



December 19, 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

**CULVERT AT STATION 14+510 (BC4)
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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GEOCRES NO.: 32D-18

Report Number: 10-1191-0044-R4

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REPORT





Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

1.0	INTRODUCTION.....	1
2.0	SITE DESCRIPTION.....	1
3.0	INVESTIGATION PROCEDURES.....	1
3.1	Foundation Investigation.....	1
4.0	SITE GEOLOGY AND SUBSURFACE CONDITIONS	3
4.1	Regional Geology	3
4.2	General Overview of Local Subsurface Conditions	3
4.2.1	Peat	4
4.2.2	Clayey Silt to Clay.....	4
4.2.2.1	Clayey Silt.....	4
4.2.2.2	Silty Clay to Clay	5
4.2.2.3	Clayey Silt.....	6
4.2.3	Silt.....	6
4.2.4	Gravelly Sand	7
4.2.5	Refusal.....	7
4.3	Groundwater Conditions	7
5.0	CLOSURE	7

PART B - FOUNDATION DESIGN REPORT

6.0	DISCUSSION AND ENGINEERING RECOMMENDATIONS.....	9
6.1	General.....	9
6.2	Culvert Types	9
6.3	Culvert Construction Timing	10
6.3.1	Culvert Construction Concurrent with Embankment Construction	10
6.3.2	Culvert Construction Following Embankment Surcharge Period.....	11
6.3.3	Culvert Construction Following Full Sub-Excavation of Compressible Soils	11
6.4	Stability, Settlement and Horizontal Strain.....	12
6.4.1	Stability	12
6.4.2	Settlement.....	12
6.4.2.1	Methodology	12



6.4.2.2	Parameter Selection	13
6.4.2.3	Results of Analysis	14
6.4.3	Horizontal Strain	15
6.4.3.1	Parameter Selection	15
6.4.3.2	Results of Analysis	16
6.5	Geotechnical Resistances	16
6.5.1	Geotechnical Axial Resistance.....	16
6.5.2	Resistance to Lateral Loads/Sliding Resistance	17
6.6	Lateral Earth Pressures	17
6.7	Culvert Construction Considerations	18
6.7.1	Embankment Monitoring	18
6.7.2	Excavation and Replacement below Culvert Bedding.....	18
6.7.3	Culvert Bedding and Backfill	18
6.7.3.1	Precast Culvert.....	18
6.7.3.2	Cast-in-Place Culvert.....	19
6.7.3.3	General.....	19
6.7.4	Erosion Protection.....	19
6.7.5	Control of Groundwater and Surface Water	20
6.8	Temporary Culvert.....	20
7.0	CLOSURE	21

REFERENCES

DRAWINGS

Drawing 1 Borehole Locations and Soil Strata

LIST OF APPENDICES

Appendix A Record of Boreholes

List of Symbols and Abbreviations
Record of Boreholes BC4-1 to BC4-3

Appendix B Laboratory Test Results

Figure B1	Grain Size Distribution – Clayey Silt
Figure B2	Plasticity Chart – Clayey Silt
Figure B3	Grain Size Distribution – Clay
Figure B4	Plasticity Chart – Silty Clay to Clay
Figure B5	Consolidation Test Summary, Borehole BC4-1, Sample 4
Figure B6	Consolidation Test Summary, Borehole BC4-1, Sample 5
Figure B7	Consolidation Test Summary, Borehole BC4-1, Sample 6
Figure B8	Grain Size Distribution – Silt



FOUNDATION REPORT – CULVERT BC4 – HIGHWAY 66 REALIGNMENT
GWP 5091-07-00

Figure B9
Figure B10

Grain Size Distribution – Gravelly Sand
Slope Stability – Effective Stress Analysis – 1.5 m Wick Drain Spacing – Approximate Culvert Location



PART A

FOUNDATION INVESTIGATION REPORT

CULVERT AT STATION 14+510 (BC4)

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 Corporation (MRC), a member of MMM Group Limited (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed Culvert BC4, crossing the proposed Highway 66 realignment at STA 14+510. 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; as well as a number of culverts. The proposed Culvert BC4 is located about 12.5 km east of Highway 624 within the Swamp Crossing H6/H7. The general location of the proposed Culvert BC4 is shown on the Key Plan on Drawing 1.

The Terms of Reference 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 plans showing the proposed horizontal and vertical alignment as well as the General Arrangements for the culvert were provided to Golder by MRC.

This report addresses the investigation carried out for the proposed Culvert BC4 only. Separate reports address the foundation investigations for the remaining culverts, swamp crossing/high fill areas and deep cut section.

The purpose of this investigation is to establish the subsurface conditions along the proposed culvert alignment by methods of borehole drilling, rock coring, in situ testing and laboratory testing on selected samples. The centreline of the proposed Highway 66 realignment was staked in the field by MRC and the foundation investigation was carried out at Culvert BC4, as defined in the Terms of Reference. The investigation area is shown in plan on Drawing 1.

2.0 SITE DESCRIPTION

The new Highway 66 alignment is oriented generally in an east-west direction within the Township of McGarry. The proposed culvert will be up to 35 m long extending across the proposed realigned Highway 66 at about STA 14+510. The land in the vicinity of Culvert BC4 is used for recreation.

In general, the topography in the vicinity of Culvert BC4 consists of a low-lying swamp area. The area is densely populated with trees and has open water ponded as a result of beaver dams. Multiple ATV trails are located near the proposed culvert. The ground surface within the limits of the culvert alignment varies between about Elevation 304 m and Elevation 305 m. A detailed description of the subsurface conditions along the culvert alignment is presented in Section 4.0.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The investigation for Culvert BC4 crossing the realigned Highway 66 was carried out between November 14 and 18, 2012, during which time a total of three boreholes were advanced along the proposed culvert alignment. The locations of the boreholes are shown on Drawing 1 and are provided on the Record of Borehole sheets in Appendix A.



The field investigation was carried out using portable drilling equipment supplied and operated by Landcore Drilling of Sudbury, Ontario. The portable equipment was set up on a floating raft to facilitate drilling in areas of open water. The boreholes were advanced through the overburden using 'NW' casing with wash boring techniques. In general, soil samples were obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm outer diameter (O.D.) split-spoon sampler driven by a cathead hammer, and carried out in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586, Standard Test Method for Standard Penetration Test). The portable equipment employed full-weight hammers, dropped from the standard SPT height. Samples of the cohesive soils were obtained at selected locations/depths 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. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 Wells (as amended).

A dynamic cone penetration test (DCPT) was advanced from the bottom of two of the boreholes and extended to refusal. The boreholes were advanced to depths ranging between 13.6 m and 18.7 m below existing ground surface (excluding the ponded water), including the DCPTs driven from the bottom of two of the boreholes.

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 Appendix A.

The fieldwork was observed by a member of our engineering and technical staff, who located the boreholes, 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 samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury Geotechnical Laboratory where the samples underwent further 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 were carried out on select samples of the cohesive deposit. The results of the laboratory testing on samples from the culvert boreholes are included in Appendix B.

The proposed centreline of the new highway alignment was staked in the field by MRC prior to drilling. The as-drilled borehole locations, in stations and offsets, were measured in reference to the centreline alignment and were subsequently converted into MTM NAD 83 coordinates in AutoCAD. Static water elevations at the borehole locations were surveyed at the time of drilling by a member of our technical staff in reference to the ground surface elevations at temporary benchmarks, which were installed by MRC prior to the commencement of fieldwork. The borehole locations given in the Record of Borehole sheets and shown on Drawing 1 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations and drilled depths are as follows:



Borehole	Location (MTM NAD 83)		Water/Ground Surface Elevation (m)	Borehole/DCPT Depth Below Ground Surface (m)
	Northing	Easting		
BC4-1	5334678.1	410115.3	305.2/305.0	15.5/18.7
BC4-2	5334695.8	410109.4	304.8/304.1	13.6
BC4-3	5334660.4	410121.2	305.2/304.9	14.3/15.5

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. (Baker, 1985)¹

Based on NOEGTS² 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.

Published literature indicates that the site is located in the Abitibi Subprovince of the Superior Province (OGS, 1991)³. 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 advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are presented on the attached Record of Borehole sheets and the soil laboratory test sheets provided in Appendices A and B. 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 locations.

¹ C.L. Baker, 1985. Quaternary Geology of the Kirkland Lake Area, Districts of Cochrane and Timiskaming; Ontario Geological Survey.

² Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Map Reference Number 32DSW.

³ 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.



The inferred soil stratigraphy based on the result of the boreholes is shown in profile on Drawing 1. 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 subsurface conditions encountered at the site consist of peat at the ground surface underlain by a deposit of very soft to firm clayey silt to clay. The cohesive deposit is underlain by a deposit of very loose to dense silt which is in turn underlain by a layer of gravelly sand at one borehole location and dense gravelly sand.

Detailed descriptions of the subsurface conditions along the investigated culvert alignment 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 or stratum.

4.2.1 Peat

A 0.2 m to 0.7 m thick deposit of amorphous to fibrous peat was encountered below the ponded water at between pond bottom Elevations 305.0 m to 304.1 m.

One SPT 'N'-value measured within the peat is 1 blow per 0.3 m of penetration, suggesting a very soft relative density.

The natural water content measured on one sample of the peat is 93 per cent.

4.2.2 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 to a lower zone of clayey silt was encountered underlying the peat in the three boreholes. The total thickness of the deposit ranges between 8.5 m and 9.9 m and the surface of the deposit was encountered between Elevations 304.6 m and 303.9 m.

The upper zone is comprised of layered, grey to brown clayey silt and is between 1.2 m and 2.3 m thick. The middle zone is comprised of grey silty clay to clay and is between 6.1 m and 7.3 m thick. This portion of the deposit was noted to be irregularly varved with clayey silt/clay varves between 1 mm and 25 mm thick as observed in the Shelby tube samples. The lower zone is comprised of grey, wet clayey silt and is about 1.5 m thick in the two boreholes encountered. Silt seams/interlayers (i.e., varves) were encountered within the cohesive deposit, between Elevations 297.8 m and 296.3 m in Borehole BC4-1, between Elevations 302.7 m and 300.0 m and between Elevations 298.5 m and 295.4 m in Borehole BC4-2, and between Elevations 297.7 m and 296.2 m in Borehole BC4-3.

4.2.2.1 Clayey Silt

The SPT 'N'-values measured within the clayey silt portion of the deposit range from 1 blow to 4 blows per 0.3 m of penetration. In situ field vane tests carried out within the upper portion of the deposit measured undrained shear strengths of 26 kPa and 34 kPa, and the sensitivity is calculated to be 9 and 18, respectively. The field vane tests results indicate that the clayey silt portion of the deposit has a firm consistency.



The natural water content measured on three samples of this portion of the deposit range from about 26 per cent to 33 per cent.

A grain size distribution test completed on one sample of the clayey silt deposit is shown on Figure B1 in Appendix B.

Atterberg limits test were carried out on three samples of the upper clayey silt portion of the deposit and measured liquid limits ranging from about 28 per cent to 31 per cent, plastic limits of about 18 per cent and plasticity indices ranging from about 11 per cent to 13 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 a clayey silt of low plasticity.

4.2.2.2 Silty Clay to Clay

The SPT 'N'-values measured within the silty clay to clay portion of the deposit range are 0 blows (weight of hammer) or 1 blow per 0.3 m of penetration. In situ field vane tests carried out within this portion of the deposit measured undrained shear strengths ranging between 20 kPa and 39 kPa, and the sensitivity is calculated to range from 3 to 13. The field vane tests results indicate that the silty clay to clay portion of the deposit has a soft to firm consistency.

The natural water content measured on eight selected samples of this portion of the deposit ranges from about 45 per cent to 70 per cent.

The results of grain size distribution tests completed on three samples of the silty clay to clay portion of the deposit are shown on Figure B3 in Appendix B.

Atterberg limits test were carried out on eight samples of the silty clay to clay portion of the deposit and measured liquid limits ranging from about 37 per cent to 57 per cent, plastic limits ranging from about 20 per cent to 25 per cent and plasticity indices ranging from about 17 per cent to 32 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B4 in Appendix B and indicate that the material is classified as a silty clay of intermediate plasticity to clay of high plasticity.

Laboratory consolidation tests were carried out on three specimens of the silty clay to clay deposit obtained from Shelby tube samples in Borehole BC4-1. Preconsolidation pressures ranging between 92 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 16.1 kN/m³ to 16.6 kN/m³ and a specific gravity between about 2.74 and 2.76 were measured on the consolidation test specimens. Details of the test results are shown on Figure B5 to B7 in Appendix B, 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 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}

*For stress range between approximately effective overburden stress and final stress due to 3 m high embankment, that is $30 \text{ kPa} \leq \sigma_v' \leq 140 \text{ kPa}$

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

4.2.2.3 Clayey Silt

The SPT 'N'-values recorded within the lower clayey silt portion of the deposit are 1 blow per 0.3 m of penetration. One in situ field vane test carried out within the lower portion of the deposit measured an undrained shear strength of 37 kPa, and the sensitivity is calculated to be 7. The field vane test result indicates that the clayey silt deposit has a firm consistency.

The natural water content measured on one sample of this deposit is 36 per cent.

An Atterberg limits test carried out on one sample of the clayey silt deposit measured a liquid limit of about 29 per cent, the plastic limit of about 20 per cent and a plasticity index of about 9 per cent. The results of the Atterberg limits test are shown on the plasticity chart on Figure B2 in Appendix B and indicate the material is classified as a clayey silt of low plasticity.

4.2.3 Silt

A deposit of grey, wet silt, 4.1 m to 4.9 m thick was encountered underlying the cohesive deposit in all the boreholes. The surface of this deposit was encountered between Elevation 295.4 m and 294.7 m.

The SPT 'N'-values measured within the silt deposit range from 4 blows to 35 blows per 0.3 m of penetration, indicating a very loose to dense relative density. Borehole BC4-2 was terminated within this deposit on refusal to further split spoon and casing advancement.

The natural water content measured on three samples of this deposit ranges from about 27 per cent to 29 per cent.

The results of grain size distribution tests completed on three samples of the silt deposit are shown on Figure B8 in Appendix B. An Atterberg limits test on one sample of the silt deposit indicates this material to be non-plastic.



4.2.4 Gravelly Sand

A 0.9 m thick deposit of grey, wet gravelly sand, some silt, trace clay was encountered below the silt deposit in Borehole BC4-1. The surface of this deposit was encountered at Elevation 290.4 m.

One SPT 'N'-value measured within the gravelly sand deposit is 40 blows per 0.3 m of penetration, indicating a dense relative density.

The natural water content measured on one sample of this deposit is about 10 per cent.

A grain size distribution test completed on one sample of the gravelly sand deposit is shown on Figure B9 in Appendix B.

4.2.5 Refusal

Dynamic Cone Penetration Tests (DCPTs) were advanced from the bottom of Boreholes BC4-1 and BC4-3. The bedrock surface at the boreholes and DCPTs is inferred by refusal to further split-spoon, casing advancement or dynamic cone penetration between depths of about 13.6 m to 18.7 m below the ground surface, corresponding to between Elevation 290.5 m and 286.3 m.

4.3 Groundwater Conditions

Groundwater levels were measured in the open boreholes during and upon completion of drilling. Between 0.2 m and 0.7 m of standing water was encountered over the ground surface in the boreholes.

The ponded water elevation as encountered at the borehole locations may not be representative of groundwater levels and both the surface water level and groundwater elevations will vary depending on seasonal fluctuations, precipitation, local soil permeability, and, in the case of the ponded water, on the condition of the beaver dams.

5.0 CLOSURE

The drilling program was supervised by Mr. Gabriel Mathieu. This report was prepared by Mr. Matt Thibeault, EIT and reviewed by Ms. Sarah 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 Principal with Golder, conducted an independent quality control review of the report.



Report Signature Page

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PART B

FOUNDATION DESIGN REPORT

CULVERT AT STATION 14+510 (BC4)

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 was retained by MRC to provide foundation engineering services for the design of the proposed up to 35 m long Culvert BC4, to be constructed across the proposed Highway 66 realignment at STA 14+510. The proposed culvert inlet (south side) and outlet (north side) are at Elevations 305.1 m and 304.6 m and the proposed embankment at the culvert area is up to 3.0 m high.

This report presents an assessment of the stability and settlement of the embankment at the culvert location and geotechnical resistances for design of the culvert. It provides recommendations for stable embankment geometry and embankment fill materials, including implementation of mitigation alternatives that may be required as a means to reduce culvert settlements and to improve embankment stability (if necessary). The report also provides recommendations to addresses potential construction concerns and geotechnical problems associated with culvert and embankment construction, sub-excavating soft/organic materials and placement of new fill materials.

The culvert is located within a swamp crossing/high fill area (designated as Swamp Crossing H6/H7), as contained in the following report (MTO, 2013):

- *Foundation Investigation and Design Report, Swamp Crossings/High Fill Areas and Deep Cut, Realignment of Highway 66 at Virginiatown from 10.6 km east of Highway 624 easterly 3.4 km, GWP 5091-07-00, by Golder Associates Ltd.*

Due to the presence of an extensive deposit of cohesive soil below the proposed Highway 66 embankment at this site, the preferred stability and settlement mitigation for this swamp crossing area, including the culvert location, consists of the installation of wick drains spaced at 1.5 m intervals, staged embankment construction including a surcharge period of 44 days, and a 5 m wide toe berm tapered to accommodate the culvert inlet/outlet ends. Further, an excess material management area will be located to the south of the swamp crossing/high fill area, with at least a 5 m wide buffer zone extending from the limits of sub-excavation to the proposed 5 m toe berm.

6.2 Culvert Types

The analyses and recommendations presented herein are based on Culvert BC4 at STA 14+510 being a concrete box culvert having a width of 1.8 m and a height of 1.2 m and a temporary culvert consisting of a 1.2 m diameter circular concrete pipe during the surcharge period, as discussed below.



6.3 Culvert Construction Timing

In general, the foundation soils at the culvert crossing will undergo settlement as a result of loading from the new overlying embankment. Therefore, the timing of culvert construction is an essential factor in determining the preferred settlement mitigation option. The following alternatives for culvert construction can be considered (where applicable, giving due consideration to the recommended foundation mitigation option for the embankment crossing the swamp in the culvert area):

- concurrent with embankment construction;
- following the embankment surcharge period; and
- following full sub-excavation of compressible deposits along the culvert alignment and concurrent with embankment construction.

In areas of culvert construction where relatively small settlements are estimated to occur due to the presence of relatively thin, compressible foundation soils, culvert construction can be carried out concurrently with the proposed new embankment construction provided that any requirements for maintaining embankment stability are implemented. If required, the culvert design could include a camber.

Where relatively large settlements are estimated to occur, as is the case at this site, it is recommended that the culvert be constructed subsequent to the embankment surcharge period or after sub-excavation of cohesive deposits, to reduce settlement and provide adequate long-term performance of the culvert and the associated overlying roadway. The following sections provide a more detailed discussion on the alternatives for culvert construction and measures to mitigate settlements and improve long-term performance of the culvert.

6.3.1 Culvert Construction Concurrent with Embankment Construction

A culvert that is constructed concurrently with the new embankment will experience settlement (both short-term and long-term), as well as lateral spreading (or horizontal strain in the longitudinal direction) as a result of the embankment loading. If the culvert structure is capable of tolerating the estimated total and differential settlements and associated strains, the culvert could be constructed with a camber (if necessary), such that once the settlement has occurred, the hydraulic flow will be maintained as originally designed. However, for a culvert designed to include a camber there is a relatively high risk of poor performance resulting in unfavourable drainage/surface water flow conditions at some locations. It is important to note that it is inherently difficult to predict settlements for the variable subsurface conditions along the culvert alignment with such a degree of accuracy to allow an accurate camber design. If the actual settlements are smaller than predicted, the culvert may not achieve the design grade or slope, which could impede the flow of water. If actual settlements are larger than expected, the culvert may sag below the design invert elevation and as a result some sediments may be deposited inside the culvert and could reduce the flow of water. Expansion joints may also be included along the length of the culvert to accommodate horizontal strain which will occur in conjunction with the vertical settlement. The analyses of settlement and horizontal strain at Culvert BC4 are discussed in Section 6.4.2 and Section 6.4.3, respectively.

Sub-excavation of all existing organic material is required prior to placement of any fill or culvert bedding material, as organic soils are highly compressible and can undergo significant secondary (creep) settlement.



6.3.2 Culvert Construction Following Embankment Surcharge Period

At locations where the magnitudes of estimated total and differential settlements and horizontal strains cannot be tolerated by a culvert and/or where removal of localized cohesive deposits and replacement with granular fill is not considered practical, as is the case at this site, the culvert should be constructed following a surcharge period in conjunction with the wick drain foundation design for the overall roadway embankment. Surcharging refers to the placement of fill beyond the proposed height of embankment (possibly in stages), in advance of construction of the permanent culvert, in order to consolidate the underlying compressible soils. If surcharging of the embankment at the culvert location is completed prior to construction of the permanent culvert, the magnitude of total and differential settlement beneath the permanent culvert and the horizontal strain along the culvert will be reduced. However, this mitigation option requires excavating through the new embankment fill to the culvert founding elevation at the end of the surcharge period in order to construct the culvert. Provided that the final fill above the culvert is properly placed and compacted, the magnitude of differential settlement between the fill embankment (that has been compressed under its self-weight in conjunction with wick drain foundation design for the entire surcharge period) and the final backfill above the culvert should be acceptable.

6.3.3 Culvert Construction Following Full Sub-Excavation of Compressible Soils

Depending on the depth and thickness of any soft, compressible foundation deposit(s), the magnitude of total and differential settlement and horizontal strain could also be reduced by means of full sub-excavation of these soils and replacement along the culvert alignment to allow for permanent culvert construction prior to embankment loading (i.e., concurrent with embankment construction). At culvert locations where the compressible deposits are thick, the resulting magnitude of settlements as well as the associated horizontal strains, even with full sub-excavation, may still be too large as a result of compression of the underlying fill itself, to accommodate standard culvert construction. However, where there is a limited thickness and depth of soft, compressible soils underlying the proposed culvert, full sub-excavation and replacement is a feasible option to reduce the settlement and allow for culvert construction in conjunction with the construction of the new embankment.

Although full sub-excavation will improve the settlement performance of the culvert and embankment in close proximity of the sub-excavation, adjacent areas of the embankment may not experience the same improvements in settlement performance depending on the mitigation measures adopted for the adjacent embankment swamp crossing. As a result, the overlying embankment may experience some differential settlements along its alignment depending on the timing of embankment construction/culvert construction, type of backfill and timing of final earthworks and paving.

It should also be noted that settlement of replacement fill beneath a culvert base will occur, primarily during construction, and could constitute a significant portion of the expected settlements, depending on the depth of sub-excavation and replacement required.

As discussed in Section 6.1, due to the presence of an extensive deposit of cohesive soil below the proposed Highway 66 embankment in this area, wick drains, coupled with a surcharge period and toe berms, will be required for this culvert site. Further, the presence of the excess material management area to the south of the embankment will make full sub-excavation impractical. Full sub-excavation of the cohesive deposit is not considered further in this report.



6.4 Stability, Settlement and Horizontal Strain

The following sections summarize the methods utilized to carry out analyses of embankment stability and settlement of the culvert and methods utilized to evaluate horizontal strains along the culvert beneath the zone of influence of the proposed embankment loading.

6.4.1 Stability

The stability analysis carried out for the proposed embankment during and following the staged embankment construction and surcharge period associated with the wick drain foundation design (consistent with the foundation mitigation recommendations for Swamp Crossing H6/H7), using the commercially available program Slide (Version 6.0), produced by Rocscience Inc., indicates that during and after completion of construction, the embankment at the culvert location will have a Factor of Safety (FoS) greater than 1.3 for deep-seated, global failure surfaces that would impact the operation of the roadway. Figure B10 presents the results of the stability analysis in the vicinity of Culvert BC4 at the beginning of the surcharge period after the installation of the wick drains and the temporary culvert. It should be noted, however, that the FoS will only be greater than or equal to 1.3 so long as:

- the rate of embankment construction at the culvert location is as recommended in MTO (2013) for construction of the Swamp Crossing H6/H7 embankment, which consists of the installation of wick drains, four stages of embankment construction to be completed in 221 days and a 44 day surcharge period; and
- toe berms are incorporated into the embankment cross-section, to be constructed to a width of 5 m and a height of 1.5 m, with the toe berms adjoining the permanent culvert tapered immediately from both sides at the ends of the culvert.

Monitoring of stability of the embankment should be carried out as per Section 6.7.1.

6.4.2 Settlement

The following sections outline the methods used to conduct the settlement analyses at the culvert location and the results of the analyses.

6.4.2.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out along the culvert alignment using the commercially available program Settle3D (Version 2.013) produced by Rocscience Inc. The rate of settlement/consolidation of the cohesive foundation soils was assessed using Terzaghi's one-dimensional consolidation theory.

The sources of settlement at this site are:

- primary time-dependent consolidation of the cohesive deposits;
- secondary time-dependent (creep) consolidation of the cohesive deposits (long-term);
- immediate settlement of the native granular soils; and



- self-weight compression of the embankment fill materials beneath the culvert (where applicable).

The thickness of the native cohesive and granular foundation soils and the height of the embankment vary along the proposed culvert alignment and therefore the settlements along the length of the culvert will similarly vary. As such, settlements have been assessed at the culvert inlet, mid-point (i.e., highway centreline median) and outlet.

The settlement analyses assume that all organic soils (i.e., peat) beneath the culvert alignment will be removed prior to construction and that granular fill will be used for replacement of sub-excavated material (as discussed in Section 6.7.1). The piezometric condition required in the analyses is based on the groundwater level at about the level of the natural ground surface.

6.4.2.2 Parameter Selection

The immediate compression of the very loose to dense silt deposit and the dense gravelly sand deposit below the cohesive soils was modeled 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 moduli values were compared with the typical range of expected values for similar soil types, as outlined in Canadian Highway Bridge Design Code and Commentary, (CHBDC, 2006) and adjusted, if necessary.

The consolidation settlement of the cohesive deposits was assessed using the results of the laboratory index tests, in situ field vane tests in the boreholes and CPTs in the area of the culvert to estimate the stress history and deformation parameters for the cohesive deposits at the culvert location. Estimates of deformation parameters (i.e., recompression and compression indices) were obtained 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 Koppula (1986) relating the natural water content and liquid limit to the compression index and Azzouz et al. (1976) relating the void ratio to the compression index were found to be the most consistent with the results of laboratory consolidation tests for the clayey soils in Swamp Crossing H6/H7 (including the tests from Borehole BC4-1), and as such were used to represent the deformation properties at this location.

The following correlation relating in situ undrained shear strength to preconsolidation stress proposed by Mesri (1975) was employed:

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

where:

$$\begin{aligned} S_{u(mob)} &= \mu S_{u(FV)} \\ \sigma_p' &= \text{preconsolidation stress (kPa)} \\ S_{u(mob)} &= \text{average mobilized undrained shear strength (kPa)} \\ S_{u(FV)} &= \text{undrained shear strength from field vane test (kPa)} \\ \mu &= \text{Bjerrum's (1973) correction factor based on Plasticity Index} \end{aligned}$$

The preconsolidation stress was also estimated from the results of the CPTs (Demer and Leroueil, 2002):

$$\begin{aligned} \sigma_p' &= \frac{q_t - \sigma_{vo}}{3.4} \\ \text{where: } q_t &= q_c - u_2(1 - A_n) \text{ (kPa)} \\ q_c &= \text{tip stress measured by the CPT (kPa)} \\ u_2 &= \text{pore pressure measured at cone 'shoulder' (kPa)} \\ A_n &= \text{cone constant} \\ \sigma_{vo} &= \text{total vertical stress (kPa)} \end{aligned}$$



As the culvert site is located in Swamp Crossing H6/H7 where wick drains will be installed, 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, as discussed in MTO (2013).

In addition to primary consolidation within cohesive deposits, secondary compression will 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 the culvert location.

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

where:

- S_c = secondary consolidation (creep) settlement (mm)
- $C_{\alpha\epsilon}$ = modified secondary compression index as estimated from laboratory consolidation tests and/or from the empirical correlation by Mesri (1973)
- H = initial thickness of normally consolidated portion of compressible clay deposit (mm)
- t = post-construction period of interest (20 years)
- t_{EOP} = time to reach end of primary consolidation (years)

The values of modified secondary compression index ($C_{\alpha\epsilon}$) estimated from the empirical correlation were compared with the values of $C_{\alpha\epsilon}$ calculated from the results of the laboratory consolidation tests, where necessary.

The simplified stratigraphy together with the associated strength and unit weight values assigned to the different native soil types at the culvert location are summarized below, as generally presented in MTO 2013.

Stratigraphic Unit	γ' (kN/m ³)	σ'_p (kPa)	e_o	C_c	C_r	E' (MPa)	c_h (cm ² /s)
Peat*	12	-	-	-	-	-	-
Clayey Silt (Upper)	17	64	0.8	0.5	0.025	2	2.0×10^{-2}
Silty Clay to Clay	16.5	64 – 93	1.7	1.0	0.050	2	5.0×10^{-3}
Clayey Silt (Lower)	16.5	93 – 160	1.0	0.5	0.025	2 – 7	2.0×10^{-2}
Silt	18	-	-	-	-	3	-
Gravelly Sand	20	-	-	-	-	50	-

Note: * The peat is to be removed prior to culvert/embankment construction.

6.4.2.3 Results of Analysis

Given that the proposed culvert location is within High Fill/Swamp Crossing H6/H7 and therefore subjected to the same settlement issues for the embankment crossing the swamp area, it is recommended that the settlement mitigation recommendations for the proposed culvert location be consistent with the foundation



mitigation recommendations for the new embankment construction (i.e., staged embankment construction, including surcharging and wick drains).

Due to the relatively thick cohesive deposit at this site and the staged embankment construction with wick drains spaced at 1.5 m, a temporary culvert (or potentially two temporary culverts, depending on the strain tolerance as discussed in Section 6.4.3.2) should be installed concurrent with embankment construction and left in place during the construction/surcharge period (221 days after wick drain installation) and replaced with a permanent culvert after surcharge removal.

Based on the results of the settlement analyses, the estimated settlement along the temporary culvert alignment for an embankment construction/surcharge period for 221 days is presented below.

	Location	Total Estimated Settlement During Construction* to End of Surcharge Period (mm)
Temporary Culvert Installed concurrent with Embankment Construction*	Inlet (south toe of stability berm)	50
	Near Midpoint	770
	Outlet (north toe of stability berm)	60

Notes: * Includes immediate settlement of native cohesionless soils.

Monitoring of settlement of the embankment should be carried out as per Section 6.7.1.

The settlement of the permanent culvert installed following the minimum recommended surcharge period will be negligible (i.e., less than 5 mm).

6.4.3 Horizontal Strain

The following sections outline the methods used to estimate the horizontal strain along the culvert and the results of the analysis.

6.4.3.1 Parameter Selection

As a result of the two-dimensional nature of the proposed embankment geometry, shear stresses will be mobilized in the foundation soils upon completion of preload embankment construction and during the surcharge period causing lateral spreading of the foundation soils and new embankment fill. This, in conjunction with the non-uniform vertical settlement of the foundation soils along the proposed culvert alignment, will generate horizontal straining along newly constructed temporary or permanent culverts, as applicable. In order to maintain structural integrity of the culvert, the culvert design must incorporate a suitable allowance for extension at the joints/couplings of the culvert segments, as applicable, to prevent the culvert from cracking and/or failing in tension.



The research work by Rutledge and Gould (1973) on the movements on articulated conduits under earth dams on compressible foundations can be used to estimate the magnitude of the horizontal strain likely to occur as a result of the proposed embankment construction at culvert sites. The following equations have been used to obtain a relationship between vertical settlement, vertical strain, horizontal strain and maximum joint opening as a result of settlement of the foundation soils:

$$\begin{aligned}\varepsilon_v &= \frac{\delta_v}{d} \\ \varepsilon_h &= \varepsilon_v \frac{\varepsilon_h}{\varepsilon_v} \\ \Delta L &= \varepsilon_h L \\ \text{where : } \Delta L &= \text{maximum joint opening (m)} \\ \varepsilon_v &= \text{maximum vertical strain} \\ \varepsilon_h &= \text{maximum horizontal strain} \\ \frac{\varepsilon_h}{\varepsilon_v} &= \text{estimated ratio of maximum horizontal strain to maximum vertical strain} \\ &\quad \text{from Figure 2 in Rutledge and Gould (1973)} \\ L &= \text{length of culvert (m)} \\ \delta_v &= \text{maximum vertical settlement of culvert as a result of immediate and} \\ &\quad \text{post-construction settlement of foundation soils and granular fill/bedding} \\ &\quad \text{material (m)} \\ d &= \text{thickness of compressible foundation deposits at culvert location (m)}\end{aligned}$$

6.4.3.2 Results of Analysis

The settlement analyses indicate that over the embankment construction/surcharge period of 220 days, the 5 m high surcharge embankment (i.e., a 2 m surcharge) and temporary culvert would undergo settlements between 50 mm and 770 mm. The differential settlement along the temporary culvert is significant and should be taken into consideration by the Contractor while the temporary culvert is in place. To mitigate the relatively large differential settlement, consideration should be given to:

- the use of a relatively flexible temporary culvert;
- the use of a non-flexible culvert incorporating a camber and expansion joints; or
- replacing the culvert mid-way through the embankment construction or surcharge period.

The horizontal strain will be significant if mitigation measures are not carried out. As noted above, through the use of temporary culverts during the surcharge period, the resulting post-construction settlement of the foundation soils along the permanent culvert will be negligible (less than 5 mm) and therefore the horizontal strain for a 29 m long permanent culvert will also be negligible.

6.5 Geotechnical Resistances

6.5.1 Geotechnical Axial Resistance

For the 1.8 m wide permanent box culvert proposed for this site, the permanent culvert should be designed on the basis of a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 150 kPa and a geotechnical reaction at Serviceability Limit States (SLS) of 75 kPa (for 25 mm of settlement) based on the permanent culvert being constructed after the period and being founded on a properly prepared subgrade/granular bedding (as discussed in Section 6.7.2). The geotechnical resistances are applicable for



loads that will be applied perpendicular to the base of the culvert. Where loads are not applied perpendicular to the base of the culvert, inclination of the loads should be taken into account in accordance with Section 6.7.4 and Section C6.7.4 of the Canadian Highway Bridge Design Code (CHBDC) and its Commentary.

The loading on the foundation soils below the culvert and the associated total settlement at the culvert location will be governed by the design height of the overlying and adjacent embankment fills. As such, it is recommended that the structural engineer exercise caution when utilizing the values of the geotechnical axial resistance at SLS in the design of the culvert. Where the culvert is constructed following completion of all foundation soil settlement resulting from the construction of embankment fills and surcharge, the SLS value as provided may be used for the culvert design for settlement of 25 mm.

6.5.2 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the base of the box culvert and the granular fill/bedding placed following sub-excavation of organic deposits should be calculated in accordance with Section 6.7.5 of the CHBDC. The following summarizes the coefficient of friction for the interface materials.

Interface Materials	Coefficient of Friction
Precast Concrete Box Culvert on Compacted Granular 'A'/Granular 'B' Type II	$\tan \phi = 0.45$
Cast-in-Place Concrete Box Culvert on Compacted Granular 'A'/Granular 'B' Type II	$\tan \phi = 0.55$

These values represent unfactored values.

6.6 Lateral Earth Pressures

The lateral earth pressures acting on the walls of the culvert will depend on the type and method of placement of backfill materials, the nature of the soils/embankment fill behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls of the culvert.

The following recommendations are made concerning the design of the culvert walls. 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.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II but with less than 5 per cent passing the No. 200 (0.075 mm) sieve should be used as backfill behind the culvert walls, and on top of the culvert for a thickness of up to 300 mm. Backfill should be placed in a maximum of 200 mm loose lift thickness and nominally compacted. Weep holes should be installed in the walls to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 501 (Compacting) as amended by SP 105S21 (Compacting).



- For a box culvert, granular fill (where utilized) should be placed in a zone with the width up to 300 mm behind the back of the culvert. The pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used:

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

If the culvert structure allows for lateral yielding, active earth pressures may be used in the foundation design. If the culvert structure does not allow lateral yielding, at-rest earth pressures should be assumed for foundation design. The movement to allow active pressures to develop within the backfill, and thereby assume a restrained structure, may be taken as per Table C6.6 of the Commentary to the CHBDC.

6.7 Culvert Construction Considerations

6.7.1 Embankment Monitoring

As discussed in MTO (2013), a monitoring program has been specified for the temporary culvert at this location consisting of two nail pins and three settlement plates to monitor settlement of the Swamp Crossing H6/H7 embankment. An adjacent monitoring section, 20 m east of the culvert, also consists of four vibrating wire piezometers to monitor excess pore pressure for rate of embankment construction.

6.7.2 Excavation and Replacement below Culvert Bedding

Prior to the placement of any bedding material or granular fill, all organic soils should be stripped from the plan limits of the proposed works. Given the design invert elevations of the proposed culvert (Elevations 304.8 m and 304.7 m at the inlet and outlet, respectively) and that the organic materials (i.e., peat) will be removed, the excavation will extend to at least 0.7 m below existing ground surface (i.e., below bottom of ponded water). Granular fill (i.e., Granular 'B' Type I) will be used to backfill the excavation of organic material for wick drain treatment.

All excavations should be carried out in accordance with OPSS 902 (Excavating and Backfilling – Structures) and must be carried out in accordance with Ontario Regulation 213 Ontario Occupational Health and Safety Act for Construction Projects (as amended).

6.7.3 Culvert Bedding and Backfill

6.7.3.1 Precast Culvert

The bedding, levelling pad and granular backfill requirements for a box culvert should be in accordance with OPSS 422 (Precast Reinforced Concrete Box Culverts) and OPSD 803.010 (Backfill and Cover for Concrete Culverts with Spans Less than or Equal to 3 m) and the thickness of the bedding below the levelling course should be a minimum of 500 mm. The bedding should be placed in lifts not exceeding 200 mm in loose



thickness, and compacted to at least 98 percent of the Standard Proctor maximum dry density of the material as specified in OPSS 501/SP 105S21 (Compacting). In addition, a minimum 75 mm thick uncompacted levelling pad consisting of OPSS.PROV 1010 Granular 'A' material or concrete fine aggregate (meeting the grading requirements specified in OPSS.PROV 1002 (Aggregates - Concrete)) should be provided as shown on OPSS 803.010 for culvert construction in dry conditions. Alternatively, the bedding material can be placed in wet conditions. In this regard, a 500 mm thick layer of Granular 'B' Type II material should be placed as bedding and partial frost protection and nominally compacted by the construction equipment.

6.7.3.2 Cast-in-Place Culvert

Should a cast-in-place culvert be preferred, the bedding and backfill requirements should be in accordance with OPSS 902 (Excavating and Backfilling – Structures). The box culvert should be provided with at least 300 mm of OPSS.PROV 1010 (Aggregates) Granular 'A' or 'B' Type II for bedding purposes and partial frost protection. The bedding should be placed in lifts not exceeding 200 mm in loose thickness, and compacted to at least 98 per cent of the Standard Proctor maximum dry density of the material as specified in OPSS 501/SP 105S21 (Compacting).

6.7.3.3 General

Backfill behind the culvert walls, should consist of granular fill meeting the specifications for OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II, but with less than 5 percent passing the No. 200 (0.075 mm) sieve. The granular backfill should be placed and compacted in accordance with OPSS 501/SP 105S21 (Compacting). The fill should also be placed concurrently on both sides of the culvert walls, ensuring that the backfill depth on one side does not exceed the other side by more than 400 mm.

Where a temporary culvert is incorporated into the works and is subsequently removed after the surcharge period, the backfill above the permanent culvert should consist of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II to minimize differential settlements along the highway embankment in the area of the permanent culvert.

The culvert should be designed for the full overburden stress and appropriate live loads, assuming a fill unit weight of 22 kN/m³ for Granular 'A' and 21 kN/m³ for Granular 'B' Type II placed above and surrounding the culvert.

Inspection and field density testing should be carried out by qualified geotechnical personnel during all engineered fill placement operations to ensure that appropriate materials are used, and that adequate levels of compaction have been achieved.

6.7.4 Erosion Protection

If the culvert is placed on a granular blanket, provisions should be made for scour and erosion protection (suitable non-woven geotextiles and/or rip-rap) at the culvert location. In order to prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a concrete cut-off wall or clay seal should be provided at the upstream end of the culvert. If a clay seal is adopted, the clay



material should meet the requirements of OPSS 1205 (Clay Seal), and the seal should be a minimum 1 m thick if constructed of natural clay or soil-bentonite mix and extend from a depth of 1 m below the scour level to a minimum horizontal distance of 2 m on either side of the culvert inlet opening, and a minimum vertical height equivalent to the high water level, including along the embankment slope. Alternatively, a 0.6 m thick clay blanket (if constructed of natural clay or a soil-bentonite mix) may be constructed, extending upstream three times the culvert height and along the adjacent slopes to a height of two times the culvert height or the high water level, whichever is greater.

The requirements for and design of erosion protection measures for the inlet and outlet of the culvert should be assessed by the hydraulics design engineer. As a minimum, rip-rap treatment for the outlet of the culvert should be consistent with OPSD 810.010 (Rip-Rap Treatment for Sewer and Culvert Outlets). Erosion protection for the inlet of the culverts should follow the standard presented in OPSD 810.010 similar to the outlet. Rip-rap should be provided over the full extent of the clay blanket, including the side slopes and fill slope over the culvert.

6.7.5 Control of Groundwater and Surface Water

Excavation within the plan limits of the proposed culvert alignment will be required to remove organic (peat) overburden prior to placement of backfill, bedding material and the actual culvert structure. As a result of the excavation, groundwater flow into the excavation can be expected to occur due to the relatively high water levels and potential ponded water conditions. Therefore, control of surface water and groundwater will be necessary at the culvert location to allow for construction to be carried out in dry conditions, as required.

Groundwater control may be required as the foundation excavations to the invert level are expected to extend below the groundwater level. Excavations will be advanced through cohesive and granular soils; however, seepage into the excavation should be adequately controlled by pumping from properly filtered sumps. Dewatering of all excavation should be carried out in accordance with OPSS 517 Dewatering. Surface water should be directed away from the excavations areas to prevent ponding of water.

6.8 Temporary Culvert

A temporary culvert will likely be required to promote drainage through the embankment fills during the surcharge period. The temporary culvert may consist of a precast concrete culvert (box or pipe) or corrugated steel pipe (CSP). Bedding recommendations should be in accordance with the corresponding OPSS and/or OPSD depending on the type of the temporary culvert chosen. Assuming the temporary culvert is a CSP, construction of this culvert should be in accordance with OPSD 802.010 (Flexible Pipe Embedment and Backfill, Earth Excavation).

The location of the temporary culvert could be offset from the actual alignment of the permanent culvert, provided that surface drainage paths are adequate. It is generally recommended that a temporary culvert be constructed within a temporary granular core (Granular 'B' Type II) for ease of removal after the completion of the surcharge period. Due to the potential size of the temporary culvert, it is recommended that the culvert be removed following the permanent culvert construction. If it is not desirable to remove the temporary culvert, consideration could be given to backfilling the temporary culvert with OPSS 1359 (Ushrinkable Fill) material.



7.0 CLOSURE

This report was prepared by Mr. Matt Thibeault, EIT and Mr. André Bom, P.Eng. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, carried out a quality control review and reviewed the technical aspects 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

Commercial Software:

Settle3D (Version 2.013) by Rocscience Inc.

Slide (Version 6.0) by Rocscience Inc.

Contract Design Estimating and Documentation (CDED):

Special Provision 105S21 Amendment to OPSS 501 – Compacting

Ontario Occupational Health and Safety Act:

Ontario Regulation 213 Construction Projects (as amended)

Ontario Provincial Standard Drawing:

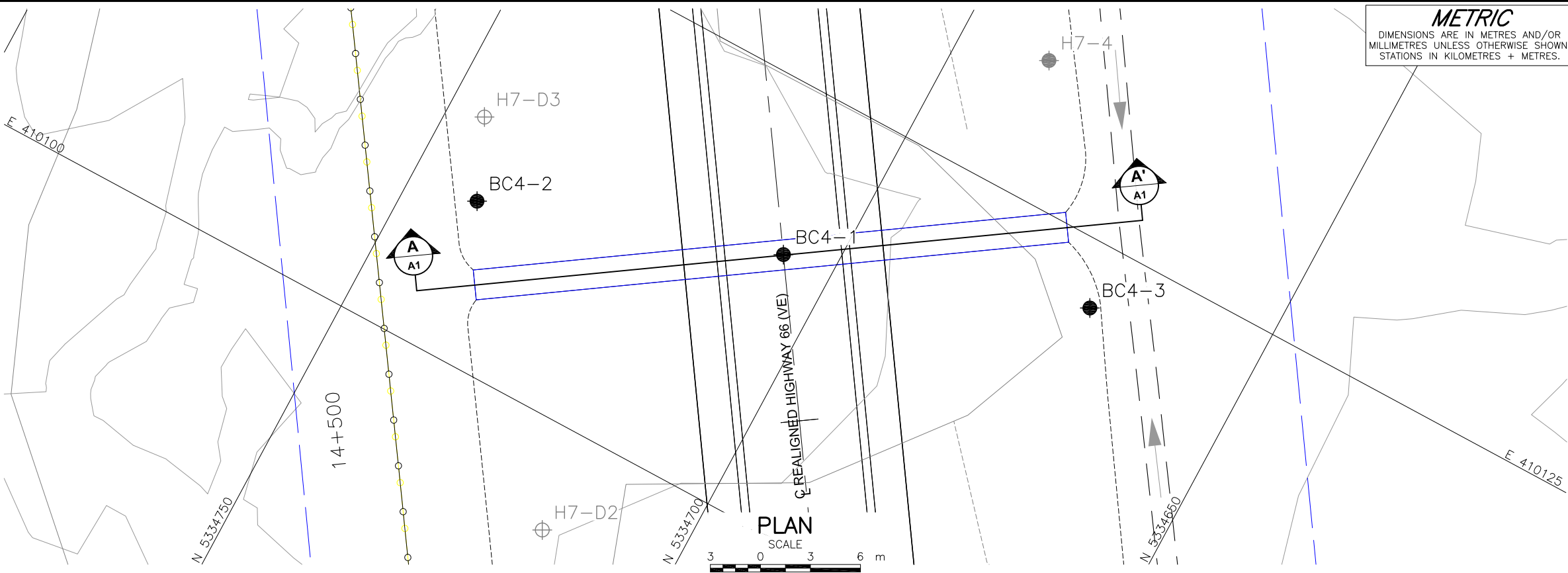
OPSD 802.010	Flexible Pipe Embankment and Backfill, Earth Excavation
OPSD 803.010	Backfill and Cover for Concrete Culverts with Spans Less Than or Equal to 3.0m
OPSD 810.010	Rip-Rap Treatment for Sewer and Culvert Outlets

Ontario Provincial Standard Specification:

OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut
OPSS 501	Construction Specification for Compacting
OPSS 902	Construction Specification for Excavating and Backfilling – Structures
OPSS 1205	Material Specification for Clay Seal
OPSS 1359	Material Specification for Unshrinkable Backfill
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
OPSS.PROV 1002	Material Specification for Aggregates – Concrete

Ontario Water Resources Act:

Ontario Regulation 903 Wells (as amended)



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

CONT No.
GWP No. 5091-07-00

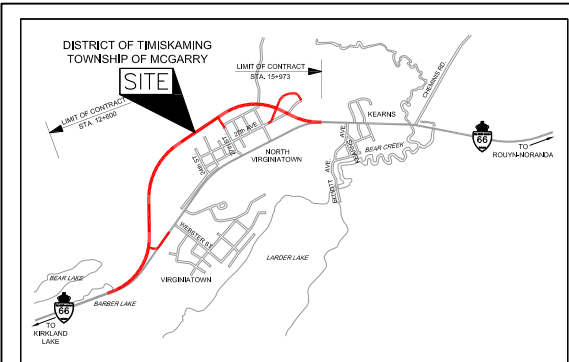


HIGHWAY 66
CULVERT AT STA 14+510
BOREHOLE LOCATIONS AND
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



KEY PLAN
SCALE
700 0 700 m

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

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
BC4-1	305.2	5334678.1	410115.3
BC4-2	304.8	5334695.8	410109.4
BC4-3	305.2	5334660.4	410121.2

NOTES

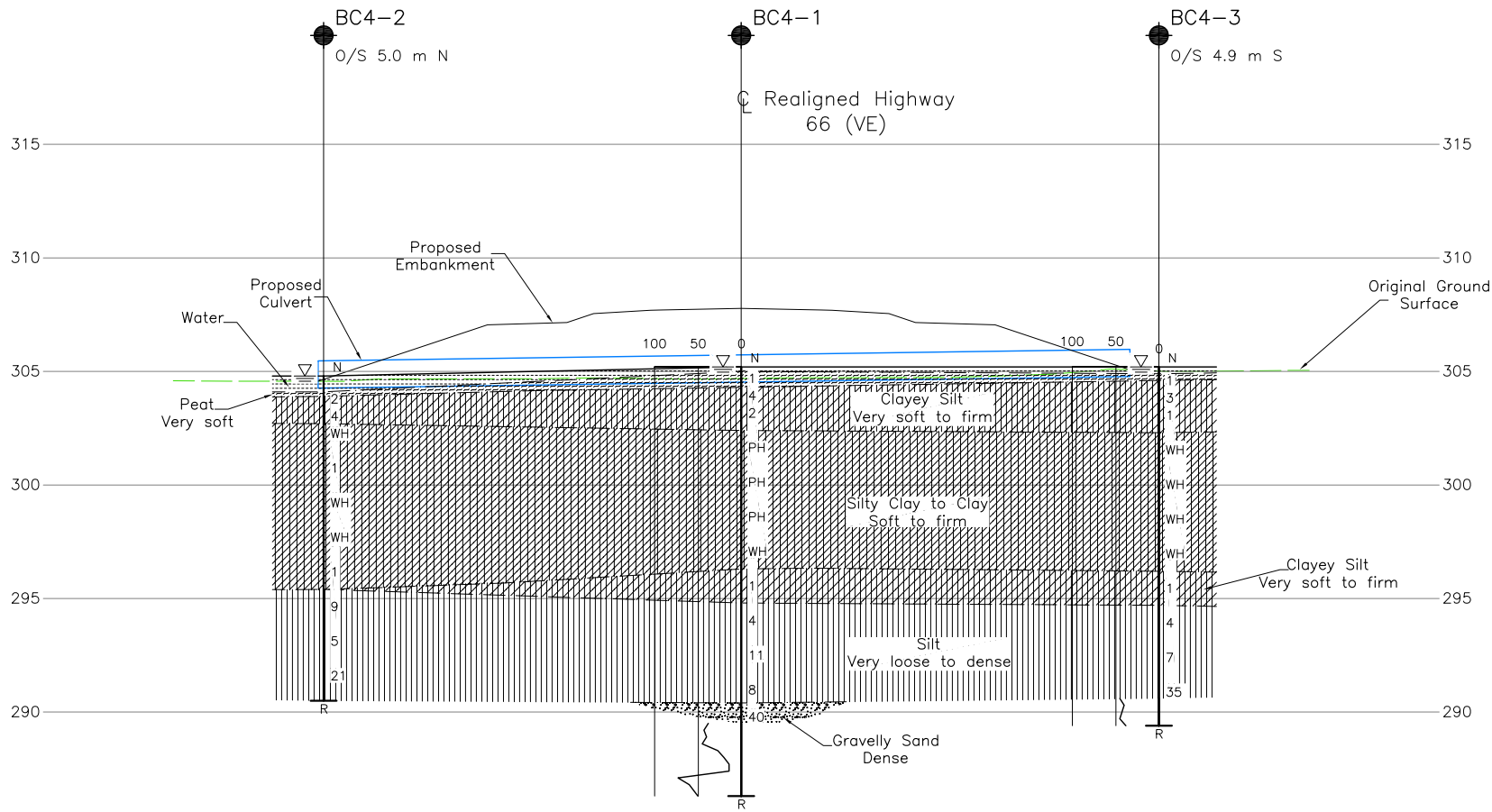
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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.



CULVERT AT STA 14+510
HIGHWAY 66
HORIZONTAL SCALE
3 0 3 6 m
VERTICAL SCALE
3 0 3 6 m



NO.	DATE	BY	REVISION
Geocres No. 32D-18			
HWY. 66		PROJECT NO. 10-1191-0044	DIST.
SUBM'D. MT	CHKD.	DATE: DEC 2013	SITE:
DRAWN: JJL	CHKD. SEMC	APPD. JMAC	DWG. 1



APPENDIX A

Highway 66 Realignment, Virginiatown – Culvert at STA 14+510 Record of Boreholes



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Notes: 1
2

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

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



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


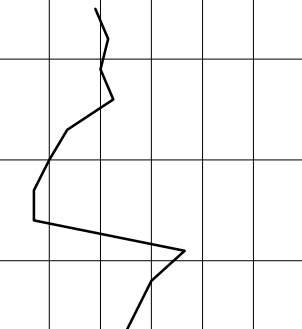
Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

PROJECT 10-1191-0044			RECORD OF BOREHOLE No BC4-1			1 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			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT 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 (%)	γ	GR SA SI CL
305.2	WATER SURFACE											
0.0	WATER											
0.2	PEAT (Amorphous) Very soft Black Moist		1	SS	1		305					
304.3												
0.9	CLAYEY SILT, layered Firm Brown Wet		2	SS	4		304					
			3	SS	2		303					0 1 70 29
302.4												
2.8	CLAY Soft to firm Grey Wet Clayey silt / silt and clay irregular varves between 1 mm and 25 mm thick noted in Shelby tubes samples 4, 5 and 6.		4	TO	PH		302				16.1	0 2 33 65
			5	TO	PH		301					
							300				16.2	
							299					
			6	TO	PH		298				16.6	0 5 39 56
							297					
			7	SS	WH							
296.3												
8.9	CLAYEY SILT Very soft Grey Wet		8	SS	1		296					
294.8							295					
10.4	SILT, trace to some clay Very loose to compact Grey Wet		9	SS	4		294					
			10	SS	11		293					0 0 88 12
							292					
			11	SS	8		291					
290.4												
14.8												

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 19/12/13 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 BC4-1				2 OF 2 METRIC									
G.W.P. <u>5091-07-00</u>				LOCATION <u>N 5334678.1; E 410115.3</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>November 16, 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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED									
--- CONTINUED FROM PREVIOUS PAGE ---																	
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																	

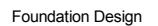
SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 19/12/13 DATA INPUT:

PROJECT 10-1191-0044			RECORD OF BOREHOLE No BC4-2			1 OF 2 METRIC															
G.W.P. 5091-07-00			LOCATION N 5334695.8; E 410109.4			ORIGINATED BY GM															
DIST _____ HWY 66			BOREHOLE TYPE Portable Equipment, NW Casing, Wash Boring			COMPILED BY MT															
DATUM GEODETIC			DATE November 14, 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		
304.8	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																				
304.1	PEAT (Amorphous)		1a				304														
0.9	CLAYEY SILT, layered Soft Brown Wet		1b	SS	2																
			2	SS	4		303														
302.7	SILTY CLAY to CLAY Firm Grey Wet		3	SS	WH		302														
2.1	Silt seams encountered between 2.1 and 4.8 m depth.																				
			4	SS	1		301														
							300														
			5	SS	WH		299												0 0 43 57		
	Silt seams encountered between 6.3 and 9.4 m depth.																				
			6	SS	WH		298														
							297														
			7	SS	1		296														
295.4	SILT, some clay Loose to compact Grey Wet																				
9.4			8	SS	9		295												NP 0 0 87 13		
							294														
			9	SS	5		293														
							292														
			10	SS	21		291														
290.5	Spoon attempted at 14.3 m depth, bouncing.																				
14.3																					

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 19/12/13 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

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 19/12/13 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			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					
305.2	WATER SURFACE							20 40 60 80 100	20 40 60 80 100	20 40 60											
0.0	WATER																				
304.9	PEAT (Fibrous)		1	SS	1																
304.6	CLAYEY SILT, layered Very soft to firm Grey to brown Wet		2	SS	3																
0.6	Trace organics above 0.9 m depth.		3	SS	1																
302.3	SILTY CLAY Soft to firm Grey Wet		4	SS	WH																
2.9			5	SS	WH																
	Silt seams encountered between 7.5 m and 9.0 m depth.		6	SS	WH																
296.2	CLAYEY SILT Firm Grey Wet		8	SS	1																
9.0			9	SS	4																
294.7	SILT, trace to some clay Loose to dense Grey Wet		10	SS	7																
10.5			11	SS	35																
290.6	END OF BOREHOLE START OF DCPT																				
14.6																					

SUD-MTO 001 10-1191-0044SUD.GPJ GAL-MISS.GDT 19/12/13 DATA INPUT:

Continued Next Page

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

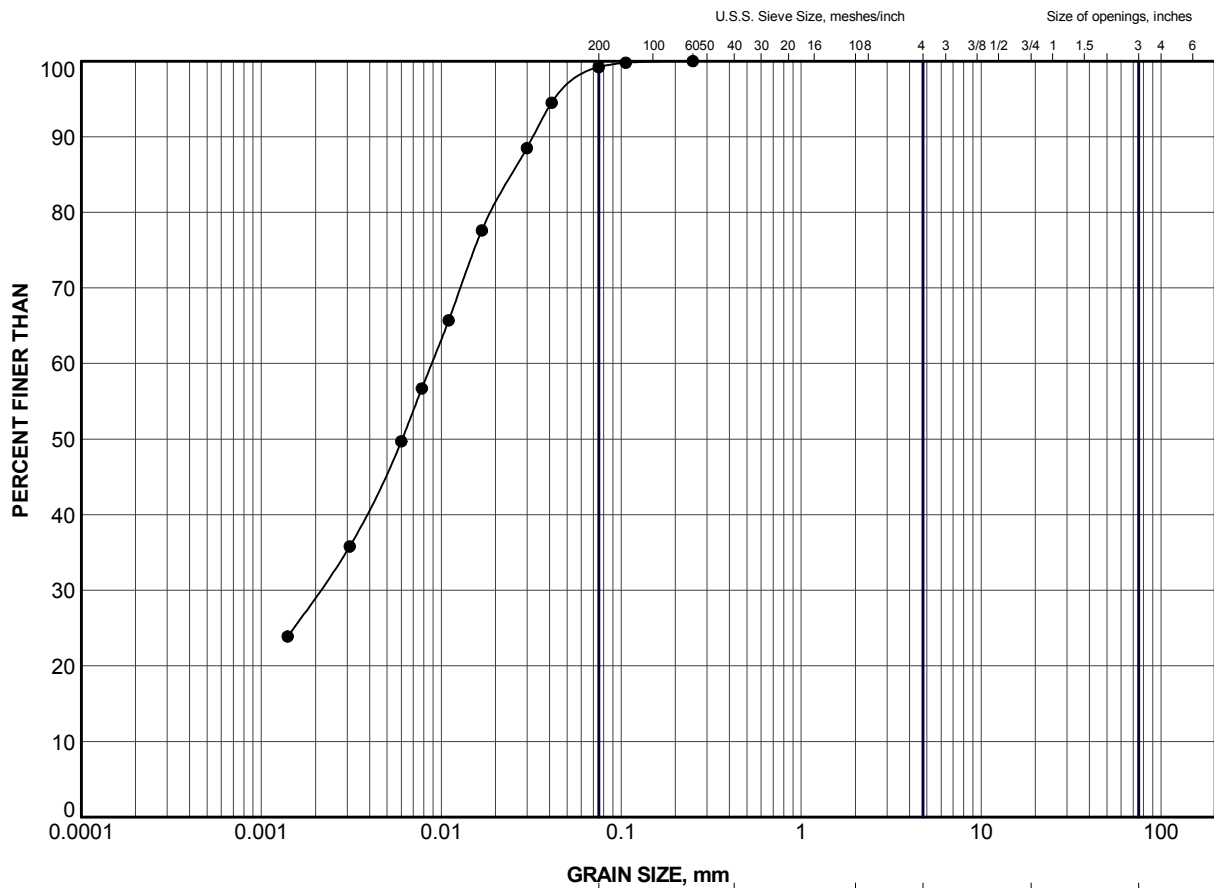


+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE



APPENDIX B

Highway 66 Realignment, Virginiatown – Culvert at STA 14+510 Laboratory Test Results



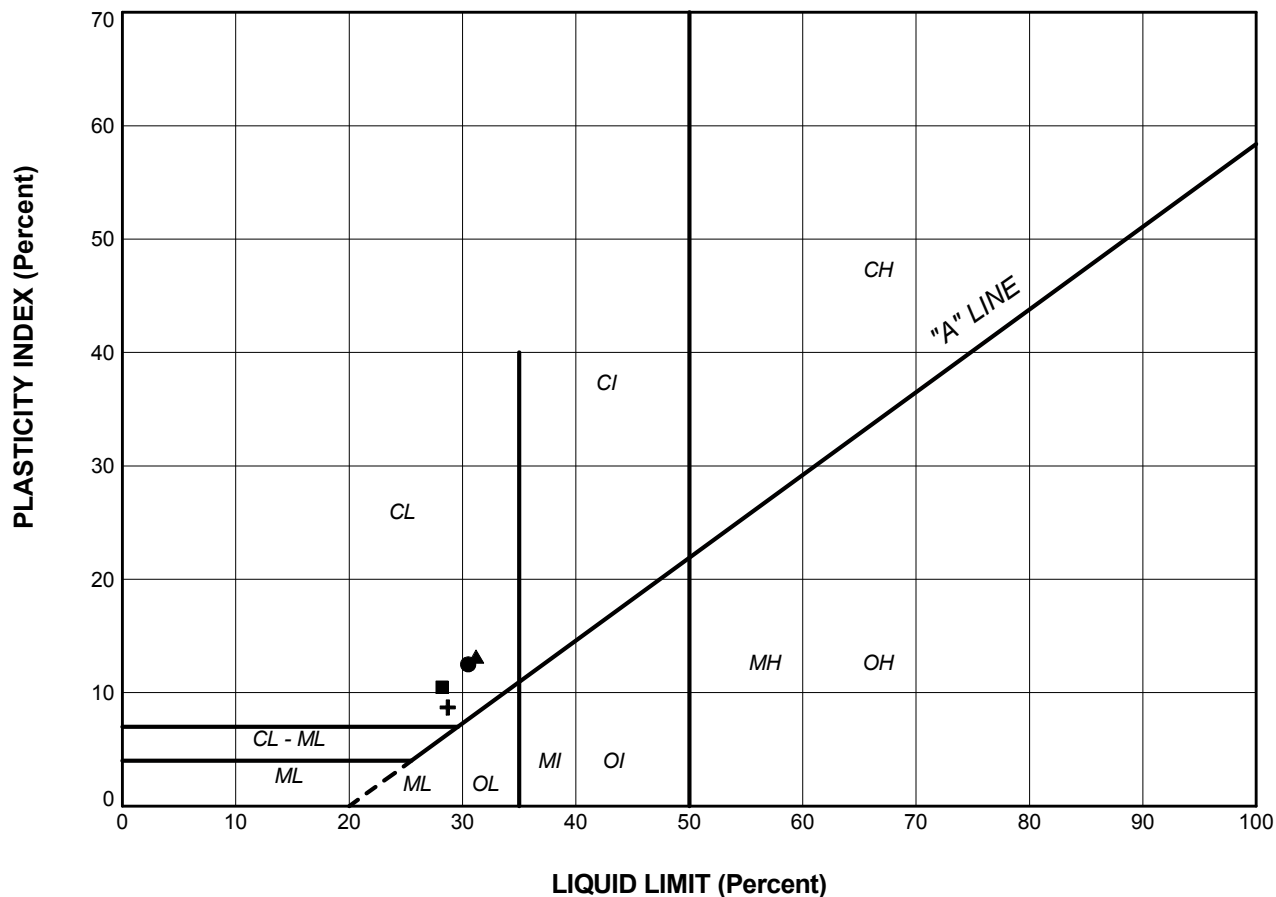
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC4-1	3	303.2

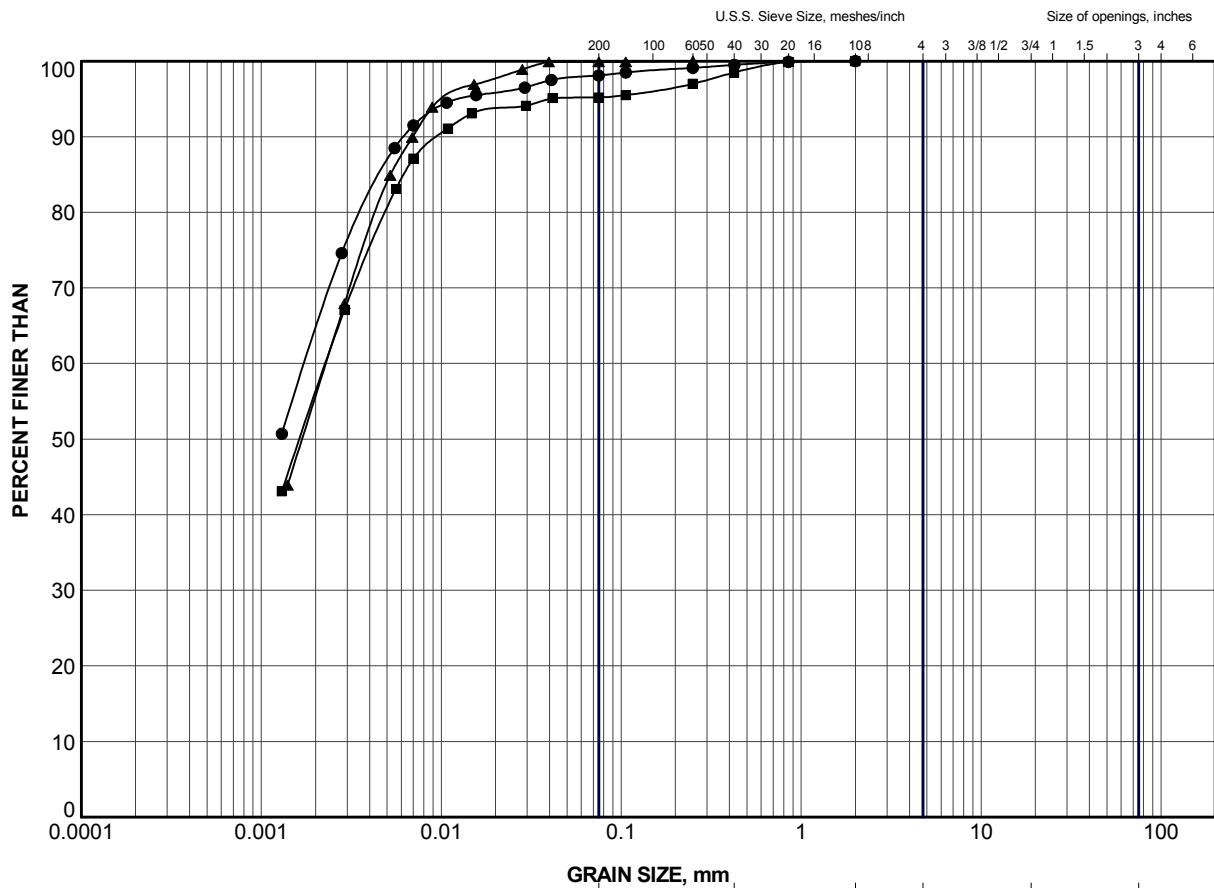
PROJECT					
HIGHWAY 66 - CULVERT BC4 STA 14+510					
TITLE					
GRAIN SIZE DISTRIBUTION CLAYEY SILT					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044C.GPJ	
DRAWN	TB	Dec 2013	SCALE	N/A	REV.
CHECK	SEMC	Dec 2013			
APPR	JMAC	Dec 2013			
			FIGURE B1		





PROJECT					
HIGHWAY 66 - CULVERT BC4 STA 14+510					
TITLE					
PLASTICITY CHART CLAYEY SILT					
PROJECT No.		10-1191-0044		FILE No.	
				10-1191-0044C.GPJ	
DRAWN	TB	Dec 2013	SCALE	N/A	REV.
CHECK	SEMC	Dec 2013			
APPR	JMAC	Dec 2013			
			FIGURE B2		





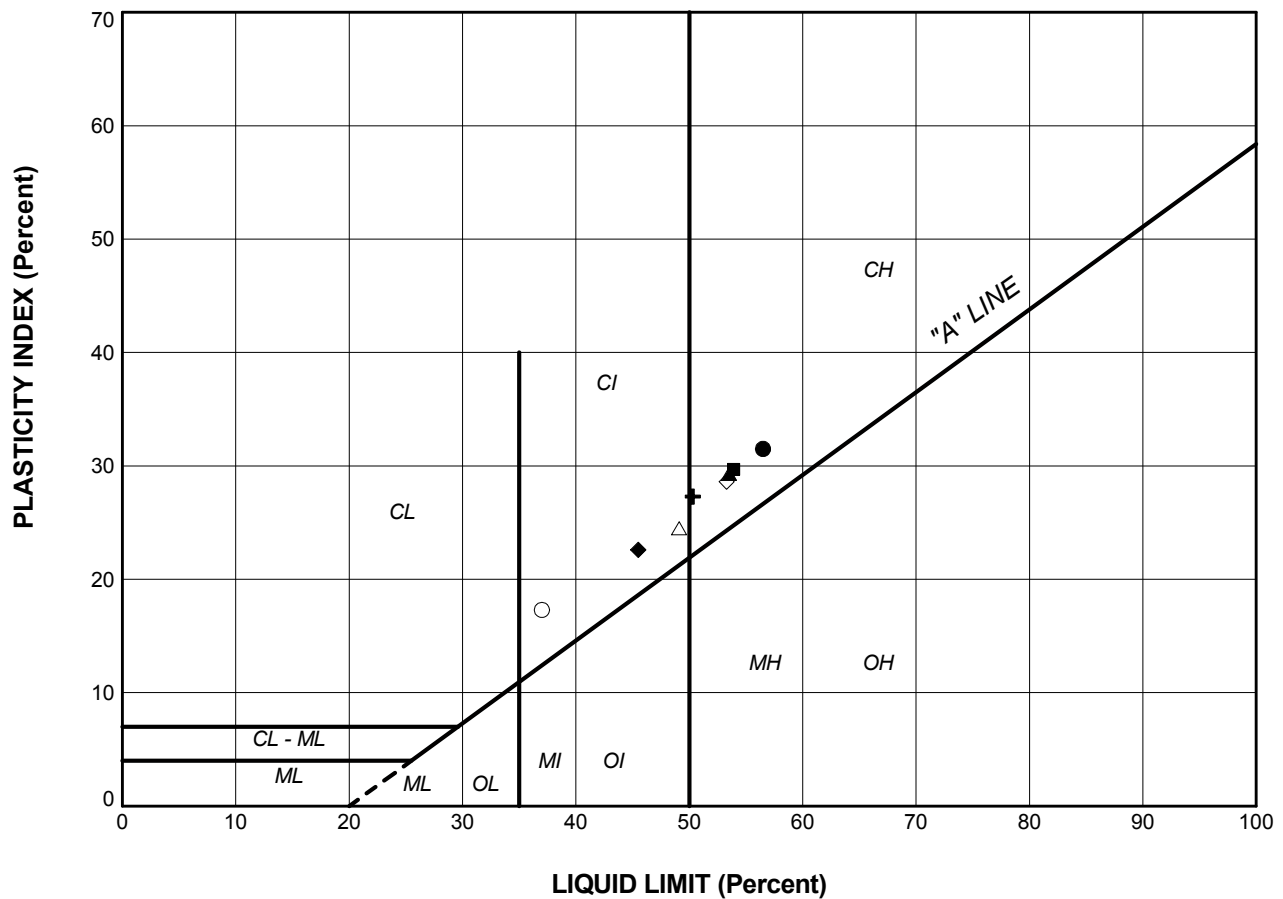
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC4-1	4	301.8
■	BC4-1	6	298.7
▲	BC4-2	5	299.2

PROJECT					
HIGHWAY 66 - CULVERT BC4 STA 14+510					
TITLE					
GRAIN SIZE DISTRIBUTION CLAY					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044C.GPJ	
DRAWN	TB	Dec 2013	SCALE	N/A	REV.
CHECK	SEMC	Dec 2013	FIGURE B3		
APPR	JMAC	Dec 2013			





SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	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

PROJECT					HIGHWAY 66 - CULVERT BC4 STA 14+510				
TITLE					PLASTICITY CHART SILTY CLAY to CLAY				
PROJECT No.			10-1191-0044		FILE No.			10-1191-0044C.GPJ	
DRAWN	TB	Dec 2013	CHECK	SEMC	Dec 2013	SCALE	N/A	REV.	
APPR	JMAC	Dec 2013				FIGURE B4			



CONSOLIDATION TEST SUMMARY**FIGURE B5**

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/m ³
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

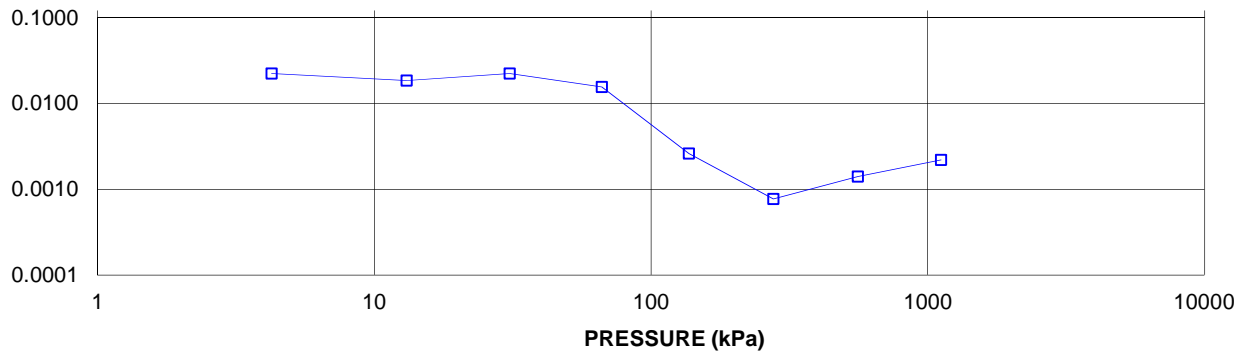
CONSOLIDATION TEST SUMMARY

FIGURE B5

Pg. 2 of 4

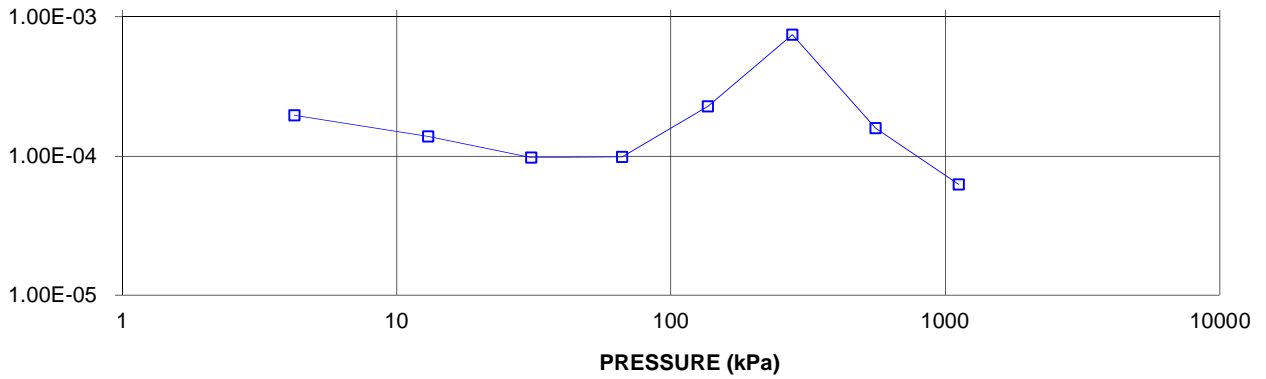
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH BC4-1 Sa 4



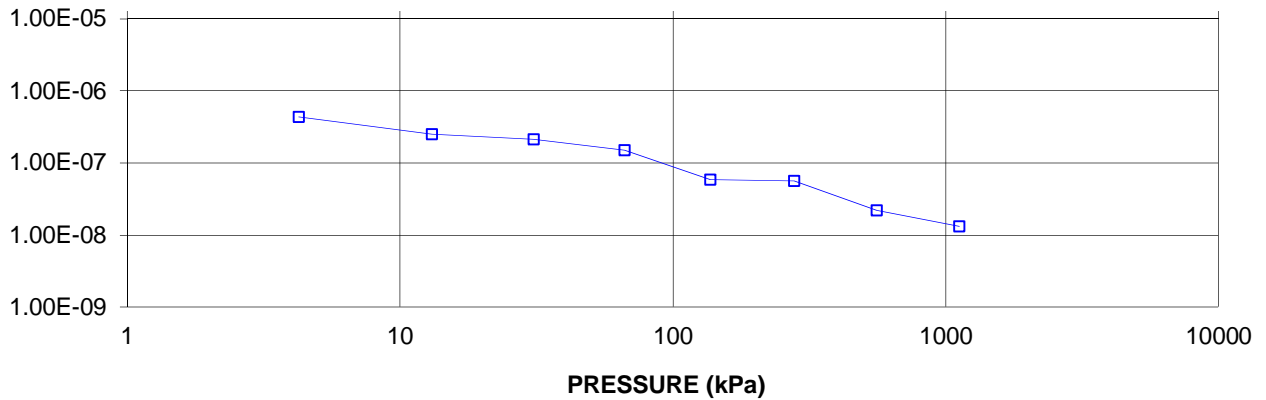
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH BC4-1 Sa 4



HYDRAULIC CONDUCTIVITY,
cm/s

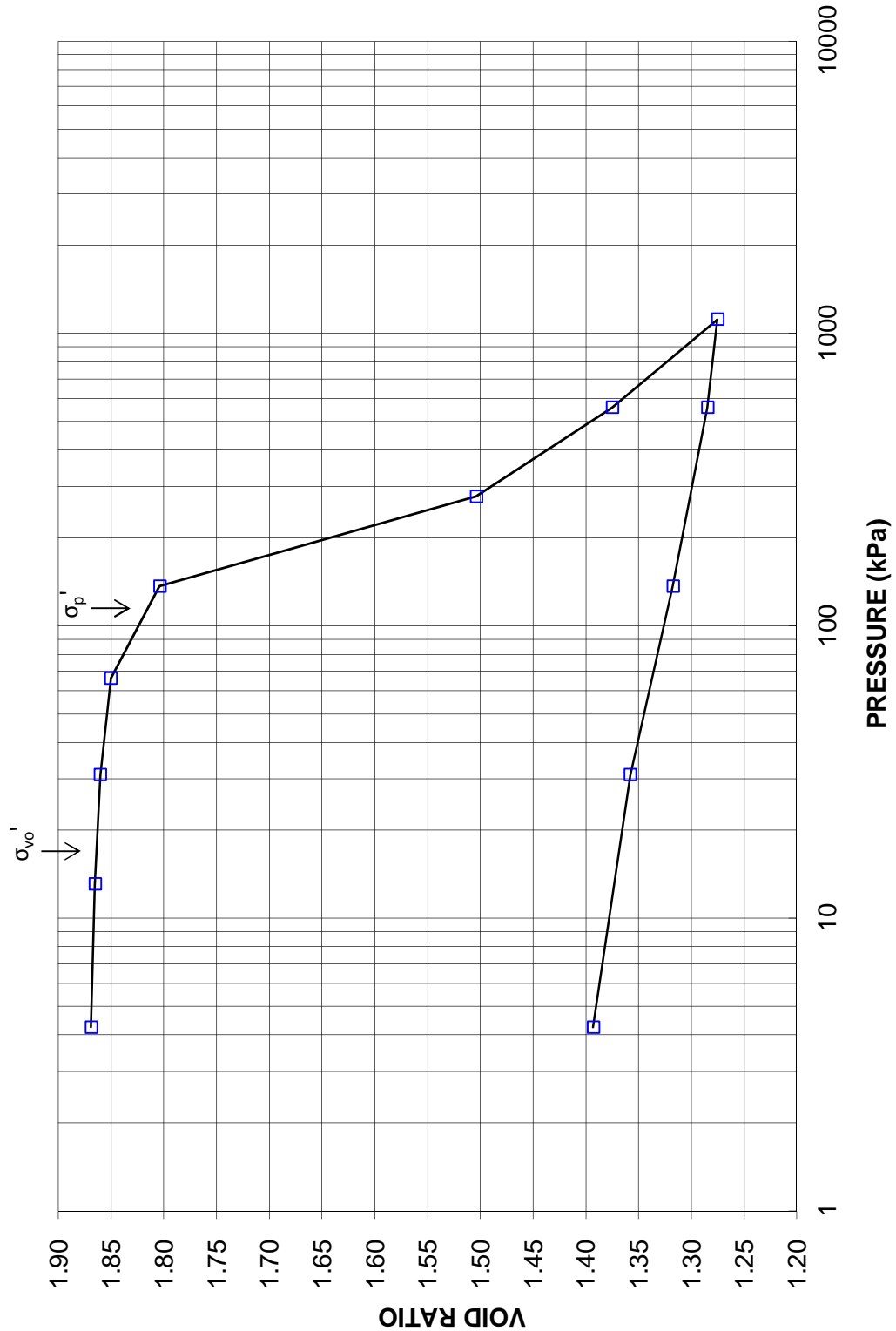
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH BC4-1 Sa 4



CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE

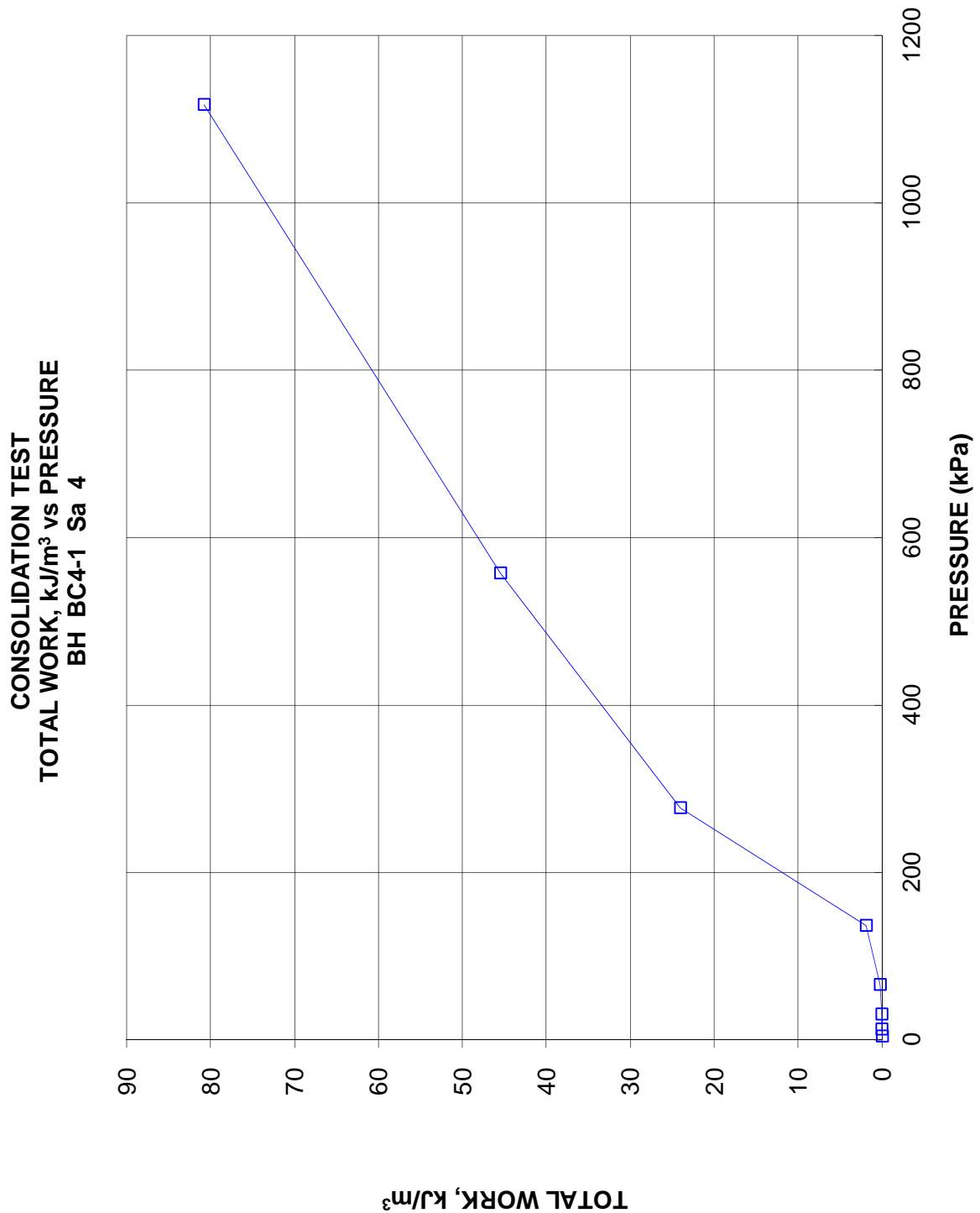
FIGURE B5
Pg. 3 of 4

CONSOLIDATION TEST
VOID RATIO VS PRESSURE
BH BC4-1 Sa 4



**CONSOLIDATION TEST
TOTAL WORK VS PRESSURE**

FIGURE B5
Pg. 4 of 4



CONSOLIDATION TEST SUMMARY**FIGURE B6****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		

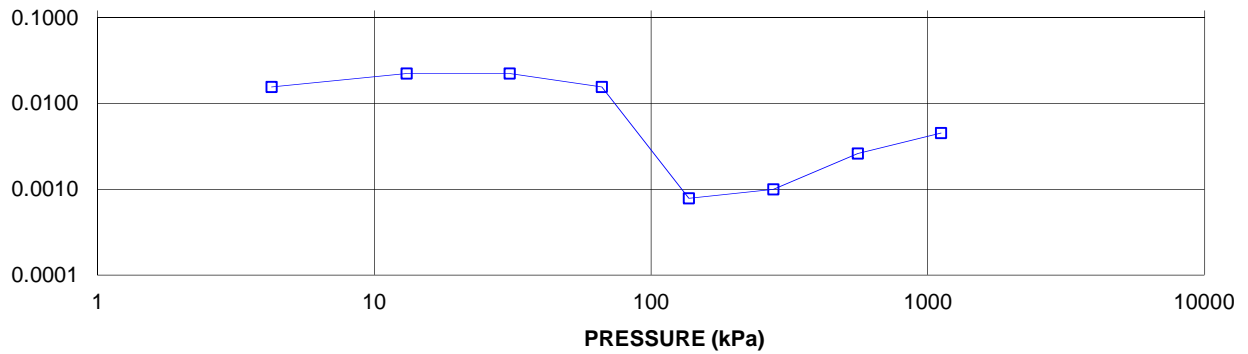
CONSOLIDATION TEST SUMMARY

FIGURE B6

Pg. 2 of 4

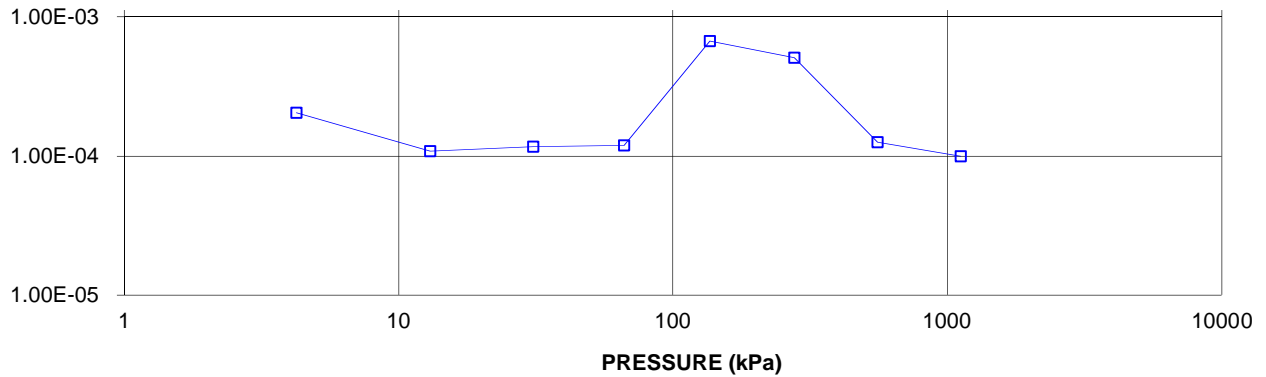
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH BC4-1 Sa 5



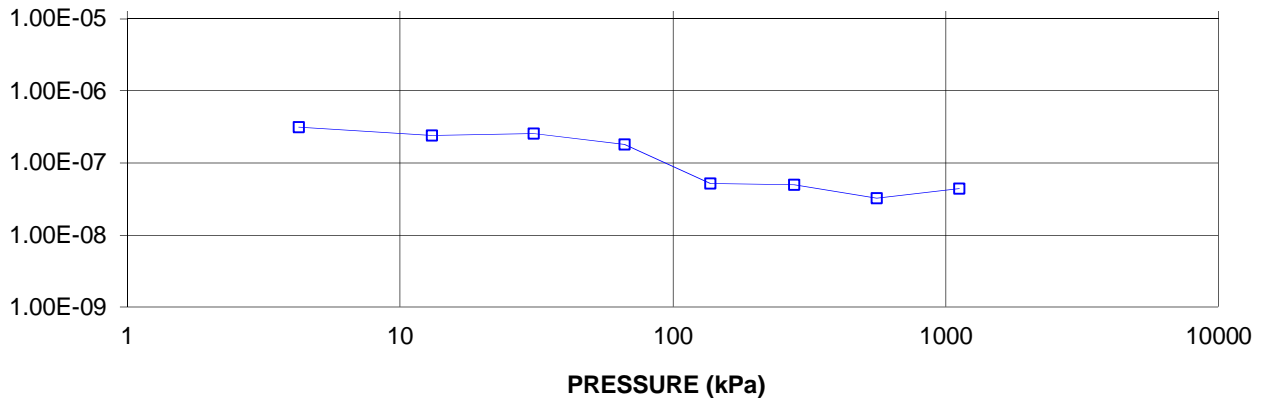
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH BC4-1 Sa 5



HYDRAULIC CONDUCTIVITY,
cm/s

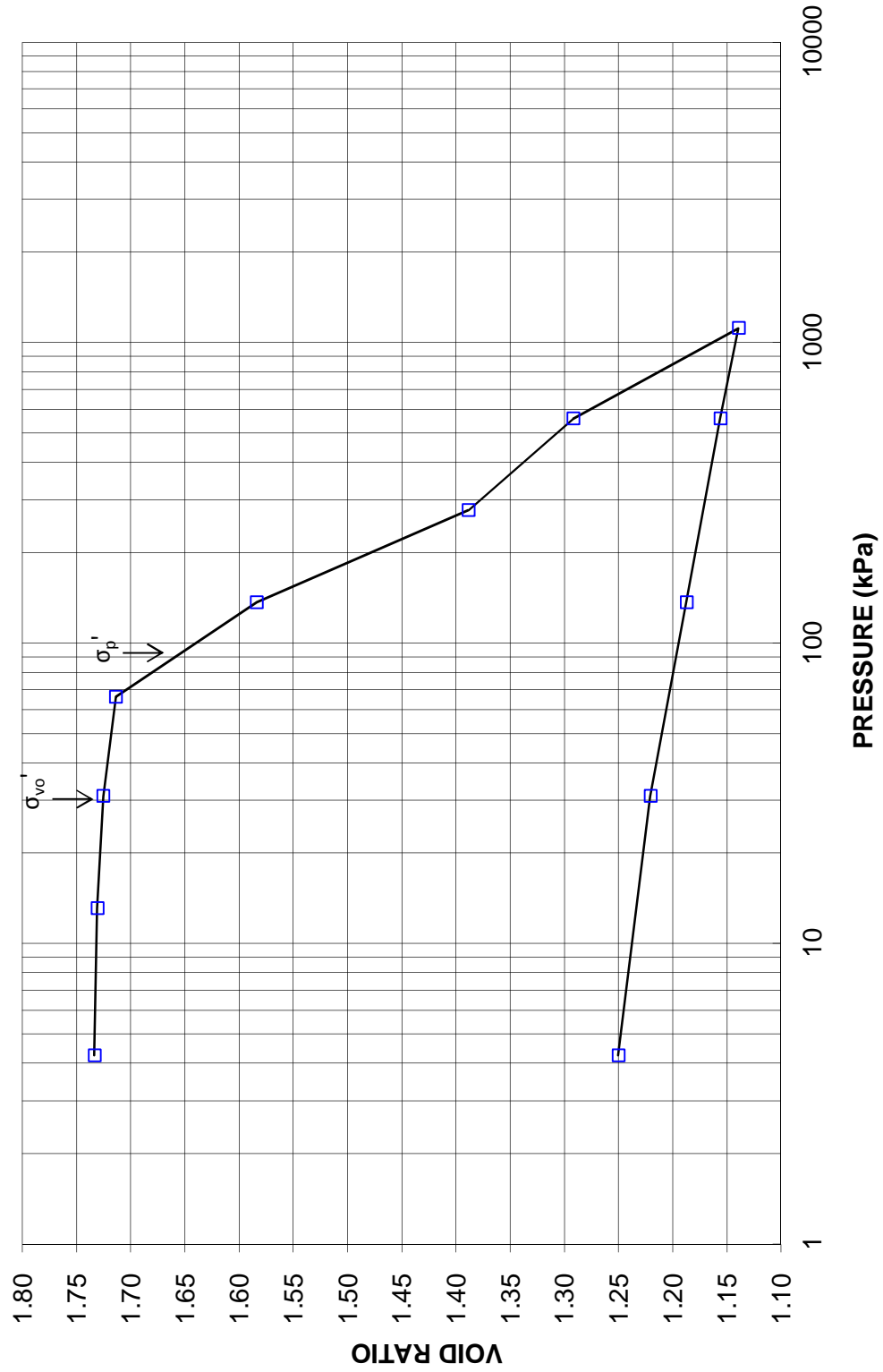
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH BC4-1 Sa 5



CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE

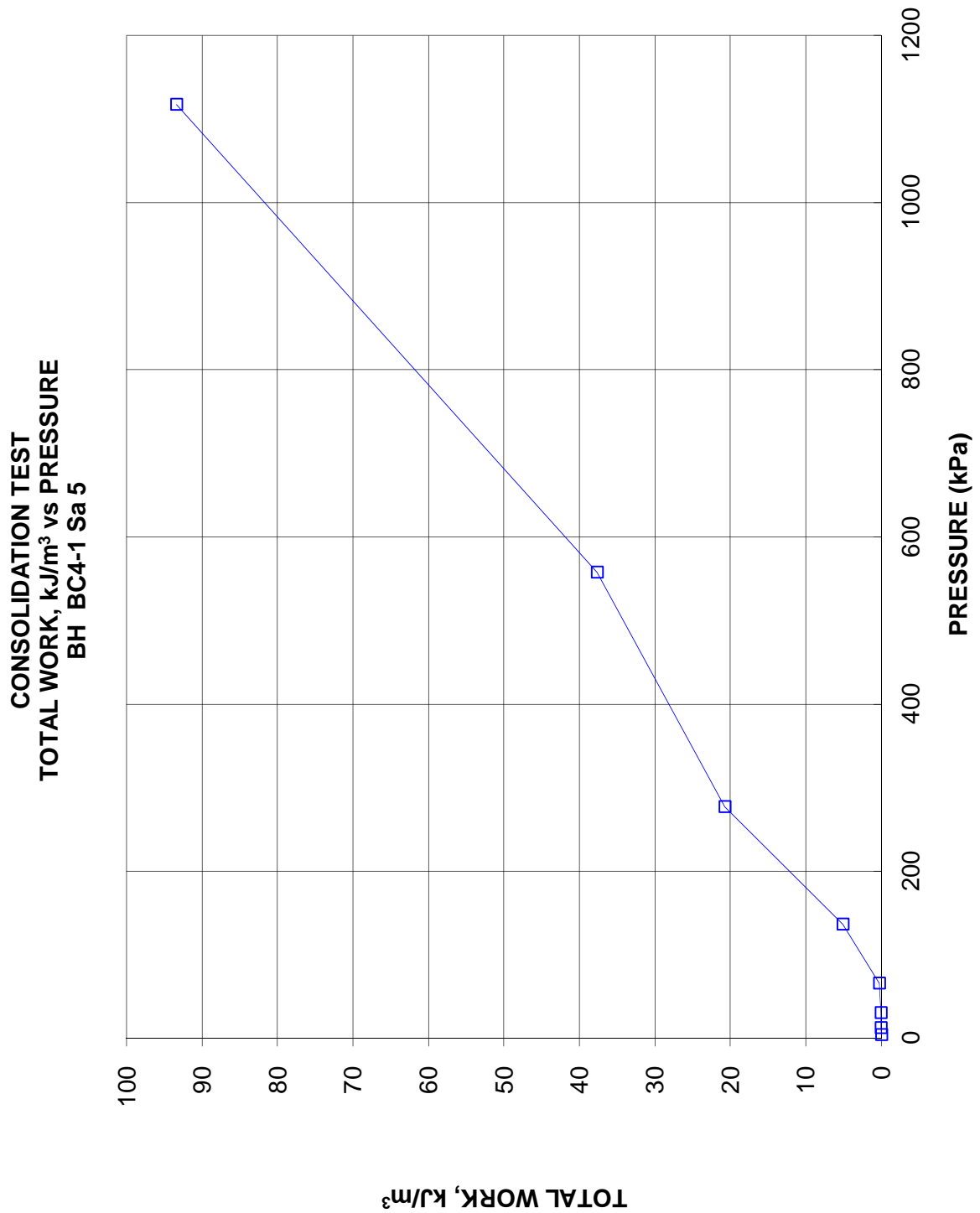
FIGURE B6
Pg. 3 of 4

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH BC4-1Sa 5



**CONSOLIDATION TEST
TOTAL WORK VS PRESSURE**

FIGURE B6
Pg. 4 of 4



CONSOLIDATION TEST SUMMARY**FIGURE B7****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					

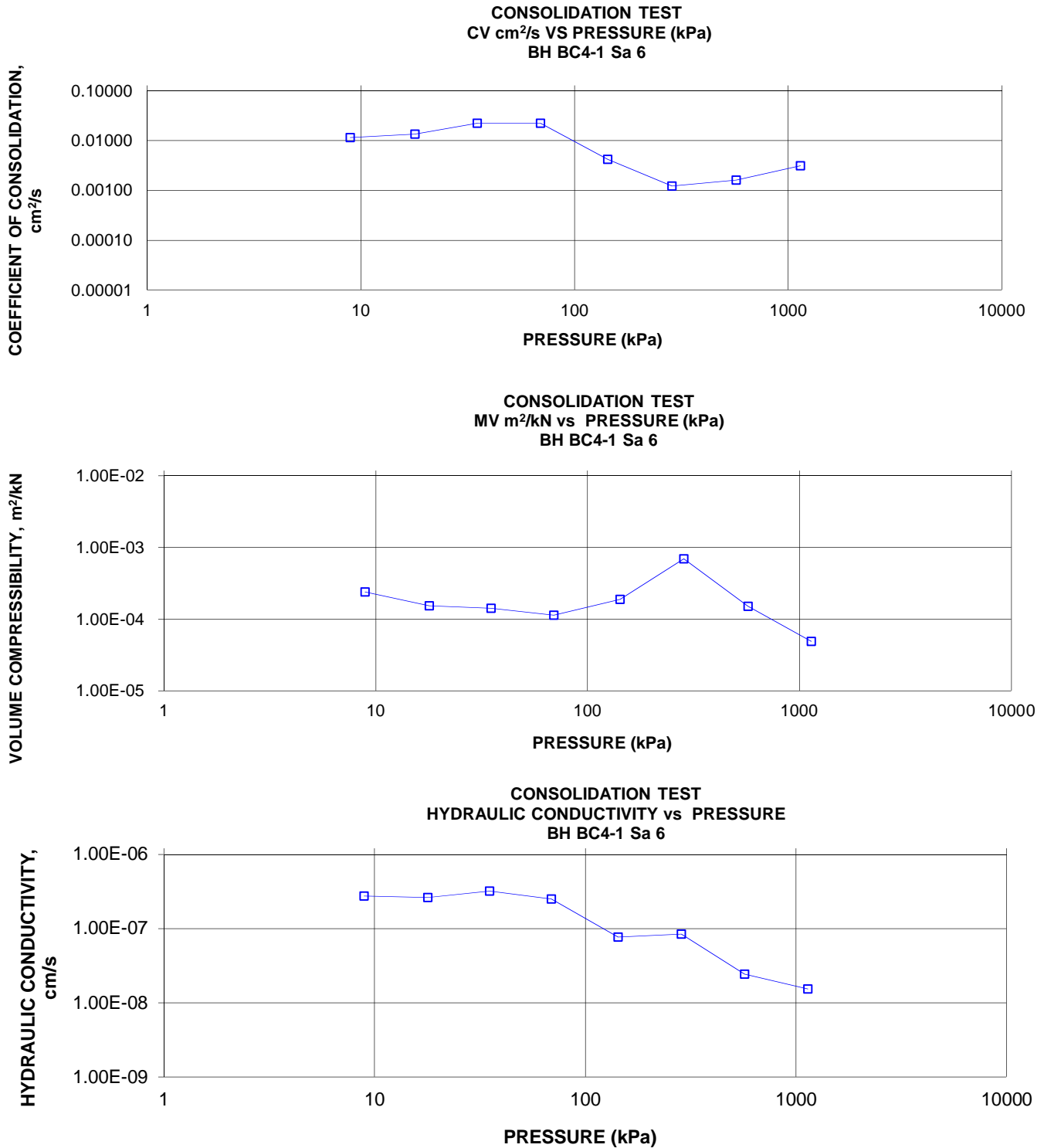
Note:

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

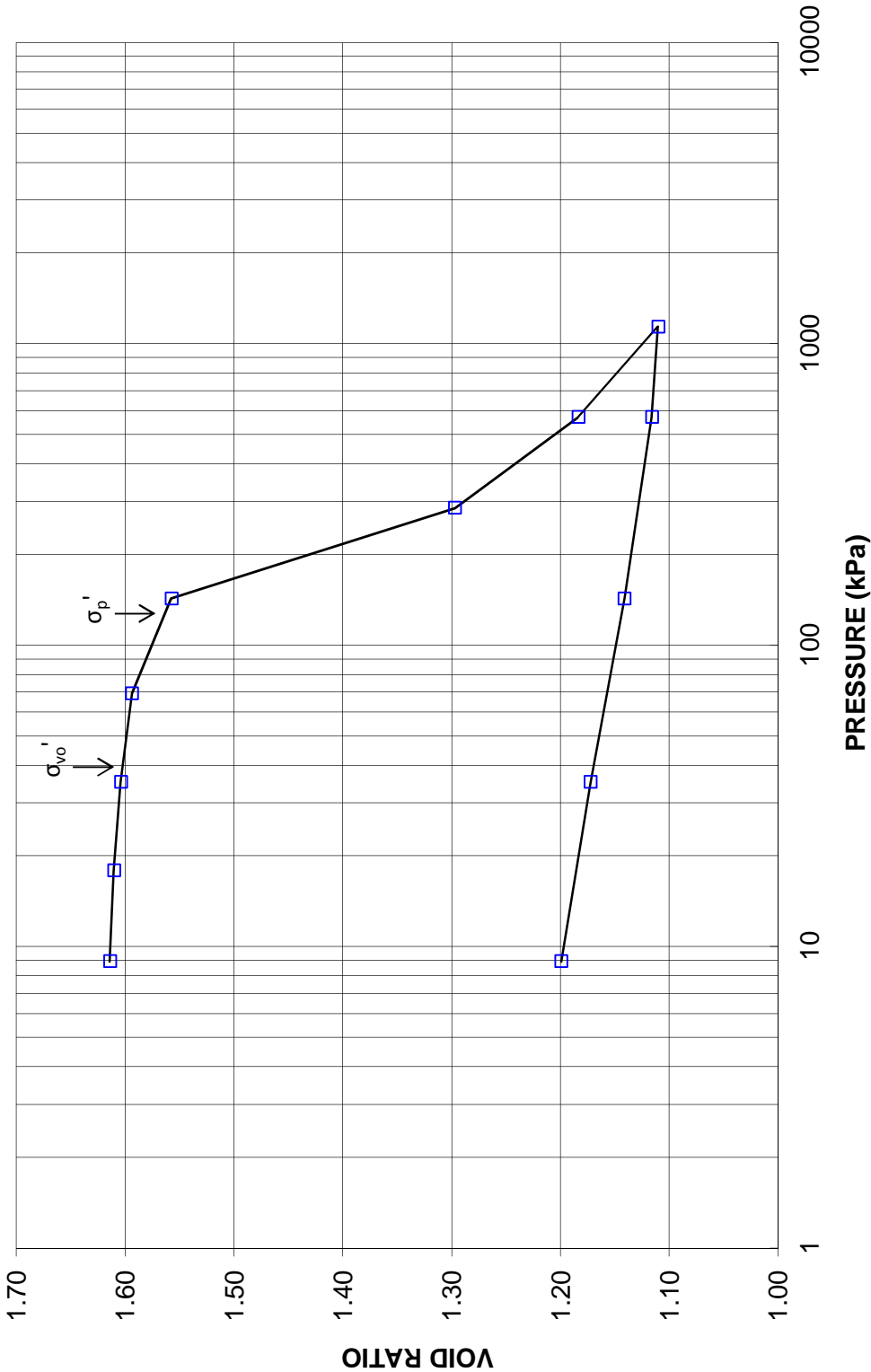
FIGURE B7
Pg. 2 of 4



CONSOLIDATION TEST VOID RATIO VS LOG PRESSURE

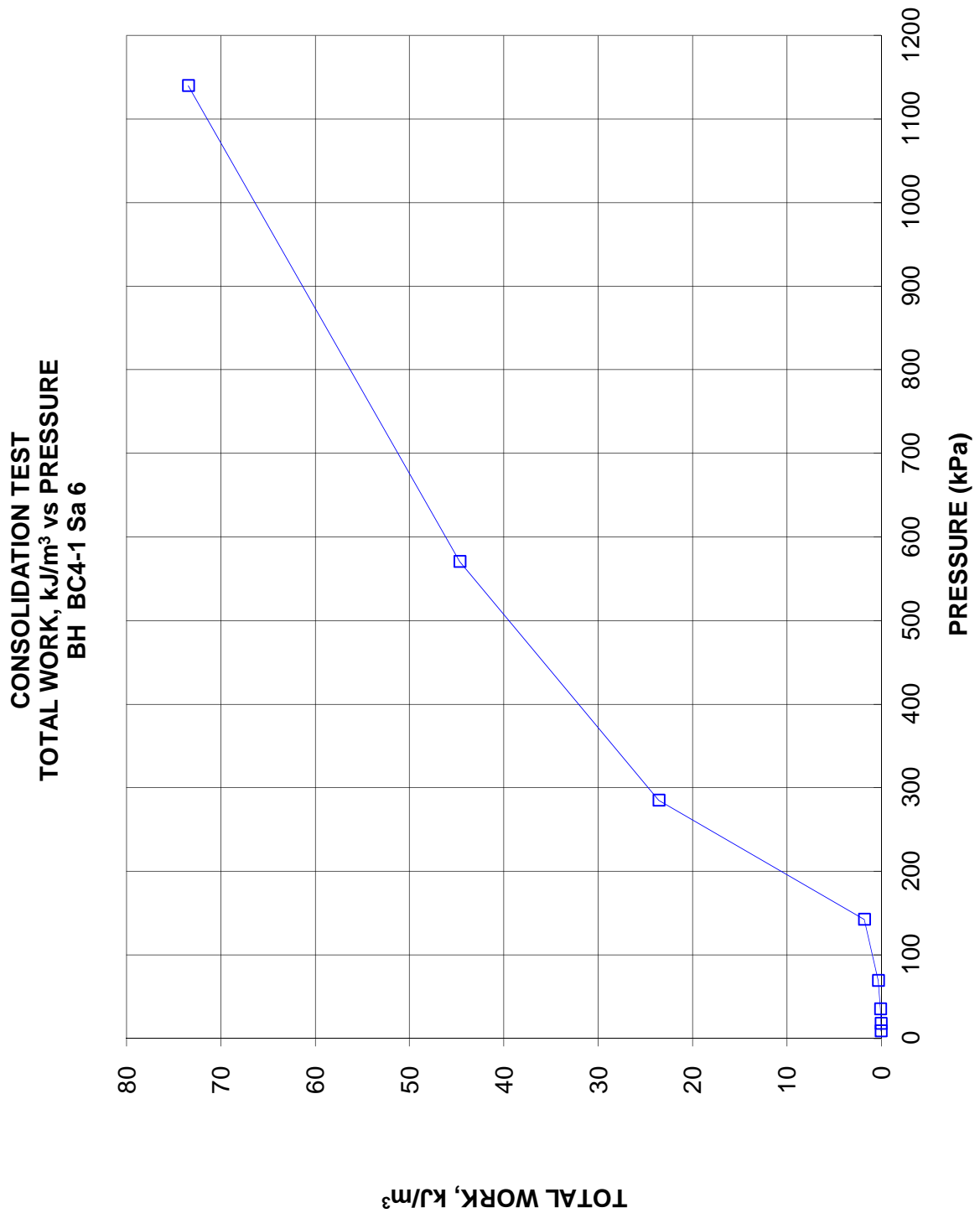
FIGURE B7
Pg. 3 of 4

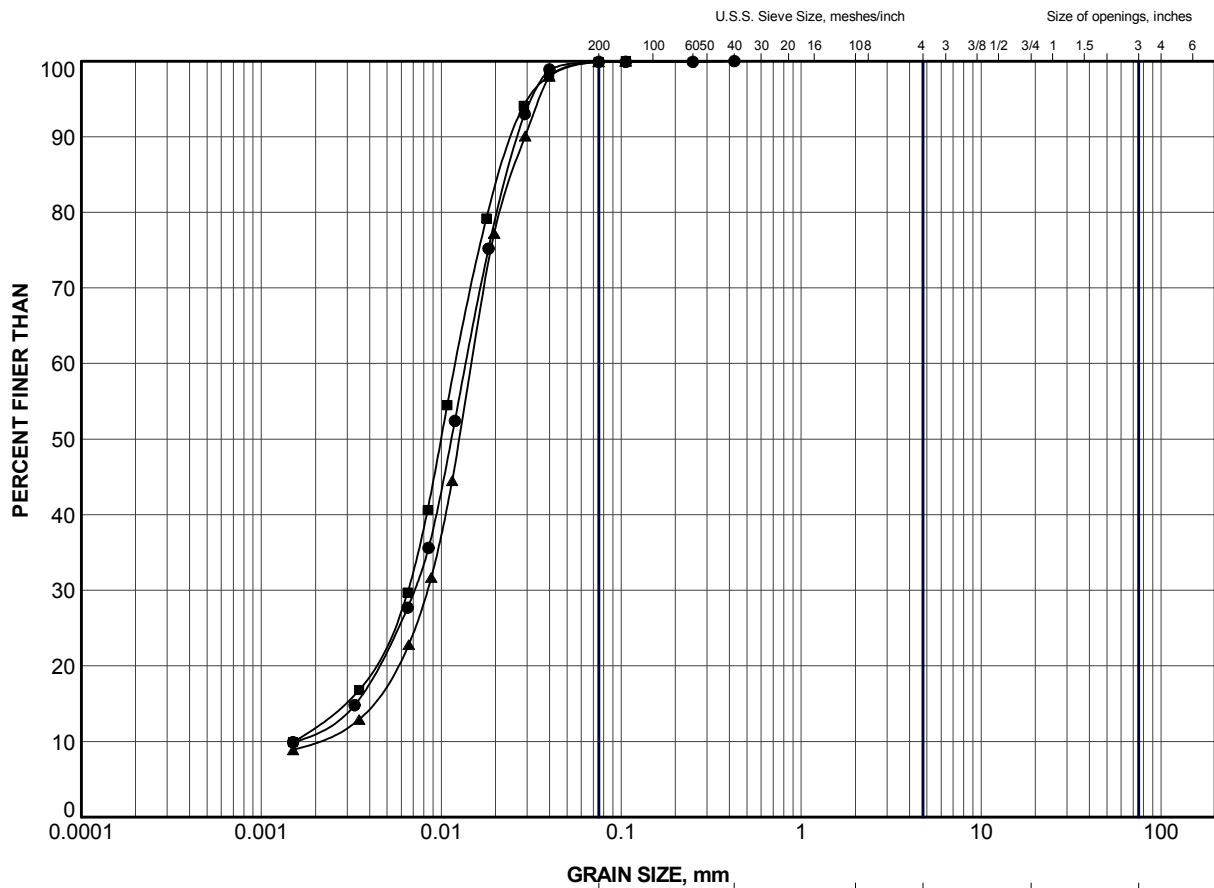
CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH BC4-1 Sa 6



**CONSOLIDATION TEST
TOTAL WORK VS PRESSURE**

FIGURE B7
Pg. 4 of 4





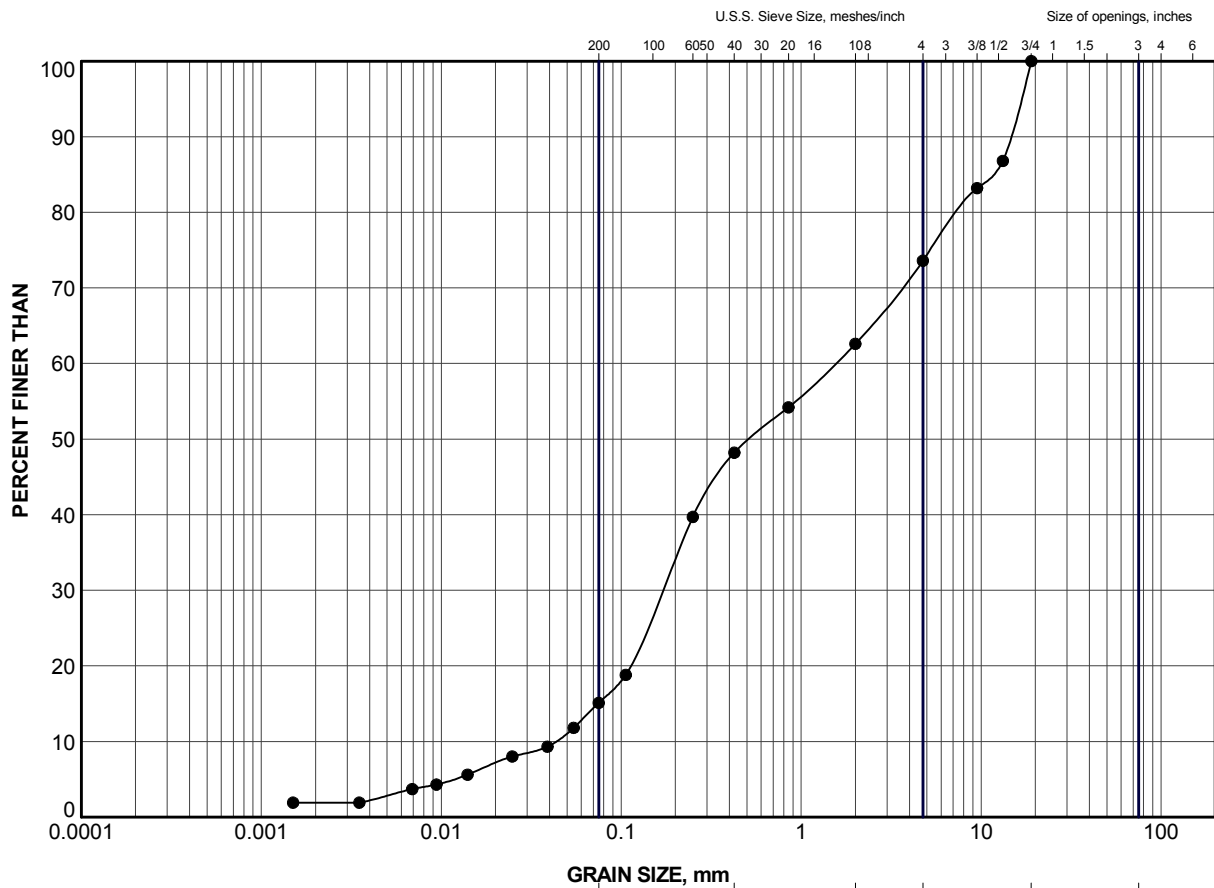
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC4-1	10	292.7
■	BC4-2	8	294.7
▲	BC4-3	10	292.4

PROJECT					
HIGHWAY 66 - CULVERT BC4 STA 14+510					
TITLE					
GRAIN SIZE DISTRIBUTION SILT					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044C.GPJ	
DRAWN	TB	Dec 2013	SCALE	N/A	REV.
CHECK	SEMC	Dec 2013	FIGURE B8		
APPR	JMAC	Dec 2013			




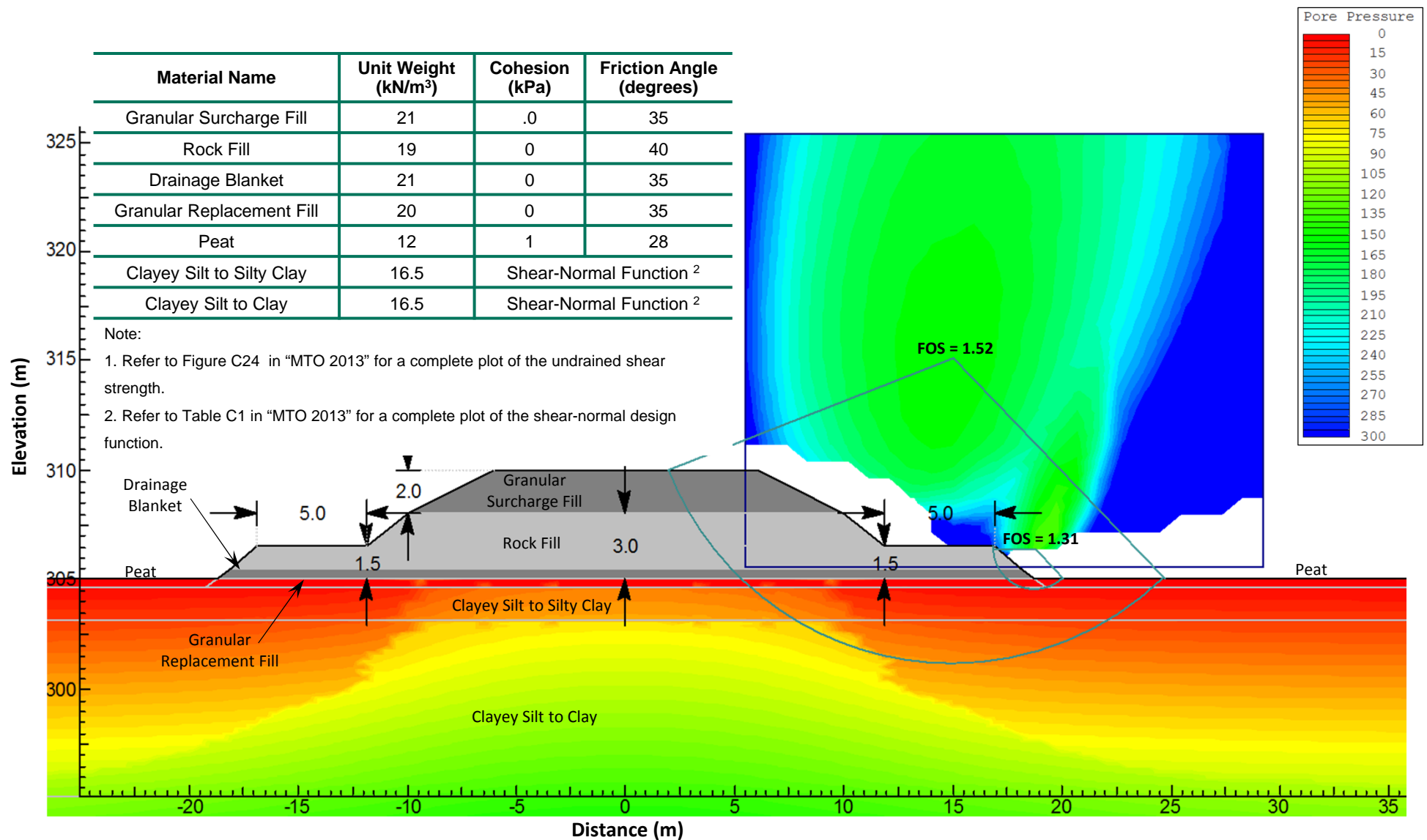


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BC4-1	12	289.8

PROJECT					
HIGHWAY 66 - CULVERT BC4 STA 14+510					
TITLE					
GRAIN SIZE DISTRIBUTION GRAVELLY SAND					
PROJECT No.		10-1191-0044		FILE No. 10-1191-0044C.GPJ	
DRAWN	TB	Dec 2013	SCALE	N/A	REV.
CHECK	SEMC	Dec 2013			
APPR	JMAC	Dec 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE B9		



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 APPROXIMATE CULVERT LOCATION			
PROJECT No. 10-1191-0044		FILE No. ----			
DESIGN	TZ	DEC 2013	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	CN	DEC 2013	FIGURE B10		
REVIEW	FJH	DEC 2013			



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