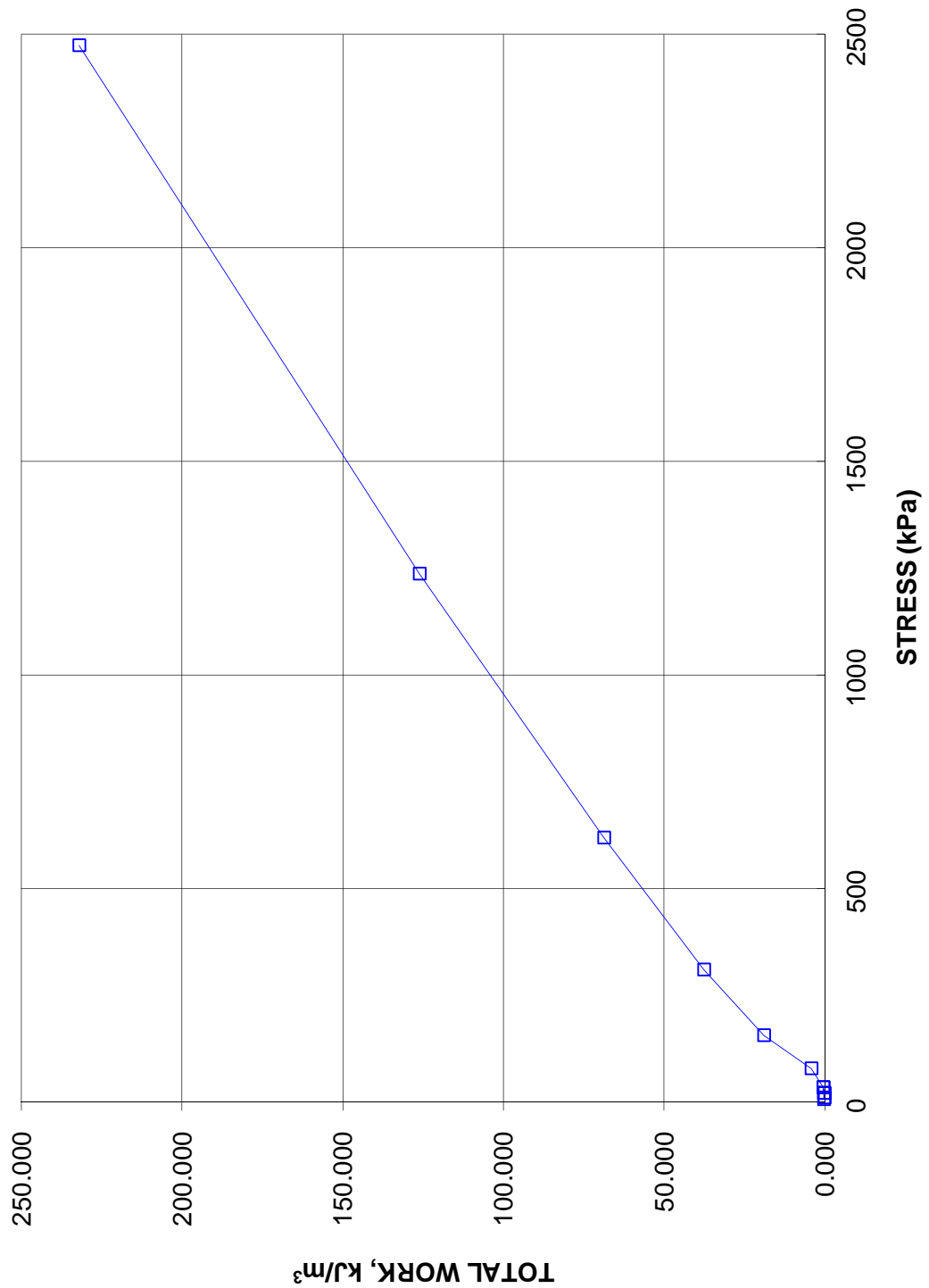
CONSOLIDATION TEST
VOID RATIO vs STRESS
BH H6-S1 SA 2 Horizontally Trimmed



**CONSOLIDATION TEST
TOTAL WORK VS STRESS**

FIGURE C13
Pg. 4 of 4

**CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs STRESS
BH H6-S1 SA 2 Horizontally Trimmed**



CONSOLIDATION TEST SUMMARY**FIGURE C14**

Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Number	10-1191-0044	Sample Number	4
Borehole Number	H6-S1	Sample Depth, m	14.11-14.20

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	3		
Date Started	5/30/2013		
Date Completed	6/20/2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.53	Unit Weight, kN/m ³	16.36
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	10.33
Area, cm ²	31.68	Specific Gravity, measured	2.75
Volume, cm ³	80.28	Solids Height, cm	0.970
Water Content, %	58.41	Volume of Solids, cm ³	30.73
Wet Mass, g	133.89	Volume of Voids, cm ³	49.54
Dry Mass, g	84.52	Degree of Saturation, %	99.7

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	2.534	1.612	2.534				
5.87	2.530	1.608	2.532	15	9.06E-02	2.62E-04	2.33E-06
10.71	2.527	1.604	2.528	296	4.58E-03	2.69E-04	1.21E-07
20.45	2.519	1.596	2.523	254	5.31E-03	3.28E-04	1.71E-07
39.85	2.499	1.576	2.509	178	7.50E-03	3.93E-04	2.88E-07
73.33	2.467	1.542	2.483	317	4.12E-03	3.88E-04	1.57E-07
20.45	2.477	1.553	2.472				
5.87	2.488	1.565	2.483				
20.45	2.481	1.557	2.485	90	1.45E-02	2.06E-04	2.93E-07
39.85	2.474	1.550	2.478	147	8.85E-03	1.28E-04	1.11E-07
78.65	2.459	1.534	2.466	167	7.72E-03	1.62E-04	1.22E-07
155.71	2.242	1.310	2.350	2076	5.64E-04	1.11E-03	6.14E-08
310.28	2.023	1.085	2.132	778	1.24E-03	5.58E-04	6.78E-08
618.29	1.889	0.947	1.956	359	2.26E-03	1.71E-04	3.80E-08
1236.56	1.784	0.839	1.837	265	2.70E-03	6.71E-05	1.78E-08
2473.38	1.691	0.743	1.737	184	3.48E-03	2.97E-05	1.01E-08
1236.56	1.696	0.748	1.693				
310.28	1.726	0.779	1.711				
78.65	1.764	0.818	1.745				
20.45	1.795	0.850	1.779				
5.87	1.813	0.869	1.804				

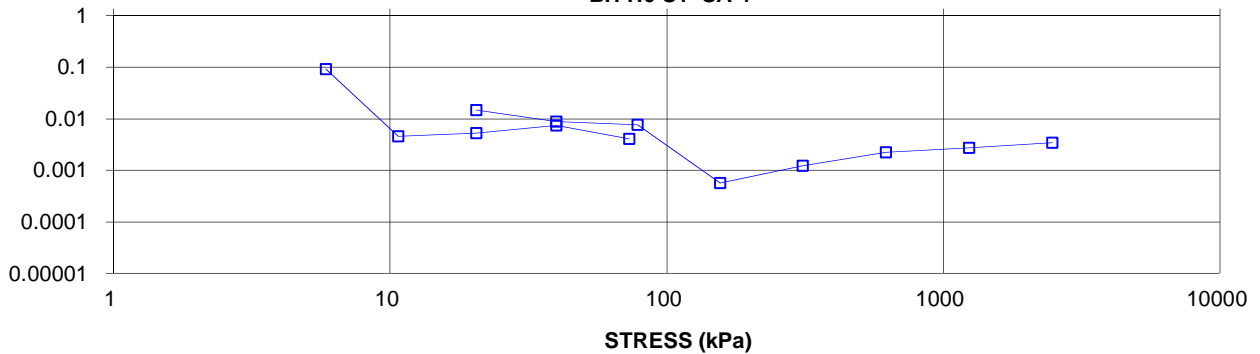
Note:

k calculated using cv based on t_{90} values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	1.69	Unit Weight, kN/m ³	20.48
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	15.47
Area, cm ²	31.68	Specific Gravity, measured	2.75
Volume, cm ³	53.57	Solids Height, cm	0.970
Water Content, %	32.32	Volume of Solids, cm ³	30.73
Wet Mass, g	111.84	Volume of Voids, cm ³	22.83
Dry Mass, g	84.52		

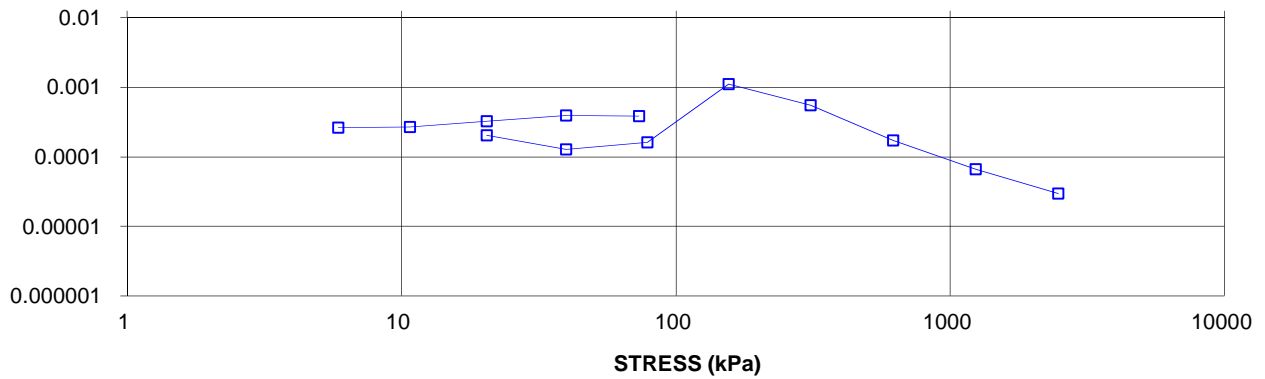
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS STRESS (kPa)
BH H6-S1 SA 4



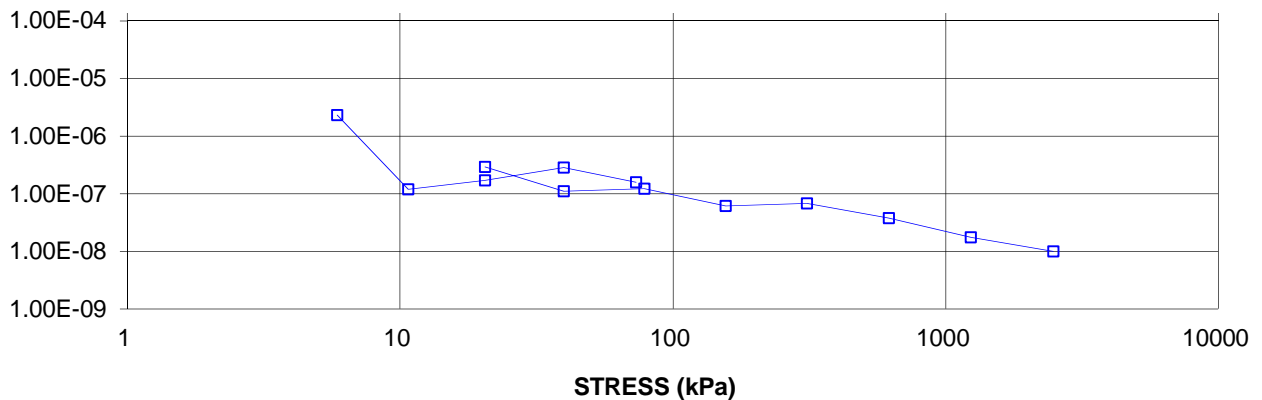
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs STRESS (kPa)
BH H6-S1 SA 4



HYDRAULIC CONDUCTIVITY,
cm/s

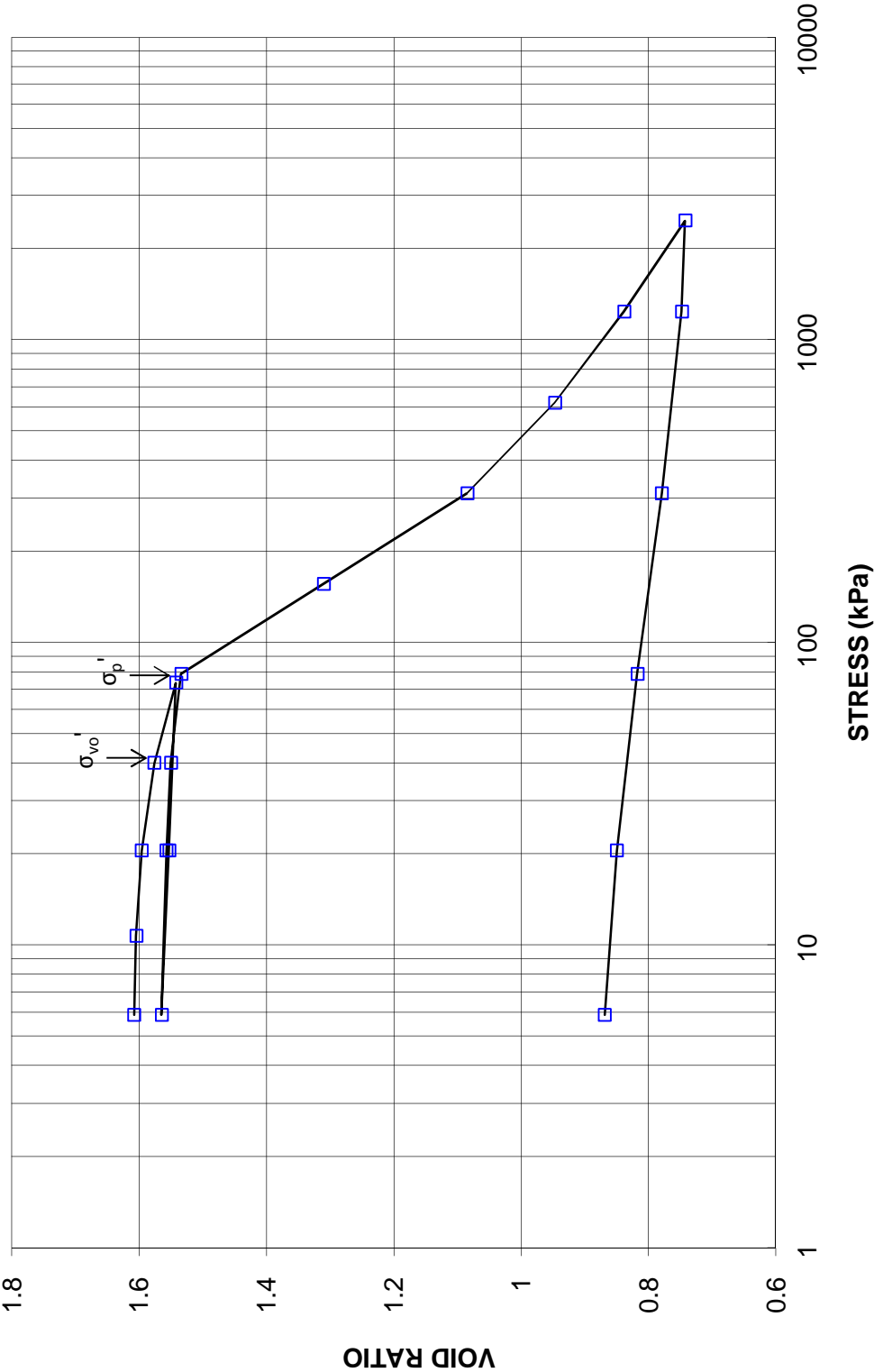
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs STRESS
BH H6-S1 SA 4



CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE C14
Pg. 3 of 4

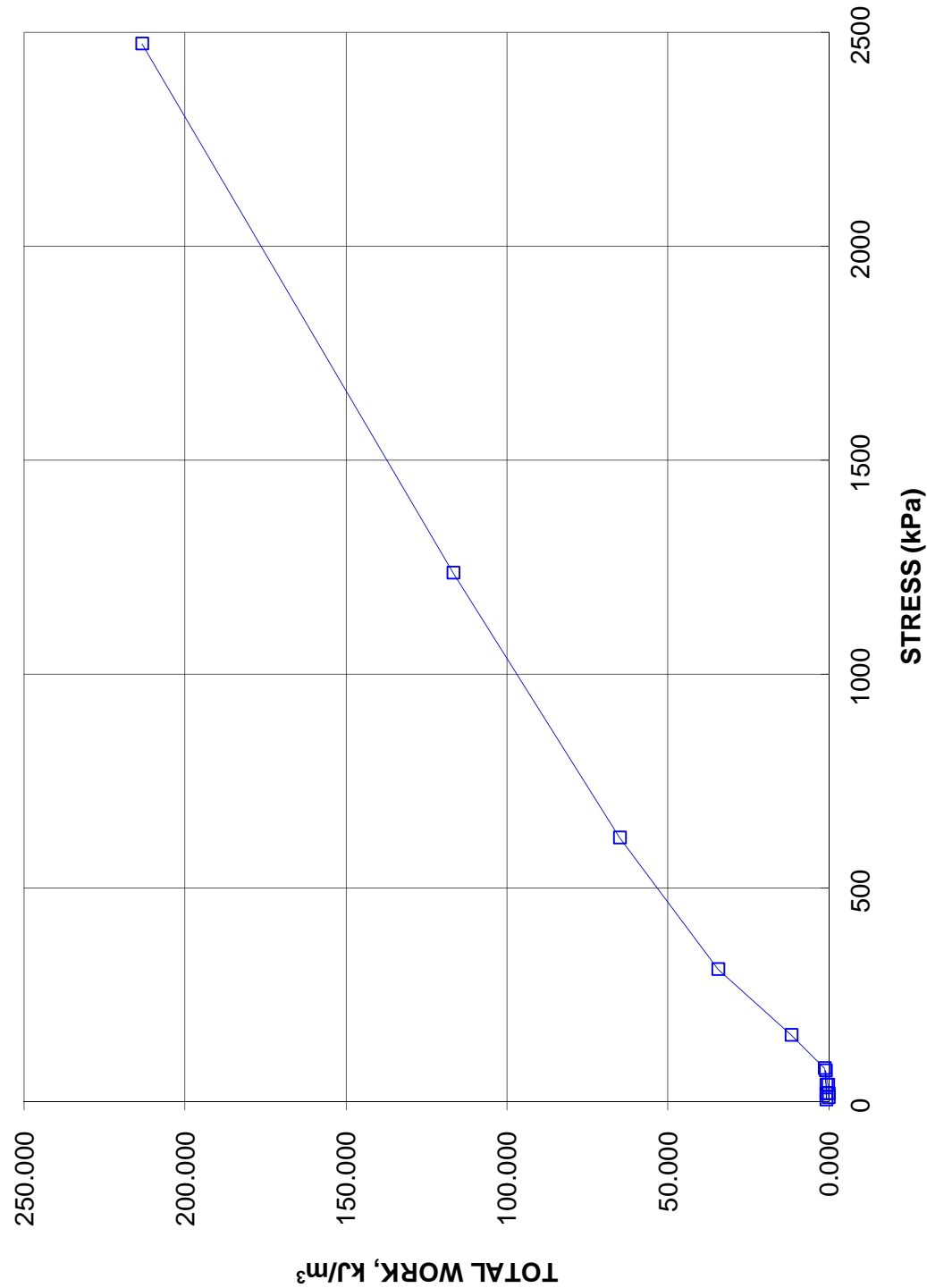
CONSOLIDATION TEST
VOID RATIO vs STRESS
BH H6-S1 SA 4



CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE C14
Pg. 4 of 4

CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs STRESS
BH H6-S1 SA 4



CONSOLIDATION TEST SUMMARY**FIGURE C15**

Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Number	10-1191-0044	Sample Number	2
Borehole Number	H6-S1 Vertically Trimmed	Sample Depth, m	7.87-7.97

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	5/30/2013		
Date Completed	6/17/2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m ³	16.14
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	9.91
Area, cm ²	31.65	Specific Gravity, measured	2.73
Volume, cm ³	60.23	Solids Height, cm	0.704
Water Content, %	62.89	Volume of Solids, cm ³	22.30
Wet Mass, g	99.15	Volume of Voids, cm ³	37.93
Dry Mass, g	60.87	Degree of Saturation, %	100.9

TEST COMPUTATIONS

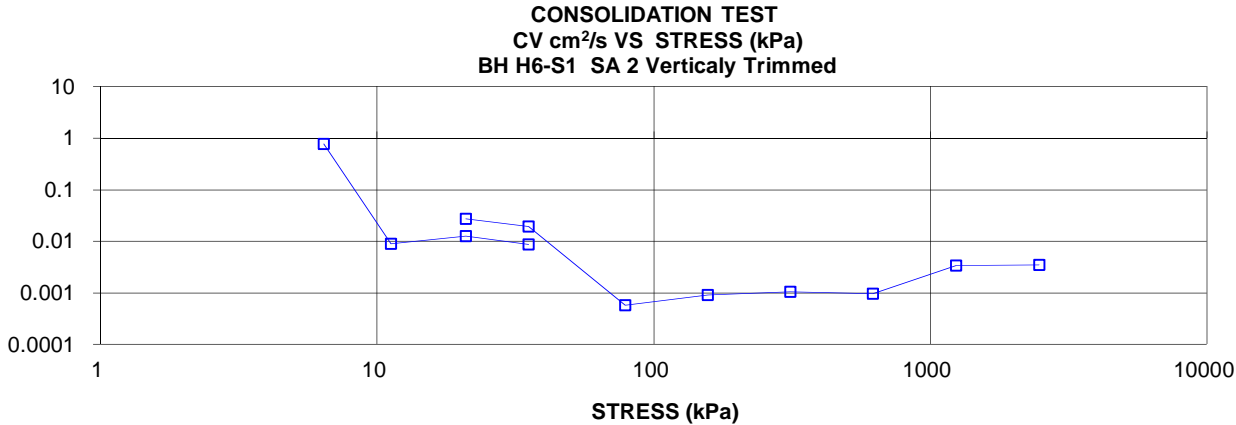
Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	1.903	1.701	1.903				
6.41	1.903	1.701	1.903	1	7.68E-01	0.00E+00	0.00E+00
11.21	1.901	1.698	1.902	86	8.92E-03	2.41E-04	2.10E-07
20.92	1.893	1.687	1.897	60	1.27E-02	4.28E-04	5.33E-07
35.21	1.875	1.662	1.884	86	8.75E-03	6.51E-04	5.58E-07
20.92	1.883	1.673	1.879				
6.41	1.893	1.687	1.888				
20.92	1.880	1.668	1.886	28	2.69E-02	4.71E-04	1.24E-06
35.32	1.872	1.658	1.876	38	1.96E-02	2.81E-04	5.41E-07
79.08	1.723	1.446	1.798	1188	5.77E-04	1.79E-03	1.01E-07
156.43	1.566	1.223	1.644	634	9.04E-04	1.07E-03	9.47E-08
311.13	1.443	1.048	1.504	452	1.06E-03	4.17E-04	4.34E-08
620.52	1.349	0.915	1.396	427	9.68E-04	1.59E-04	1.51E-08
1238.08	1.272	0.805	1.311	109	3.34E-03	6.58E-05	2.15E-08
2476.05	1.203	0.707	1.237	94	3.45E-03	2.93E-05	9.93E-09
620.52	1.219	0.730	1.211				
156.43	1.242	0.764	1.231				
35.21	1.266	0.797	1.254				
20.92	1.272	0.806	1.269				
6.41	1.288	0.828	1.280				

Specimen taken 11 to 21cm from bottom of the tube.

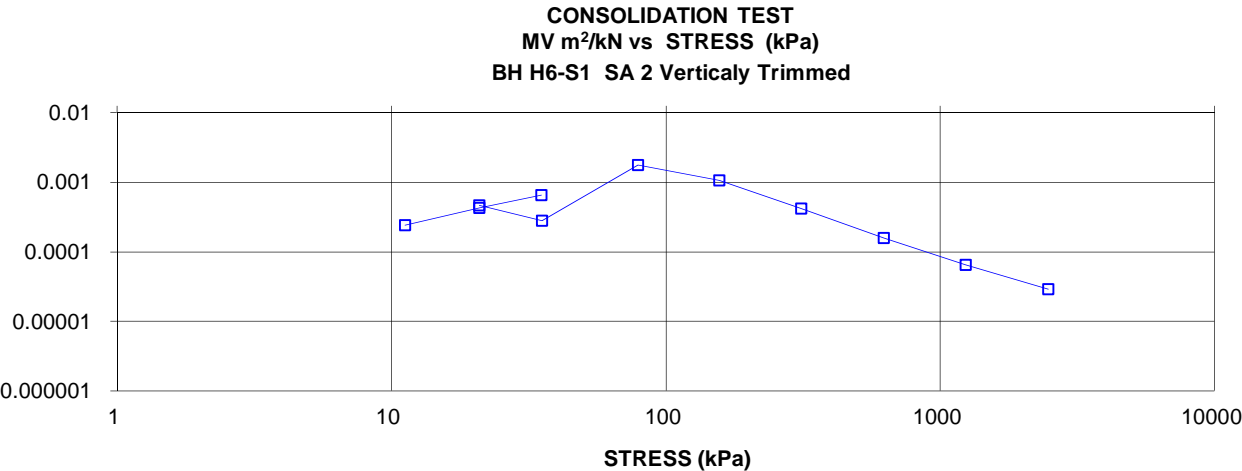
k calculated using cv based on t₉₀ values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	1.29	Unit Weight, kN/m ³	19.20
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	14.64
Area, cm ²	31.65	Specific Gravity, measured	2.73
Volume, cm ³	40.76	Solids Height, cm	0.704
Water Content, %	31.13	Volume of Solids, cm ³	22.30
Wet Mass, g	79.82	Volume of Voids, cm ³	18.46
Dry Mass, g	60.87		

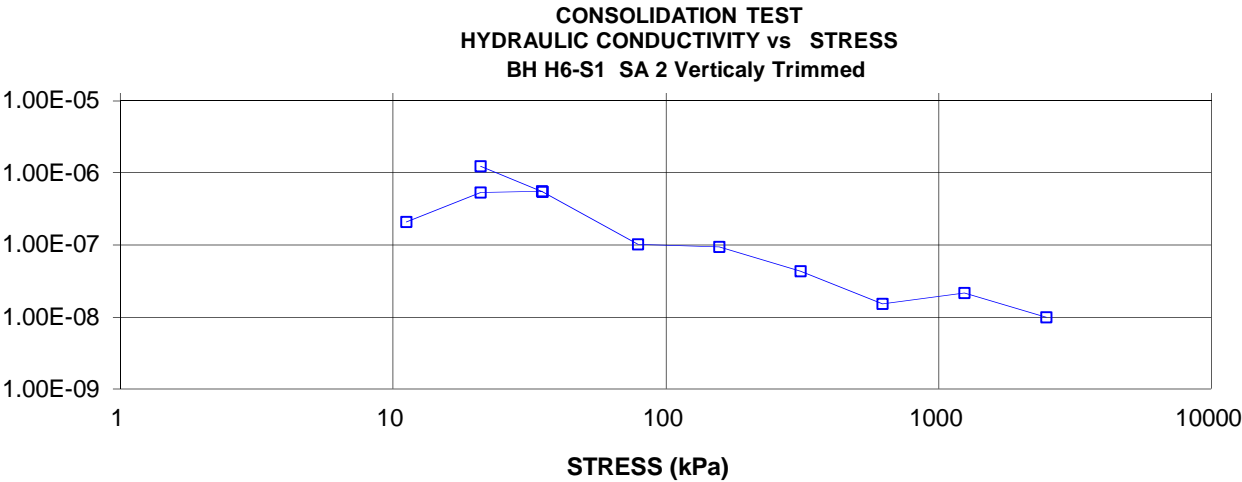
COEFFICIENT OF CONSOLIDATION,
cm²/s



VOLUME COMPRESSIBILITY, m²/kN

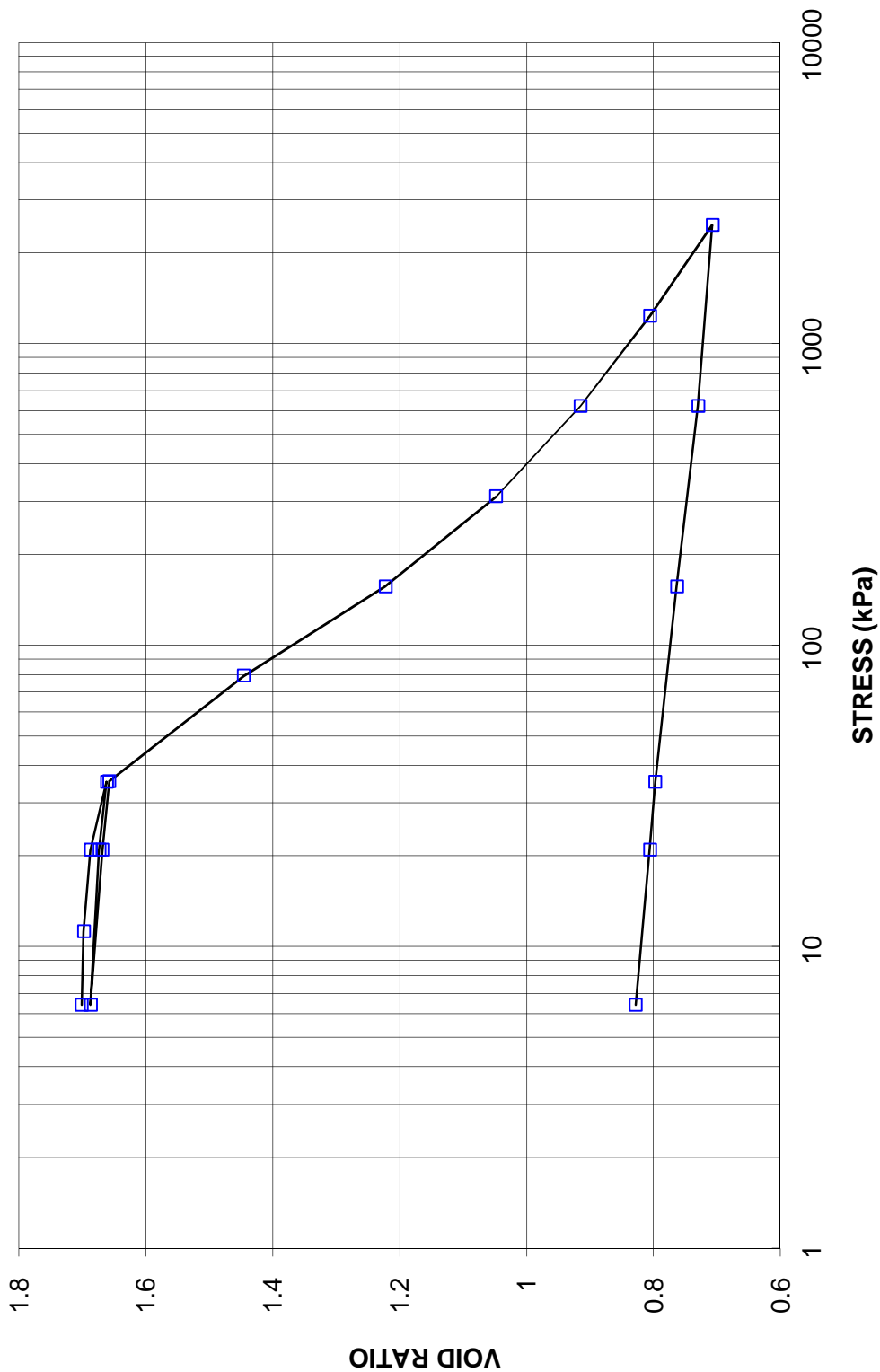


HYDRAULIC CONDUCTIVITY,
cm/s



CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

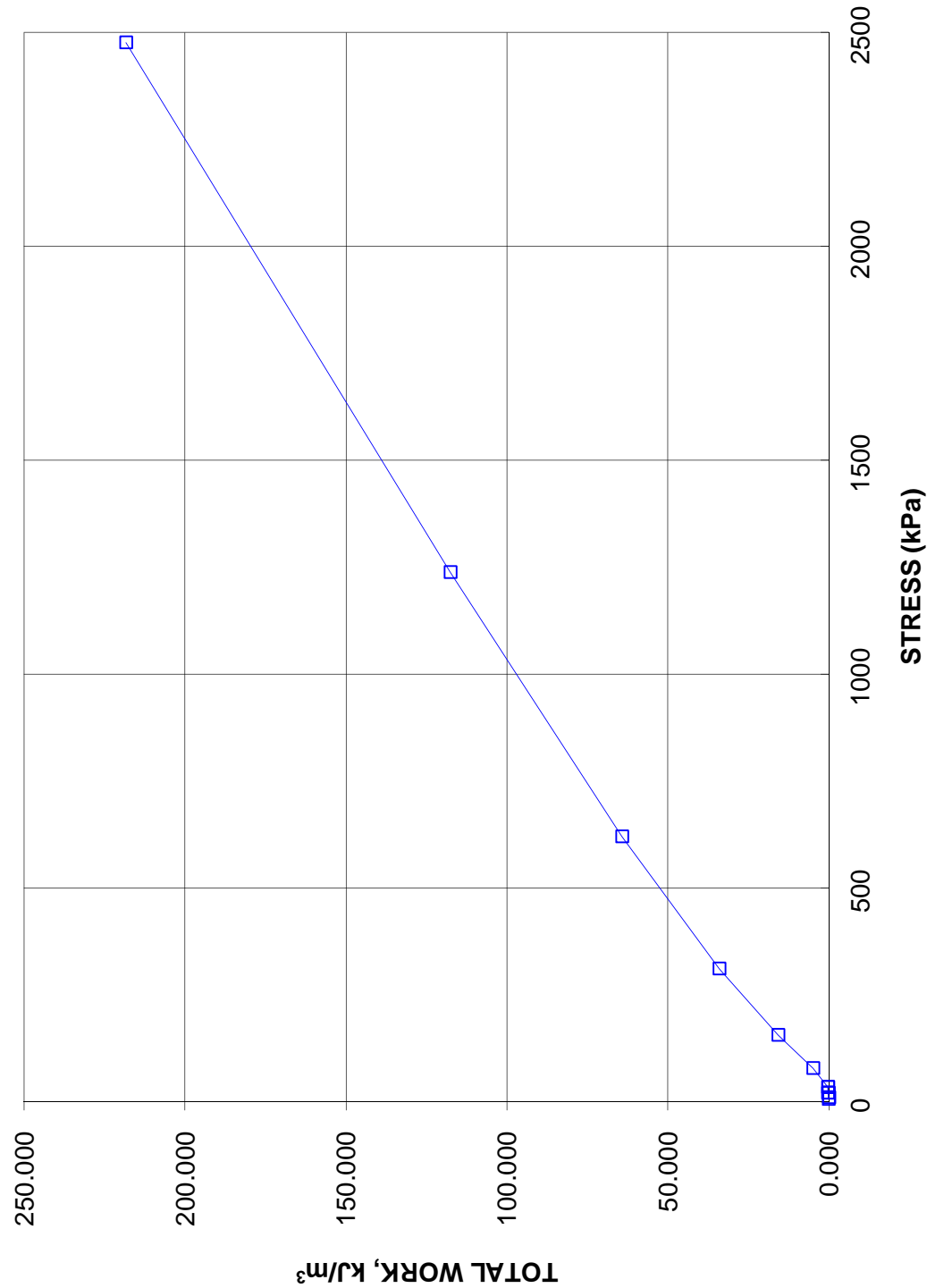
CONSOLIDATION TEST
VOID RATIO vs STRESS
BH H6-S1 SA 2 Vertically Trimmed



CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE C15
Pg. 4 of 4

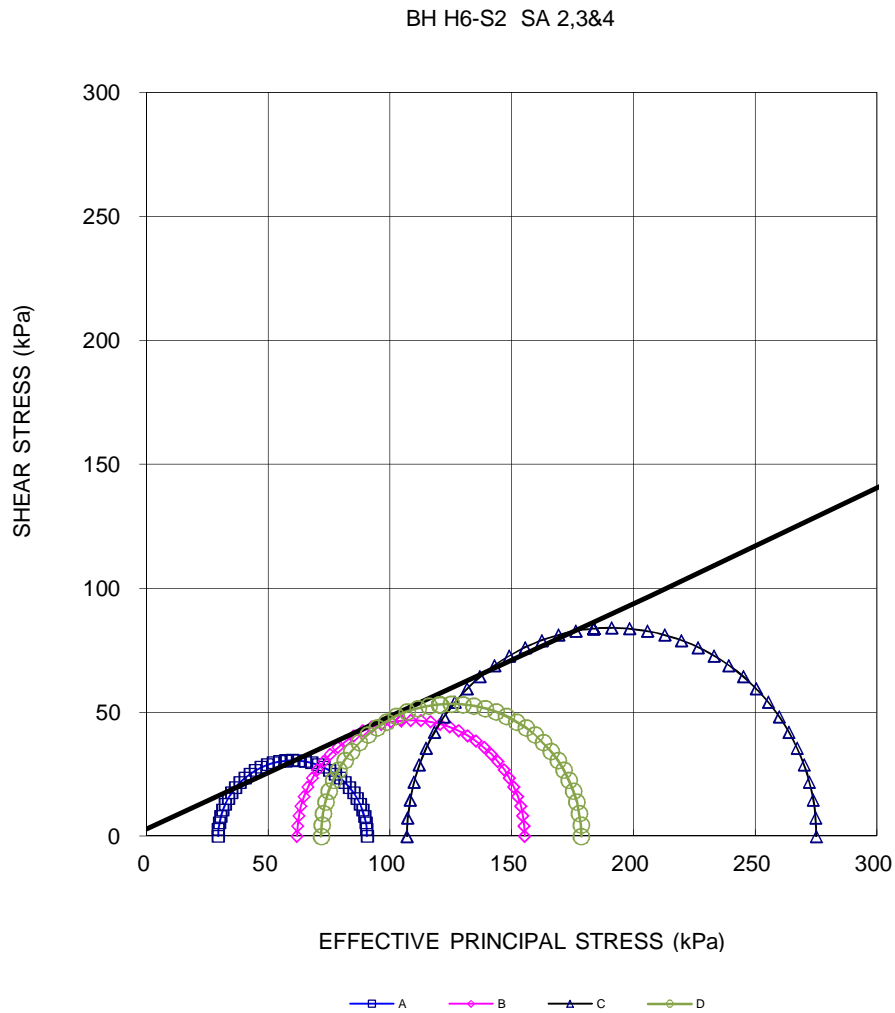
CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs STRESS
BH H6-S1 SA 2 Vertically Trimmed



CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS SHEET 1 OF 4			FIGURE C16	
TEST STAGE	A	B	C	D
BOREHOLE NUMBER	H6-S2	H6-S2	H6-S2	H6-S2
SAMPLE	3	3	4a	2
DEPTH, m	10.67-10.98	10.67-10.98	13.72-14.25	7.62-8.10
SPECIMEN DIAMETER, cm	5.03	5.05	5.06	5.05
SPECIMEN HEIGHT, cm	10.09	10.06	9.97	10.12
NATURAL WATER CONTENT, %	64.0	61.5	54.5	56.1
DRY DENSITY, Mg/m ³	0.99	1.02	1.11	1.07
WATER CONTENT AFTER SATURATION, %	65.9	61.8	55.4	57.0
CELL PRESSURE, σ_3 , kPa	295.0	300.0	460.0	335.0
BACK PRESSURE, kPa	205.0	135.0	135.0	135.0
PORE PRESSURE PARAMETER "B"	0.99	0.96	0.97	0.97
CONSOLIDATION PRESSURE, σ_c , kPa	90.0	165.0	325.0	200.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	10.2	14.8	16.1	18.5
WATER CONTENT AFTER CONSOLIDATION, %	55.6	47.7	40.8	39.7
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5	0.5
TIME TO FAILURE, HOURS	5.0	4.3	4.8	5.0
WATER CONTENT AFTER TEST, %	52.7	43.2	35.1	37.9
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$, kPa	61.1	93.6	168.1	106.8
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ maximum, %	2.5	2.2	2.4	2.5
MAX EFFECTIVE PRINCIPAL STRESS RATIO, (σ'_1 / σ'_3) maximum	4.2	3.8	3.6	3.1
DEVIATOR STRESS AT (σ'_1 / σ'_3) maximum, kPa	40.7	77.8	166.3	98.8
AXIAL STRAIN AT (σ'_1 / σ'_3) maximum, %	16.4	18.0	14.4	9.1
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1 - \sigma_3)$ maximum	0.99	1.10	1.30	1.20
PORE PRESSURE PARAMETER, Af, AT (σ'_1 / σ'_3) maximum	1.90	1.76	1.57	1.55
FILTER DRAINS USED, y/n	y	y	y	y
TEST NOTES: <div style="margin-left: 40px;"> Specimen A taken 17-30 cm from top of tube. Specimen B taken 30-43 cm from top of tube. Specimen C taken 36-49 cm from top of tube. Specimen D taken 34-47 cm from top of tube. </div>				
FAILURE PLANE NUMBER	-	-	-	-
ANGLE OF FAILURE, DEGREES	bulged	bulged	bulged	bulged
<div style="display: flex; justify-content: space-between; align-items: flex-end; padding-top: 20px;"> <div> Date: 6/3/2013 Project No. 10-1191-0044 </div> <div style="text-align: center;"> Golder Associates </div> <div> Prepared By: LH Checked By: MT </div> </div>				

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 2 OF 4

FIGURE C16



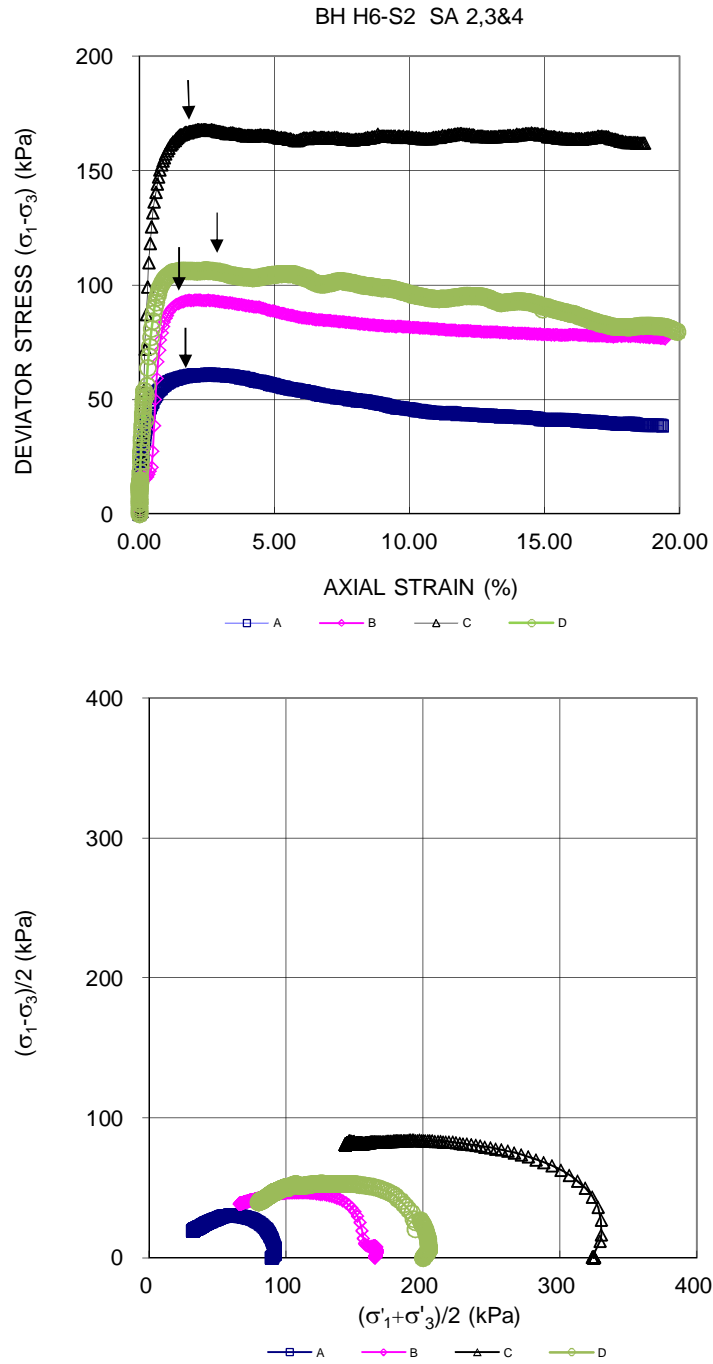
Date: 6/3/2013
Project No. 10-1191-0044

Golder Associates

Prepared By: LH
Checked By: MT

**CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 3 OF 4**

FIGURE C16



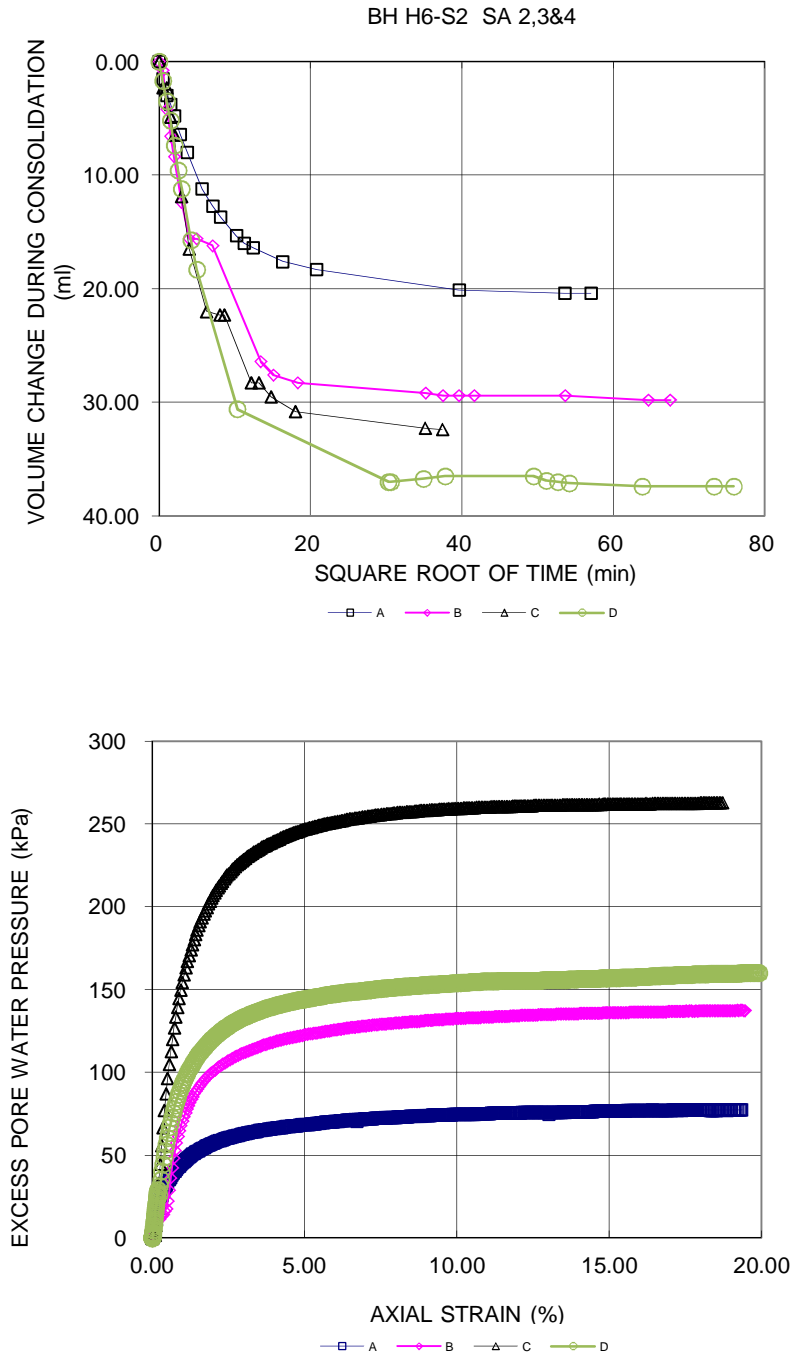
Date: 6/3/2013
Project No. 10-1191-0044

Golder Associates

Prepared By: LH
Checked By: MT

**CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 4 OF 4**

FIGURE C16

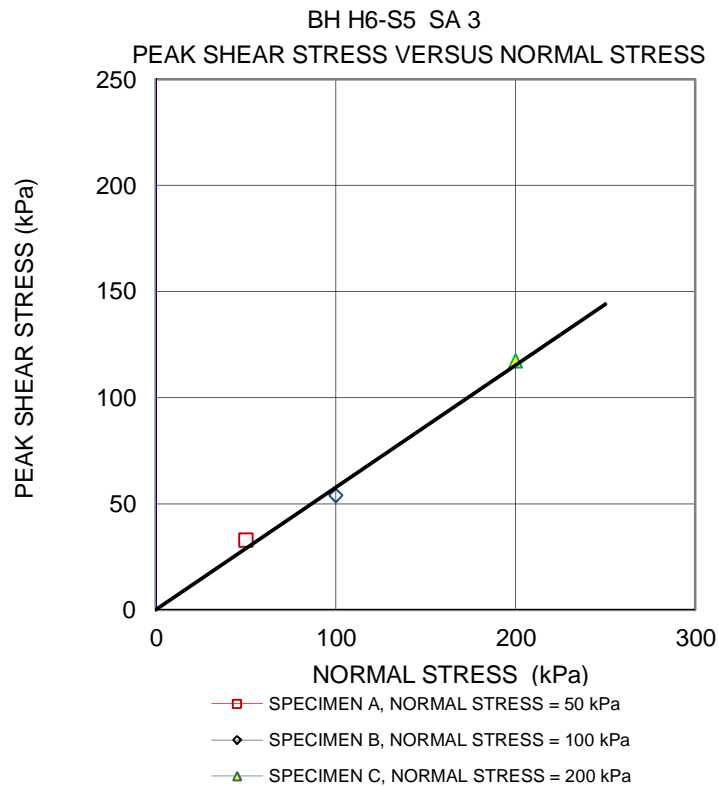
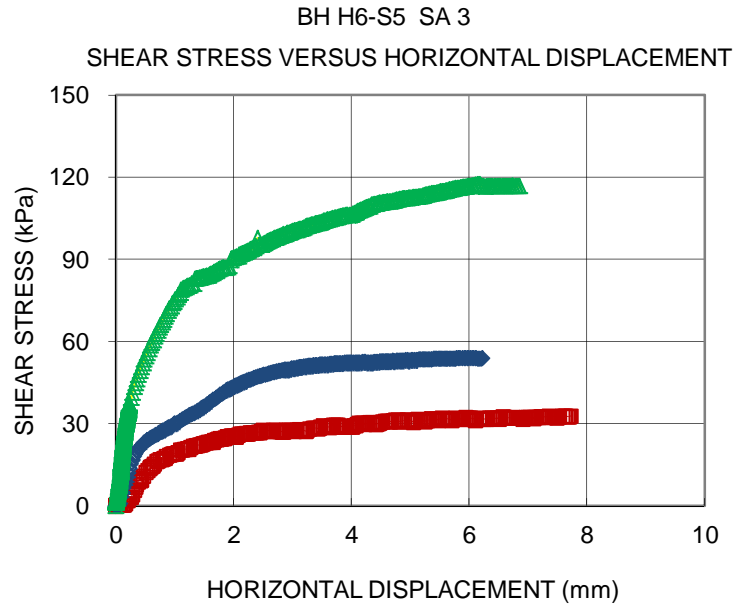


Date: 6/3/2013
Project No. 10-1191-0044

Golder Associates

Prepared By: LH
Checked By: MT

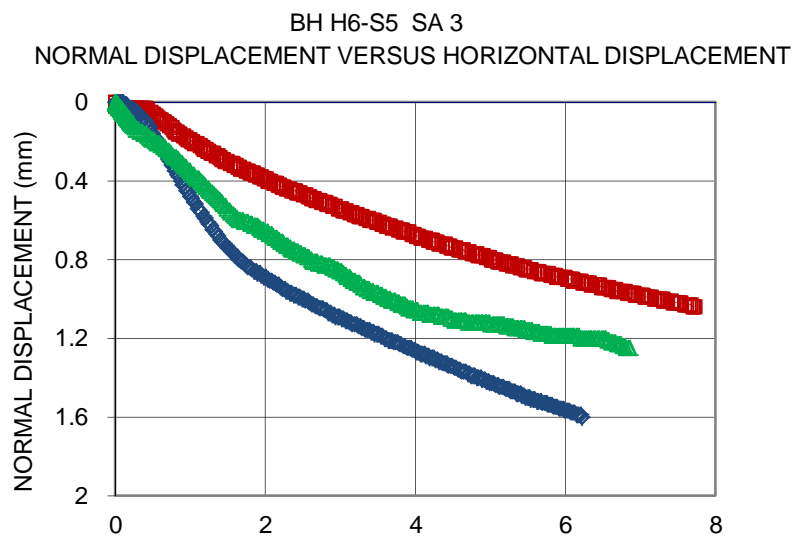
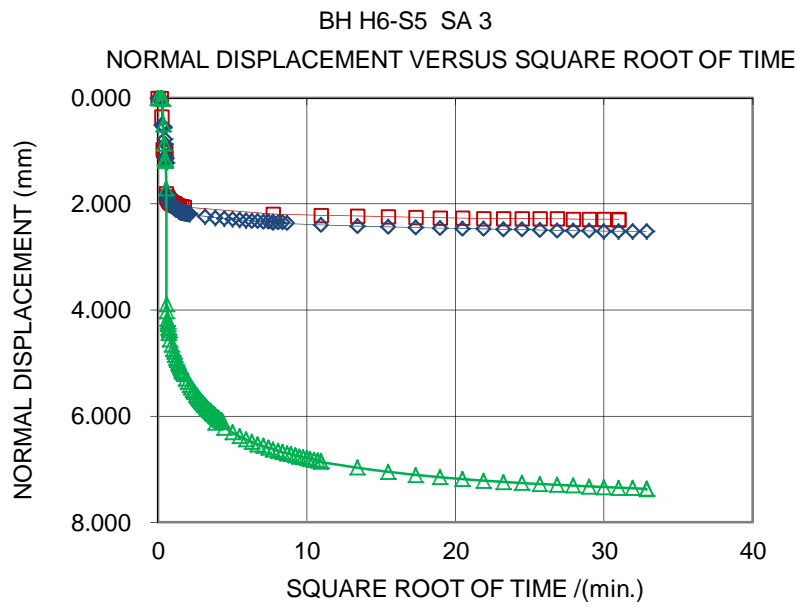
CONSOLIDATED DRAINED DIRECT SHEAR TEST SHEET 1 OF 3		FIGURE C17	
TEST STAGE	A	B	C
BOREHOLE NUMBER	H6-S5	H6-S5	H6-S5
SAMPLE NUMBER	3	3	3
SAMPLE DEPTH, (m)	7.32-7.80	7.32-7.80	7.32-7.80
SAMPLE HEIGHT, (mm)	25.00	25.00	25.20
SAMPLE LENGTH, (mm)	60.00	60.00	60.00
WATER CONTENT, BEFORE TEST, (%)	57.69	61.35	65.87
NORMAL (CONSOLIDATION) STRESS, (kPa)	50.00	100.00	200.00
WATER CONTENT, AFTER TEST, (%)	63.76	61.63	50.24
DISPLACEMENT RATE, mm/min	0.0048	0.0048	0.0048
TIME TO FAILURE, HOURS	26	20	21
PEAK SHEAR STRESS, (kPa)	32.88	53.88	117.35
HORIZONTAL DISPLACEMENT AT PEAK, (mm)	7.35	5.89	6.15
DRY DENSITY, initial, Mg/m ³	1.02	1.00	0.95
WET DENSITY, initial, Mg/m ³	1.61	1.61	1.57
TEST NOTES:			
<div> <div>Date: 6/14/2013</div> <div>Project No. 10-1191-0044</div> </div> <div> <div>Golder Associates</div> </div> <div> <div>Prepared By: LH</div> <div>Checked By: MT</div> </div>			



Date: 6/14/2013
 Project No. 10-1191-0044

Golder Associates

Prepared By: LH
 Checked By: MT



HORIZONTAL DISPLACEMENT (mm)

—□— SPECIMEN A, NORMAL STRESS = 50 kPa

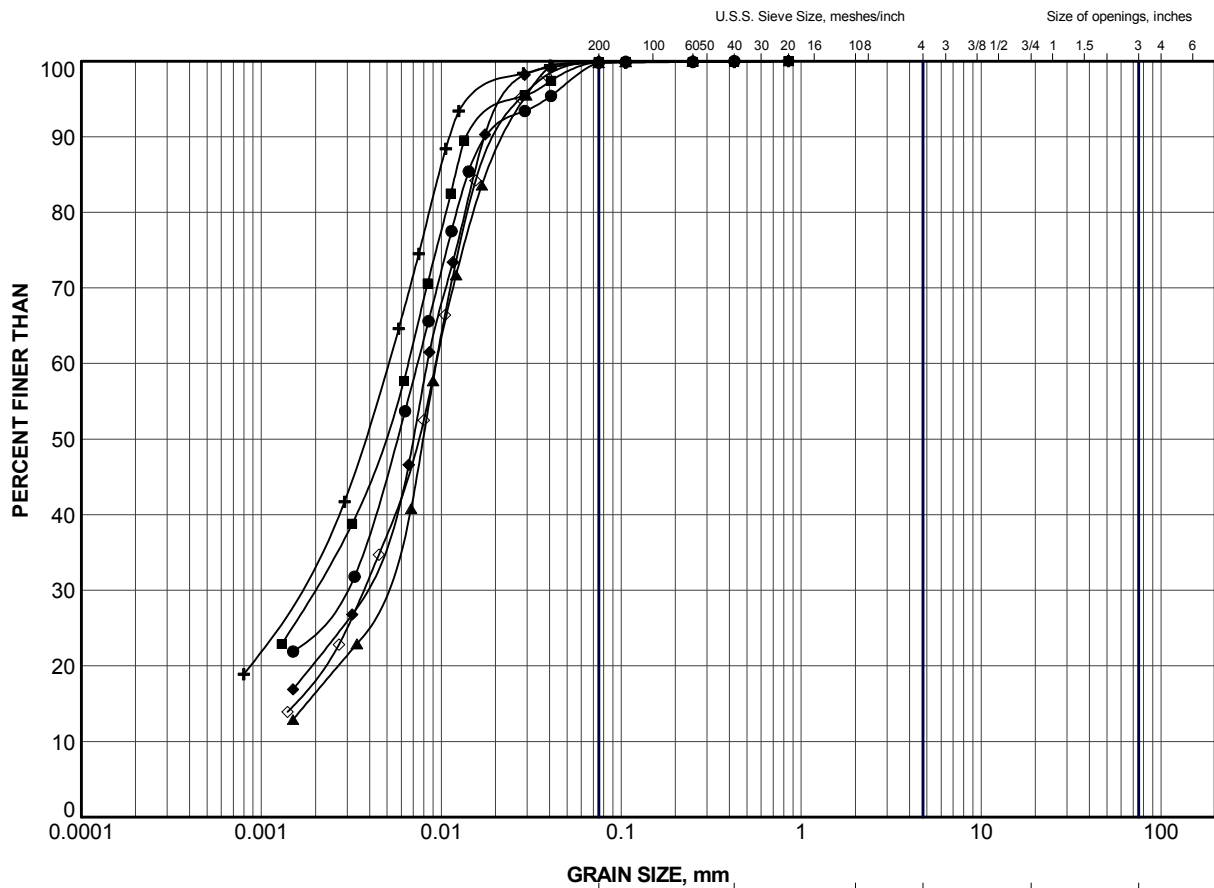
—◇— SPECIMEN B, NORMAL STRESS = 100 kPa

—△— SPECIMEN C, NORMAL STRESS = 200 kPa

Date: 6/14/2013
 Project No. 10-1191-0044

Golder Associates

Prepared By: LH
 Checked By: MT



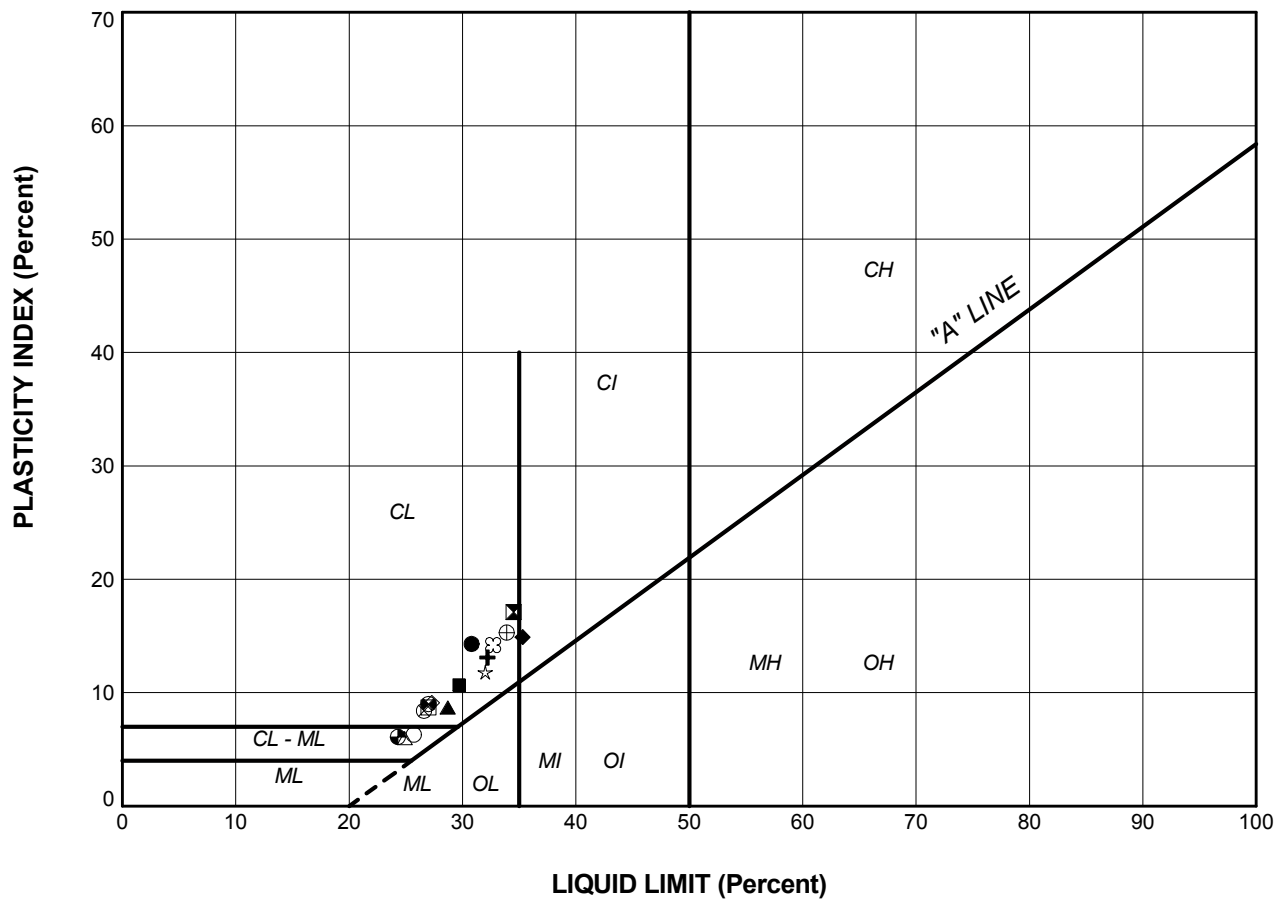
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC3-1	14	293.2
■	BC3-2	14	293.3
▲	H6-7	15	290.4
+	H6-16	8	297.3
◆	H7-2	8	295.5
◇	H7-5	8	295.8

PROJECT					
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650					
TITLE					
GRAIN SIZE DISTRIBUTION CLAYEY SILT					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013			
APPR	JMAC	Jul 2013			
			FIGURE C18		





LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BC3-1	14	30.8	16.5	14.3
■	BC3-2	14	29.7	19.0	10.7
▲	BC4-3	8	28.7	20.0	8.7
+	H6-2	14	32.2	19.1	13.1
◆	H6-5	14	35.3	20.4	14.9
◇	H6-6	16	27.3	18.2	9.1
○	H6-7	15	25.7	19.4	6.3
△	H6-9	15	24.9	18.9	6.0
⊗	H6-10	15	26.6	18.2	8.4
⊕	H6-16	8	33.9	18.6	15.3
□	H7-1	9	27.0	18.3	8.7
⊗	H7-2	8	27.0	18.0	9.0
⊕	H7-3	10	24.3	18.2	6.1
☆	H7-4	8	32.0	20.2	11.8
⊗	H7-7	7a	32.7	18.5	14.2
⊕	H7-14	4	34.5	17.4	17.1

PROJECT					
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650					
TITLE					
PLASTICITY CHART CLAYEY SILT					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013			
APPR	JMAC	Jul 2013			
			FIGURE C19		



CONSOLIDATION TEST SUMMARY**FIGURE C20**

Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Number	10-1191-0044	Sample Number	5
Borehole Number	H6-S1	Sample Depth, m	17.02-17.10

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	10		
Date Started	5/31/2013		
Date Completed	6/19/2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.53	Unit Weight, kN/m ³	18.81
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	14.16
Area, cm ²	31.67	Specific Gravity, measured	2.74
Volume, cm ³	80.25	Solids Height, cm	1.335
Water Content, %	32.87	Volume of Solids, cm ³	42.28
Wet Mass, g	153.93	Volume of Voids, cm ³	37.97
Dry Mass, g	115.85	Degree of Saturation, %	100.3

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	2.534	0.898	2.534				
5.95	2.524	0.890	2.529	94	1.44E-02	6.96E-04	9.84E-07
10.67	2.515	0.884	2.519	296	4.55E-03	6.94E-04	3.09E-07
20.40	2.503	0.875	2.509	231	5.78E-03	5.07E-04	2.87E-07
39.80	2.486	0.862	2.494	254	5.19E-03	3.48E-04	1.77E-07
78.51	2.463	0.845	2.474	187	6.94E-03	2.32E-04	1.58E-07
92.81	2.457	0.840	2.460	421	3.05E-03	1.60E-04	4.78E-08
39.80	2.462	0.844	2.460				
10.67	2.469	0.849	2.466				
39.80	2.462	0.844	2.465	15	8.59E-02	9.62E-05	8.10E-07
92.84	2.453	0.837	2.457	18	7.11E-02	6.40E-05	4.46E-07
155.77	2.427	0.818	2.440	360	3.51E-03	1.64E-04	5.64E-08
309.81	2.337	0.750	2.382	254	4.74E-03	2.30E-04	1.07E-07
619.22	2.216	0.660	2.277	214	5.13E-03	1.54E-04	7.74E-08
1241.26	2.136	0.600	2.176	113	8.88E-03	5.10E-05	4.44E-08
2479.48	2.063	0.545	2.100	73	1.28E-02	2.32E-05	2.91E-08
619.22	2.077	0.555	2.070				
155.77	2.100	0.573	2.088				
78.51	2.111	0.581	2.106				
20.38	2.136	0.600	2.123				
5.95	2.149	0.610	2.142				

Note:

k calculated using cv based on $\dot{\epsilon}_0$ values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	2.15	Unit Weight, kN/m ³	21.12
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	16.69
Area, cm ²	31.67	Specific Gravity, measured	2.74
Volume, cm ³	68.06	Solids Height, cm	1.335
Water Content, %	26.54	Volume of Solids, cm ³	42.28
Wet Mass, g	146.60	Volume of Voids, cm ³	25.78
Dry Mass, g	115.85		

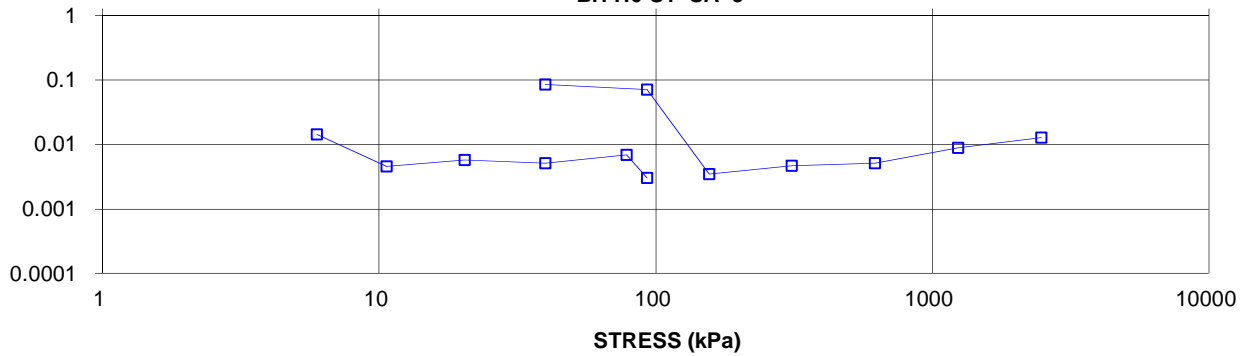
Prepared By:

Golder Associates

Checked By: MT

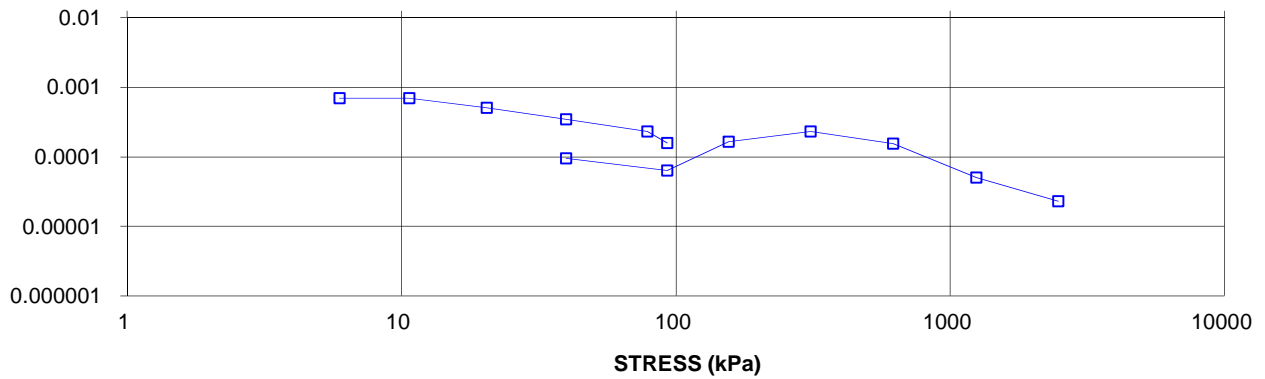
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS STRESS (kPa)
BH H6-S1 SA 5



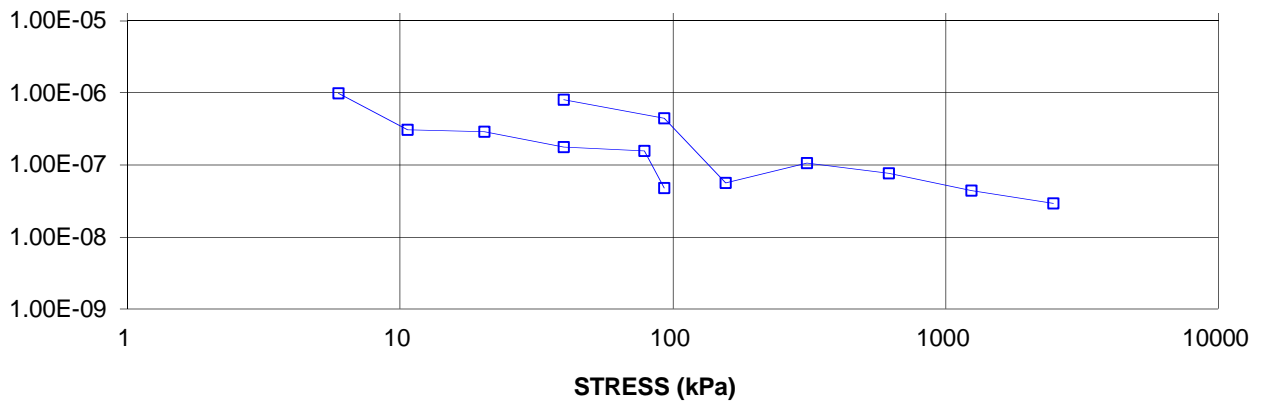
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs STRESS (kPa)
BH H6-S1 SA 5



HYDRAULIC CONDUCTIVITY,
cm/s

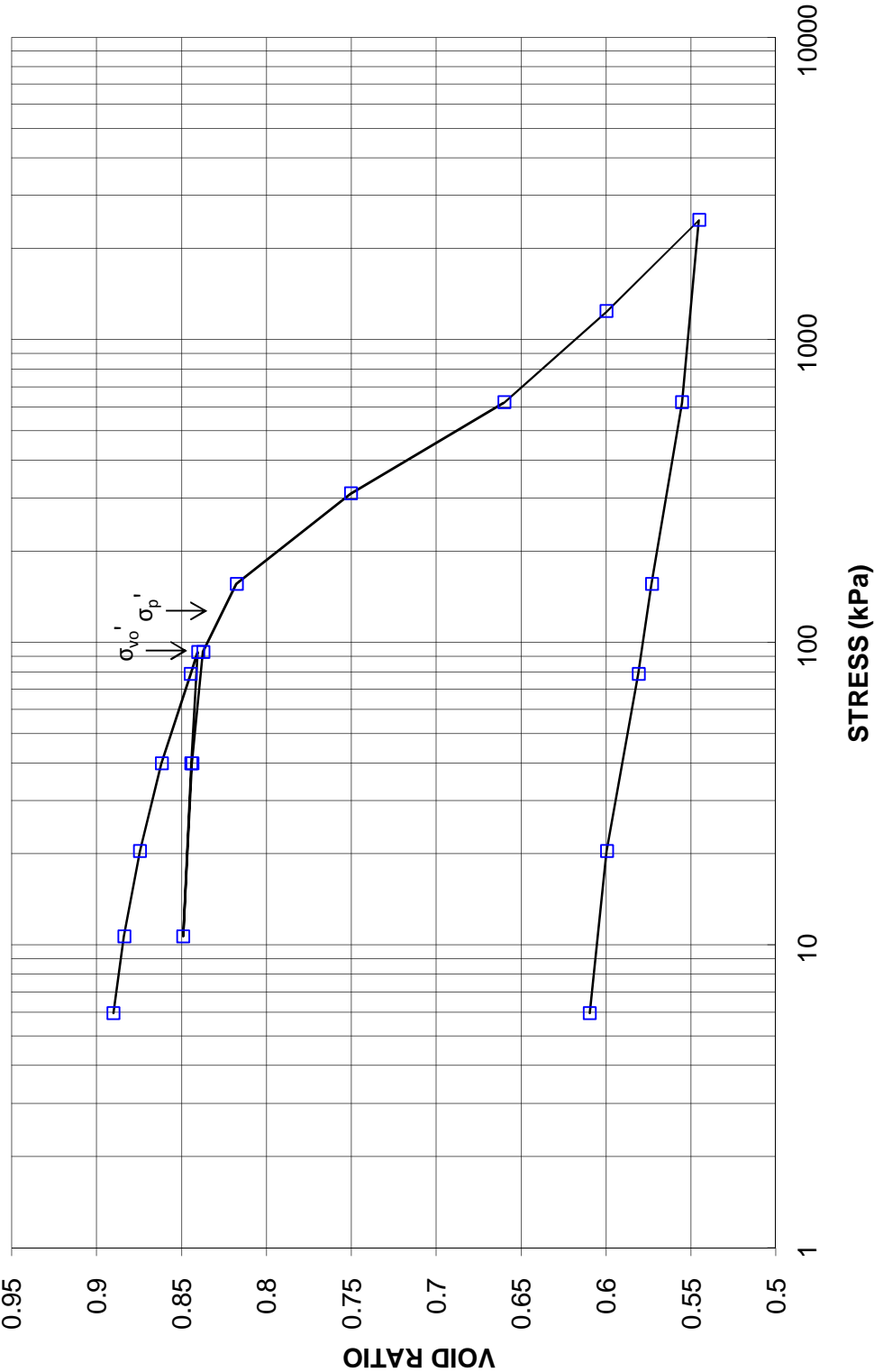
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs STRESS
BH H6-S1 SA 5



CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE C20
Pg. 3 of 4

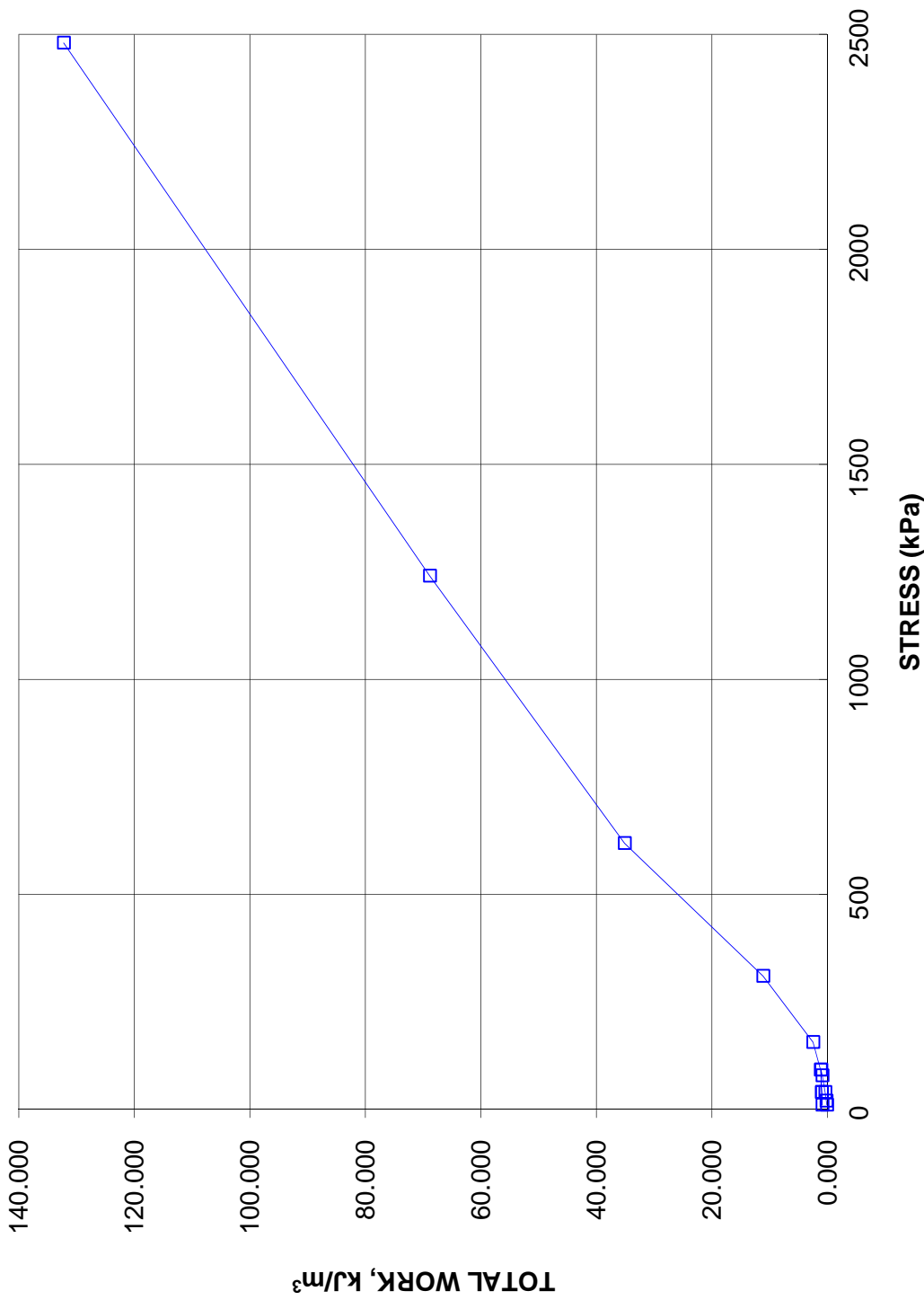
CONSOLIDATION TEST
VOID RATIO vs STRESS
BH H6-S1 SA 5

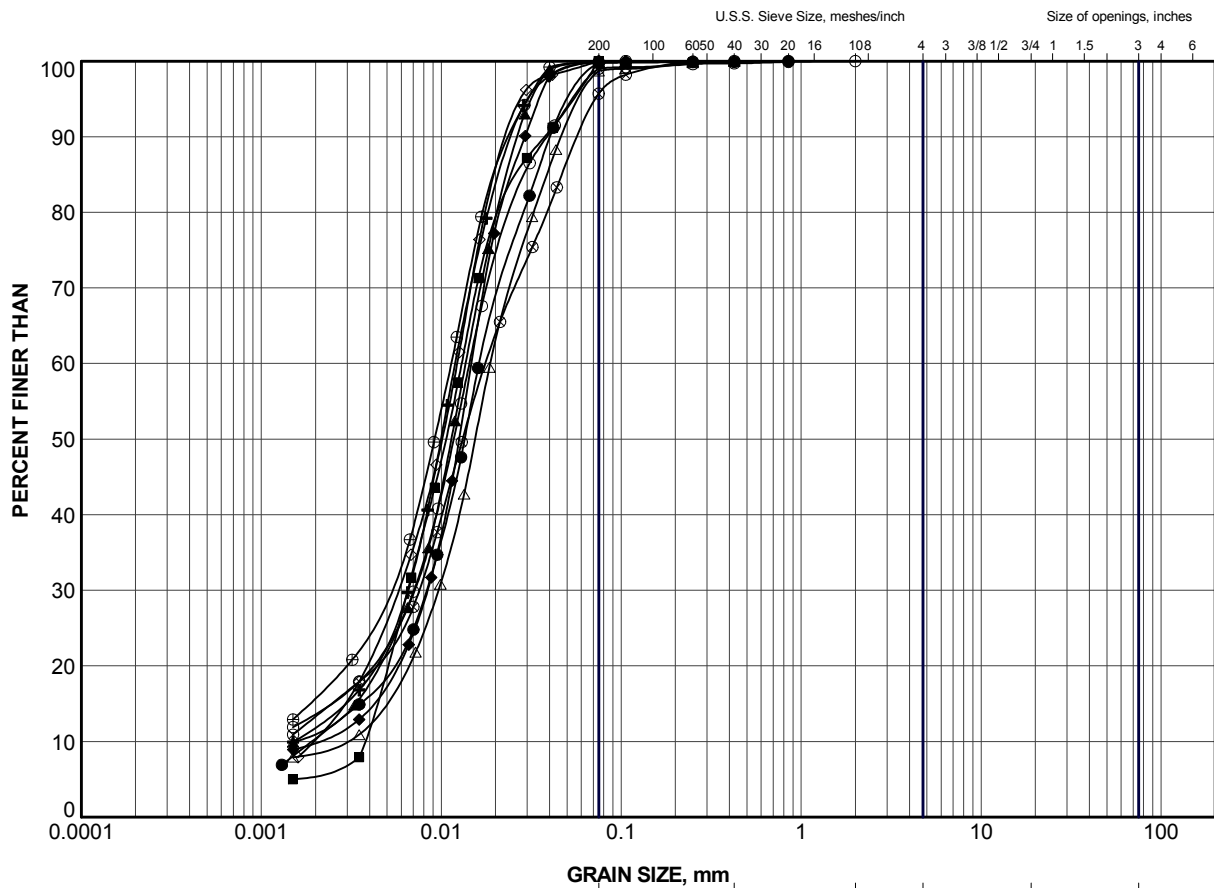


CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE C20
Pg. 4 of 4

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs STRESS
BH H6-S1 SA 5





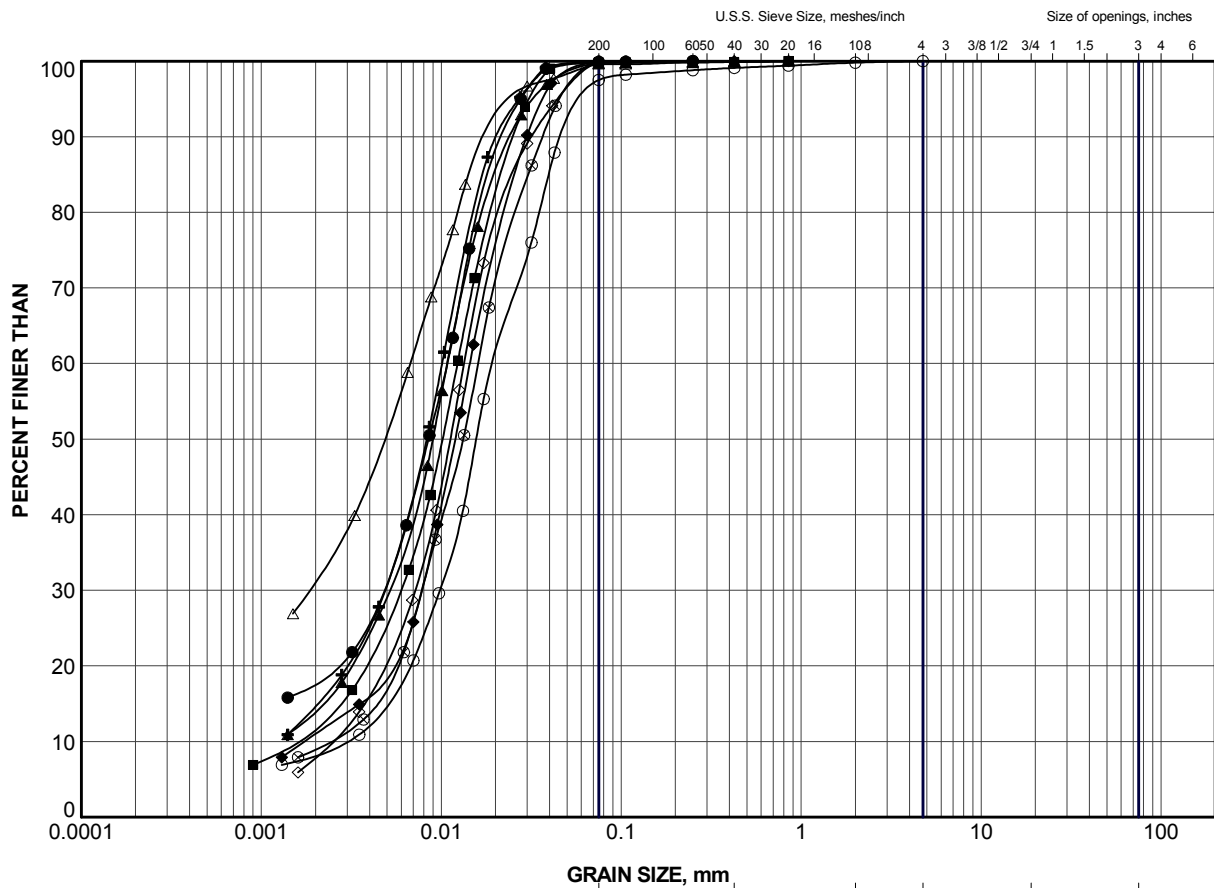
CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC3-2	16	287.2
■	BC3-3	15b	289.8
▲	BC4-1	10	292.5
+	BC4-2	8	294.7
◆	BC4-3	10	292.4
◇	H6-4	15	290.4
○	H6-5	16	287.4
△	H6-6	17	287.3
⊗	H6-9	17	284.2
⊕	H6-12	13	292.8

PROJECT					
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650					
TITLE					
GRAIN SIZE DISTRIBUTION SILT					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013	FIGURE C21.1		
APPR	JMAC	Jul 2013			





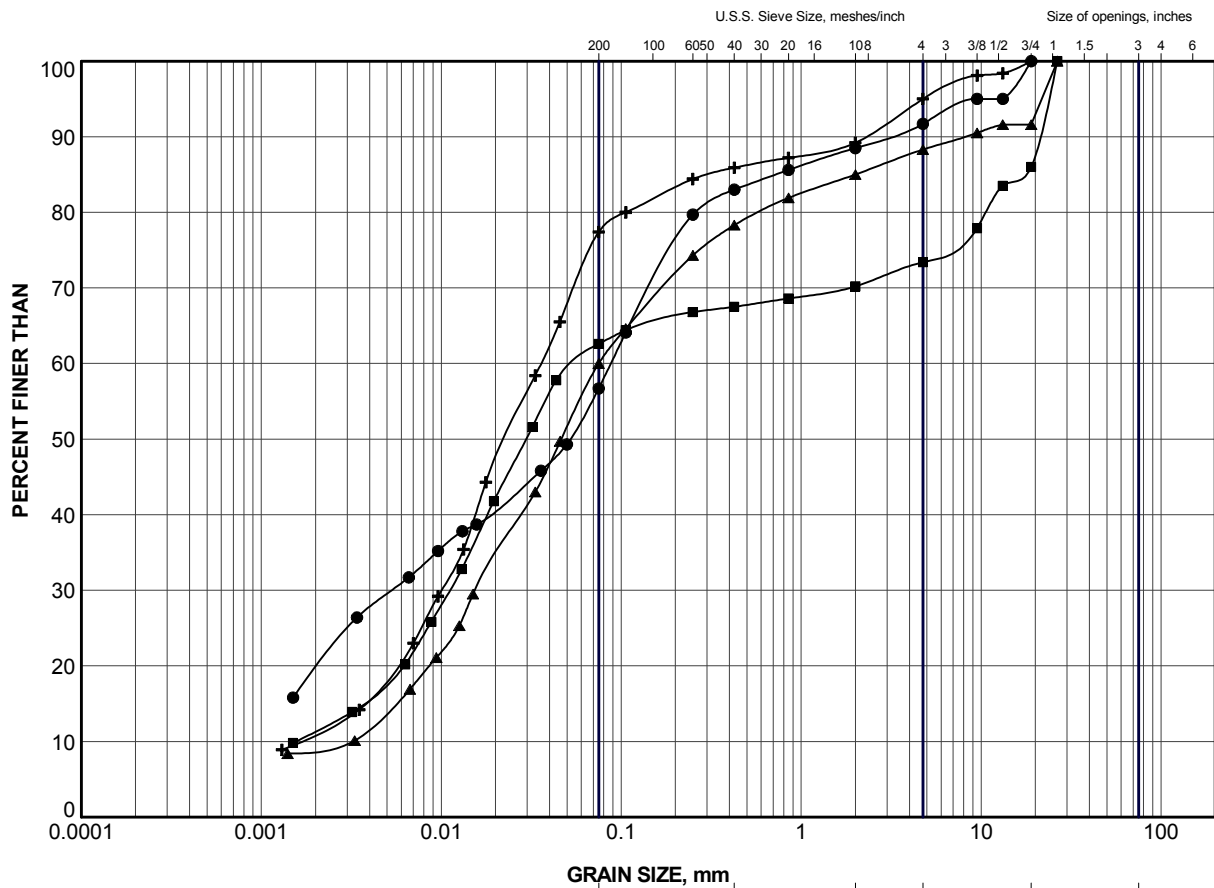
CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	H6-15	10	295.8
■	H6-17	9	295.2
▲	H6-18	9	296.6
+	H6-19	7	297.7
◆	H7-3	12	291.1
◇	H7-4	9	294.3
○	H7-8	4	302.1
△	H7-9	3	304.2
⊗	H7-10	4	301.5

PROJECT					
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650					
TITLE					
GRAIN SIZE DISTRIBUTION SILT					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013	FIGURE C21.2		
APPR	JMAC	Jul 2013			




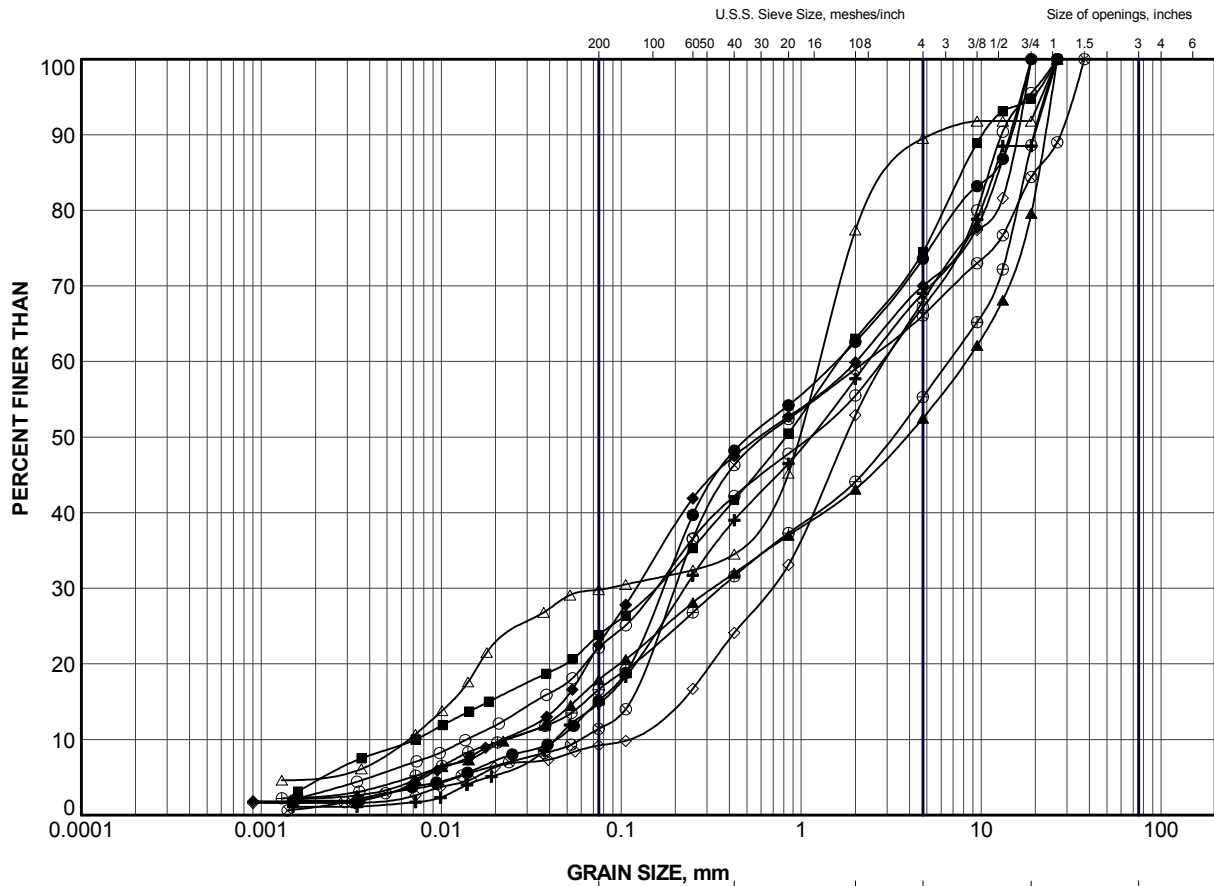


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	H6-1	13	293.2
■	H6-11	16	285.2
▲	H6-14	11	294.9
+	H7-12	2	306.7

PROJECT					
HIGHWAY 66 - HIGH FILL H6 / H7 STA 14+020 TO 14+650					
TITLE					
GRAIN SIZE DISTRIBUTION SILT to SAND AND SILT to GRAVELLY SILT					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013			
APPR	JMAC	Jul 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE C21.3		



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC4-1	12	289.8
■	H6-2	15	290.4
▲	H6-12	15	286.7
+	H6-15	12	292.8
◆	H6-16	11	292.9
◇	H6-17	11	292.2
○	H7-2	10	292.6
△	H7-3	13	289.7
⊗	H7-6	10	291.9
⊕	H7-8	5b	301.2

PROJECT

HIGHWAY 66 - HIGH FILL H6 / H7
STA 14+020 TO 14+650

TITLE

GRAIN SIZE DISTRIBUTION

SILTY SAND to SAND AND GRAVEL



Golder Associates
SUDBURY, ONTARIO

PROJECT No. 10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE N/A
CHECK	SEMC	Jul 2013	REV.
APPR	JMAC	Jul 2013	

FIGURE C22



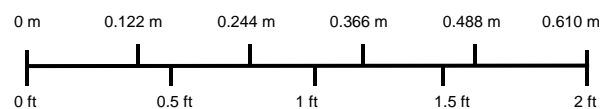
Borehole BC2-1
Elevation 305.9 m to 302.9 m




Borehole BC2-2
Elevation 302.3 m to 298.9 m



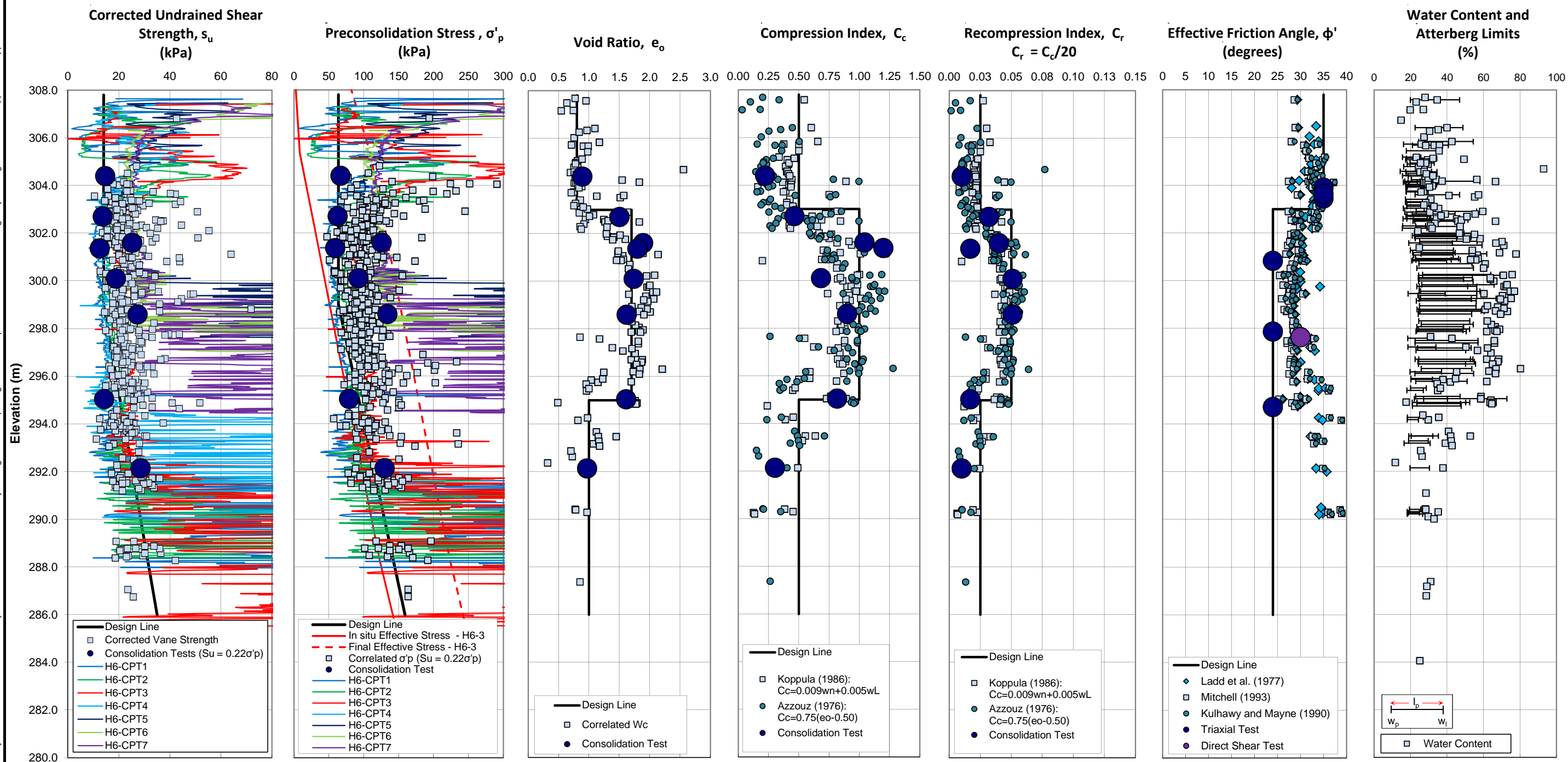
Borehole BC2-3
Elevation 307.2 m to 304.3 m

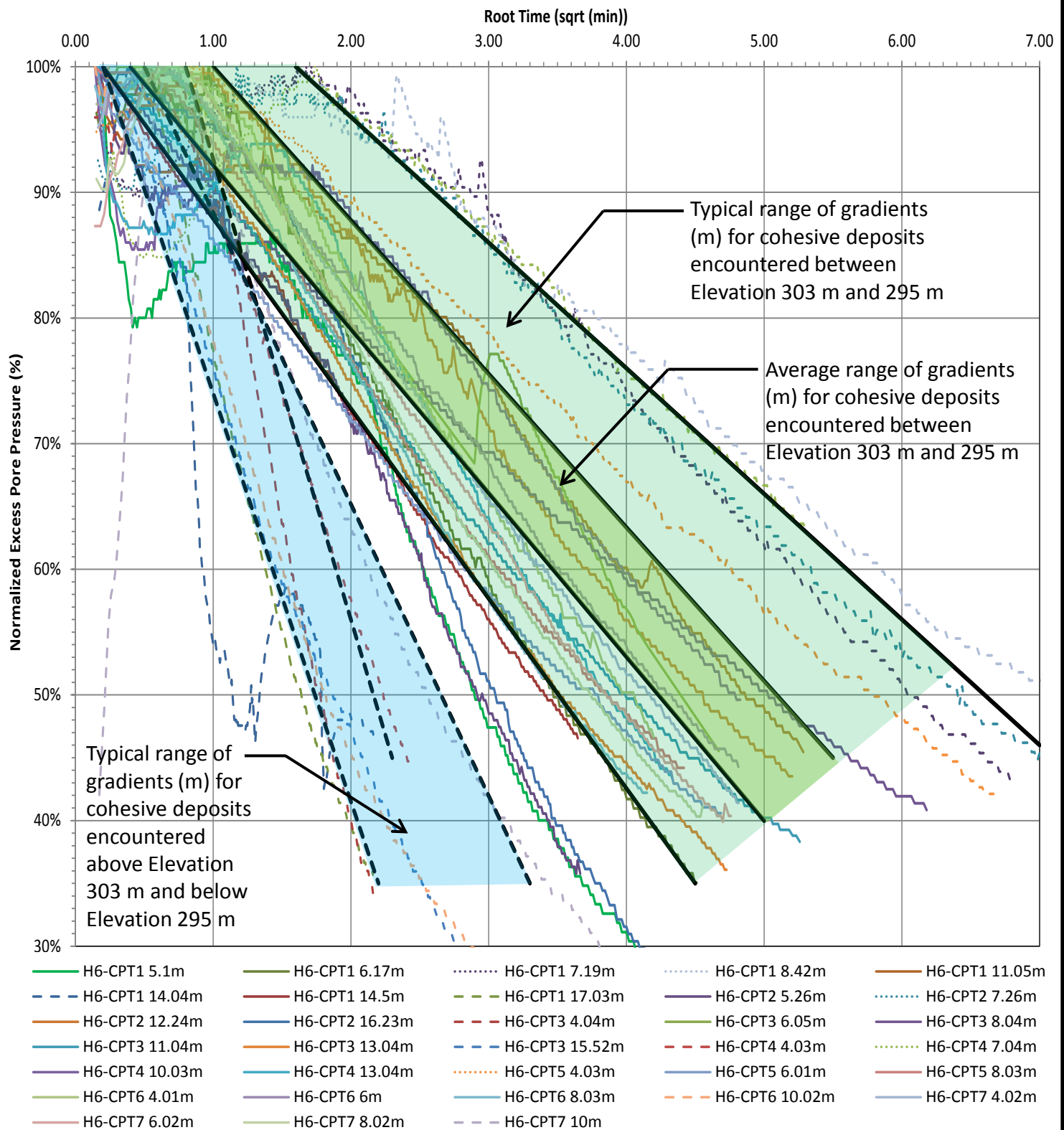


PROJECT		HIGHWAY 66 – HIGH FILL H6 / H7 STA 14+020 to 14+650			
TITLE		BEDROCK CORE PHOTOGRAPHS			
	PROJECT No.	10-1191-0044		FILE No. ----	
	DESIGN	MT	APR 2013	SCALE	AS SHOWN
	CADD	--		REV.	
	CHECK	SEMC	APR 2013	FIGURE C23	
	REVIEW	JMAC	JUN 2013		

**SUMMARY OF ENGINEERING PARAMETERS
FOR COHESIVE DEPOSITS**
Highway 66 Realignment, Virginiatown - STA 14+020 to 14+650 (Swamp
Crossing H6/H7)


FIGURE C24

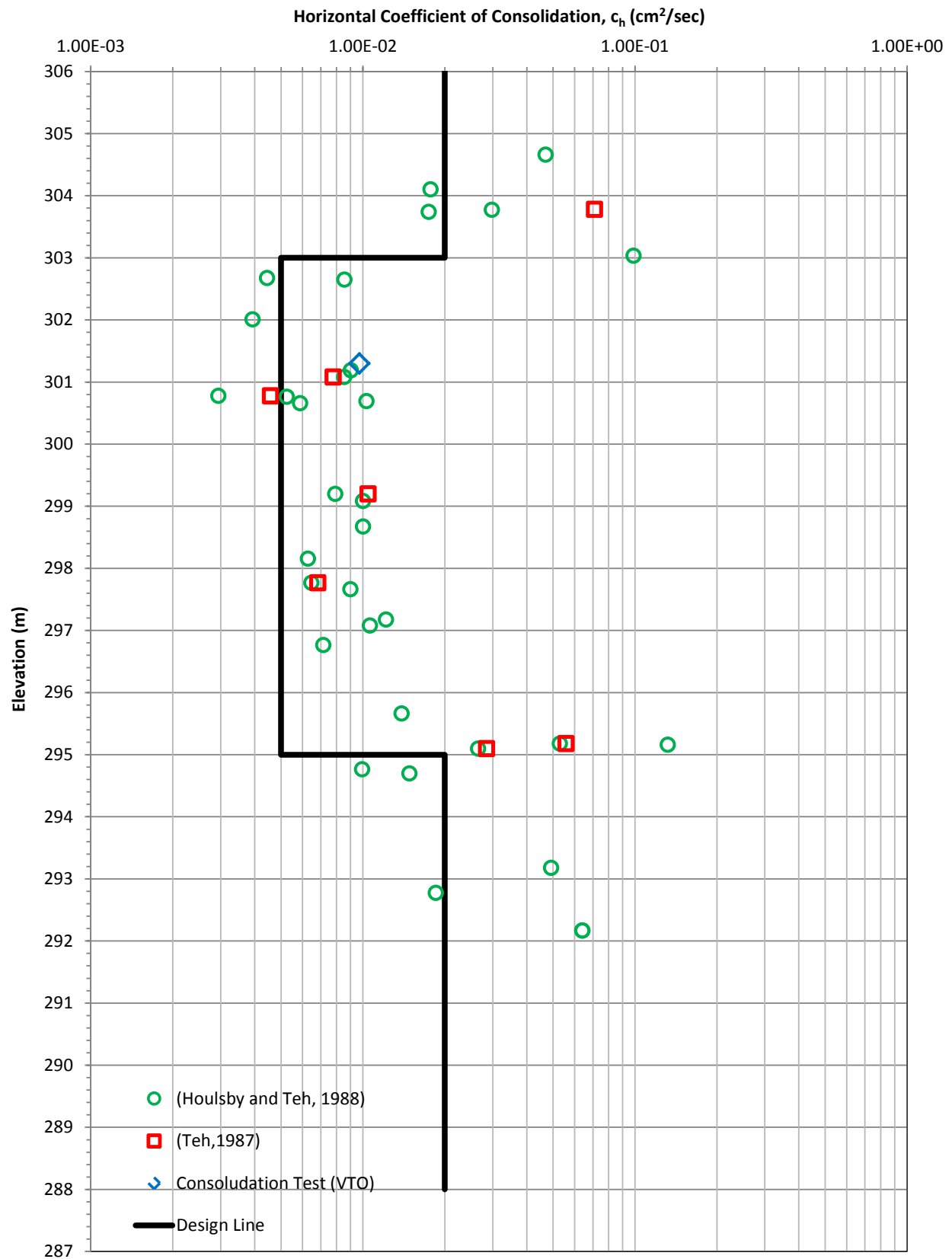





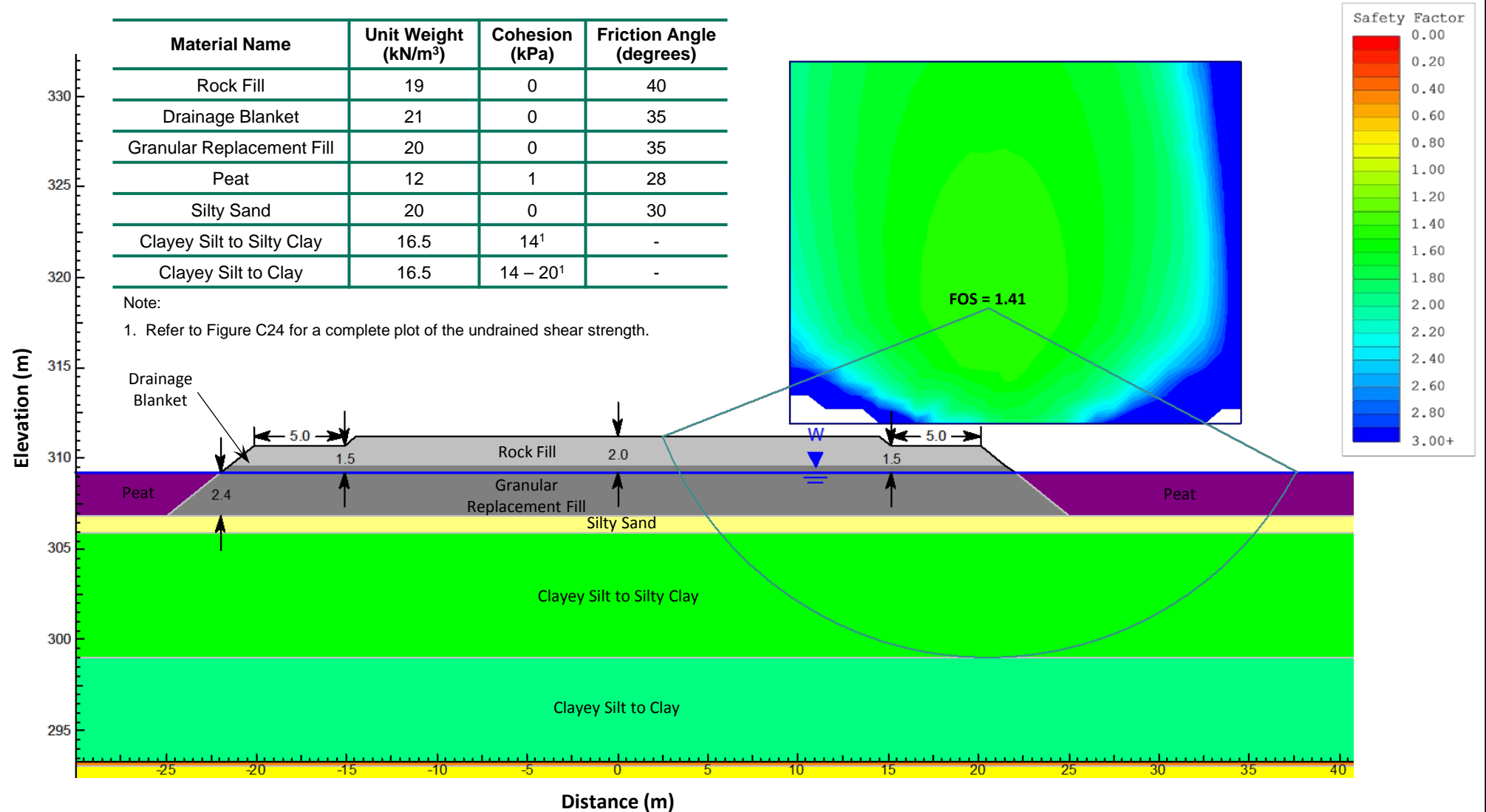
Note:

1. A total of 33 dissipation tests were carried out during the Cone Penetration Test (CPT) investigation.

PROJECT		HIGHWAY 66 SWAMP CROSSING H6/H7	
TITLE		CONE PENETRATION TESTS PORE PRESSURE DISSIPATION TESTS	
		PROJECT No.	10-1191-0044
		DESIGN	TZ
		CADD	--
		CHECK	CN
		REVIEW	JPD
		FILE No.	----
		SCALE	AS SHOWN
		REV.	
		FIGURE C25	



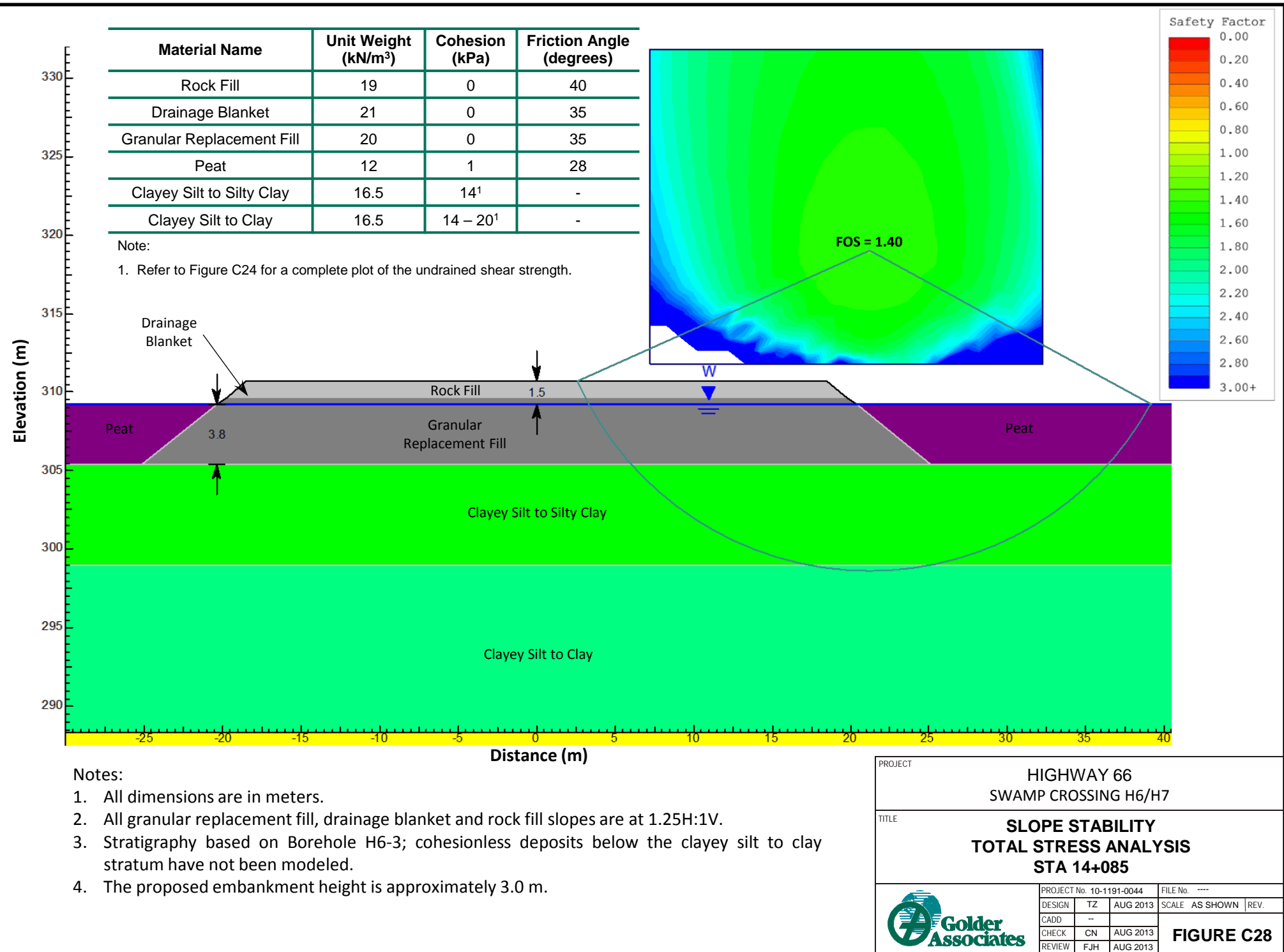
PROJECT		HIGHWAY 66 SWAMP CROSSING H6/H7	
TITLE		SUMMARY OF HORIZONTAL COEFFICIENTS OF CONSOLIDATION FOR COHESIVE DEPOSITS	
	PROJECT No.	10-1191-0044	FILE No. ----
	DESIGN	TZ	AUG 2013
	CADD	--	SCALE AS SHOWN
	CHECK	CN	AUG 2013
REVIEW		JPD	AUG 2013
		FIGURE C26	

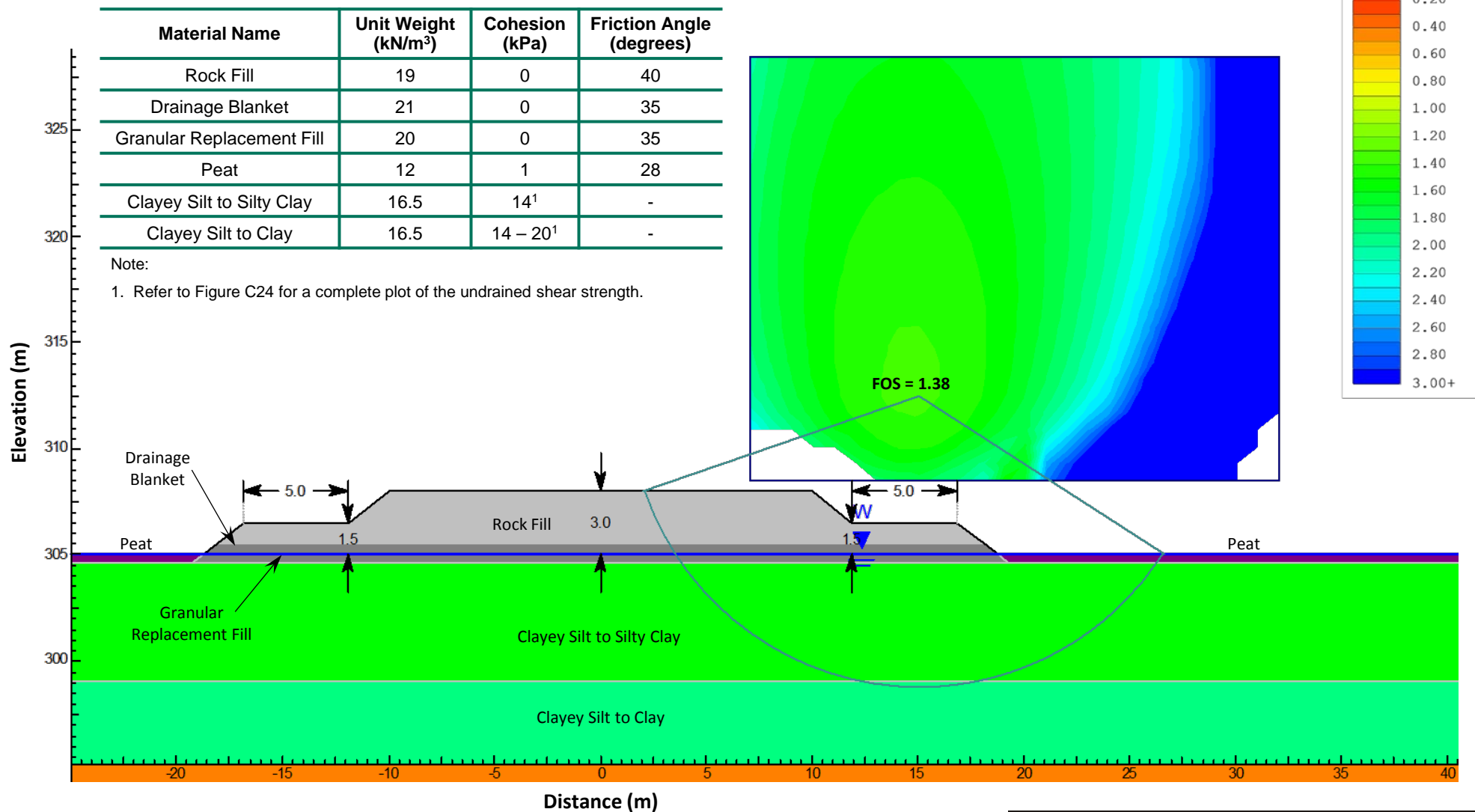


Notes:

1. All dimensions are in meters.
2. All granular replacement fill, drainage blanket and rock fill slopes are at 1.25H:1V.
3. Stratigraphy based on Borehole H6-1; cohesionless deposits below the clayey silt to clay stratum have not been modeled.
4. The proposed embankment height is approximately 3.5 m.

PROJECT		HIGHWAY 66 SWAMP CROSSING H6/H7			
TITLE		SLOPE STABILITY TOTAL STRESS ANALYSIS STA 14+060			
		PROJECT No. 10-1191-0044		FILE No. ----	
		DESIGN	TZ	AUG 2013	SCALE AS SHOWN
		CADD	--	--	REV.
		CHECK	CN	AUG 2013	
		REVIEW	FJH	AUG 2013	
		FIGURE C27			





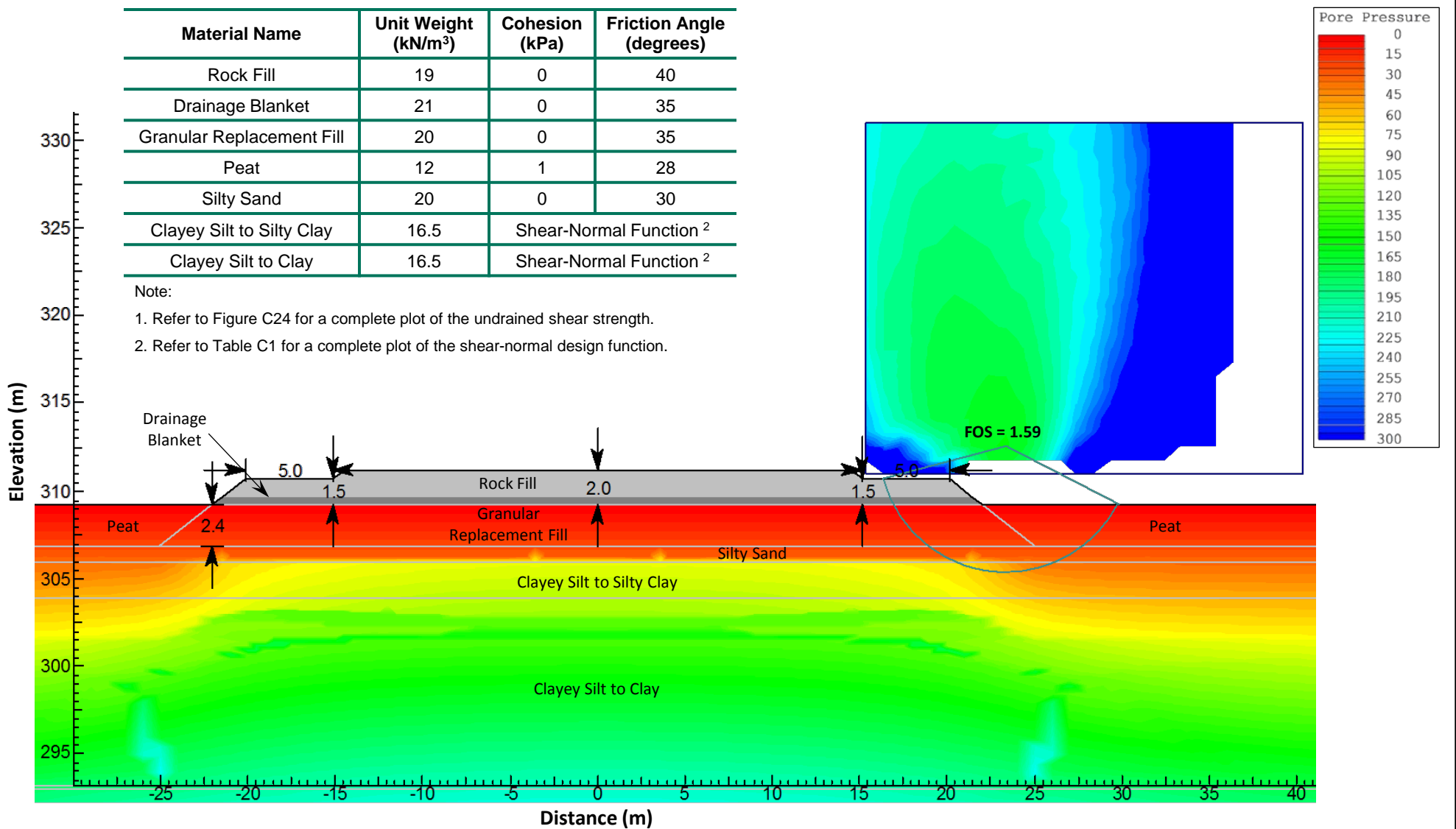
Notes:

1. All dimensions are in meters.
2. All granular replacement fill, drainage blanket and rock fill slopes are at 1.25H:1V.
3. Stratigraphy based on Borehole H7-5; cohesionless deposits below the clayey silt to clay stratum have not been modeled.
4. The proposed embankment height is approximately 3.0 m.

PROJECT		HIGHWAY 66 SWAMP CROSSING H6/H7			
TITLE		SLOPE STABILITY TOTAL STRESS ANALYSIS STA 14+530			
		PROJECT No. 10-1191-0044	FILE No. ----		
DESIGN	TZ	AUG 2013	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	CN	AUG 2013			
REVIEW	FJH	AUG 2013			




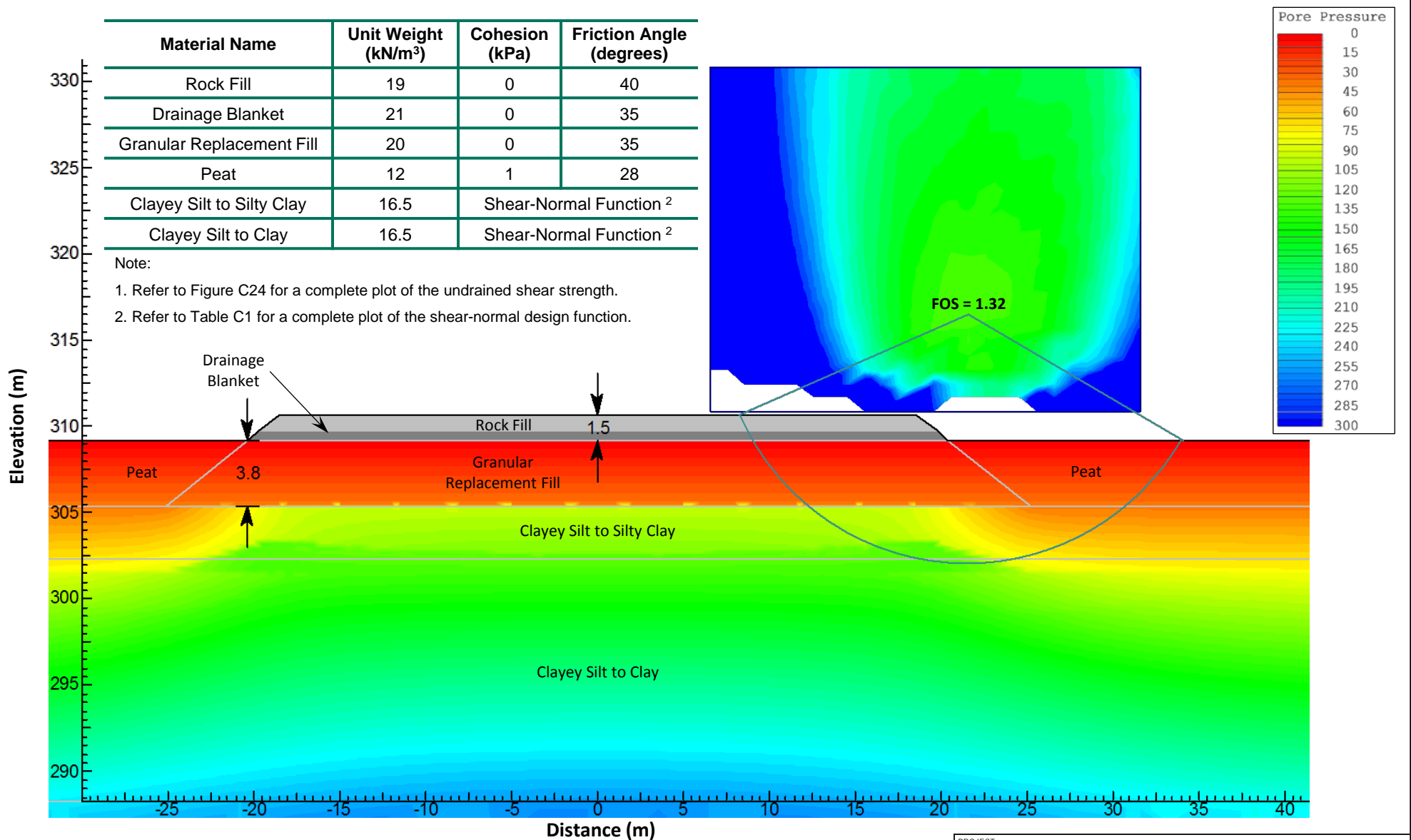
FIGURE C29



Notes:

1. All dimensions are in meters.
2. All granular replacement fill, drainage blanket and rock fill slopes are at 1.25H:1V.
3. Stratigraphy based on Borehole H6-1; cohesionless deposits below the clayey silt to clay stratum have not been modeled.
4. The proposed embankment height is approximately 3.5 m.

PROJECT		HIGHWAY 66 SWAMP CROSSING H6/H7				
TITLE		SLOPE STABILITY EFFECTIVE STRESS ANALYSIS (RAPID LOADING) STA 14+060				
	PROJECT No. 10-1191-0044		FILE No. ----			
	DESIGN	TZ	AUG 2013	SCALE	AS SHOWN	REV.
	CADD	--				
	CHECK	CN	AUG 2013	FIGURE C30		
	REVIEW	FJH	AUG 2013			



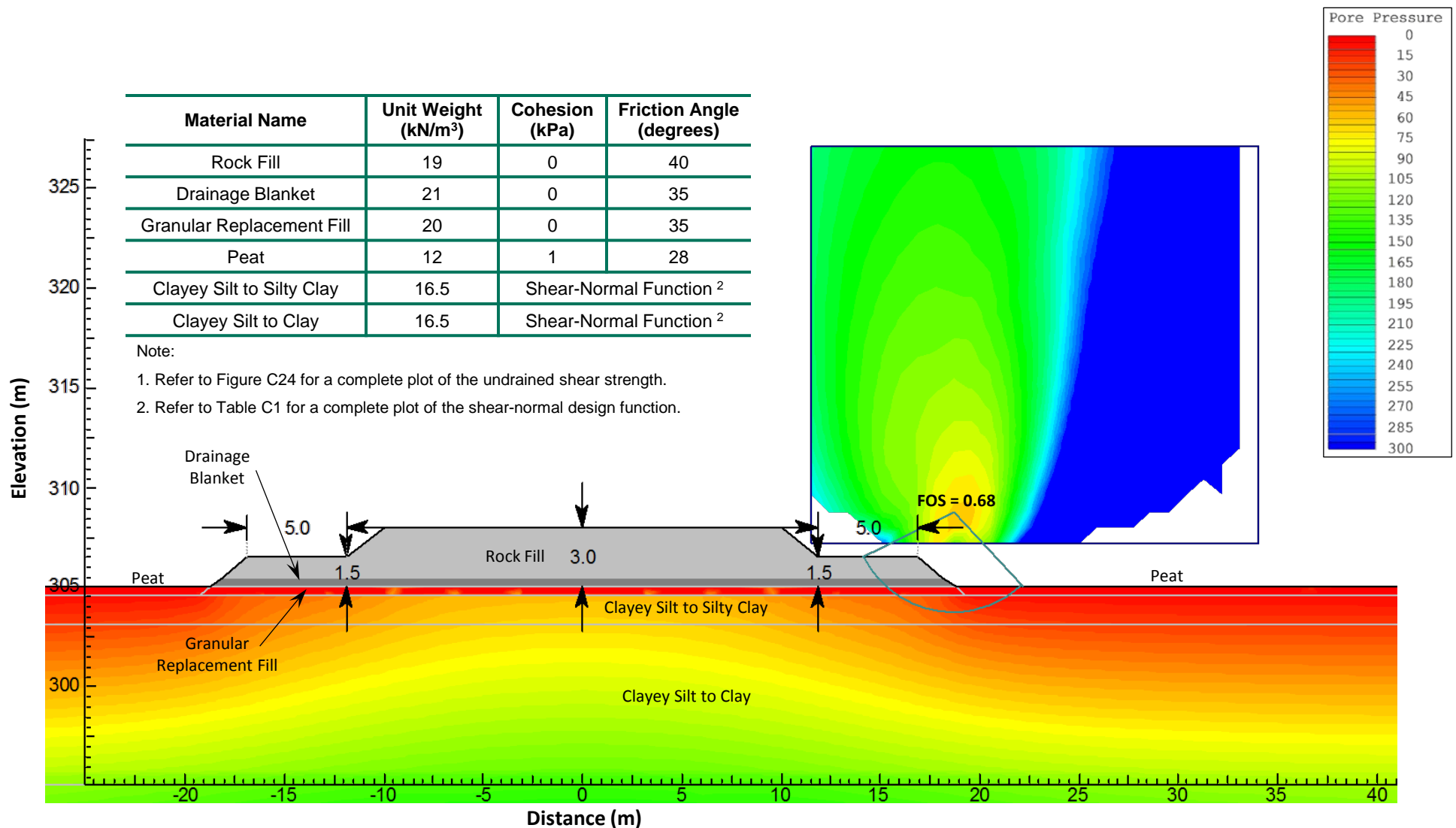
Notes:

- All dimensions are in meters.
- All replacement fill, drainage blanket and rock fill slopes are at 1.25H:1V.
- Stratigraphy based on Borehole H6-3; cohesionless deposits below the clayey silt to clay stratum have not been modeled.
- The proposed embankment height is approximately 3.0 m.

PROJECT		HIGHWAY 66 SWAMP CROSSING H6/H7			
TITLE		SLOPE STABILITY EFFECTIVE STRESS ANALYSIS (RAPID LOADING) STA 14+085			
PROJECT No. 10-1191-0044		FILE No. ----			
DESIGN	TZ	AUG 2013	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	CN	AUG 2013			
REVIEW	FJH	AUG 2013			



FIGURE C31

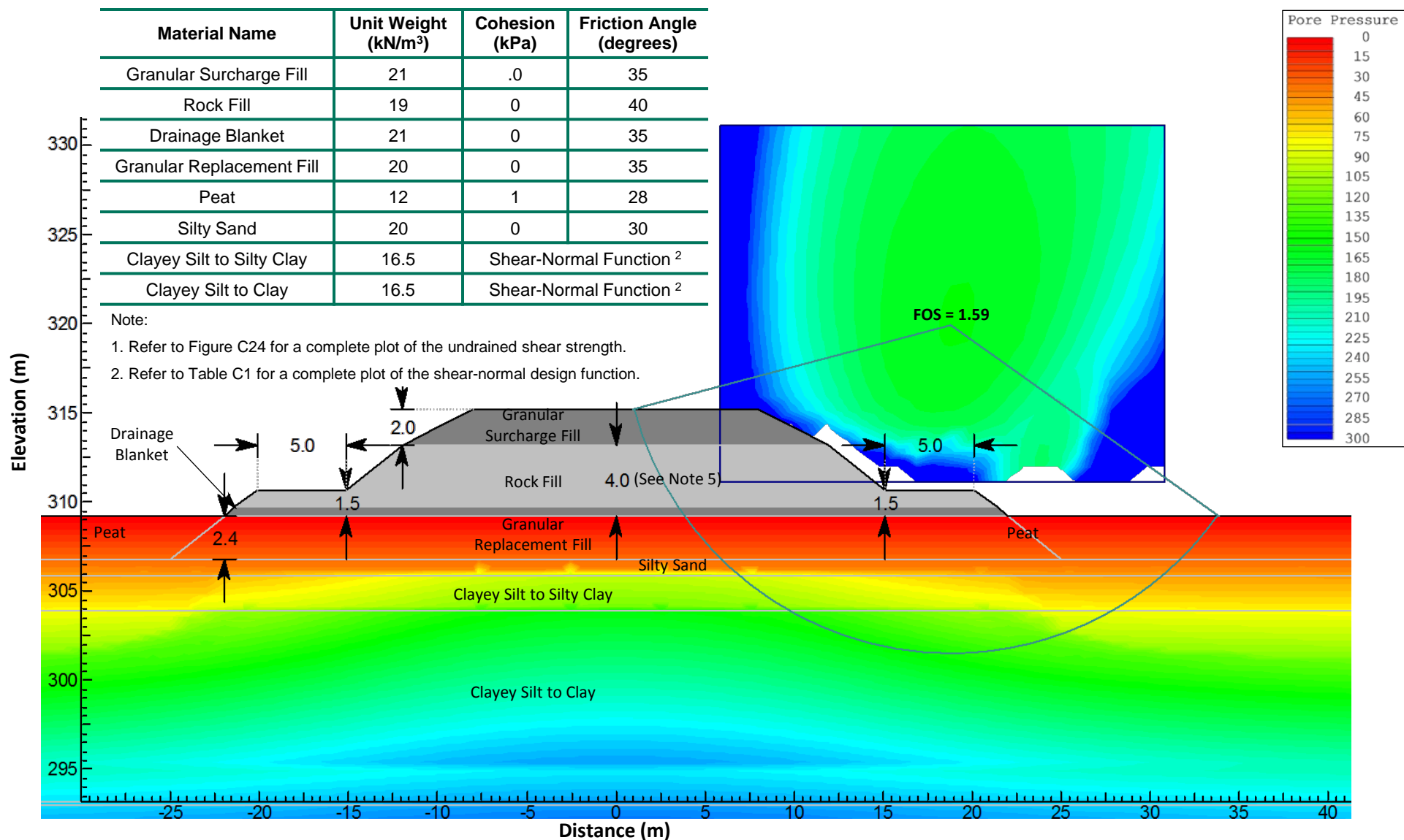


Notes:

1. All dimensions are in meters.
2. All replacement fill, drainage blanket and rock fill slopes are at 1.25H:1V.
3. Stratigraphy based on Borehole H7-5; cohesionless deposits below the clayey silt to clay stratum have not been modeled.
4. The proposed embankment height is approximately 3.0 m.

PROJECT		HIGHWAY 66 SWAMP CROSSING H6/H7			
TITLE		SLOPE STABILITY EFFECTIVE STRESS ANALYSIS (RAPID LOADING) STA 14+530			
		PROJECT No. 10-1191-0044	FILE No. ----		
DESIGN	TZ	AUG 2013	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	CN	AUG 2013	FIGURE C32		
REVIEW	FJH	AUG 2013			

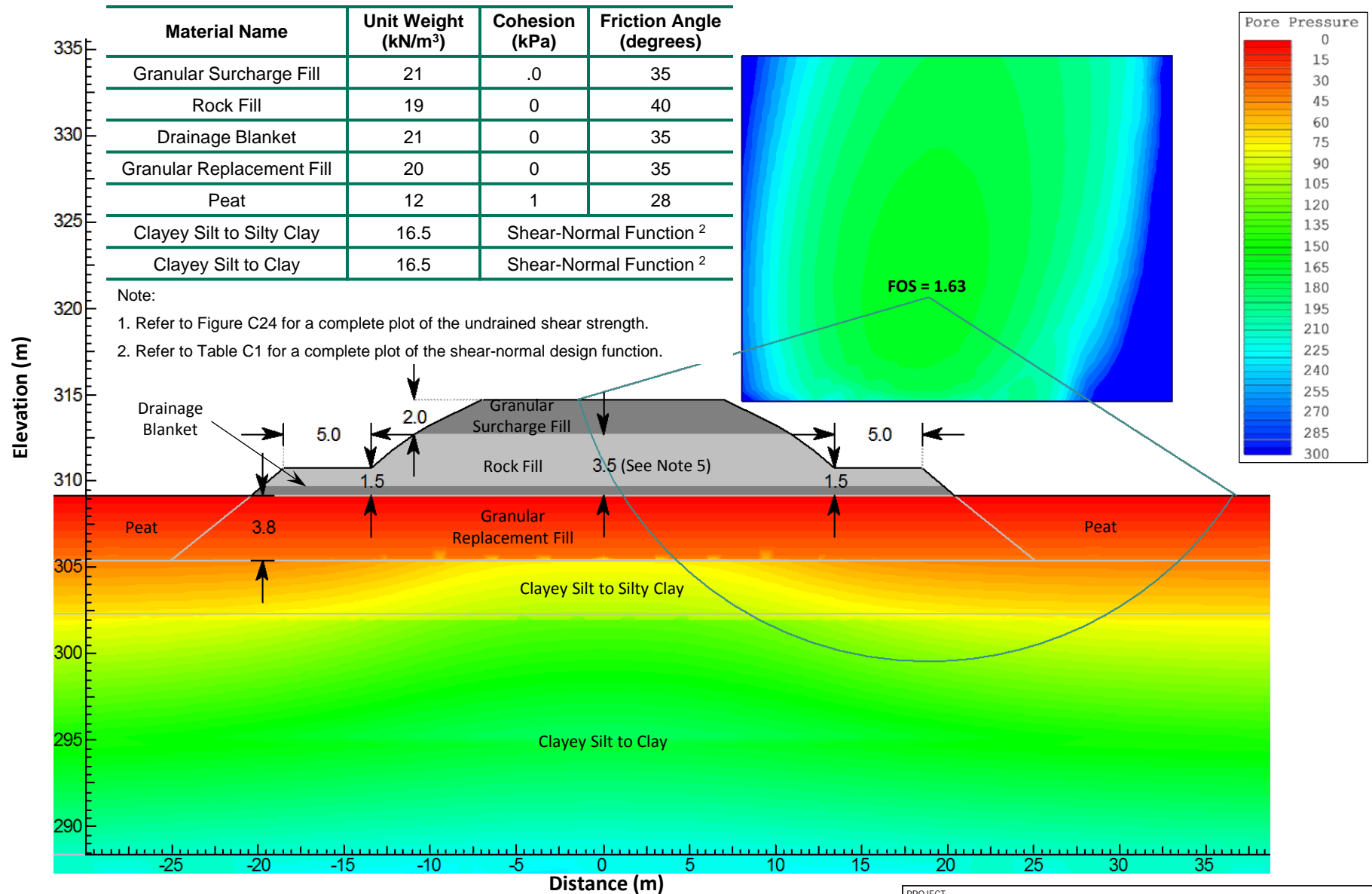




Notes:

1. All dimensions are in meters.
2. All granular replacement fill, drainage blanket and rock fill slopes are at 1.25H:1V.
3. Granular surcharge fill slopes are at 2H:1V.
4. Stratigraphy based on Borehole H6-1; cohesionless deposits below the clayey silt to clay stratum have not been modeled.
5. Final embankment height is 3.5 m, but was analyzed with an additional 0.5 m thick rock fill top-up to compensate for the large settlements during construction.

PROJECT		HIGHWAY 66 SWAMP CROSSING H6/H7			
TITLE		SLOPE STABILITY EFFECTIVE STRESS ANALYSIS - 1.5 m WICK DRAIN SPACING (STA 14+060)			
		PROJECT No. 10-1191-0044		FILE No. ----	
		DESIGN	TZ	AUG 2013	SCALE AS SHOWN
		CADD	--	AUG 2013	REV.
		CHECK	CN	AUG 2013	
		REVIEW	FJH	AUG 2013	
FIGURE C33					

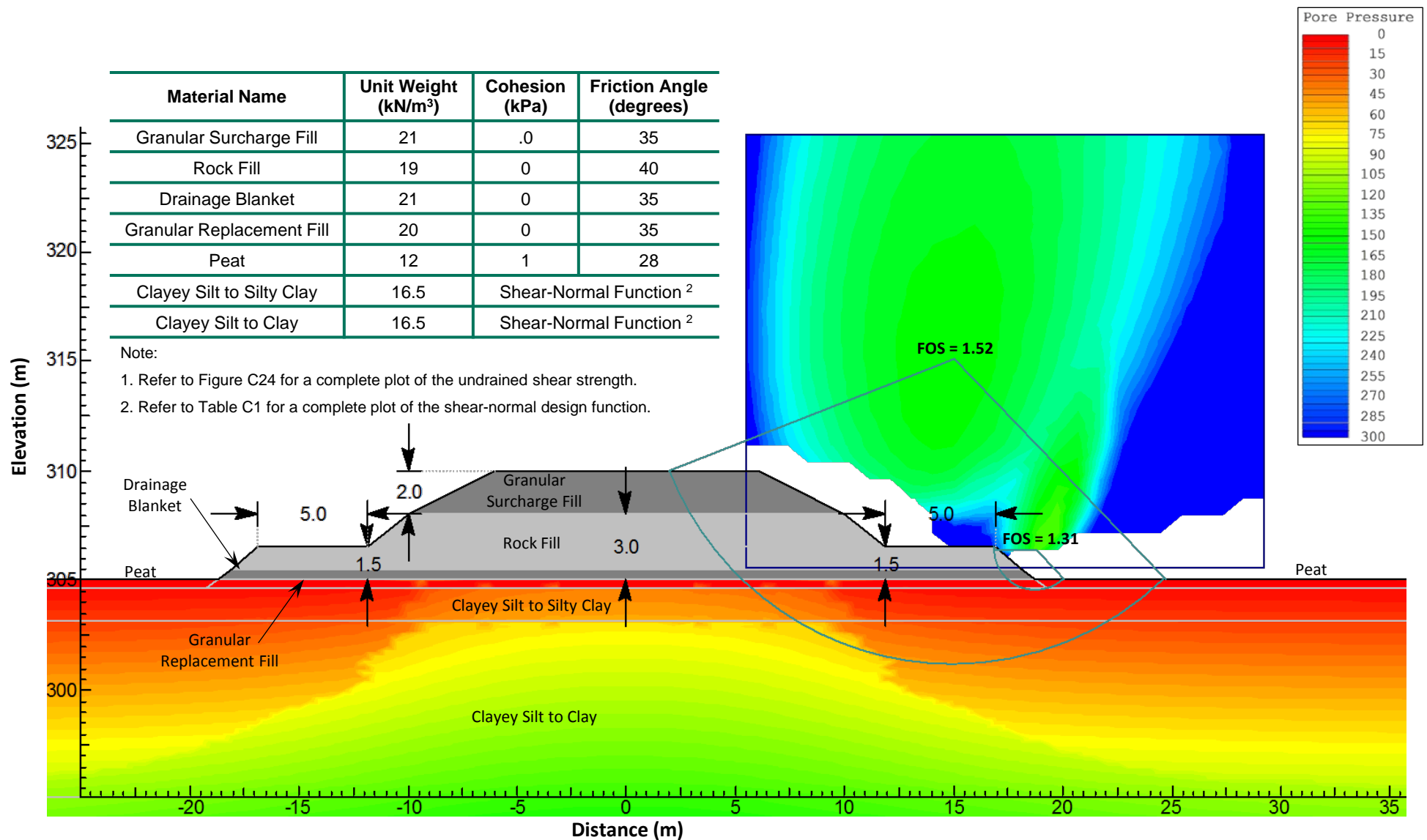


Notes:

1. All dimensions are in meters.
2. All granular replacement fill, drainage blanket and rock fill slopes are at 1.25H:1V.
3. Granular surcharge fill slopes are at 2H:1V.
4. Stratigraphy based on Borehole H6-3; cohesionless deposits below the clayey silt to clay stratum have not been modeled.
5. Final embankment height is 3.0 m, but was analyzed with an additional 0.5 m thick rock fill top-up to compensate for the large settlements during construction.

PROJECT		HIGHWAY 66 SWAMP CROSSING H6/H7	
TITLE		SLOPE STABILITY EFFECTIVE STRESS ANALYSIS - 1.5 m WICK DRAIN SPACING (STA 14+085)	
PROJECT No. 10-1191-0044		FILE No. ----	
DESIGN	TZ	AUG 2013	SCALE AS SHOWN
CADD	--		REV.
CHECK	CN	AUG 2013	FIGURE C34
REVIEW	FJH	AUG 2013	

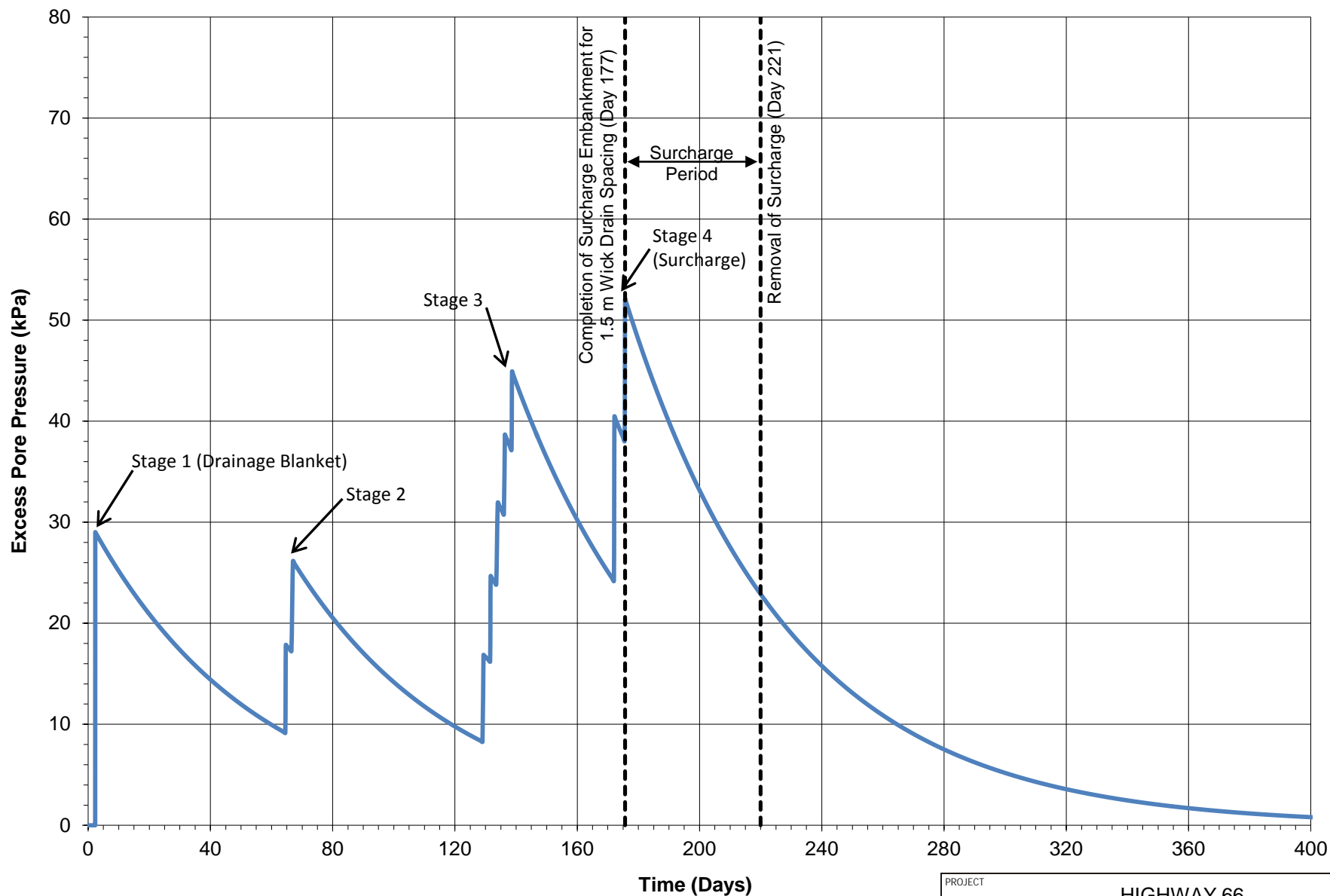




Notes:


1. All dimensions are in meters.
2. All replacement fill, drainage blanket and rock fill slopes are at 1.25H:1V.
3. Stratigraphy based on Borehole H7-5; cohesionless deposits below the clayey silt to clay stratum have not been modeled.
4. The proposed embankment height is approximately 3.0 m.

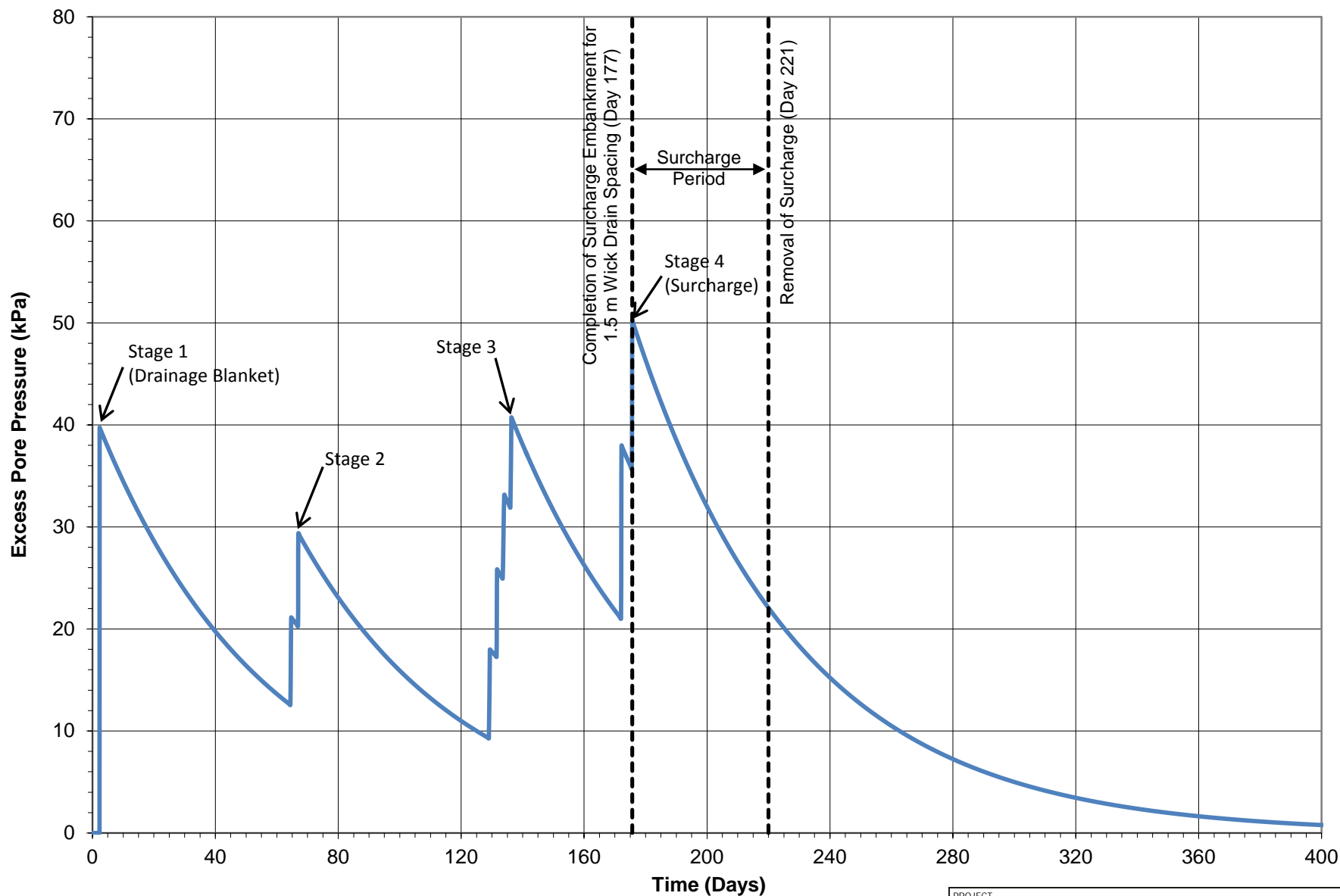
PROJECT		HIGHWAY 66 SWAMP CROSSING H6/H7			
TITLE		SLOPE STABILITY EFFECTIVE STRESS ANALYSIS - 1.5 m WICK DRAIN SPACING (STA 14+530)			
PROJECT No. 10-1191-0044		FILE No. ----			
DESIGN	TZ	AUG 2013	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	CN	AUG 2013			
REVIEW	FJH	AUG 2013			
Golder Associates		FIGURE C35			



Note:

1. Excess pore pressure plot corresponds to a point located below the embankment centerline at a depth of 7.8 m below existing ground surface, corresponding to Elevation 301.4 m.

PROJECT		HIGHWAY 66 SWAMP CROSSING H6/H7			
TITLE		EXCESS PORE PRESSURE RESPONSE STA 14+060			
		PROJECT No. 10-1191-0044		FILE No. ----	
		DESIGN	TZ	AUG 2013	SCALE AS SHOWN
		CADD	--		REV.
		CHECK	CN	AUG 2013	
		REVIEW	FJH	AUG 2013	
		FIGURE C36			

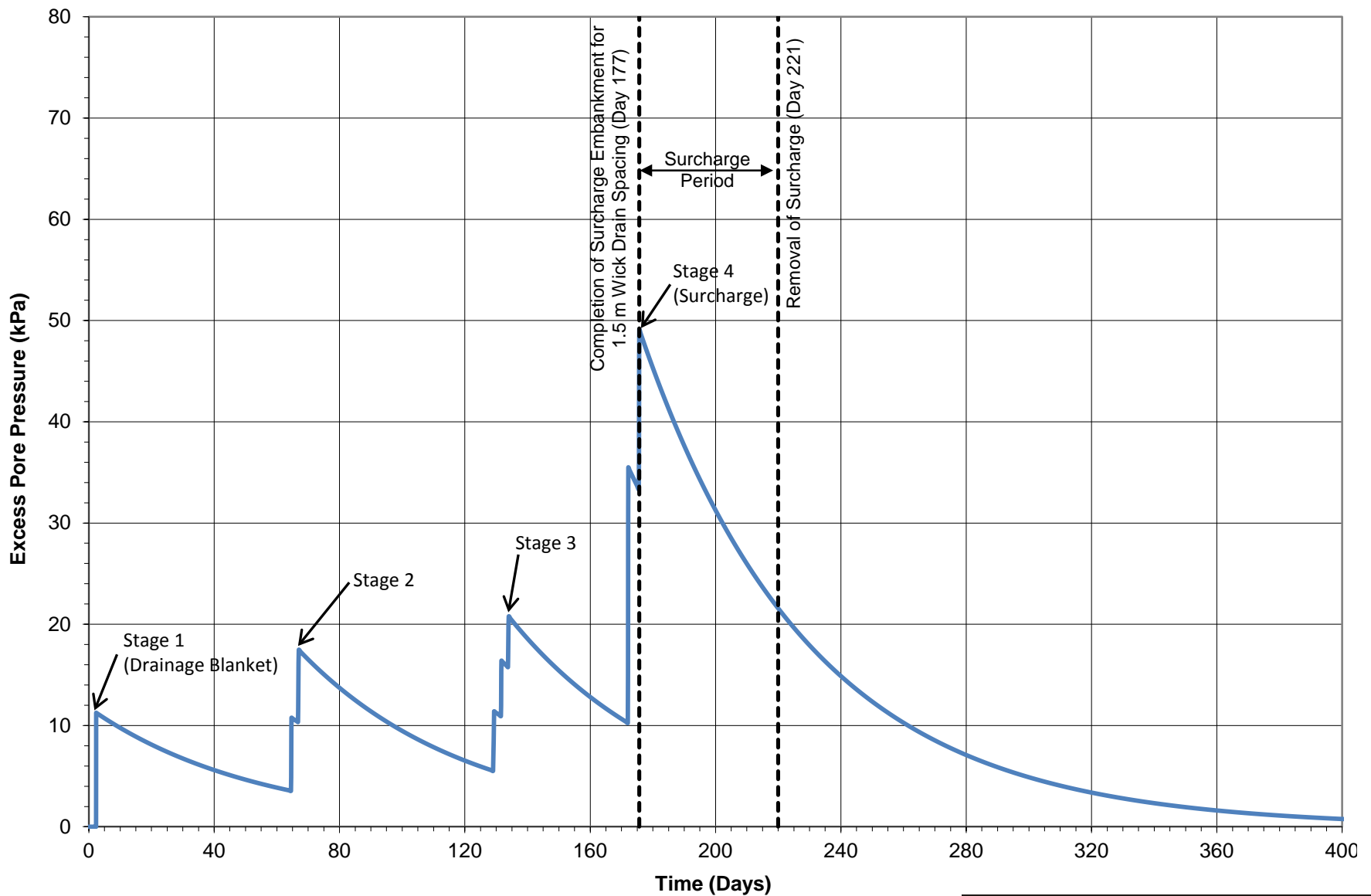


Note:

1. Excess pore pressure plot corresponds to a point located below the embankment centerline at depth of 10.0 m below existing ground surface, corresponding to Elevation 299.2 m.

PROJECT		HIGHWAY 66 SWAMP CROSSING H6/H7	
TITLE		EXCESS PORE PRESSURE RESPONSE STA 14+085	
PROJECT No. 10-1191-0044		FILE No. ----	
DESIGN	TZ	AUG 2013	SCALE AS SHOWN
CADD	--	--	REV.
CHECK	CN	AUG 2013	FIGURE C37
REVIEW	FJH	AUG 2013	





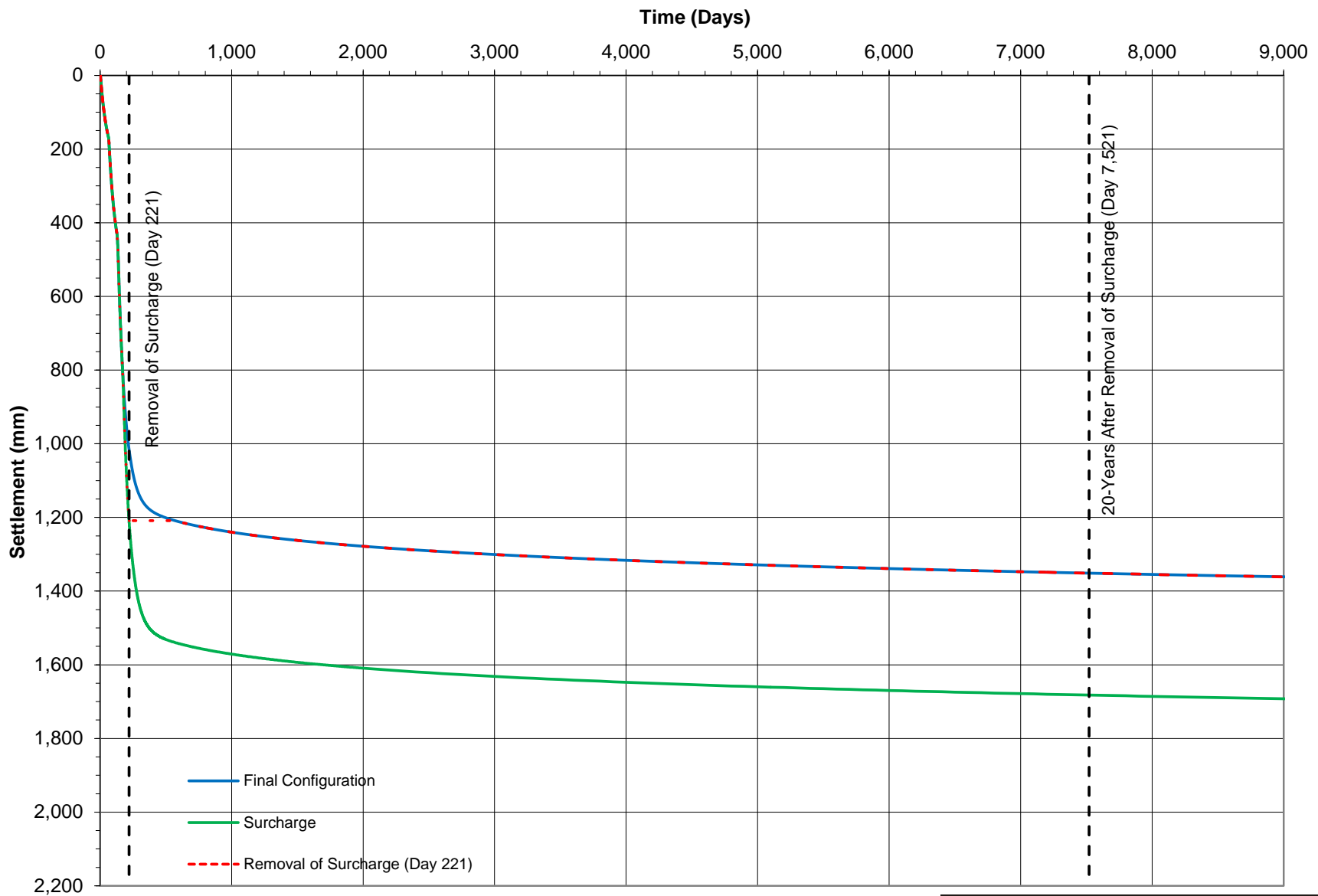
Note:

1. Excess pore pressure plot corresponds to a point located below the embankment centerline at a depth of 4.0 m below existing ground surface, corresponding to Elevation 301.0 m.

PROJECT		HIGHWAY 66 SWAMP CROSSING H6/H7			
TITLE		EXCESS PORE PRESSURE RESPONSE STA 14+530			
		PROJECT No. 10-1191-0044	FILE No. ----		
DESIGN	TZ	AUG 2013	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	CN	AUG 2013			
REVIEW	FJH	AUG 2013			



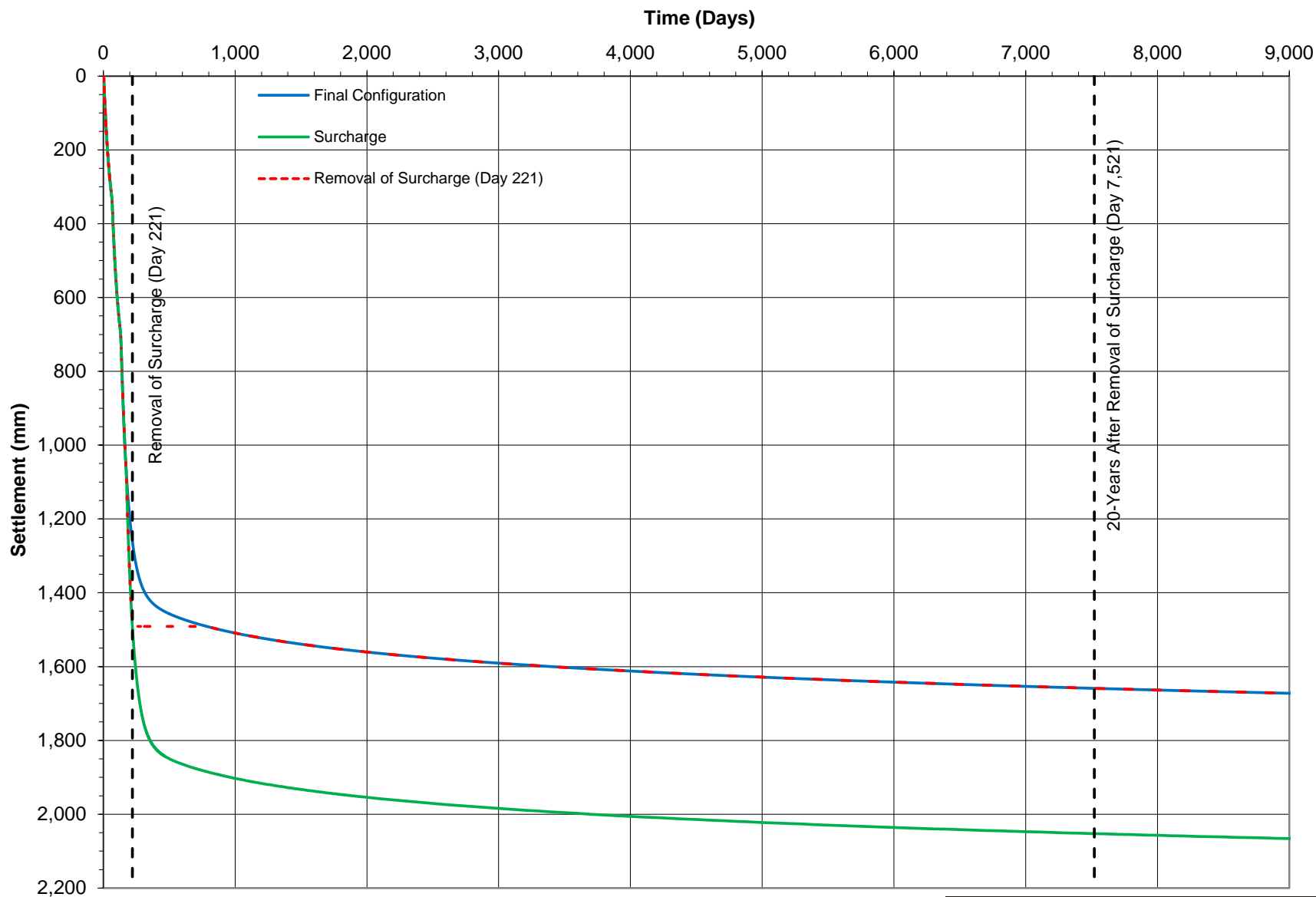
FIGURE C38



PROJECT		HIGHWAY 66 SWAMP CROSSING H6/H7			
TITLE		TIME-RATE CONSOLIDATION SETTLEMENT AT EMBANKMENT CENTRELINE STA 14+060			
PROJECT No. 10-1191-0044		FILE No. ----			
DESIGN	TZ	AUG 2013	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	CN	AUG 2013			
REVIEW	FJH	AUG 2013			



FIGURE C39




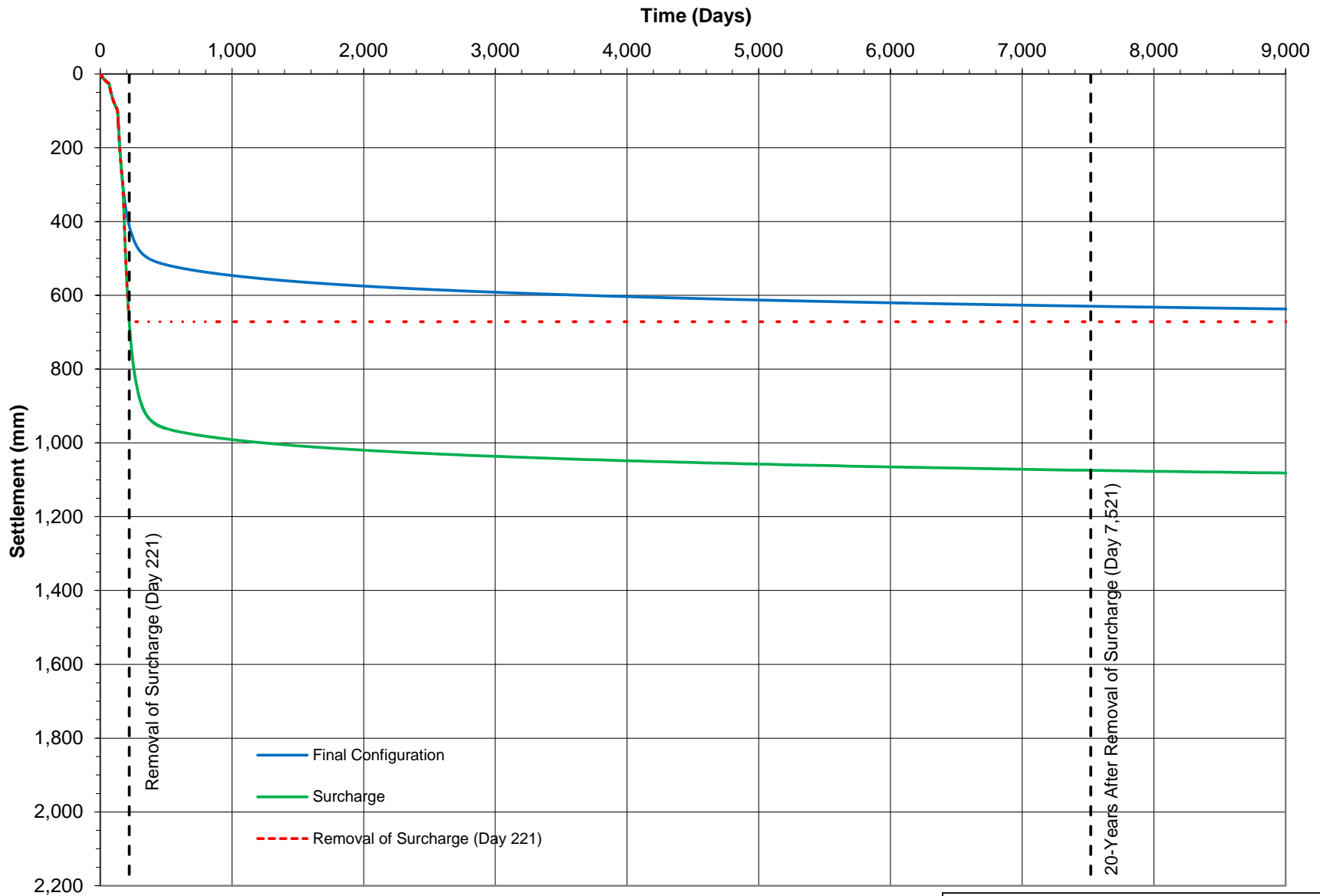
PROJECT		HIGHWAY 66 SWAMP CROSSING H6/H7			
TITLE		TIME-RATE CONSOLIDATION SETTLEMENT AT EMBANKMENT CENTRELINE STA 14+085			
		PROJECT No. 10-1191-0044		FILE No. ----	
		DESIGN	TZ	AUG 2013	SCALE AS SHOWN
		CADD	--		REV.
		CHECK	CN	AUG 2013	
		REVIEW	FJH	AUG 2013	

FIGURE C40



PROJECT		HIGHWAY 66 SWAMP CROSSING H6/H7			
TITLE		TIME-RATE CONSOLIDATION SETTLEMENT AT EMBANKMENT CENTRELINE STA 14+530			
PROJECT No. 10-1191-0044		FILE No. ----			
DESIGN	TZ	AUG 2013	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	CN	AUG 2013			
REVIEW	FJH	AUG 2013			

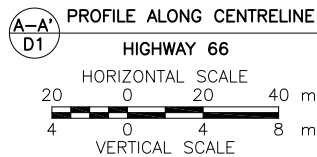
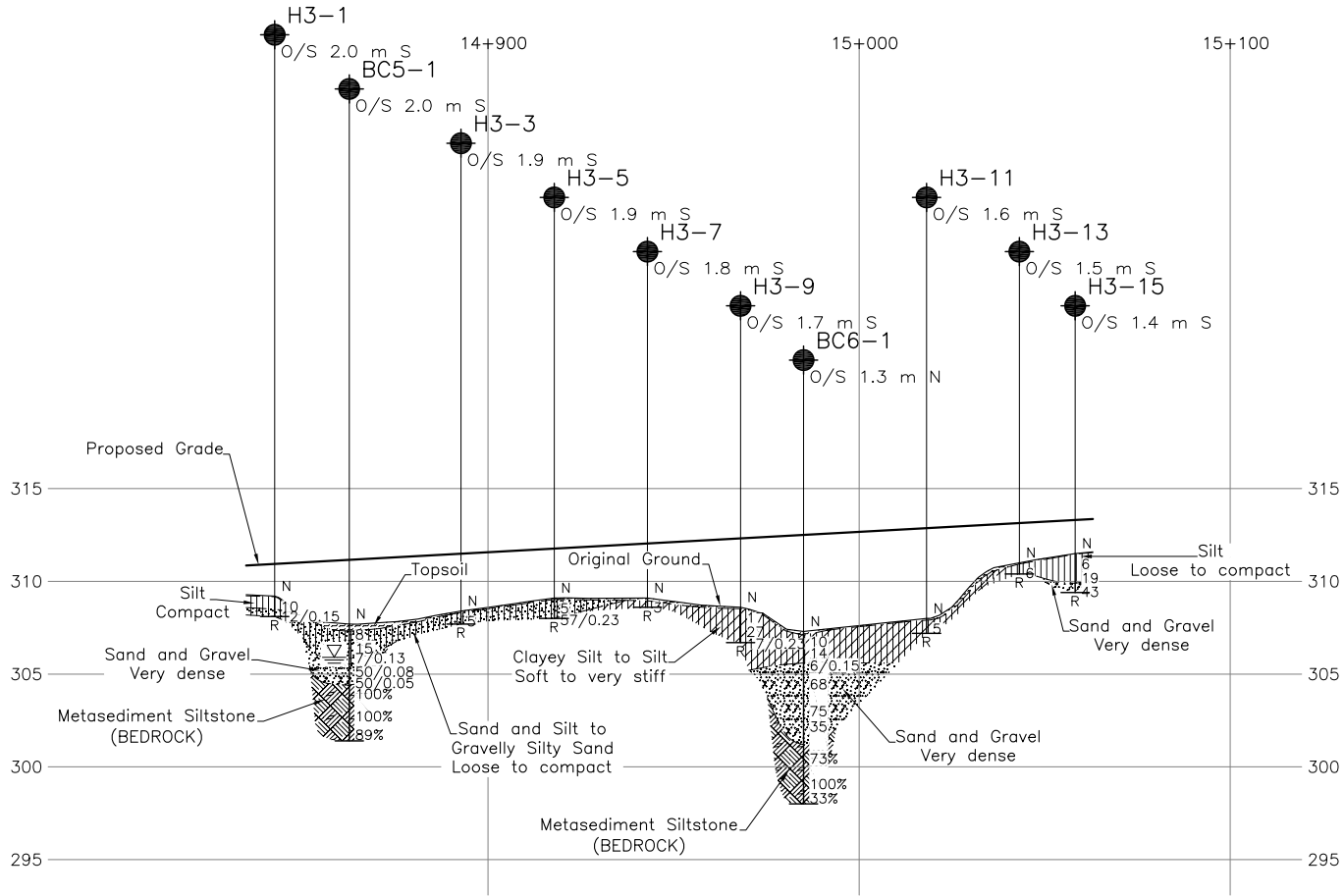
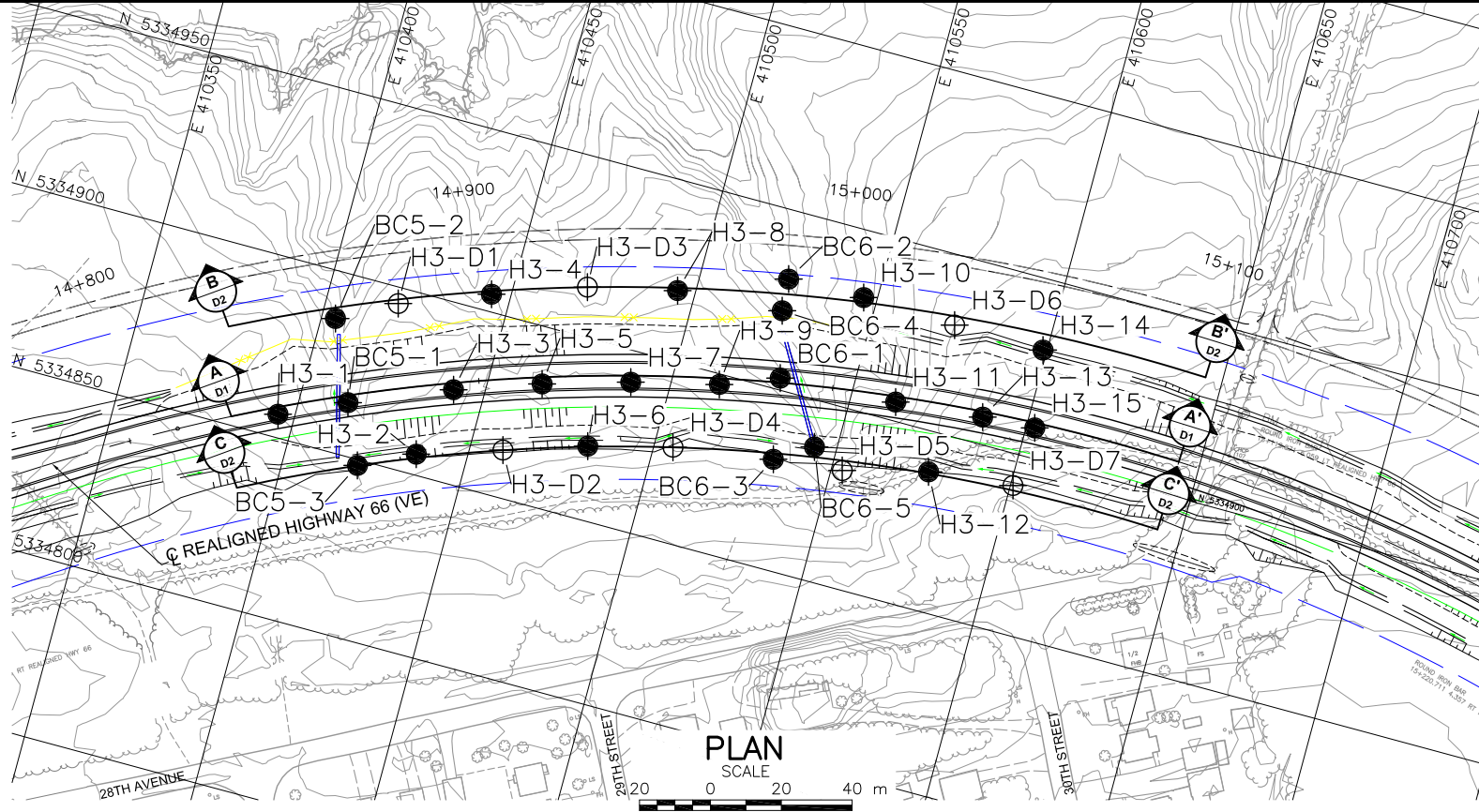


FIGURE C41



APPENDIX D

Highway 66 – STA 14+840 to 15+060 (High Fill H3)



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

CONT No.
GWP No. 5091-07-00

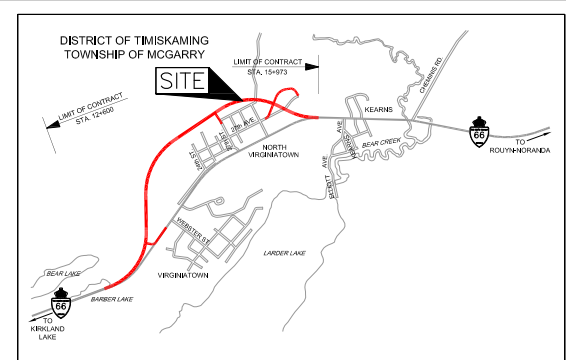


HIGHWAY 66
HWY 66 - STA 14+840 TO 15+060
BOREHOLE LOCATIONS AND
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



LEGEND

- Borehole
- ⊕ Dynamic Cone Penetration Test
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- R Refusal
- ≡ WL upon completion of drilling

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.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by MMM, drawing file nos. H3211009D16 ROLL PLAN-ULTIMATE and PDR.dwg, received DEC 3, 2012. Keyplan drawing file nos. H3211009G02 received JAN 24, 2013.

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
BC5-1	307.7	5334864.8	410413.5
BC5-2	306.6	5334886.8	410403.9
BC5-3	309.0	5334848.4	410420.7
BC6-1	308.8	5334903.3	410529.4
BC6-2	306.8	5334930.9	410524.4
BC6-3	305.5	5334880.7	410533.5
BC6-4	305.7	5334921.8	410525.0
BC6-5	307.6	5334887.1	410543.8
H3-1	309.2	5334856.5	410395.3
H3-2	309.6	5334855.8	410435.9
H3-3	308.4	5334876.1	410441.3
H3-4	306.0	5334904.9	410444.6
H3-5	309.1	5334884.2	410465.0
H3-6	309.8	5334870.6	410482.0
H3-7	309.1	5334891.2	410489.0
H3-8	307.8	5334919.6	410495.0
H3-9	308.6	5334897.1	410513.3
H3-10	307.3	5334931.4	410546.2
H3-11	308.0	5334905.3	410562.6
H3-12	310.9	5334888.7	410576.7
H3-13	311.0	5334907.6	410587.4
H3-14	310.5	5334930.3	410598.9
H3-15	311.5	5334908.5	410602.4
H3-D1	307.5	5334895.5	410419.9
H3-D2	309.5	5334863.3	410459.2
H3-D3	306.9	5334913.9	410470.2
H3-D4	309.7	5334876.6	410505.3
H3-D5	307.7	5334882.7	410553.1
H3-D6	309.7	5334930.6	410573.0
H3-D7	311.5	5334891.2	410600.7



NO.	DATE	BY	REVISION
Geocres No. 32D-17			
HWY. 66	PROJECT NO. 10-1191-0044		DIST.
SUBM'D. TR	CHKD.	DATE: DEC 2013	SITE:
DRAWN: JJJ	CHKD. SEMC	APPD. JMAC	DWG. D1

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

CONT No.
GWP No. 5091-07-00

HIGHWAY 66
HWY 66 - STA 14+840 TO 15+060
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

LEGEND

- Borehole
- Dynamic Cone Penetration Test
- Seal
- Piezometer
- Standard Penetration Test Value
- Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- R Refusal
- WL upon completion of drilling
- WL in piezometer, measured on Nov. 15, 2012

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
BC5-2	306.6	5334886.8	410403.9
BC5-3	309.0	5334848.4	410420.7
BC6-2	306.8	5334930.9	410524.4
BC6-3	305.5	5334880.7	410533.5
BC6-5	307.6	5334887.1	410543.8
H3-2	309.6	5334855.8	410435.9
H3-4	306.0	5334904.9	410444.6
H3-6	309.8	5334870.6	410482.0
H3-8	307.8	5334919.6	410495.0
H3-10	307.3	5334931.4	410546.2
H3-12	310.9	5334888.7	410576.7
H3-14	310.5	5334930.3	410598.9
H3-D1	307.5	5334895.5	410419.9
H3-D2	309.5	5334863.3	410459.2
H3-D3	306.9	5334913.9	410470.2
H3-D4	309.7	5334876.6	410505.3
H3-D5	307.7	5334882.7	410553.1
H3-D6	309.7	5334930.6	410573.0
H3-D7	311.5	5334891.2	410600.7

NOTES

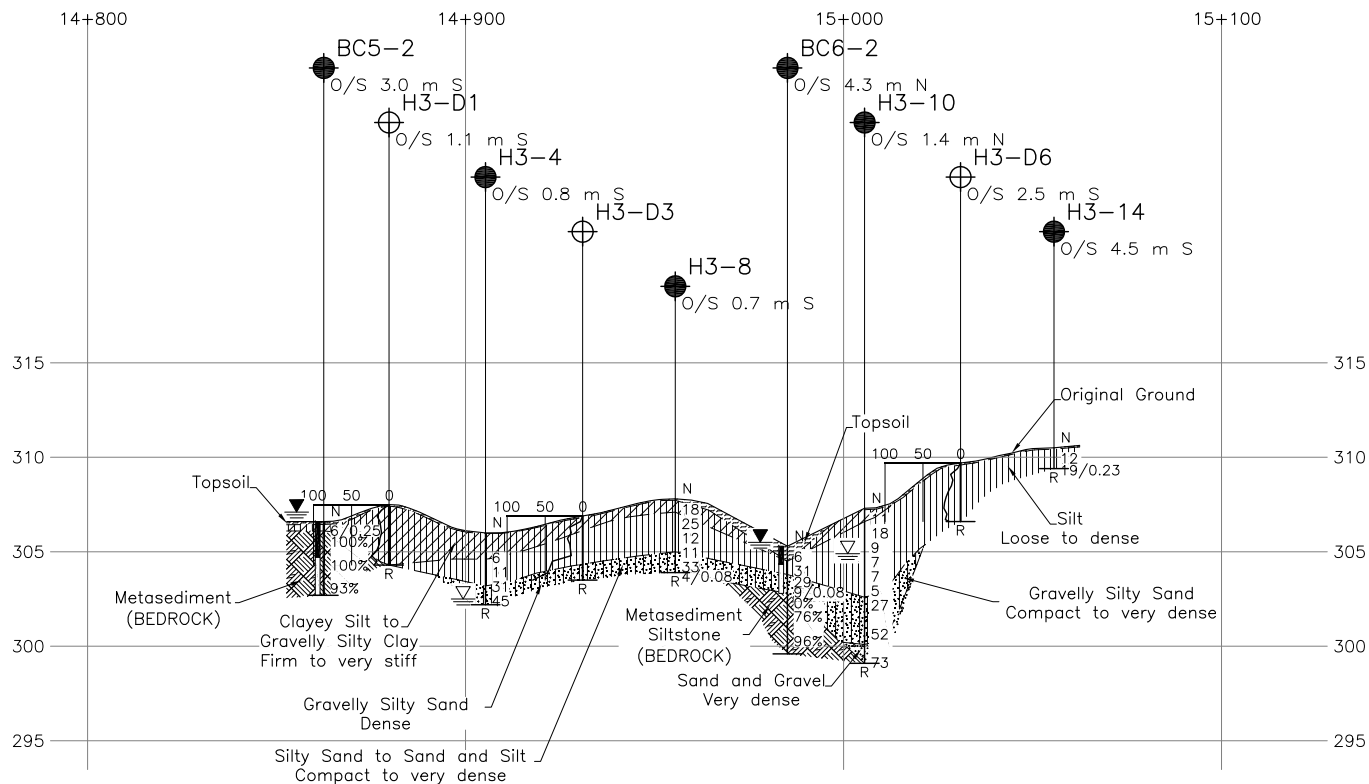
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

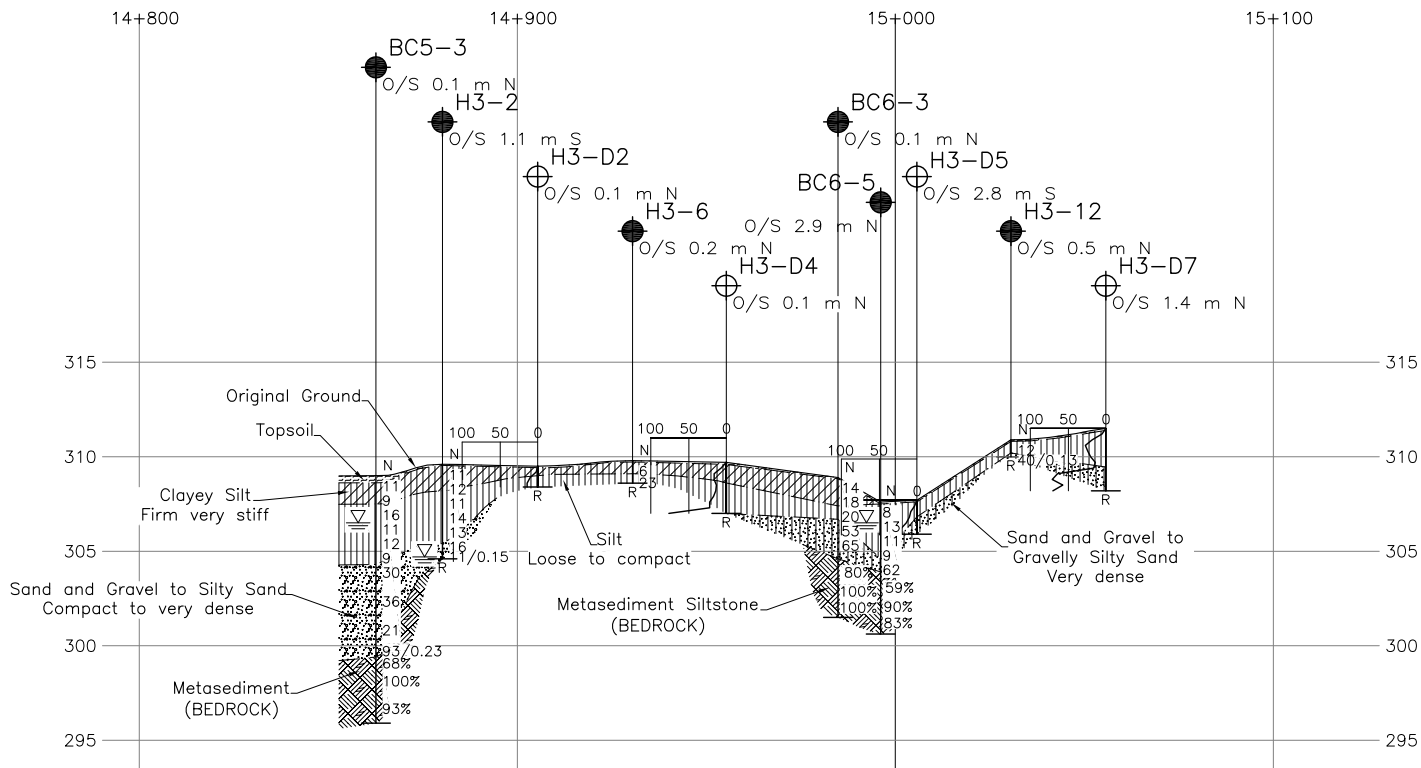
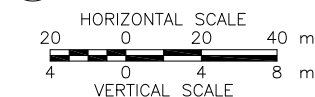
REFERENCE

Base plans provided in digital format by MMM, drawing file nos. H3211009D16 ROLL PLAN-ULTIMATE and PDR.dwg, received DEC 3, 2012. Keyplan drawing file nos. H3211009G02 received JAN 24, 2013.



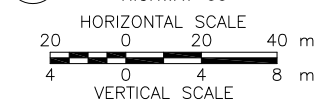
PROFILE ALONG NORTH TOE

HIGHWAY 66



PROFILE ALONG SOUTH TOE

HIGHWAY 66



NO.	DATE	BY	REVISION
Geocres No. 32D-17			
HWY. 66	PROJECT NO. 10-1191-0044		DIST.
SUBM'D. TR	CHKD.	DATE: DEC 2013	SITE:
DRAWN: JJJ	CHKD. SEMC	APPD. JMAC	DWG. D2

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No H3-1				1 OF 1 METRIC												
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334856.5; E 410395.3</u>				ORIGINATED BY <u>MT</u>												
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>MT</u>												
DATUM <u>GEODETIC</u>		DATE <u>August 9, 2011</u>				CHECKED BY <u>SEMC</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	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
309.2	GROUND SURFACE							20	40	60	80	100						
0.0	TOPSOIL		1	SS	10		309											
308.5	SILT, trace clay Compact Grey Moist		2	SS	12/0.15												14	43 36 7
308.1	SAND and SILT, some gravel, trace to some clay Compact Grey Moist																	
1.1	END OF BOREHOLE AUGER REFUSAL																	
Note: 1. Borehole dry upon completion of drilling.																		

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044		RECORD OF BOREHOLE No H3-2				1 OF 1 METRIC											
G.W.P. 5091-07-00		LOCATION N 5334855.8; E 410435.9				ORIGINATED BY MT											
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT											
DATUM GEODETIC		DATE August 9, 2011				CHECKED BY SEMC											
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									
309.6	GROUND SURFACE							20	40	60	80	100					
8.9	TOPSOIL		1	SS	11		309										
	CLAYEY SILT, trace sand, trace rootlets		2	SS	12												
	Stiff																
	Brown to grey																
	Moist																
308.2	SILT, trace to some clay, trace sand		3	SS	11		308										
1.4	Compact																
	Grey		4	SS	14		307										
	Wet																
			5	SS	13		306										
305.4	Silty SAND, some gravel, trace clay		6	SS	16												
4.2	Compact																
	Brown		7	SS	11/0.15		305										
	Wet																
304.6	END OF BOREHOLE AUGER REFUSAL																
5.0	Note: 1. Water level at a depth of 4.9 m below ground surface (Elev. 304.7 m) upon completion of drilling.																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044		RECORD OF BOREHOLE No H3-3				1 OF 1 METRIC						
G.W.P. 5091-07-00		LOCATION N 5334876.1; E 410441.3				ORIGINATED BY MT						
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT						
DATUM GEODETIC		DATE August 9, 2011				CHECKED BY SEMC						
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	20 40 60 80 100	W _p W W _L			
308.4	GROUND SURFACE											
0.0	TOPSOIL		1a									
0.1	SAND and SILT, trace to some gravel, trace clay		1b	SS	5		308					9 49 37 5
307.7	Loose to compact Brown Moist											
0.7	Spoon attempted at 0.7 m depth, bouncing.											
	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING)											
	Notes: 1. Borehole dry upon completion of drilling. 2. Auger refusal encountered at 0.5 m depth.											

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044				RECORD OF BOREHOLE No H3-4				1 OF 1 METRIC									
G.W.P. 5091-07-00				LOCATION N 5334904.9; E 410444.6				ORIGINATED BY MT									
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT									
DATUM GEODETIC				DATE August 8, 2011				CHECKED BY SEMC									
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									
306.0	GROUND SURFACE							20	40	60	80	100					
0.0	TOPSOIL		1	SS	9												
	CLAYEY SILT, trace sand Firm to stiff Grey Moist		2	SS	6												
304.6																	
1.4	SILT, trace to some clay, trace sand Compact to dense Wet Grey		3	SS	11												
303.2			4	SS	31												
2.8	Gravelly Silty SAND, trace clay Dense Brown Wet		5	SS	45												
302.2	Spoon attempted at 3.8 m depth.																
3.8	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING)																
	Note: 1. Water level at a depth of 3.5 m below ground surface (Elev. 302.5 m) upon completion of drilling.																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044				RECORD OF BOREHOLE No H3-5				1 OF 1 METRIC									
G.W.P. 5091-07-00				LOCATION N 5334884.2; E 410465.0				ORIGINATED BY MT									
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT									
DATUM GEODETIC				DATE August 8, 2011				CHECKED BY SEMC									
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									
309.1	GROUND SURFACE							20	40	60	80	100					
0.7	TOPSOIL		1	SS	5		309									NP	
308.4	SILT, trace to some clay, trace sand Loose Grey and brown Moist																
308.0	Gravelly Silty SAND, trace clay Very dense Brown and grey Moist		2	SS	57/0.23		308										26 43 26 5
1.1	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING)																
Notes: 1. Borehole dry upon completion of drilling. 2. Auger refusal encountered at 0.8 m depth.																	

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044				RECORD OF BOREHOLE No H3-6				1 OF 1 METRIC									
G.W.P. 5091-07-00				LOCATION N 5334870.6; E 410482.0				ORIGINATED BY MT									
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT									
DATUM GEODETIC				DATE August 8, 2011				CHECKED BY SEMC									
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									
309.8	GROUND SURFACE							20	40	60	80	100					
0.7	TOPSOIL		1	SS	6												
309.1	CLAYEY SILT, trace sand, trace rootlets																
0.7	Firm																
308.6	Grey Moist		2	SS	23												
1.2	SILT, some clay, trace sand																
	Loose																
	Grey																
	Wet																
	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING)																
Notes: 1. Borehole dry upon completion of drilling. 2. Auger refusal encountered at 1.1 m depth.																	

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No H3-7				1 OF 1 METRIC							
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334891.2; E 410489.0</u>				ORIGINATED BY <u>MT</u>							
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>MT</u>							
DATUM <u>GEODETIC</u>		DATE <u>August 8, 2011</u>				CHECKED BY <u>SEMC</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		WATER CONTENT (%)		γ	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	W _p W W _L	20 40 60			
309.1	GROUND SURFACE												
0.0	TOPSOIL		1a	SS	3		309						
308.6	CLAYEY SILT, trace sand, trace rootlets		1b										
0.5	Soft Brown Moist												
	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING)												
	Notes: 1. Borehole dry upon completion of drilling. 2. Auger refusal encountered at 0.4 m depth.												

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT		10-1191-0044		RECORD OF BOREHOLE No H3-8		1 OF 1 METRIC								
G.W.P.		5091-07-00		LOCATION		N 5334919.6; E 410495.0								
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers								
DATUM		GEODETIC		DATE		August 8, 2011								
						ORIGINATED BY MT								
						COMPILED BY MT								
						CHECKED BY SEMC								
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						
307.8	GROUND SURFACE							20 40 60 80 100	20 40 60					
0.0	TOPSOIL		1a											
0.1	CLAYEY SILT, trace sand		1b	SS	18									
307.1	Very stiff Grey Moist													
0.7	SILT, trace to some clay, trace sand		2	SS	25									
	Compact Grey Moist to wet													
			3	SS	12									
			4	SS	11									
305.0	SAND and SILT, some gravel, trace clay													
2.8	Dense Brown Wet		5	SS	33									
			6	SS	40.08									
303.9	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING)													
3.9	Notes: 1. Borehole dry upon completion of drilling. 2. Auger refusal encountered at 3.8 m depth.													

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044		RECORD OF BOREHOLE No H3-9				1 OF 1 METRIC								
G.W.P. 5091-07-00		LOCATION N 5334897.1; E 410513.3				ORIGINATED BY MT								
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT								
DATUM GEODETIC		DATE August 7, 2011				CHECKED BY SEMC								
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 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED		W _p	W	W _L	γ	GR SA SI CL
308.6	GROUND SURFACE							20 40 60 80 100	20 40 60					
0.7	TOPSOIL		1	SS	17		308							
	CLAYEY SILT, trace sand		2	SS	27									0 1 78 21
	Very stiff													
	Grey													
	Moist													
306.7	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING)		3	SS	17/0.23		307							
1.9	Notes:													
	1. Borehole dry upon completion of drilling.													
	2. Auger refusal encountered at 1.6 m depth.													

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044			RECORD OF BOREHOLE No H3-10				1 OF 1 METRIC									
G.W.P. 5091-07-00			LOCATION N 5334931.4; E 410546.2				ORIGINATED BY MT									
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT									
DATUM GEODETIC			DATE August 7, 2011				CHECKED BY SEMC									
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								
307.3	GROUND SURFACE															
0.7	TOPSOIL		1	SS	11											
306.6	CLAYEY SILT, trace sand Stiff Grey Moist		2	SS	18											
0.7	SILT, some clay Loose to compact Grey Wet		3	SS	9											
			4	SS	7											
			5	SS	7											
			6	SS	5											
302.6	Gravelly Silty SAND, trace clay Compact to very dense Brown Wet		7	SS	27											
4.7			8	SS	52											
300.1	SAND and GRAVEL, trace to some silt, trace clay Very dense Brown Wet		9	SS	73											
7.2																
299.1	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING)															
8.2	Note: 1. Water level at a depth of 2.4 m below ground surface (Elev. 304.9 m) upon completion of drilling. 2. Auger refusal at 7.9 m depth.															

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No H3-11				1 OF 1 METRIC						
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334905.3; E 410562.6</u>				ORIGINATED BY <u>MT</u>						
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>MT</u>						
DATUM <u>GEODETIC</u>		DATE <u>August 7, 2011</u>				CHECKED BY <u>SEMC</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 (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L			20 40 60
308.0	GROUND SURFACE											
0.0	TOPSOIL		1a									
0.1	CLAYEY SILT, trace sand		1b	SS	5							
307.2	Firm											
0.8	Grey Moist											
	Spoon attempted at 0.8 m depth.											
	END OF BOREHOLE											
	SPOON AND AUGER REFUSAL (HAMMER BOUNCING)											
	Note:											
	1. Borehole dry upon completion of drilling.											


SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044				RECORD OF BOREHOLE No H3-12				1 OF 1 METRIC									
G.W.P. 5091-07-00				LOCATION N 5334888.7; E 410576.7				ORIGINATED BY MT									
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT									
DATUM GEODETIC				DATE August 6, 2011				CHECKED BY SEMC									
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									
310.9	GROUND SURFACE																
0.0	TOPSOIL		1a														
0.1	SILT, trace clay, trace rootlets		1b	SS	12											0 9 82 9	
310.2	Compact Brown Moist		2	SS	40/0.13												
0.9	SAND and GRAVEL, some silt Very dense Brown and grey Moist																
	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING)																
Notes: 1. Borehole dry upon completion of drilling. 2. Auger refusal at 0.8 m depth.																	

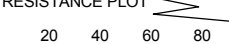
SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT		10-1191-0044				RECORD OF BOREHOLE No H3-13				1 OF 1 METRIC							
G.W.P.		5091-07-00		LOCATION		N 5334907.6; E 410587.4				ORIGINATED BY		MT					
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY		MT					
DATUM		GEODETIC		DATE		August 6, 2011				CHECKED BY		SEMC					
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									
311.0	GROUND SURFACE																
0.0	TOPSOIL		1a														
0.1	SILT, some clay, some sand, trace gravel Loose Brown Moist		1b	SS	6												1 13 74 12
310.4	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING)																
0.6																	
Note: 1. Borehole dry upon completion of drilling.																	

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No H3-14				1 OF 1 METRIC								
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334930.3; E 410598.9</u>				ORIGINATED BY <u>MT</u>								
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>MT</u>								
DATUM <u>GEODETIC</u>		DATE <u>August 6, 2011</u>				CHECKED BY <u>SEMC</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		W _p	W	W _L	γ	GR SA SI CL
310.5	GROUND SURFACE							20 40 60 80 100						
0.7	TOPSOIL		1	SS	12		310							
	SILT, some clay, trace sand Compact Grey Moist													
309.4			2	SS	19/0.23									0 5 81 14
1.1	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING)													
	Note: 1. Borehole dry upon completion of drilling.													

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044			RECORD OF BOREHOLE No H3-15				1 OF 1 METRIC				
G.W.P. 5091-07-00			LOCATION N 5334908.5; E 410602.4				ORIGINATED BY MT				
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT				
DATUM GEODETIC			DATE August 6, 2011				CHECKED BY SEMC				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
311.5	GROUND SURFACE										
0.0	TOPSOIL		1	SS	6		311				
	SILT, some gravel, trace to some clay, trace sand Loose to compact Brown to grey Moist		2	SS	19						12 3 79 6
309.9							310				
1.6	SAND and GRAVEL, some silt Very dense Brown		3	SS	43						
309.4											
2.1	Moist										
	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING)										
	Note: 1. Borehole dry upon completion of drilling.										

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044		RECORD OF BOREHOLE No BC5-1				1 OF 2 METRIC						
G.W.P. 5091-07-00		LOCATION N 5334864.8; E 410413.5				ORIGINATED BY EHS						
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring				COMPILED BY MT						
DATUM GEODETIC		DATE August 16, 2011				CHECKED BY SEMC						
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	20 40 60 80 100	W _p W W _L			20 40 60
307.7	GROUND SURFACE											
0.0	TOPSOIL		1	SS	8							
307.3												
0.4	SILT and SAND, trace clay, trace gravel Very dense Brown to grey Wet		2	SS	15							
			3	SS	7/0.13							
305.5												
2.2	SAND and GRAVEL Very dense Brown to grey Wet		4	SS	50/0.08							
304.4			5	SS	50/0.05							
3.3	METASEDIMENT (SILTSTONE) (BEDROCK) Bedrock cored from depths of 3.3 m to 6.3 m For bedrock coring details, refer to Record of Drillhole C5-1		1	RC	REC 100%							
			2	RC	REC 100%							
			3	RC	REC 100%							
301.4												
6.3	END OF BOREHOLE Note: 1. Water level at a depth of 1.8 m below ground surface (Elev. 305.9 m) on August 18, 2011. 2. Spoon bouncing at Samples 3, 4 and 5											

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC5-1

SHEET 2 OF 2

LOCATION: N 5334864.8 ; E 410413.5

DRILLING DATE: August 16, 2011

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D50

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	RECOVERY TOTAL CORE %	R.Q.D. % SOLID CORE %	FRACT. INDEX METRES	B Angle	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s	Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE		304.4															
	NW	METASEDIMENT (SILTSTONE)		3.3															
4		Very strong Very fine grained Fresh Dark grey			1	GREY 100%						JN,IR,Ro							
		Vertical joint encountered between 3.3 m and 4.5 m depth.										JN,UN,SM							
5	August 17, 2011 NQ Coring				2	GREY 100%						JN,IR,Ro							
6		Vertical joint encountered between 6.0 m and 6.3 m depth.			3	GREY 100%						JN,UN,Ro JN,IR,Ro						143 MPa	
		END OF DRILLHOLE		301.4															
6.3																			
7																			
8																			
9																			
10																			
11																			
12																			
13																			

DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:

PROJECT 10-1191-0044				RECORD OF BOREHOLE No BC5-2				1 OF 2 METRIC								
G.W.P. 5091-07-00				LOCATION N 5334886.8; E 410403.9				ORIGINATED BY EHS								
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring				COMPILED BY MT								
DATUM GEODETIC				DATE August 16, 2011				CHECKED BY SEMC								
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								
306.6	GROUND SURFACE															
0.0	TOPSOIL		1a	SS	6/0.25											
306.1	Gravelly SILTY CLAY, trace to some sand, trace rootlets		1b													26 7 43 24
0.5	Firm Brown Wet METASEDIMENT (BEDROCK)		1	RC	REC 100%											RQD = 100%
	Bedrock cored from depths of 0.5 m to 3.9 m															
	For bedrock coring details, refer to Record of Drillhole C5-2		2	RC	REC 89%											RQD = 89%
			3	RC	REC 97%											RQD = 93%
302.7	END OF BOREHOLE															
3.9	Note: 1. Water level at a depth of 2.7 m below ground surface (Elev. 303.9 m) upon completion of drilling. 2. Water level at a depth of 1.0 m below ground surface (Elev. 305.6 m) upon completion of piezometer installation. 3. Water level in piezometer at 0.5 m above ground surface (Elev. 307.1 m) on November 15, 2012 and at 0.6 m above ground surface (Elev. 307.2 m) on May 15, 2013.															

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC5-2

SHEET 2 OF 2

LOCATION: N 5334886.8 ; E 410403.9

DRILLING DATE: August 16, 2011

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D50

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE		306.1									
1	NW	METASEDEMENT Very fine grained Fresh Dark grey		0.5	1	GREY	100%						
2	August 16, 2011 NQ Coring				2	GREY	100%						
3					3	GREY	100%						
4		END OF DRILLHOLE		302.7									
5				3.9									
6													
7													
8													
9													
10													

DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:

PROJECT		10-1191-0044		RECORD OF BOREHOLE No BC5-3		1 OF 2 METRIC	
G.W.P.		5091-07-00		LOCATION		N 5334848.4; E 410420.7	
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring	
DATUM		GEODETC		DATE		August 18, 2011	
				ORIGINATED BY		EHS	
				COMPILED BY		MT	
				CHECKED BY		SEMC	
SOIL PROFILE				SAMPLES		DYNAMIC CONE PENETRATION RESISTANCE PLOT	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
309.0	GROUND SURFACE						
0.0	TOPSOIL						
308.6			1	SS	11		
0.4	CLAYEY SILT, trace sand, trace organics, clay seams Stiff Brown Moist		2	SS	9		
307.5							
1.5	SILT, trace to some clay Loose to compact Brown to grey Wet		3	SS	16		
			4	SS	11		
			5	SS	12		
			6	SS	9		
304.3							
4.7	SAND and GRAVEL, some silt, trace clay Compact to very dense Grey to brown Wet		7	SS	30		
			8	SS	36		
			9	SS	21		
299.4			10	SS	93/0.23		
9.6	METASEDIMENT (BEDROCK) Bedrock cored from depths of 9.6 m to 13.1 m For bedrock coring details, refer to Record of Drillhole C5-3		1	RC	REC 100%		
			2	RC	REC 100%		
			3	RC	REC 93%		
295.9							
13.1	END OF BOREHOLE Note: 1. Water level inside casing at a depth of 2.5 m below ground surface (Elev. 306.5 m) upon completion of drilling.						

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

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC5-3

SHEET 2 OF 2

LOCATION: N 5334848.4 ; E 410420.7

DRILLING DATE: August 18, 2011

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D50

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE		299.4									
	NW	METASEDIMENT		9.6									
10		Very strong			1	GREY	100%						
		Very fine to very coarse grained											
		Fresh to completely weathered											
		Grey											
11	August 18, 2011				2	GREY	100%						
	NQ Coring												
12													
		Completely weathered zone between			3	GREY	100%						
		12.5 m and 12.6 m depth.											
13		END OF DRILLHOLE		295.9									
				13.1									
14													
15													
16													
17													
18													
19													

DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:

PROJECT 10-1191-0044				RECORD OF BOREHOLE No BC6-1				1 OF 2 METRIC									
G.W.P. 5091-07-00				LOCATION N 5334903.3; E 410529.4				ORIGINATED BY MT									
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring				COMPILED BY MT									
DATUM GEODETIC				DATE August 20 and 21, 2011				CHECKED BY SEMC									
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									
307.3	GROUND SURFACE																
0.9	TOPSOIL		1	SS	10												
	CLAYEY SILT, trace sand Stiff Grey Moist		2	SS	14												0 4 61 35
305.6			3	SS	6/0.15												
1.7	SAND and GRAVEL, some silt, trace clay Very dense Brown to grey Moist																
	Cobble encountered between 3.0 m and 3.2 m depth.		4	SS	68												41 44 12 3
	Cobble encountered between 5.2 and 5.3 m depth.		5	SS	75												
	Split spoon bouncing at 4.3 m depth.		6	SS	35												
301.2																	
6.1	METASEDIMENT (SILTSTONE) (BEDROCK)		1	RC	REC 100%												RQD = 73%
	Bedrock cored from depths of 6.1 m to 9.3 m For bedrock coring details, refer to Record of Drillhole C6-1		2	RC	REC 100%												RQD = 100%
298.0			3	RC	REC 100%												RQD = 33%
9.3	END OF BOREHOLE																
	Note: 1. Water level not recorded.																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC6-1

SHEET 2 OF 2

LOCATION: N 5334903.3 ; E 410529.4

DRILLING DATE: August 20 and 21, 2011

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D50

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	CORRELATION LOG														NOTES WATER LEVELS INSTRUMENTATION		
						COLOUR % RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA					HYDRAULIC CONDUCTIVITY		Diametral Point Load Index (MPa)	RMC -Q AVG.			
							FLUSH	TOTAL CORE %			SOLID CORE %	B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn				k, cm/s	T
								JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate			BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage											
		REFER TO PREVIOUS PAGE		301.2																		
	NW	METASEDIMENT (SILTSTONE)		6.1																		
7		Strong Very fine to medium grained Slightly weathered to fresh Greenish grey			1	GREY / BROWN 100%																
8	August 20 and 21, 2012 NQ Coring	Well banded between 6.1 m and 7.5 m depth.			2	GREY 100%																64 MPa
9					3	GREY 100%																
		END OF DRILLHOLE		298.0 9.3																		
10																						
11																						
12																						
13																						
14																						
15																						
16																						

DEPTH SCALE

1 : 50



LOGGED: MT

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:

PROJECT 10-1191-0044				RECORD OF BOREHOLE No BC6-2				1 OF 2 METRIC									
G.W.P. 5091-07-00				LOCATION N 5334930.9; E 410524.4				ORIGINATED BY MT									
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring				COMPILED BY MT									
DATUM GEODETIC				DATE August 21 and 22, 2011				CHECKED BY SEMC									
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									
305.3	GROUND SURFACE																
0.0	TOPSOIL		1	SS	6												
304.5			2	SS	31												
0.8	SILT, some sand, trace to some clay, trace gravel Dense Grey to brown Moist																
303.8			3	SS	29												
1.5	Silty SAND, some gravel, trace clay Compact Brown Wet																
302.8			4	SS	9/0.08												
2.5	METASEDIMENT (SILTSTONE) (BEDROCK)		1	RC	REC 100%												RQD = 0%
	Bedrock cored from depths of 2.5 m to 5.7 m																
	For bedrock coring details, refer to Record of Drillhole C6-2		2	RC	REC 100%												RQD = 76%
			3	RC	REC 100%												RQD = 96%
299.6																	
5.7	END OF BOREHOLE																
	Note: 1. Water level in the casing at a depth of 1.2 m below ground surface (Elev. 304.1 m) upon completion of drilling. 2. Water level in piezometer at 0.2 m above ground surface (Elev. 305.5 m) on November 15, 2012 and on May 15, 2013.																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

SHEET 2 OF 2

DATUM: GEODETIC

DRILLING CONTRACTOR: Walker Drilling Ltd.

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:

PROJECT		10-1191-0044				RECORD OF BOREHOLE No BC6-3				1 OF 2 METRIC							
G.W.P.		5091-07-00		LOCATION		N 5334880.7; E 410533.5				ORIGINATED BY							
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring				COMPILED BY							
DATUM		GEODETIC		DATE		August 22, 2011				CHECKED BY							
										SEMC							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
308.9	GROUND SURFACE						20	40	60	80	100						
308.9	TOPSOIL		1	SS	14												
	CLAYEY SILT		2	SS	18												
307.4	Stiff to very stiff																
	Brown to grey																
	Moist																
307.4	SILT, trace clay, trace sand		3	SS	20												
	Compact																
	Grey																
	Wet																
306.7	Gravelly Silty SAND, trace clay		4	SS	53												
	Very dense																
	Brown																
	Moist																
	Split spoon bouncing at 2.7 m depth.		5	SS	65												
304.6	METASEDIMENT (SILTSTONE)		1	RC	REC 100%												
	(BEDROCK)																
	Bedrock cored from depths of																
	4.3 m to 7.4 m																
	For bedrock coring details, refer to																
	Record of Drillhole C6-2																
			2	RC	REC 100%												
			3	RC	REC 100%												
301.5	END OF BOREHOLE																
	Note:																
	1. Water level not recorded.																

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC6-3

SHEET 2 OF 2

LOCATION: N 5334880.7 ; E 410533.5

DRILLING DATE: August 22, 2011

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D50

DRILLING CONTRACTOR: Walker Drilling Ltd.

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.				NOTES WATER LEVELS INSTRUMENTATION
							FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA						HYDRAULIC CONDUCTIVITY			Diametral Point Load Index (MPa)	RMC -Q AVG.				
								TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	k, cm/s	10 ⁰	10 ¹			10 ²			
		REFER TO PREVIOUS PAGE		304.6																						
5	August 22, 2012 NQ Coring	METASEDIMENT (SILTSTONE) Very strong Fine to medium grained Fresh to moderately weathered Greenish Grey		4.3	1	GREY 100%																				
6					2	GREY 100%																				
7					3	GREY 100%																				
		END OF DRILLHOLE		301.5 7.4																						
8																										
9																										
10																										
11																										
12																										
13																										
14																										

DEPTH SCALE

1 : 50



LOGGED: MT

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:

PROJECT 10-1191-0044		RECORD OF BOREHOLE No BC6-4				1 OF 1 METRIC											
G.W.P. 5091-07-00		LOCATION N 5334921.8; E 410525.0				ORIGINATED BY MR											
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT											
DATUM GEODETIC		DATE May 16, 2013				CHECKED BY SEMC											
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									
305.7	GROUND SURFACE																
0.0	SILT, trace organics, trace to some sand, trace to some clay, trace to some gravel Very loose Brown Wet		1	SS	1												
305.1																	
0.6	SAND and GRAVEL, trace to some silt, trace clay Compact to dense Brown Wet		2	SS	35												
303.6			3	SS	19												39 47 11 3
2.1	METASEDIMENT (BEDROCK)																
	Bedrock cored from 2.1 m depth to 5.7 m depth. For coring details see record of Drillhole BC6-4.		1	RC	REC 100%												RQD = 95%
			2	RC	REC 100%												RQD = 88%
			3	RC	REC 100%												RQD = 97%
300.0	END OF BOREHOLE																
5.7	Note: 1. Water level at a depth of 0.1 m below ground surface (Elev. 305.6 m) upon completion of drilling.																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

SHEET 1 OF 1

DATUM: GEODETIC

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:

PROJECT 10-1191-0044				RECORD OF BOREHOLE No BC6-5				1 OF 1 METRIC									
G.W.P. 5091-07-00				LOCATION N 5334887.1; E 410543.8				ORIGINATED BY MR									
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY MT									
DATUM GEODETIC				DATE May 16, 2013				CHECKED BY SEMC									
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									
307.6	GROUND SURFACE							20	40	60	80	100					
0.1	TOPSOIL		1	SS	8		307										
	SILT, trace to some clay, trace organics Loose to compact Grey Moist to wet		2	SS	13		306										
			3	SS	11		305										
			4	SS	9		304										
304.6	Silty SAND and GRAVEL Very dense Brown Wet		5	SS	62		303										
303.9	METASEDIMENT (BEDROCK)		1	RC	REC 100%		302										
3.7	Bedrock cored from 3.7 m depth to 7.0 m depth. For coring details see Record of Drillhole BC6-5.		2	RC	REC 100%		301										
			3	RC	REC 100%												
300.6	END OF BOREHOLE																
7.0	Note: 1. Water level at a depth of 1.1 m below ground surface (Elev. 306.5 m) upon completion of drilling.																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT: 10-1191-0044

RECORD OF DRILLHOLE: BC6-5

SHEET 1 OF 1

LOCATION: N 5334887.1 ; E 410543.8

DRILLING DATE: May 16, 2013

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME55

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

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.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
							FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA										HYDRAULIC CONDUCTIVITY		Diametral Point Load Index (MPa)	RMC -Q' AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
								TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	k, cm/s																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
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DEPTH SCALE

1 : 50



LOGGED: MR

CHECKED: SEMC

SUD-RCK 10-1191-0044SUD.GPJ GAL-MISS.GDT 05/12/13 DATA INPUT:



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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST No H3-D2				1 OF 1 METRIC											
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334863.3; E 410459.2</u>				ORIGINATED BY <u>MT</u>											
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>MT</u>											
DATUM <u>GEODETIC</u>		DATE <u>August 9, 2011</u>				CHECKED BY <u>SEMC</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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)
309.5 0.0	GROUND SURFACE						20	40	60	80	100						
308.4 1.1	END OF DCPT REFUSAL TO FURTHER PENETRATION 14 BLOWS / 0.15 m (HAMMER BOUNCING)						20	40	60	80	100						

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



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



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


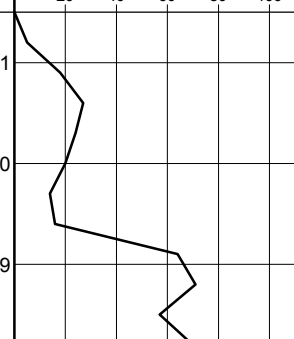
SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST No H3-D5				1 OF 1 METRIC											
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334882.7; E 410553.1</u>				ORIGINATED BY <u>MT</u>											
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>MT</u>											
DATUM <u>GEODETIC</u>		DATE <u>August 7, 2011</u>				CHECKED BY <u>SEMC</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	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)
307.7 0.0	GROUND SURFACE						20	40	60	80	100						
307																	
305.9 1.8	END OF DCPT REFUSAL TO FURTHER PENETRATION 20 BLOWS / 0.30 m (HAMMER BOUNCING)																

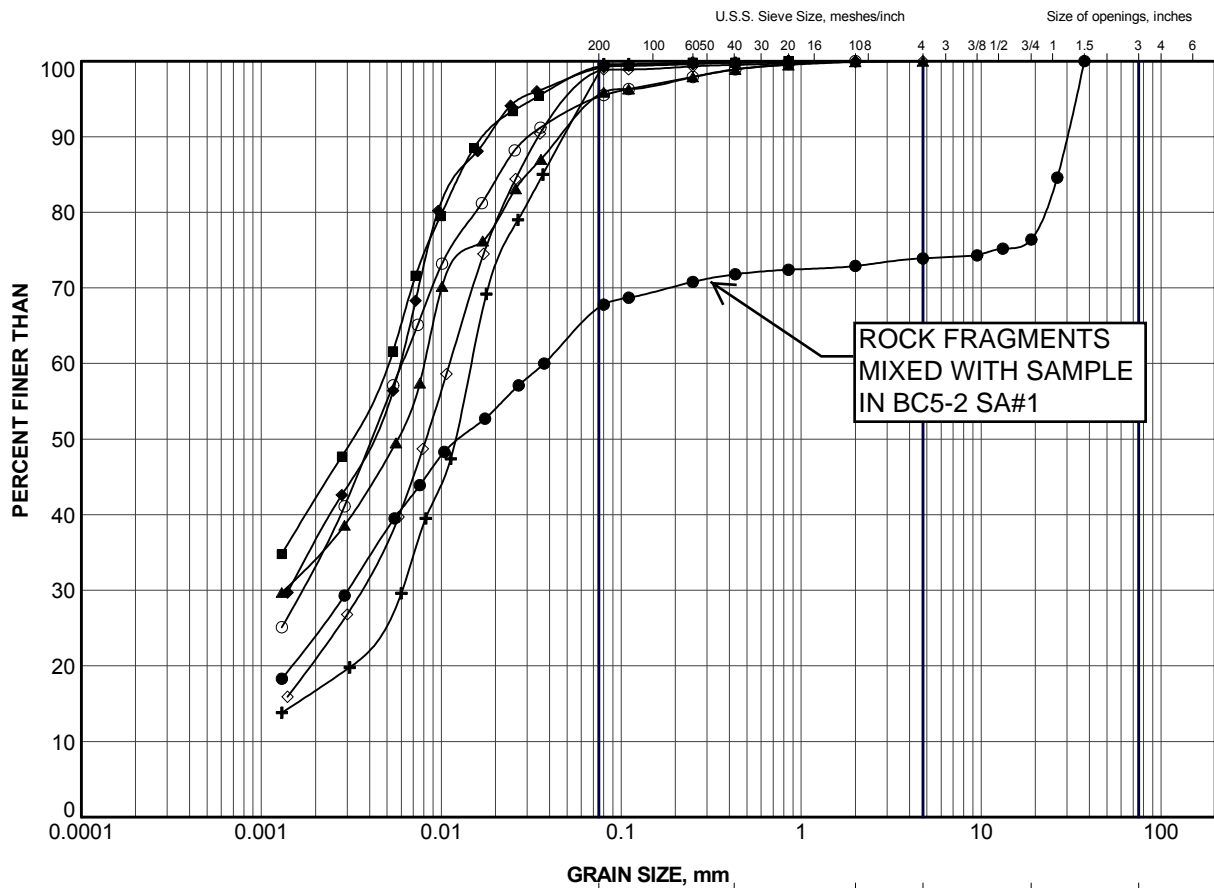
SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



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

PROJECT <u>10-1191-0044</u>		RECORD OF PENETRATION TEST No H3-D7				1 OF 1 METRIC				
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334891.2; E 410600.7</u>				ORIGINATED BY <u>MT</u>				
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>MT</u>				
DATUM <u>GEODETIC</u>		DATE <u>August 6, 2011</u>				CHECKED BY <u>SEMC</u>				
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
311.5 0.0	GROUND SURFACE									
										
308.2 3.3	END OF DCPT REFUSAL TO FURTHER PENETRATION 70 BLOWS / 0.23 m (HAMMER BOUNCING)									

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC5-2	1b	306.3
■	BC5-3	2	307.9
▲	BC6-1	2	306.4
+	BC6-3	2	307.8
◆	H3-2	2	308.5
◇	H3-9	2	307.5
○	H3-11	1b	307.7

PROJECT

HIGHWAY 66 - HIGH FILL H3
STA 14+840 TO 15+060

TITLE

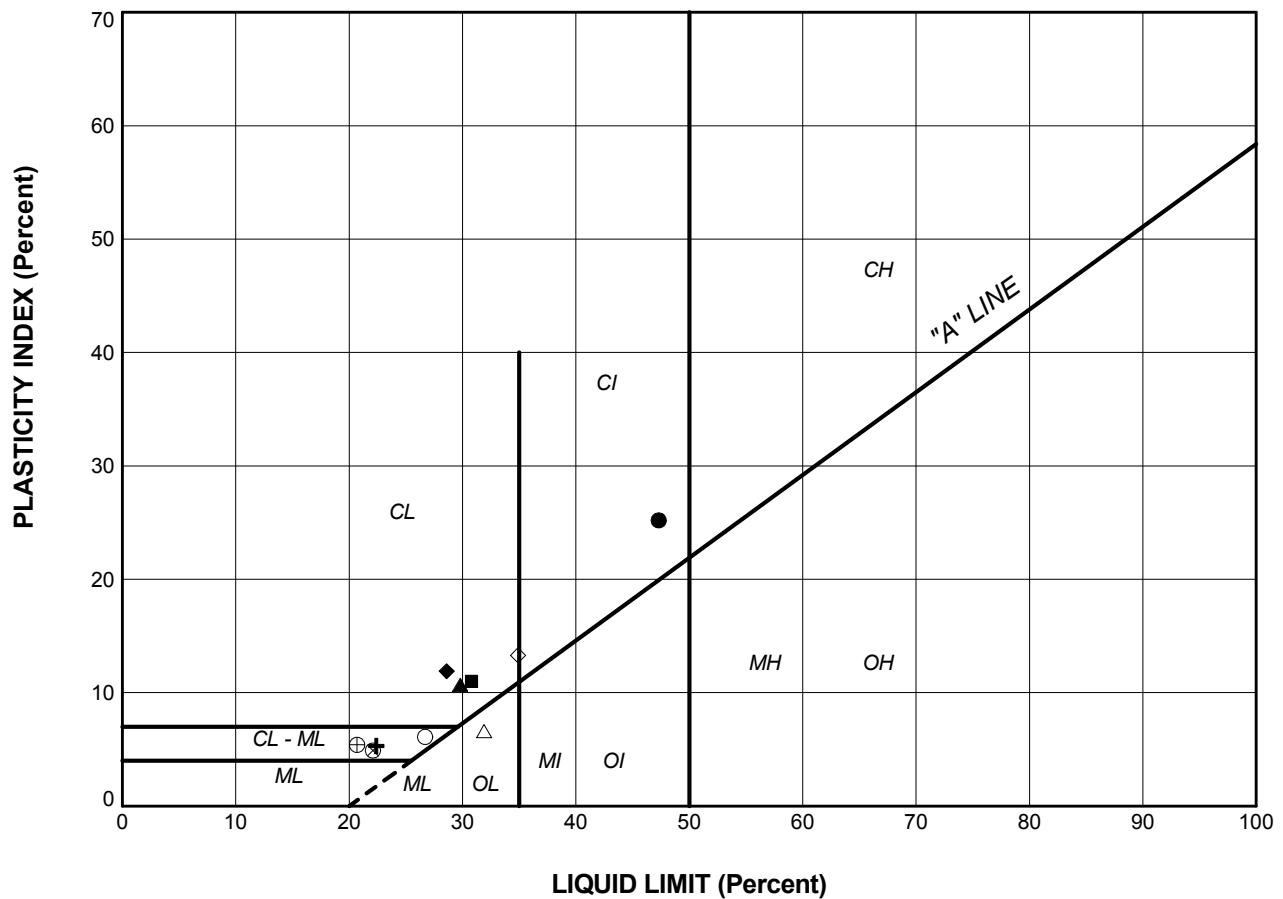
GRAIN SIZE DISTRIBUTION

CLAYEY SILT to SILTY CLAY




PROJECT No.	10-1191-0044	FILE No.	10-1191-0044SUD.GPJ
DRAWN	JJL	Jun 2013	SCALE N/A
CHECK	SEMC	Jun 2013	REV.
APPR	JMAC	Jun 2013	

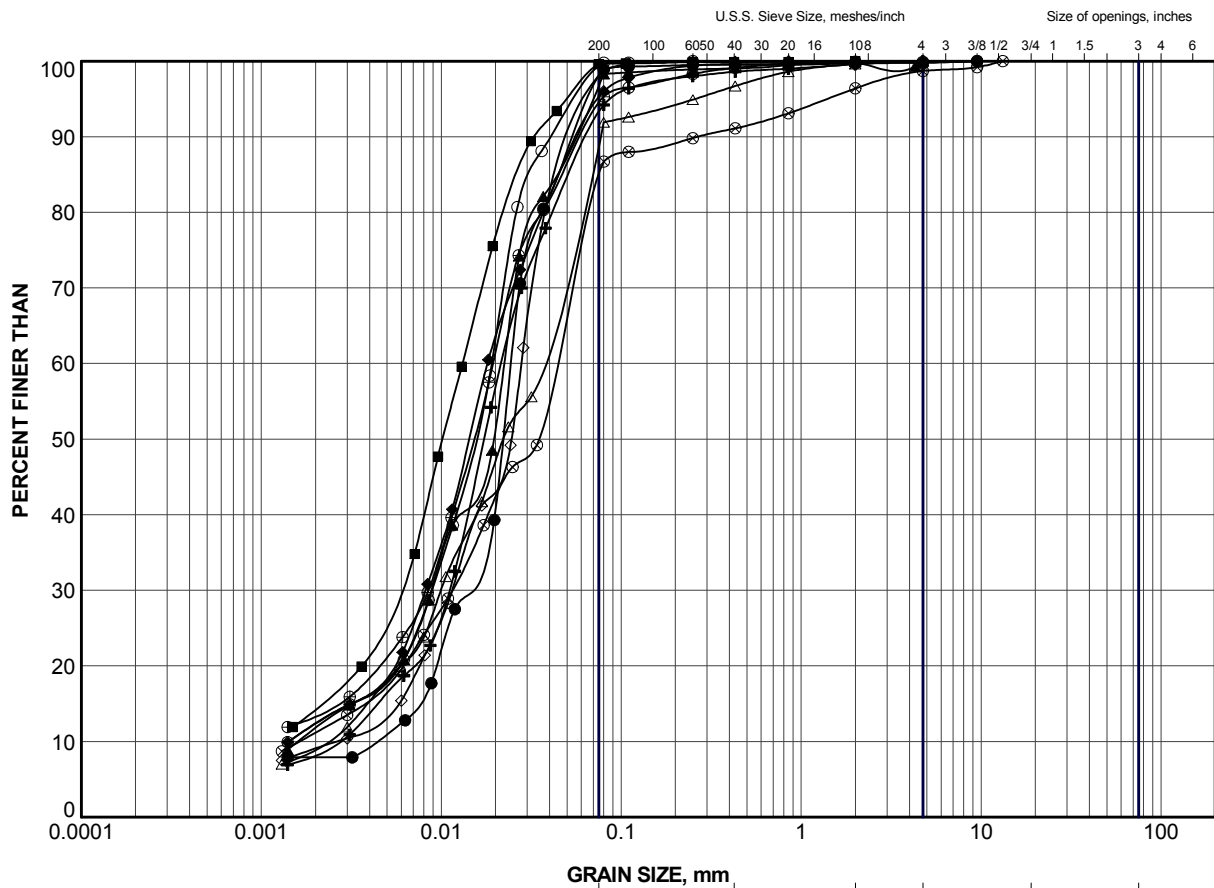
FIGURE D1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BC5-2	1b	47.3	22.1	25.2
■	BC5-3	2	30.8	19.8	11.0
▲	BC6-1	2	29.8	19.1	10.7
+	BC6-3	2	22.4	17.1	5.3
◆	H3-2	2	28.6	16.7	11.9
◇	H3-4	2	34.9	21.6	13.3
○	H3-6	1	26.7	20.6	6.1
△	H3-7	1b	31.9	25.3	6.6
⊗	H3-8	1b	22.1	17.2	4.9
⊕	H3-9	2	20.7	15.3	5.4

PROJECT				
HIGHWAY 66 - HIGH FILL H3 STA 14+840 TO 15+060				
TITLE				
PLASTICITY CHART CLAYEY SILT to SILTY CLAY				
PROJECT No. 10-1191-0044		FILE No. 10-1191-0044SUD.GPJ		
DRAWN	JJL	May 2013	SCALE	N/A
CHECK	SEMC	May 2013	REV.	
APPR	JMAC	May 2013		
 Golder Associates SUDBURY, ONTARIO			FIGURE D2	



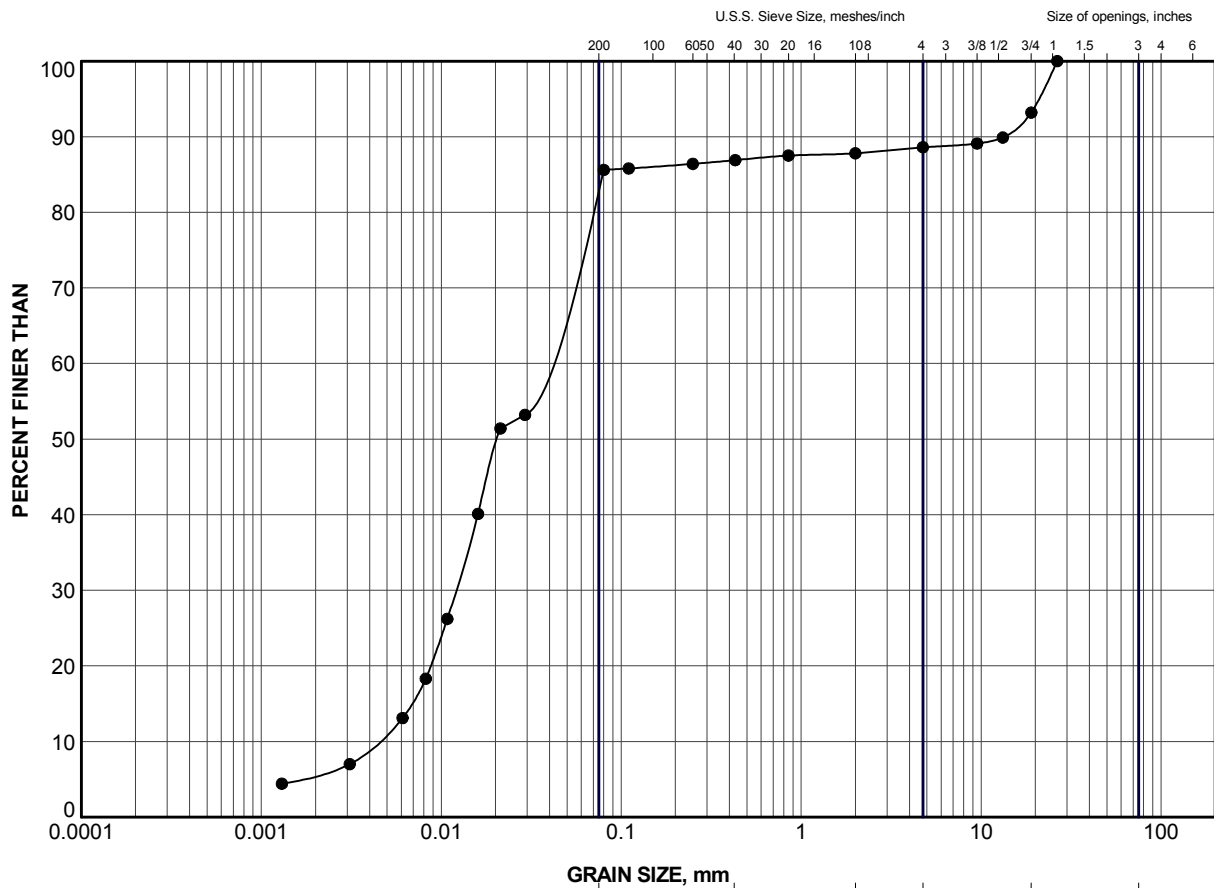
CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC5-3	5	305.6
■	BC6-5	3	305.8
▲	H3-2	5	306.2
+	H3-4	4	303.4
◆	H3-6	2	308.8
◇	H3-8	3	306.0
○	H3-10	3	305.5
△	H3-12	1b	310.6
⊗	H3-13	1b	310.7
⊕	H3-14	2	309.5

PROJECT					
HIGHWAY 66 - HIGH FILL H3 STA 14+840 TO 15+060					
TITLE					
GRAIN SIZE DISTRIBUTION SILT					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013	FIGURE D3.1		
APPR	JMAC	Jul 2013			




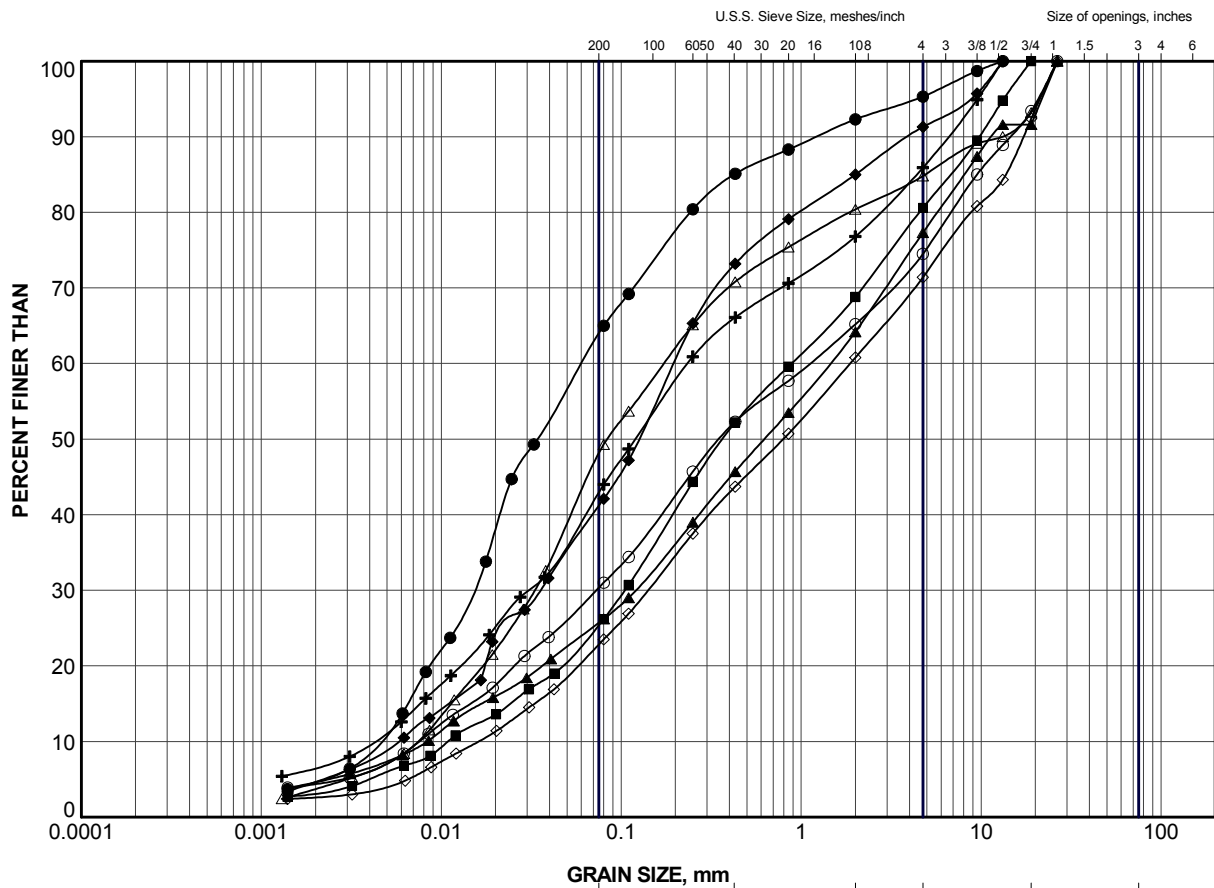


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	H3-15	2	310.4

PROJECT					
HIGHWAY 66 - HIGH FILL H3 STA 14+840 TO 15+060					
TITLE					
GRAIN SIZE DISTRIBUTION SILT					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	Jul 2013	SCALE	N/A	REV.
CHECK	SEMC	Jul 2013			
APPR	JMAC	Jul 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE D3.2		



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC5-1	3	306.1
■	BC6-2	3	303.5
▲	BC6-3	5	305.5
+	H3-1	2	308.4
◆	H3-3	1b	308.1
◇	H3-4	5	302.6
○	H3-5	2	308.1
△	H3-8	5	304.5

PROJECT

HIGHWAY 66 - HIGH FILL H3
STA 14+840 TO 15+060

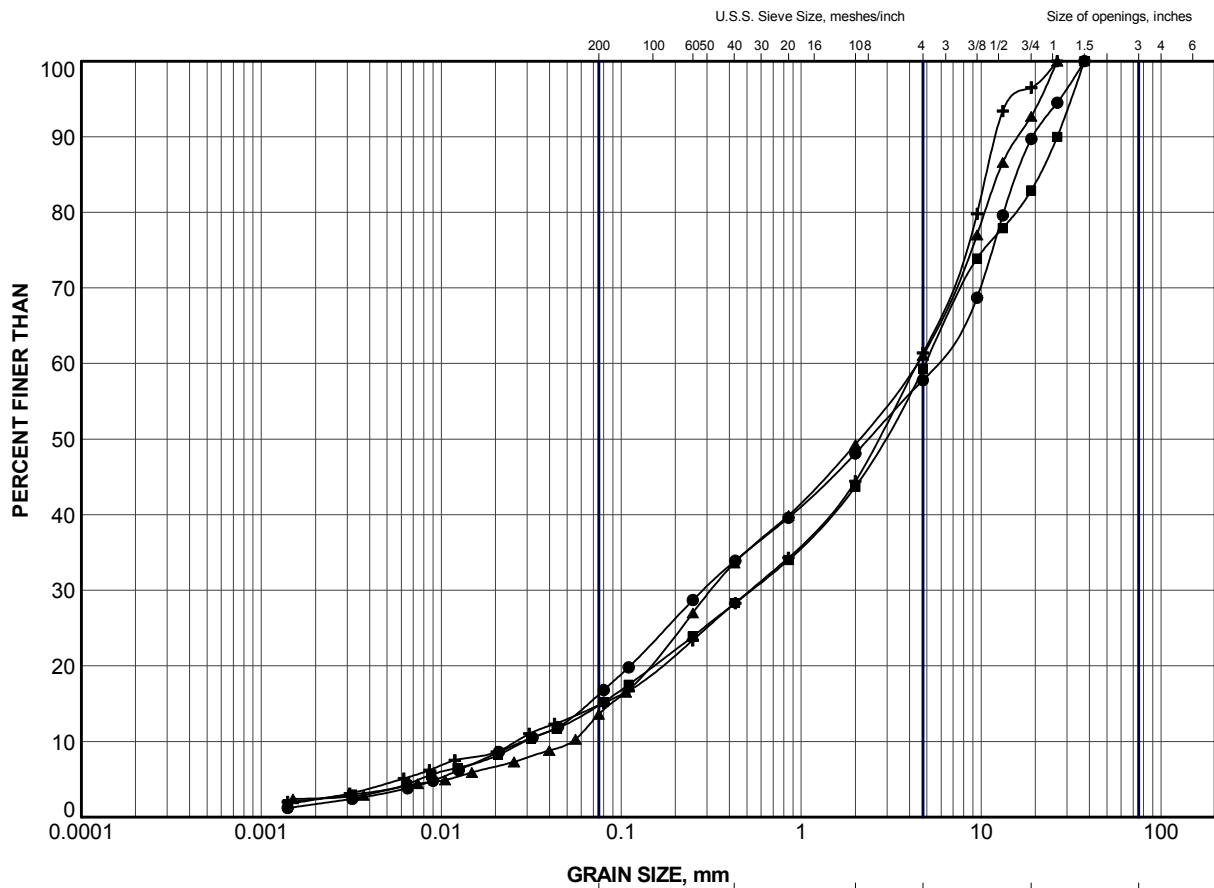
TITLE

GRAIN SIZE DISTRIBUTION
SAND AND SILT to GRAVELLY SILTY SAND



PROJECT No.	10-1191-0044	FILE No.	10-1191-0044SUD.GPJ
DRAWN	JJL	Jun 2013	SCALE N/A
CHECK	SEMC	Jun 2013	REV.
APPR	JMAC	Jun 2013	

FIGURE D4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC5-3	8	302.6
■	BC6-1	4	304.7
▲	BC6-4	3	304.0
+	H3-10	9	299.4

PROJECT

HIGHWAY 66 - HIGH FILL H3
STA 14+840 TO 15+060

TITLE

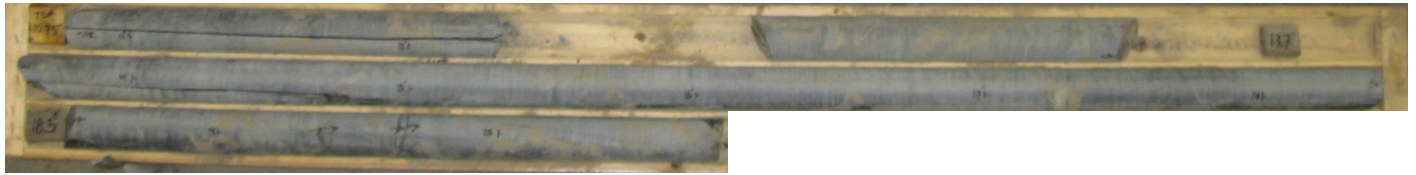
GRAIN SIZE DISTRIBUTION SAND AND GRAVEL



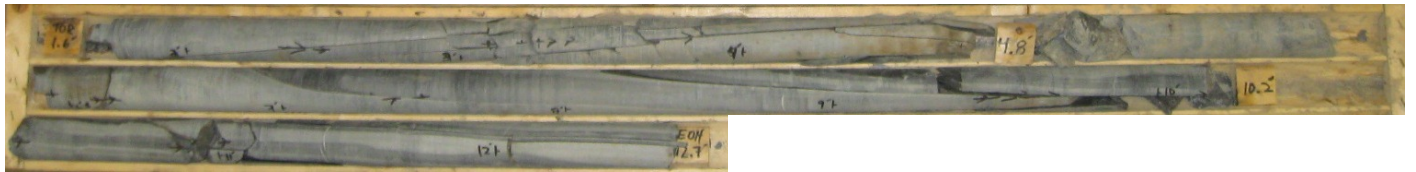
**Golder
Associates**
SUDBURY, ONTARIO

PROJECT No.	10-1191-0044	FILE No.	10-1191-0044SUD.GPJ
DRAWN	JJL	JUL 2013	SCALE N/A
CHECK	SEMC	JUL 2013	REV.
APPR	JMAC	JUL 2013	

FIGURE D5



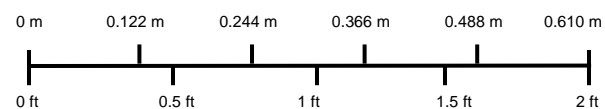
Borehole BC5-1
Elevation 304.4 m to 301.4 m




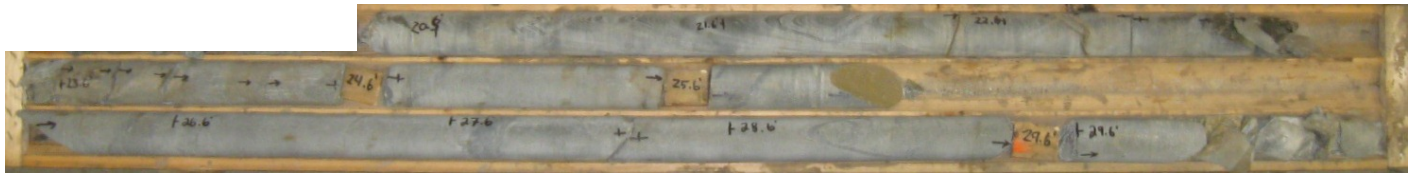
Borehole BC5-2
Elevation 306.1 m to 302.7 m



Borehole BC5-3
Elevation 299.4 m to 295.9 m



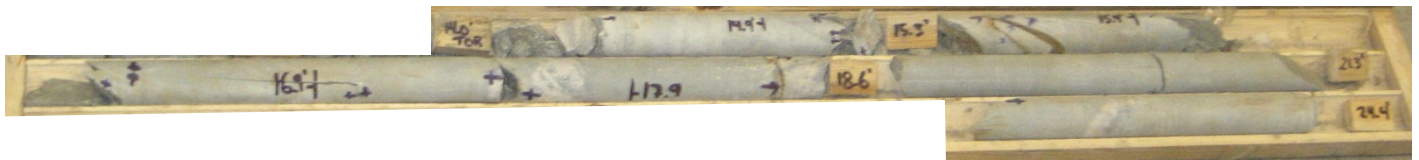
PROJECT		HIGHWAY 66 HIGH FILL H3 STA 14+840 to 15+060			
TITLE		BEDROCK CORE PHOTOGRAPHS			
		PROJECT No. 10-1191-0044		FILE No. ----	
		DESIGN	MT	Aug 2013	SCALE AS SHOWN
		CADD	--		REV.
		CHECK	SEMC	Aug 2013	FIGURE D6.1
		REVIEW	JMAC	Aug 2013	



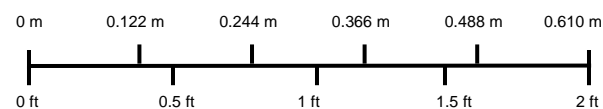
Borehole BC6-1
Elevation 302.7 m to 299.5 m




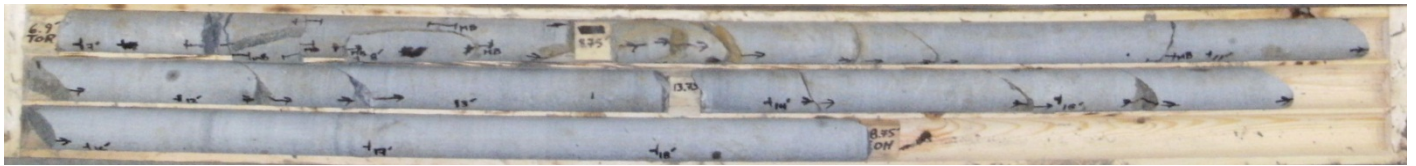
Borehole BC6-2
Elevation 304.3 m to 301.1 m



Borehole BC6-3
Elevation 301.2 m to 298.1 m



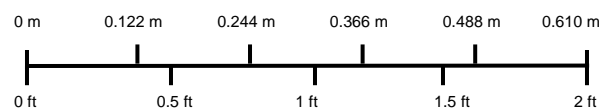
PROJECT		HIGHWAY 66 HIGH FILL H3 STA 14+840 to 15+060			
TITLE		BEDROCK CORE PHOTOGRAPHS			
		PROJECT No. 10-1191-0044		FILE No. ----	
		DESIGN	MT	Aug 2013	SCALE AS SHOWN
		CADD	--		REV.
		CHECK	SEMC	Aug 2013	FIGURE D6.2
		REVIEW	JMAC	Aug 2013	




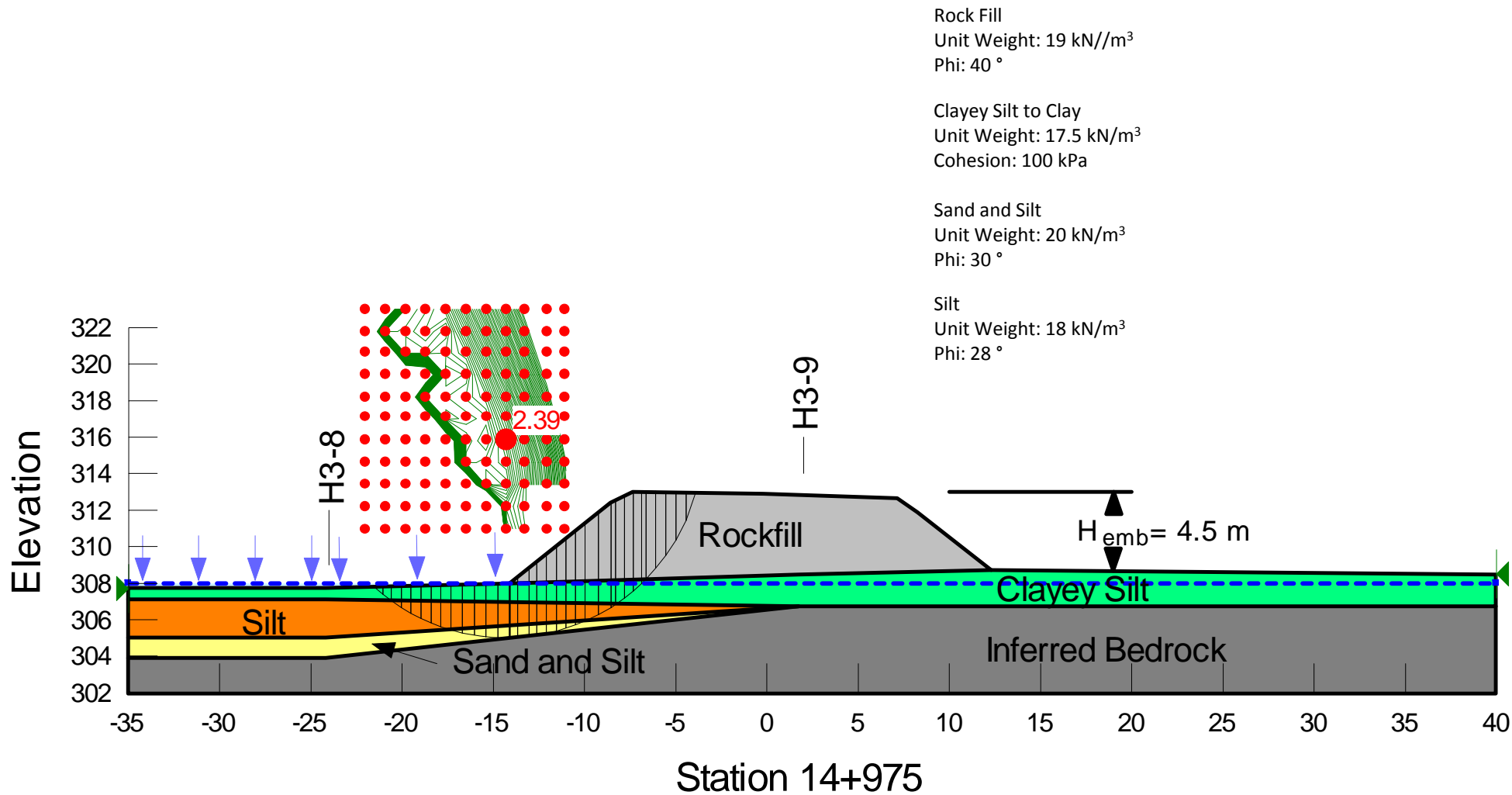
Borehole BC6-4
Elevation 303.6 m to 300.0 m



Borehole BC6-5
Elevation 303.9 m to 300.6 m



PROJECT		HIGHWAY 66 HIGH FILL H3 STA 14+840 to 15+060			
TITLE		BEDROCK CORE PHOTOGRAPHS			
		PROJECT No. 10-1191-0044		FILE No. ----	
		DESIGN	MT Aug 2013	SCALE	AS SHOWN
		CADD	--	REV.	
		CHECK	SEMC Aug 2013	FIGURE D6.3	
		REVIEW	JMAC Aug 2013		



PROJECT		HIGHWAY 66 HIGH FILL H3	
TITLE		STABILITY ANALYSIS NORTH SIDE SLOPE AT STA 14+975	
PROJECT No. 10-1191-0044		FILE No. ----	
DESIGN	MT	MAY 2013	SCALE AS SHOWN REV.
CADD	--		
CHECK	SEMC	MAY 2013	
REVIEW	JMAC	MAY 2013	

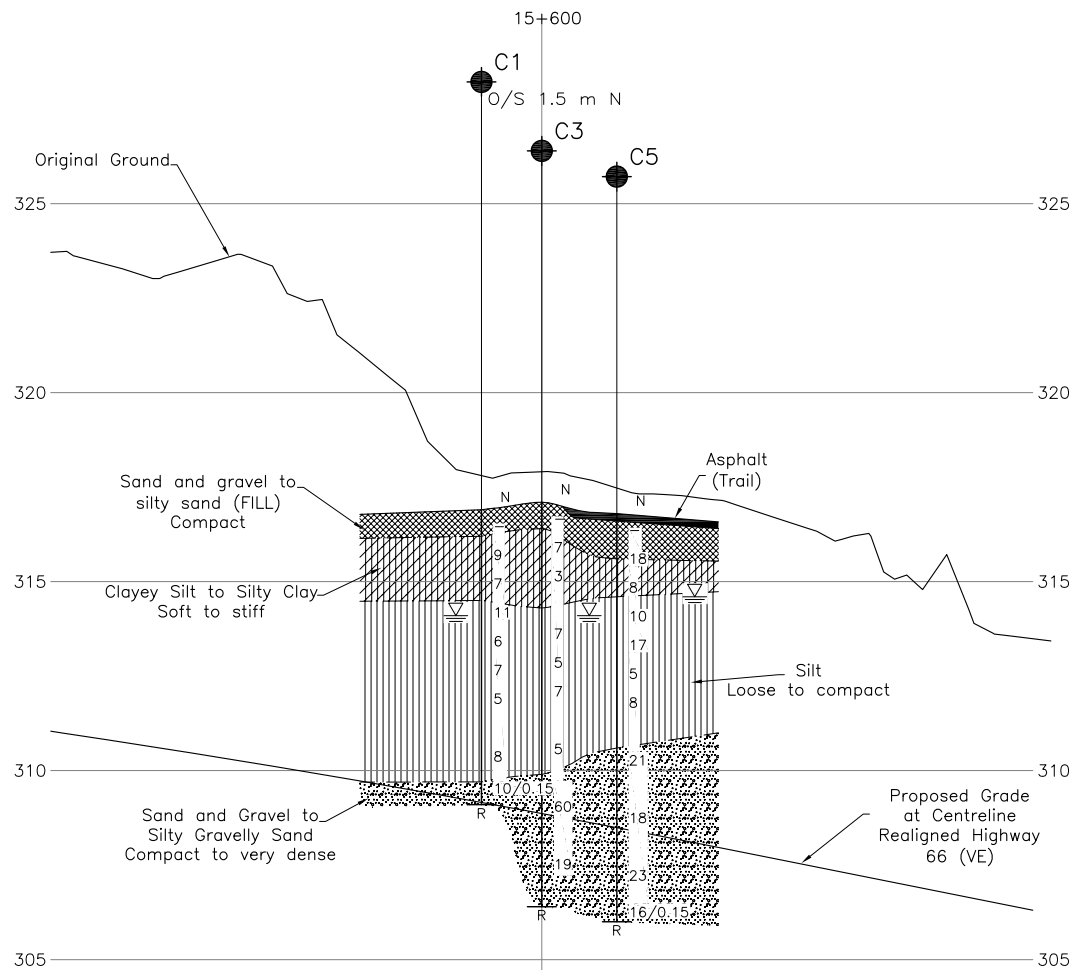
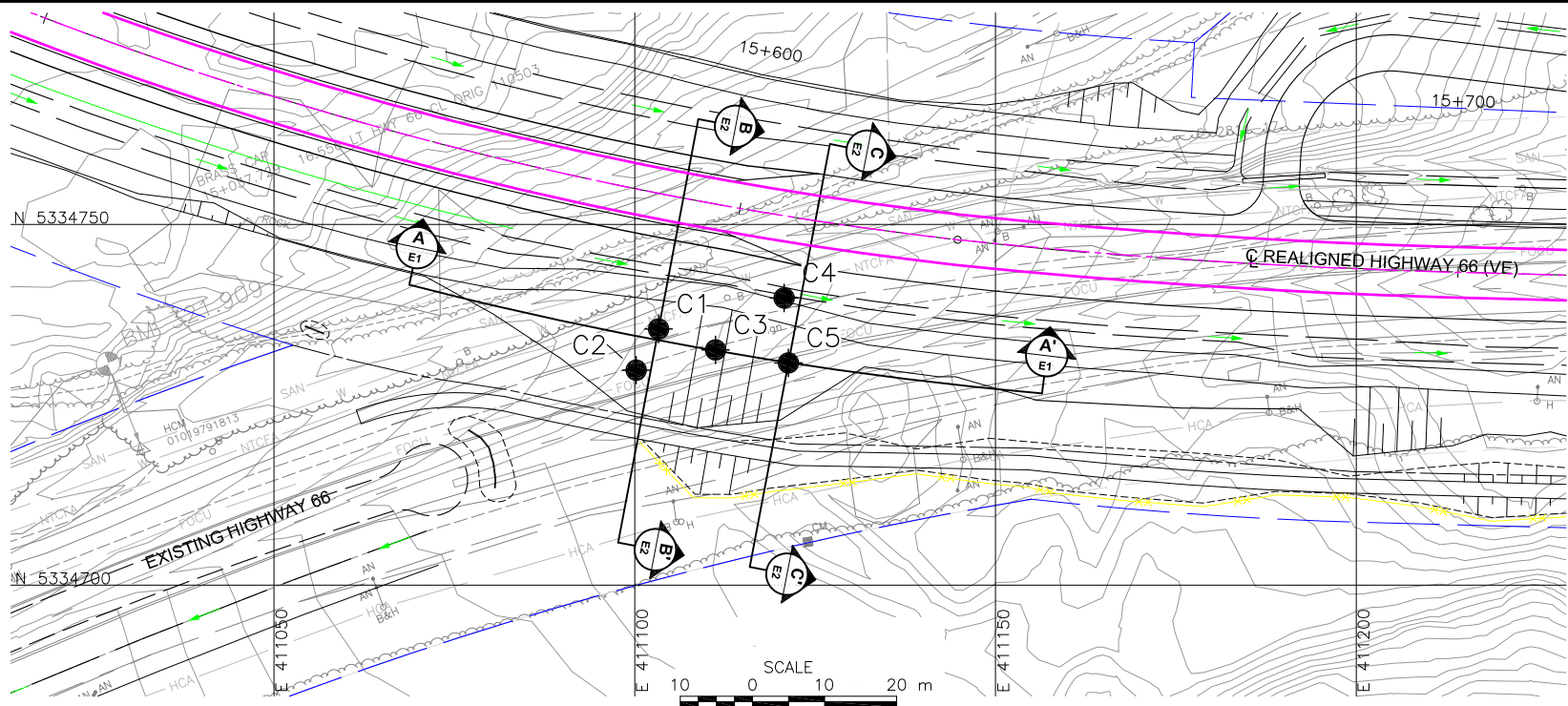


FIGURE D7



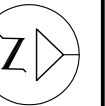
APPENDIX E

Highway 66 – STA 15+590 to 15+610 (Deep Cut)



A-A'
E1
PROFILE ALONG CREST OF SLOPE OFFSET 20 m SOUTH OF HWY 66 CENTRELINE
HIGHWAY 66
HORIZONTAL SCALE
10 0 10 20 m
2 0 2 4 m
VERTICAL SCALE

CONT No.
GWP No. 5091-07-00

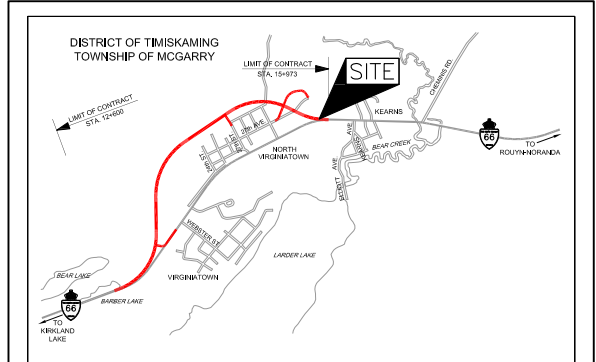


HIGHWAY 66
HWY 66 - STA 15+590 TO 15+610
BOREHOLE LOCATIONS AND
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



KEY PLAN
SCALE
700 0 700 m

LEGEND

- Borehole
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- R Refusal
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
C1	316.9	5334735.5	411103.3
C2	317.5	5334729.8	411100.2
C3	317.1	5334732.6	411111.2
C4	315.9	5334739.8	411120.7
C5	316.8	5334730.8	411121.3

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.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by MMM, drawing file nos. H3211009D16 ROLL PLAN-ULTIMATE and PDR.dwg, received DEC 3, 2012. Keyplan drawing file nos. H3211009G02 received JAN 24, 2013.



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Geocres No. 32D-17

HWY. 66	PROJECT NO. 10-1191-0044	DIST.
SUBM'D. TR	CHKD.	DATE: DEC 2013
DRAWN: JJJ	CHKD. SEMC	APPD. JMAC
		DWG. E1

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

CONT No.
GWP No. 5091-07-00

HIGHWAY 66
HWY 66 - STA 15+590 TO 16+610
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

LEGEND

- Borehole
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- R Refusal
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
C1	316.9	5334735.5	411103.3
C2	317.5	5334729.8	411100.2
C4	315.9	5334739.8	411120.7
C5	316.8	5334730.8	411121.3

NOTES

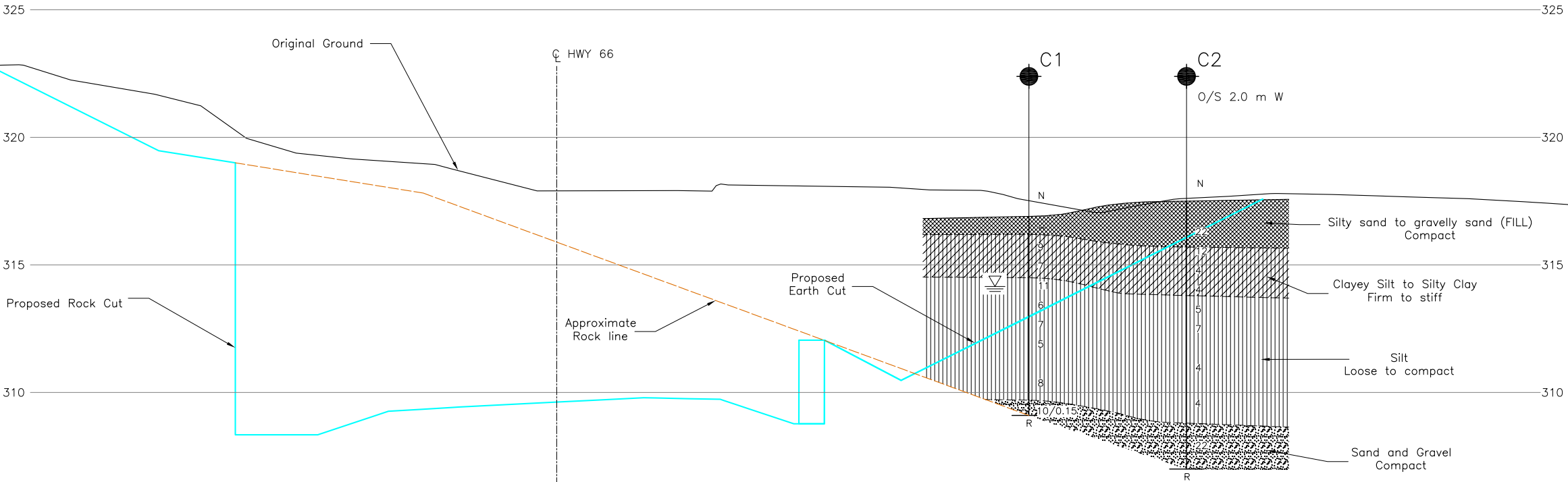
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

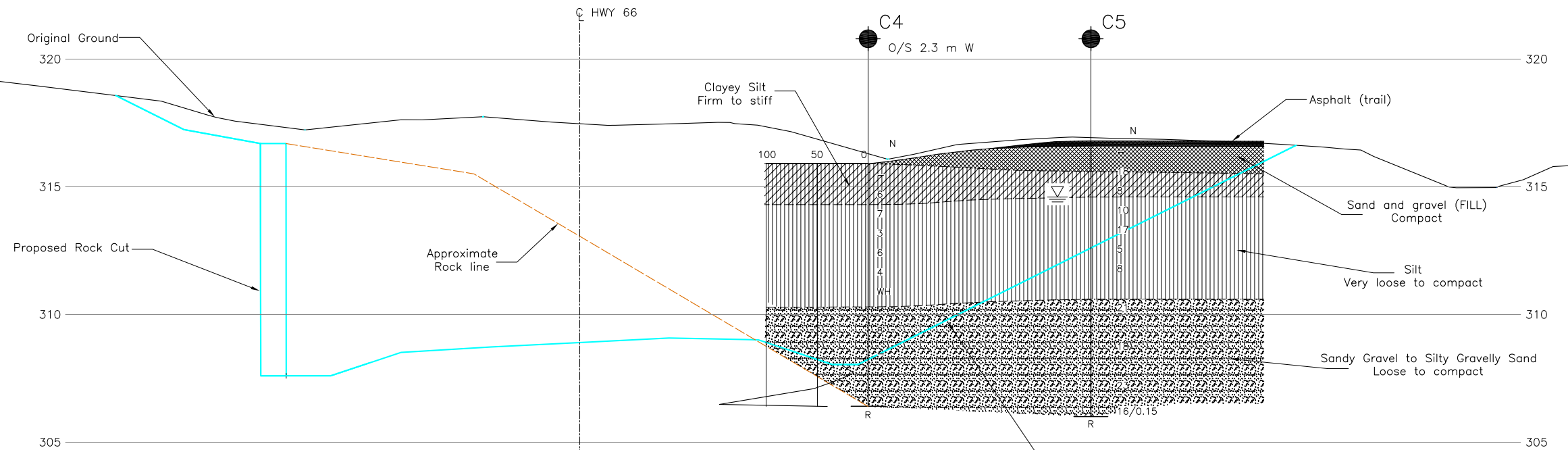
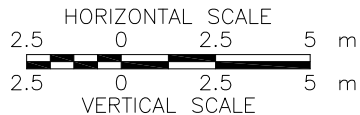
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

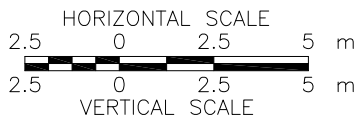
Base plans provided in digital format by MMM, drawing file nos. H3211009D16 ROLL PLAN-ULTIMATE and PDR.dwg, received DEC 3, 2012. Keyplan drawing file nos. H3211009G02 received JAN 24, 2013.



B-B'
E2 CROSS-SECTION AT STA. 15+590
HIGHWAY 66



C-C'
E2 CROSS-SECTION AT STA. 15+610
HIGHWAY 66



NO.	DATE	BY	REVISION
Geocres No. 32D-17			
HWY. 66		PROJECT NO. 10-1191-0044	DIST.
SUBM'D. TR	CHKD.	DATE: DEC 2013	SITE:
DRAWN: JJJ	CHKD. SEMC	APPD. JMAC	DWG. E2

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

PROJECT		10-1191-0044		RECORD OF BOREHOLE No C2				1 OF 1 METRIC									
G.W.P.		5091-07-00		LOCATION		N 5334729.8; E 411100.2		ORIGINATED BY		MT							
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY		MT							
DATUM		GEODETIC		DATE		November 14, 2012		CHECKED BY		SEMC							
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									
317.5	GROUND SURFACE						20	40	60	80	100						
0.0	Gravelly sand (FILL) Compact Brown Moist		1	AS	-												28 67 2 3
			2	SS	22												
315.7																	
1.8	CLAYEY SILT, trace to some sand Firm Brown Moist		3	SS	12												
			4	SS	4												
			5	SS	4												
313.8																	
3.7	SILT, some clay, trace sand Loose Grey Wet		6	SS	5												
			7	SS	7												
			8	SS	4												
			9	SS	4												
308.8																	
8.7	SAND and GRAVEL, trace silt Compact Grey Wet		10	SS	22												
	Spoon attempted at 10.5 m depth, bouncing.																
307.0	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING)																
10.5	Note: 1. Borehole caved to 0.6 m below ground surface. Open borehole dry to 0.6 m depth.																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

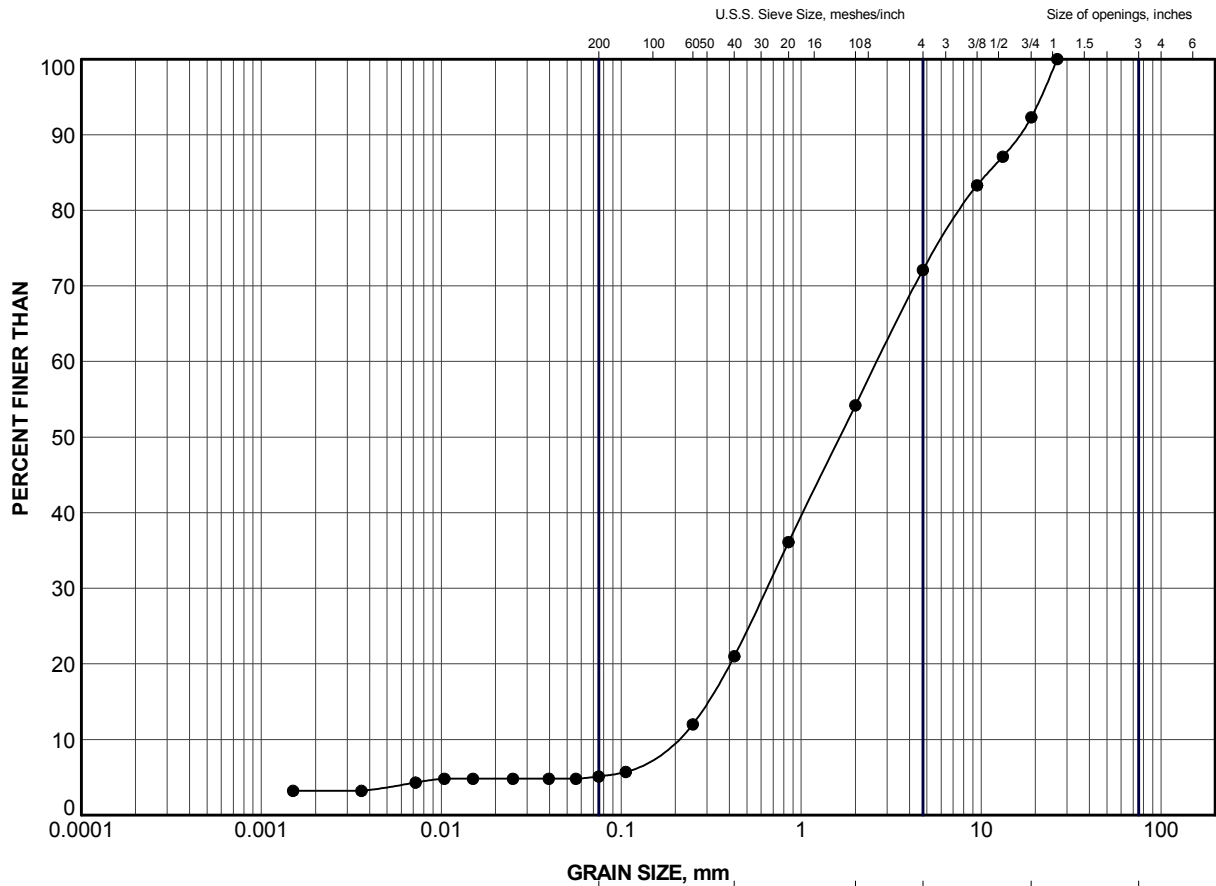
[illegible]

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

PROJECT		10-1191-0044		RECORD OF BOREHOLE No C4		1 OF 1 METRIC							
G.W.P.		5091-07-00		LOCATION		N 5334739.8; E 411120.7							
DIST		HWY 66		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers							
DATUM		GEODETIC		DATE		November 15, 2012							
				ORIGINATED BY		MT							
				COMPILED BY		MT							
				CHECKED BY		SEMC							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
315.9	GROUND SURFACE												
0.0	CLAYEY SILT, trace sand, trace organics Firm Brown Moist Trace organics above 0.6 m depth.		1	AS	-								
			2	SS	6								0 1 75 24
314.3	SILT, trace to some sand, some clay Very loose to loose Grey Wet		3	SS	7								
1.6			4	SS	3								0 0 87 13
			5	SS	6								
			6	SS	4								
			7	SS	WH								
310.3	Sandy GRAVEL, some silt, trace clay Loose Grey Wet		8	SS	8								61 23 13 3
5.6			9	AS	-								
308.3	Approximately 0.9 m of heave in augers at 7.6 m depth.												
7.6	END OF BOREHOLE START OF DCPT												
306.4	END OF DCPT REFUSAL TO FURTHER PENETRATION 40 BLOWS / 0.05 M (HAMMER BOUNCING)												
9.5	Notes: 1. Borehole caved to 1.8 m depth below ground surface. Open borehole dry to 1.8 m depth.												


SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

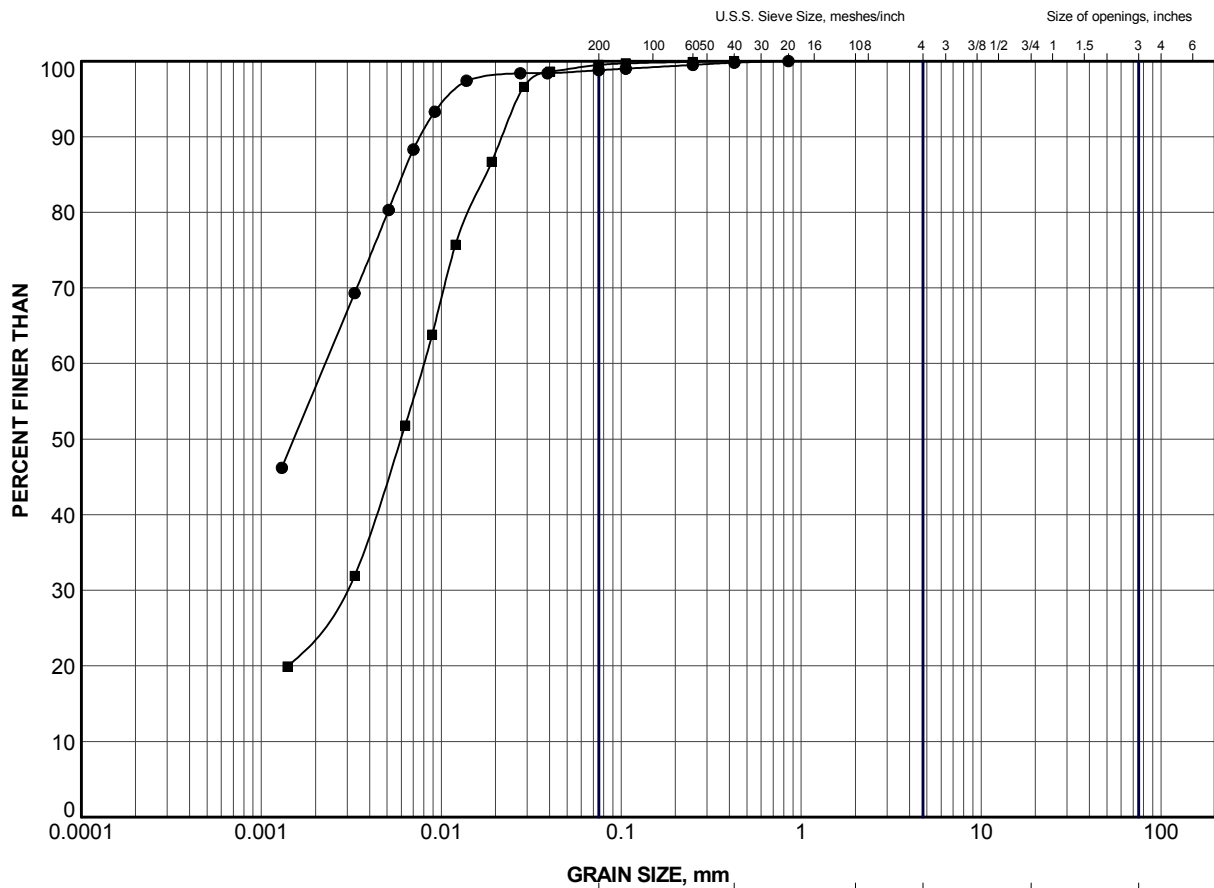


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C2	1	317.2

PROJECT					
HIGHWAY 66 - CUT SECTION STA 15+590 TO 15+610					
TITLE					
GRAIN SIZE DISTRIBUTION GRAVELLY SAND (FILL)					
		PROJECT No. 10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
		DRAWN	JJL	May 2013	SCALE N/A
		CHECK	SEMC	May 2013	REV.
		APPR	JMAC	May 2013	
<div style="display: flex; justify-content: space-between;"> FIGURE E1 </div>					

SUD-MTO GSD GLDR_LDN.GDT



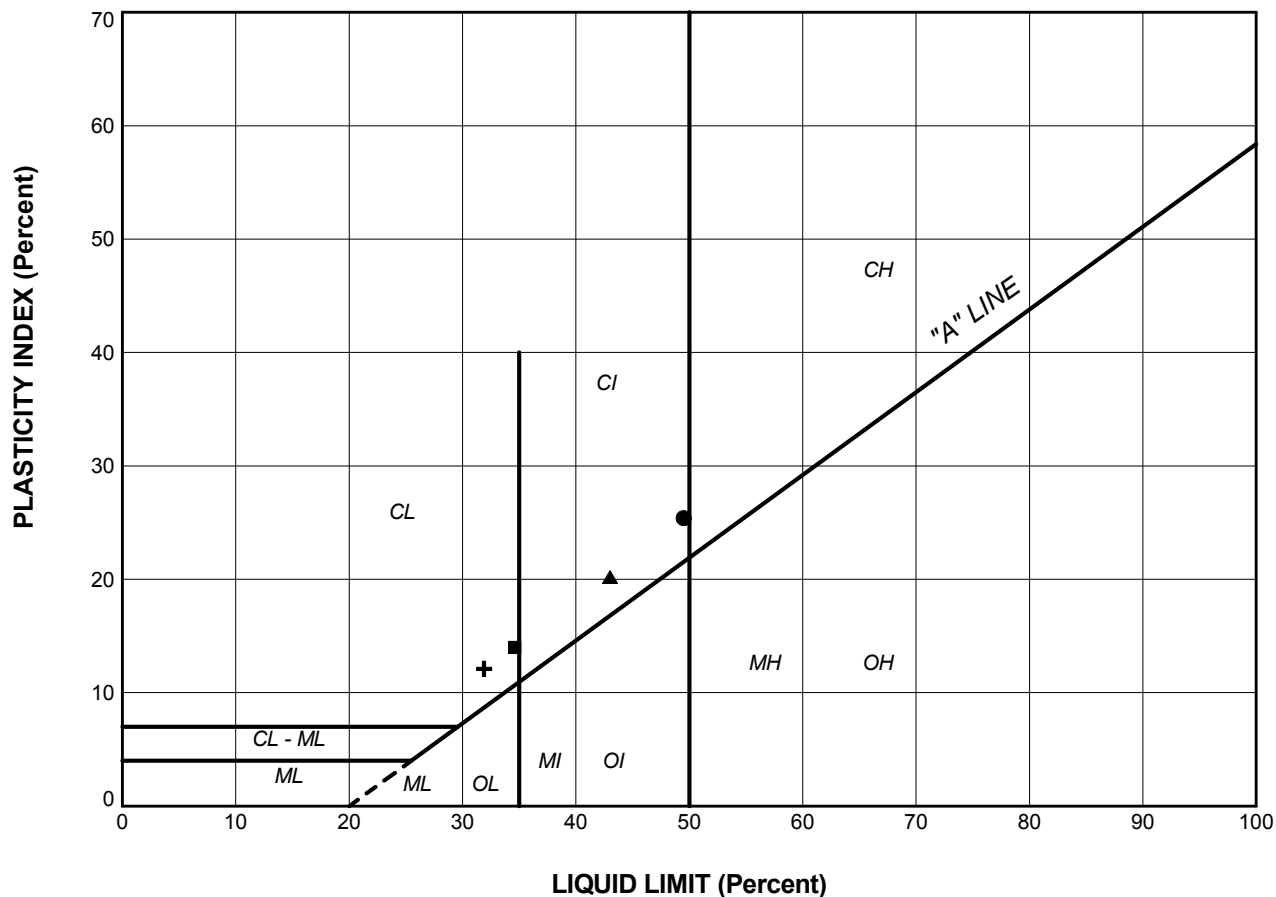
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C1	2	315.8
■	C4	2	314.8


PROJECT					
HIGHWAY 66 - CUT SECTION STA 15+590 TO 15+610					
TITLE					
GRAIN SIZE DISTRIBUTION CLAYEY SILT to SILTY CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	May 2013	SCALE	N/A	REV.
CHECK	SEMC	May 2013			
APPR	JMAC	May 2013			
			FIGURE E2		

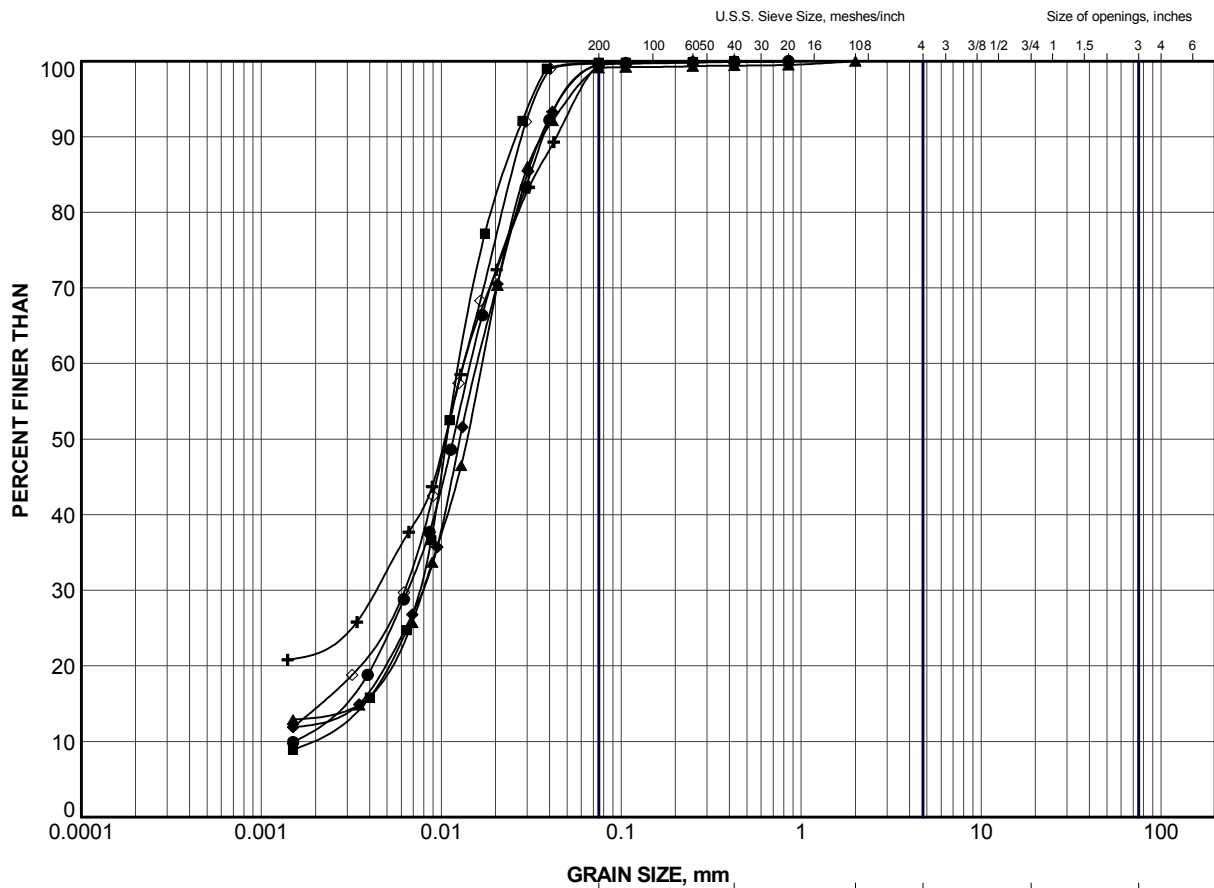




LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	C1	2	49.5	24.1	25.4
■	C2	4	34.6	20.6	14.0
▲	C3	2	43.0	22.8	20.2
+	C4	2	31.9	19.8	12.1


PROJECT					
HIGHWAY 66 - CUT SECTION STA 15+590 TO 15+610					
TITLE					
PLASTICITY CHART CLAYEY SILT to SILTY CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	May 2013	SCALE	N/A	REV.
CHECK	SEMC	May 2013			
APPR	JMAC	May 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE E3		

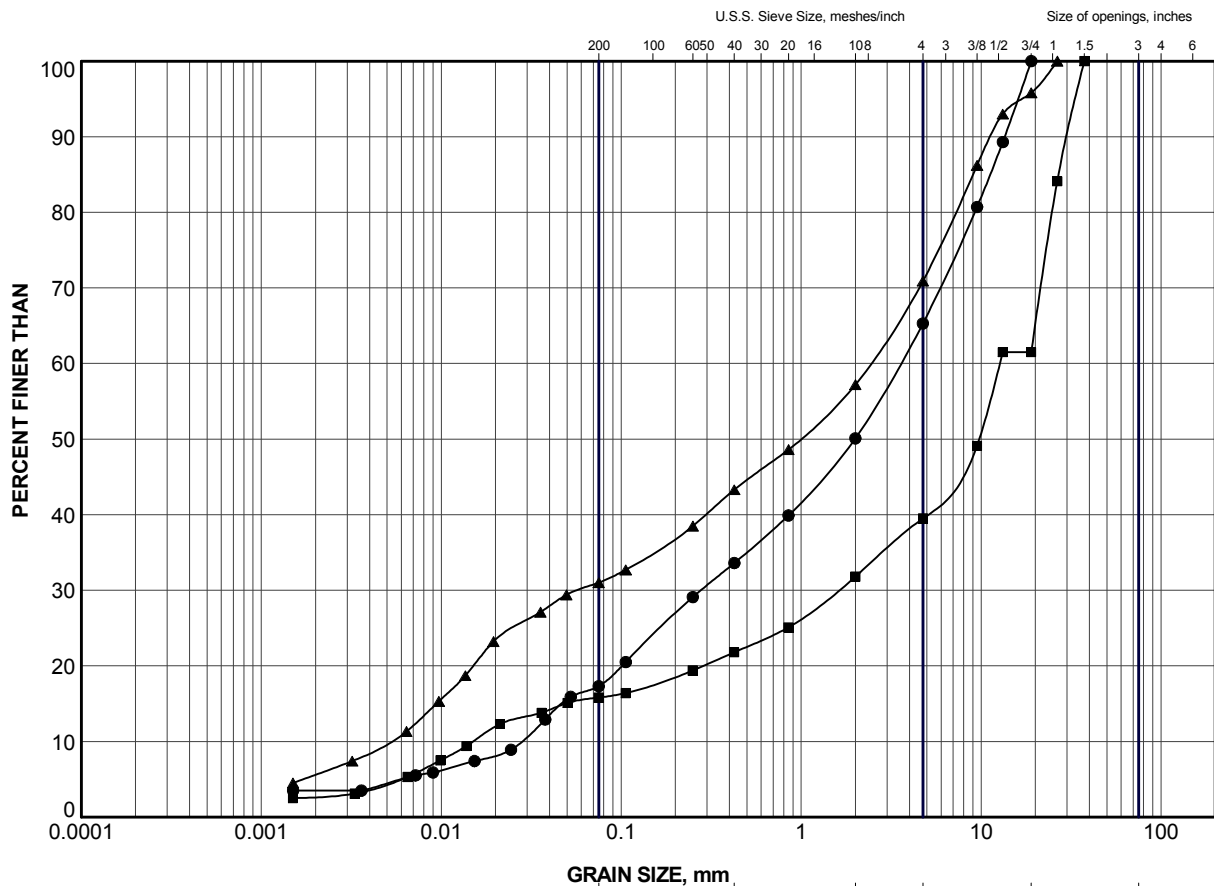


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C1	4b	314.3
■	C1	7	312.0
▲	C2	8	311.1
+	C3	4	313.7
◆	C4	4	313.3
◇	C5	6	312.7


PROJECT					
HIGHWAY 66 - CUT SECTION STA 15+590 TO 15+610					
TITLE					
GRAIN SIZE DISTRIBUTION SILT					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	May 2013	SCALE	N/A	REV.
CHECK	SEMC	May 2013			
APPR	JMAC	May 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE E4		

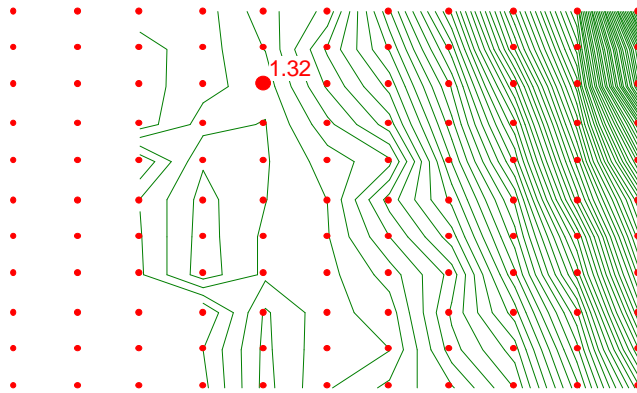


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C3	8	309.2
■	C4	8	309.5
▲	C5	9	308.9

PROJECT					
HIGHWAY 66 - CUT SECTION STA 15+590 TO 15+610					
TITLE					
GRAIN SIZE DISTRIBUTION SILTY GRAVELLY SAND to SAND AND GRAVEL					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	JJL	May 2013	SCALE	N/A	REV.
CHECK	SEMC	May 2013			
APPR	JMAC	May 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE E5		

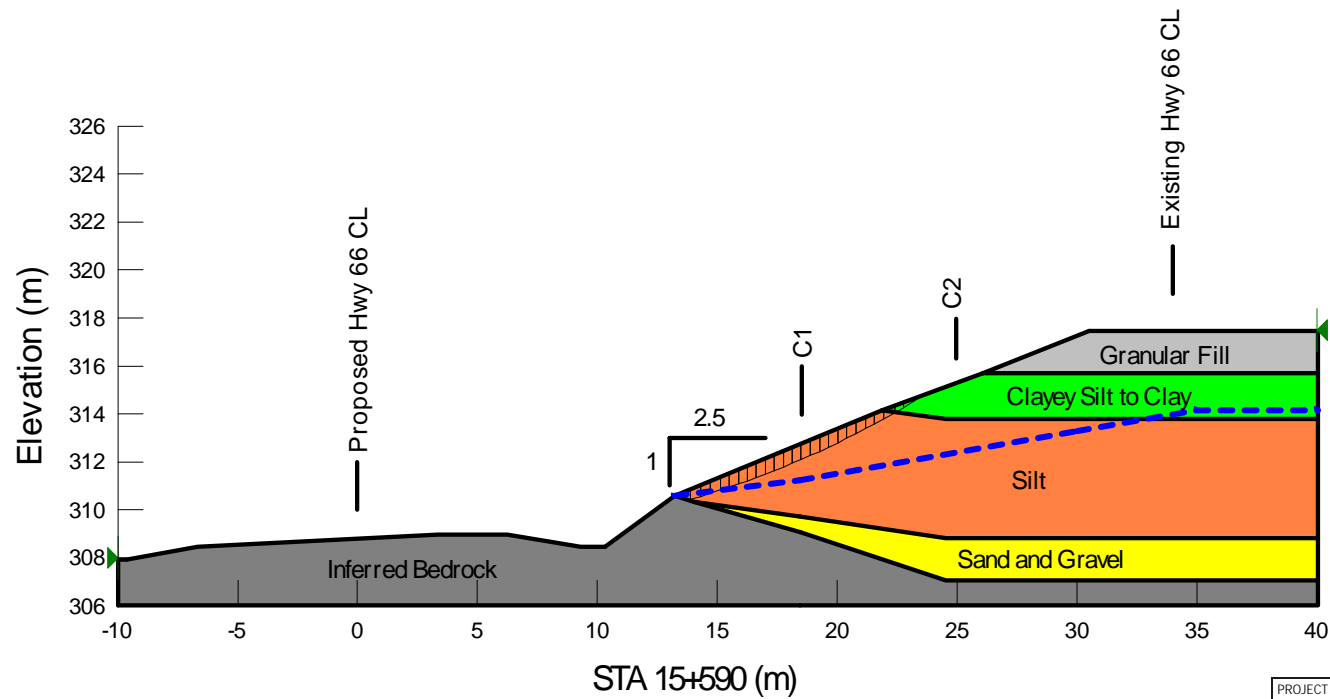


Granular Fill
Unit Weight: 21 kN/m³
Phi: 35°

Clayey Silt to Clay
Unit Weight: 17.5 kN/m³
Phi: 30°

Silt
Unit Weight: 18 kN/m³
Phi: 28°

Sand and Gravel to Gravel
Unit Weight: 20 kN/m³
Phi: 32°



PROJECT		HIGHWAY 66 DEEP CUT	
TITLE		STABILITY ANALYSIS FINAL CUT SLOPE AT STA 15+590	
PROJECT No. 10-1191-0044		FILE No. ----	
DESIGN	MT	MAY 2013	SCALE AS SHOWN REV.
CADD	--		
CHECK	SEMC	MAY 2013	
REVIEW	JMAC	MAY 2013	

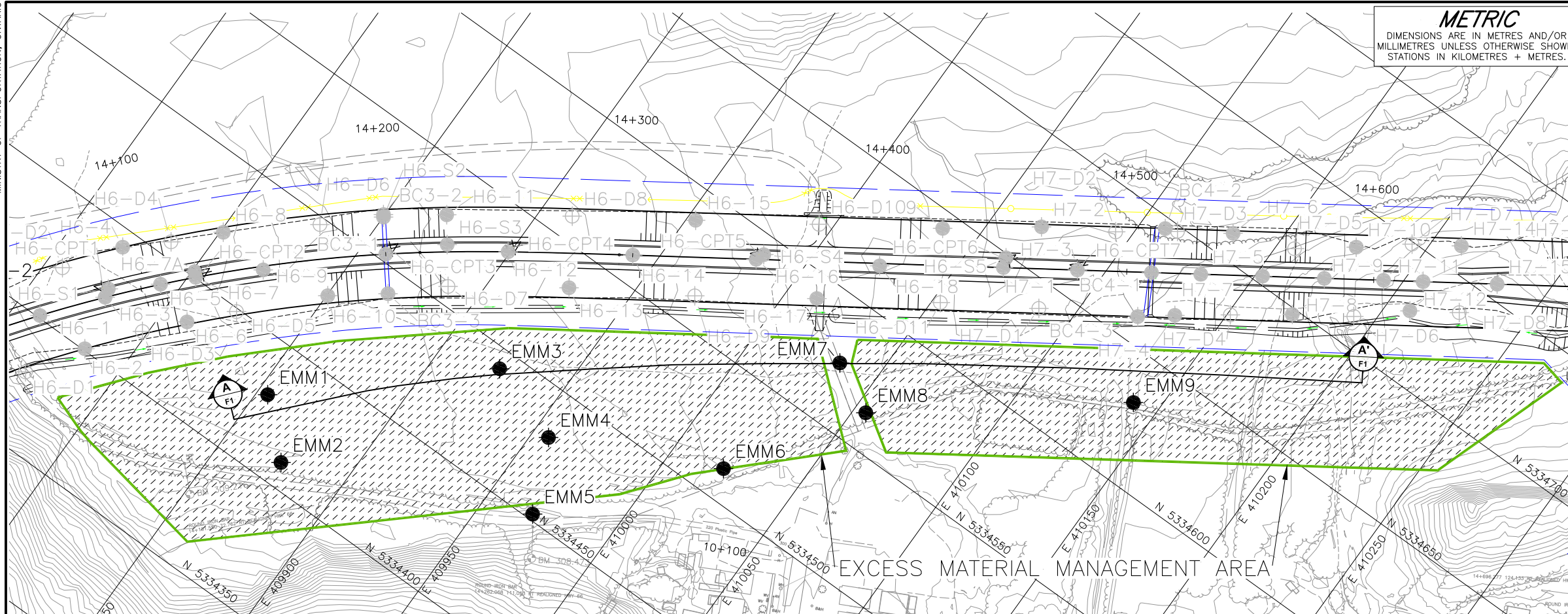


FIGURE E6

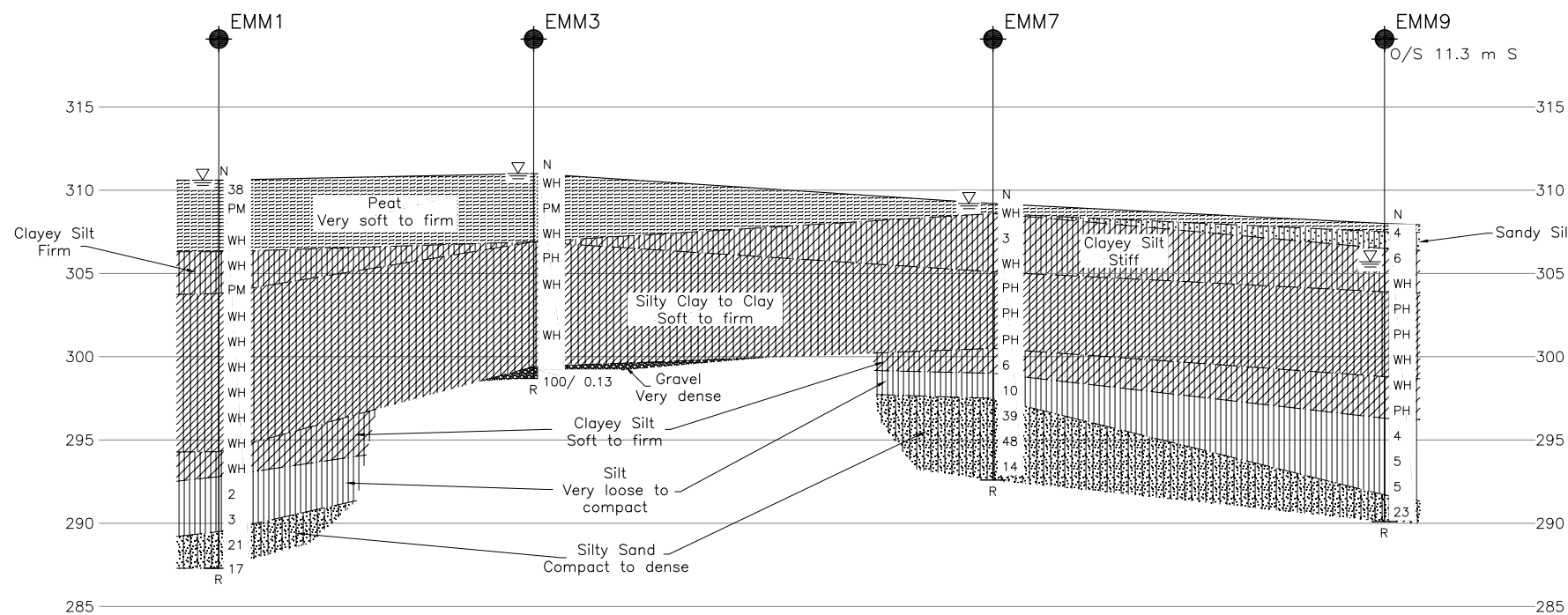
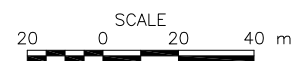
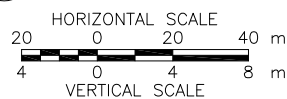
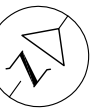


APPENDIX F

Excess Material Management Area



PLAN

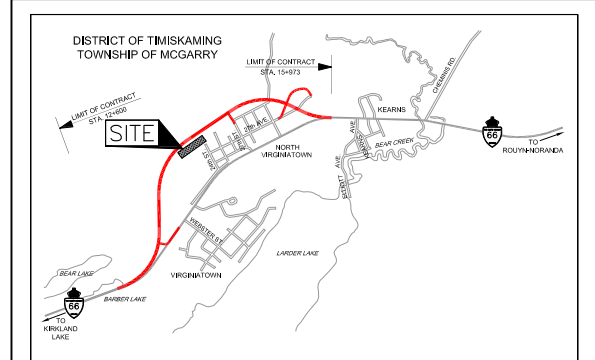
A-A'
F1PROFILE
HIGHWAY 66CONT No.
GWP No. 5091-07-00

HIGHWAY 66

Excess Material Management Area
STA 14+050 TO 14+675

SHEET

BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

KEY PLAN

SCALE
700 0 700 m

LEGEND

- Borehole
- Dynamic Cone Penetration Test
- CPT
- Standard Penetration Test Value
- Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Refusal
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
EMM1	310.6	5334428.9	409854.1
EMM2	311.3	5334409.8	409874.5
EMM3	311.0	5334492.3	409924.1
EMM4	309.1	5334481.3	409956.4
EMM5	310.6	5334452.4	409969.3
EMM6	309.4	5334512.5	410021.2
EMM7	309.2	5334574.7	410034.4
EMM8	310.3	5334564.5	410054.7
EMM9	308.0	5334631.2	410140.1

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.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

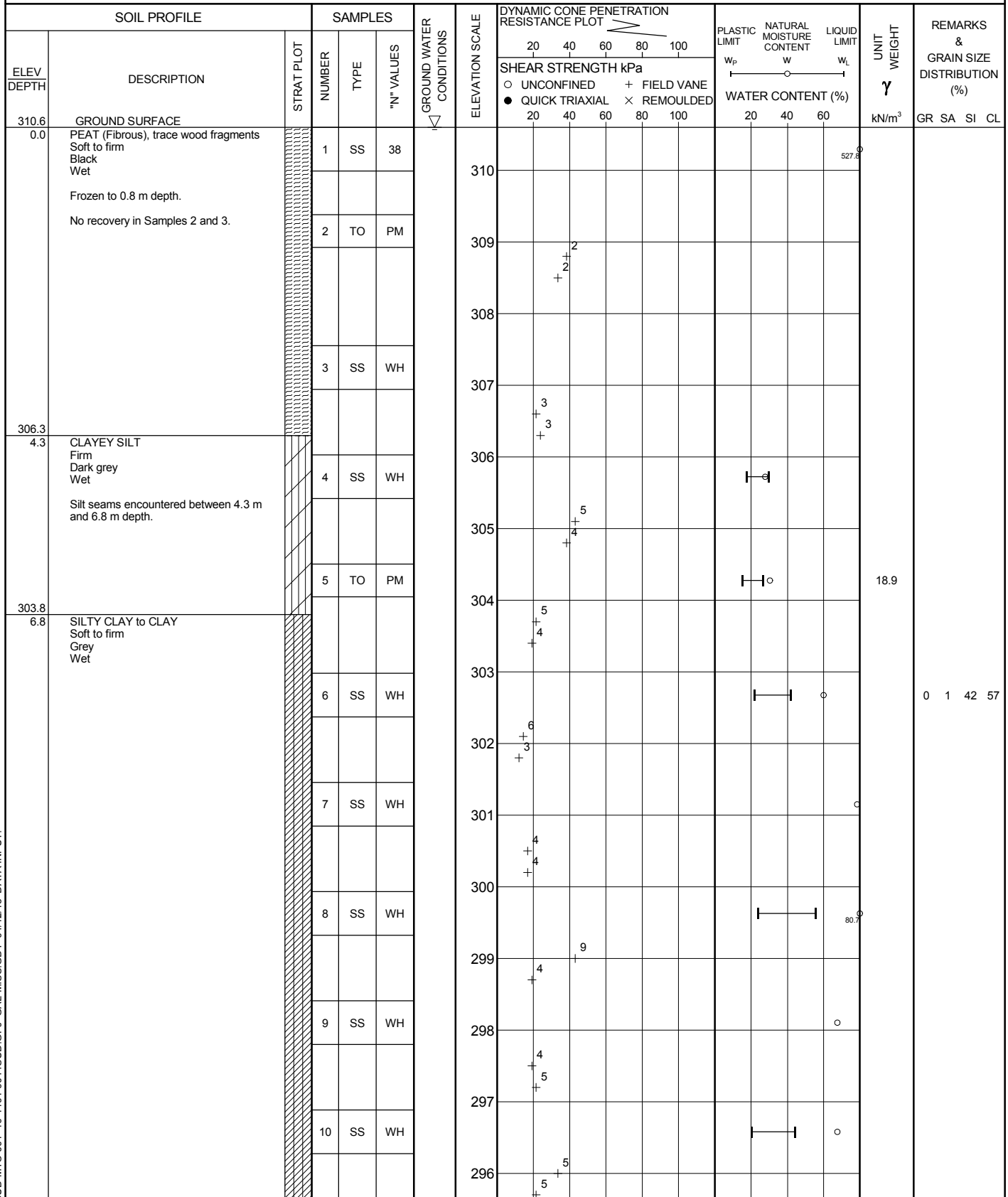
REFERENCE

Base plans provided in digital format by MMM, drawing file nos. H3211009D16 ROLL PLAN-ULTIMATE and PDR.dwg, received DEC 3, 2012. Keyplan drawing file nos. H3211009G02 received JAN 24, 2013.



NO.	DATE	BY	REVISION
Geocres No. 32D-17			
HWY. 66	PROJECT NO. 10-1191-0044		DIST.
SUBM'D. MT	CHKD.	DATE: DEC 2013	SITE:
DRAWN: TB	CHKD. SEMC	APPD. JMAC	DWG. F1

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No EMM1		1 OF 2 METRIC	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334428.9; E 409854.1</u>		ORIGINATED BY <u>BM</u>	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u>		COMPILED BY <u>AC</u>	
DATUM <u>GEODETIC</u>		DATE <u>May 5, 2013</u>		CHECKED BY <u>SEMC</u>	



Continued Next Page

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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044		RECORD OF BOREHOLE No EMM1				2 OF 2 METRIC										
G.W.P. 5091-07-00		LOCATION N 5334428.9; E 409854.1				ORIGINATED BY BM										
DIST _____ HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring				COMPILED BY AC										
DATUM GEODETIC		DATE May 5, 2013				CHECKED BY SEMC										
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 ---															
294.3	SILTY CLAY to CLAY Soft to firm Grey Wet		11	SS	WH											
16.3	CLAYEY SILT, trace sand Soft to firm Grey Wet		12	SS	WH											
292.8	SILT, trace to some clay, trace sand Very loose Grey Wet		13	SS	2											
17.8	SILT, trace to some clay, trace sand Very loose Grey Wet		14	SS	3											
289.5	Silty SAND, some gravel, trace sand Compact Grey Wet		15	SS	21											
21.1	One 25 mm gravel piece recovered in Sample 16.		16	SS	17											
287.3	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING) Note: 1. Water level at ground surface (Elev. 310.6 m) upon completion of drilling.															

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

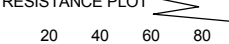
PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE		No EMM2	1 OF 2	METRIC
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334409.8; E 409874.5</u>		ORIGINATED BY <u>BM</u>		
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u>		COMPILED BY <u>AC</u>		
DATUM <u>GEODETIC</u>		DATE <u>May 5, 2013</u>		CHECKED BY <u>SEMC</u>		

[illegible]




Continued Next Page

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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>			RECORD OF BOREHOLE No EMM2				2 OF 2 METRIC				
G.W.P. <u>5091-07-00</u>			LOCATION <u>N 5334409.8; E 409874.5</u>				ORIGINATED BY <u>BM</u>				
DIST <u> </u> HWY <u>66</u>			BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u>				COMPILED BY <u>AC</u>				
DATUM <u>GEODETIC</u>			DATE <u>May 5, 2013</u>				CHECKED BY <u>SEMC</u>				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
294.8	SILT, trace clay, trace sand		11	SS	8		296				
295							295				
16.5	Silty SAND, some gravel, trace sand Compact Grey Wet		12	SS	27		294				
293.5	One 25 mm gravel piece recovered in Sample 13.		13	SS	100/ 0.08						
17.8	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING) Note: 1. Water level at ground surface (Elev. 311.3 m) upon completion of drilling.										

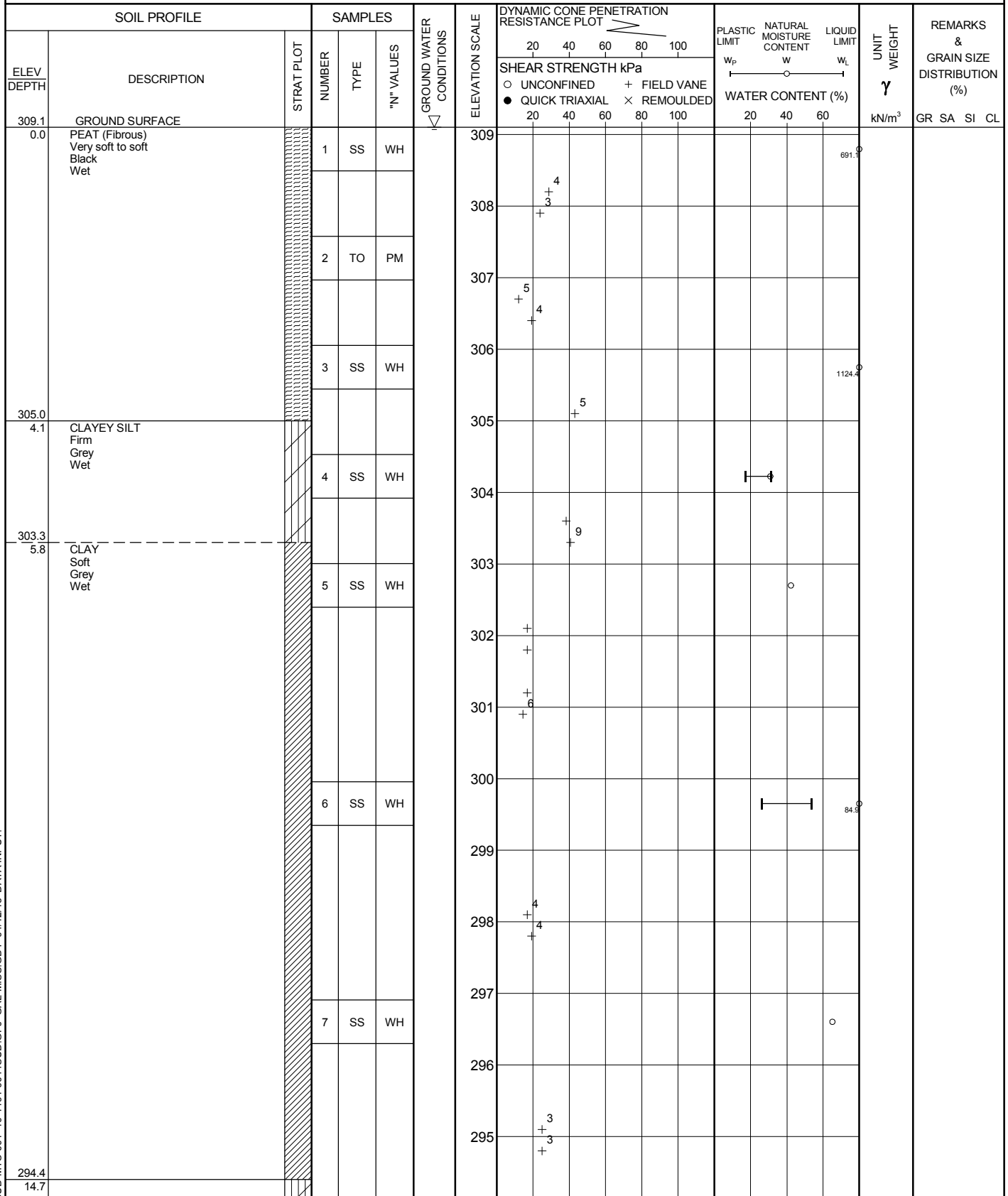
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PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No EMM3		1 OF 1 METRIC																	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334492.3; E 409924.1</u>		ORIGINATED BY <u>BM</u>																	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u>		COMPILED BY <u>AC</u>																	
DATUM <u>GEODETIC</u>		DATE <u>May 7, 2013</u>		CHECKED BY <u>SEMC</u>																	
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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 (%) W _p — W — W _L			γ			GR SA SI CL		
311.0 0.0	GROUND SURFACE PEAT (Fibrous) Soft to firm Black Wet		1	SS	WH		310	3													
			2	TO	PM																
			3	SS	WH																
306.9 4.1	SILTY CLAY Soft to firm Grey Wet		4	TO	PH				307												
							306	7													
							305	10													
		5	SS	WH			304														
							303	7													
							302														
		6	SS	WH			301														
							300	2													
							299	3													
299.4 11.6	GRAVEL, some sand, trace silt Very dense Brown Wet																				
298.7 12.3	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING) Note: 1. Water level at ground surface (Elev. 311.0 m) upon completion of drilling.		7	SS	100/ 0.13																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No EMM4		1 OF 2 METRIC	
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334481.3; E 409956.4</u>		ORIGINATED BY <u>BM</u>	
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u>		COMPILED BY <u>AC</u>	
DATUM <u>GEODETIC</u>		DATE <u>May 6, 2013</u>		CHECKED BY <u>SEMC</u>	



SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

Continued Next Page

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

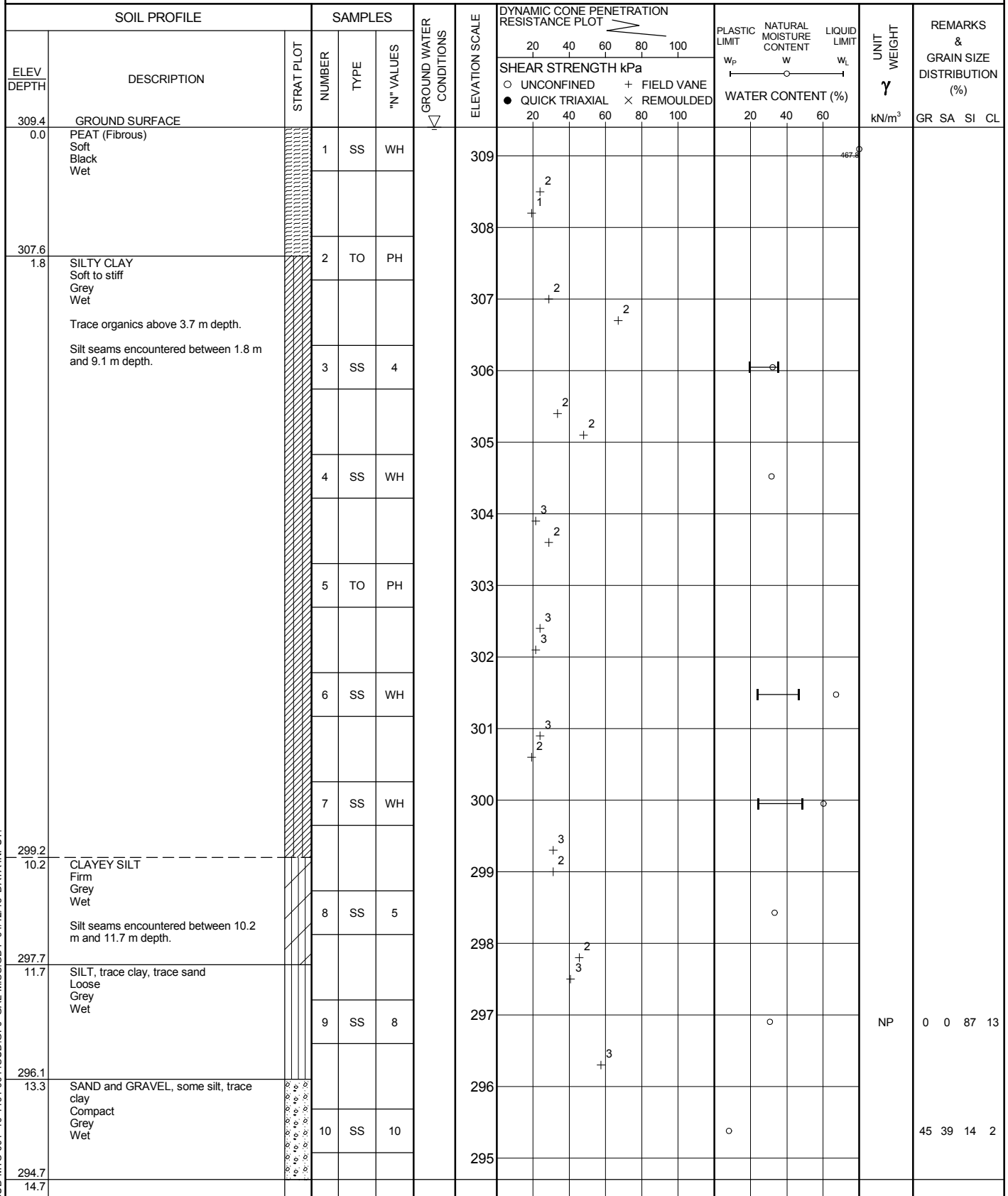
PROJECT <u>10-1191-0044</u>			RECORD OF BOREHOLE No EMM4				2 OF 2 METRIC															
G.W.P. <u>5091-07-00</u>			LOCATION <u>N 5334481.3; E 409956.4</u>				ORIGINATED BY <u>BM</u>															
DIST <u> </u> HWY <u>66</u>			BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u>				COMPILED BY <u>AC</u>															
DATUM <u>GEODETIC</u>			DATE <u>May 6, 2013</u>				CHECKED BY <u>SEMC</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			SHEAR STRENGTH kPa					W _p W W _L									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100										
289.6	CLAYEY SILT, trace sand Soft to firm Grey Wet		8	SS	8																	
			9	SS	3																	
			10	SS	2																	
289.0																						
19.5	Gravelly Silty SAND, trace clay Loose Grey Wet		11	SS	8																	
			12	SS	5																	
286.1	Gravel pieces recovered in Sample 13.																					
23.0	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING) Note: 1. Water level at ground surface (Elev. 309.1 m) upon completion of drilling.		13	SS	100/ 0.13																	

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044			RECORD OF BOREHOLE No EMM5			1 OF 1 METRIC							
G.W.P. 5091-07-00			LOCATION N 5334452.4; E 409969.3			ORIGINATED BY BM							
DIST _____ HWY 66			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring			COMPILED BY AC							
DATUM GEODETIC			DATE May 6, 2013			CHECKED BY SEMC							
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					
310.6	GROUND SURFACE												
0.0	PEAT (Fibrous), trace gravel, trace sand Very soft to firm Black Wet		1	SS	4								
			2	TO	PM								
307.6			3	SS	5								
3.0	CLAYEY SILT, trace sand, trace organics Soft Grey Wet												
			4	SS	1								
305.3													
5.3	SILTY CLAY Soft Grey Wet												
			5	SS	WH								
			6	TO	PM								
			7	SS	51								
300.2													
299.9	Silty SAND, trace to some clay, trace gravel Very dense Grey Wet		8	SS	100/0.05								
10.7	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING)												
	Note: 1. Water level at ground surface (Elev. 310.6 m) upon completion of drilling.												

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT 10-1191-0044		RECORD OF BOREHOLE No EMM6		1 OF 2 METRIC	
G.W.P. 5091-07-00		LOCATION N 5334512.5; E 410021.2		ORIGINATED BY MR	
DIST HWY 66		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring		COMPILED BY AC	
DATUM GEODETIC		DATE May 14, 2013		CHECKED BY SEMC	



SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

Continued Next Page

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




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

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE		No EMM7	1 OF 2	METRIC
G.W.P. <u>5091-07-00</u>	LOCATION <u>N 5334574.7; E 410034.4</u>	ORIGINATED BY <u>MR</u>				
DIST <u> </u> HWY <u>66</u>	BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>	COMPILED BY <u>AC</u>				
DATUM <u>GEODETIC</u>	DATE <u>May 14, 2013</u>	CHECKED BY <u>SEMC</u>				

[illegible]

Continued Next Page

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

PROJECT <u>10-1191-0044</u>				RECORD OF BOREHOLE No EMM7				2 OF 2 METRIC									
G.W.P. <u>5091-07-00</u>				LOCATION <u>N 5334574.7; E 410034.4</u>				ORIGINATED BY <u>MR</u>									
DIST <u> </u> HWY <u>66</u>				BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>AC</u>									
DATUM <u>GEODETIC</u>				DATE <u>May 14, 2013</u>				CHECKED BY <u>SEMC</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									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)				
								20	40	60	80	100	20	40	60		
292.6	Silty SAND, trace to some gravel Compact to dense Grey Wet Spoon attempted at 16.6 m depth; spoon bouncing.		11	SS	14		294										
16.6	END OF BOREHOLE SPOON REFUSAL (SPOON BOUNCING) Note: 1. Water level at ground surface (Elev. 309.2 m) upon completion of drilling.						293										

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

RECORD OF BOREHOLE No EMM8

1 OF 2 **METRIC**

PROJECT 10-1191-0044

G.W.P. 5091-07-00

LOCATION N 5334564.5; E 410054.7

ORIGINATED BY BM

DIST HWY 66





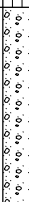
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring

COMPILED BY AC

DATUM GEODETIC

DATE May 8, 2013

CHECKED BY SEMC

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 (%)					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED							20	40	60	80	100	W _P
310.3	GROUND SURFACE							20	40	60	80	100	20	40	60		GR	SA	SI	CL
0.0	PEAT (Fibrous) Very soft Black Wet		1	SS	WH		310									159.8				
309.7																				
0.6	CLAYEY SILT, trace organics, trace sand Soft to firm Grey Wet		2	SS	5		309	10												
							308		3											
							307													
			3	SS	WH															
306.2																				
4.1	SILTY CLAY to CLAY Soft Grey Wet						306		12											
							305													
			5	SS	WH		304													
							303		9											
							302													
							301													
			6	TO	PM															
300.1																				
10.2	SILT, trace sand, trace clay Compact Grey Wet						300													
							299													
							298													
			7	SS	10															
297.4																				
12.9	SAND and GRAVEL, some silt, trace clay Loose to dense Grey Wet						297													
			8	SS	25															
			9	SS	4		296													

Continued Next Page

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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

PROJECT <u>10-1191-0044</u>		RECORD OF BOREHOLE No EMM8				2 OF 2 METRIC					
G.W.P. <u>5091-07-00</u>		LOCATION <u>N 5334564.5; E 410054.7</u>				ORIGINATED BY <u>BM</u>					
DIST <u> </u> HWY <u>66</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u>				COMPILED BY <u>AC</u>					
DATUM <u>GEODETIC</u>		DATE <u>May 8, 2013</u>				CHECKED BY <u>SEMC</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	20 40 60 80 100	W _p W W _L		
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SAND and GRAVEL, some silt, trace clay Loose to dense Grey Wet		10	SS	11		295				
							294				
			11	SS	17		293				
			12	SS	41						
292.0	Spoon attempted at 18.3 m depth; spoon bouncing.						292				
18.3	END OF BOREHOLE SPOON REFUSAL Note: 1. Water level at ground surface (Elev. 310.3 m) upon completion of drilling.										

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

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

PROJECT 10-1191-0044				RECORD OF BOREHOLE No EMM9				2 OF 2 METRIC									
G.W.P. 5091-07-00				LOCATION N 5334631.2; E 410140.1				ORIGINATED BY MR									
DIST _____ HWY 66				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing				COMPILED BY AC									
DATUM GEODETIC				DATE May 15, 2013				CHECKED BY SEMC									
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									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
291.7	SILT, some clay Loose Grey Wet		11	SS	5		292										
16.3	Silty SAND Compact Grey Wet																
	No recovery in Sample 12.		12	SS	23		291										
290.1	Spoon attempted at 17.9 m depth; spoon bouncing.																
17.9	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING) AND REFUSAL TO FURTHER CASING ADVANCEMENT Note: 1. Water level at a depth of 2.3 m below ground surface (Elev. 305.7 m) upon completion of drilling.																

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 04/12/13 DATA INPUT:

CONSOLIDATION TEST SUMMARY**FIGURE F1**

Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Number	10-1191-0044	Sample Number	2
Borehole Number	EMM-5	Sample Depth, m	1.52 - 2.13

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	6		
Date Started	05/24/2013		
Date Completed	07/09/2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.89	Unit Weight, kN/m ³	10.15
Sample Diameter, cm	6.37	Dry Unit Weight, kN/m ³	2.28
Area, cm ²	31.87	Specific Gravity, measured	1.58
Volume, cm ³	60.36	Solids Height, cm	0.279
Water Content, %	344.24	Volume of Solids, cm ³	8.90
Wet Mass, g	62.46	Volume of Voids, cm ³	51.46
Dry Mass, g	14.06	Degree of Saturation, %	94.1

TEST COMPUTATIONS

Stress	Corr. Height	Void	Average Height	t ₉₀	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm ² /s	m ² /kN	cm/s
0.00	1.894	5.783	1.894				
5.91	1.873	5.709	1.884	83	9.06E-03	1.84E-03	1.63E-06
10.53	1.850	5.624	1.861	58	1.27E-02	2.73E-03	3.39E-06
20.37	1.799	5.442	1.824	540	1.31E-03	2.72E-03	3.48E-07
39.48	1.719	5.157	1.759	408	1.61E-03	2.20E-03	3.47E-07
78.02	1.591	4.697	1.655	1329	4.37E-04	1.76E-03	7.53E-08
154.75	1.408	4.041	1.499	1192	4.00E-04	1.26E-03	4.93E-08
307.71	1.201	3.300	1.304	1500	2.40E-04	7.14E-04	1.68E-08
638.16	1.014	2.633	1.108	1500	1.73E-04	2.98E-04	5.06E-09
1252.96	0.853	2.054	0.934	1220	1.51E-04	1.39E-04	2.06E-09
2481.79	0.713	1.552	0.783	1017	1.28E-04	6.02E-05	7.53E-10
1252.96	0.736	1.635	0.724				
307.71	0.824	1.951	0.780				
78.02	0.912	2.264	0.868				
20.37	0.988	2.539	0.950				
5.88	1.070	2.832	1.029				

Note:

Consolidation loading and unloading schedule assigned by the client.

Specimen taken 33cm from top of the tube

k calculated using cv based on ϕ_0 values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

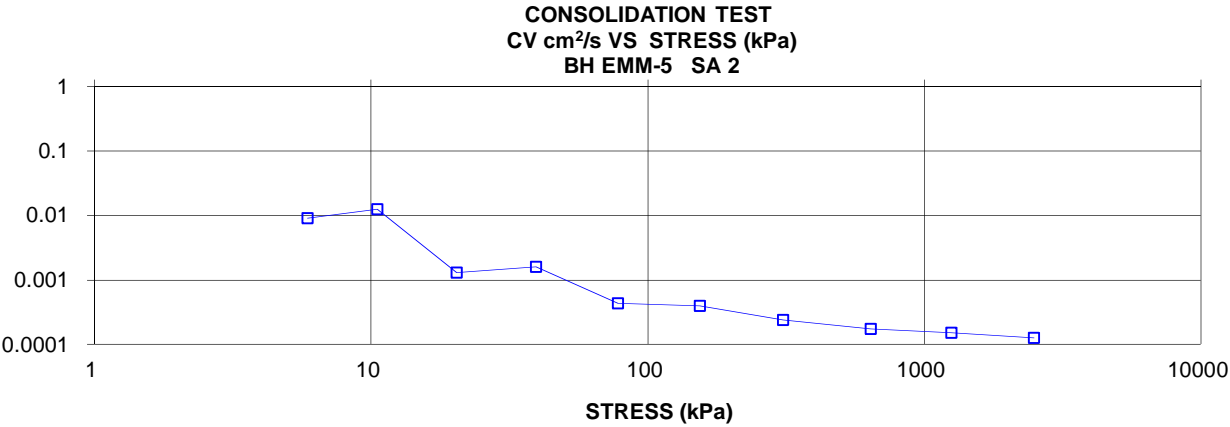
Sample Height, cm	1.07	Unit Weight, kN/m ³	11.80
Sample Diameter, cm	6.37	Dry Unit Weight, kN/m ³	4.04
Area, cm ²	31.87	Specific Gravity, measured	1.58
Volume, cm ³	34.10	Solids Height, cm	0.279
Water Content, %	191.82	Volume of Solids, cm ³	8.90
Wet Mass, g	41.03	Volume of Voids, cm ³	25.20
Dry Mass, g	14.06		

Prepared By: RD

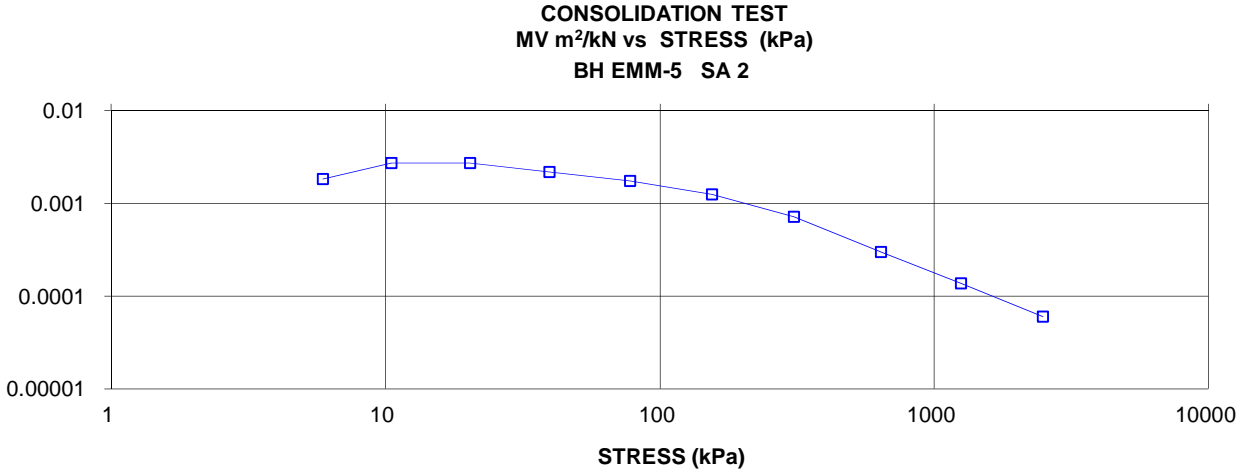
Golder Associates

Checked By: MT

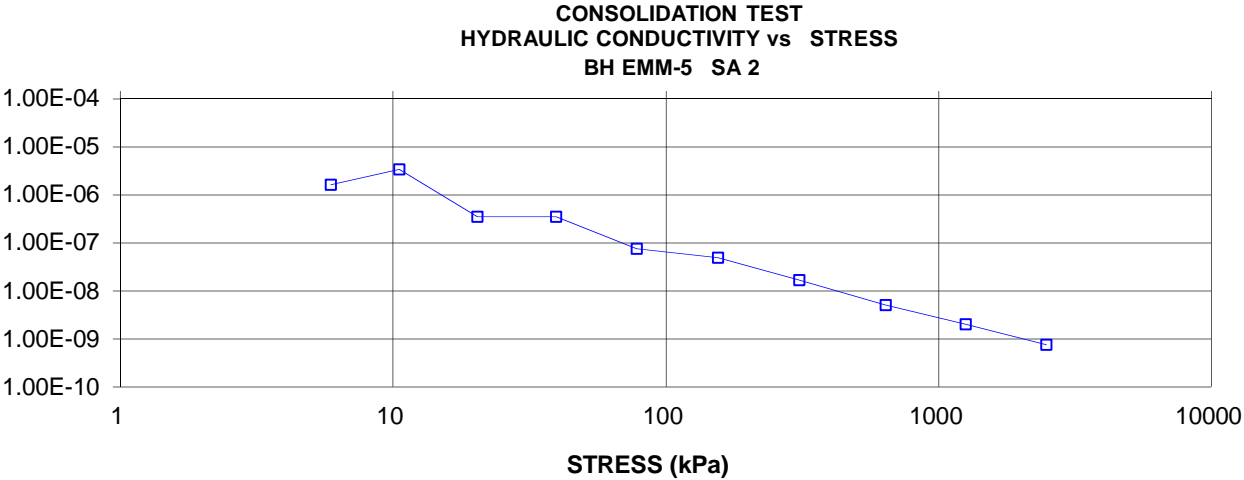
COEFFICIENT OF CONSOLIDATION,
cm²/s



VOLUME COMPRESSIBILITY, m²/kN



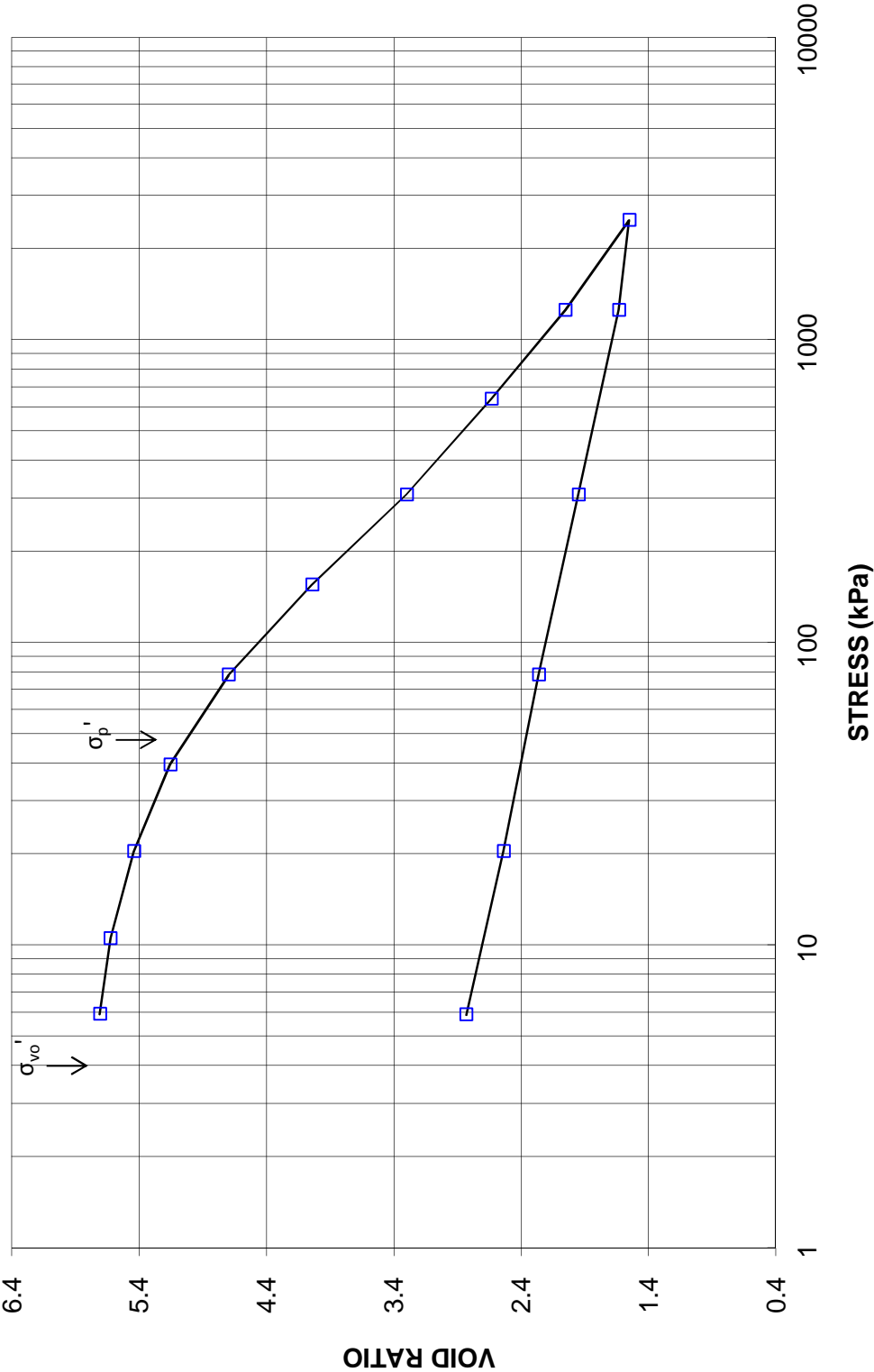
HYDRAULIC CONDUCTIVITY,
cm/s



CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE F1
Pg. 3 of 4

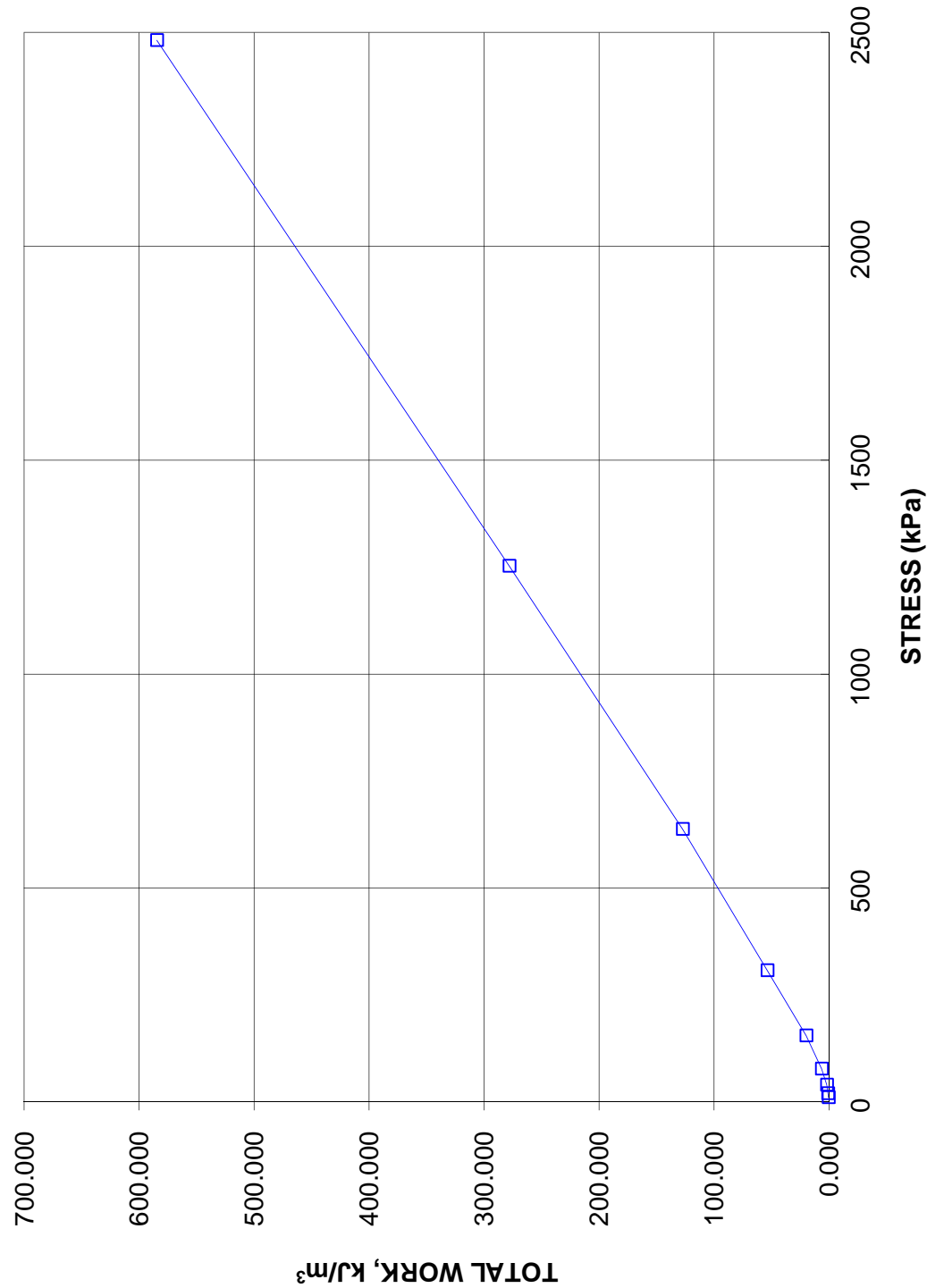
CONSOLIDATION TEST
VOID RATIO vs STRESS
BH EMM-5 SA 2

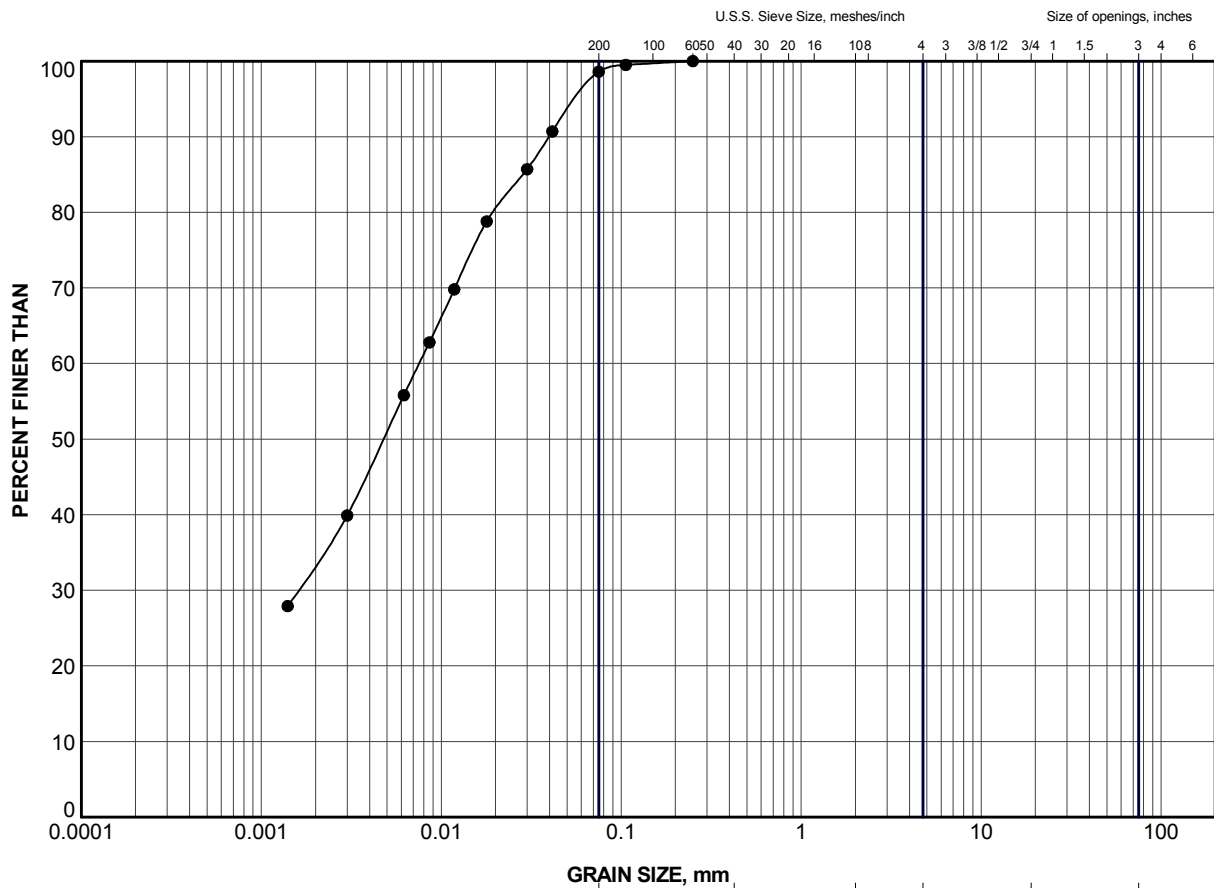


CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE F1
Pg. 4 of 4

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs STRESS
BH EMM-5 SA 2




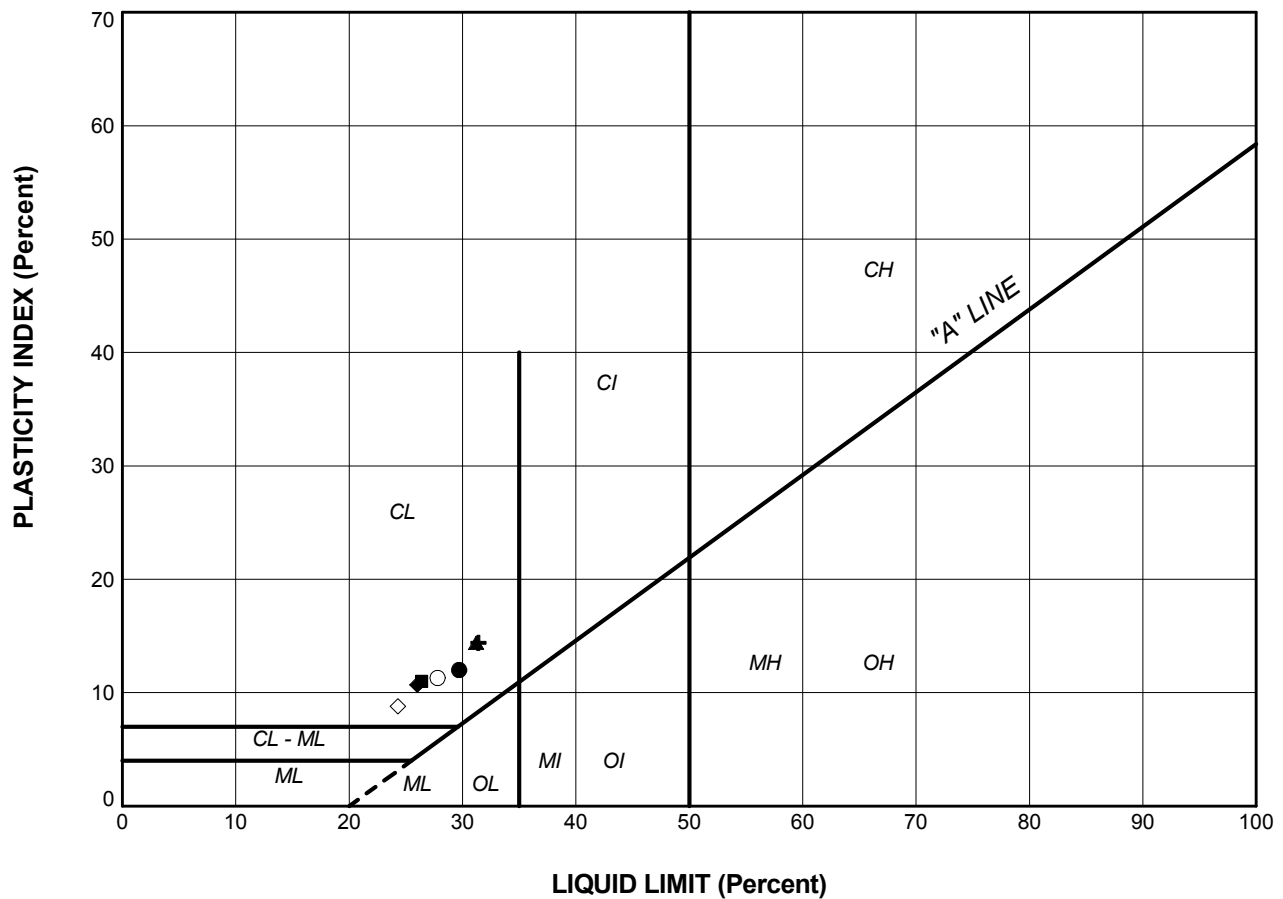


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	EMM7	3	305.9

PROJECT					
HIGHWAY 66 EXCESS MATERIAL MANAGEMENT AREA					
TITLE					
GRAIN SIZE DISTRIBUTION CLAYEY SILT					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	TB	Nov 2013	SCALE	N/A	REV.
CHECK	SEMC	Nov 2013			
APPR	JMAC	Nov 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE F2		




SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	EMM1	4	29.7	17.7	12.0
■	EMM1	5	26.4	15.4	11.0
▲	EMM2	5	31.2	16.8	14.4
+	EMM4	4	31.4	17.0	14.4
◆	EMM5	4	26.0	15.3	10.7
◇	EMM7	3	24.3	15.5	8.8
○	EMM9	3	27.8	16.5	11.3

PROJECT				
HIGHWAY 66 EXCESS MATERIAL MANAGEMENT AREA				
TITLE				
PLASTICITY CHART CLAYEY SILT				
PROJECT No. 10-1191-0044		FILE No. 10-1191-0044SUD.GPJ		
DRAWN	TB	Nov 2013	SCALE	N/A
CHECK	SEMC	Nov 2013	REV.	
APPR	JMAC	Nov 2013		
 Golder Associates SUDBURY, ONTARIO			FIGURE F3	

CONSOLIDATION TEST SUMMARY**FIGURE F4**

Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Number	10-1191-0044	Sample Number	5
Borehole Number	EMM-1	Sample Depth, m	6.1-6.6

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	5/24/2013		
Date Completed	6/10/2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m ³	18.91
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	14.34
Area, cm ²	31.69	Specific Gravity, measured	2.76
Volume, cm ³	60.18	Solids Height, cm	1.006
Water Content, %	31.84	Volume of Solids, cm ³	31.88
Wet Mass, g	116.01	Volume of Voids, cm ³	28.30
Dry Mass, g	87.99	Degree of Saturation, %	99.0

TEST COMPUTATIONS

Stress	Corr. Height	Void	Average Height	t ₉₀	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm ² /s	m ² /kN	cm/s
0.00	1.899	0.888	1.899				
10.59	1.899	0.887	1.899	73	1.05E-02	9.95E-06	1.02E-08
20.62	1.897	0.885	1.898	74	1.03E-02	1.05E-04	1.06E-07
40.00	1.895	0.883	1.896	38	2.01E-02	5.71E-05	1.12E-07
20.62	1.898	0.887	1.896				
10.59	1.900	0.888	1.899				
40.00	1.894	0.882	1.897	27	2.82E-02	1.04E-04	2.87E-07
78.59	1.882	0.871	1.888	60	1.26E-02	1.57E-04	1.94E-07
155.80	1.844	0.833	1.863	304	2.42E-03	2.61E-04	6.20E-08
309.06	1.769	0.758	1.806	173	4.00E-03	2.59E-04	1.01E-07
617.88	1.707	0.696	1.738	101	6.34E-03	1.05E-04	6.54E-08
1236.51	1.650	0.640	1.679	73	8.18E-03	4.79E-05	3.84E-08
2473.55	1.595	0.586	1.623	107	5.22E-03	2.35E-05	1.20E-08
1236.51	1.598	0.588	1.596				
309.06	1.618	0.608	1.608				
78.59	1.633	0.623	1.625				
20.62	1.650	0.640	1.641				
10.59	1.656	0.646	1.653				

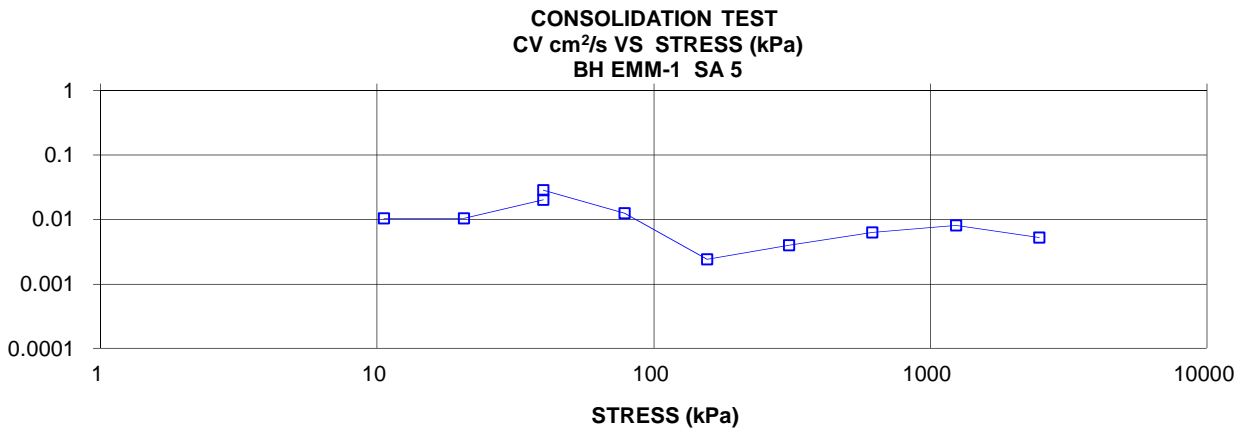
Note:

Specimen taken 6"-9" from top of the tube
k calculated using cv based on λ_0 values.

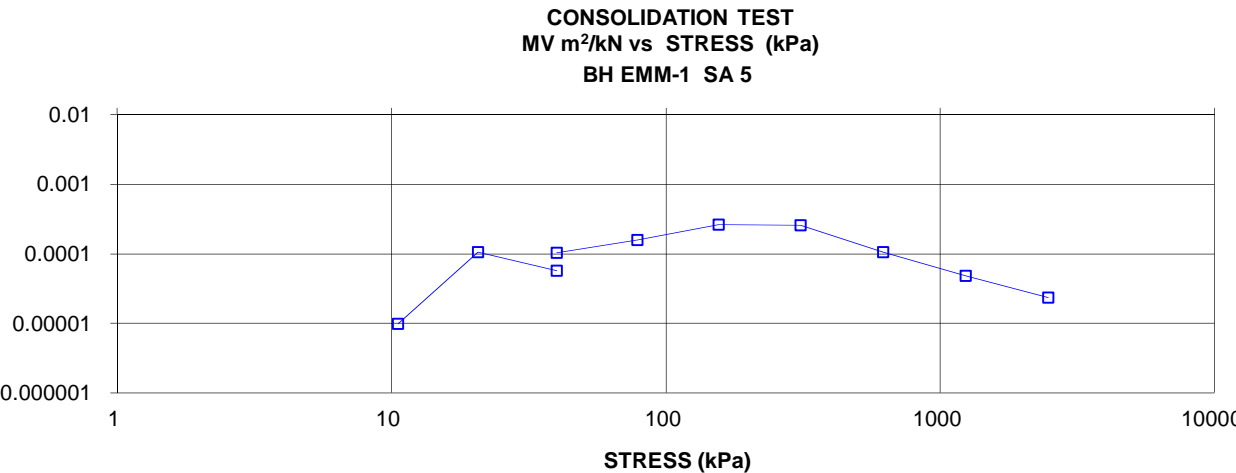
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.66	Unit Weight, kN/m ³	20.08
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	16.45
Area, cm ²	31.69	Specific Gravity, measured	2.76
Volume, cm ³	52.46	Solids Height, cm	1.006
Water Content, %	22.10	Volume of Solids, cm ³	31.88
Wet Mass, g	107.44	Volume of Voids, cm ³	20.58
Dry Mass, g	87.99		

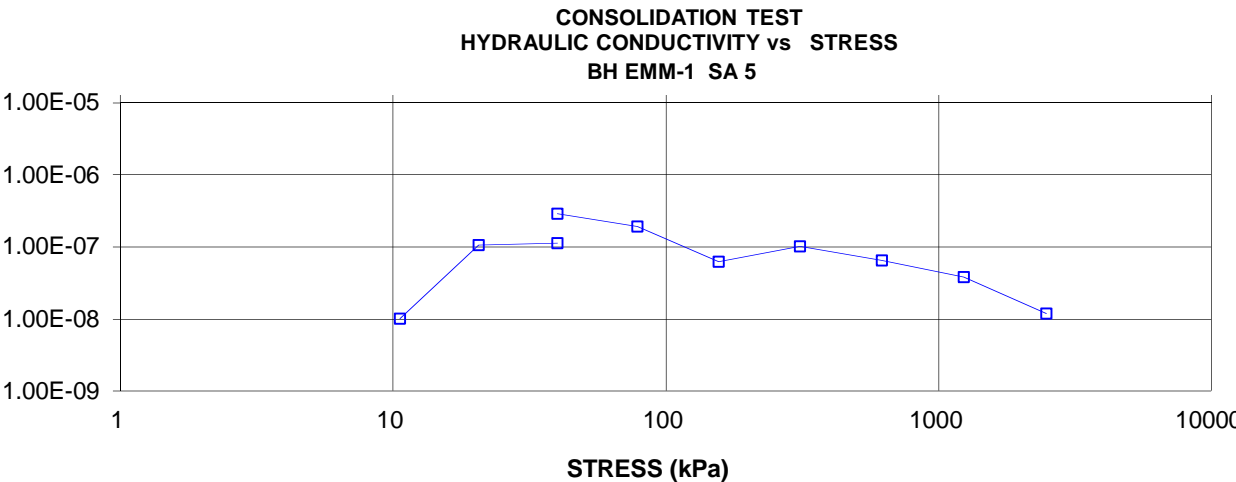
COEFFICIENT OF CONSOLIDATION,
cm²/s



VOLUME COMPRESSIBILITY, m²/kN



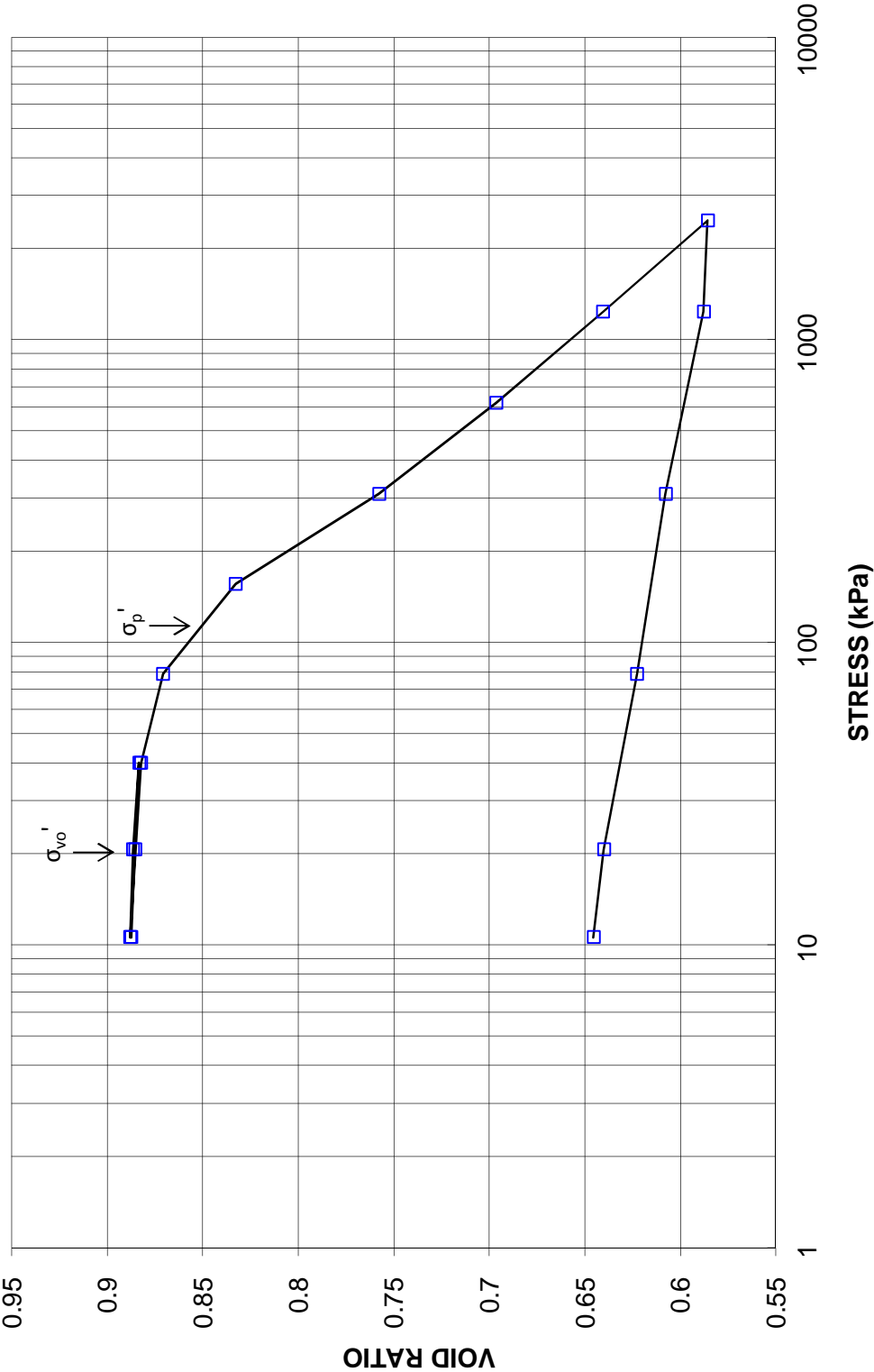
HYDRAULIC CONDUCTIVITY,
cm/s



CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE F4
Pg. 3 of 4

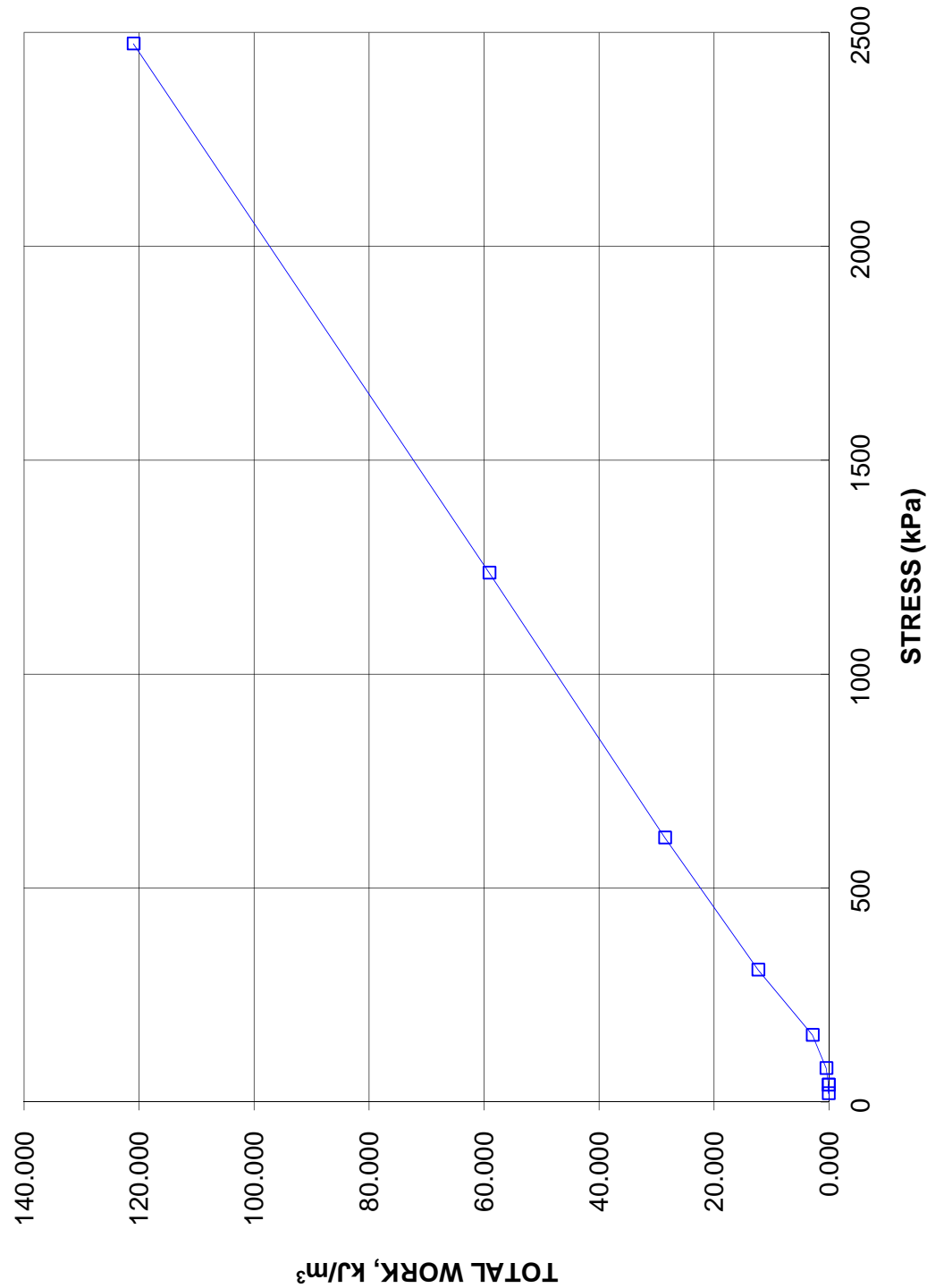
CONSOLIDATION TEST
VOID RATIO vs STRESS
BH EMM-1 SA 5

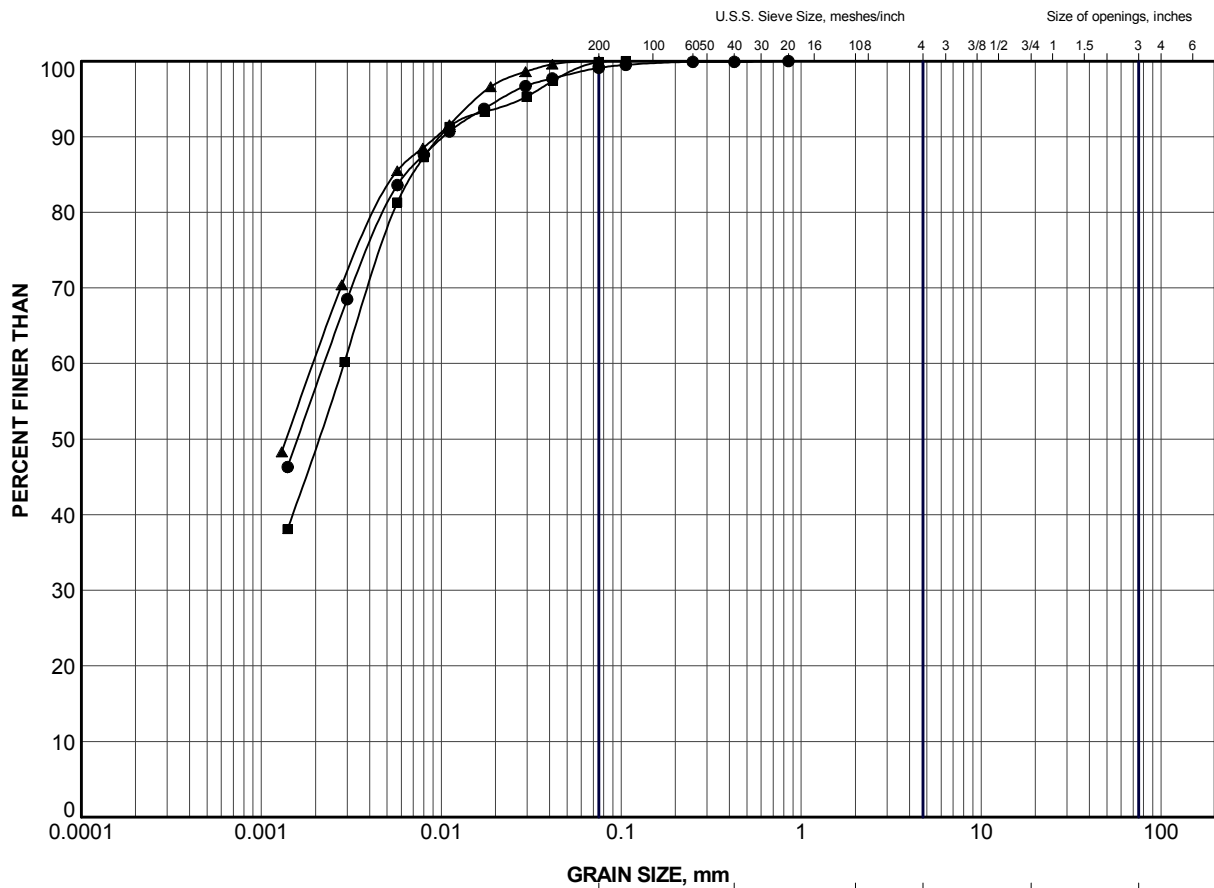


CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE F4
Pg. 4 of 4


CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs STRESS
BH EMM-1 SA 5

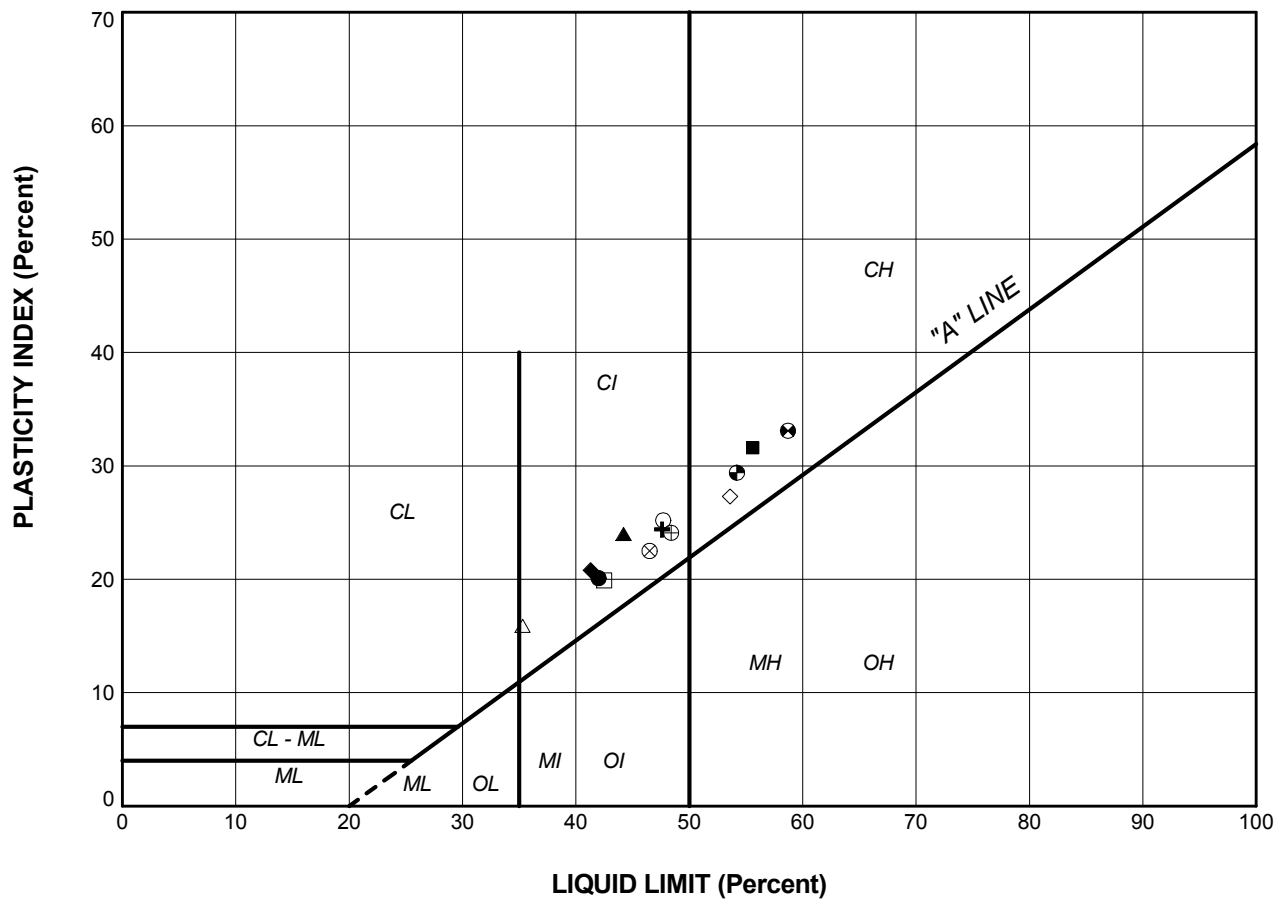




LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	EMM1	6	302.7
■	EMM2	8	300.3
▲	EMM8	5	303.9

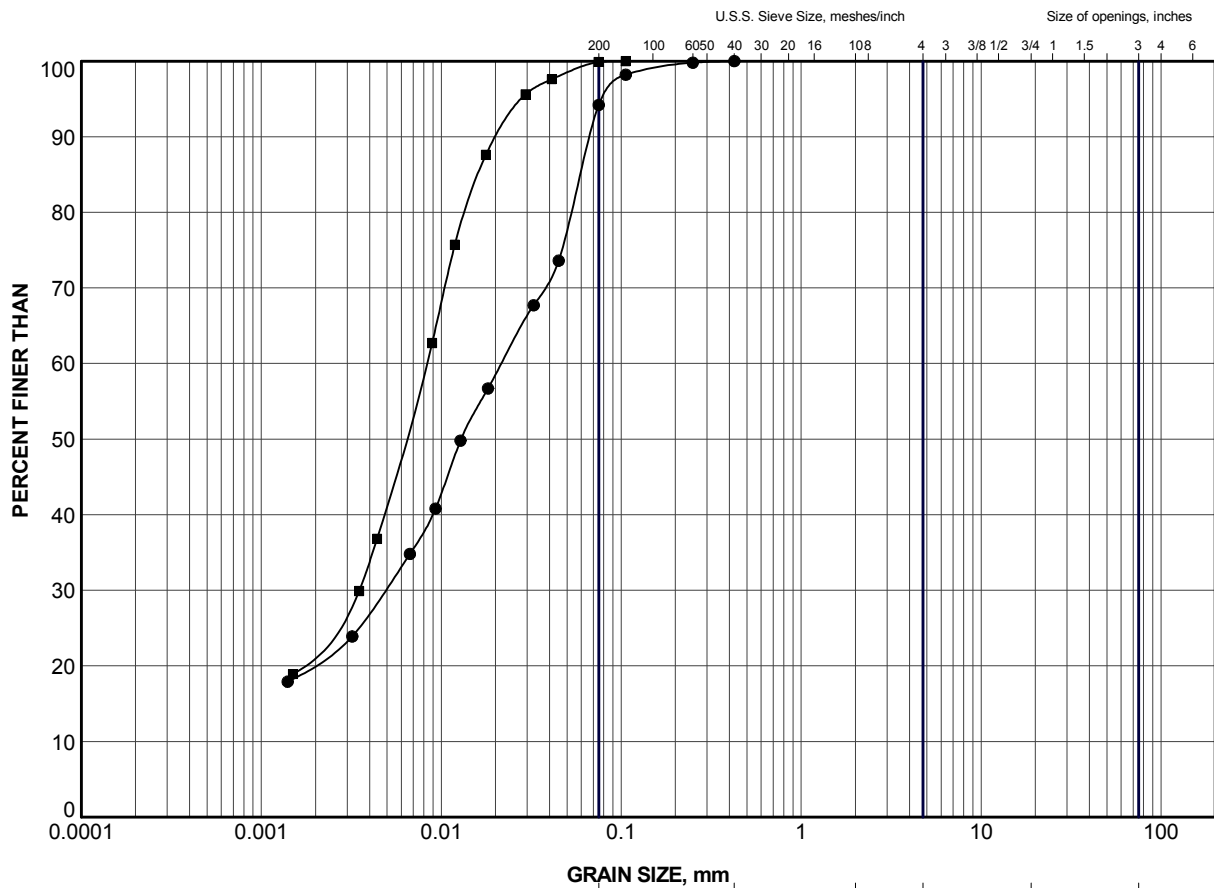
PROJECT					
HIGHWAY 66 EXCESS MATERIAL MANAGEMENT AREA					
TITLE					
GRAIN SIZE DISTRIBUTION SILTY CLAY to CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	TB	Nov 2013	SCALE	N/A	REV.
CHECK	SEMC	Nov 2013			
APPR	JMAC	Nov 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE F5		



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	EMM1	6	42.0	21.9	20.1
■	EMM1	8	55.6	24.0	31.6
▲	EMM1	10	44.2	20.2	24.0
+	EMM2	8	47.6	23.2	24.4
◆	EMM3	5	41.3	20.5	20.8
◇	EMM4	6	53.6	26.3	27.3
○	EMM5	5	47.7	22.5	25.2
△	EMM6	3	35.3	19.4	15.9
⊗	EMM6	6	46.5	24.0	22.5
⊕	EMM6	7	48.4	24.3	24.1
□	EMM8	4	42.5	22.6	19.9
⊗	EMM8	5	58.7	25.6	33.1
⊕	EMM9	6	54.2	24.8	29.4


PROJECT					
HIGHWAY 66 EXCESS MATERIAL MANAGEMENT AREA					
TITLE					
PLASTICITY CHART SILTY CLAY to CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	TB	Dec 2013	SCALE	N/A	REV.
CHECK	SEMC	Dec 2013			
APPR	JMAC	Dec 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE F6		

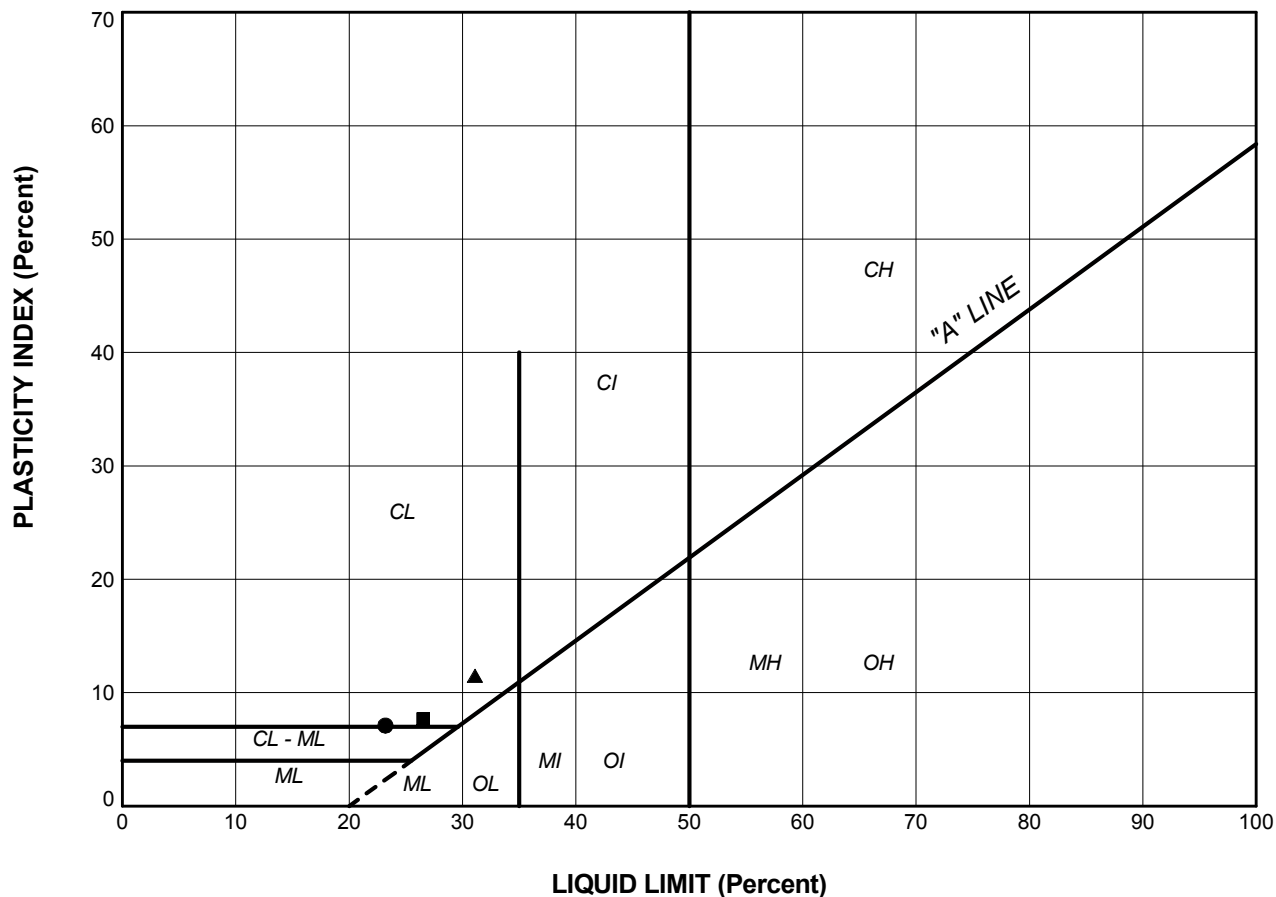



GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

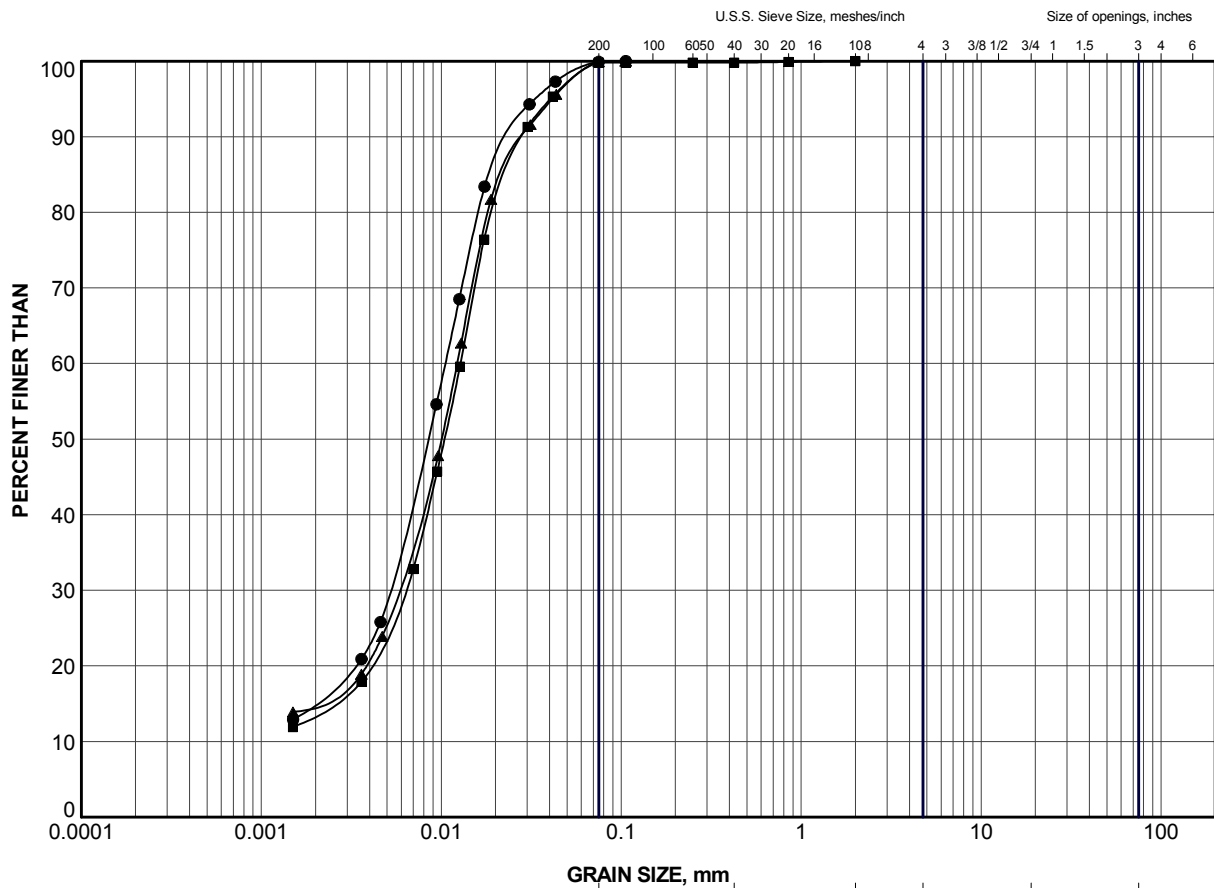
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	EMM4	10	290.5
■	EMM7	7	299.8

PROJECT					
HIGHWAY 66 EXCESS MATERIAL MANAGEMENT AREA					
TITLE					
GRAIN SIZE DISTRIBUTION CLAYEY SILT					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	TB	Nov 2013	SCALE	N/A	REV.
CHECK	SEMC	Nov 2013			
APPR	JMAC	Nov 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE F7		




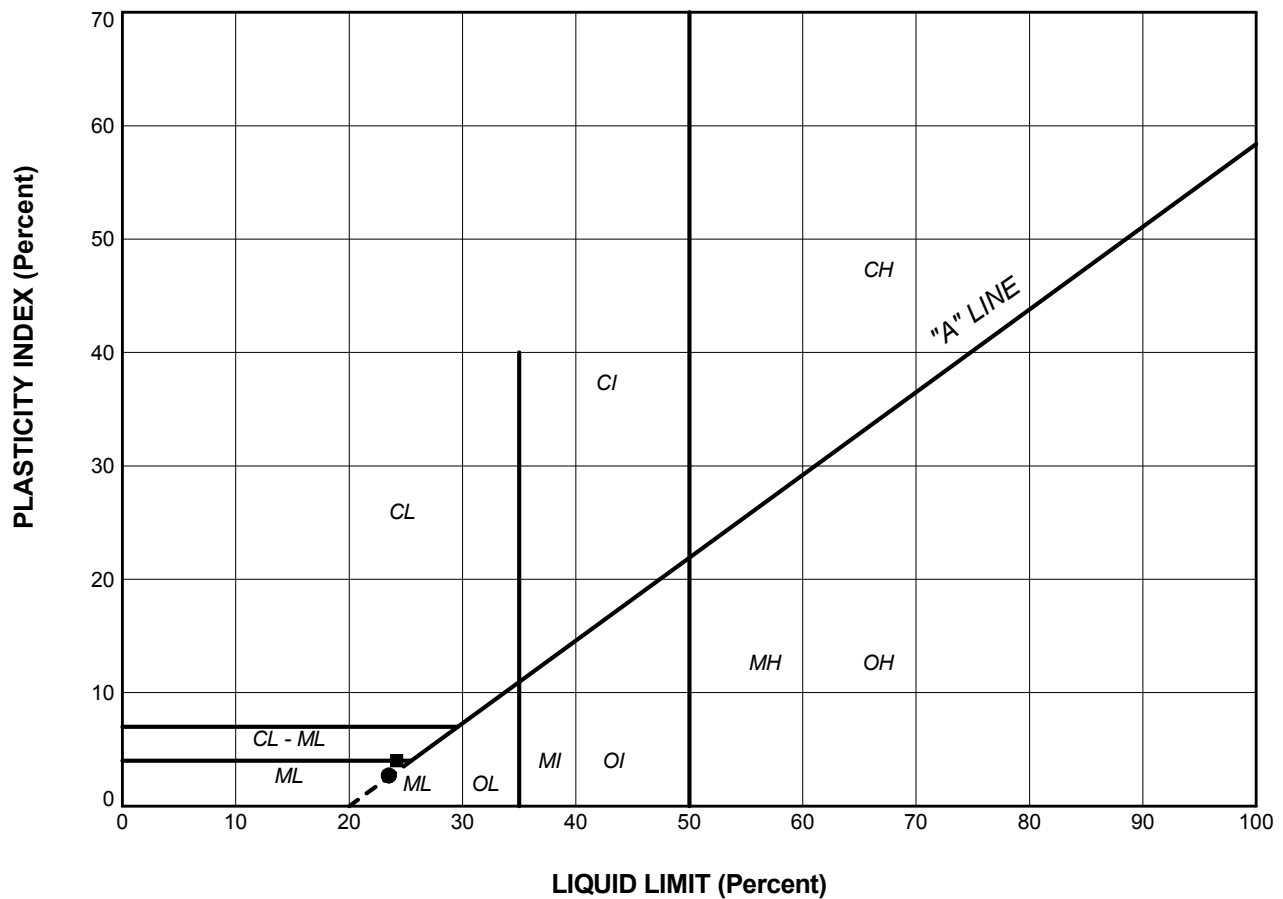
PROJECT					
HIGHWAY 66 EXCESS MATERIAL MANAGEMENT AREA					
TITLE					
PLASTICITY CHART CLAYEY SILT					
PROJECT No. 10-1191-0044			FILE No. 10-1191-0044SUD.GPJ		
DRAWN	TB	Nov 2013	SCALE	N/A	REV.
CHECK	SEMC	Nov 2013			
APPR	JMAC	Nov 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE F8		




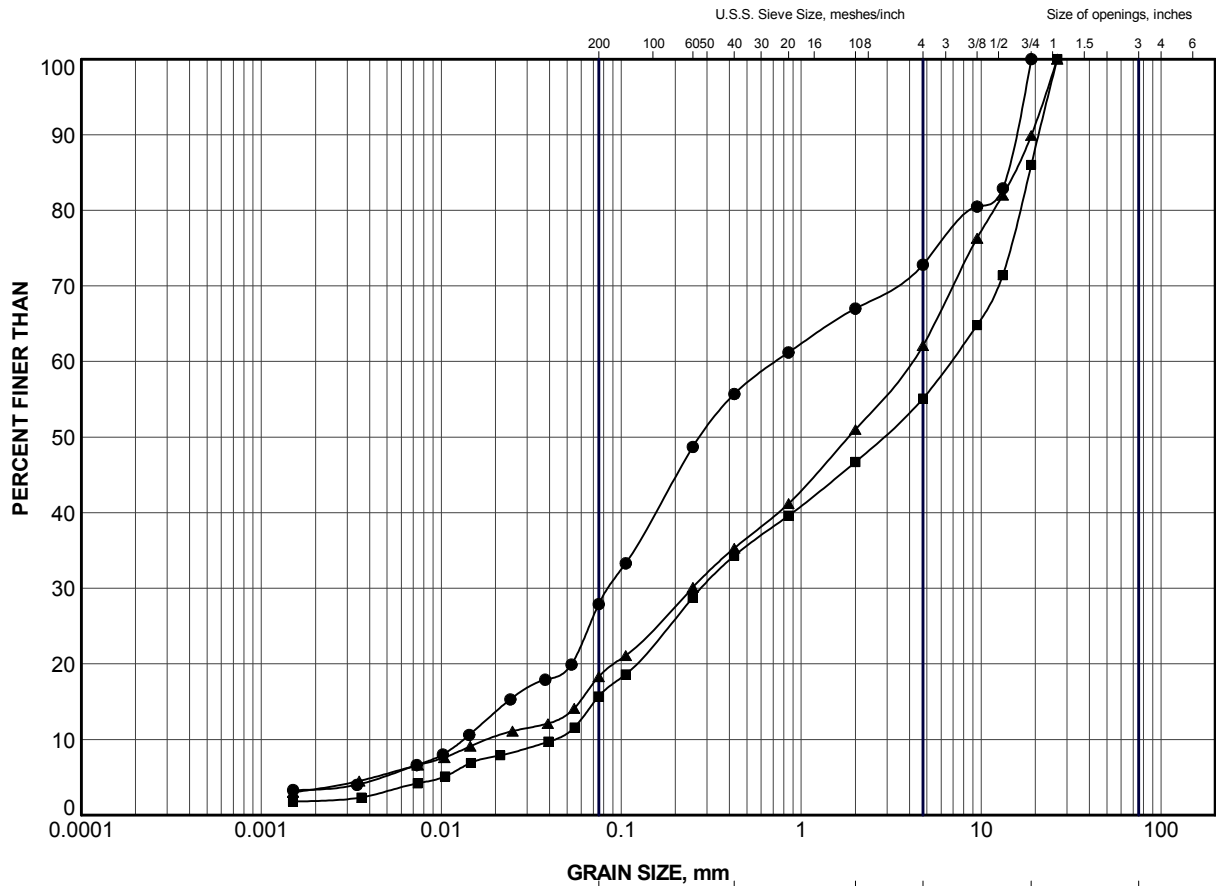
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	EMM1	13	292.0
■	EMM6	9	296.9
▲	EMM9	9	295.5

PROJECT					
HIGHWAY 66 EXCESS MATERIAL MANAGEMENT AREA					
TITLE					
GRAIN SIZE DISTRIBUTION SILT					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	TB	Nov 2013	SCALE	N/A	REV.
CHECK	SEMC	Nov 2013			
APPR	JMAC	Nov 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE F9		




PROJECT					
HIGHWAY 66 EXCESS MATERIAL MANAGEMENT AREA					
TITLE					
PLASTICITY CHART SILT					
PROJECT No. 10-1191-0044			FILE No. 10-1191-0044SUD.GPJ		
DRAWN	TB	Nov 2013	SCALE	N/A	REV.
CHECK	SEMC	Nov 2013			
APPR	JMAC	Nov 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE F10		



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

LEGEND

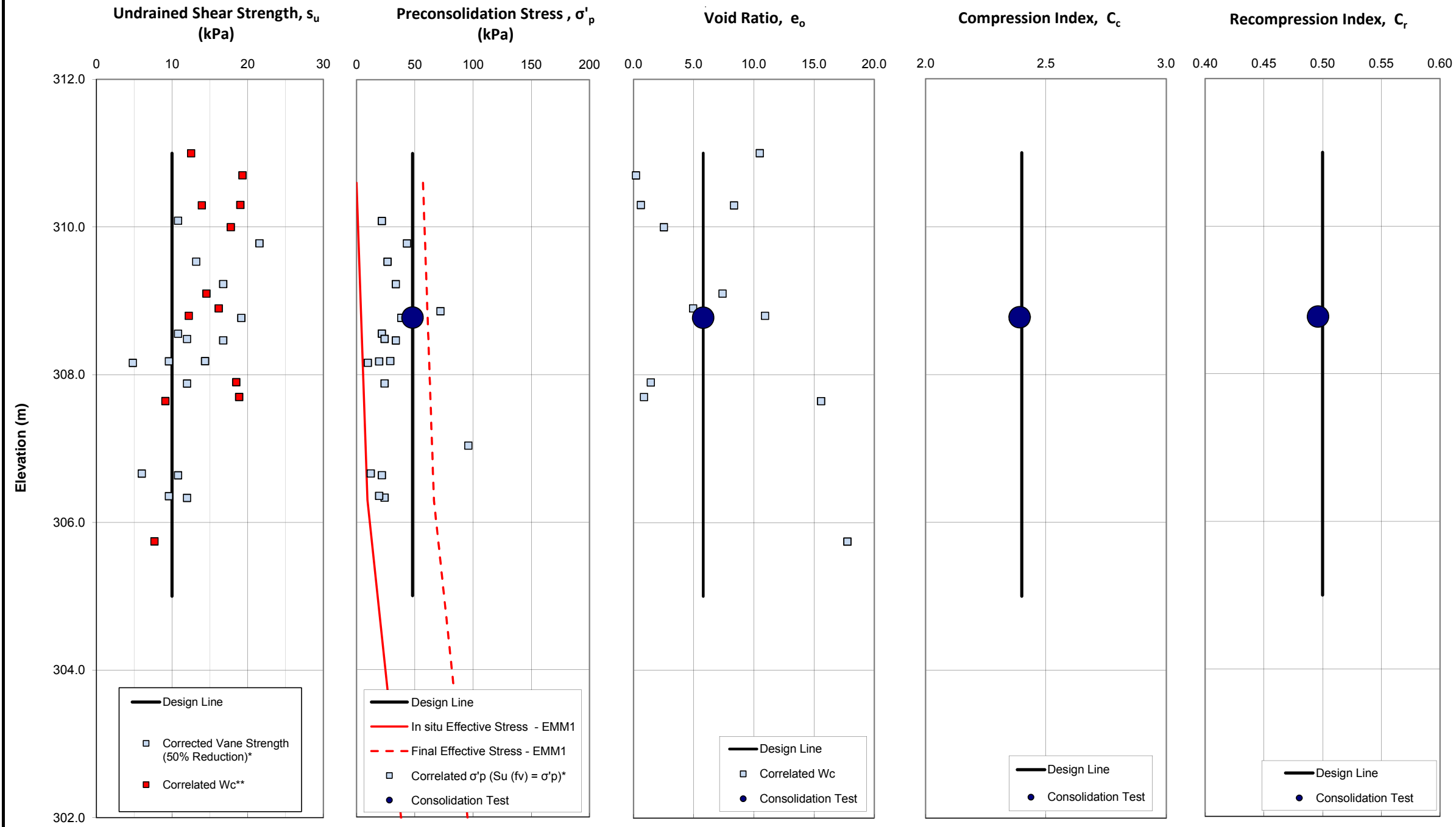
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	EMM4	12	287.5
■	EMM6	10	295.4
▲	EMM8	8	296.9

PROJECT					
HIGHWAY 66 EXCESS MATERIAL MANAGEMENT AREA					
TITLE					
GRAIN SIZE DISTRIBUTION GRAVELLY SILTY SAND to SAND and GRAVEL					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044SUD.GPJ	
DRAWN	TB	Nov 2013	SCALE	N/A	REV.
CHECK	SEMC	Nov 2013			
APPR	JMAC	Nov 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE F11		

N:\Active\2010\1190 Sudbury\1191\10-1191-0044 MRC Hwy 66 VirginatownAnalyses\Excess Material Management\Parameters\10-1191-0044-Parameters and Design Lines EMM.xlsx[EMM Plots - Final (PEAT)]

SUMMARY PLOT OF ENGINEERING PARAMETERS FOR
PEAT
Highway 66 Realignment (Excess Material Management Area)

FIGURE F12



* From Mesri and Ajlouni (2007)

** From Muskeg Engineering Handbook (1969)

Date: November 2013
Project No: 10-1191-0044

Prepared By: MT
Checked By: SEMC

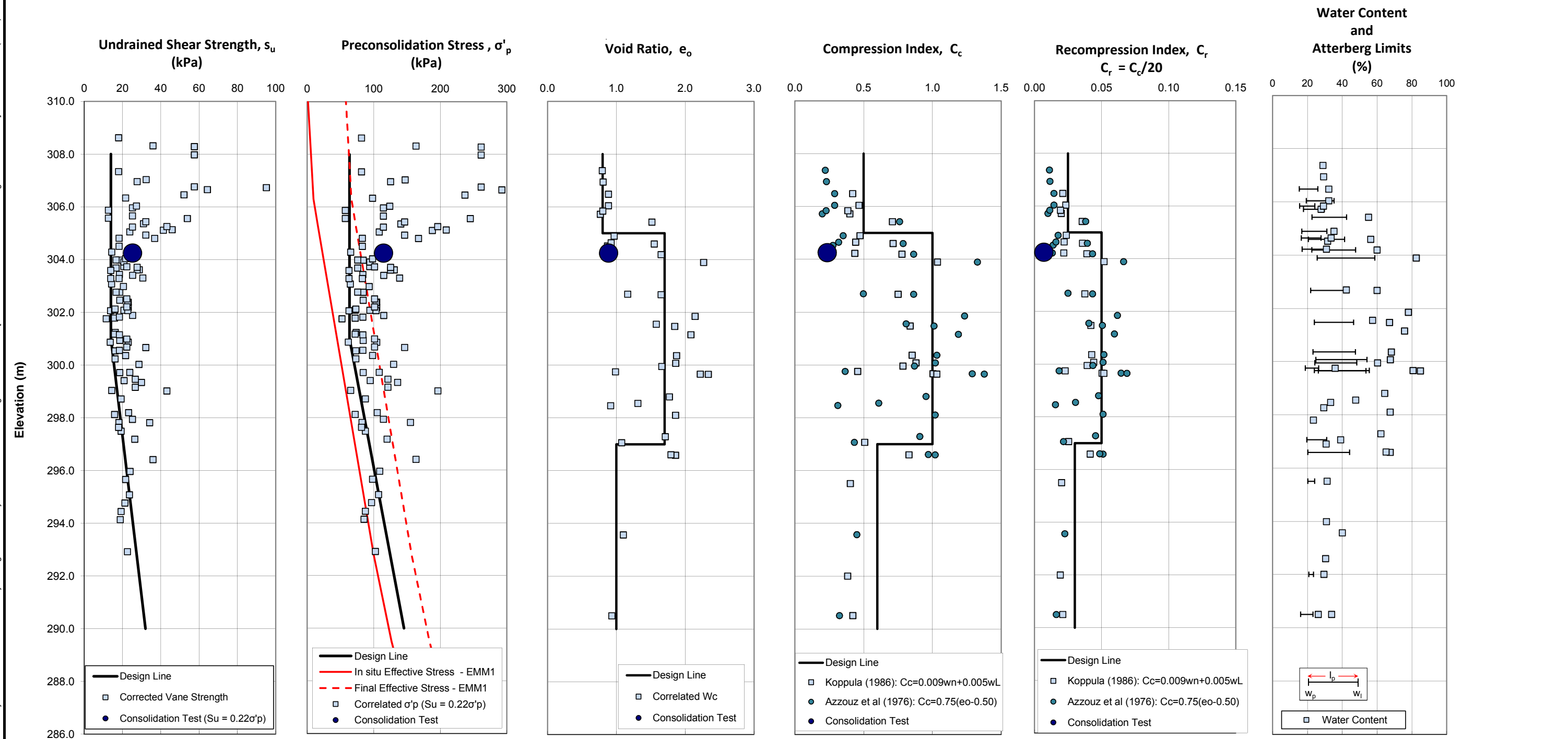
Golder Associates

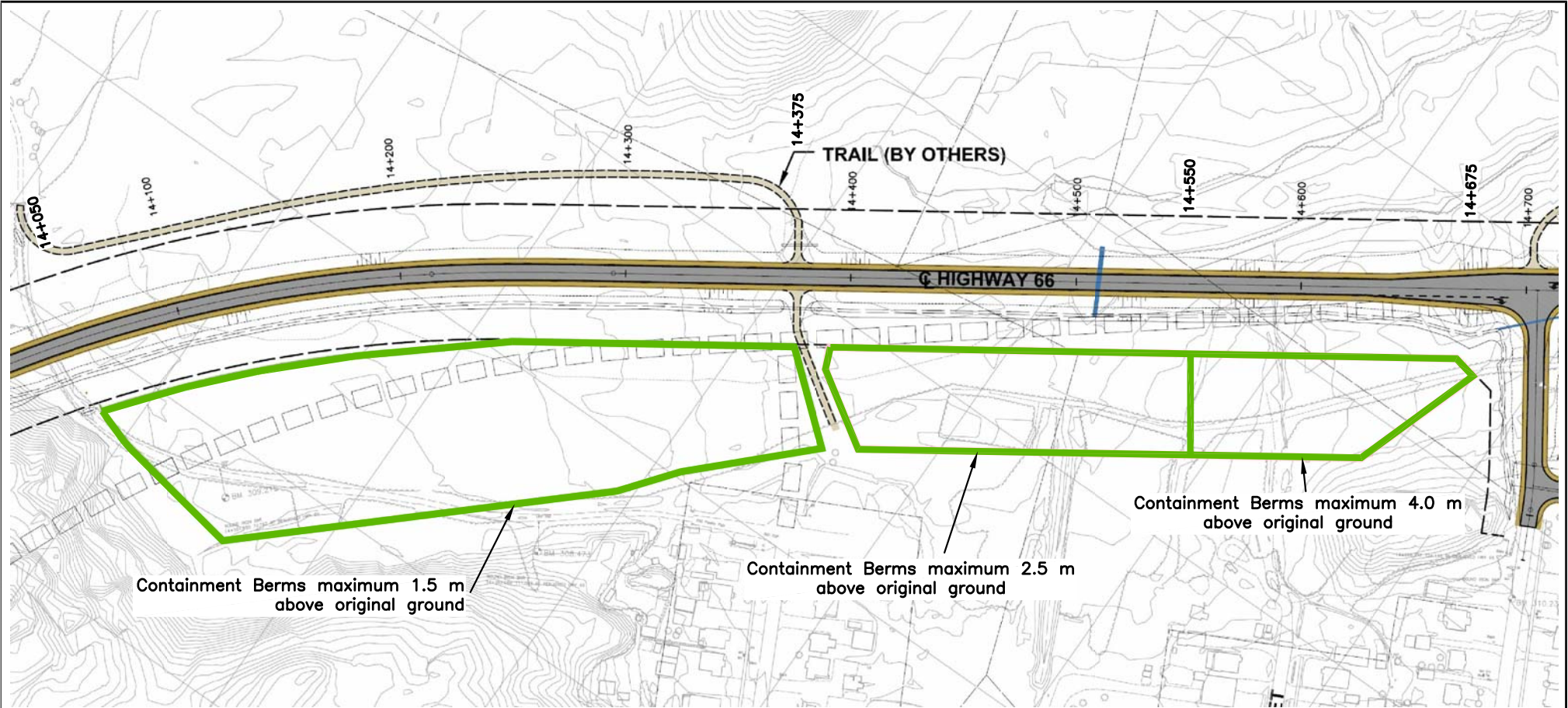


N:\Active\2010\1190 Sudbury\1191\10-1191-0044 MRC Hwy 66 VirginiatownAnalyses\Excess Material Management\Parameters\10-1191-0044-Parameters and Design Lines EMM.xlsx\EMM Plots - Final (CLAY)

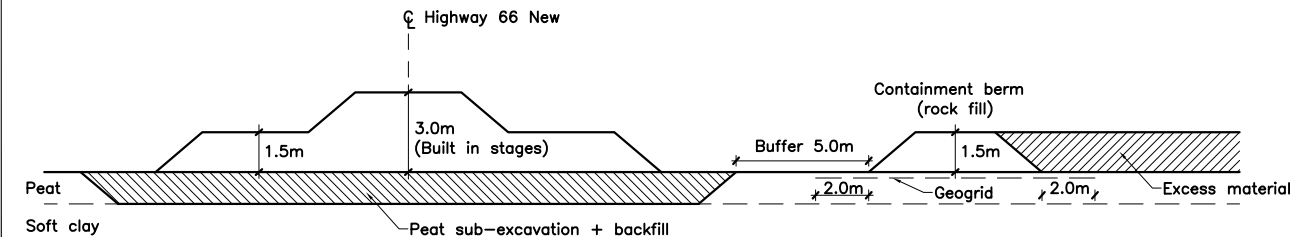
SUMMARY PLOT OF ENGINEERING PARAMETERS FOR
COHESIVE DEPOSITS
Highway 66 Realignment (Excecss Material Management Area)

FIGURE F13





NOTES:
For details of method and of timing placement of containment berms and excess material, see F.I.D.R Section 6.5.6.



TYPICAL CROSS SECTION
STA 14+200

REV	DATE	DES	REVISION DESCRIPTION	CAD	CHK	RVW
PROJECT						
HIGHWAY 66 EXCESS MATERIAL MANAGEMENT AREA STA 14+050 TO 14+675						
TITLE						
CONTAINMENT BERM DETAILS						
PROJECT No.			10-1191-0044	FILE No. Figure F14.dwg		
DESIGN				SCALE		
CAD			TB	NTS		
CHECK			SEMC	REV.		
REVIEW			JMAC	FIGURE No.		
			NOV 2013			
			NOV 2013			
			NOV 2013			





APPENDIX G

Non-Standard Special Provisions and Operational Constraints

SUPPLY AND INSTALLATION OF EMBANKMENT MONITORING EQUIPMENT –
Item No.

Non-Standard Special Provision

1.0 SCOPE

This Special Provision contains the requirements for the supply and installation of Temporary Survey Benchmarks (TBM), Settlement Plates (SP), Nail Pins (NP) and Vibrating Wire Piezometers (VWP) to monitor settlements and porewater pressures in the foundation soils during construction of the Highway 66 embankments as follows:

Highway 66 STA 13+080 to 13+185 (High Fill H4)
Highway 66 Connection STA 10+000 to 10+050 (High Fill H1)
Highway 66 STA 14+040 to 14+560 (Swamp Crossing H6/H7)

The purpose of the SPs is to monitor settlements of the embankment base or the base of a temporary culvert. The settlement readings shall help to establish the timing for placement of fill and the removal of the surcharge. Settlement is measured by survey of the top of the rod with reference to stable, non-settling TBMs.

The purpose of the VWPs installed within the footprint of the embankments / stability berms is to monitor piezometric head at depth within the foundation soil. The piezometer readings will assist in establishing the timing for the placement and the removal of surcharge.

The purpose of the VWPs installed outside the footprint of the embankments / stability berms is to monitor the background hydrostatic piezometric head within the compressible clay deposits not subjected to the embankment loadings.

The purpose of the NPs is to monitor settlement of the temporary culverts. Settlement is measured by survey of the top of the NP with reference to stable, non-settling TBMs.

The rate of fill placement for construction of the embankments and the timing for placement of fill and the removal of the surcharge shall be controlled by the instrumentation readings.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction

OPSS 905 Steel Reinforcement for Concrete

Ontario Provincial Standards Specifications, Material

OPSS1010 Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSS.PROV 1350 Concrete – Materials and Production

OPSS 1205 Clay Seal

OPSS 1301 Cementing Materials

OPSS 1801 Corrugated Steel Pipe (CSP) Products

Ontario Water Resources Act RRO 1990:

Regulation 903 Wells

3.0 DEFINITIONS

Contractor means the Contractor and his Geotechnical Consultant.

Geotechnical Engineering Consultant means a consultant with MTO classification of “Geotechnical (Structures and Embankments) - High Complexity”, to undertake the supply and installation of geotechnical instruments.

Temporary Survey Benchmark means a non-yielding, deep-seated survey reference point.

Monitoring Program means the monitoring readings conducted by others as part of the Contract Administration Assignment.

Settlement Pin/Stake means a bolt or stake embedded in a concrete plug for the purposes of settlement monitoring.

Settlement Plate means a plate installed at the defined level with a series of rods attached to a plate for the purposes of settlement monitoring.

Vibrating Wire Piezometer means a sensor attached to a cable installed in a borehole for the purposes of measuring pore pressure response.

Equal shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Design Requirements

4.01.01 Underground Utilities

The Contractor shall be responsible for locating and protecting all underground utilities and existing wick drains prior to drilling boreholes for installing instruments. Any damage to underground utilities and/or existing wick drains caused by the Contractor’s work shall be repaired by the Contractor at no cost to the Owner or Contract Administrator.

4.01.02 Boreholes

The Contractor shall document subsurface conditions at the locations of instruments and prepare Record of Borehole sheets (borehole logs).

4.01.03 Marking and Labelling

The location of any above-ground monitoring fixtures shall be made clearly visible to nearby traffic before, during and after embankment construction. Markings shall be of sufficient size to be visible from a reversing vehicle and after heavy snow falls.

Instruments and their data cables shall be clearly labelled in the field, each instrument having a unique identifier. The labelling shall remain legible for at least 3 years.

4.01.04 Protection of Instruments

The Contractor shall adequately protect all instruments such that they are not damaged during construction. Any instrument damaged by the Contractor's work shall be immediately replaced at no cost to the Owner or Contract Administrator.

4.02 Submission Requirements

4.02.01 Notification

The Contract Administrator shall be notified a minimum of fifteen (15) working days in advance of commencing the installation of instruments.

4.02.02 Installation Methods

The Contractor shall submit details of the proposed installation methods including locations and types of the data acquisition system, monitoring enclosure, temporary survey benchmarks and installation schedule, to the Contract Administrator, a minimum of fifteen (15) working days before the start of instrument installation.

5.0 MATERIALS

5.01 General

The Contractor shall supply all materials and equipment required for the installation of instrumentation unless noted otherwise.

5.02 Temporary Benchmarks (TBM)

5.02.01 Rod

The Contractor shall supply a steel pipe Schedule 40 with an outside diameter not less than 25.4 mm, supplied in lengths as required to complete the installation as described in Section 2.3.

The top end of each length of TBM rod shall be threaded to receive a cap or to allow for connection of successive lengths of rods. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

5.02.02 Sand

The Contractor shall supply clean, washed sand. The sand shall be Sakcrete washed general-purpose sand – or equal.

5.02.03 Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design consists of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU – OPSS 1301).

5.02.04 Rod Anchor Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design consists of 14 kg of bentonite (OPSS 1205), 49 litres of water and 40 kg of cement (Type GU – OPSS 1301).

5.02.05 Friction Reducing Sleeve

The Contractor shall supply a friction reducing sleeve for the full length of rod consisting of Schedule 40 – 50.8 mm (2") O.D. PVC pipe cut perpendicular to the axis of the pipe.

5.03 Settlement Plates (SP)

5.03.01 Plate

The Contractor shall supply a steel plate with a thickness of at least 6.35 mm. The plate shall be at least 0.5 m wide by 0.5 m long.

5.03.02 Rod

The Contractor shall supply a steel pipe Schedule 40 with an outside diameter not less than 25.4 mm, supplied in lengths as required to complete the installation as described in Section 7.03.04.

The top end of the full length of rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

5.03.03 Friction Reducing Sleeve

The Contractor shall supply a friction reducing sleeve consisting of Schedule 40 – 50.8 mm O.D. PVC pipe cut perpendicular to the axis of the pipe.

5.03.04 Protective Surround

The Contractor shall supply a protective surround for the portion of the rod within the embankment.

The surround shall consist of 300 mm diameter corrugated steel pipe (CSP – OPSS 1801) with the ends cut perpendicular to the axis of the pipe and free of burrs and sharp edges. The space between the CSP and the Friction Reduction Sleeve (PVC pipe) shall be filled with medium to coarse sand.

5.04 Nail Pins (NP)

5.04.01 Pin

The Contractor shall supply a 25.4 mm minimum diameter reinforcing steel bar (OPSS 905) cut 0.15 m long or equivalent.

The top of the reinforcing steel bar shall be angled or rounded in such a way that a single survey point can be clearly identified and repeated

5.04.02 Concrete

The Contractor shall supply concrete (OPSS Prov. 1350) of minimum 25 MPa compressive strength and set time sufficient to secure the Nail Pin within two (2) days of pouring.

5.05 Vibrating Wire Piezometers (VWP)

5.05.01 Vibrating Wire Piezometer Sensors

The vibrating wire piezometer sensors shall be:

- Slope Indicator model 52611020 (-5 to 50 psi); or
- RST model VW2100-0.35; or
- Equal.

The VWPs shall be compatible with the Slope Indicator VW Minilogger, model 52613310, or equal. All VWPs shall be of the same make/supplier.

All VWPs shall be calibrated prior to installation and the calibration data for each piezometer shall be provided to the Contract Administrator.

5.05.02 Signal Cable

The signal cable shall be:

- Slope Indicator model 50613524 cable; or
- RST model EL380004 cable; or
- Equal.

The length of cable for each piezometer shall be carefully estimated from the construction drawings to ensure that there is sufficient length of signal cable for each piezometer to provide enough slack in the borehole and along the trenches until each cable is out of the embankment footprint area where they shall be protected from earthmoving equipment and extended to the monitoring station.

5.05.03 Bentonite

Bentonite to form borehole plugs as required shall be in accordance with OPSS 1205 in pellet form in sufficient quantity.

5.05.04 Filter Sand

Sand for filters around VWP sensors shall be clean washed sand, such as “Sakcrete” washed general-purpose sand; or similar.

5.05.05 Grout

Grout shall be cement-bentonite mix consisting of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU - OPSS 1301).

5.05.06 Trench Burial and Conduit

The signal cable for each piezometer shall be buried in a shallow trench as shown in the Contract Drawings, and taken out of the embankment footprint area if possible and/or to an area that will not be impacted by construction operations. Conduits to protect the signal cables in the trenches and above ground surface shall consist of Schedule 40 – 75 mm - 3" - steel pipe or Schedule 80 – 75 mm - 3" - rigid PVC pipe. If appropriate, several signal cables may be housed in a single conduit and laid in a common trench.

5.05.07 Data Acquisition System (Data Logger)

The signal cables from the vibrating wire piezometers shall be connected to the nearest data-logger. A minimum of two (2) data-loggers shall be installed in the High Fill H1/H4 Areas and a minimum of three (3) data-loggers shall be installed in the Swamp Crossing H6/H7. The data acquisition systems shall be from the same supplier as the VWPs and shall consists of:

- Slope Indicator Model 56701000 (CR1000); or
- RST Model ELGL1200; or
- Equal.

The data-logger shall consist of the following:

- ENC 16/18 Water-proof Enclosure Model 56705020, Model ELF0638, or equal;
- SC32A Serial Interface (with RS232 transfer cable) Model 56704010, Model CS-SC32A, or equal;
- VW Interface Model 56701510 or 56701500, Model CS-AVW200, or equal;
- AM16/32 Multiplexer Model 56702110, Model ELGL2042, or equal;
- A suitable power supply which shall be able to last for 5 years for long term settlement monitoring (i.e. large capacity rechargeable battery coupled with solar panel);
- LoggerNet Software Model 56708020, Model CS-Loggernet, or equal.

The data-loggers shall be programmed according to the following:

- Recording Software: VWP data shall be recorded four (4) times a day (i.e. one (1) reading every 6 hours); and,
- Test Software: once this program is transferred to the data-logger, the system

shall be able to be tested and record data manually on site.

The real-time data shall be retrieved on site by direct wire (i.e. RS232 Cable) with a portable laptop computer as specified in Section 5.05.08.

5.05.08 Portable Laptop Computer

For the purposes of monitoring the VWP's the Contractor shall supply:

- A new Portable Laptop Computer (with a three (3) year warranty): Intel Core i5 or equivalent (2.4 GHz or higher) with Windows 7 (English), 4 GB memory, a minimum of 250 GB hard drive storage, a DVD+/-RW and Microsoft Office 2010, to retrieve, read and store the VWP readings.
- An extra battery for the above portable laptop computer and a cigarette lighter adaptor for the computer charger.

5.05.09 Wooden Posts

Wooden posts for the support of the data acquisition system enclosures shall be:

- 100 mm x 100 mm (4"x4"), minimum 3 m (10') long pressured treated lumber.

6.0 EQUIPMENT

6.01 Equipment Operations and Weather Conditions

All installation and monitoring equipment and associated materials shall be capable of withstanding the range of temperatures possible for their location within the ground or on the surface. The instruments shall be capable of operating within the manufacturer's stated accuracy throughout the temperature range. Monitoring shall be conducted year round.

6.02 Data Logger

The Contractor shall submit a detailed proposal on the setup of the data-logging system (i.e. numbers and locations of the data-logging unit(s)) to the Contract Administrator for review, prior to ordering the data-logger(s).

6.03 Laptop Computer

The portable laptop computer will become property of the MTO and shall be handed to the Contract Administrator after the installation of the instruments for the Monitoring Program.

The calibration factors for the VWP's shall be entered into the portable laptop computer by the Contractor for initialization of the instruments.

7.0 CONSTRUCTION

7.01 Subsurface Conditions

The subsurface conditions at the site are described in Foundation Investigation Report as specified elsewhere in the Contract Documents.

7.02 Drawings

Reference shall be made to the following drawings that are contained elsewhere in the Contract:

- Monitoring Section Location Plans;
- Embankment Monitoring Program Plans and Section Types A, B, C, D, E and F; and,
- Typical Instrument Installation Details.

7.03 Instrumentation Installation

7.03.01 Instrument Locations

The quantity and location of instruments are as shown in the Contract Documents and in Table 1A below.

Table 1A – Instrument Quantities and Locations

Monitoring Section			Quantities			
<i>High Fill/Swamp Crossing</i>	<i>Hwy 66 Station</i>	<i>Type</i>	<i>TBM</i>	<i>SP</i>	<i>VWP</i>	<i>NP</i>
H1/H4 High Fill Section	13+080	A	0	4	2	2
	13+120	n/a	1	0	0	0
	13+150	B	0	2	3	0
	10+020 ¹	n/a	1	0	0	0
	10+050 ¹	C	0	2	3	0
H6/H7 Swamp Crossing	14+030	n/a	1	0	0	0
	14+060	D	0	3	5	0
	14+090	D	0	3	4	0
	14+180	D	0	3	5	0
	14+270	E	0	3	2	0
	14+360	E	0	3	2	0
	14+450	D	0	3	4	0
	14+510	F	0	3	0	2
	14+530	D	0	3	5	0
	14+530	n/a	1	0	0	0
TOTAL			4	32	35	4

NOTES: 1. Station provided along Highway 66 Connection.

Prior to the installation of instruments, the Contractor shall accurately survey and stake the location of each instrument and obtain a ground surface elevation at each instrument location.

The locations of the monitoring instruments should be adjusted in the field such that they will not be damaged by the sub-excavation procedures for the new embankment, by highway maintenance equipment on the existing highway, or by earth moving equipment.

7.03.02 Installation Program

Instrument installation shall commence immediately after wick drain installation or immediately after construction of any temporary culvert and before any embankment construction. Table 1B presents a summary of the installation schedule requirements.

Table 1B – Installation Program

High Fill/Swamp Crossing	Type	Start of Installation	Completion of Installation
H1/H4 High Fill Section	SP	Before embankment construction	At completion of embankment construction
	VWP	Before embankment construction	Before embankment construction
	NP	After construction of temporary culvert	Before embankment construction
H6/H7 Swamp Crossing	SP	After wick drain installation and after construction of temporary culvert	At completion of embankment construction
	VWP	After wick drain installation	Before embankment construction
	NP	After construction of temporary culvert	Before embankment construction

7.03.03 Temporary Survey Benchmarks

7.03.03.01 General

The locations of the TBMs are as shown in the Contract Drawings and in Table 2A. The TBMs shall be installed prior to embankment construction. The TBMs shall consist of a steel rod anchored to the bottom of a borehole.

The number and locations of TBMs shall be such that direct sighting is possible from all geotechnical instruments to at least one (1) TBM. The Contractor shall establish the geodetic elevation of each such TBM.

Table 2A – Approximate Temporary Benchmark Locations¹

Swamp Crossing	Hwy 66 Station	Offset ²	Approximate Elevation of the Bottom of Rod Anchor ³ (m)	Estimated Final Length of Steel Rod including 1 m Stickup ³ (m)
H1/H4 High Fill Section	13+120	25.0 m Lt	301.4	5.4
	10+020 ⁴	25.0 m Lt	301.4	4.2
H6/H7 Swamp Crossing	14+030	25.0 m Rt	307.2	3.1
	14+545	22.0 m Lt	291.5	14.5

NOTES: 1. Location to be agreed upon by Contractor and Contract Administrator prior to installation.
2. Offset from centreline.
3. The rod anchor elevations shown are approximate and should be adjusted in the field so that the rod anchor is installed a minimum of 2 m into the bedrock.
4. Station from Highway 66 Connection Alignment.

7.03.06.02 Borehole Installation

The borehole shall be advanced to the rod anchor elevations provided in Table 2A using suitable drilling techniques. The diameter of the borehole shall be sufficient to fit the rod, friction reducing sleeve and rod anchor grout. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

7.03.06.03 Rod

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

7.03.06.04 Rod Anchor

The rod shall be installed vertically in the borehole with its bottom end resting at the bottom of the borehole.

The elevation of the bottom of the rod anchor shall be determined by measuring the length of the rod to the ground surface elevation. The bottom portion of the rod shall be fixed against the surrounding native soil by grouting the bottom 0.5 m of the borehole using the rod anchor grout mix to form a concrete/soil anchor.

Once grouting is completed and the rod anchor grout has set, the contractor shall pour clean sand in the lower 0.5 m length of the borehole above the concrete/soil anchor to create a base for the end of the friction reducing sleeve to rest on.

7.03.06.05 Friction Reducing Sleeve

The friction reducing sleeve shall be over the entire length of the rod above the rod anchor and sand.

7.03.06.06 Installation Details

The elevation, easting and northing of the top of the Benchmark rod shall be surveyed.

7.03.04 Settlement Plates**7.03.04.01 General**

The locations of the SPs are shown on the Contract Drawings and are given in Table 2B. As embankment construction proceeds the rods shall be extended above the new top of embankment. Sleeves around the rods shall be installed to reduce friction and allow uninhibited movement of the rod with the plate.

In non-wick drain areas, the SPs shall be placed on undisturbed native soil just below the existing ground surface (i.e. where sub-excavation of organic deposits is not required) or on top of the backfill after sub-excavation and replacement of organic deposits, where required, or at the top of a temporary culvert. In wick drain areas, the Settlement Plates shall be placed on top of the drainage blanket or at the top of a temporary culvert. As embankment construction proceeds the settlement measuring rods shall be extended above the new top of embankment.

Table 2B – Settlement Plate Locations

Swamp Crossing	Monitoring Section Type	Hwy 66 Station	Offset from Centreline	Approximate Elevation of Existing Ground Surface (m)	Estimated Thickness of Embankment Fill Above Ground Surface¹ (m)
H1/H4 High Fill Section	A	13+080	15.0 m Lt	305.5	3.5
	A	13+080	3.0 m Lt	305.1	3.5
	A	13+080	3.0 m Rt	305.0	3.5
	A	13+080	14.0 m Rt	304.9	3.5
	B	13+150	3.0 m Lt	305.2	3.5
	B	13+150	4.0 m Rt	304.8	3.5
	C	10+050 ²	3.0 m Lt	303.1	6.0
	C	10+050 ²	3.0 m Rt	303.2	6.0
H6/H7 Swamp Crossing	D	14+060	5.0 m Lt	309.2	3.5
	D	14+060	0 m	309.2	3.5
	D	14+060	5.0 m Rt	309.4	3.5
	D	14+090	5.0 m Lt	309.2	3.0
	D	14+090	0 m	309.2	3.0
	D	14+090	5.0 m Rt	309.1	3.0
	D	14+180	5.0 m Lt	308.8	2.5
	D	14+180	0 m	308.8	2.5
	D	14+180	5.0 m Rt	308.8	2.5
	E	14+270	5.0 m Lt	307.7	2.0
	E	14+270	0 m	307.8	2.0
	E	14+270	5.0 m Rt	307.9	2.0

Swamp Crossing	Monitoring Section Type	Hwy 66 Station	Offset from Centreline	Approximate Elevation of Existing Ground Surface (m)	Estimated Thickness of Embankment Fill Above Ground Surface ¹ (m)
	E	14+360	5.0 m Lt	306.5	2.5
	E	14+360	0 m	306.5	2.5
	E	14+360	5.0 m Rt	306.5	2.5
	D	14+450	5.0 m Lt	305.2	3.0
	D	14+450	0 m	305.2	3.0
	D	14+450	5.0 m Rt	305.2	3.0
	F	14+510	5.0 m Lt	304.6	2.5
	F	14+510	0 m	305.0	2.5
	F	14+510	5.0 m Rt	305.0	2.5
	D	14+530	5.0 m Lt	304.7	3.0
	D	14+530	0 m	305.0	3.0
	D	14+530	5.0 m Rt	305.3	3.0

NOTES: 1. Embankment fill thickness excludes surcharge.
2. Station from Highway 66 Connection Alignment.

The elevation, easting and northing of the centre of the base of the plate and top of the rod shall be surveyed after installation.

The total distance from the base of the plate to the top of the rod shall be measured to an accuracy of ± 2 mm or better.

7.03.04.02 Plate

For Monitoring Sections Types A and F specified in the Contract, the SPs shall be installed on top of the temporary culvert. For Monitoring Sections Types B and C specified in the Contract, the settlement plate shall be installed horizontally on the undisturbed native soil just below the excavation. For Monitoring Sections Types D and E specified in the Contract, the settlement plate shall be installed horizontally on top of the drainage blanket.

7.03.04.03 Rod

The SP rod shall be fixed to the centre of the plate and perpendicular to the plate. The coupling of the rods shall be such that all sections have the same axis and that no separation or contraction will occur at the couplings.

7.03.04.04 Friction Reducing Sleeve

The friction reducing sleeve shall be over the entire length of the rod that is below ground and within the embankment fill except that the cap on top of the SP rod shall extend 25 mm above the top of the friction sleeve at all times.

7.03.04.05 Extension of Rod

The SP rods shall be extended upwards as the embankment is constructed so that the top of the rod is always at least 0.3 m but not more than 2 m above the surrounding fill.

7.03.04.06 Protective Surround

The CSP, Friction Reducing Sleeve and sand protective surround shall be extended concurrent with the rods. The SP rod shall be in the centre of the CSP and friction-reducing sleeve. The annulus between the CSP and the friction-reducing sleeve shall be filled with sand to a level not higher than the top of the sleeve.

7.03.05 Nail Pins

7.03.05.01 General

The locations of the NPs are shown on the Contract Drawings and are given in Table 2C.

For Monitoring Section Type A and F specified in the Contract, two (2) NPs shall be installed on the top of the temporary culvert, one (1) at each end of the culvert. The specified location shall be 0.5 m from each end and on the centreline of the culvert.

Table 2C – Nail Pin Locations

Swamp Crossing	Monitoring Section Type	Culvert Designation	Culvert Station ¹	Nail Pin Location
H1/H4 High Fill Section	A	BC1	13+080	0.5 m from the north end of the temporary culvert
	A	BC1	13+080	0.5 m from the south end of the temporary culvert
H6/H7 Swamp Crossing	F	BC4	14+510	0.5 m from the north end of the temporary culvert
	F	BC4	14+510	0.5 m from the south end of the temporary culvert

NOTE: 1. Stations for the nail pins are approximate.

7.03.06 Vibrating Wire Piezometers

7.03.06.01 General

The locations of the Vibrating Wire Piezometers are as shown in the Contract Documents and in Table 2D. Installation of the Vibrating Wire Piezometers shall be as per the manufacturer's recommendations in addition to what is stated or emphasised below.

The piezometers shall be installed in boreholes after wick drain installation but prior to any embankment construction.

The VWP signal cables shall be extended to the data-logger enclosure areas through a metal or plastic conduit buried in trenches, as shown on the Contract Drawings.

Table 2D – Vibrating Wire Piezometer Locations and Elevations

Swamp Crossing	Monitoring Section Type	Hwy 66 Station	Offset from Centreline ¹	Approximate Elevation of Existing Ground Surface (m)	Tip Elevation (m)
H1/H4 High Fill Section	A	13+080	0 m	305.0	301.4
	A	13+080	14.0 m Rt	304.9	300.6
	B	13+150	0 m	305.0	303.7
	B	13+150	19.5 m Rt	304.0	300.0
	B	13+150	34.0 m Rt	304.1	300.0
	C	10+050 ¹	15.5 m Lt	303.6	300.9
	C	13+050 ¹	0 m	303.2	303.3
	C	13+050 ¹	15.5 m Rt	303.0	299.7
H6/H7 Swamp Crossing	D	14+059	0 m	309.2	304.7
	D	14+060	14.5 m Lt	309.2	305.0
	D	14+060	34.0 m Lt	309.2	301.7
	D	14+060	16.0 m Rt	309.2	304.0
	D	14+061	0 m	309.2	299.7
	D	14+089	0 m	309.2	303.9
	D	14+090	14.5 m Lt	309.1	303.9
	D	14+090	14.0 m Rt	309.0	304.0
	D	14+091	0 m	309.2	299.1
	D	14+179	0 m	308.8	303.6
	D	14+180	14.0 m Lt	308.8	303.9
	D	14+180	26.0 m Lt	308.8	297.5
	D	14+180	13.5 m Rt	308.8	303.4
	D	14+181	0 m	308.8	298.1
	E	14+269	0 m	307.8	304.2
	E	14+271	0 m	307.8	299.0
	E	14+359	0 m	306.5	303.9
	E	14+361	0 m	306.5	300.4
	D	14+449	0 m	305.2	303.9
	D	14+450	13.0 m Lt	304.9	303.6
	D	14+450	16.0 m Rt	305.5	303.6
	D	14+451	0 m	305.2	298.8
	D	14+529	0 m	305.0	303.4
	D	14+530	13.5 m Lt	304.4	302.3
	D	14+530	26.0 m Lt	304.4	299.4
	D	14+530	13.5 m Rt	305.3	303.8
	D	14+531	0 m	305.0	299.9

NOTE: 1. Station from Highway 66 Connection Alignment.

7.03.06.02 Borehole Installation

The borehole at each VWP location shall be advanced to 300 mm below the lowest tip elevation using suitable drilling techniques. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

The borehole location shall be at the centre of the triangular wick drain grid (where applicable) and shall be determined in the field, after the wick drain installation.

7.03.06.03 Protection for Long Term Monitoring (Monitoring Shed)

The data-logger shall be installed in a walk-in Monitoring Shed to prevent vandalism and prolonged wear-out of the data-loggers against extreme weather. The Monitoring Shed shall be lockable and weather proofed surrounded by 2 m high chain link fence and a lockable gate. The Monitoring Shed shall also be seated on a gravel pad and securely tied down to the ground. The location of the Monitoring Shed shall not be susceptible to ground settlement. The Contractor shall submit a detailed proposal of the Monitoring Shed (i.e. materials and location(s) etc.) to the Contract Administrator for review, prior to construction.

The Contractor shall ensure access to the Monitoring Shed at all times, including but not limited to snow clearing in the winter.

7.03.06.04 Completion of Installation

It is known that the process of installing VWPs can temporarily alter the pore water pressure acting on the piezometer tip. The installation of a VWP shall not be considered to be complete until the pore pressure acting on the piezometer has returned to and stabilized at the value prevailing in the surrounding, unaffected soil mass. The Contractor shall take daily reading of the pore pressures until the value has stabilized. Stabilization shall be deemed to have occurred:

- When no change in the measured value has occurred over a period of five (5) consecutive days and the measured value is within 10 per cent of the anticipated hydrostatic value; and,
- When the daily rate of change is less than four (4) kPa per day for three (3) consecutive days and the measured value is within 5 per cent of the anticipated hydrostatic value.

The Contractor should be prepared to wait for a period of 10 days to 15 days after completion of installation of the instruments for the baseline readings to stabilize.

7.04 Coordination with Monitoring Program

7.04.01 Notification

The Contractor shall notify the Contract Administrator no later than three (3) days the completion of installation of TBMs, SPs, NPs and VWPs.

7.04.02 Reporting

The Contractor shall supply the information outlined in the following sections to the Contract Administrator within three (3) days of completion of installation of each instrument.

7.04.02.01 Temporary Survey Benchmarks

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- TBM Northing and Easting in MTM NAD 83 coordinates;
- Elevation of the rod anchor bottom rod anchor length and top of rod in Geodetic datum;
- Date of installation;
- Stratigraphic log of subsurface conditions at the TBMs, including notes on drilling method obstructions it encountered;
- Installation notes/sketches; and,
- Description of TBM (rod), sleeves and rod anchors.

7.04.02.02 Settlement Plates

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- SP Northing and Easting in MTM NAD 83 coordinates;
- Elevation of base of plate and top of rod in Geodetic datum;
- Date of installation;
- Installation notes/sketches; and,
- Description of SP rods, sleeves and plates.

Adjustments in the length of any SP rod shall be coordinated with the Contract Administrator to allow surveying by others of the elevation of the top of the rod immediately before and immediately after adjustment. This surveying is necessary to accurately track the settlement data.

7.04.02.03 Nail Pins

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- NP Northings and Eastings in MTM NAD 83 coordinates;
- Elevation of pin in Geodetic datum;
- Date(s) of installation; and,
- Installation notes / sketches.

7.04.02.04 Vibrating Wire Piezometers

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- VWP Northings and Eastings in MTM NAD 83 coordinates;
- Elevations of VW sensors in Geodetic datum;
- Dates of installation;
- Stratigraphic log of subsurface conditions, including drilling method notes;

- Installation notes / sketches;
- Model, make and serial numbers of VW sensors, readout unit and signal cable; and,
- Calibration details of VW sensors.

7.04.03 Monitoring

7.04.03.01 Temporary Survey Benchmarks

Monitoring of settlements with reference to the TBMs shall be done by others. Monitoring shall be conducted during the embankment construction. The Contractor shall provide installation information as specified above and provide access to the TBMs for monitoring including, but not limited to, snow clearing in the winter. The Contractor shall provide electric power and general area lighting as needed.

7.04.03.02 Settlement Plates

Monitoring of the SPs shall be done by others. Monitoring shall be conducted during the embankment and surcharge construction. The Contractor shall provide installation information as specified above and provide access to the SPs for monitoring including, but not limited to a scaffolding platform and ladder if required and snow clearing in the winter. The Contractor shall provide electric power and general area lighting as needed for reading the instruments.

7.04.03.03 Nail Pins

Monitoring of the NPs shall be done by others. Monitoring shall be conducted during the embankment fill construction and surcharge period. The Contractor shall provide installation information as specified above and provide access to the NPs for monitoring.

7.04.03.04 Vibrating Wire Piezometers

Monitoring of the VWP's shall be done by others. Monitoring shall be conducted during and after the embankment and surcharge construction. The Contractor shall provide installation information as specified above and provide access to the data-loggers for monitoring.

The Contractor shall transfer the Portable Laptop Computer to the Contract Administrator, including all the data-logging software and hardware, operation instructions and calibration constants. The contractor shall also transfer the keys for the locks of the Monitoring Shed(s). The Contractor shall be available for one site meeting with the Contract Administrator to transfer and explain about any questions from the Contract Administrator regarding the data-logging system.

7.05 Decommissioning of Instruments

7.05.01 General

The Contractor shall decommission all the TBMs, SPs and NPs at the end of the monitoring program unless advised otherwise by the Contract Administrator. The VWP's shall be kept and protected for long-term monitoring and shall not be decommissioned. Decommissioning of instrumentation shall be carried out according to the Ontario Water Resources Act, R.R.O. 1990, Regulation 903.

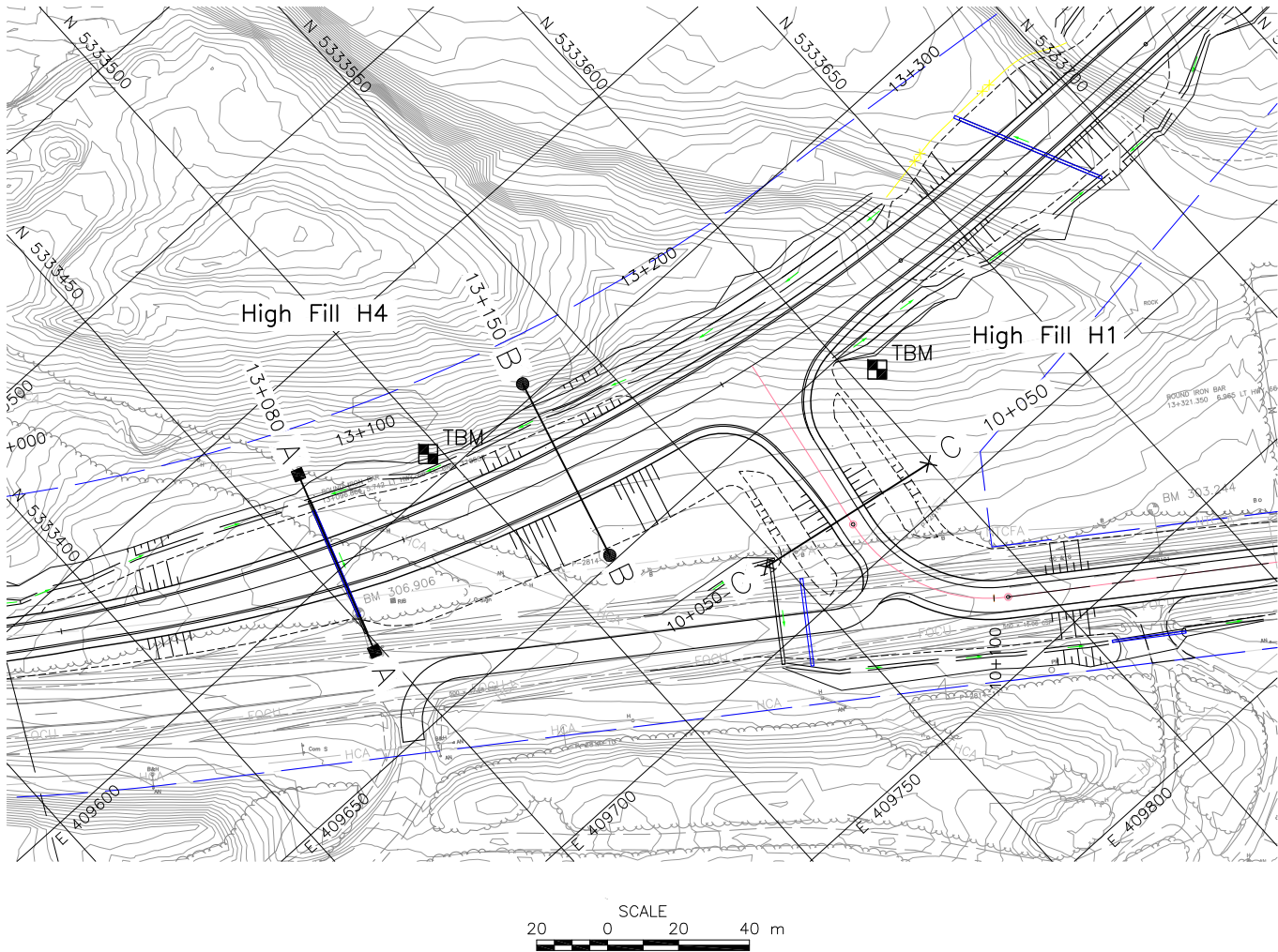
8.0 PAYMENT

8.0.1 Measurement for Payment

Measurement for Payment will be made on the basis of the number of units of survey TBMs, SPs, VWPs and NPs installed.

8.02 Basis of Payment

Payment at the Lump Sum price for this tender item shall be full compensation for all labour, monitoring equipment and material to do the work.



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

CONT No.
GWP No. 5091-07-00

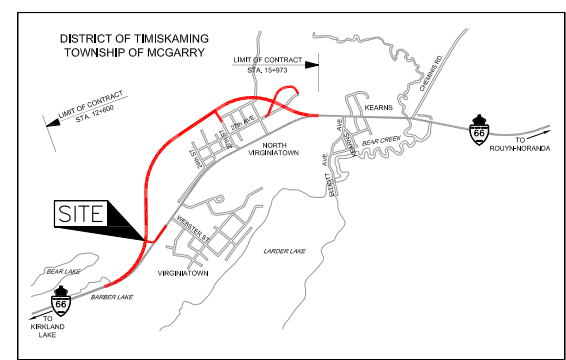


HIGHWAY 66
HWY 66 - STA 13+080 TO 13+185
HWY 66 CONNECTION - STA 10+000 TO 10+050
MONITORING SECTION LOCATION PLAN

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



KEY PLAN
SCALE
700 0 700 m

LEGEND

- Monitoring Section and Type
- Monitoring Section and Type
- Monitoring Section and Type
- TBM
Proposed Temporary Benchmark Installation

NOTES

The location of the Temporary Benchmarks (TBM) are shown for illustration purposes only.
The final location of the Temporary Benchmarks shall be agreed upon by the Contractor and Contract Administrator prior to installation.

REFERENCE

Base plans provided in digital format by MMM, drawing file nos. H3211009D16 ROLL PLAN-ULTIMATE and PDR.dwg, received DEC 3, 2012.
Keyplan drawing file nos. H3211009G02 received JAN 24, 2013.



NO.	DATE	BY	REVISION
Geocres No. 32D-17			
HWY. 66	PROJECT NO. 10-1191-0044		DIST.
SUBM'D. MT	CHKD. TZ	DATE: NOV 2013	SITE:
DRAWN: TB	CHKD. SEMC	APPD. JMAC	DWG. G1

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

CONT No.
GWP No. 5091-07-00

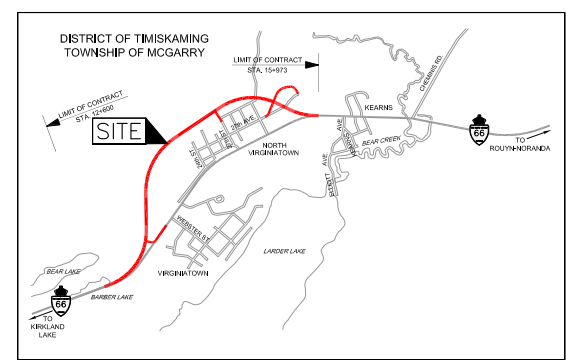


HIGHWAY 66
HWY 66 - STA 14+020 TO 14+650
MONITORING SECTION
LOCATION PLAN

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



LEGEND

- Monitoring Section and Type
- Monitoring Section and Type
- Monitoring Section and Type
- TBM
- Proposed Temporary Benchmark Installation

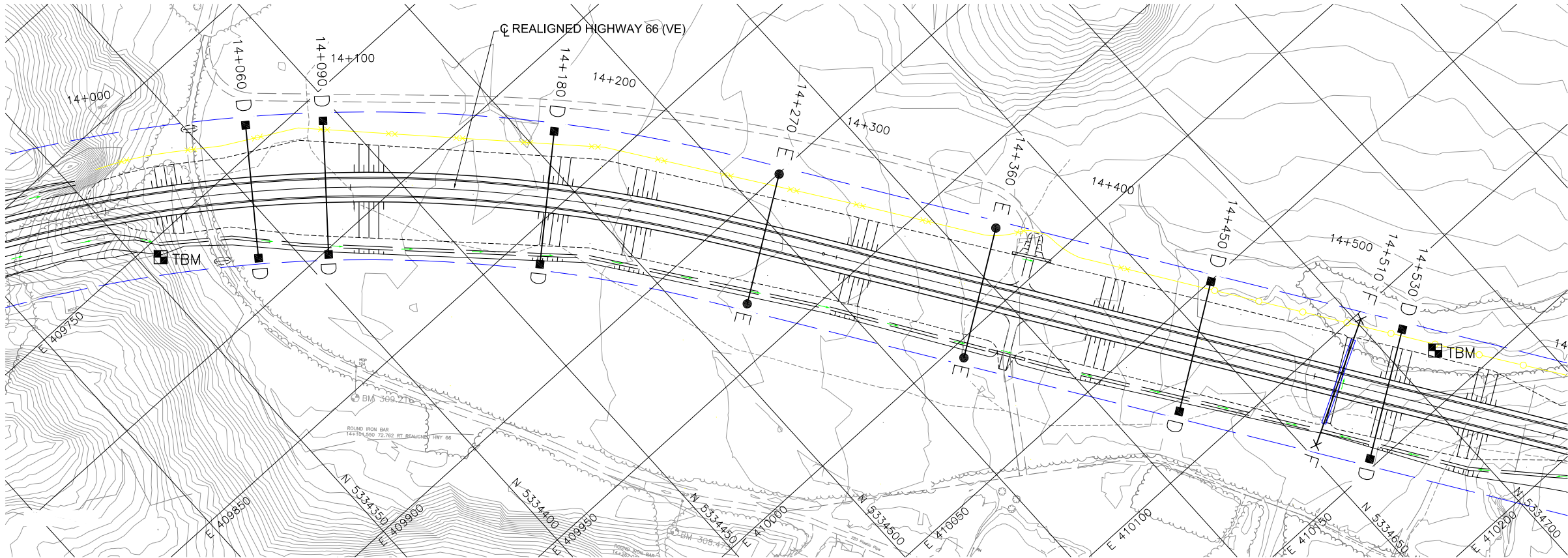
NOTES

The location of the Temporary Benchmarks (TBM) are shown for illustration purposes only.
The final location of the Temporary Benchmarks shall be agreed upon by the Contractor and Contract Administrator prior to installation.

REFERENCE

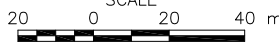
Base plans provided in digital format by MMM, drawing file nos. H3211009D16 ROLL PLAN-ULTIMATE and PDR.dwg, received DEC 3, 2012.
Keyplan drawing file nos. H3211009G02 received JAN 24, 2013.

Swamp H6/H7



PLAN

SCALE



NO.	DATE	BY	REVISION
Geocres No. 32D-17			
HWY. 66	PROJECT NO. 10-1191-0044		DIST.
SUBM'D. MT	CHKD. TZ	DATE: NOV 2013	SITE:
DRAWN: TB	CHKD. SEMC	APPD. JMAC	DWG. G2

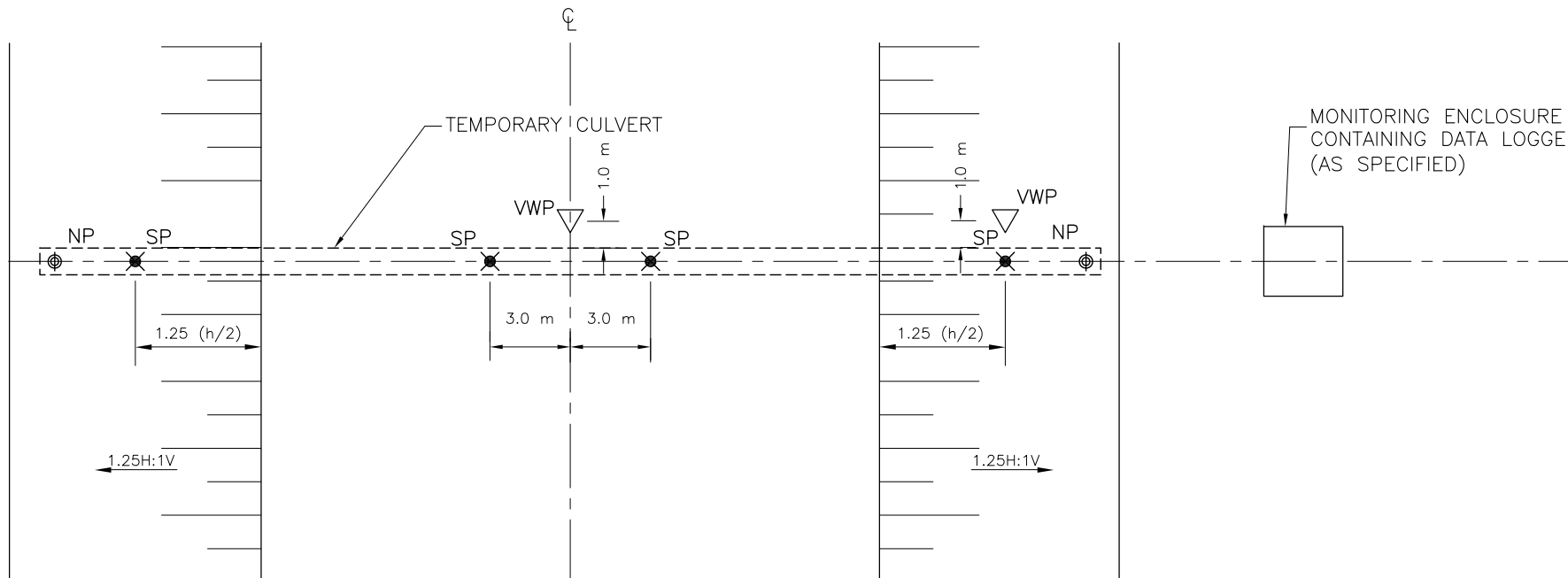







Diagram illustrating the cross-section of a temporary culvert structure. The culvert is shown as a horizontal pipe with two vertical access points labeled "SP" (Service Port) and one labeled "NP" (Nipple Port). The structure is supported by "ROCK FILL" on the left and a slope with a 1.25:1 ratio on the right. A "MONITORING ENCLOSURE CONTAINING DATA LOGGER (AS SPECIFIED)" is attached to the right side. Below the culvert, two "VWP" (Vertical Water Port) are shown, one on the left and one on the right. The ground is labeled "Clayey Silt to Clay" and the bottom layer is "Sandy Gravel to Sand and Gravel".

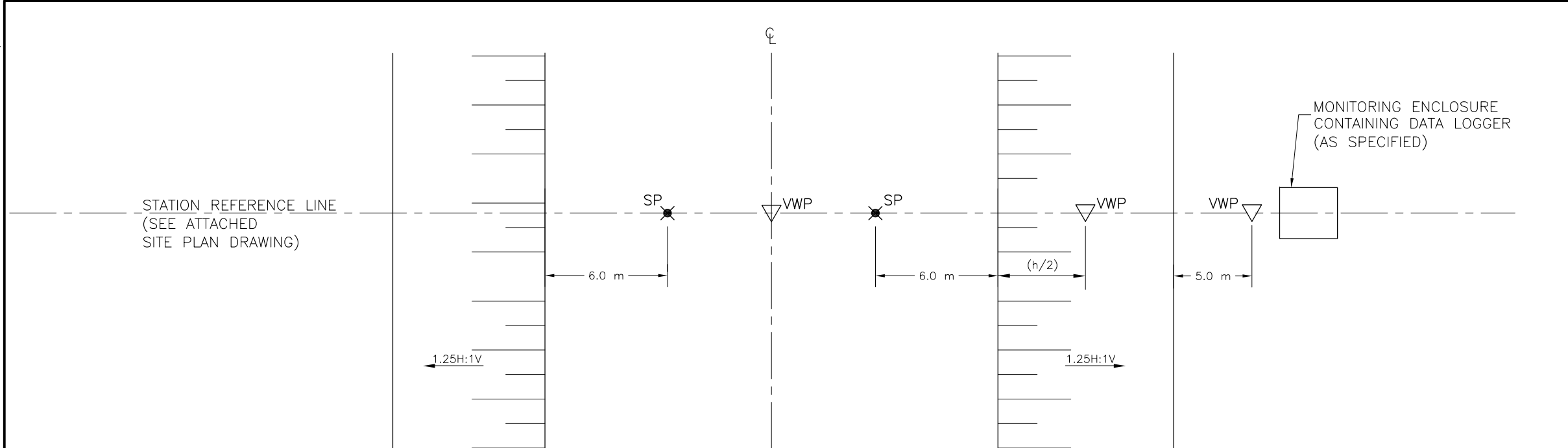
NOT TO SCALE

LEGEND	
NP 	NAIL PIN
SP 	SETTLEMENT PLATE (PLAN)
SP 	SETTLEMENT PLATE (SECTION)
VWP 	VIBRATING WIRE PIEZOMETER (PLAN)
VWP 	VIBRATING WIRE PIEZOMETER (SECTION)


- ## NOTES
1. SEE SPECIFICATIONS FOR LOCATION, DEPTH AND NUMBER OF INSTRUMENTS.
 2. "h" REFERS TO THE HEIGHT OF THE EMBANKMENT AT THE MONITORING SECTION.
 3. SUBSURFACE STRATUM SHOWN FOR ILLUSTRATION PURPOSES ONLY. FOR DETAILED SUBSURFACE INFORMATION, SEE FOUNDATION INVESTIGATION REPORT.



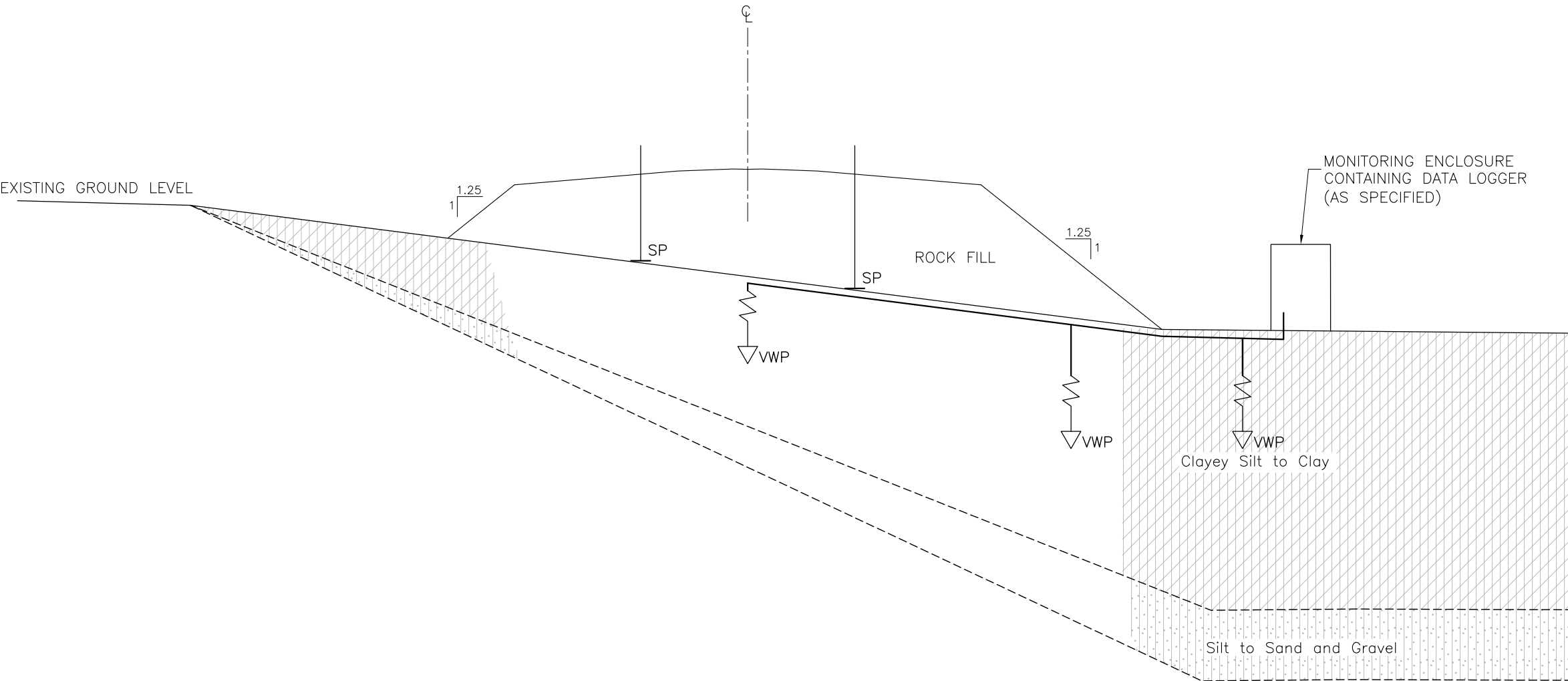
NO.			DATE		
BY			REVISION		
Geocres No. 32D-17					
HWY. 66		PROJECT NO. 10-1191-0044			DIST.
SUBM'D. MT		CHKD.	DATE: NOV 2013		SITE:
DRAWN: TB		CHKD. SEMC	APPD. JMAC		DWG. G3



MONITORING PLAN – TYPE B
NOT TO SCALE

CONT No. WP No. 5091-07-00		
HIGHWAY 66 HIGH FILL H1/H4 EMBANKMENT MONITORING PROGRAM PLAN AND SECTION – TYPE B		SHEET
 Golder Associates Ltd. SUDBURY, ONTARIO, CANADA		

LEGEND	
SP x	SETTLEMENT PLATE (PLAN)
SP	SETTLEMENT PLATE (SECTION)
VWP ▽	VIBRATING WIRE PIEZOMETER (PLAN)
VWP ▽	VIBRATING WIRE PIEZOMETER (SECTION)



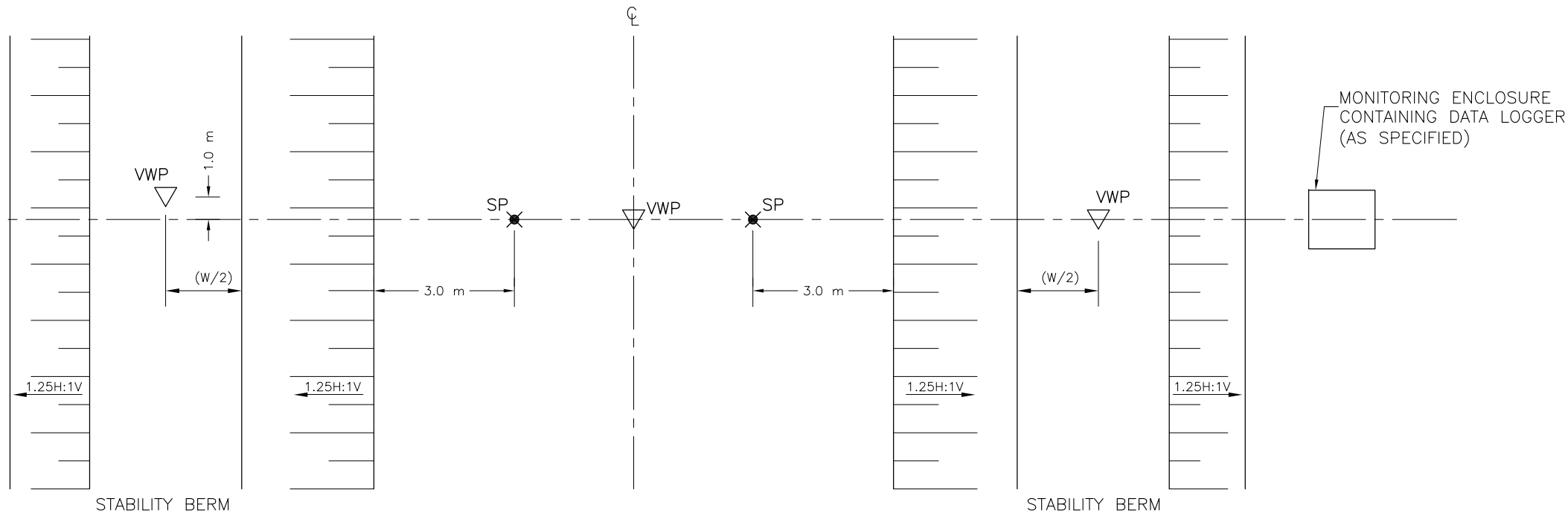
MONITORING SECTION – TYPE B
NOT TO SCALE

- NOTES
- SEE SPECIFICATIONS FOR LOCATION, DEPTH AND NUMBER OF INSTRUMENTS.
 - "h" REFERS TO THE HEIGHT OF THE EMBANKMENT AT THE MONITORING SECTION.
 - SUBSURFACE STRATUM SHOWN FOR ILLUSTRATION PURPOSES ONLY. FOR DETAILED SUBSURFACE INFORMATION, SEE FOUNDATION INVESTIGATION REPORT.

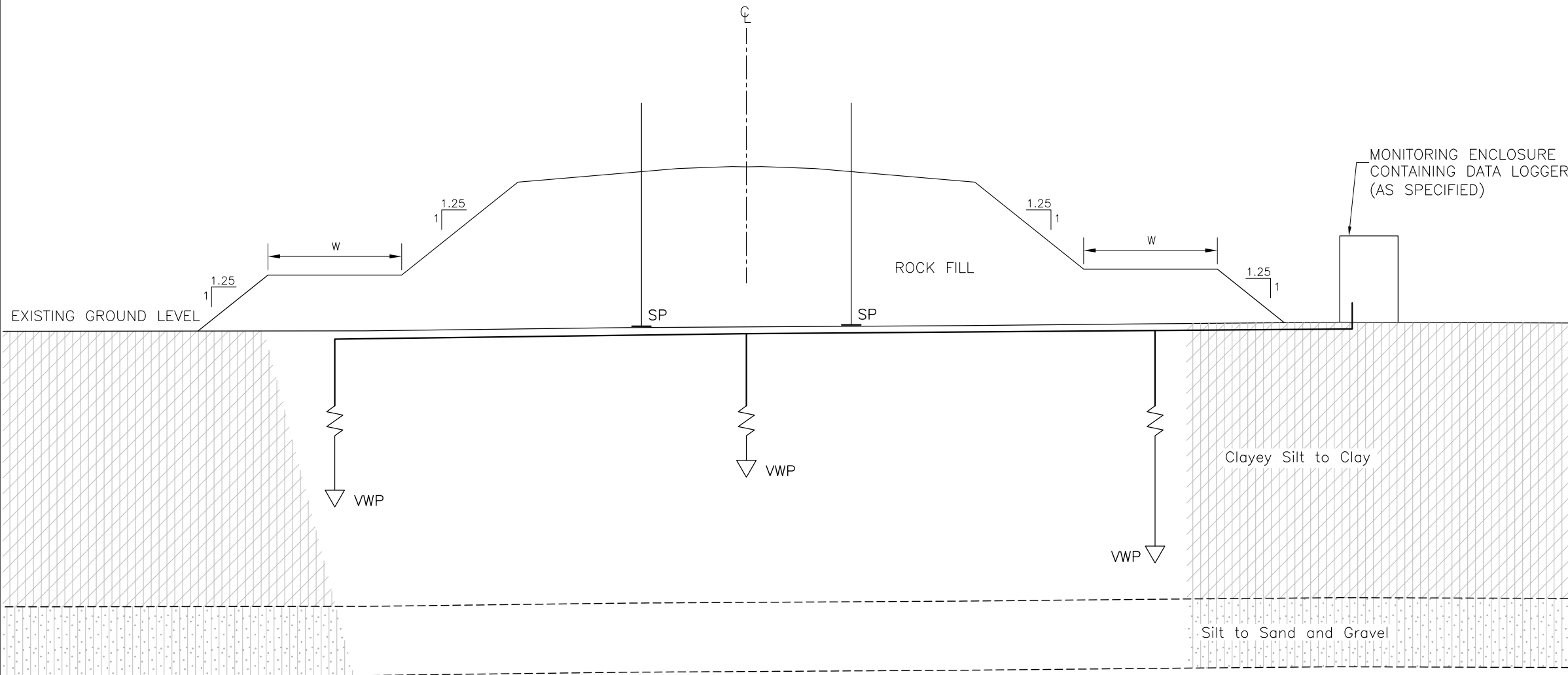


NO.	DATE	BY	REVISION
Geocres No. 32D-17			
HWY. 66	PROJECT NO. 10-1191-0044		DIST.
SUBM'D. MT	CHKD.	DATE: NOV 2013	SITE:
DRAWN: TB	CHKD. SEMC	APPD. JMAC	DWG. G4

STATION REFERENCE
LINE
(SEE ATTACHED
SITE PLAN DRAWING)



MONITORING PLAN – TYPE C
NOT TO SCALE



MONITORING SECTION – TYPE C
NOT TO SCALE

CONT No.
WP No. 5091–07–00

HIGHWAY 66
HIGH FILL H1/H4
EMBANKMENT MONITORING PROGRAM
PLAN AND SECTION – TYPE C

SHEET



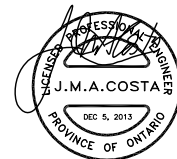
Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

LEGEND

- SP ✕ SETTLEMENT PLATE (PLAN)
- SP ⊥ SETTLEMENT PLATE (SECTION)
- VWP ▽ VIBRATING WIRE PIEZOMETER (PLAN)
- VWP ▽↓ VIBRATING WIRE PIEZOMETER (SECTION)

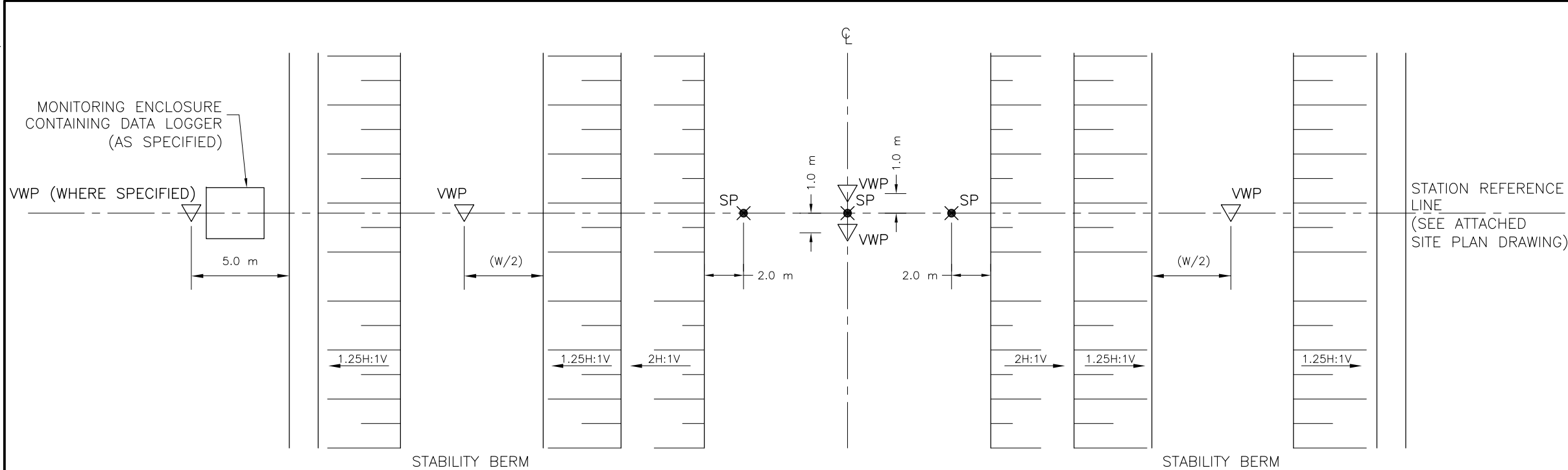
NOTES

- SEE SPECIFICATIONS FOR LOCATION, DEPTH AND NUMBER OF INSTRUMENTS.
- "W" REFERS TO THE WIDTH OF THE STABILITY BERM AT THE MONITORING SECTION.
- SUBSURFACE STRATUM SHOWN FOR ILLUSTRATION PURPOSES ONLY. FOR DETAILED SUBSURFACE INFORMATION, SEE FOUNDATION INVESTIGATION REPORT.




NO.	DATE	BY	REVISION
Geocres No. 32D–17			
HWY. 66	PROJECT NO. 10–1191–0044		DIST.
SUBM'D. MT	CHKD.	DATE: NOV 2013	SITE:
DRAWN: TB	CHKD. SEMC	APPD. JMAC	DWG. G5

MINISTRY OF TRANSPORTATION, ONTARIO
PLOT DATE: November 21, 2013
FILENAME: \\pwr\proj\10-1191-0044\Drawings\Monitoring\1011910044M020_G3-08.dwg



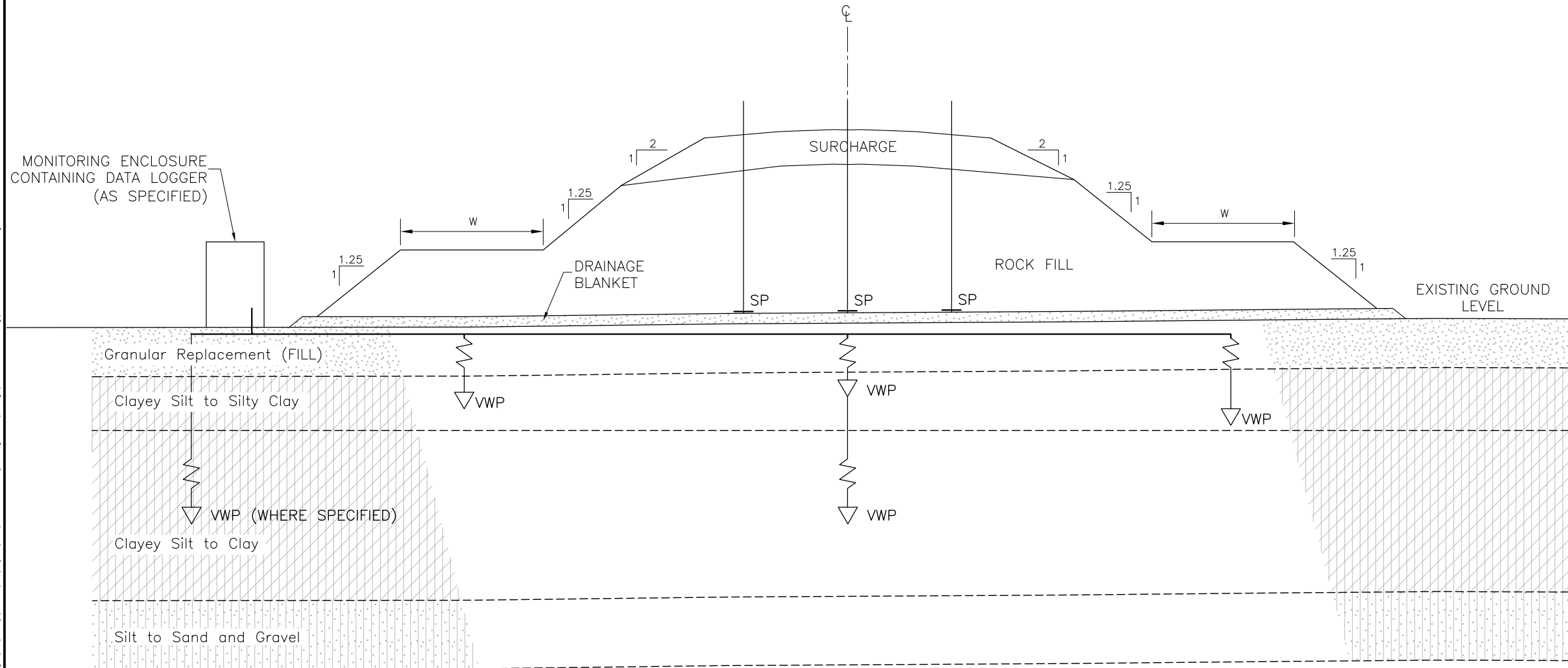
CONT No.
WP No. 5091-07-00

HIGHWAY 66
SWAMP CROSSING H6/H7
EMBANKMENT MONITORING PROGRAM
PLAN AND SECTION - TYPE D

**Golder Associates Ltd.**
SUDBURY, ONTARIO, CANADA

SHEET

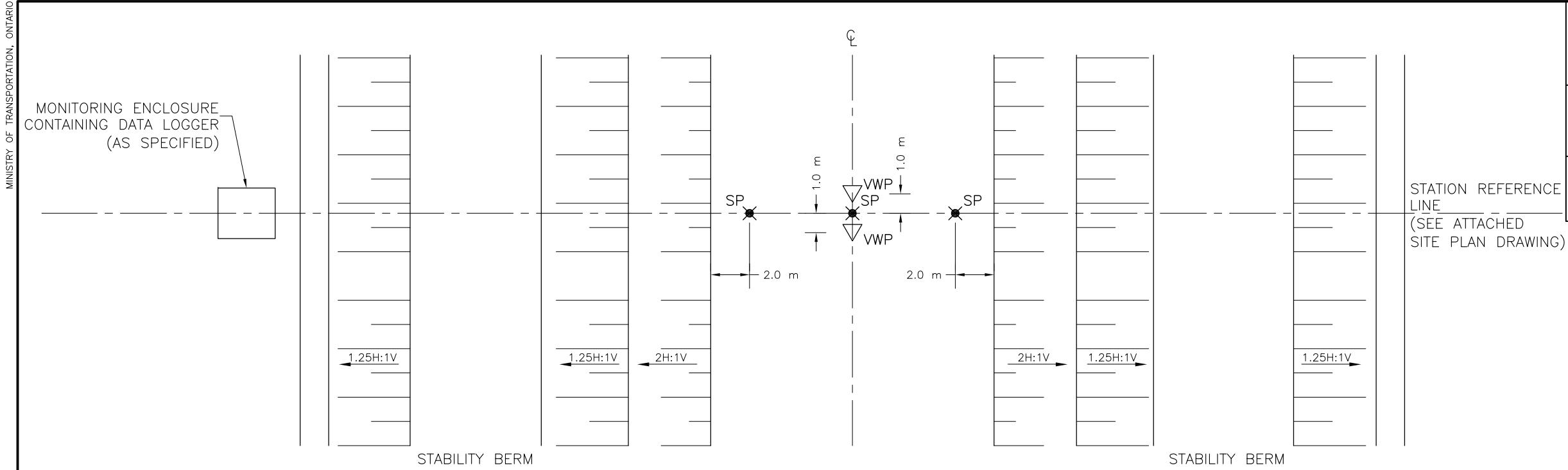
LEGEND	
SP	SETTLEMENT PLATE (PLAN)
SP	SETTLEMENT PLATE (SECTION)
VWP	VIBRATING WIRE PIEZOMETER (PLAN)
VWP	VIBRATING WIRE PIEZOMETER (SECTION)



- NOTES
- SEE SPECIFICATIONS FOR EXACT LOCATION, DEPTH AND NUMBER OF INSTRUMENTS.
 - "W" REFERS TO THE WIDTH OF THE STABILITY BERM AT THE MONITORING SECTION.
 - SUBSURFACE STRATUM SHOWN FOR ILLUSTRATION PURPOSES ONLY. FOR DETAILED SUBSURFACE INFORMATION, SEE FOUNDATION INVESTIGATION REPORT.




NO.	DATE	BY	REVISION
Geocres No. 32D-17			
HWY. 66	PROJECT NO. 10-1191-0044		DIST.
SUBM'D. TZ	CHKD. TZ	DATE: NOV 2013	SITE:
DRAWN: TB	CHKD. SEMC	APPD. JMAC	DWG. G6



CONT No.
WP No. 5091-07-00

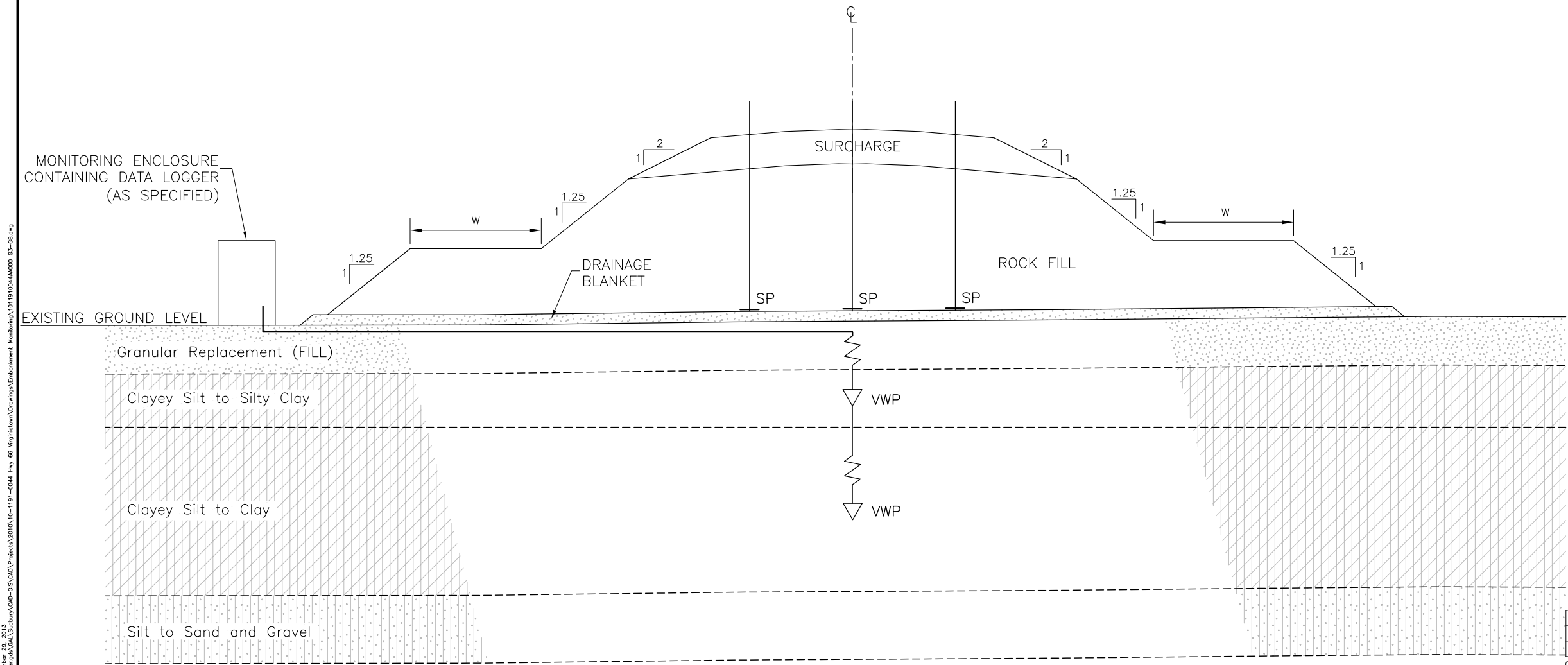
HIGHWAY 66
SWAMP CROSSING H6/H7
EMBANKMENT MONITORING PROGRAM
PLAN AND SECTION – TYPE E

**Golder Associates Ltd.**
SUDBURY, ONTARIO, CANADA

SHEET

STATION REFERENCE
LINE
(SEE ATTACHED
SITE PLAN DRAWING)

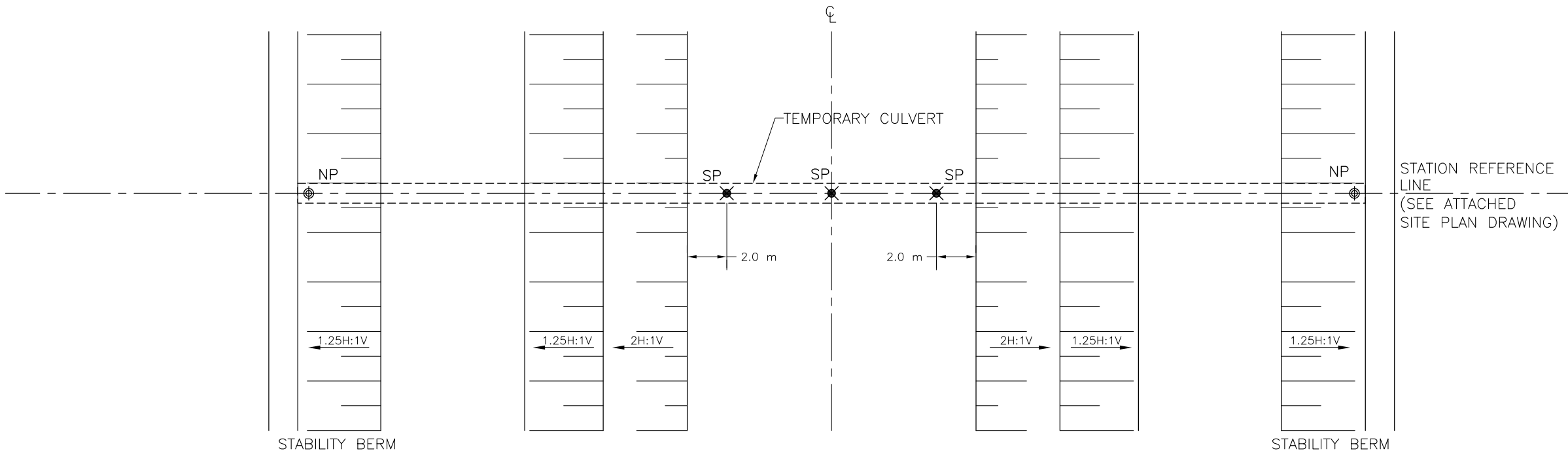
LEGEND	
SP ✕	SETTLEMENT PLATE (PLAN)
SP ↓	SETTLEMENT PLATE (SECTION)
VWP ▽	VIBRATING WIRE PIEZOMETER (PLAN)
VWP ↓	VIBRATING WIRE PIEZOMETER (SECTION)



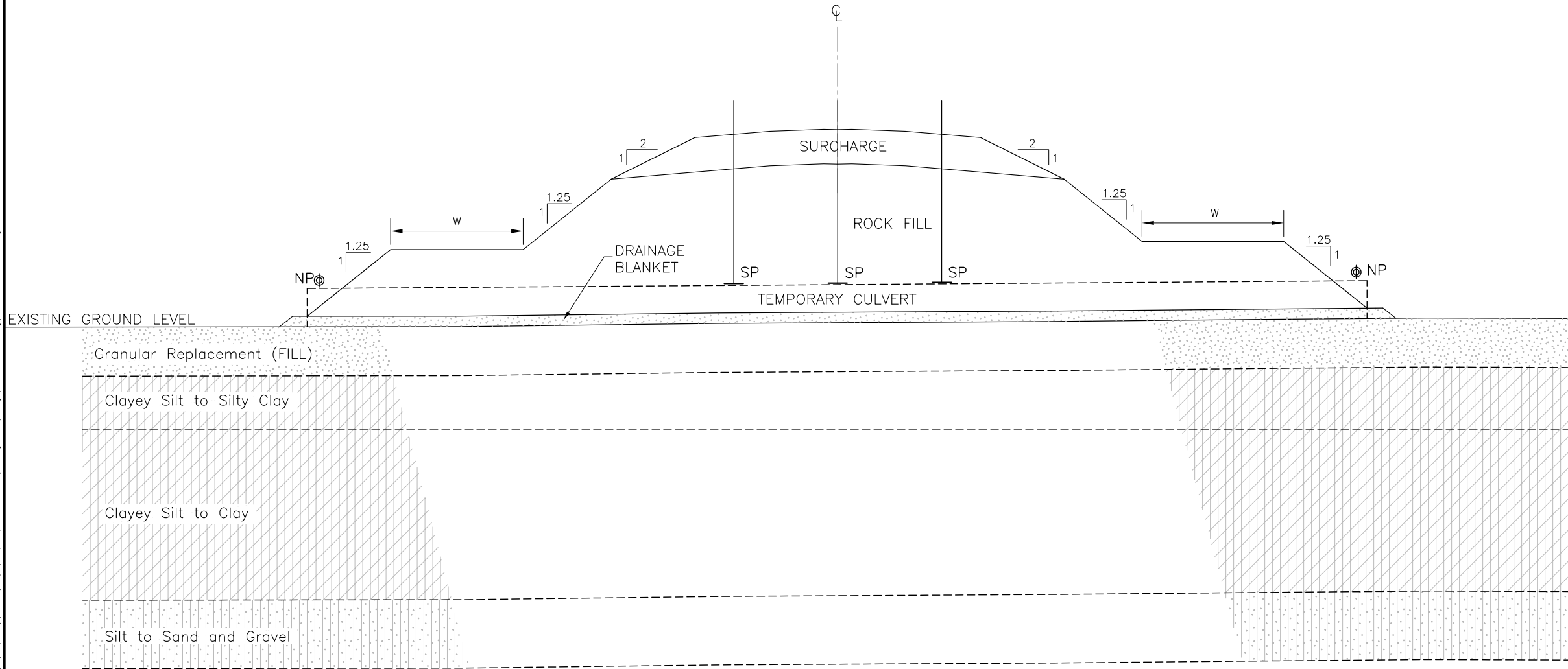
- NOTES
- SEE SPECIFICATIONS FOR LOCATION, DEPTH AND NUMBER OF INSTRUMENTS.
 - SUBSURFACE STRATUM SHOWN FOR ILLUSTRATION PURPOSES ONLY. FOR DETAILED SUBSURFACE INFORMATION, SEE FOUNDATION INVESTIGATION REPORT.



NO.	DATE	BY	REVISION
Geocres No. 32D-17			
HWY. 66	PROJECT NO. 10-1191-0044		DIST.
SUBM'D. TZ	CHKD. TZ	DATE: NOV 2013	SITE:
DRAWN: TB	CHKD. SEMC	APPD. JMAC	DWG. G7




MONITORING PLAN – TYPE F
NOT TO SCALE






MONITORING SECTION – TYPE F
NOT TO SCALE

CONT No.
WP No. 5091-07-00

HIGHWAY 66
SWAMP CROSSING H6/H7
EMBANKMENT MONITORING PROGRAM
PLAN AND SECTION – TYPE F

**Golder Associates Ltd.**
SUDBURY, ONTARIO, CANADA

SHEET

LEGEND	
NP 	NAIL PIN
SP 	SETTLEMENT PLATE (PLAN)
SP 	SETTLEMENT PLATE (SECTION)

- NOTES
1. SEE SPECIFICATIONS FOR LOCATION, DEPTH AND NUMBER OF INSTRUMENTS.

3. SUBSURFACE STRATUM SHOWN FOR ILLUSTRATION PURPOSES ONLY. FOR DETAILED SUBSURFACE INFORMATION, SEE FOUNDATION INVESTIGATION REPORT.



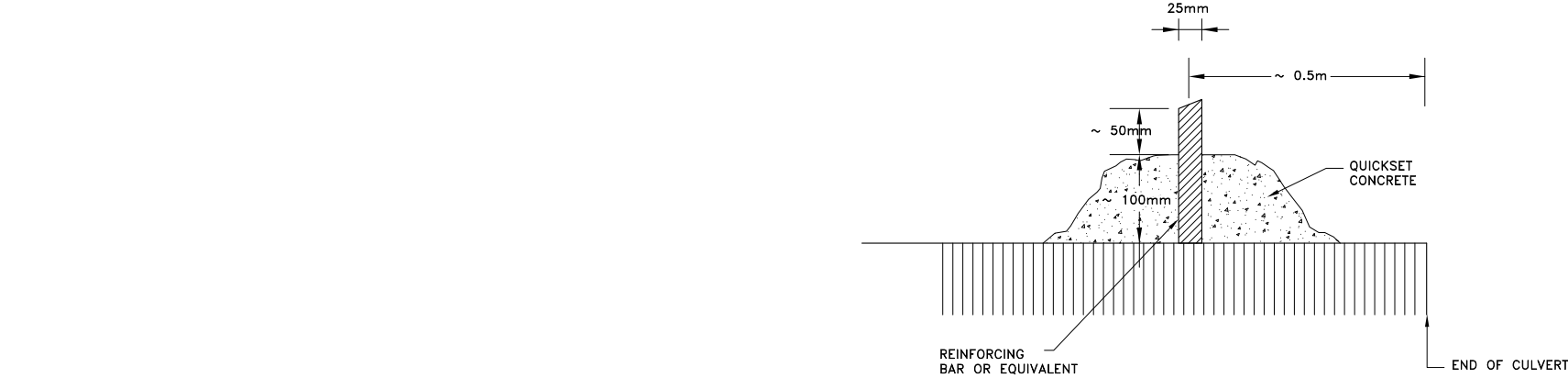
NO.	DATE	BY	REVISION
Geocres No. 32D-17			
HWY. 66		PROJECT NO. 10-1191-0044	DIST.
SUBM'D. TZ	CHKD. TZ	DATE: NOV 2013	SITE:
DRAWN: TB	CHKD. SEMC	APPD. JMAC	DWG. G8

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

CONT No.
GWP No. 5091-07-00

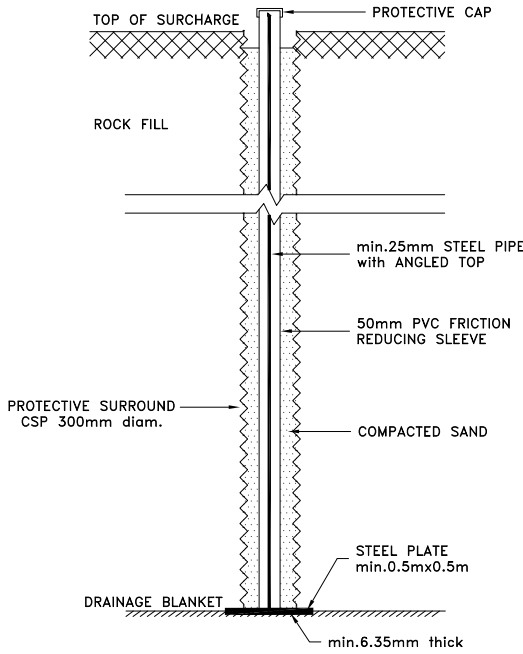
HIGHWAY 66
TYPICAL INSTRUMENT
INSTALLATION DETAILS

Golder Associates
Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



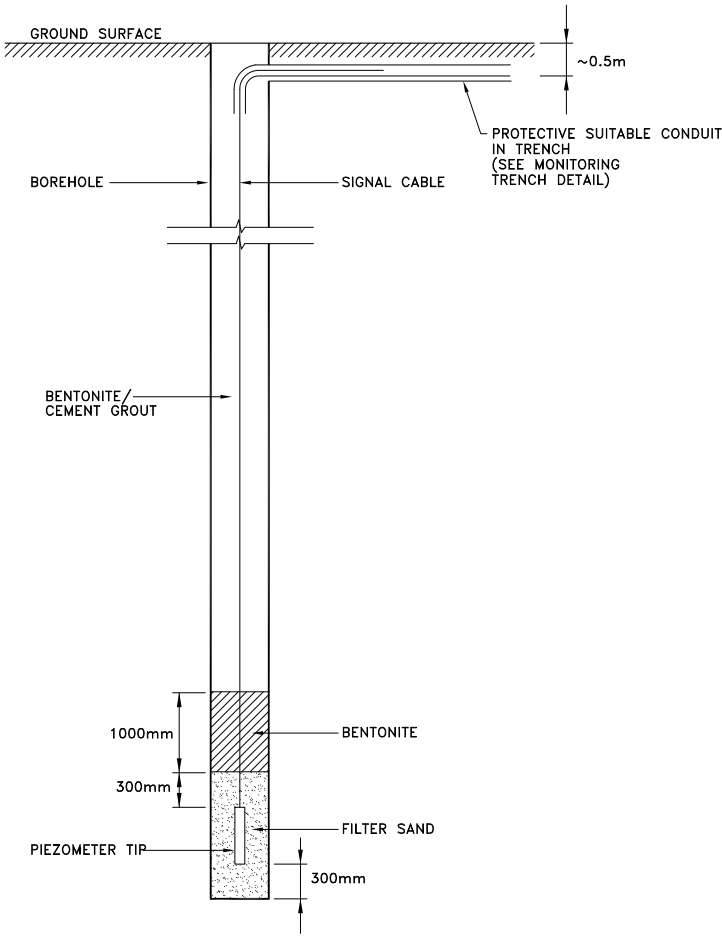
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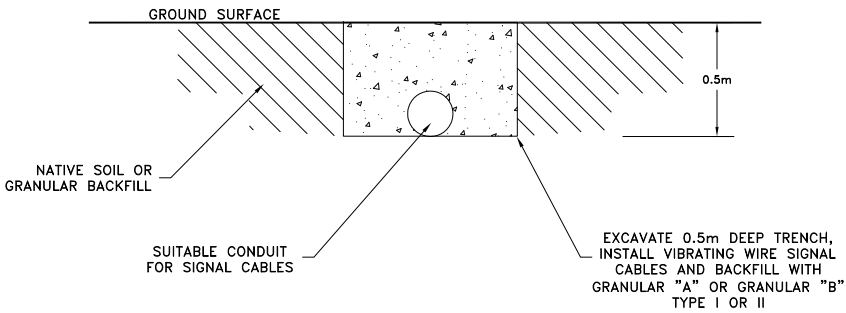
SETTLEMENT PLATE (SP)

NOT TO SCALE



VIBRATING WIRE PIEZOMETER (VWP)

NOT TO SCALE



MONITORING TRENCH

NOT TO SCALE



NO.	DATE	BY	REVISION
Geocres No. 32D-17			
HWY. 66		PROJECT NO. 10-1191-0044	DIST.
SUBM'D. TZ	CHKD. TZ	DATE: NOV 2013	SITE:
DRAWN: TB	CHKD. SEMC	APPD. JMAC	DWG. G9

DRAINAGE BLANKET – Item No.

Non-Standard Special Provision

1.0 SCOPE

This non-standard special provision specifies the requirements for the surface preparation, supply, placement and compaction of the Drainage Blanket (Blanket) in connection with the installation of the prefabricated wick drains.

2.0 MATERIALS

The Drainage Blanket shall be pit run Granular 'B' Type I and shall satisfy the physical and gradation requirements as specified in OPSS.PROV 1010 except that:

- 100% shall pass the 26.5 mm sieve; and,
- no more than 5% shall pass the 0.075 mm sieve.

3.0 CONSTRUCTION

3.1 The Drainage Blanket shall be placed and compacted to the limits and grades shown on the Contract Drawings or as directed by the Contract Administrator.

3.2 The Blanket shall be placed subsequent to the required stripping/sub-excavation of organic deposits, to an elevation meeting the following requirements:

- at least 0.5 m above the groundwater table during the installation of the wick drains;
- a minimum thickness of 0.5 m; and,
- at least 0.3 m above the original grade.

3.3 The Drainage Blanket shall be end-dumped for placement below water.

3.4 The Drainage Blanket shall be placed and compacted in lift thicknesses not exceeding 250 mm except below water as described in Item 3.3.

4.0 PAYMENT

4.1 Measurement for Payment

Measurement for payment shall be by the tonne. The method of determining the mass of materials for payment shall conform to OPSS 102.

4.2 Basis of Payment

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

EXCAVATION OF ORGANICS – Item No.

Non-Standard Special Provision

Scope

This non-standard special provision outlines the procedures(s) to be used for excavating organic deposits at the following area:

- Highway 66 – STA 14+040 to 14+560 (H6/H7 Swamp Crossing)

The excavation procedures to be followed are:

- Work may be carried out simultaneously starting from both ends of the swamp and progressing towards the centre along the embankment footprint;
- Removal of the organic deposits within the proposed wick drain treatment footprint and backfilling of the excavation shall be carried out simultaneously in accordance with OPSS 209. The limits of the removal of organics is described elsewhere in the Contract Documents.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

WICK DRAINS – Item No.

Non-Standard Special Provision

1.0 SCOPE

This non-standard special provision specifies the requirements for the supply and installation of wick drains in accordance with the details shown on the plan and profile/cross-section drawings and with the requirements of this specification.

2.0 REFERENCES

3.0 DEFINITIONS

Quality Verification Engineer (QVE): means an Engineer who has a minimum of five (5) years experience related to the design and installation of wick drains or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the Contract Documents and to issue Certificate(s) of Conformance.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Design Requirements

4.01.01 Qualifications

The work shall be undertaken by a recognized specialist Subcontractor who has proven satisfactory experience in work of this type and magnitude and has completed a minimum of two wick drain installation projects in the last five (5) years, each project with the following characteristics:

- Minimum installation depth: not less than 15 m; and,
- Total length of wick drains: not less than 100,000 m.

The specialist Subcontractor's qualifications shall be submitted to the Contract Administrator not later than fifteen (15) working days in advance of commencing the installation of wick drains.

4.01.02 Installation Method Proposal Submission

At least fifteen (15) working days prior to the installation of the wick drains, the Contractor shall submit to the Quality Verification Engineer, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Size, type, weight, maximum pushing force, and configuration of the installation rig;
- Dimensions and length of mandrel;
- Details of wick drain anchorage;
- Detailed description of proposed installation procedures;
- Proposed methods for overcoming obstructions; and,

- Proposed methods for splicing wick drains.

4.02 Submission Requirements

4.02.01 Certificate of Conformance – Material

The Contractor shall submit a sample of the wick drain to the Quality Verification Engineer for review prior to the installation of the wick drains on the Contract. The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one (1) week prior to commencement of work under this item. The Certificate shall state that the wick drain material is in conformance with the requirements and specifications of the Contract Documents.

4.02.02 Certificate of Conformance – Installation Method

The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of fifteen (15) days prior to commencement of work under this item. The Certificate shall state that the proposed installation procedures are in conformance with the requirements and specifications of the Contract Documents.

4.02.03 Certificate of Conformance – Final

Upon completion of the wick drain installation, the Contractor shall submit to the Contract Administrator a final Certificate of Conformance sealed and signed by the Quality Verification Engineer. The certificate shall state that the work has been carried out in general conformance with the installation procedures and specifications of the Contract Documents.

5.0 MATERIALS

5.01 General

The Contractor shall supply all materials and equipment required for the installation of wick drains unless noted otherwise.

5.02 Wick Drains

The prefabricated wick drain shall consist of a continuous plastic drainage core wrapped in a non-woven geotextile. The core configuration shall be 'Studded' or 'Grooved' ('Filament' or 'Cusped' are not acceptable).

The prefabricated wick drain material shall meet the minimum requirements specified in Table 1.

The Contractor shall submit a 1 m long, full width, sample of the wick drain material to the Contract Administrator for information at least one (1) month prior to commencement of work under this item. The sample shall be stamped or labeled by the manufacturer as being representative of the wick drain material having the specified trade name. Documentation indicating the source and the physical and mechanical properties of the wick drain shall be provided.

Manufacturer certification shall be provided for all wick drain material delivered to the project. Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation.

All wick drains supplied shall be free of defects, rips, holes or flaws. During shipment the wick drain shall be protected from damage. During on-site storage the storage area shall be such that the wick drain is protected from sunlight, dirt, dust, mud, debris and any other detrimental substances and construction equipment.

Table 1 – Wick Drain Material Specification

Property	Test Method	Units	Specification
Physical			
Drain Body Material	--	--	Polypropylene – Studded or Grooved
Filter Material	--	--	Polypropylene – Non-Woven Geotextile
Width	--	mm	≥ 100
Core Thickness	ASTM D-5199	mm	≥ 2
Composite Thickness	ASTM D-5199	mm	≥ 3
Mass of Core	ASTM D-3776	g/m	≥ 40
Mass of Filter	ASTM D-5261	g/m ²	≥ 130
Mechanical			
Tensile Strength Core	ASTM D-638	N	≥ 800
Grab Tensile Strength Filter	ASTM D-4632	N	≥ 600
Puncture Strength	ASTM D-4533	N	≥ 200
Filter Trapezoidal Tear	ASTM D-4833	N	≥ 200
Apparent Opening Size (AOS) of Filtered Material	ASTM D-4751	µm	≥ 40
Discharge Capacity @ 10 kPa	ASTM D-4716	m ³ /s	≥ 1.2 x 10 ⁻⁴
Discharge Capacity @ 240 kPa	ASTM D-4716	m ³ /s	≥ 1.0 x 10 ⁻⁴
Permittivity	ASTM D-4491	sec ⁻¹	≥ 0.25

6.0 EQUIPMENT

Wick drains shall be installed vertical, with equipment which will minimize disturbance to the drainage blanket or the native subsoil during the installation operation. Static or vibratory methods are considered acceptable. Falling weight impact hammers will not be allowed.

The Contractor is advised that the site is considered as an environmentally sensitive area and therefore the control of any water effluent needs to be carefully planned and organized. Jetting techniques to install the wick drains, therefore, shall be subjected to the approval of the Contract Administrator.

The Contractor shall be permitted to use augering equipment to predrill or to loosen the native soils and the drainage blanket if required to facilitate the installation of the wick drains.

Each prefabricated wick drain shall be installed using a mandrel or sleeve that shall be advanced through the drainage blanket and the underlying soil. The mandrel shall protect the prefabricated wick drain material from tears, cuts and abrasions during installation and shall be withdrawn after the installation of the wick drain. The mandrel shall be provided with an “anchor” rod or plate at the bottom to prevent the soil from entering the bottom of the mandrel during installation of the wick drain and to anchor the bottom of the wick drain at the

required depth at the time of mandrel removal. The projected cross-sectional area of the mandrel and anchor combination shall not exceed 7,700 mm².

7.0 CONSTRUCTION

7.01 Site Conditions

The Contractor shall refer to the Foundation Investigation Reports in the Contract Documents for a description of subsurface conditions at this site. The Record of Borehole sheets are not represented as a complete description of the subsurface conditions, but only present what was found in borings at the indicated locations on the date boreholes were drilled. The subsurface conditions may be variable between the borehole locations. The Contractor should verify existing subsurface and surface conditions.

7.02 Construction Sequence

Wick drains shall be installed vertical, subsequent to the construction of the drainage blanket and prior to installation of monitoring instruments and placement of the embankment material.

7.03 Trial Wick Drains

Prior to the installation of prefabricated wick drains within the areas designated on the Contract Drawings, the Contractor shall demonstrate that the proposed materials, equipment and installation method produces a satisfactory wick drain installation in accordance with this specification. The Contractor will be required to install a total of ten (10) trial wick drains at locations within the work area as designated by the Contractor Administrator.

Should the ten (10) trial wick drains be installed to the satisfaction of the Quality Verification Engineer, the trial wick drains can be incorporated as part of the permanent installation. The Contractor will be compensated for each trial wick drain if the installation satisfies the requirements of this specification, at the same unit price as the production wick drains. The Contractor shall not be compensated for unsatisfactory trial wick drains.

Full time monitoring of the Contractor's method of installation will be required by the Contractor's Quality Verification Engineer. If, at any time, the Quality Verification Engineer considers that the method of installation does not produce a wick drain that satisfies the project requirements, the Contractor shall alter the method and/or equipment as necessary to comply with this specification.

7.04 Layout

The layout of the prefabricated wick drains locations/pattern shall be staked by the Contractor. The location of the drains shall not vary by more than 150 mm from the locations indicated on the Contract Drawings.

7.05 Plumbness

Wick drains shall be installed vertically, within a tolerance of not more than 10 mm per 500 mm. The equipment shall be carefully checked for plumbness and the Contractor shall provide the Contract Administrator with a suitable means of verifying the plumbness of the mandrel and of determining the depth of the drain at any time.

7.06 Splices

Splices or connections in the wick drain material shall be done in a manner that ensures continuity and avoids any reduction of the flow characteristics of the wick material. Splices shall overlap by a minimum length of 150 mm.

7.07 Cut-off

The wick drain shall be cut above the ground surface such that at least a 150 mm length protrudes above the top of the granular blanket at each wick drain location.

7.08 Tip Elevation

The wick drains shall be installed to the elevations as summarized in Table 2.

Table 2 – Tip Elevations for Wick Drain Installations

Station	WBL – North Toe	Median Centreline	EBL – South Toe
<i>H6/H7 Swamp Crossing – STA 14+040 to 14+560</i>			
14+040	300.2	304.5	303.5
14+050	299.0	294.0	303.2
14+060	296.5	293.0	300.5
14+070	294.8	291.2	291.2
14+080	293.5	288.3	291.0
14+090	290.3	287.3	290.0
14+100	290.3	287.1	288.5
14+110	289.2	287.1	287.0
14+120	288.9	287.1	287.0
14+130	287.0	287.0	287.0
14+140	287.0	287.0	287.0
14+150	287.2	287.0	287.0
14+160	287.4	287.5	287.0
14+170	287.6	288.0	286.9
14+180	287.8	289.0	286.9
14+190	289.0	289.5	287.9
14+200	289.1	289.9	288.9
14+210	290.1	290.2	289.4
14+220	291.1	290.7	289.9
14+230	291.1	291.1	290.4
14+240	291.5	291.5	290.9
14+250	291.9	292.1	291.4
14+260	292.3	292.7	291.9
14+270	292.8	293.3	294.8
14+280	293.3	293.9	294.8
14+290	293.8	294.5	295.3

Station	WBL – North Toe	Median Centreline	EBL – South Toe
<i>H6/H7 Swamp Crossing – STA 14+040 to 14+560</i>			
14+300	294.3	295.1	295.3
14+310	294.8	295.0	295.3
14+320	295.2	295.0	295.3
14+330	295.2	295.0	295.3
14+340	295.2	295.0	295.3
14+350	295.4	295.1	295.0
14+360	295.6	295.3	294.5
14+370	295.8	295.5	294.6
14+380	296.0	295.7	294.6
14+390	296.2	295.9	294.5
14+400	296.4	296.2	294.5
14+410	296.6	296.4	294.5
14+420	297.3	295.4	294.0
14+430	297.3	294.6	294.0
14+440	296.1	293.8	293.5
14+450	294.9	292.5	293.5
14+460	293.7	292.8	293.5
14+470	293.7	292.6	293.5
14+480	293.7	291.7	293.5
14+490	293.7	292.4	293.5
14+500	294.1	293.1	293.7
14+510	294.4	293.8	293.7
14+520	294.4	293.9	293.7
14+530	293.9	294.1	295.5
14+540	293.3	296.3	299.5
14+550	294.2	296.6	300.5
14+560	297.0	296.4	301.0

Notes: 1. The anticipated Tip elevations have been interpreted from the available borehole information assuming the wick drains will terminate at the top of bedrock or top of the sandy gravel to sand and gravel or penetrate about 1 m into the cohesionless deposit(s) underlying the compressible cohesive deposit. The wick drain tip elevations between and beyond the borehole locations were estimated by interpolation and extrapolation of the available data, respectively, and as such, the actual tip elevations may vary during wick drain installation. The lateral extent of the wick drain installation should be at least two (2) times the wick drain spacing beyond the proposed embankment toe.

7.09 Obstructions

Where obstructions are encountered below the working surface that cannot be penetrated by the wick drain

installation equipment, the Contractor shall complete the wick drain from the elevation of the obstruction to the working surface and notify the Contract Administrator. At the direction of the Contract Administrator, the Contractor shall attempt to install a new drain within a 500 mm radius of the obstructed drain. A maximum of two (2) attempts shall be made as directed by the Contract Administrator. The Contractor will be compensated for each obstructed wick drain unless the wick drain is improperly completed, in which case no compensation will be allowed.

7.10 Pre-augering

It may be necessary to pre-auger through the native soils and the drainage blanket at some locations to facilitate the installation of the prefabricated wick drain. Pre-augering shall not extend more than 1 m into the cohesive soils at the site. Any additional cost for preaugering shall be incorporated into the unit price.

7.11 Rejected Drains

Wick drains that are installed beyond the plan location by more than 150 mm, or that are damaged or are not installed in accordance with this specification shall be rejected. Rejected wick drains may be removed at the Contractor's own expense and time. The Contractor shall not be compensated for the materials and work associated with rejected wick drains.

Replacement wick drains shall be installed within a 500 mm radius from the location of the rejected drain as directed by the Contract Administrator.

7.12 Geotechnical Instrumentation

Installation of the wick drains should be coordinated with the placement of geotechnical instrumentation as shown on the Contract Drawings. Special care shall be taken to install wick drains in such a manner so as not to disturb instrumentation already in place. The replacement of instrumentation damaged as a result of the Contractor's activities will be the responsibility of the Contractor.

7.0 PAYMENT

7.0.1 Measurement for Payment

Measurement of the item "WICK DRAINS", as may be revised by Adjusted Plan Quantity shall be by the linear metre for all accepted drains installed including the 150 mm length protruding above the granular drainage blanket. Properly completed obstructed wick drains and properly installed replacement wick drains and accepted trial wick drains will be measured for payment.

8.0 Basis of Payment

Payment at the Contract unit price per linear metre for the above item shall be full compensation for all labour, materials and equipment to complete the work in accordance with the Plans and Specifications.

No payment shall be made for unacceptable wick drains or delays or expenses incurred by the Contractor as a result of improper or unacceptable material or installation.

Expanded Polystyrene Embankment – Item No.

Special Provision

REQUIREMENTS FOR EXPANDED POLYSTYRENE EMBANKMENT FILL

1.0 SCOPE

This special provision covers the requirements for the supply and construction of the rigid expanded polystyrene embankment fill and associated works as shown on the contract drawings.

2.0 REFERENCES

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

2.1 National Standards of Canada

CAN/ULC – 5.701-11

2.2 ASTM

ASTM D6817 Standard Specification for Rigid Cellular Polystyrene Geofoam

ASTM D1621 Test Method for Compressive Properties of Rigid Cellular Plastics

ASTM C203 Test Method for Breaking Load and Flexural Properties of Block Type Thermal Insulation

ASTM C177 Test Method for Steady State Heat Flux Measurements and Thermal Transmission Properties by Means of the Heat Flow Apparatus

ASTM D2842 Test Method for Water Absorption by Rigid Cellular Plastics

ASTM D2863 Test Method for Measuring the Minimum Oxygen Content

ASTM D2126 Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

2.3 OPSS - Ontario Provincial Standard Specification

OPSS 212 Borrow

OPSS 501 Compaction

OPSS 517 Dewatering

OPSS 1010 Aggregates – Granular A, B, M, and Selected Subgrade Material

OPSS 1860 Geotextiles

3.0 SUBSURFACE CONDITIONS

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

4.0 DEFINITIONS

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

Rigid Expanded Polystyrene: Moulded rigid blocks produced by a process of pre-expansion, aging and forming of petroleum based raw material.

Rigid Extruded Expanded Polystyrene: Rigid boards made by extrusion of expanded polystyrene beads.

Production Lot: The quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

Quality Verification Engineer: Quality Verification Engineer means an Engineer with a minimum of five (5) years experience related to the design and/or construction of expanded polystyrene systems of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue of certificate(s) of conformance.

5.0 QUALIFICATION

The Contractor shall have on site at the commencement of the work, a representative of the supplier of the rigid expanded polystyrene to advise on recommended construction procedure.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

6.0 SUBMISSION AND DESIGN REQUIREMENTS

6.1 Submission of Shop Drawings

At least three (3) weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six (6) copies of the shop drawings and method statement signed and sealed by the Quality Verification Engineer that provides full details of materials and construction procedure.

6.2 Delivery, Storage, Handling, and Protection

The Contractor shall submit the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the rigid expanded polystyrene manufacturer's requirement.

6.3 Construction

The contractor shall submit full details of the following.

- a) The method of foundation excavation and preparation.
- b) Construction of 300 mm thick levelling pad.
- c) The method of placement of expanded polystyrene blocks including temporary ballasting and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer-by-layer basis.
- d) The method and limits of placement of polyethylene sheeting.
- e) The method of placement of subbase material.
- f) The method of placement of side slope cover.

6.4 Quality Verification Engineer

- (1) The Contractor shall submit details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted at least three weeks prior to the installation of the rigid expanded polystyrene embankments. The Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.
- (2) The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation. Upon completion of the Expanded Polystyrene Embankment the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the Expanded Polystyrene Embankment has been constructed in conformance with the installation procedures and specifications of the contract documents.

7.0 MATERIALS

7.1 Granular Levelling Pad

The levelling pad shall consist of a Granular 'A' material with gradation and physical requirements as specified in Special Provision 110S13.

7.2 Rigid Expanded Polystyrene

7.2.1 General

7.2.1.1 The Contractor shall submit:

1. A general statement as to the type, composition, and method of production of the material.
2. The manufacturer's name, address, phone number, identification of a contact person and description of background experience in the manufacturing of the rigid expanded polystyrene.
3. Certification of compliance of physical and mechanical properties.
4. An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the rigid expanded polystyrene.
5. The physical and mechanical properties of the rigid expanded polystyrene including:
 1. Geometry
 2. Nominal Density
 3. Compressive Strength
 4. Flexural Strength
 5. Thermal Resistance
 6. Dimensional Stability
 7. Flammability
 8. Water Absorption
6. Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
7. A sample of the expanded polystyrene material to the Quality Verification Engineer for review.
8. To the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one (1) week prior to commencement of work under this item. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents.

7.2.1.2 Production Lots

Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The polystyrene shall be free from defects affecting serviceability.

7.2.2 Detail Requirements

Requirements shall be as shown in Table 1 and as described below.

Table 1 – Material Properties

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear Dimensions - Flatness - Squareness	mm (min)	1200 x 600 x 300 ± 1% 10 mm in 3 m ± 0.5%	--
Nominal Density	kg/m ³ (max)	50	--
Compressive Strength at 5% Deformation	kPa (min)	115	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	240	ASTM C203
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Thermal Resistance	m ² .°C/W (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	4	ASTM D2842

7.2.2.1 Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel blocks of minimum acceptable dimensions of 1200 mm x 600 mm x 300 mm. The maximum deviation from the specified linear dimensions shall be ± 1%.

The flatness of the block faces shall be within ± 10 mm of a line formed by a 3 m straight edge.

The maximum difference in corner-to-corner dimensions (squareness) shall be 0.5%.

7.2.2.2 Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 115 kPa at a strain of not more than 5%. The maximum permissible permanent stress level should not exceed 30% of the compressive strength of the material at 5% deformation.

7.2.2.3 Flexural Strength

The minimum flexural strength of the polystyrene shall be 240 kPa. The flexural strength shall be determined in accordance to ASTM C203, Method 1, Procedure B.2.7.4 Dimensional Stability.

7.2.2.4 Dimensional Stability

Dimensional Stability shall be determined in accordance with ASTM D2126, Procedure G. A tolerance of 1.5% shall be satisfied.

7.2.2.5 Thermal Resistance

The thermal resistance shall be 0.7 m².°C/W for a 25 mm thickness using the following equation and using the average value from three specimens:

$$R_{25\text{ mm}} = \frac{R}{\text{thickness (mm)}} \cdot 25\text{mm}$$

The thermal resistance shall be measured in accordance with ASTM C177 or C518.

7.2.2.6 Flammability

The expanded polystyrene shall be classified as to surface burning characteristics in accordance with CAN/ULC – 51022 having a flame spread rating less than 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863.

7.2.2.7 Water Absorption

The water absorption as measured by ASTM D2842 shall be limited to 4% by volume.

7.2.2.8 Chemical Resistance

The expanded polystyrene shall be resistant to common inorganic acids and alkalis. A table identifying the chemical resistance as either resistant limited or not resistant shall be submitted.

7.2.2.9 Biological Resistance

The expanded polystyrene shall be resistant to biological degradation caused by organisms or enzymes.

7.2.2.10 Environmental

The expanded polystyrene shall be inert, non-nutritive and highly stable and shall not produce undesirable gases or leachate.

8.0 DELIVERY, STORAGE AND HANDLING

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

9.0 CONSTRUCTION

9.1 Foundation Excavation

Foundation excavation shall be carried out to the design elevations shown on the Contract Drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with Granular 'A' or Granular 'B' material.

9.2 Leveling Pad

Place, level and compact a layer of Granular 'A' material in accordance with OPSS 501 to within ± 30 mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any

place on a 3 m straight edge over the limits of the bottom course of blocks. The leveling pad shall not be placed on frozen ground.

9.3 Installation of Blocks

- (1) The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.
- (2) Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.
- (3) A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.
- (4) Sloping end adjustments at the abutments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.
- (5) Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.
- (6) The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.
- (7) The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction. The proposed method of protection during construction shall be submitted to the Contractor's Quality Verification Engineer for review and to the Contract Administrator for information purposes.
- (8) Exposed blocks shall be covered immediately to avoid possible burrowing by animals.
- (9) Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.
- (10) The top surface and side surfaces of the expanded polystyrene shall be covered with 6 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.
- (11) The side slope of the rigid expanded polystyrene embankment shall be covered with granular fill as detailed elsewhere in the Contract Drawings.

10.0 EQUIPMENT

All cutting of polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

11.0 QUALITY ASSURANCE

11.1 General

The Contract Administrator may undertake an independent testing program of the expanded polystyrene. Sampling and testing will be carried out in conformance with the relevant test procedure. The physical and thermal property testing identified in Table 1 will be conducted. A recognized testing laboratory accredited by the Standards Council of Canada shall conduct the testing.

11.2 Sampling Frequency

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. As a minimum, three (3) blocks shall be tested.

11.3 Acceptance/Rejection

Failure of any one of the sample blocks 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 blocks shall be at the Contractor's expense.

12.0 MEASUREMENT FOR PAYMENT

12.1 Actual Measurement

Measurement will be by volume in cubic metres measured in its original position and based on cross-sections.

13.0 PAYMENT

13.1 Basis of Payment

The Concrete Base pad and granular leveling pad shall be paid for with the appropriate tender items as detailed elsewhere in the contract.

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 and no extra payments will be made.

EMBANKMENT CONSTRUCTION IN WICK DRAIN AND PRELOAD AREAS

Operational Constraint

Scope

Fill placement for embankment construction shall not commence sooner than five (5) working days following completion of installation of all monitoring instrumentation, including notification and submission of required information (to the Contract Administrator) for all instrumentation. In any case, fill placement shall not commence before establishment of baseline readings. Baseline readings shall be conducted no sooner than three (3) days following notification of completion of installation of instrumentation and receipt of required installation information. If the baseline monitoring shows that the baseline has been established (i.e. consistent readings reflecting initial conditions obtained over the three (3) day period), embankment construction may commence. If the baseline is not established within a three (3) day period, additional daily readings shall be completed until three (3) consistent readings on consecutive days have been obtained prior to commencement of embankment construction.

The Contractor shall confirm that the elevation of the top of the surcharge is within 150 mm of the design top of surcharge. Elevations shall be provided to the Contract Administrator within five (5) working days of placement of the surcharge. The Contractor shall keep records of the thickness of each layer of fill placed and provide these records to the Contract Administrator within five (5) working days of reaching the top of each layer.

Embankment Construction – Highway 66 STA 13+080 to 13+180 (High Fill H4) and Highway 66 Connection – STA 10+000 to 10+050 (High Fill H1)

After the organic deposits have been excavated and the excavation backfilled with granular fill within the area of the embankment and stability berm footprint, where applicable, embankment construction may proceed in the following manner:

Fill Type	Fill Thickness	Estimated Construction Time	Wait Time After Completion of Embankment Construction to the Design Grade (Preload Period)
Rock Fill	Up to 4.5 m	Up to 14 days	12 months

The preload period given above is estimated based on the detailed foundation analysis and design and is intended to be a guide for bidding purposes only. The actual construction schedule will be determined by the results of the foundation monitoring program in place during construction.

Embankment Construction – Highway 66 STA 14+040 to 14+560 (H6/H7 Swamp Crossing)

After the organic deposits have been excavated and the excavation backfilled with granular fill within the area of the embankment and stability berm footprint, and wick drains and required instrumentation has been installed and baselined, embankment construction may proceed in the following manner:

Fill Type	Fill Thickness	Estimated Construction Time	Wait Time After Completion of Layer (before proceeding to the next layer)
Granular 'B' Type I (Drainage Blanket)	0.5 m	2.5 days	60 days*
Rock Fill	1.0 m	4.5 days	60 days
Rock Fill	Up to 4.0 m	Up to 12 days	30 days
Granular 'A' or Granular 'B' Type II (Surcharge)	2.0 m	7 days	44 days

* The wick drains and monitoring instrumentation are to be installed during this wait period; a minimum of 60 days wait period is required.

The wait times after completion of each layer, and surcharge period given above are estimated based on the detailed wick drain foundation analysis and design and are intended to be a guide for bidding purposes only. The actual construction schedule will be determined by the results of the foundation monitoring program in place during construction. The Contractor shall not begin placement of a new layer or removal of surcharge until permission is given by the Contract Administrator.

Operational Constraint

EXCESS MATERIAL MANAGEMENT AREA

This operational constraint covers the requirements for the construction of rock fill containment berms and for disposal of excess material in the area between STA 14+050 and STA 14+675 as shown on the Contract Drawings.

The Contractor shall ensure that the EMMA operations do not negatively impact the new construction works.

The Contractor shall submit full details of the following:

- a) The location and height of rock fill containment berms.
- b) The method of placement of geogrid and rock fill containment berms and excess material.
- c) The timing of placement of rock fill containment berms and excess material.

The construction of rock fill containment berms and the disposal of excess material shall be as follows:

- (1) There shall be a minimum buffer zone of 5 m between the outside edge of the rock fill containment berms and any excavation, structures or roadways.
- (2) The area under the entire EMM footprint shall be prepared such that any trees, brush or surface irregularities are removed to provide an approximately clean and even subgrade for placement of the geogrid and subsequent berm construction. Preparation of the surface is only required where geogrid will be placed.
- (3) Geogrid shall be placed below the rock fill containment berms in a direction parallel to the berm. The geogrid shall extend a minimum of 2 m beyond the outside edges of the base of the berm. The geogrid shall consist of extruded polypropylene uniaxial geogrid having a tensile stiffness of 1,000 kN/m. The geogrid shall be placed under all containment berms located between STA 14+050 and STA 14+675.
- (4) Prior to placing geogrid, a 300 mm thick granular bedding layer should be placed on the prepared surface. The bedding may consist on granular material or general soil as approved by the Contract Administrator. The bedding should be nominally compacted by the construction equipment prior to placement of the geogrid. The geogrid shall be placed such that the strong direction is oriented parallel to the cross section of the berm. Adjacent sections of geogrid shall be overlapped a minimum of 0.5 m except where major changes in direction occur (i.e. corners). Where major changes in the berm direction occur, a minimum 100 mm vertical separation shall be maintained between adjacent geogrid sections and filled with bedding material, or as specified by the manufacturer.

- (5) All materials shall be placed in a manner such that any mud waves generated move away from the new embankment, structures and roadways (i.e. they should start at the sides of the disposal area to the centre).
- (6) Rock fill containment berms between STA 14+050 and STA 14+375 may be constructed to a maximum height of 1.5 m above original ground surface. Between STA 14+375 and STA 14+550, rock fill containment berms may be constructed to a maximum height of 2.5 m above original ground surface. Between STA 14+550 and STA 14+675, rock fill containment berms may be constructed to a maximum height of 4.0 m above original ground surface. The berms may be “topped up” during the disposal operations provided that they do not exceed the maximum indicated height above the original ground surface.
- (7) The original ground surface shall be determined by survey. The amount that the berms can be topped up shall be relative to the survey of the original ground. The thickness of fill material placed shall be recorded and submitted to the Contract Administrator.
- (8) After construction of the new Highway 66 embankment commences above ground surface (i.e. after installation of wick drains and monitoring instrumentation), no more material may be placed in the EMM Area within a distance of 25 m from the nearest wick drain on the main alignment, or nearest structure or roadway.



APPENDIX H

Construction Administration Assignment

FOUNDATION MONITORING PROGRAM – Item No.

Non-Standard Special Provision

1.0 GENERAL

This Non-Standard Special Provision contains the requirements for the monitoring of the following geotechnical instruments:

- Settlement Plates (SP);
- Vibrating Wire Piezometers (VWP);
- Nail Pins (NP);

The instrumentation monitoring services include: data collection; data reduction and reporting; and adherence to criteria used to assess the embankment performance based on the monitoring data collected from the instruments installed by others.

1.0.1 Specialist Qualifications

The Foundation Engineering Consultant services required for this assignment have been categorized as “Geotechnical Specialty – High Complexity”.

The Foundation Engineering Consultants that are registered in MTO’s consultant registry acquisition system (RAQS) at the complexity rating in the required specialty that meets the identified complexity requirement for this assignment are eligible to provide Foundation Engineering services for this project. The Foundation Consultant collecting, assessing and reporting the monitoring data shall not be the same Foundation Consultant retained by the Contractor for the supply and installation of embankment monitoring equipment.

The Foundation Engineer shall have a minimum of five (5) years experience in the monitoring assessment of data and reporting for vibrating wire piezometers, settlement plates, nail pins and survey benchmarks data or alternatively demonstrate expertise through providing satisfactory monitoring services for the instrumentation specified for a minimum of two (2) projects in which the work was similar in scope to that in the contract.

1.0.2 Services, Deliverables and Records

The Foundation Engineering Consultant shall:

- Review the monitoring program and, if deemed necessary, submit in writing to the Contract Administrator recommendations for modifications to the Monitoring Program;
- Meet with the Contractor in order to receive the Portable Laptop Computer used for downloading data from the DataLoggers that are monitoring the vibrating wire piezometers, and to receive reports with details about installation of instruments installed by the Contractor and calibration certificates, as specified in the Special Provision titled, “Supply and Installation of Embankment Monitoring Equipment”, included in the contract documents;

- With the exception of the Portable Laptop Computer and DataLoggers referred to above and all instruments installed by the Contractor, supply all materials and equipment that are required for the Monitoring Program;
- Calibrate and maintain monitoring equipment;
- Take instrument readings, reduce data, prepare reports;
- Provide transmittal of instrumentation readings and reports to the Contract Administrator;
- Interpret instrumentation readings as needed for the purpose of on-going construction;
- Notify the Contract Administrator of required modifications to the construction procedures accordingly, if necessary. Interpretation shall include making correlations between instrumentation data and specific construction activities; and
- Notify the Contract Administrator if critical instrument readings (Review and Alert Levels), as specified herein, for any instrumentation are reached. Discuss as soon as possible (within 48 hours) with the Contract Administrator response action(s), and submit a plan of actions, to prevent the instrument readings from exceeding the critical levels.

Progress reports shall be submitted to the Contract Administrator, the MTO Contract Services Administrator and the MTO Foundations Engineer. Weekly reports shall be issued from the beginning of construction monitoring to the end of the one month period immediately after the top of the surcharge fill is reached. Thereafter, one report shall be submitted after each set of readings is taken. As a minimum, progress reports shall be submitted on a monthly basis. The progress reports shall discuss the Contractor's operations with respect to the installation of instrumentation, extent of embankment fill placed and a summary of the monitoring completed.

The Foundation Engineering Consultant shall maintain a Foundations Monitoring Diary. The diary shall document original conditions, work in progress, including any unusual or problem situations that arise, record of actions taken by the Contractor to rectify the situation, and restored conditions. The diary shall be supported by photographs of these conditions.

1.0.3 Submission of Foundation Monitoring Plan

The Foundation Engineering Consultant shall, in a brief narrative, discuss the applicable experience and qualifications of specialist staff, the role each will play in administration of the Contract, the authority to be assumed, and the reporting relationships with the construction administration staff.

The Consultant shall also complete the Foundation Monitoring Plan table in the format provided below.

Foundation Monitoring Plan		
<i>Major Inspection Tasks</i>	<i>Level of Inspection</i>	<i>Deliverable Record(s)</i>
List major inspection tasks associated with foundation monitoring.	State frequency/level of inspection.	List associated Deliverable Records for each task.

1.0.4 Purpose

The purpose of this Monitoring Program is to monitor settlements and pore water pressures in the foundation soils at select locations during construction of the new Highway 66 embankments (including surcharge) over high fill and swamp areas for the following extents of embankment construction:

- Highway 66 – STA 13+080 to STA 13+185 (High Fill H4)
- Highway 66 Connection – STA 10+000 to STA 10+050 (High Fill H1)
- Highway 66 – STA 14+040 to STA 14+560 (H6/H7 Swamp Crossing)

The rate of fill placement and the timing for the end of preloading and the removal of the surcharge shall be controlled by the instrumentation readings.

1.0.5 Drawings

Reference shall be made to the drawings titled, “Monitoring Section Location Plan”, “Embankment Monitoring Program – Plan and Section” and “Typical Instrument Installation Details” included in the Contract Package.

1.0.6 Subsurface Conditions

The subsurface Conditions at the site are described in the Foundation Investigation Reports as referenced elsewhere in the Contract Documents.

1.0.7 Equipment Operation

Monitoring shall be conducted year round. All monitoring equipment shall be maintained and rendered operational throughout the monitoring period.

Any equipment malfunction shall be investigated and attempts shall be made to remedy the malfunction. Notification of any equipment malfunction and equipment that cannot be repaired shall be made to the Contract Administrator. Documentation of the possible causes and suggested remedial measures shall be forwarded to the Contract Administrator.

1.0.8 Reading Schedule and Frequency

The Foundation Engineering Consultant shall save and archive raw data in electronic and hard copy format.

Monitoring shall commence immediately after the installation of an instrument. Monitoring is to continue during a period from the start of embankment construction to at least six (6) months following surcharge removal. The actual length of the monitoring period depends on the construction schedule and the results of monitoring amongst other factors.

The minimum monitoring frequencies along with the anticipated number of readings for the embankments in this contract is given in Table 1 and Table 2. The monitoring frequency is the same for each individual instrument indicated in the following tables. Instruments shall be read more or less frequently if judged to be required by the Contract Administrator.

It should be noted that the number of readings given in Table 1 and Table 2 are estimates and may vary depending on the actual construction schedule.

Table 1 – Minimum Monitoring Frequency for the Staged Construction of Highway 66 Embankment in H1/H4 High Fill Section

Stage	Frequency	Anticipated Number of Readings Per Monitoring Section ¹
Baseline Readings ²	Three readings on 3 consecutive days, no sooner than 7 days following installation	3
Immediately prior to start of embankment construction	Once	1
During embankment construction	Once every 1.5 m thick fill lift within 20 m of the monitoring section	3
Preload Period (anticipated duration: about 12 months)	Weekly: First month Bi-weekly: Second month Monthly: Third month to end of preload	16
After end of preload period to completion of consultant assignment	Monthly: Twelfth month to 6 months following preload period	6

NOTE: 1. Due to uncertainty of the construction schedule, the anticipated number of readings per monitoring section is not equivalent to the number of site visits required to carry out the monitoring program described herein.
2. Baseline Readings: value of instrumentation readings taken prior to construction to provide a baseline against which all subsequent readings are compared to assess movements of the ground and changes in piezometric head.

Table 2 – Minimum Monitoring Frequency for the Staged Construction of Highway 66 Embankment in H6/H7 Swamp Crossing Section

Stage	Frequency	Anticipated Number of Readings Per Monitoring Section¹
Baseline Readings ²	Three readings on 3 consecutive days, no sooner than 7 days following installation	3
Immediately prior to start of each stage of embankment construction (Stages 2 to 4)	Once	3
Waiting period after end of Stage 1 of embankment construction to top of drainage blanket and prior to Stage 2 construction (anticipated duration: about 60 days)	Weekly: first two weeks Bi-weekly: second week to eight week	5
During Stage 2 of embankment construction	Once during Stage 2 filling within 20 m of the monitoring section	1
Waiting period after end of Stage 2 of embankment construction to top of toe berm level and prior to Stage 3 construction (anticipated duration: about 60 days)	Twice per week: first 2 weeks Weekly: second 2 weeks Bi-weekly: Second month	8
During Stage 3 of embankment construction	Twice during Stage 3 filling within 20 m of the monitoring section approximately once every 1 m to 2 m fill lift within 20 m of the monitoring section	2
Waiting period after end of Stage 3 of embankment construction to top of profile grade level and prior to Stage 4 (surcharge) construction (anticipated duration: about 30 days)	Twice per week: first 2 weeks Weekly: second 2 weeks	6
During Stage 4 of embankment construction (i.e. surcharge placement)	Twice during Stage 4 filling within 20 m of the monitoring section, approximately once every 1 m fill lift within 20 m of the monitoring section	2

Waiting period after end of Stage 4 of embankment construction to top of surcharge and prior to surcharge removal (anticipated duration: about 44 days)	Twice per week: first 2 weeks Weekly: second 4 weeks	8
After surcharge removal to completion of consultant assignment	Weekly: First month Monthly: Second month to 6 months following surcharge removal	9

NOTE: 1. Due to uncertainty of the construction schedule and placement of fill, the anticipated number of readings per monitoring section is not equivalent to the number of site visits required to carry out the monitoring program described herein.

2. Baseline Readings: value of instrumentation readings taken prior to construction to provide a baseline against which all subsequent readings are compared to assess movements of the ground and changes in piezometric head.

2.0 INSTRUMENTATION SPECIFIC REQUIREMENTS

2.0.1 Settlement Plates (SP) and Nail Pins (NP)

Surveying

The elevations of Settlement Plates and Nail Pins shall be surveyed to an accuracy of plus/minus 2 mm or better and shall be reported to the nearest millimetre.

Surveying for settlement monitoring shall be conducted by a registered surveyor with appropriate equipment and experience. The surveyor shall be retained by the Foundation Engineering Consultant.

Reporting

A brief interpretation of the updated monitoring data shall be reported to the Contract Administrator within five (5) working days after each set of readings is obtained. A full set of up-to-date and processed monitoring data shall be presented in tabular and graphical form in the Progress Reports.

As a minimum, the following shall be submitted to the Contract Administrator in the Progress Reports based on the readings collected from the SPs and NPs:

- A plot of settlement of the base of the embankments (SPs) versus time;
- A plot of settlement of the temporary culvert (SPs and NPs) versus time;
- Fill height within 20 m of the instruments versus time;
- Plan view, cross section and profile sketches showing the top of fill location while the SPs and NPs were being surveyed.

Review and Alert Levels

Typically, embankment failures result in an acceleration of settlements after placement of a lift of fill. If this condition is observed or the maximum settlement measured exceeds the Review Levels in Table 3a, the Foundation Monitoring Consultant shall immediately inform the Contract Administrator and discuss response action(s). The Foundation Monitoring Consultant shall submit a plan of action(s) to prevent Alert Levels being reached. All construction work shall be continued such that instrument Alert Levels are not reached.

If the maximum settlement measured exceeds the Alert Levels in Table 3a, the Foundation Monitoring Consultant shall immediately inform the Contract Administrator and the Contract Administrator shall instruct the Contractor to stop all construction activities on and within the embankment. No construction shall take place on the affected embankment until all the following conditions are satisfied:

- The cause of the accelerated settlement has been identified and analyzed by the Foundation Engineer;
- Any corrective action deemed necessary by the Foundation Engineer has been implemented;
- The Contract Administrator deems it safe to proceed.

Table 3a – Review and Alert Levels for Instruments Monitoring Settlements

Instrument Type	Location	Monitoring Section	Hwy 66 Station	Offset from CL of Specified Lane (m)	Settlement Response Levels (mm)	
					Review	Alert
Settlement Plates (SP)	H1/H4 High Fill Section	A	13+080	12.0 m Lt	130	170
		A	13+080	3.0 m Lt	130	170
		A	13+080	3.0 m Rt	130	170
		A	13+080	14.0 m Rt	220	290
		B	13+150	3.0 m Lt	130	170
		B	13+150	4.0 m Rt	130	170
		C	10+050 ²	3.0 m Lt	330	430
		C	10+050 ²	3.0 m Rt	330	430
Nail Pin (NP)	H1/H4 High Fill Section	A	13+080 ¹	0.5 m from north end of the temporary culvert	20	30
				0.5 m from south end of the temporary culvert	20	30
Settlement Plates (SP)	H6/H7 Swamp Crossing	D	14+060	5.0 m Lt	1,500	1,900
				0 m	1,550	2,050
				5.0 m Rt	1,500	1,900
		D	14+090	5.0 m Lt	1,750	2,300

Instrument Type	Location	Monitoring Section	Hwy 66 Station	Offset from CL of Specified Lane (m)	Settlement Response Levels (mm)	
					Review	Alert
				0 m	1,900	2,450
Settlement Plates (SP)	H6/H7 Swamp Crossing	D	14+180	5.0 m Rt	1,750	2,300
				5.0 m Lt	1,350	1,750
				0 m	1,500	1,900
		E	14+270	5.0 m Rt	1,350	1,750
				5.0 m Lt	1,350	1,750
				0 m	1,450	1,850
		E	14+360	5.0 m Rt	1,350	1,750
				5.0 m Lt	900	1,150
				0 m	950	1,250
		D	14+450	5.0 m Rt	900	1,150
				5.0 m Lt	900	1,150
				0 m	1,050	1,350
		F	14+510	5.0 m Rt	900	1,150
				5.0 m Lt	440	550
				0 m	950	1,250
		D	14+530	5.0 m Rt	440	550
				5.0 m Lt	980	1,250
				0 m	1,050	1,350
				5.0 m Rt	980	1,250
Nail Pin (NP)	H6/H7 Swamp Crossing	F	14+510 ¹	0.5 m from north end of the temporary culvert	100	150
				0.5 m from south end of the temporary culvert	100	150

- NOTES: 1. Stations for the nail pins are approximate. All nail pins shall be installed on the top of the temporary culverts and along the culvert(s) centreline based on the locations above.
2. Station from Highway 66 Connection Alignment.

Vibrating Wire Piezometers (VWP)

Readout Unit

The VWPs shall be read using the Portable Laptop Computer supplied by the Contractor.

Reporting

A brief interpretation of the updated monitoring data shall be reported to the Contract Administrator within one (1) day during construction and within five (5) working days after each set of readings is obtained. A full set of up-to-date and processed monitoring data shall be presented in tabular and graphical form in the Progress Reports.

As a minimum, the following shall be submitted to the Contract Administrator in the Progress Reports based on the readings collected from the VWPs:

- A plot of piezometric head (elevation), background piezometer head (elevation) and fill elevation versus time for each VWP;
- A plot of excess pore pressure (EPP) and embankment vertical effective stress versus time for each VWP;
- Plan view, cross section and profile sketches showing the top of fill location while the VWP readings were being taken.
- Review and Alert levels on EPP versus time plots.
- All plots should be presented in both log time and liner time.

Review and Alert Levels

The increase in pore pressure in the foundation soils associated with the placement of successive lifts of fill should be equal to or lower than the increase in total vertical stress due to the fill placement. The failure of embankments founded on soft soils is usually associated with increases in pore pressure to levels which exceed the increase in total stress as described above. If this condition is observed or the maximum excess pore pressure measured exceeds the Review Levels in Table 3b, the Foundation Monitoring Consultant shall immediately inform the Contract Administrator and discuss response action(s). The Foundation Monitoring Consultant shall submit a plan of action(s) to prevent Alert Levels being reached. All construction work shall be continued such that the instrument Alert Levels are not reached.

If the maximum excess pore pressure measured exceeds the Alert Levels in Table 3b, the Foundation Monitoring Consultant shall immediately inform the Contract Administrator and the Contract Administrator shall instruct the Contractor to stop all construction activities on and within the embankment until all the following conditions are satisfied:

- The cause of the excess pore pressure has been identified and analyzed by the Foundation Engineer;
- Any corrective action deemed necessary by the Foundation Engineer has been implemented;
- The Contract Administrator deems it safe to proceed.

Table 3b – Review and Alert Levels for Excess Pore Pressures

Location	Section Type	Hwy 66 Station	Offset from Centreline (m)	Tip Elevation (m)	Fill Stage	Excess Pore Pressure (EPP) – Response Levels (kPa)		
						Review Level	Alert Level	Maximum EPP Before Following Stage
H1/H4 High Fill Section	A	13+080	0 m	301.4	1	25	45	N/A
	A	13+080	14.0 m Rt	300.6	1	25	45	N/A
	B	13+150	0 m	303.7	1	25	45	N/A
	B	13+150	17.5 m Rt	300.0	1	25	45	N/A
	B	13+150	34.0 m Rt	300.0	1	25	45	N/A
	C	10+050 ¹	15.5 m Lt	300.9	1	45	80	N/A
	C	10+050 ¹	0 m	303.3	1	45	80	N/A
	C	10+050 ¹	15.5 m Rt	299.7	1	45	80	N/A

H6/H7 Swamp Crossing	D	14+059	0 m	304.7	1	48	54	0
					2	48	54	0
					3	48	54	3
					4 ²	48	54	1
		14+060	14.5 m Lt	305.0	1	57	63	0
					2	45	50	0
					3	33	39	2
					4 ²	24	30	1
			14.0 m Rt	304.0	1	57	63	0
					2	45	50	0
					3	33	39	2
					4 ²	24	30	1
		14+061	0 m	299.7	1	64	71	9
					2	64	71	8
					3	64	71	14
					4 ²	64	71	12
H6/H7 Swamp Crossing	D	14+089	0 m	303.9	1	59	68	0
					2	42	48	0
					3	42	48	3
					4 ²	42	48	2
		14+090	14.5 m Lt	303.9	1	55	64	0
					2	41	48	0
					3	26	31	2
					4 ²	20	25	0
			14.0 m Rt	304.0	1	55	64	0
					2	41	48	0
					3	26	31	2
					4 ²	20	25	0
		14+097	0 m	299.1	1	62	72	13
					2	62	72	9
					3	62	72	22
					4 ²	62	72	22
H6/H7 Swamp Crossing	D	14+179	0 m	303.6	1	48	54	0
					2	48	54	0
					3	48	54	3
					4 ²	48	54	1
		14+180	14.0 m Lt	303.9	1	57	63	0
					2	45	50	0
					3	33	39	2
					4 ²	24	30	1
			13.5 m Rt	303.4	1	57	63	0
					2	45	50	0
					3	33	39	2
					4 ²	24	30	1
		14+181	0 m	298.1	1	64	71	9
					2	64	71	8
					3	64	71	14
					4 ²	64	71	12

	E	14+269	0 m	304.2	1	34	38	0
					2	44	51	0
					3	44	51	1
					4 ²	44	51	1
		14+271	0 m	299.0	1	36	41	8
					2	61	70	7
					3	61	70	11
					4 ²	61	70	16
H6/H7 Swamp Crossing	E	14+359	0 m	303.9	1	30	35	0
					2	42	49	0
					3	42	49	1
					4 ²	42	49	1
		14+361	0 m	300.4	1	35	40	8
					2	60	68	7
					3	60	68	11
					4 ²	60	68	16
H6/H7 Swamp Crossing	D	14+449	0 m	303.9	1	13	15	0
					2	18	24	0
					3	29	50	2
					4 ²	36	58	2
		14+450	13.0 m Lt	303.6	1	12	13	0
					2	14	19	0
					3	6	10	0
					4 ²	4	6	0
			13.5 m Rt	303.6	1	12	13	0
					2	14	19	0
					3	6	10	0
					4 ²	4	6	0
		14+451	0 m	298.8	1	11	12	3
					2	19	26	5
					3	31	54	10
					4 ²	49	79	20
H6/H7 Swamp Crossing	D	14+529	0 m	303.4	1	13	15	0
					2	18	24	0
					3	29	50	2
					4 ²	36	58	2
		14+530	13.5 m Lt	302.3	1	12	13	0
					2	14	19	0
					3	6	10	0
					4 ²	4	6	0
			13.5 m Rt	303.8	1	12	13	0
					2	14	19	0
					3	6	10	0
					4 ²	4	6	0
		14+531	0 m	299.9	1	11	12	3
					2	19	26	5
					3	31	54	10
					4 ²	49	79	20

NOTES: 1. Station from Highway 66 Connection Alignment
2. Surcharge Stage.

3.0 CONTROL MONITORING LEVELS

General

The monitoring program will provide input for the control of the appropriate time for embankment staged construction and surcharge placement, end of preloading, removal of surcharge and installation of permanent culverts.

Stabilization of Settlements due to Primary Consolidation

Settlement data monitored at the SPs and NPs allow for an approximate assessment of the total settlement that will occur due to primary consolidation and the approximate time required for settlements due to primary consolidation to stabilize.

The anticipated total settlement that will occur and the required time for settlements due to primary consolidation to stabilize shall be assessed for each of the SPs and NPs using an appropriate method.

4.0 FINAL REPORT

At the completion of the monitoring program, a final monitoring report shall be issued to the Contract Administrator. The monitoring results shall be presented in tabular and graphical form as described above for each instrument type. Interpretation of the monitoring data shall be included in the report.

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