May 2013

REPORT ON

Foundation Investigation and Design Proposed Bridge Replacement Salmon River Bridge Highway 7 G.W.P. 4034-09-00

Submitted to:
Morrison Hershfield Limited
2440 Don Reid Drive
Ottawa, Ontario
K1H 1E1

GEOCRES Number: 31C- 213

Report Number: 11-1111-0017

Distribution:

3 copies - Morrison Hershfield Limited

3 copies - Ministry of Transportation, Ontario, Kingston, Ontario

1 copy - MTO Foundations Section, Downsview, Ontario

2 copies - Golder Associates Ltd.

REPORT

Golder Associates

Table of Contents

TABLES

DRAWINGS

APPENDICES

PART A

Foundation Investigation
Proposed Bridge Replacement
Salmon River Bridge
Highway 7
G.W.P. 4034-09-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited. on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations for two existing structures along Highway 7 and along Highway 41 in Ontario.

Foundation investigation services are required on this project for the following components:

- Highway 7 Salmon River Bridge replacement; and,
- Highway 41 Culvert (at site 29-235) replacement or rehabilitation.

This report addresses the replacement of the Highway 7 Salmon River Bridge located about 0.2 km west of the Henderson Road/Arden Road intersection under G.W.P. 4034-09-00.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated January 2011 and in Section 5.8 (Foundations Engineering) of the *Technical Proposal* for this assignment as well as Addendum (4) dated July 25, 2011.

The work was carried out in accordance with Golder's Quality Control Plan dated April 22, 2013.

2.0 SITE DESCRIPTION

The existing Salmon River Bridge structure in this assignment (G.W.P. 4034-09-00) is located on Highway 7 at about 0.2 km west of the Henderson Road/Arden Road intersection, and 18 km east of Kaladar, Ontario.

Through this area, Highway 7 is a two lane undivided highway with a rural cross-section. The existing structure is aligned approximately east-west. The highway profile grade over the structures is at Elevations 191.3 m and 191.6 m at the east and west abutments, respectively. The existing structure consists of a two-span rigid concrete frame structure, with two arched decks, measuring about 26 m in overall length and 9 m in width. The construction drawings of the existing structure, dated March 1952, indicates that the east abutment, the north half of the west abutment, and the centre pier are supported by spread footings on bedrock. However, the construction drawings also indicate that the south half of the west abutment is supported on rock fill, which increases in thickness towards the south in a wedge shape.

The existing approach embankments are close to the surrounding land level, and about 3 m high relative to the banks of the Salmon River. Based on visible signs of blast holes on the north side of the bridge, the existing structure was built partly within a rock cut. This is also evident from the site survey plans, which appears to indicate that the original Salmon River flowed about 70 m west of its current location. The land on the south side of the bridge is about 1 m lower than the roadway with very gentle side slopes at about 8H:1V. On the north side, the land is slightly lower, and 1.5 to 2 m side slopes are present at about 3H:1V. No signs of embankment instability were observed.

The Salmon River runs beneath the Highway 7 structure with a high water level at Elevation 188.92 m.

The highway profile at the approaches does not seem to indicate that significant differential settlement of the roadway relative to the bridge has occurred, although the maintenance history at this location is not currently known.

3.0 INVESTIGATION PROCEDURES

The subsurface investigation was carried out for the Salmon River Bridge replacement project at two different times, and a total of eleven borehole locations were investigated during both visits. The preliminary foundation investigation was carried out between December 8 and 14, 2011, at which time four boreholes (number 11-1 to 11-4, inclusive) were advanced at the locations shown on Drawings 1 and 2. The detailed design foundation investigation was carried out between December 18, 2012 and January 3, 2013, at which time seven borehole locations (numbered 12-101 to 12-107, inclusive) were investigated at the locations shown on Drawings 1 and 2. At two of the borehole locations, borehole 12-105 and borehole 12-106, an adjacent borehole was put down, labelled BH 12-105A and BH 12-106A, to retrieve a Shelby tube of the soft grey clay at each borehole location.

The boreholes were advanced using a combination of 108 mm inside diameter continuous flight hollow stem augers and NW size wash casing, by truck and track-mounted drill rigs supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to depths of between about 4.3 and 15.4 m below the existing ground surface.

Soil samples were obtained at intervals ranging from 0.75 m to 1.5 m of depth, using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. Insitu vane testing (using an N-size vane) was carried out within the cohesive deposits where possible. Four relatively undisturbed, 73 mm diameter thin-walled Shelby tube samples of the silty clay were retrieved in boreholes 11-3, 12-105A, 12-106A and 12-107 using a fixed piston sampler. A fifth undisturbed sampling was attempted in borehole 12-104, but was unsuccessful.

A standpipe piezometer was installed in borehole 11-3 and 12-106 to monitor the groundwater level at the site. The standpipe consists of a 19 mm diameter PVC pipe with a 0.6 m long slotted screen section, installed within silica sand backfill and sealed by a 0.6 and 0.3 m long section of bentonite pellet backfill at boreholes 11-3 and 12-106, respectively. The water level in the standpipe piezometers were measured on April 16, 2012 and January 15, 2013. The water level in borehole 11-3 was also measured in the open borehole during the fieldwork on December 13, 2011.

The boreholes were backfilled with bentonite pellets, mixed with native soils, and the site conditions restored following completion of work. The standpipe piezometers will be decommissioned prior to construction, unless instructed otherwise by the Ministry.

The field work was supervised throughout by a member of Golder's technical staff, who located the boreholes, supervised the drilling, sampling and insitu 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 Golder's laboratories in Ottawa and Mississauga for further examination. Index and classification tests consisting of grain size distribution, water content, organic content, and Atterberg limit testing were carried out on selected soil samples at the Ottawa laboratory. Unconfined Compression (UC) tests were carried out on four samples of bedrock from boreholes 11-1, 11-2, 12-105 and 12-106. Diametrical Point Load (PL) tests were also carried out on nine samples of bedrock from boreholes 11-1, 11-2, and 11-4. The UC and PL testing as well as two oedometer tests on samples of the silty clay in boreholes 105A and 107 were carried out at the Mississauga laboratory. All of the laboratory tests were carried out to MTO and/or ASTM standards as appropriate.

The borehole elevations and locations were surveyed by Golder Associates personnel using a Trimble R-8 GPS unit. The borehole locations, including MTM Zone 9 NAD83 northing and easting coordinates and ground surface elevations referenced to Geodetic datum are summarized in the following table and are shown on Drawings 1 and 2.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
11-1	West bridge abutment, westbound lane	4955513.0	270261.3	191.7	7.3
11-2	East bridge abutment, westbound lane	4955527.7	270303.9	191.4	7.0
11-3	South of west bridge abutment	4955498.3	270267.1	190.5	10.8
11-4	South of east bridge abutment	4955513.5	270308.7	190.7	11.4
12-101	North of east bridge abutment	4955531.0	270299.3	191.5	4.3
12-102	23 m west of west bridge abutment, in shoulder of westbound lane	4955509.4	270248.1	191.5	6.9
12-103	40 m west of west bridge abutment, in shoulder of westbound lane	4955504.6	270232.1	191.3	12.8
12-104	58 m west of west bridge abutment, beyond shoulder of westbound lane	4955501.2	270214.5	191.0	15.4
12-105	South of west bridge abutment	4955503.1	270271.3	190.4	12.8
12-105A	South of west bridge abutment	4955502.9	270270.3	190.6	7.3
12-106	South of east bridge abutment	4955518.0	270306.0	191.5	9.9
12-106A	South of east bridge abutment	4955518.1	270307.0	191.4	3.6
12-107	20 m east of east bridge abutment, behind guardrail for eastbound lane	4955521.3	270317.5	191.4	7.6

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

As delineated in *The Physiography of Southern Ontario*[1], the study area for this assignment lies within the physiographic region known as the Georgian Bay Fringe.

The Georgian Bay Fringe is a broad belt that borders Georgian Bay, extending from Muskoka and Perry Sound, across the area north of the Kawartha Lakes, and reaches the western limits of Lanark County. It covers an estimated 3,340 km^2 and is characterized by very shallow soil and bare rock knobs and ridges. The study area is located within the Frontenac Axis where the Precambrian bedrock is exposed and extends from the Canadian Shield to the Adirondack mountain range in New York. From the Ontario Ministry of Natural Resources geology maps, the Precambrian bedrock in this area is indicated to be Clastic Metasediments of the Grenville Supergroup.[1]

4.2 Site Stratigraphy

As part of the preliminary subsurface investigation at this site, four boreholes were advanced. Subsequent to the preliminary study, a further nine boreholes were advanced in the vicinity of the Salmon River Bridge as part of the detailed design foundation investigation. The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during the current investigation, together with the results of the insitu and laboratory tests carried out on selected soil and rock samples, are given on the attached Record of Borehole sheets and on Figures A1 to A15 in Appendix A. The Record of Borehole sheets and Figures B1 to B4 from the preliminary investigation are included in Appendix B.

The soil stratigraphy section projected along the north side of Highway 7 is shown on Drawing 1, and the soil stratigraphy sections projected along the abutment areas are shown on Drawing 2. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsurface conditions encountered in the boreholes vary significantly from north to south, and perpendicular to the bridge. On the north side of Highway 7, exposed gneiss/schist/quartzite bedrock is visible on both the east and west sides of the bridge. Further west of the existing bridge, the presence of visible rock fill on the north side of Salmon River in the area of boreholes 12-102, 12-103 and 12-104, and the presence of drill holes for blasting on the north side of the existing bridge would indicate that the original Salmon River flowed about 70 west of its present location. From boreholes 12-102 to 12-104 put down in this area, the previous river channel seems to have been filled in with rock fill prior to the construction of the road structure.

On the south side of the bridge, the overburden soils consists of sandy fill, over organic silty soil, over silty clay, over silty sand to sand and gravel. The very dense sand and gravel deposit encountered below the silty clay on this side of the Salmon River was not fully penetrated in borehole 11-3, and the depth to bedrock was not established at this location. However, bedrock at borehole 12-105 was encountered at a depth of about 9.6 m (i.e., about Elevation 180.8 m).

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

On the southeast side of the bridge, the organic soils have a higher clay content, ranging from silty clay to clayey silt. The silty sand deposit below the silty clay at this location was fully penetrated and is underlain by schist bedrock at about 6.4 and 8.5 m depth (i.e., about Elevation 185.1 and 182.2 m) at boreholes 12-106 and 11-4, respectively.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Pavement Structure/Fill/Topsoil

The pavement structure was penetrated at boreholes 11-1, 11-2, 12-102 and 12-103 (all in westbound lane) with thicknesses ranging from about 0.3 m to 1.0 m. The pavement structure encountered at the boreholes consists of about 0.2 m of asphaltic concrete overlying 0.3 m to 0.6 m of crushed sand and gravel base and 0.3 m to 0.5 m of sand and gravel with cobbles subbase. The subbase was not encountered at borehole 12-102.

At borehole 11-3, 12-101, 12-104 and 12-105, a 0.1 m to 0.2 m thick layer of topsoil was encountered at ground surface, and at borehole 11-3 a 0.3 m thick buried layer of topsoil was encountered below the fill at about 1.4 m depth.

The fill was fully penetrated at all of the borehole locations and varied in thickness from 1.0 m to 5.5 m (including the pavement structure where present). The fill material generally consists of sand, silty sand, sandy silt and clayey silt. The lower portion of the fill at borehole 11-1, 12-102, 12-103, 12-104, 12-106 and 12-107 consists of rock fill with some sandy silt and gravel infill. Varying amounts of organic matter was also observed in the fill samples.

The result of grain size distribution testing carried out on six samples of the fill from borehole 11-3, 12-102, 12-103, 12-104, 12-105 and 12-106 are provided on Figures A1 and B1.

Standard penetration test N values for the embankment fill in borehole 11-1, 12-102, 12-103 and 12-104 were measured to be about 4 to greater than 50 blows per 0.3 m of penetration indicate it to be loose to dense, although the higher N values could reflect the presence of the larger rocks in the rock fill, rather than the state of packing of the soil matrix. Diamond Drilling was required to penetrate some sections of the fill. The N values to the south of the embankment were measured to be about 2 to 9 blows per 0.3 m indicating a very loose to loose material.

The water content on three samples of the embankment fill was measured to be about 8 to 39 percent. The water content on three samples of the fill south of the embankment was measured to be about 7 to 44 percent which reflects the trace of organic material in the sample.

4.2.2 Organic Soil

A deposit of black organic soil was encountered in borehole 11-1, 11-3, and 12-105 near the west abutment, in boreholes 11-4, 12-106 and 12-107 near the east abutment, and in boreholes 12-103 and 12-104 along the proposed retaining wall on the north side of Highway 7. In boreholes 11-1, 11-3, 12-103 to 12-107, this deposit consists of organic silty sand to clayey silt and extends to depths ranging from about 2.0 m to 8.2 m (i.e., Elevation 189.5 m to 182.8 m). In borehole 11-4, this deposit consists of organic silty clay to clayey silt, and extends to about 3.7 m depth (i.e., Elevation 186.9 m).

Standard penetration test N values for this material ranging from the weight of the sampler and rods to 6 blows per 0.3 m of penetration indicate a very loose to loose state of packing.

The measured organic content on nine selected samples of this deposit ranges from approximately 5 to 23 percent. The measured water content on these samples ranges from approximately 34 to 189 percent.

Due to the high organic content of this deposit and its effect on hydrometer readings, grain size distribution testing was not carried out. However, an Atterberg limits test was completed on one selected sample of this deposit from borehole 11-4 and measured a plastic limit of 36 percent, a liquid limit of 94 percent, and a plasticity index of 58 percent. This test result, which is plotted on a plasticity chart on Figure B4, indicates that the deposit consists of clay of high plasticity, although these values and corresponding soil plasticity may have been affected by the organic content of the samples.

In borehole 11-3, a 0.7 m thick layer of brown fine sand with some silt was encountered between the buried topsoil and the organic soil deposit at about 1.7 m depth (i.e., Elevation 188.7 m). In this same borehole, a 0.2 m thick grey sand layer was encountered between the organic soil deposit and the silty clay at about 5.3 m depth (i.e., Elevation 185.1 m). Standard penetration test N values for the sand layers in borehole 11-3 of about 1 to 2 blows per 0.3 m of penetration indicates a generally very loose state of packing.

In borehole 12-102, a layer that was more easily penetrated during the coring process was encountered from 5.0 to 5.4 m depth. The colour and composition of the wash water return indicated that this layer is likely the organic soil layer that was encountered in the other nearby boreholes in this area.

4.2.3 Silty Clay

A deposit of grey silty clay was encountered on the south side of the existing bridge and within the original Salmon River channel to the west of the existing bridge below the organic soil deposit in boreholes 11-3, 11-4 and 12-105 to 12-107. This deposit was encountered at Elevations ranging from 182.8 m and 190.4 m and extends down to depths ranging from 5.0 to 9.2 m (i.e., Elevations 186.5 to 180.6 m) in boreholes 11-3, 11-4 and 12-103 to 12-107. In boreholes 12-101 and 11-2 located on the north side of the east abutment, a deposit of silty clay that has been weathered to a grey brown crust was also encountered below the topsoil or fill.

The results of grain size distribution tests completed on ten selected samples of the silty clay are shown on Figures A2 and B2. Atterberg limits tests were also completed on these ten selected sample and on two additional samples of this deposit, which were also subjected to two oedometer consolidation tests, and measured a plastic limit ranging from 11 to 19 percent, a liquid limit ranging from 25 to 49 percent, and a plasticity index of 9 to 31 percent. These test results, which are also plotted on the plasticity charts on Figures A5 and B4, indicate that this deposit consists of clayey silt to silty clay of low to intermediate plasticity.

The measured water content on thirteen select samples of the silty clay ranges from approximately 28 to 53 percent which were at or slightly above the corresponding liquid limit values.

The measured SPT “N” values within the silty clay deposit generally only required the weight of the hammer for 0.3 m of penetration. The results of insitu vane testing in this material gave undrained shear strengths ranging from 9 to 34 kPa, indicating a very soft to firm consistency. Insitu vane testing carried out on remoulded grey silty clay gave undrained shear strengths ranging from 3 to 4 kPa, reflecting a sensitive material (i.e., sensitivity ratio of 4 to 7).

In borehole 11-2 and 12-101, the silty clay was encountered at about Elevation 190.3 m and 191.4 m, respectively. At these locations, the silty clay is at a higher elevation than at the other boreholes, and has been weathered to a grey-brown stiff to very stiff crust. The N values measured in this material were 5 and 7 blows per 0.3 m.

Oedometer consolidation testing was also carried out on two intact samples of the grey silty clay, and the results of this testing, which are provided on Figures A6 to A13, are summarized in the table below.

Borehole/ Sample Number	Sample Depth/Elevation (m)	Unit Weight (kN/m^3)	σ_P' (kPa)	σ_{VO}' (kPa)	$\sigma_P' - \sigma_{vo}'$ (kPa)	Cc	Cr	e_o	OCR
12-105A / 1	6.9 / 183.7	18.4	75	60	15	0.285	0.019	1.03	1.3
12-107 / 5	4.7 / 186.7	18.5	60	55	5	0.319	0.029	1.04	1.1

Notes:

σ_P' - Apparent preconsolidation pressure

σ_{VO}' - Computed existing vertical effective stress

Cc - Compression index

Cr - Recompression index

e_o - Initial void ratio

OCR - Overconsolidation ratio

The consolidation tests indicate an apparent preconsolidation pressure that is approximately 60 to 75 kPa, and only slightly in excess of the calculated existing effective stresses within the grey silty clay at the depth of the samples. Therefore, the grey silty clay is very close to being normally consolidated with an OCR of 1.1 and 1.3.

4.2.4 Sandy Gravel, Sand and Gravel, Silty Sand and Sand

Grey sandy gravel to grey brown silty sand was encountered below the silty clay at depths ranging from about 6.3 to 11.9 m in boreholes 11-3, 11-4, 12-103, 12-104, 12-105, 12-106, and 12-107 (i.e., Elevations 185.2 m to 179.1 m, respectively).

Standard penetration test N values for this material ranging from 4 to greater than 100 blows per 0.3 m of penetration indicate a loose to very dense state of packing, although the higher N values could also reflect the presence of cobbles and boulders, rather than the state of packing of the soil matrix. Refusal to advancement of the augers was encountered in borehole 12-104 and frequent refusal to advancement of the sampler in borehole 11-3, on cobbles and boulders in the deposit, and in some instances rotary diamond drilling/coring techniques were required to advance the borehole within the sand and gravel deposit.

4.2.5 Gravelly Sandy Silt Till

A deposit of gravelly sandy silt till was encountered in Borehole 11-2 below the stiff silty clay crust. The 1.3 m thick deposit of till was encountered at about 1.5 m depth (i.e., Elevation 189.8 m). This till deposit consists of gravelly sandy silt with trace clay and cobbles. The result of grain size distribution test carried out on one selected sample of the till is shown on Figure B3.

A 0.1 m diameter cobble was encountered between the organic soil deposit and the bedrock surface in Borehole 11-1. This cobble could be part of a very thin, discontinuous glacial till veneer above the bedrock surface in this area.

Two measured SPT "N" values within the till gave 16 and 35 blows per 0.3 m of penetration suggesting a compact to dense state of packing.

4.2.6 Refusal and Bedrock

Sampler refusal was encountered at Elevations 179.7 m (10.8 m depth) in borehole 11-3. Practical refusal to advancement of the augers was encountered at Elevation 183.8 m (7.6 m depth) in borehole 12-107. Refusal may indicate the bedrock surface; however, it could also represent cobbles and/or boulders within the silty sand to sand and gravel deposit. Bedrock was encountered beneath the till and/or sandy deposits and cored for 1.5 m to 4.2 m depth, in all of the remaining boreholes.

The following table summarizes the bedrock surface depths and elevations as encountered at the nine borehole locations where bedrock was cored. Refusal was obtained at the two remaining boreholes BH 11-3 and BH 12-107.

Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
11-1	191.6	3.4	188.2
11-2	191.3	2.8	188.5
11-4	190.7	8.5	182.2
12-101	191.5	1.4	190.1
12-102	191.5	5.4	186.1
12-103	191.3	11.2	180.1
12-104	191.0	11.9	179.1
12-105	190.4	9.6	180.8
12-106	191.5	6.4	185.1

The bedrock encountered in the boreholes typically consists of grey to greenish grey gneiss or grey schist. The gneiss bedrock varies between granitic gneiss to calcareous granitic gneiss to calcareous gneiss. In addition, the schist varies between mica schist to quartz mica schist. In borehole 12-106, the quartz mica schist was underlain by quartzite bedrock, which was encountered at about 9.5 m depth (i.e., Elevation 182.1 m). The bedrock is slightly weathered to fresh and typically very strong.

The Rock Quality Designation (RQD) values measured on recovered bedrock core samples ranged from about 68 to 100 percent, indicating excellent quality rock.

Laboratory point load index testing was carried out, axially, on selected specimens from the bedrock core. Laboratory unconfined compressive strength testing was also carried out on selected specimens of the bedrock core. The results for the existing bridge abutment boreholes are summarized on Figures A14 and A15. The calculated compressive strengths from the point load index testing on nine bedrock core samples range from 64 MPa to 244 MPa. Four unconfined compressive strength tests indicate values ranging from about 41 MPa to 92 MPa at the bridge abutments.

4.3 Groundwater Conditions

The groundwater level in the piezometer in borehole 11-3 was measured on April 16, 2012, and the water levels in the piezometers in boreholes 11-3 and 12-106 were measured on January 15, 2013. The observed groundwater level is summarized in the table below:

Borehole Number	Existing Ground Surface Elevation (m)	December 13, 2012		April 16, 2012		January 15, 2013	
		Water Level Depth (m)	Water Level Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)
11-3	190.5	1.7*	188.7*	1.8	188.6	1.5	189.0
12-106	191.5	NA	NA	NA	NA	1.7	189.8

Note: * Water level taken in the open borehole in the morning, prior to the start of the daily drilling fieldwork.

Surveying of the rock outcrop areas on December 12, 2011 established a level of 189.05 m, some 100 mm to 150 mm above the river level at that time.

The high water level in the Salmon River at the site is reported as 188.92.

It should be noted that groundwater levels in the area are subject to fluctuations both seasonally and with precipitation events.

5.0 CLOSURE

This report was prepared by Mr. Nicolas LeBlanc, P.Eng. and reviewed by Mr. Fintan Heffernan, P.Eng., Golder's Designated MTO Contact for this project, who conducted a technical and independent quality control review of the report.

GOLDER ASSOCIATES LTD.

Nicolas LeBlanc, P.Eng.
Geotechnical Engineer

Fintan J Heffernan, P.Eng.
Designated MTO Foundations Contact

WAM/NRL/FJH/bg

n:\active\2011\1111 -mississauga\11-1111-0017-1000 salmon river\foundations - detailed design (ph 3000)\report and memo\final to mto\11-1111-0017-3000 rpt-002 foundations detailed design may 2013.docx

PART B

Foundation Design
Proposed Bridge Replacement
Salmon River Bridge
Highway 7
G.W.P. 4034-09-00

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed replacement of the existing Highway 7 Salmon River Bridge. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this subsurface investigation and our previous subsurface investigation. The interpretation and recommendations provided are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations.

Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Bridge Foundation Options

Based on the preliminary plans completed to date for the Highway 7 Salmon River Bridge replacement, it is understood that three alignments, all with a 1.35 m grade raise from the existing alignment grades, are currently being proposed. However, we understand that there may be a further minor increase in this grade. The three proposed alignments are as follows:

1) The centre line of the new bridge would remain on the current alignment (Option 1);

2) The centre line of the new bridge would be moved 7 m to the north (Option 2); and,

3) The centre line of the new bridge would be moved 4.2 m to the south (Option 3).

It is understood that the limited space on the north side of the bridge due to nearby private property would require the use of retaining walls for an alignment that remains on the current alignment (Option 1) or that is moved 7 m to the north (Option 2).

In addition to the above 3 options for the bridge location, consideration had been given to either a one span, or a two span bridge during the preliminary design stage. The use of a two span bridge would allow the bridge girders to be thinner, and the option of re-using the existing center pier for the new bridge. The preliminary design concluded the single span bridge configuration is more desirable and the two span bridge configuration is no longer being considered.

At the initial stages of this project, the use of a detour located on the south side of the existing bridge that would include a temporary modular bridge was being considered. Following the fieldwork, it was determined, considering the deep compressible organic soil and compressible clay on this south alignment, to be impractical and expensive to utilize a modular bridge. Therefore, the current preferred option is to replace the existing bridge in two halves, while utilizing traffic lights on Highway 7 to allow traffic in either direction on the opposite half of the bridge.

The existing structure consists of a two-span rigid concrete frame structure, with two arched decks, measuring about 29 m in overall length and 9 m in width. The construction drawings of the existing structure, dated March 1952, indicates that the east abutment, the north half of the west abutment, and the centre pier are supported by spread footings on bedrock. However, the construction drawings also indicate that the south half of the west abutment is supported on a wedge of rock fill, which increases in thickness towards the south. The existing abutments are indicated to be supported on 1.1 m wide spread footings founded at varying elevations on the bedrock surface, with the exception of the south portion of the west abutment which is supported on 2.4 m wide spread footings founded on the rock fill wedge. We understand that there has been some settlement related to this footing on rock fill. A large crack which is located in the center of the western span and parallel to the bridge was noted during the fieldwork, and is suspected to be the result of differential settlements between the footings on bedrock and those on rock fill. The existing bridge also has concrete gravity retaining wing walls which are connected by flexible connections to the end of the abutments, and run parallel to the roadway.

Based on the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments for the new Highway 7 Salmon River Bridge. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 1 for the new Salmon River Bridge following the text of this report.

- **Strip or spread footings founded on the bedrock:** Strip or spread footings are feasible for support of the new abutments for the bridge with a 7 m shift to the north (Option 2), and potentially for a new bridge at its current location (Option 1). However, the footings would have to be sloped or stepped to follow the existing bedrock surface. In addition, the use of spread footings on the south portion of the west abutment for the Option 1 alignment is not practical due to the deeper bedrock in this area. The use of spread footings would require excavation to depths of approximately 3 m to 5 m relative to the existing roadway grades at the site for Option 1, and to depths of approximately 0 m to 5 m relative to the existing roadway grades at the site for Option 2. Temporary excavation support would likely be required both adjacent to Highway 7 and along the creek channel. It is not considered feasible to use spread footings within the overburden soils on the south side of the bridge. For all three alignment options, strip or spread footings are also feasible for the support of the associated wing walls or retaining walls on the north side of Highway 7 for both the east and west sides of the proposed bridge, but only within about 10 m (or less for a 4.2 m centre line shift to the south) from the existing west abutment. Beyond that approximate 10 m and on the south side of the proposed new bridge, the depth to bedrock becomes too great for the practical use of strip or spread footings;
- **Driven steel H-piles:** Driven steel H-piles are feasible and suitable for support of the abutments of the new bridge and the associated wing walls or retaining walls. There is a minor risk associated with the piles "hanging up" on cobbles or boulders within the glacial till and the sand/gravel soils, and for variable pile lengths. Titus-type Injector points or equivalent are recommended to, not only penetrated the steeply sloping bedrock in this site area, but to also protect piles during driving in the anticipated hard/very dense ground conditions above the bedrock surface. The Titus-type ejector points can be used for rock slopes up to 30-35 degrees which is about the maximum overall slope at the abutment areas. Locally steeper sloping bedrock could cause some piles to deflect and/or pile damage, requiring additional piles; and,

- **Caissons:** Caissons socketed into the bedrock are feasible for the new bridge, retaining walls or wing walls at this site given the relatively shallow depth to bedrock at the abutments. However, the gneiss/schist/ quartzite bedrock are very strong and abrasive, and drilling of the rock socket would be difficult. As such, steel H-piles with injector points are considered more favorable/practical than caissons for this project.

The following sections provide recommendations for spread footing foundations and driven steel H-pile or steel pipe pile foundations to support the proposed new bridge, and associated wing walls or retaining walls. However, based on the above considerations, the preferred option from a geotechnical/foundations perspective is for the Highway 7 alignment to be moved 7 m to the north, and to support the new bridge abutments entirely on spread footings on bedrock. If the current Highway 7 alignment is maintained or moved towards the south, then a portion or all of the abutments may require the use of deep foundations. Because some of the piles at this location will be relatively short (i.e., less than 3 m) to mobilize sufficient lateral support, these short piles would have to be pre-drilled and socketed into the bedrock. Because of the presence of boulders in the till, as well as the deeper sand and gravel deposit, driven steel H-pile foundations are preferred for both the new bridge and the proposed retaining wall.

6.3 Retaining Wall and Reinforced Earth Slope Options

For Option 1, no centre line shift, an approximate 2 m high retaining wall is currently proposed on the north side of Highway 7 to accommodate the raised highway embankment in this area. For Option 2, a 7 m centre line shift to the north, the retaining wall would be approximately 4 m high on the north side of Highway 7.

Due to the proposed grade raise and the presence of the compressible organic soils and silty clay deposit, there is the potential for significant differential ground settlements at this site (50-100 mm) (see Section 6.7). From that perspective, the following wall types and reinforced earth options are considered appropriate:

- **Concrete Retaining Wall on Shallow and Deep Foundations:** A concrete retaining wall supported on shallow foundations (concrete strip footings) is geotechnically feasible for the proposed retaining walls where shallow bedrock or dense to compact sand and gravel/glacial till is present. Further to the west of the proposed new bridge where the depth to the competent soils and bedrock is greater, and where compressible soils are present, the use of deep foundations, such as steel H-piles, would be required for a concrete retaining wall. Temporary excavations to allow for construction of the spread footings and pile caps would be required and depending on the proximity of the excavation to adjacent property limits, Salmon River, existing structures and the Highway 7, a temporary protection system and/or cofferdams are expected to be required;

- **Reinforced Soil System (RSS) Wall:** The conditions at this site are not considered suitable for retained soil systems (RSS), since the settlements would be beyond the acceptable limits and could result in the formation of gaps between the RSS wall facing panels. Alternatively, the area of the retaining wall could be pre-loaded or surcharged to reduce the magnitude of post-construction settlements to an acceptable level for the use of RSS walls. This option will be able to accommodate some minor long term settlements along the wall length, and as such, some slip joints are recommended in the RSS wall facing panels. The excavation for this option would not be as deep as for shallow foundations. However, excavation must still be completed within the zone of the reinforced soil mass, which width is typically about 80 per cent of the wall height. As with the concrete retaining wall option, a temporary protection system is expected to be required along the north side of Highway 7 in the existing embankment side slope to facilitate excavation and construction of the reinforced soil mass;

- **Mechanically Stabilized Embankment (MSE):** A flexible type retaining wall such as a Mechanically Stabilized Embankment (MSE) is considered feasible for this site. This type of wall construction is typically supported on a granular pad. The wall facing could be either concrete blocks, rock filled galvanized steel baskets (similar to gabions), or vegetated at very steep slope angles (i.e., almost vertical for concrete blocks and typically 3H:8V to 1H:1V with steel baskets or vegetated slopes). Given the flexible nature of this type of construction, this option could accommodate the expected differential settlements from the proposed grade raise. In addition, the 1.8 m frost protection requirement can be waived for an MSE wall. MSE construction is manufacturer dependent and, as such, the manufacturer's design should be followed including the type of foundation support that is required. Furthermore, the resistance to sliding at the base of the wall between the existing granular fill and the backfill material should be calculated using an unfactored friction angle of 25 degrees. The manufacturer's guidelines should be carried out with respect to the preparation of the granular base and the wall backfill;

- **Soldier Pile and Concrete Panel Walls:** A soldier pile and concrete panel wall may be considered for the proposed retaining structure. This type of wall is generally more advantageous in "top-down" construction applications (i.e., as part of a cut, rather than a new fill construction). However, this option would minimize excavation into the existing embankment side slopes on the north side of Highway 7, and would allow fill placement to proceed above the existing ground surface behind the proposed wall alignment. This type of wall construction would require pre-drilling of the soldier piles in areas where the bedrock is shallow. Some cost savings could be achieved if the bridge piling driving contractor also installs the retaining wall soldier piles; and,

- **Gabion Walls:** The nature of the soil conditions at this site and the large magnitude of expected settlements requires a consideration of flexible type retaining structures. Construction of a gabion wall is geotechnically feasible at this site. Gabion walls require the least amount of space behind the wall. Temporary shoring should not be necessary if this wall type is constructed. Gabion walls do not require an embedment depth equivalent to the frost depth provided it is founded on a granular pad of 300 mm compacted thickness, and the foundations of nearby structures have adequate embedment to provide a stable structure. Advantages of gabion walls compared to more rigid structures include the ability to accommodate differential settlements, and are free draining provided a suitable filter is placed behind the wall. Gabion walls can be constructed relatively quickly with minimal equipment and materials. The life expectancy of a gabion wall can be extended by utilizing PVC-coated galvanized steel baskets. Gabion walls are to be constructed in accordance with OPSS 512 if greater than 2 m in height or SP 512 S03 if less than 2 m in height.

6.4 Shallow Foundations

Spread footing founded on the bedrock may be designed based on a factored geotechnical resistance at ULS resistance of 10 MPa. However, this full ULS valve would not be needed for this bridge and footings as nominal size should be used. SLS resistances do not apply to the design of footings on the gneiss/schist/quartzite bedrock since the SLS resistance for 25 mm of settlement is greater than the factored geotechnical resistance at ULS.

These values will need to be confirmed based on the actual footing size, geometry, location and founding level. The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Canadian Highway Bridge Design Code (CHBDC).

A working slab of 20 MPa concrete and 100 mm thickness should be placed on the abutment footing areas. A granular pad of OPSS Granular A or Granular B Type II some 300 mm in thickness should be placed below the retaining wall footings. An NSSP is included in Appendix C.

6.4.1 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on a concrete working slab that is cast on top of the bedrock or a compacted granular pad, the coefficient of friction, tan δ or tan ϕ', can be taken as follows:

- Cast-in-place footing to concrete working slab: tan $\delta = 0.70$
- Cast-in-place footing to compacted granular pad: tan $\phi' = 0.55$
- Cast-in-place concrete working slab to unweathered/sound bedrock: tan $\delta = 0.70$
- Concrete working slab to fine sand (at west abutment) tan $\phi' = 0.45$
- Concrete working slab to stiff clay (at east abutment) tan $\phi' = 0.40$

6.5 Steel H-Pile Foundations

Steel H-piles driven through the organic soils, clay and granular deposits to found on the gneiss/schist/quartzite bedrock may be used for support of the new bridge abutments and the new retaining walls/wing walls, if required.

The sand and gravel, rock fill, and the till deposits that overly the bedrock at this site contain cobbles and boulders. In addition, the bedrock in this area is known to be sloped, and could be steeply sloping in localized areas. The piles should therefore be provided with Titus-type injector bearing points or equivalent to protect the pile tips during driving through the cobbles and boulders in the overburden and for seating on the sloping bedrock.

The piles should be designed to be founded on bedrock. However some of the piles could have difficulty penetrating to depth and could “hang up” at shallower depth in the overburden deposits due to the presence of cobbles and boulders. In that case pre-drilling of the overburden could be considered. Alternatively a reduced capacity may apply to these piles, as discussed below.

Based on the bedrock depths, an integral or semi-integral abutment design could be considered for a new bridge that is shifted 4.2 m to the south (Option 3). To accommodate an integral abutment, the piles may need to be socketted into the bedrock to provide the minimum pile length of about 5 m required for this type of abutment. It should be noted that drilling into the gneiss/schist/quartzite bedrock would be difficult. Alternatively, the pile cap may be perched in the embankment fill to provide the required 5 m pile length above the bedrock surface. Semi-integral and conventional abutments are also considered feasible at this location. However, the abutment type (i.e., conventional, semi-integral or integral) should be consistent at both abutment locations.

6.5.1 Axial Geotechnical Resistance

The following factored axial resistances at ULS may be assumed for design of piles that are successfully driven to found on the bedrock:

Pile Size	Factored ULS Resistance (kN)
HP 310 x 110	2,000
HP 360 x 132	2,400
HP 360 x 152	2,750

The above values represent structural limitations for the piles rather than geotechnical limitations.

SLS resistances do not apply to piles founded on the gneiss/schist/quartzite bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. ULS conditions will govern for this foundation type, providing the piles are successfully driven to bedrock. The depth to bedrock and bedrock surface elevation as encountered in the recent borehole investigation at the proposed abutment locations is summarized below.

Foundation Unit	Borehole Number	Existing Ground Surface Level Elevation(m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
West Abutment	11-1	191.7	3.4	188.3
West Abutment	12-105	190.4	9.6	180.8
West Abutment	11-3	190.5	> 10.8[1]	< 179.7[1]
East Abutment	12-101	191.5	1.4	190.1
East Abutment	11-2	191.4	2.8	188.6
East Abutment	12-106	191.5	6.4	185.1
East Abutment	11-4	190.7	8.5	182.2
25 m West of West Abutment and East End of Retaining Wall	12-102	191.5	5.4	186.1
Centre of Retaining Wall	12-103	191.3	11.2	180.1
West End of Retaining Wall	12-104	191.0	11.9	179.1

Note: [1] Bedrock not encountered in boreholes 11-3 and 12-107.

As discussed previously, it is expected that some of the piles may not fully penetrate the cobbly overburden deposits to reach the bedrock surface; these piles could "hang up" at shallower depth in these materials. In addition, piles driven on the south portion of the west abutment of the existing bridge will likely encounter the previously placed rock fill in this area below the existing bridge foundations, as shown on the old contract drawings, and could also "hang up" on the larger rocks. In those cases, predrilling of the overburden could be considered. A Non-Standard Special Provision (NSSP) is provided in Appendix C to address this issue.

Alternatively, the piles could be designed for a reduced capacity for the piles that have stopped in native overburden. The ULS factored axial resistance of these piles will depend on the depth to which they penetrate and the set that is achieved. As a preliminary guideline, for HP 310 x 110 piles founded within the sand and gravel or glacial till soils, a ULS factored geotechnical resistance of 1,600 kN may be used. The axial resistance at SLS for 25 mm of settlement would likely be in the order of 1,200 kN. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The set criteria should be established using the Hiley formula, using a resistance factor of 0.5 on the factored axial resistance. For this situation, the piles should be driven in accordance with Standard SS 103-11, using an ultimate capacity of 3,200 kN per pile.

If short piles with less than 3 m in length area used, the piles should have a minimum bedrock embedment length of 1.0 m to provide adequate lateral support. It should be noted that drilling into the hard gneiss/schist/ quartzite bedrock would be difficult.

Pile installation should be in accordance with OPSS 903. The drawings should incorporate the appropriate note stating that the piles should be equipped with bearing points and should be driven to bedrock. For piles driven to refusal on bedrock, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and to then gradually increase the energy over a series of blows to seat the pile. An example Non-Standard Special Provision (NSSP) is provided in Appendix C for a HP 310 x 110 pile to address a reduced pile driving energy when bedrock is reached. A revised NSSP can be provided should a different pile size be chosen for this project.

Depending on the chosen alignment and the final bridge configuration, the settlements induced during and over time following construction of the approach embankments are expected to induce downdrag loads into the deep foundation systems. Depending on the sequence of construction and bridge location, these downdrag loads could be up to 370 kN for a HP 360 x 152 steel driven pile, and 320 kN for a HP 310 x 110 steel driven pile. These maximum downdrag loads were calculated at the location of borehole 12-104 were the compressible overburden is the thickest (i.e., about 10.4 m). Lower downdrag loads are expected where the compressible overburden is not as thick. It is recommended that at final design there is a liaison on the downdrag forces when the alignment and grades have been determined.

6.5.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. Alternatively, in the case of integral abutments the resistance to lateral loading will have to be derived from the soil in front of the piles, and it may be assumed that this resistance will be nearly the same for vertical and inclined piles as indicated in Section C6.8.7.2 of the Commentary to the CHBDC.

The SLS geotechnical response of the soil in front of the piles under lateral loading may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the equations given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (3rd Edition).

For cohesionless soils:

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

Where: n_h is the constant of horizontal subgrade reaction, as given below;

z is the depth (m); and,

B is the pile diameter/width (m).

The following values of n_h may be assumed in the structural analysis.

Location	Elevation (m)	Soil Type	n_h (MN/m^3)
East and West Abutment	PCL[1] to 189.0	Loose granular fill above water table	2.2
	Note 2	Granular fill below water table	1.3
	Note 2	Loose organic granular deposits	1.3
	Note 2	Soft clay deposit	See below
	Note 2	Compact sand and gravel deposit	4.4

Notes: [1] Pile Cap Level.
[2] Elevations to be determined once a final alignment is chosen.

For clay soils:

$$k_h = \frac{67 \cdot c_u}{d}$$

Where: c_u is the undrained shear strength of the clay, use 20 kPa; and,

d is the pile diameter (or width) (m).

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor as follows:

Pile Spacing in Direction of Loading (d = Pile Diameter)	Reduction Factor
8d	1.0
6d	0.7
4d	0.4
3d	0.25

For establishing the ULS factored *structural* resistance, the shear force and bending moment distribution in the piles under factored loading can be established using the procedures and parameters given above for evaluating the SLS response of the pile.

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure theory as outlined in Section C6.8.7 of the *Commentary to the CHBDC*. For individual piles in cohesionless soils (i.e., loose sandy and silty fill) the passive resistance may be assumed to act over the pile shaft to a depth equal to six pile diameters below the underside of the pile cap and may be calculated as:

Above the water table: $P_p(z) = 3\ d\ K_p\ \gamma\ z$

Below the water table: $P_p(z) = 3dK_p\ \gamma\ D_w + 3dK_p\ (z - D_w)\ (\gamma - \gamma_w)$

Where:

- $P_p(z)$ is the ULS lateral resistance at depth 'z' below ground surface (kN/m);
- γ is average unit weight of overlying soil, use 18 kN/m^3;
- K_p is the coefficient of passive earth pressure;
- D_w is the depth to groundwater table below ground surface (m);
- γ_w is the unit weight of water, use 9.8 kN/m^3; and,
- d is the pile diameter (m).

The ULS lateral resistance of a pile group may be estimated as the sum of the individual pile resistances across the face of the pile group, perpendicular to the direction of the applied lateral force.

The ULS resistances obtained using the above parameters represent unfactored values; in accordance with the *CHBDC*, a resistance factor of 0.5 is to be applied in calculating the horizontal resistance.

For design purposes, the ULS *geotechnical* resistance can also be estimated using the "Assessed Horizontal Passive Resistance and Geotechnical Reaction at SLS" provided in Table C6.4 of the *Commentary* to the CHBDC. On that basis, a maximum lateral resistance of 130 kN at ULS (unfactored), and a maximum lateral resistance of 40 kN at SLS (for 10 mm of horizontal deflection at pile cap level) is recommended for HP 310 x 110 piles in the sandy/silty fill.

6.5.3 Frost Protection

The pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection purposes.

6.6 Seismic Site Response Classification

The site falls within the Western Quebec Seismic Zone (WQSZ) according to the geological survey of Canada, but is located in a less seismically active area near the limits with the Southern Great Lakes Seismic Zone (SGLSZ). The WQSZ constitutes a large area that extends from Montreal to Témiscaming, and encompasses the Ottawa River Valley area. Within the WQSZ recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montreal-Maniwaki Axis. Historical seismicity within the WQSZ from 1900 to 2000 includes the 1935 Témiscaming event which had a magnitude (i.e., a measure of the intensity of the earthquake) of 6.2 and the 1944 Cornwall-Massena event which had a magnitude of 5.6 and most recently the 2010 Echo Lake Québec event which had a magnitude of 5.0. In comparison to other seismically active areas in the world (e.g., California, Japan, New Zealand), the frequency of earthquake activity within the WQSZ is significantly lower but there still exists the potential for significant earthquake events to be generated.

Given the above, a seismic Site Coefficient also needs to be assigned, as given in Section 6.4.1., to be used by the structural designer.

6.6.1 Site Coefficient

In accordance with Section 4.4.6 of the CHBDC, the Site Coefficient, S, for this site will be dependant on the chosen alignment.

For an alignment that is moved 7 m to the north from the current alignment where less than 9 m of predominantly loose fill, soft clays and loose organic silts and sands are present, the Site Coefficient, S, may be taken as 1.2, consistent with Soil Profile Type II, may be taken for seismic design purposes.

For the existing alignment or an alignment that is moved to the south, more than 9 m of predominantly loose fill soft clays and loose organic silts and sands are present, and the Site Coefficient, S, should be taken as 1.5 for seismic design purposes, consistent with Soil Profile Type III.

6.7 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems, wing walls and retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the abutment stems and retaining walls:

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II but with less than 5 percent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 and 3121.150;
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS 501. Other surcharge loadings should be accounted for in the design, as required;
- The granular fill may be placed either in a zone with width equal to at least 1.8 m behind the back of the abutment stem [Case (a) in Figure C6.20 of the Commentary to the CHBDC] or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing [Case (b) in Figure C6.20 of the Commentary to the CHBDC]. Consideration should be given to placing the granular fill behind the abutments first before placing any embankment rock fill above the granular fill. If the granular fill is placed over the rock fill, a separation layer will be required;

- For Case (a), the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of select subgrade material or rock fill:

	Earth Fill	**Rock Fill**
Soil Unit Weight:	20 kN/m^3	19 kN/m^3
Coefficients of Static Lateral Earth Pressure: Active, K_a At rest, K_o	 0.35 0.50	 0.22 0.36

- For Case (b), the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

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

If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

- Seismic loading will result in increased lateral earth pressures acting on the walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. The site-specific zonal acceleration ratio for this site is 0.1. Based on experience, for the subsurface conditions at this site, no significant amplification of the ground motion is expected. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.1; and,

- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which do not allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient is taken as 1.5 times the zonal acceleration ratio (i.e., k_h = 0.15). For structures which allow lateral yielding), k_h is taken as 0.5 times the zonal acceleration ratio (i.e., k_h = 0.05).

The following seismic active pressure coefficients (K_{AE}) for the two backfill cases [Case (a) and Case (b)] may be used in design. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

Seismic Active Pressure Coefficients, K_{AE}

	Case (a)		Case (b)	
	Earth Fill	**Rock Fill**	**Granular 'A'**	**Granular 'B' Type II**
Yielding wall	0.35	0.24	0.29	0.29
Non-yielding wall	0.49	0.35	0.41	0.41

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.1. This corresponds to displacements of up to approximately 25 mm at this site; and,
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a \gamma d + (K_{AE} - K_a) \gamma (H-d)$$

Where: $\sigma_h(d)$ is the lateral earth pressure at depth, d, (kPa);

K_a is the static active earth pressure coefficient;

K_{AE} is the seismic active earth pressure coefficient;

γ is the unit weight of the backfill soil (kN/m^3), as given previously;

d is the depth below the top of the wall (m); and,

H is the total height of the wall (m).

6.8 Approach Embankment Design and Construction

The approach embankments for the new bridge, as currently proposed, will be constructed with 2H:1V side slopes above the existing approach embankments. The proposed profile indicates that the embankments will be up to about 1.4 m in grade raise above the current ground surface. The maximum embankment height from the top of the new embankment to the toe of the existing embankment will be about 3.5 m.

The maximum embankment heights noted above are near the abutment locations where the ground surface is lower than along the remainder of the alignment. The maximum embankment height will be up to about 3.5 m at about 8 m behind the existing east and west abutments.

Based on the borehole results, the embankment widening subgrade soils will consist of granular fill over rock fill which are in turn underlain by loose to very loose organic silty sand to clayey silt, over soft to firm silty clay, over a compact to very dense sand and gravel deposit over bedrock.

The clay soils at this site have a low to intermediate plasticity, and are soft and compressible. In addition, the organic silty sands to clayey silts are loose and very compressible. The settlement analyses (discussed in more detail below) indicate that the consolidation settlements due to the approach embankment loading will be excessive and that settlement mitigation measures such as preloading and/or surcharging and/or light-weight fill (i.e., expanded polystyrene) will need to be considered.

The stability analyses (also discussed in more detail below) indicate that the full height embankments proposed at this site will have static factors of safety of more than 1.3, as shown on the Sigma/W output in Appendix D, and a seismic factor of safety of more than 1.1 against global instability.

The settlement mitigation measures that may be undertaken at this site are discussed in Section 6.7.4.

6.8.1 Subgrade Preparation and Embankment Construction

It is recommended that all topsoil and softened/loosened soils be stripped from below the approach embankment areas, to minimize differential settlement between the existing and widened portions of the approach embankments.

Embankment fill should be placed in regular lifts with a loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the material's Standard Proctor maximum dry density.

The final lift prior to placement of the granular subbase and base courses should be compacted to 100 percent of the Standard Proctor maximum dry density in accordance with MTO Special Provision 105S10. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

To reduce surface water erosion on the embankment side slopes, placement of topsoil and seeding or pegged sod is recommended.

6.8.2 Approach Embankment Stability

6.8.2.1 *Static Slope Stability*

The slope stability analyses for this embankment configuration were carried out using the commercially available program SLOPE/W produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis.

The table below summarizes the soil parameters that have been used in the stability analyses. The undrained shear strengths used in the analyses are based on the corrected undrained shear strength (based on Bjerrum's correction method) from insitu vane testing.

Soil Conditions	Bulk Unit Weight (kN/m^3)	Effective Friction Angle	Undrained Shear Strength (kPa)
Pavement Structure	21	35°	-
Embankment Fill	20	30°	-
Organic Silty Sand To Clayey Silt	12.5	35°	-
East of Salmon River – Soft Silty Clay	18.4	0°	17
West of Salmon River – Soft Silty Clay	18.4	0°	15
Very Dense to Compact Silt Till / Sand and Gravel Deposit	18	32°	-

The results of the slope stability analyses using these parameters indicate that the proposed approach embankments analyzed at Station 22+620, Station 22+660 (i.e., at about section B-B'), and Station 22+720 (i.e., at about section C-C') less than about 3.5 m in total height (i.e., about 1.4 m above the existing roadway), with side slopes orientated at 2H:1V, will have a factor of safety of greater than 1.3 against deep-seated global instability.

6.8.2.2 *Seismic Slope Stability*

The seismic slope stability evaluations were carried out assuming that the design earthquake would correspond to an event with a 5 percent probability of occurrence in 50 years (i.e., 7.5 percent probability of occurrence in 75 years). This design criteria corresponds to the design earthquake in the CHBDC. The design ground accelerations associated with an earthquake with a 5 percent probability of accidence in 50 years result from the cumulative contributions of a variety of earthquake magnitudes occurring at various distances from the site. Consistent with this criteria, a “firm ground” peak horizontal ground acceleration (PHGA) of 0.13 g (g=acceleration due to gravity) was selected.

In consideration of the potential for ground accelerations to be generated during the design earthquake at this site, the seismic performance of these slopes was assessed using a pseudo-static slope stability analysis. For the seismic stability analyses, the seismic loads imposed on a slope are modelled in a simplified manner by applying a horizontal “pseudo-static” force to the soil mass. The “pseudo-static” force, F_s, is calculated as:

$$F_s = k_s \times M$$

Where: k_s Is horizontal seismic coefficient, taken as 0.065; and,

M Is mass of soil contained within the failure surface.

If the factor of safety obtained using the pseudo-static approach is greater than 1.1, the slope is expected to perform acceptably. The horizontal seismic coefficient which results in a factor of safety of 1.0 is commonly referred to as the yield acceleration, k_y.

All the slopes analyzed had a pseudo-static factor of safety of more than 1.1.

6.8.3 Approach Embankment Settlement

Settlement analyses for the anticipated soil conditions below the widened and raised approach embankments were carried out using both hand calculations and the commercially available computer program *SIGMA/W* produced by Geo Slope International Ltd., using a finite element analysis with estimated elastic deformation moduli and the Soft Clay model parameters as given in the table below. These parameters were based on correlations with the SPT "N" values, results from consolidation tests on samples of the soft silty clay and engineering judgement from experience with similar soils in this region of Ontario (Das, 2007). The consolidation parameters for the organic soils were chosen based on the correlations provided in Mesri and Ajlouni (2007).

Soil Conditions	Model	Bulk Unit Weight (kN/m^3)	Elastic Modulus (MPa)	Cc	Cr	OCR	Initial Void Ratio
New Pavement Structure	Linear Elastic	22	80	-	-	-	-
New Embankment Fill	Linear Elastic	20	12	-	-	-	-
Existing Embankment Fill	Linear Elastic	20	12	-	-	-	-
Soft Organic Soils – East of Salmon River	Soft Clay (MCC)	18.5	-	0.330	0.017	1.0	0.87
Soft Organic Soils – West of Salmon River	Soft Clay (MCC)	12.5	-	1.890	0.095	1.0	5.01
Soft to Firm Silty Clay	Soft Clay (MCC)	18.4	-	0.319	0.029	1.0	1.04
Dense Sand and Gravel / Glacial Till	Linear Elastic	20	25	-	-	-	-
Bedrock	Linear Elastic	-	Incompressible	-	-	-	-

Based on this assessment, the settlement of the foundation soils under the proposed 1.35 m grade raise for Highway 7 approach embankments will be dependent on the chosen alignment option. Because of the sloping bedrock in this area, the highest magnitude settlements will occur on the south side of Highway 7, and for the 4.2 m centre line shift to the south (Option 3). The estimated settlements of the foundation soils under the proposed 1.35 m grade raise to accommodate the new embankments was calculated for each alignment option using the assumed soil profiles determined from the boreholes at Cross-Sections B-B' and C-C'. In addition, a third section at 22+620 was also analyzed using an assumed horizontal soil profile based on borehole 12-104. At this location, it is suspected that the previous Salmon River channel was filled in with rock fill, and the variation in the bedrock surface and overburden layers would be less. In addition, this area had the most overburden based on the boreholes, which would also yield significant settlements from a grade raise. The table below provides the estimated maximum settlement results at these three cross-sections using the Sigma/W finite element analyses.

Station / Location	Alignment Option	Estimated Maximum Settlements (mm)
22+660 (Cross-Section B-B’)	No centre line shift	100 – 150
22+660 (Cross-Section B-B’)	7 m centre line shift to the North	< 25
22+660 (Cross-Section B-B’)	4.2 m centre line shift to the south	150 – 300
22+720 (Cross-Section C-C’)	No centre line shift	< 25
22+720 (Cross-Section C-C’)	7 m centre line shift to the North	< 25
22+720 (Cross-Section C-C’)	4.2 m centre line shift to the south	25 – 50
22+620 (at borehole 12-104)	No centre line shift	50 – 100
22+620 (at borehole 12-104)	7 m centre line shift to the North	50 – 100
22+620 (at borehole 12-104)	4.2 m centre line shift to the south	100 – 150

The north alignment would be preferred from a settlement viewpoint with values at the approaches within allowable limits.

From the Sigma/W finite element outputs, it was possible to determine the magnitude of expected settlement in each of the soil layers. The results consistently indicated that the most compression would occur in the organic soil layer. The primary consolidation of the organic sandy/silty deposit is expected to be quick (i.e., about 1 to 3 months). The primary consolidation calculated from the consolidation test data for the silty clay deposit is expected to take much longer (i.e., about 1.5 to 2 years). However, the layering of the silty clay with silt seams that was observed in the intact samples would indicate that the primary consolidation of the silty clay would take less time than the calculated values from the consolidation test data (i.e., about 4 to 8 months).

The organic soils typically have secondary consolidation (or creep) magnitudes that can be significant over time due to the breakdown of organic matter. Because of the significant variation in the thickness and the organic content of the organic silty sand to clayey silt layer, it is very difficult to accurately predict the amount of secondary settlement. Moreover, the empirical method in Mesri and Ajlouni (2007) for calculating the secondary settlements in organic soils is also very dependent on the surcharge/preload program that is used. However, the estimated amount of secondary settlement over 20 years could be in the order of about 100 mm based on the measured thickness of asphaltic concrete near the bridge (i.e., about 200 mm), which would indicate that about 100 mm of padding was required over time. This would require maintenance over time to pad the asphalt at the bridge.

Settlement of the approach embankments will occur as a result of compression of the new embankment fill itself as well as consolidation of the organic and clayey soils on which the approaches will be founded.

The above settlement estimates include compression of the new fill itself, which would occur during or shortly after the construction of the embankment depending on the type of materials used. The magnitude of new fill compression would be less than 25 mm, assuming approximately 95 percent compaction of the new embankment fill is achieved, relative to the material’s standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the new fill itself is expected to occur essentially during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time. The use of granular fill for the new embankment construction, would reduce the magnitude of post-construction settlement of the new embankment fill (likely to less than 10 mm), since the majority of settlement of granular fills will occur during construction.

It is recommended that settlement monitoring consisting of settlement pins and settlement points be established on the raised embankment to determine the magnitude of the settlement and the time rate as required for the timing to final paving.

6.8.4 Embankment Settlement Mitigation Options

The above settlements would be entirely differential relative to the structures (which would be supported on deep foundations on bedrock). These settlement values exceed the usual values accepted by MTO for the approaches to bridges, as shown in the following table:

Distance from Abutment (m)	Tolerable Settlement (mm)
0 to 30	10 to 25
30 to 70	25 to 75
70 to 250	75 to 150

Within 5 m of the abutments of these particular bridges, it is understood that essentially negligible settlements are desired, in part because of the limited tolerance of the structures to accept approach slab settlements.

The following options could be considered for mitigation of post-construction settlement of the approach embankments:

- **Option 1:** Excavate the fill and underlying organic soils, and replace with engineered fill. This mitigation option would limit the amount of anticipated consolidation and creep settlement;
- **Option 2a:** Employ lightweight fill (i.e., expanded polystyrene – EPS) in the construction of the approach embankments to reduce the magnitude of primary consolidation settlement. The use of EPS fill would reduce the post-construction settlements;
- **Option 2b:** Employ lightweight or ultra-lightweight slag fills in the construction of the approach embankments to reduce the magnitude of primary consolidation settlement. The use of slag fills may reduce the post-construction settlements;
- **Option 2c:** Employ lightweight cellular concrete in the construction of the approach embankments to reduce the magnitude of primary consolidation settlement. The use of lightweight cellular concrete may reduce the post-construction settlements;

- **Option 3:** Preload the new embankment and allow the settlements to occur prior to paving. This option would reduce the primary settlement consolidation magnitudes prior to paving. However, some primary consolidation settlement would still occur after paving. Additionally, this option would not reduce the long term creep settlements;
- **Option 4:** Install wick drains to accelerate the consolidation settlement within the silty clay. This option may be used, with or without surcharge (see Option 5 below), to reduce the post-paving settlements. However, wick drains would not reduce the secondary creep within the organic layer;
- **Option 5:** Surcharge the new embankment to increase the magnitude of settlement during the preload period, prior to paving. This option would significantly reduce the post-paving consolidation and creep settlements. However, the increased height of fill would further reduce the factor of safety against embankment stability and additional mitigation measures such as berms or ground improvement would be required; and,
- **Option 6:** Carry out insitu soil improvement below the affected sections of the embankments by using deep soil mixing or rammed aggregate piers.

Some additional information regarding each of these mitigation options is provided in the sub-sections that follow, and the advantages, disadvantages, relative costs, and risks/consequences are summarized in tabular format in Table 2 following the text of this report.

6.8.4.1 *Option 1 – Excavate and Replace with Engineered Fill*

Option 1, excavating the existing fill and underlying organic soils, and replacing it with engineered fill is likely not feasible at this site. The fill and underlying organic soils extend to depths of up to 8.2 m, which is within the reach of typical mechanical excavation equipment. But, the costs for subexcavation would likely be high due to the extensive temporary excavation support that would be required adjacent to the existing roadway and along the creek. Additionally, the excavation would need to extend below the groundwater level at this site, and below the water level in the adjacent creek, requiring the use of water tight shoring or cofferdams.

The shoring options are limited since it would likely not be possible to cantilever sheet piling into the loose/soft overburden, and which is also indicated to be relatively thin in thickness below the required excavation depths. Tie-backs into the bedrock would therefore be required. Soldier pile and lagging shoring would similarly require tiebacks, unless the soldier piles were socketed into the rock which is also potentially costly. In addition, soldier piles and lagging is not generally regarded as a suitable option for watertight shoring. Other shoring methods that are suited to wet conditions, such as a concrete secant pile wall or slurry walls, would be very costly and therefore also likely not cost-effective or practical. The time required to effect this excavation and replacement would result in a longer period for one lane traffic.

6.8.4.2 *Option 2 – Lightweight Fill*

As noted in Option 2a, the amount of time-dependent settlement and the associated roadway maintenance may be reduced by employing EPS ‘Geofoam’ fill material, with a unit weight of less than 1 kN/m^3, below the pavement structure. EPS fill could be used in place of conventional earth fill to reduce the applied loading to below the pre-consolidation range. To prevent icing of the roadway during the winter months, EPS should have a granular cover of at least a metre.

If EPS fill is adopted for construction of the approach embankments up to a 1.4 m thick layer of EPS along the approach embankments near the bridge would reduce the applied load sufficiently to limit the post-paving, primary consolidation settlement to less than 25 mm. The extent and thickness of the light weight fills, and at which distance from the abutments it can be reduced in thickness will depend on the chosen alignment option, to meet the permissible settlement of the roadway at and near the bridge abutments.

The Geofoam will need to be covered with a concrete slab to protect it from being overstressed by the traffic loads; overstressing of the Geofoam could lead to rutting of the pavement surface. That slab would be placed at pavement subgrade level. A thickness of 125 mm is typical for the protective slab.

A sufficient pavement granular thickness is required to limit the potential for icing of the roadway due to the insulating properties of the Geofoam. From that perspective, a minimum of 800 mm combined thickness of granular base and subbase should be planned.

A suitable Geofoam type would be EPS22 in accordance with ASTM D6817-02, having a compressive strength at 5 percent strain of at least 115 kPa.

The EPS is potentially soluble in hydrocarbons. To guard against dissolution of the EPS in the case of an accidental release and infiltration of fuel (such as could occur in the case of a collision), it is general practice to cover the outside surface of the EPS with polyethylene sheeting.

A 0.3 m thick layer of OPSS Granular A would be appropriate as a levelling pad beneath the EPS Geofoam, covered with up to 100 mm of mortar sand.

Special frost taper treatments would be required at the longitudinal and transverse limits of the EPS to avoid severe differential frost heaving of the pavement surface. The longitudinal treatment could consist of the sub-excavation of the subgrade soils beneath the end of the EPS to a depth of 1.8 m below pavement grade (the design frost depth), replacement with compacted Granular B Type II, and the construction of a longitudinal 20H:1V frost taper beyond the edge of the EPS, in a manner similar to OPSD 205.060.

For Option 2b, two types of slag fill are available for use:

- Ultra-lightweight slag fill from Hamilton (Litex-143), with a bulk unit weight of about 11.5 kN/m^3; and,
- Lightweight slag fill (Superior Slag) from Sault Ste. Marie or from Hamilton (Litex-149), with a bulk unit weight of about 14 kN/m^3.

Ultra-lightweight (Litex-143) and lightweight slag fill (Superior Slag) could also be used to construct the embankment. However, since it will not be possible to achieve the required reduction in loading with either ultra-lightweight slag fill or lightweight slag fill, especially since these fills should not be installed below the anticipated high-water levels due to environmental concerns, Option 2b would not eliminate future settlement; some ongoing settlement will occur due to consolidation settlement and creep.

6.8.4.3 *Option 3 – Preload*

Option 3, preloading without a surcharge is considered to be feasible at this site, but it would depend on the chosen option and staging of the construction work. This option would reduce the magnitude of post paving primary consolidation settlements but would not reduce the long term settlement magnitudes due to secondary compression. Also, it may not be possible to use a preload adjacent to the new retaining wall, and a preload in combination with the use of light weight fill may need to be considered for some of the alignment options.

6.8.4.4 *Option 4 – Wick Drains*

Option 4, in combination with either preloading (Option 3) or surcharging (Option 5, as discussed below), would involve installing wick drains beneath the embankment footprint to accelerate the consolidation settlement. Installing wick drains for this relatively small project and for the limited thickness of clay present at this site would likely not be practical or economic. In addition, while the wick drains would accelerate the settlements in the silty clay, it would have very limited to no impact on the settlement of the organic layer, especially the secondary creep, which would not justify the additional cost of wick drains for this project.

6.8.4.5 *Option 5 –Surcharging*

Option 3, preloading with a surcharge is considered to be feasible at this site, but it would depend on the chosen option and staging of the construction work. This option would reduce the magnitude of post paving primary consolidation settlements, and potentially a significant amount of secondary creep settlement. The amount of secondary creep settlement reduction would depend on the height of the surcharge, and the amount of time it is left in place. The slope stability analyses carried out for an embankment surcharge indicates that the slopes under both static and seismic loading would not be stable in some areas, such as at Station 22+660 for a 4.2 m alignment shift to the south, and the use of temporary berms at the toe for surcharging would be required.

For this project, surcharge amounts of 1 m or 1.5 m of granular fill would be required in the approach embankment areas for a 3 to 6 month period.

6.8.4.6 *Option 6 –Soil improvement*

As an alternative to Option 5, and to conventional subexcavation of the organic soil and silty clay deposits below the approach embankments, the use of deep soil mixing or rammed aggregate piers could be considered to improve the performance the embankments.

The mobilization costs associated with the deep soil mixing equipment would be high; it would likely not be practical to mobilize equipment to this site for the relatively limited improvement works required for this approximately 100 m length of embankment, and so a deep soil mixing option for subsoil improvement is not considered economical.

Rammed aggregate piers (RAP), which can be installed to a maximum of about 7 m depth, could be feasible for improvement below the approach embankments provided that some of the existing fill is excavated out prior to the installation of the RAP. The rammed aggregate piers would likely be installed in an array under the full width and length of the affected embankment areas.

However, rammed aggregate piers may not be competitive with the alternative use of temporary berms, preloading and surcharging and some use of EPS fill at the abutments and retaining wall. The mobilization and installation costs would be relatively high for a project of this small size. Temporary berms are relatively cost effective, and the cost of the limited amount of EPS fill that would be eliminated on this project, may not be sufficient to offset the costs for rammed aggregate piers.

In addition, cost advantages would likely be lost due to the relatively small size of this project. Although their use is not uncommon in the United States, we understand that rammed aggregate piers have also not been used to date on an MTO project and there is potential for impacts to the schedule.

Based on the above, it is therefore considered that Options 2a, 3 and 5, or a combination thereof, are the most technically feasible, practicable and cost effective options. The preferred option, or combination of options, would depend on the chosen alignment. For example, although a 4.2 m centre line shift to the south would yield the highest magnitude of settlements, the use of a surcharge or preload for all of the approach embankments with a limited amount of EPS near the abutment is considered to be feasible, and likely the most economical.

6.9 Design and Construction Considerations

6.9.1 Excavations

The excavations for the construction of abutments will extend mostly through the surficial sandy fill, rock fill and embankment fill, and potentially into the underlying organic soil deposit. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The fill above the water table is classified as a Type 3 soil according to the OHSA and excavations should be made with side slopes no steeper than 1 horizontal to 1 vertical. When below the water table, the fill and organic soils are classified as Type 4 soils according to the OHSA and therefore excavations should be made with side slopes no steeper than 3 horizontal to 1 vertical.

6.9.2 Groundwater and Surface Water Control

The bridge span length should be chosen to avoid construction in the Salmon River.

The groundwater level is considered to be in the range of Elevation 188 to 189 m and highly dependent on the level of Salmon River. There is a risk that foundation excavations for the pile cap construction will intersect water-bearing organic soils or fill materials associated with the former river channel, contributing to higher groundwater inflows into the excavation. If appropriate, groundwater inflows could be minimized with the use of cofferdams at the abutments, and at the retaining wall (if required). Due to the presence of rock fill within the previous river channel and at the existing abutments, the use of an interlocking sheet pile system is not recommended, unless the location of such a system can be chosen to avoid the rock fill. In areas where limited overburden is present, the use of interlocking sheet piles is not recommended.

6.9.3 Obstructions

The soils at this site contain cobbles and boulders, which could affect the installation of deep foundations or protection systems. A Non-Standard Special Provision (NSSP) is provided in Appendix C to alert the contractor to the presence of cobbles and boulders.

7.0 CLOSURE

This report was prepared by Mr. Nicolas LeBlanc, P.Eng. and reviewed by Mr. Fintan Heffernan, P.Eng., Golder's Designated MTO Contact for this project, who conducted a technical and independent quality control review of the report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
May 30, 2013
N. R. LEBLANC
100169943
PROVINCE OF ONTARIO

Nicolas LeBlanc, P.Eng.
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
May 30, 2013
F. J. HEFFERNAN
PROVINCE OF ONTARIO

Fintan J Heffernan, P.Eng.
Designated MTO Foundations Contact

WAM/NRL/FJH/bg

n:\active\2011\1111 -mississauga\11-1111-0017-1000 salmon river\foundations - detailed design (ph 3000)\report and memo\final to mto\11-1111-0017-3000 rpt-002 foundations detailed design may 2013.docx

Table 1: Comparison of Foundation Alternatives
Salmon River Bridge Replacement
G.W.P. 4034-09-00

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread or Strip Footings on Bedrock	▪ Feasible only for Option 1 and a portion of Option 2	▪ High bearing resistance ▪ Negligible settlement ▪ Conventional construction	▪ Could require dewatering ▪ Deep excavations may be required ▪ May require extensive shoring/roadway protection ▪ Lower or higher than expected bedrock could require modifications to the foundations (i.e., stepped footings)	▪ Least expensive for Option 1	▪ Generally low risk option
Steel H-piles Founded on Bedrock	▪ Feasible	▪ High bearing resistance ▪ Negligible settlement ▪ Integral abutments possible for south alignment Option 3	▪ Possibility of encountering cobbles or boulders during pile driving, and need to pre-auger some pile locations or use reduced pile capacity	▪ Least expensive for Option 3	▪ Generally low risk option
Caissons Founded on Bedrock	▪ Feasible	▪ High bearing resistance	▪ High water table will require the use of temporary or permanent liners ▪ Presence of cobbles and boulders may cause difficulty in drilling caissons	▪ Most expensive option	▪ Generally higher risk option

Table 2: Comparison of Ground Improvement Options
Salmon River Bridge Replacement
G.W.P. 4034-09-00

Mitigation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Lightweight Fill	▪ Feasible	▪ Reduces the post-construction settlements	▪ EPS should be used below granular cover to avoid icing of the roadway in winter ▪ Lightweight slag fills will not sufficiently reduce the loading on the underlying silty clay and organics ▪ Lightweight slag fills may not be used below the groundwater or river level	▪ More costly than preloading or surcharging	▪ EPS may de-stabilise embankment by floatation if very high water flows in river occur ▪ Icing of the roadway if EPS used without appropriate granular cover
Preload the Embankment	▪ Feasible	▪ Would reduce the post-construction settlement magnitudes	▪ Long-term settlements (post-construction) would still exceed 50 mm (i.e., about 100 mm) ▪ For south alignment Option 3, EPS required behind abutments ▪ Final paving delayed	▪ Least costly	▪ Long term (post-construction) settlement magnitudes higher than acceptable
Surcharging	▪ Feasible	▪ Would reduce post-construction settlement to acceptable magnitudes	▪ Unacceptable factor of safety against global instability during construction in some areas ▪ Berms required in some areas where embankment is surcharged. ▪ For south alignment Option 3, EPS required behind abutments	▪ Slightly more costly than preloading	▪ EPS may de-stabilise embankment by floatation if very high water flows in river occur ▪ Icing of the roadway if used without adequate granular cover

Mitigation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Wick Drains (with or without surcharging)	▪ Feasible, not practical	▪ Would accelerate time to achieve primary consolidation settlement in the clay	▪ Would not accelerate settlements in the organic soil deposit, which will yield the most compression from the proposed grade raise	▪ Not cost effective for time gain	▪ Not practical to reduce overall settlements
Ground Improvement	▪ Feasible, not practical	▪ Reduces the post-construction settlements to less than 25 mm	▪ Uncommon on MTO projects ▪ May be difficult to mobilise contractor for a relatively small project	▪ Likely the most costly option	▪ Higher risk of construction difficulties with this method

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

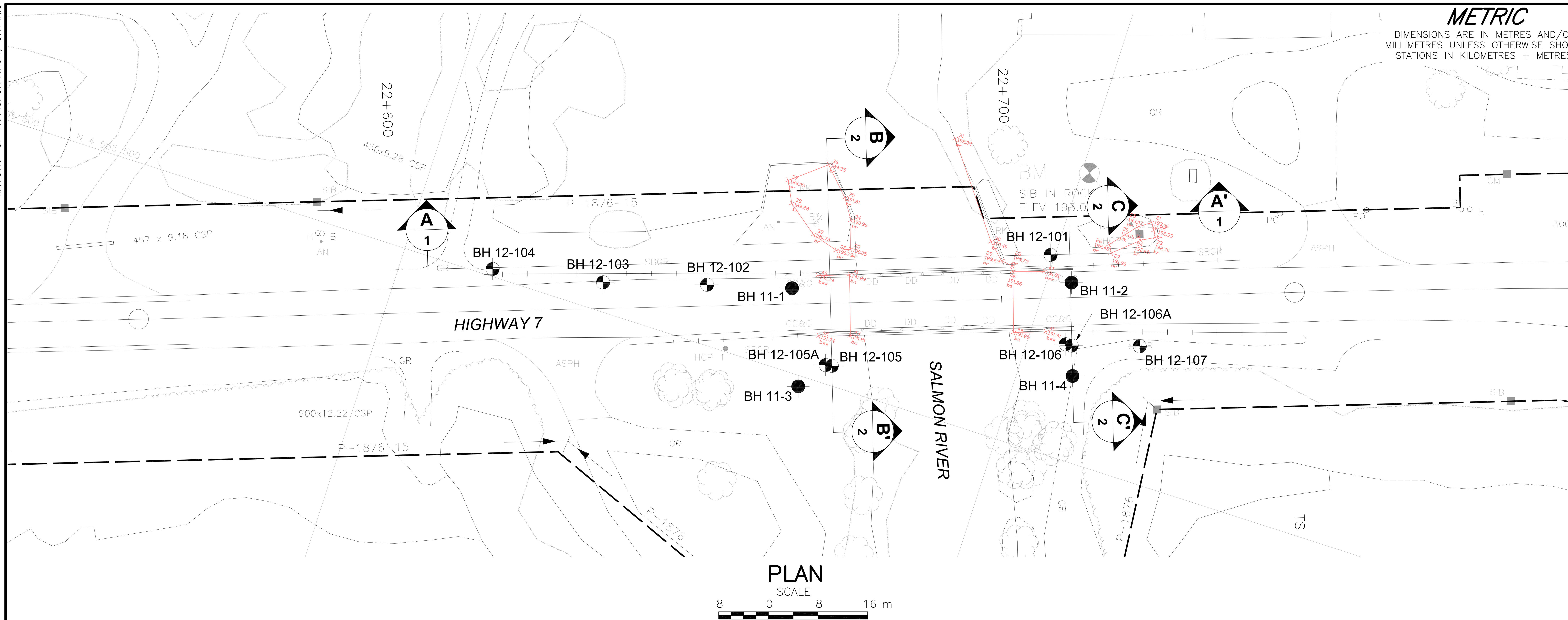
CONT No.
WP No. 4034-09-00

SHEET

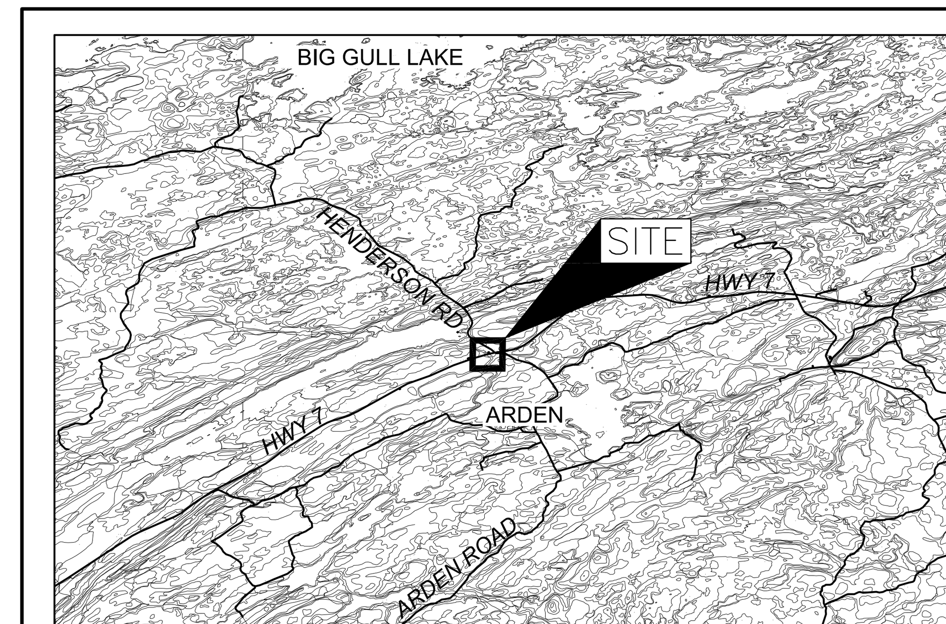
HIGHWAY 7
SALMON RIVER BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



PLAN
SCALE
8 0 8 16 m



KEY PLAN
SCALE
3 0 3 6 km

LEGEND

Borehole – Current Investigation
Borehole – Previous Investigation
N Standard Penetration Test Value
16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100% Rock Quality Designation
Seal
Piezometer
WL in piezometer
HWL, measured May 23, 2003
Spot elevation on bedrock outcrop

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
11-1	191.7	4955513.1	270261.3
11-2	191.4	4955527.7	270303.9
11-3	190.5	4955498.3	270267.1
11-4	190.7	4955513.5	270308.7
12-101	191.5	4955531.0	270299.3
12-102	191.5	4955509.4	270248.1
12-103	191.3	4955504.6	270232.1
12-104	191.0	4955501.2	270214.5
12-105	190.4	4955503.1	270271.3
12-105A	190.6	4955502.9	270270.3
12-106	191.5	4955518.0	270306.0
12-106A	191.4	4955518.1	270307.0
12-107	191.4	4955521.3	270317.5

REFERENCE

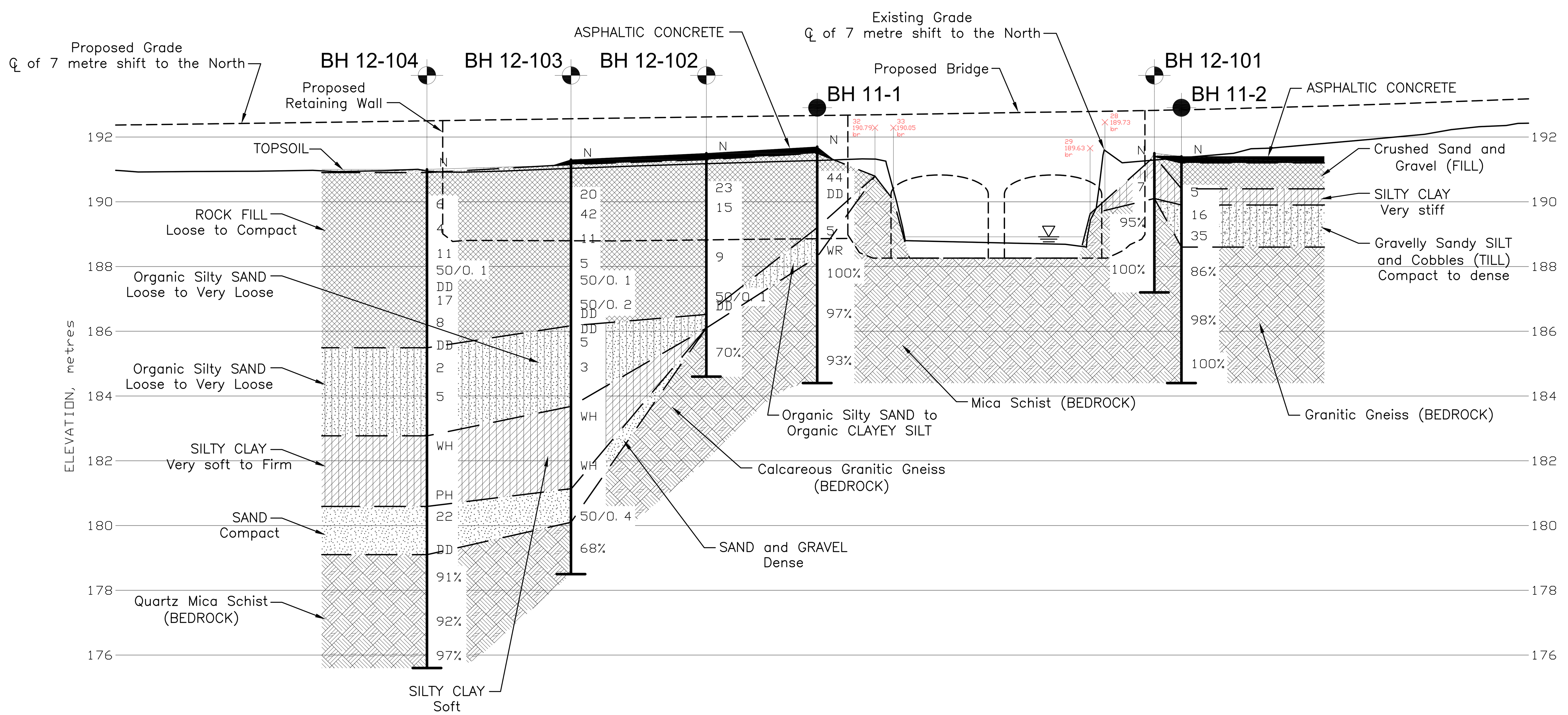
Base plans provided in digital format by Morrison Hershfield Limited, DATUM: NAD 83, COORDINATE SYSTEM: MTM ZONE 9

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 Preliminary Design Report.

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

The complete Preliminary 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.



PROFILE A–A'
HORIZONTAL SCALE
8 0 8 16 m
2 0 2 4 m
VERTICAL SCALE

NO.	DATE	BY	REVISION

Geocres No. 31C-213

HWY. 7	PROJECT NO. 11-1111-0017		DIST.
SUBM'D. NRL	CHKD. NRL	DATE: JANUARY 2013	SITE:
DRAWN: JM	CHKD. NRL	APPD. FJH	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC

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

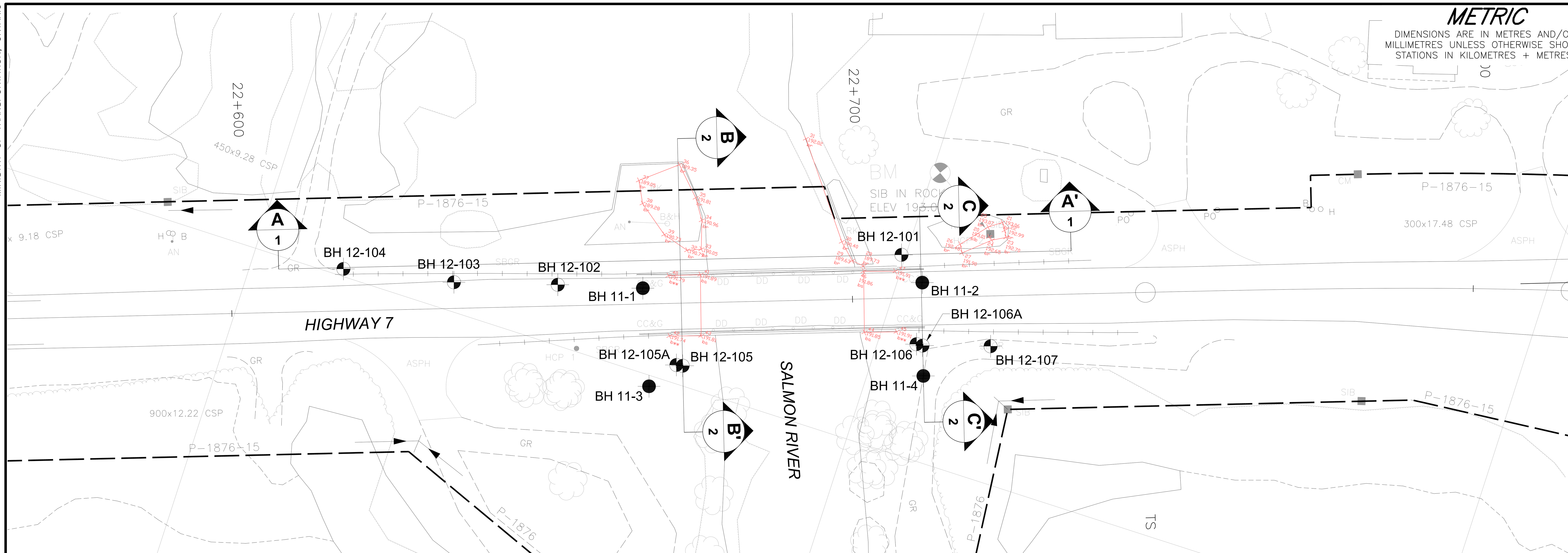
CONT No.
WP No. 4034-09-00

SHEET

HIGHWAY 7
SALMON RIVER BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

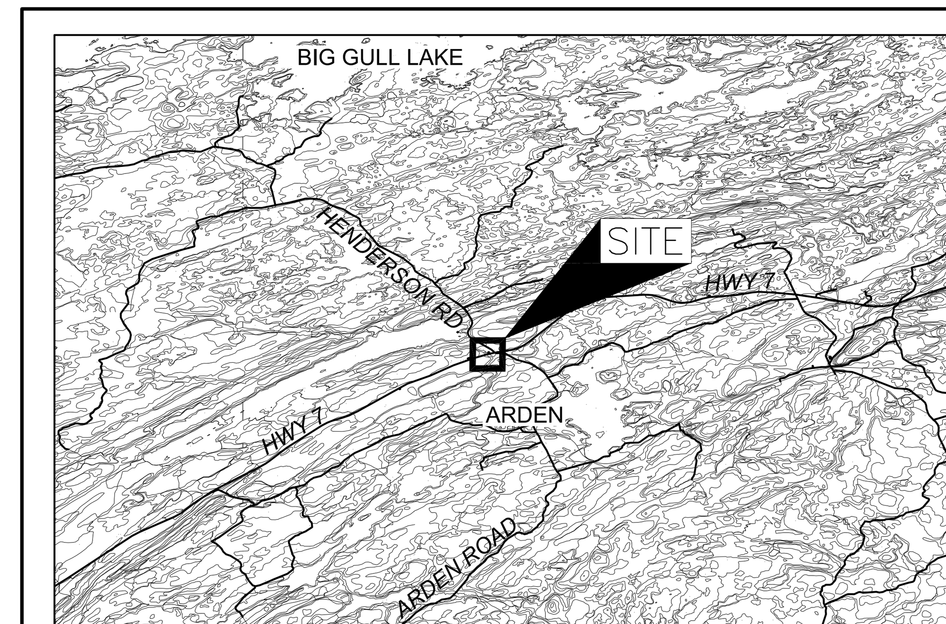
Golder Associates

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



PLAN

SCALE 8 0 8 16 m



KEY PLAN

SCALE 3 0 3 6 km

LEGEND

- Borehole – Current Investigation
- Borehole – Previous Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation
- Seal
- Piezometer
- WL in piezometer
- Spot elevation on bedrock outcrop

No.	ELEVATION	CO-ORDINATES NORTHING	EASTING
11-1	191.7	4955513.1	270261.3
11-2	191.4	4955527.7	270303.9
11-3	190.5	4955498.3	270267.1
11-4	190.7	4955513.5	270308.7
12-101	191.5	4955531.0	270299.3
12-102	191.5	4955509.4	270248.1
12-103	191.3	4955504.6	270232.1
12-104	191.0	4955501.2	270214.5
12-105	190.4	4955503.1	270271.3
12-105A	190.6	4955502.9	270270.3
12-106	191.5	4955518.0	270306.0
12-106A	191.4	4955518.1	270307.0
12-107	191.4	4955521.3	270317.5

REFERENCE

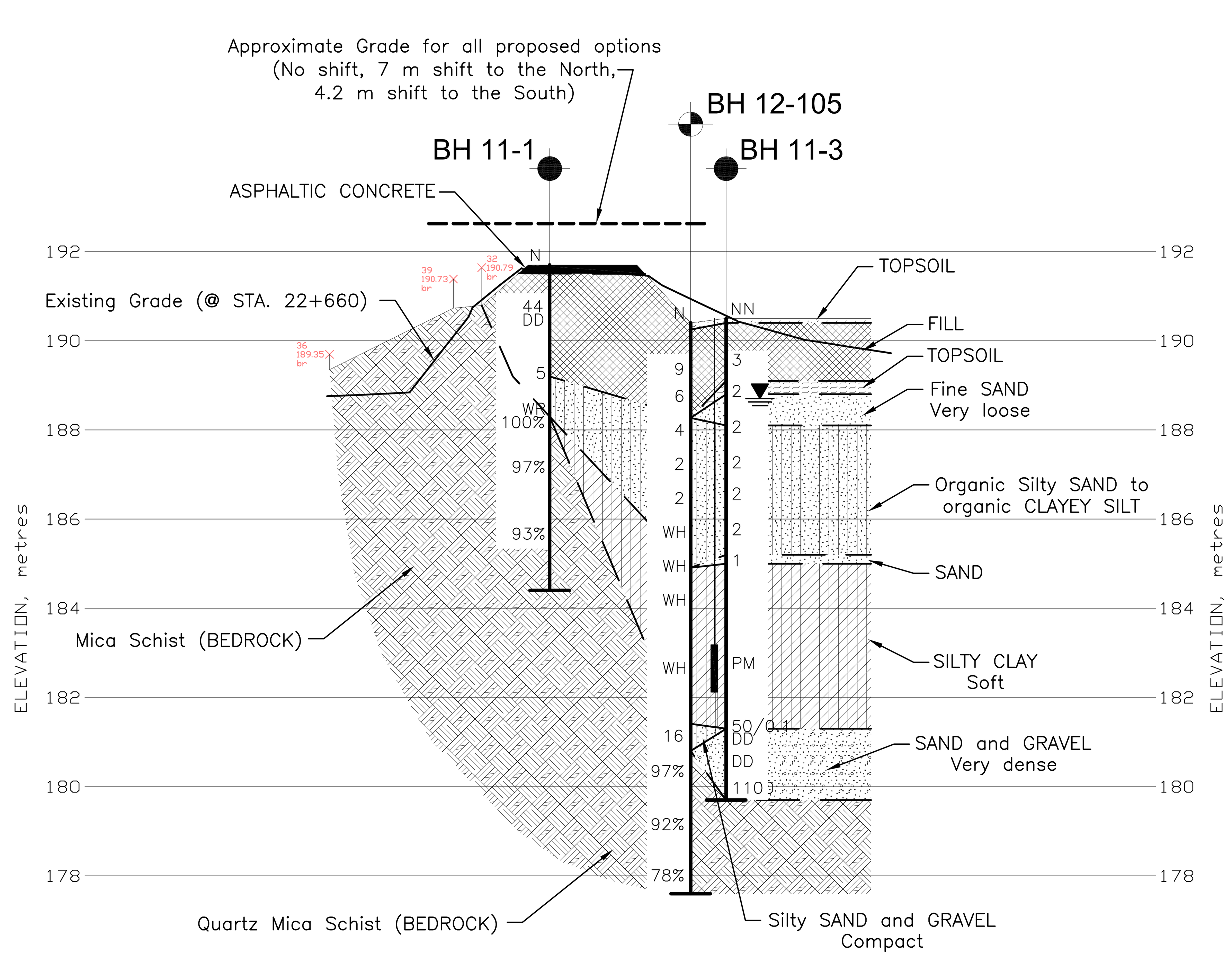
Base plans provided in digital format by Morrison Hershfield Limited, DATUM: NAD 83, COORDINATE SYSTEM: MTM ZONE 9

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 Preliminary Design Report.

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

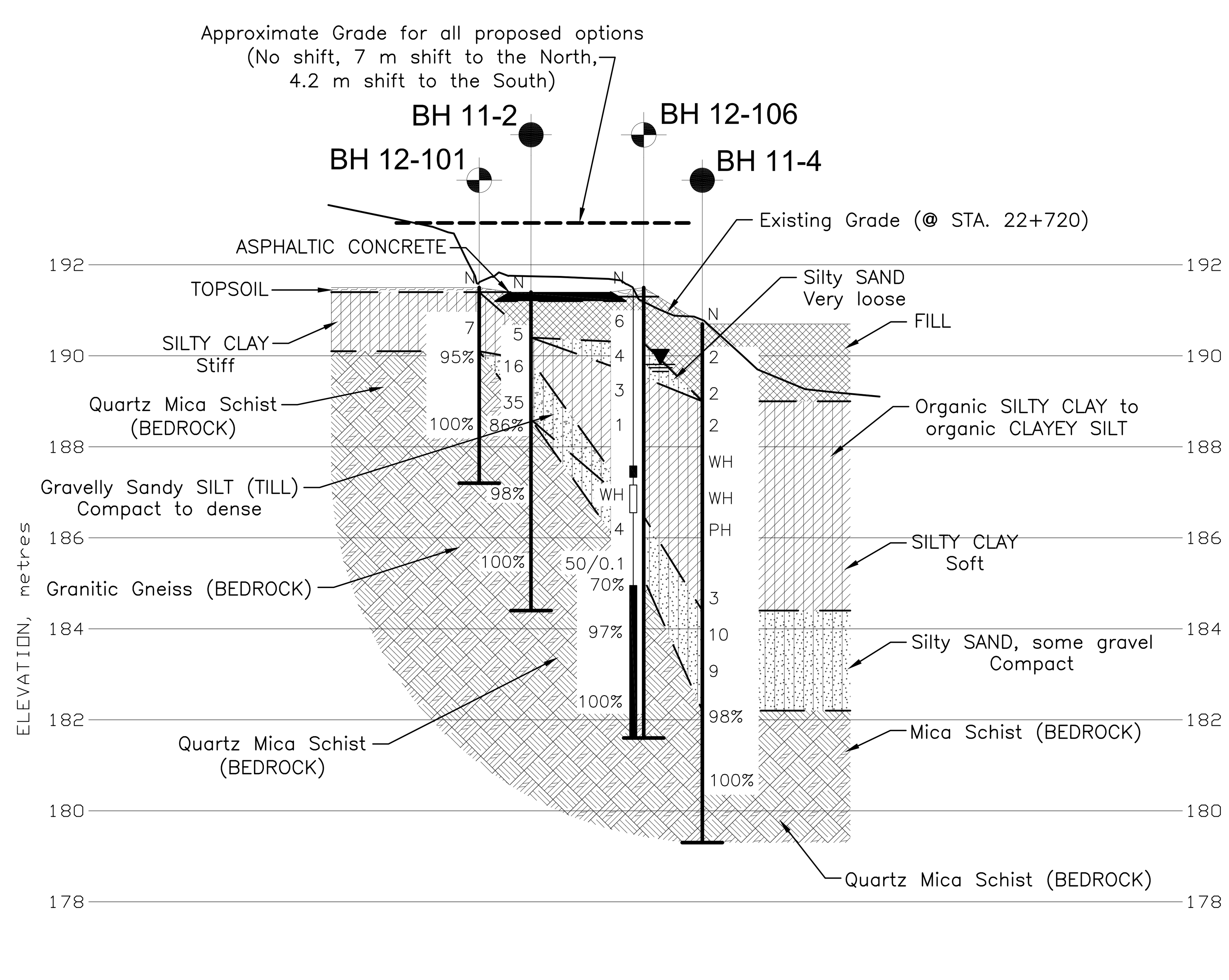
The complete Preliminary Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section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



CROSS-SECTION B-B'

HORIZONTAL SCALE 8 0 8 16 m

VERTICAL SCALE 2 0 2 4 m



CROSS-SECTION C-C'

HORIZONTAL SCALE 8 0 8 16 m

VERTICAL SCALE 2 0 2 4 m

NO.	DATE	BY	REVISION

Geocres No. 31C-213

HWY. 7	PROJECT NO. 11-1111-0017		DIST.
SUBM'D. NRL	CHKD. NRL	DATE: JANUARY 2013	SITE:
DRAWN: JM	CHKD. NRL	APPD. FJH	DWG. 2

APPENDIX A

Record of Boreholes and Drillholes and Laboratory Test Results Current Investigation (2012)

RECORD OF BOREHOLE No 12-101

 METRIC

PROJECT 11-1111-0017-3000

G.W.P. 4034-09-00 LOCATION N 4955531.0 ;E 270299.3 ORIGINATED BY W.A.M.

DIST HWY 7 BOREHOLE TYPE 200 mm Diam. Power Auger (Hollow Stem) COMPILED BY J.M.

DATUM Geodetic DATE January 3, 2013 CHECKED BY N.R.L.

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.5	GROUND SURFACE										
0.0	TOPSOIL										
0.1	SILTY CLAY Stiff Grey-brown Moist						191				
			1	SS	7				○ (~27)		
190.1 1.4	Quartz Mica Schist (BEDROCK), with thin to medium felsic granitic gneiss bands Fresh Dark grey Bedrock cored between 1.4 m and 4.3 m depth. For bedrock coring details refer to Record of Drillhole 12-101.						190				
			1	RC	REC 100%		189				RQD = 95%
			2	RC	REC 100%		188				RQD = 100%
187.2 4.3	End of Borehole										

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

PROJECT: 11-1111-0017-3000

RECORD OF DRILLHOLE: 12-101

LOCATION: N 4955531.0 ;E 270299.3

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: January 3, 2013

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole 12-101		190.10												
2		Quartz Mica Schist (BEDROCK), with thin to medium felsic granitic gneiss bands Fresh Dark grey		1.40	1											
3					2											
4				187.20 4.30												
		End of Borehole														

Depth scale: 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16

MIS-RCK 001 1111110017-1200 (ROCK).GPJ GAL-MISS.GDT 05/29/13 JM

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: W.A.M.

CHECKED: ______

RECORD OF BOREHOLE No 12-102

 METRIC

PROJECT 11-1111-0017-3000

G.W.P. 4034-09-00 LOCATION N 4955509.4 ;E 270248.1 ORIGINATED BY W.A.M.

DIST ___ HWY 7 BOREHOLE TYPE 200 mm Diam. Power Auger (Hollow Stem)/Wash Boring, NW Casing COMPILED BY J.M.

DATUM Geodetic DATE December 20, 2012 CHECKED BY N.R.L.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.5	GROUND SURFACE						
0.0	ASPHALTIC CONCRETE						
0.2	Sand and gravel (BASE) Brown Dry				191		
190.7 0.8	Sandy gravel, some silt, trace clay, with cobbles and boulders (ROCKFILL) Compact to loose Grey to grey-brown Moist to wet	1	SS	23			
		2	SS	15	190	○	52 27 16 5
					189		
		3	SS	9	188		
		~~4~~	~~SS~~	~~50/0.1~~	187		
		5	RC	DD			
186.5 5.0	Probable organic layer						
186.1 5.4	Calcareous Gneiss (BEDROCK) Fresh to slightly weathered Light grey Bedrock cored between 5.4 m and 6.9 m depth. For bedrock coring details refer to Record of Drillhole 12-102.	1	RC	REC 100%	186 185		RQD = 70%
184.6 6.9	End of Borehole						

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

PROJECT: 11-1111-0017-3000

RECORD OF DRILLHOLE: 12-102

SHEET 1 OF 1

LOCATION: N 4955509.4 ;E 270248.1

DRILLING DATE: December 20, 2012

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole 12-102		186.10												
6		Calcareous Gneiss (BEDROCK) Fresh to slightly weathered Light grey		5.40	1											
				184.60 6.90												
7		End of Borehole														

Depth scale: 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: W.A.M.

CHECKED: ______

MIS-RCK 001 1111110017-1200 (ROCK).GPJ GAL-MISS.GDT 05/29/13 JM

RECORD OF BOREHOLE No 12-103

1 OF 1 **METRIC**

PROJECT: 11-1111-0017-3000
G.W.P.: 4034-09-00
DIST: HWY: 7
DATUM: Geodetic
LOCATION: N 4955504.6 ;E 270232.1
BOREHOLE TYPE: 200 mm Diam. Power Auger (Hollow Stem)/Wash Boring, NW Casing
DATE: December 19-20, 2012
ORIGINATED BY: W.A.M.
COMPILED BY: J.M.
CHECKED BY: N.R.L.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.3	GROUND SURFACE				
0.0	ASPHALTIC CONCRETE *(0.0 m- 0.2 m)*				
0.3	Crushed stone (BASE) *(0.2 m-0.3 m)* Grey				
	Silty sand, some gravel (FILL) Compact Brown Moist	1	SS	20	
190.0 / 1.3	Silty sand and gravel, with cobbles and boulders (ROCK FILL) Dense to loose Grey Moist	2	SS	42	
		3	SS	11	
		4	SS	5	
		5	SS	50/0.1	
		6	SS	50/0.2	40 41 16 3
		7	RC	DD	
186.2 / 5.1	Organic silty SAND Loose to very loose Dark brown to black Wet	8	RC	DD	
		9	SS	5	
		10	SS	3	Org=17.6%
183.7 / 7.6	SILTY CLAY, trace to some sand Very soft to soft Grey Wet	11	SS	WH	0 34 30 36
		12	SS	WH	0 2 59 39
181.1 / 10.2	SAND and GRAVEL, trace silt Dense Grey Wet	13	SS	50/0.4	
180.1 / 11.2	Calcareous Granitic Gneiss (BEDROCK) Fresh Light grey Bedrock cored between 11.2 m and 12.8 m depth. For bedrock coring details refer to Record of Drillhole 12-103.	1	RC	REC 94%	RQD = 68%
178.5 / 12.8	End of Borehole				

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (191–179); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³; STRAT PLOT.

Water content note: 182

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

PROJECT: 11-1111-0017-3000

RECORD OF DRILLHOLE: 12-103

LOCATION: N 4955504.6 ;E 270232.1

DRILLING DATE: December 19-20, 2012

DATUM: Geodetic

DRILL RIG: CME 55

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Marathon Drilling

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE
CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK
SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING
VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole 12-103		180.10												
12		Calcareous Granitic Gneiss (BEDROCK) Fresh Light grey		11.20 178.50	1											
13		End of Borehole		12.80												

Depth scale ticks: 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: W.A.M.

CHECKED: ______

Golder Associates

RECORD OF BOREHOLE No 12-104

 METRIC

PROJECT 11-1111-0017-3000
G.W.P. 4034-09-00 LOCATION N 4955501.2 ;E 270214.5 ORIGINATED BY W.A.M.
DIST HWY 7 BOREHOLE TYPE 200 mm Diam. Power Auger (Hollow Stem)/Wash Boring, NW Casing COMPILED BY J.M.
DATUM Geodetic DATE December 17-18, 2012 CHECKED BY N.R.L.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
191.0	GROUND SURFACE
0.0	TOPSOIL
0.1	Silty sand, some gravel, with cobbles and boulders (ROCK FILL) Loose to compact Brown Moist to wet
185.5 / 5.5	Organic Silty SAND Loose to very loose Brown to black Moist
182.8 / 8.2	SILTY CLAY, trace to some sand Firm Grey Wet
180.6 / 10.4	SAND, trace gravel Compact Grey Wet
179.1 / 11.9	Quartz Mica Schist (BEDROCK) Fresh to slightly weathered Dark grey
177.4 / 13.6	Quartz Mica Schist (BEDROCK) Fresh to slightly weathered Red
176.8 / 14.2	

SAMPLES

NUMBER	TYPE	"N" VALUES	WATER CONTENT / UNIT WEIGHT NOTES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	6		
2	SS	4		19 47 27 7
3	SS	11		
4	SS	50/0.1		
5	RC	DD		
6	SS	17		
7	SS	8		
8	RC	DD		
9	SS	2	126; Org=14.0%	
10	SS	5	156	
11	SS	WH		0 2 41 57
12	TP	PH		
13	SS	22		
14	RC	DD		
1	RC	REC 96%		RQD = 91%
2	RC	REC 100%		RQD = 92%
3	RC	REC 97%		RQD = 97%

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (190, 189, 188, 187, 186, 185, 184, 183, 182, 181, 180, 179, 178, 177); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³.

Continued Next Page

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

RECORD OF BOREHOLE No 12-104

 METRIC

PROJECT 11-1111-0017-3000

G.W.P. 4034-09-00 LOCATION N 4955501.2 ;E 270214.5 ORIGINATED BY W.A.M.

DIST HWY 7 BOREHOLE TYPE 200 mm Diam. Power Auger (Hollow Stem)/Wash Boring, NW Casing COMPILED BY J.M.

DATUM Geodetic DATE December 17-18, 2012 CHECKED BY N.R.L.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) (25 50 75)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
175.6 15.4	Quartz Mica Schist (BEDROCK) Fresh to slightly weathered Dark grey		3	RC	REC 97%						RQD = 97%
	Bedrock cored between 11.9 m and 15.4 m depth. For bedrock coring details refer to Record of Drillhole 12-104. End of Borehole										

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

PROJECT: 11-1111-0017-3000

RECORD OF DRILLHOLE: 12-104

LOCATION: N 4955501.2 ;E 270214.5

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: December 17-18, 2012

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR / % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole 12-104		179.10												
12		Quartz Mica Schist (BEDROCK) Fresh to slightly weathered Dark grey		11.90	1											
13																
		Quartz Mica Schist (BEDROCK) Fresh to slightly weathered Red		177.38 / 13.62	2											
14		Quartz Mica Schist (BEDROCK) Fresh to slightly weathered Dark grey		176.77 / 14.23												
15					3											
		End of Borehole		175.60 / 15.40												
16																
17																
18																
19																
20																
21																
22																
23																
24																
25																
26																

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: W.A.M.

CHECKED: ______

RECORD OF BOREHOLE No 12-105

 METRIC

PROJECT 11-1111-0017-3000

G.W.P. 4034-09-00 LOCATION N 4955503.1 ;E 270271.3 ORIGINATED BY W.A.M.

DIST HWY 7 BOREHOLE TYPE 200 mm Diam. Power Auger (Hollow Stem) COMPILED BY J.M.

DATUM Geodetic DATE December 13, 2012 CHECKED BY N.R.L.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
190.4	GROUND SURFACE					
0.0	TOPSOIL					
0.2	Silty sand, some gravel and cobbles, trace clay (FILL) Loose to very loose Brown Moist				190	
		1	SS	9		13 62 20 3
		2	SS	6	189	
188.3 / 2.1	Organic silty SAND Very loose Dark brown to black Moist	3	SS	4	188	
		4	SS	2	187	189 Org=23.4%
		5	SS	2		162
		6	SS	WH	186	175
184.9 / 5.5	SILTY CLAY, trace to some sand Soft Grey Moist	7	SS	WH	185	
		8	SS	WH	184	0 3 67 30
		9	SS	WH	183	
					182	
181.4 / 9.0	Sandy GRAVEL, some silt, trace clay Compact Grey Moist to wet	10	SS	16	181	55 32 11 2
180.8 / 9.6	Calcareous Granitic Gneiss (BEDROCK) Fresh Light grey	1	RC	REC 100%	180	RQD = 97%
179.2 / 11.3	Quartz Mica Schist (BEDROCK) Fresh Dark grey Bedrock cored between 9.6 m and 12.8 m depth. For bedrock coring details refer to Record of Drillhole 12-105.	2	RC	REC 100%	179	RQD = 92%
		3	RC	REC 100%	178	RQD = 78%
177.6 / 12.8	End of Borehole					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³.

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

RECORD OF DRILLHOLE: 12-105

PROJECT: 11-1111-0017-3000

SHEET 1 OF 1

LOCATION: N 4955503.1 ;E 270271.3

DRILLING DATE: December 13, 2012

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

Legend: FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	NOTES WATER LEVELS INSTRUMENTATION
	Continued from Record of Borehole 12-105	180.80		
10	Calcareous Granitic Gneiss (BEDROCK) Fresh Light grey	9.60	1	UC = 40.9 MPa
11			2	
	Quartz Mica Schist (BEDROCK) Fresh Dark grey	179.15 11.25		
12			3	
	End of Borehole	177.60 12.80		

Other column headings: DRILLING RECORD; SYMBOLIC LOG; PENETRATION RATE (m/min); COLOUR % RETURN; FLUSH; RECOVERY: TOTAL CORE % (80 60 40 20), SOLID CORE % (80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX PER 0.3 (5 10 15 20); DISCONTINUITY DATA: DIP w.r.t. CORE AXIS (0 30 60 90), TYPE AND SURFACE DESCRIPTION; HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); DIAMETRAL POINT LOAD INDEX (MPa) (2 4 6)

Depth scale: 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24

DEPTH SCALE 1 : 75

Golder Associates

LOGGED: W.A.M.

CHECKED: ______

MIS-RCK 001 1111110017-1200 (ROCK).GPJ GAL-MISS.GDT 05/29/13 JM

Golder Associates

RECORD OF BOREHOLE No 12-105A

 METRIC

PROJECT 11-1111-0017-3000

G.W.P. 4034-09-00 LOCATION N 4955502.9 ;E 270270.3 ORIGINATED BY W.A.M.

DIST HWY 7 BOREHOLE TYPE 200 mm Diam. Power Auger (Hollow Stem) COMPILED BY J.M.

DATUM Geodetic DATE December 17, 2012 CHECKED BY N.R.L.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
190.6	GROUND SURFACE										
0.0	For soil description details refer to Record of Borehole 12-105						190				
							189				
							188				
							187				
							186				
							185				
							184				
183.3			1	TP	PH					18.4	
7.3	End of Borehole										

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

Golder Associates

RECORD OF BOREHOLE No 12-106

 METRIC

PROJECT	11-1111-0017-3000		
G.W.P.	4034-09-00	LOCATION	N 4955518.0 ;E 270306.0
ORIGINATED BY	W.A.M.		
DIST		HWY 7	
BOREHOLE TYPE	200 mm Diam. Power Auger (Hollow Stem)	COMPILED BY	J.M.
DATUM	Geodetic	DATE	January 2-3, 2013
CHECKED BY	N.R.L.		

SOIL PROFILE / SAMPLES / GROUND WATER CONDITIONS / ELEVATION SCALE / DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100) / PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L – WATER CONTENT (%) (25, 50, 75) / UNIT WEIGHT γ kN/m³ / REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT	GR	SA	SI	CL
191.5	GROUND SURFACE								
0.0	COBBLES and BOULDERS								
0.2	Sand, some gravel and silt, with cobbles (ROCK FILL) Loose Brown Moist	1	SS	6		14	75	10	1
190.3 / 1.2	Silty SAND, with organic matter Very loose Brown to black Moist	2	SS	4	Org=4.5%				
189.5 / 2.0	SILTY CLAY, trace sand Soft Grey Moist	3	SS	3					
		4	SS	1		0	3	73	24
		5	SS	WH		0	2	74	24
186.5 / 5.0	Silty SAND Loose Grey wet	6	SS	4					
		~~7~~	~~SS~~	~~50/0.1~~					
185.1 / 6.4	Quartz Muscovite Schist (BEDROCK) Fresh Light grey	1	RC	REC 98%		RQD = 70%			
184.2 / 7.3	Quartz Mica Schist (BEDROCK) Fresh Dark grey	2	RC	REC 100%		RQD = 97%			
182.1 / 9.5	Quartzite (BEDROCK) Fresh Light and dark grey bands	3	RC	REC 100%		RQD = 100%			
181.6 / 9.9	Bedrock cored between 6.4 m and 9.9 m depth. For bedrock coring details refer to Record of Drillhole 12-106. End of Borehole								

Elevation scale: 191, 190, 189, 188, 187, 186, 185, 184, 183, 182

Note:
1. Water level in Standpipe at 1.7 m depth (Elev. 189.8 m) on Jan. 15, 2013.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 11-1111-0017-3000

RECORD OF DRILLHOLE: 12-106

SHEET 1 OF 1

LOCATION: N 4955518.0 ;E 270306.0

DRILLING DATE: January 2-3, 2013

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN / FLUSH	RECOVERY TOTAL CORE % (80 60 40 20)	RECOVERY SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER 0.3 (5 10 15 20)	DISCONTINUITY DATA DIP w.r.t. CORE AXIS (0 30 60 90)	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3})	DIAMETRAL POINT LOAD INDEX (MPa) (2 4 6)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole 12-106		185.10												
7		Quartz Muscovite Schist (BEDROCK) Fresh Light grey		6.40	1											UC = 87.0 MPa
		Quartz Mica Schist (BEDROCK) Fresh Dark grey		184.18 7.32												
8					2											
9																
		Quartzite (BEDROCK) Fresh Light and dark grey bands		182.05 9.45	3											
10		End of Borehole		181.60 9.90												
11																
12																
13																
14																
15																
16																
17																
18																
19																
20																
21																

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: W.A.M.

CHECKED: ______

Golder Associates

RECORD OF BOREHOLE No 12-106A

 METRIC

PROJECT 11-1111-0017-3000			
G.W.P. 4034-09-00	LOCATION N 4955518.1 ;E 270307.0	ORIGINATED BY W.A.M.	
DIST ___ HWY 7	BOREHOLE TYPE 200 mm Diam. Power Auger (Hollow Stem)	COMPILED BY J.M.	
DATUM Geodetic	DATE January 3, 2013	CHECKED BY N.R.L.	

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	SAMPLES – TYPE	SAMPLES – "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.4	GROUND SURFACE										
0.0	For soil description details refer to Record of Borehole 12-106						191				
							190				
							189				
187.8			1	TP	PH		188				
3.6	End of Borehole										

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

RECORD OF BOREHOLE No 12-107

 METRIC

PROJECT 11-1111-0017-3000
G.W.P. 4034-09-00 LOCATION N 4955521.3 ;E 270317.5 ORIGINATED BY W.A.M.
DIST HWY 7 BOREHOLE TYPE 200 mm Diam. Power Auger (Hollow Stem) COMPILED BY J.M.
DATUM Geodetic DATE January 2, 2013 CHECKED BY N.R.L.

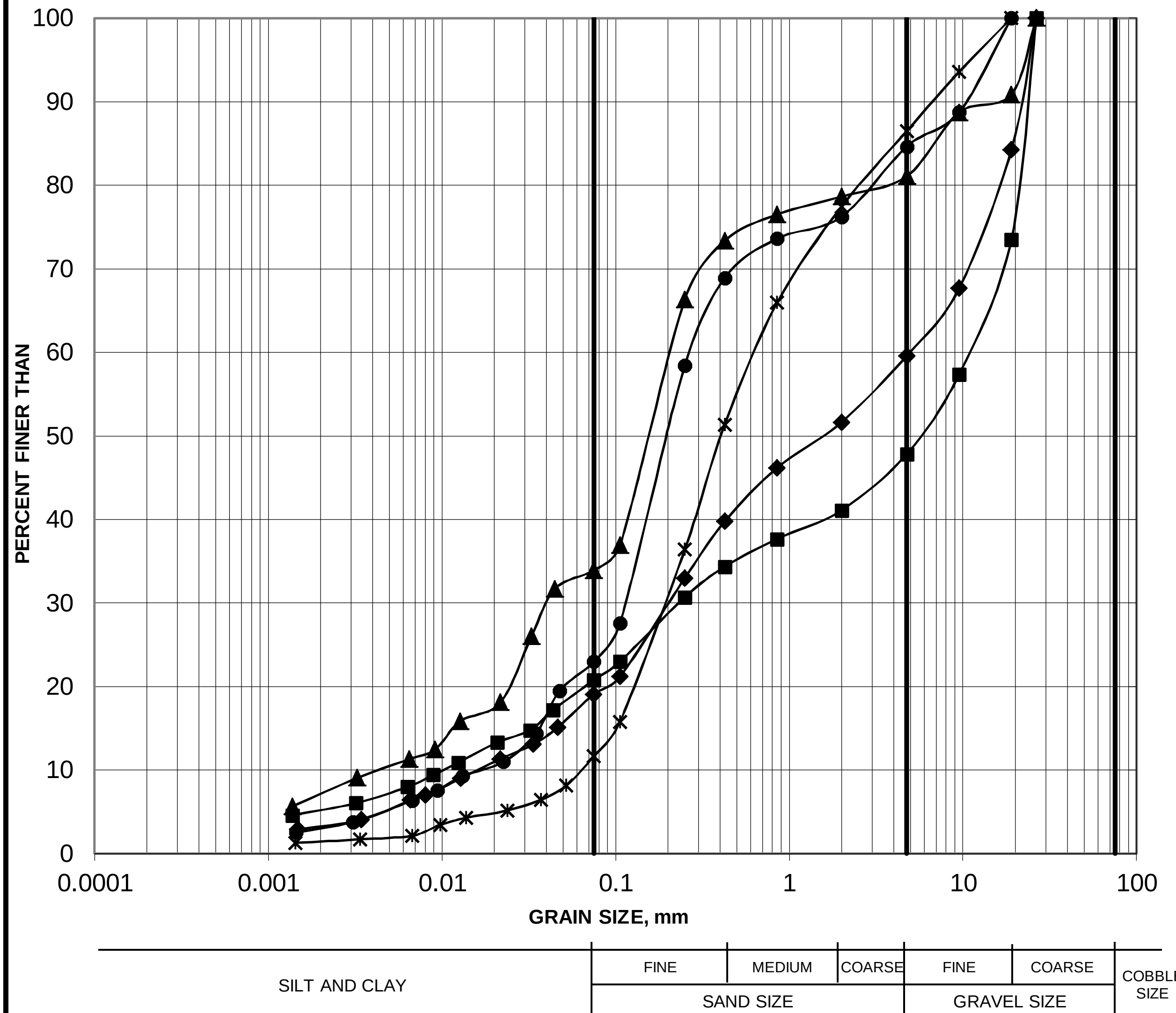
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT	UNIT WEIGHT γ (kN/m³)	GR	SA	SI	CL
191.4	GROUND SURFACE										
0.0	Sand, some gravel, with cobbles (ROCK FILL) Loose Brown Moist	1	SS	9	191						
190.0 / 1.4	Organic CLAYEY SILT Firm Brown to black Moist	2	SS	4	190						
189.3 / 2.1	Silty SAND, some clay, trace gravel, with organic matter Loose Brown to black Moist	3	SS	6	189			2	64	21	13
188.5 / 2.9	SILTY CLAY, trace sand Soft Grey-brown to grey Moist to wet	4	SS	1	188			0	8	66	26
		5	TP	PH	187		18.5				
		6	SS	WR	186						
185.6 / 5.8	Silty SAND Compact Brown Wet	7	SS	24	185						
		8	SS	50/0.1	184						
183.8 / 7.6	End of Borehole Auger Refusal										

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; STRAT PLOT; GROUND WATER CONDITIONS; REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

GRAIN SIZE DISTRIBUTION	FIGURE A1

FILL



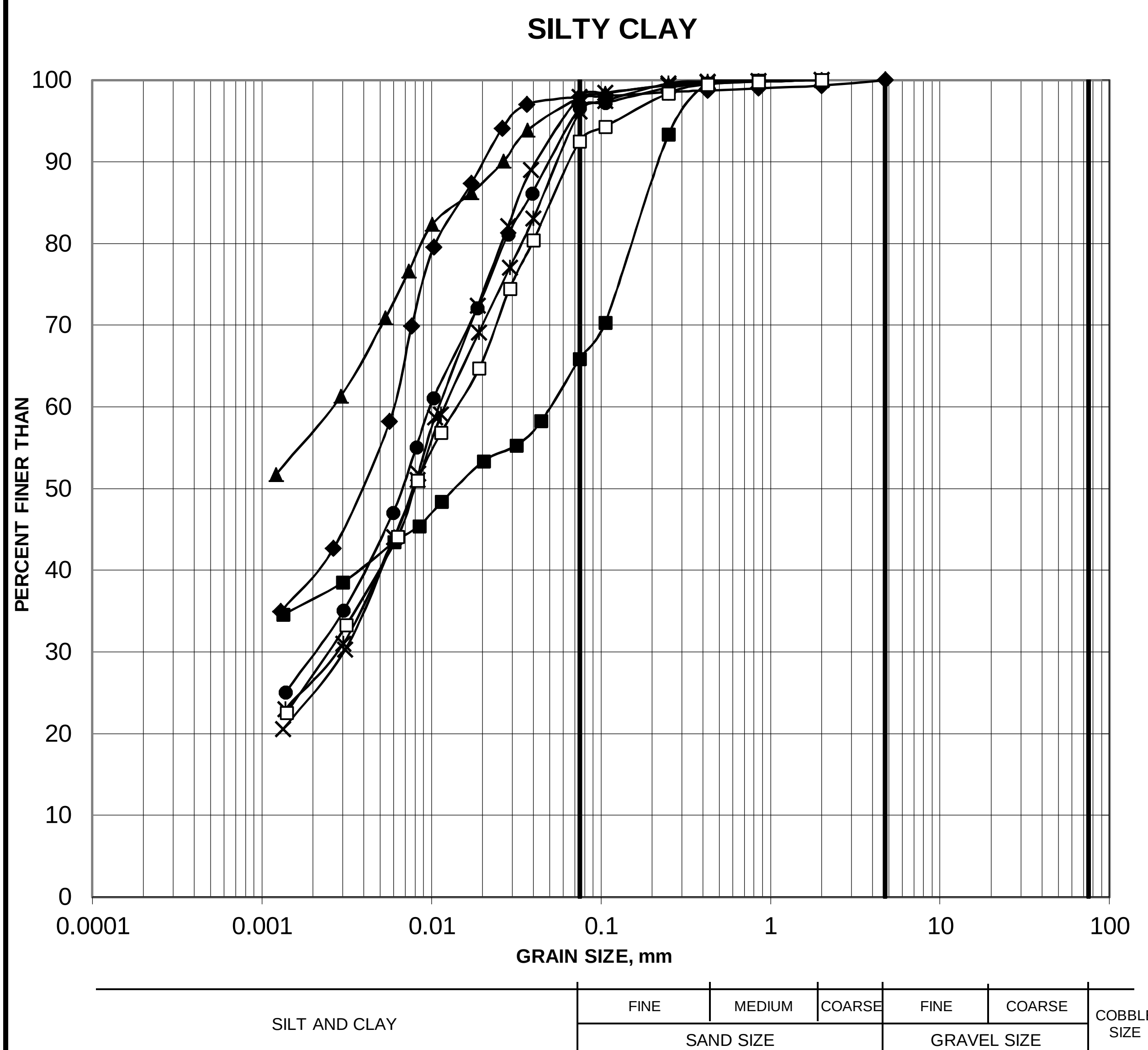
Borehole	Sample	Depth (m)
12-102	2	1.37-1.98
12-103	6	4.42-4.59
12-104	2	1.52-2.13
12-105	1	0.76-1.37
12-106	1	0.61-1.22

Project: 11-1111-0017/3000

Golder Associates

Created by: CW
Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE A2

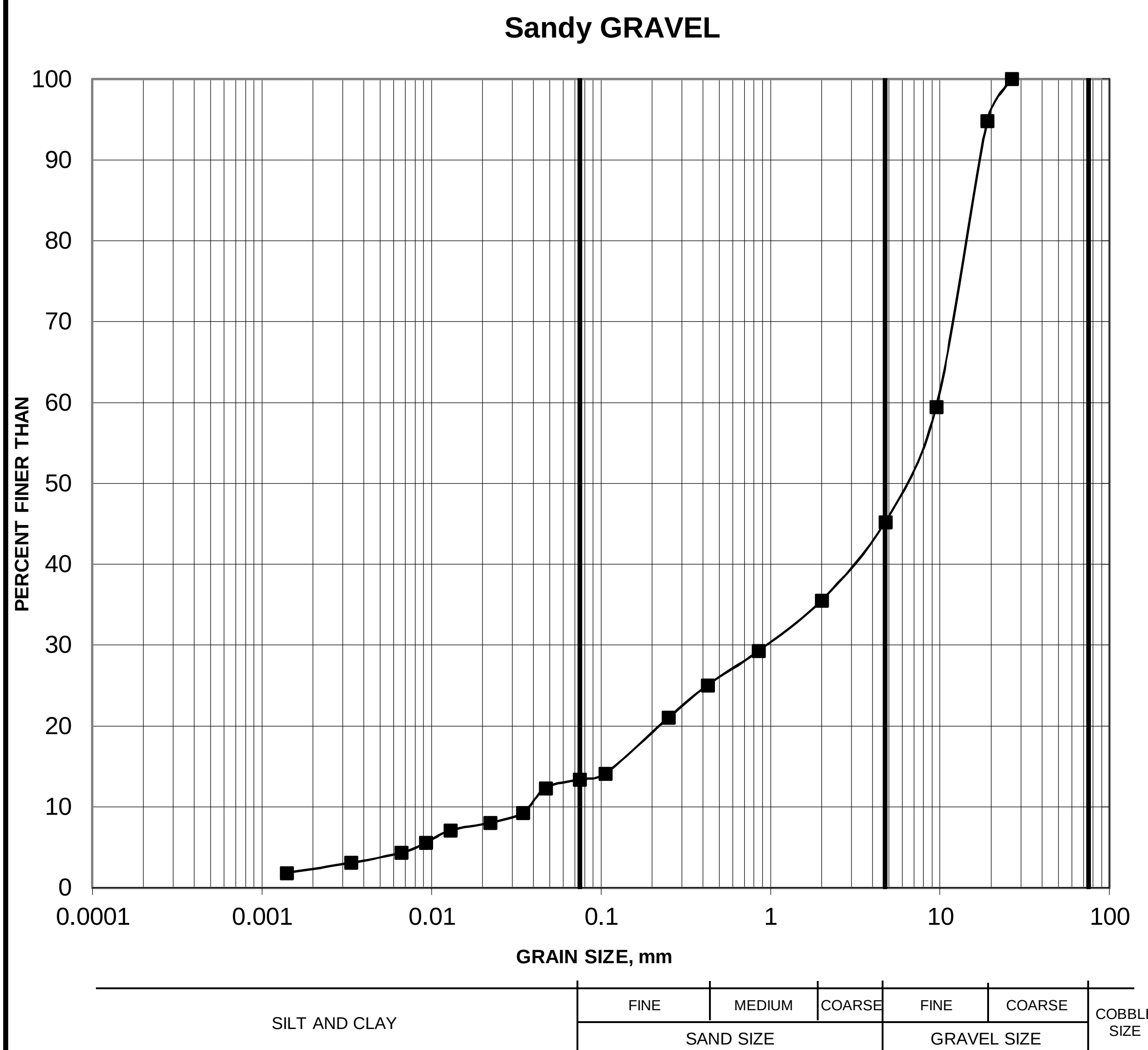


Borehole	Sample	Depth (m)
12-103	11	7.62-8.23
12-103	12	9.15-9.76
12-104	11	8.23-8.84
12-105	8	5.95-6.55
12-106	4	2.90-3.51
12-106	5	4.42-5.03
12-107	4	2.90-3.51

Created by: CW
Checked by: CNM

Project: 11-1111-0017/3000 **Golder Associates**

GRAIN SIZE DISTRIBUTION	FIGURE A3



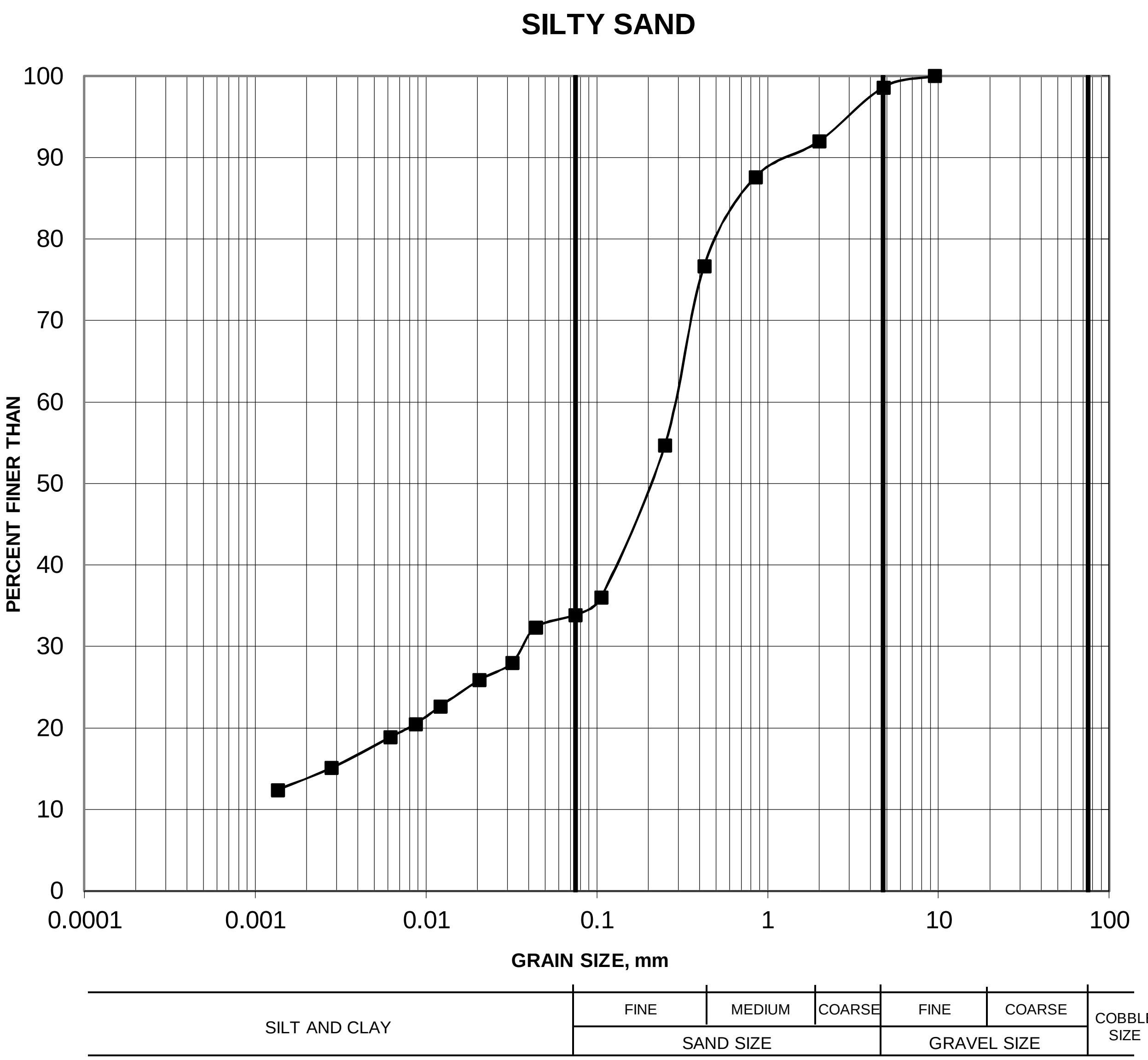
Borehole	Sample	Depth (m)
12-105	10	8.99-9.55

Project: 11-1111-0017/3000

Golder Associates

Created by:	CW
Checked by:	CNM

GRAIN SIZE DISTRIBUTION	FIGURE A4



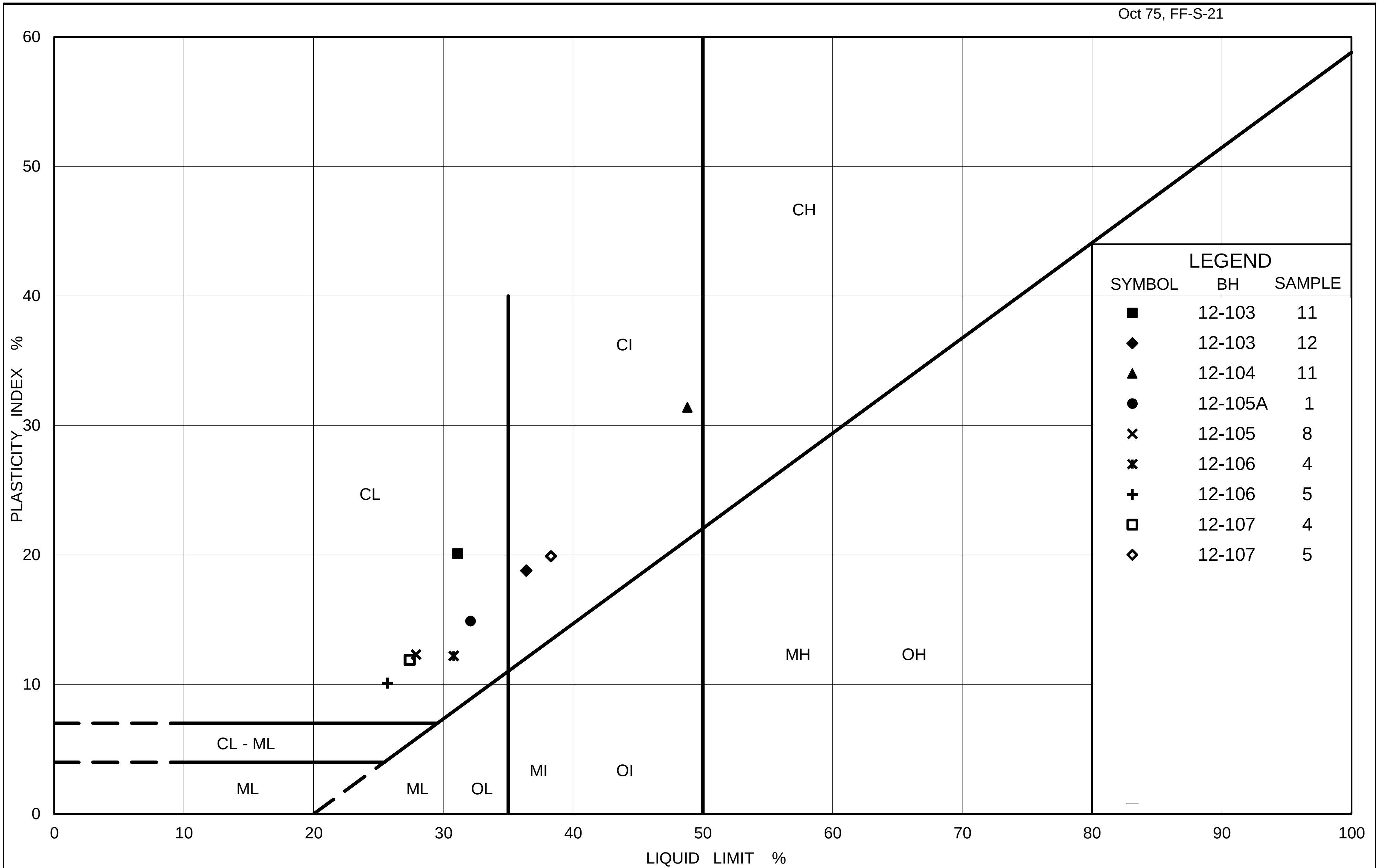
Borehole	Sample	Depth (m)
12-107	3	2.13-2.74

Project: 11-1111-0017/3000

Golder Associates

Created by: CW

Checked by: CNM



Ministry of Transportation

Ontario

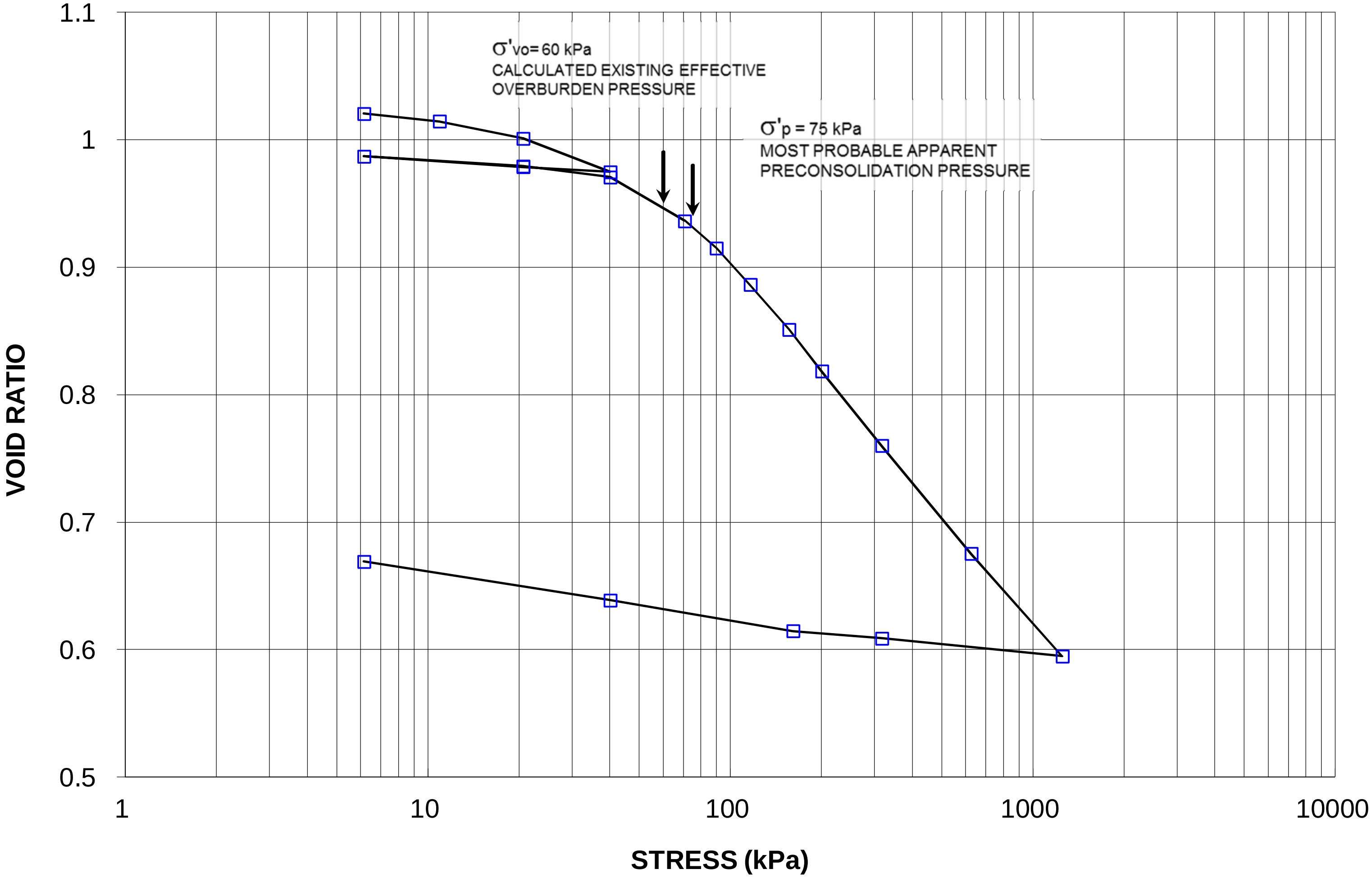
PLASTICITY CHART
Silty Clay

FIG No. A5

Project No. 11-1111-0017/3000

Compiled By : CW Checked By : CNM

CONSOLIDATION TEST
VOID RATIO vs STRESS
BH 12-105A



Project No. 11-1111-0017
Prepared By: LFG

Golder Associates

Checked By: CNM

CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE A6

CONSOLIDATION TEST SUMMARY	FIGURE A7

SAMPLE IDENTIFICATION

Project Number	11-1111-0017	Sample Number	-
Borehole Number	12-105A	Sample Depth, m	6.7-7.2

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	4		
Date Started	1/14/2013		
Date Completed	2/02/2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.53	Unit Weight, kN/m^3	18.41
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	13.37
Area, cm^2	31.71	Specific Gravity, measured	2.76
Volume, cm^3	80.22	Solids Height, cm	1.249
Water Content, %	37.74	Volume of Solids, cm^3	39.62
Wet Mass, g	150.60	Volume of Voids, cm^3	40.61
Dry Mass, g	109.34	Degree of Saturation, %	101.6

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	2.530	1.025	2.530				
6.14	2.524	1.020	2.527	304	4.45E-03	3.73E-04	1.63E-07
10.91	2.517	1.014	2.520	694	1.94E-03	6.38E-04	1.21E-07
20.65	2.500	1.001	2.508	227	5.88E-03	6.78E-04	3.90E-07
40.02	2.467	0.975	2.483	311	4.20E-03	6.69E-04	2.76E-07
20.65	2.472	0.979	2.470				
6.14	2.482	0.987	2.477				
20.65	2.473	0.979	2.478	94	1.38E-02	2.59E-04	3.51E-07
40.02	2.462	0.971	2.467	202	6.39E-03	2.20E-04	1.38E-07
70.71	2.419	0.936	2.441	2120	5.96E-04	5.54E-04	3.23E-08
89.97	2.392	0.915	2.406	12615	9.73E-05	5.50E-04	5.24E-09
116.30	2.356	0.886	2.374	5134	2.33E-04	5.37E-04	1.23E-08
156.44	2.313	0.851	2.335	3197	3.61E-04	4.30E-04	1.52E-08
200.49	2.272	0.818	2.292	6490	1.72E-04	3.68E-04	6.19E-09
316.41	2.199	0.760	2.235	882	1.20E-03	2.49E-04	2.93E-08
625.75	2.093	0.675	2.146	221	4.42E-03	1.35E-04	5.84E-08
1250.58	1.993	0.595	2.043	194	4.56E-03	6.36E-05	2.84E-08
316.41	2.010	0.609	2.001				
161.19	2.017	0.615	2.014				
40.02	2.047	0.639	2.032				
6.14	2.085	0.669	2.066				

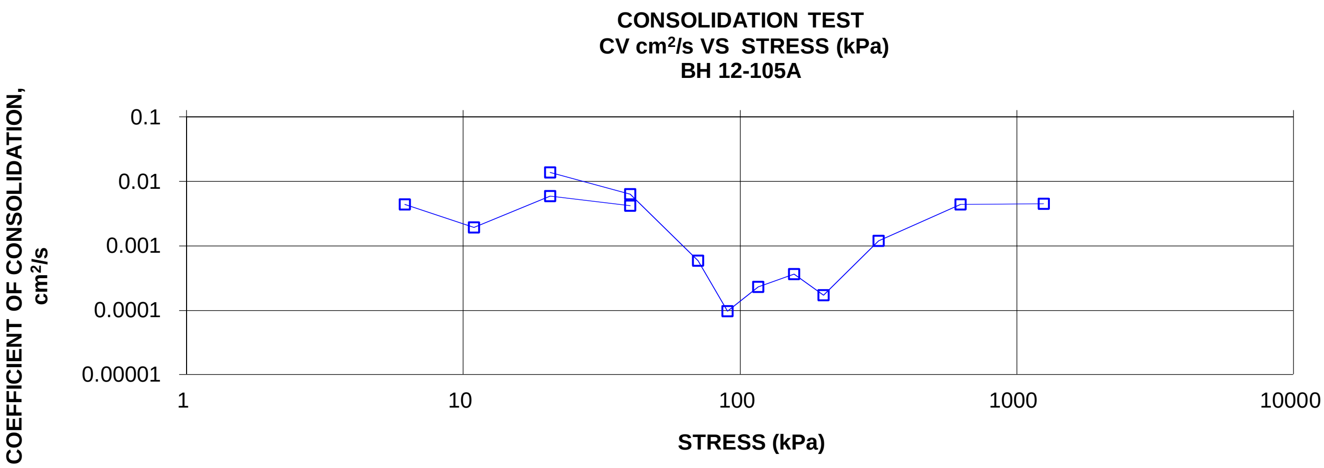
Note:
k calculated using cv based on t_{90} values.

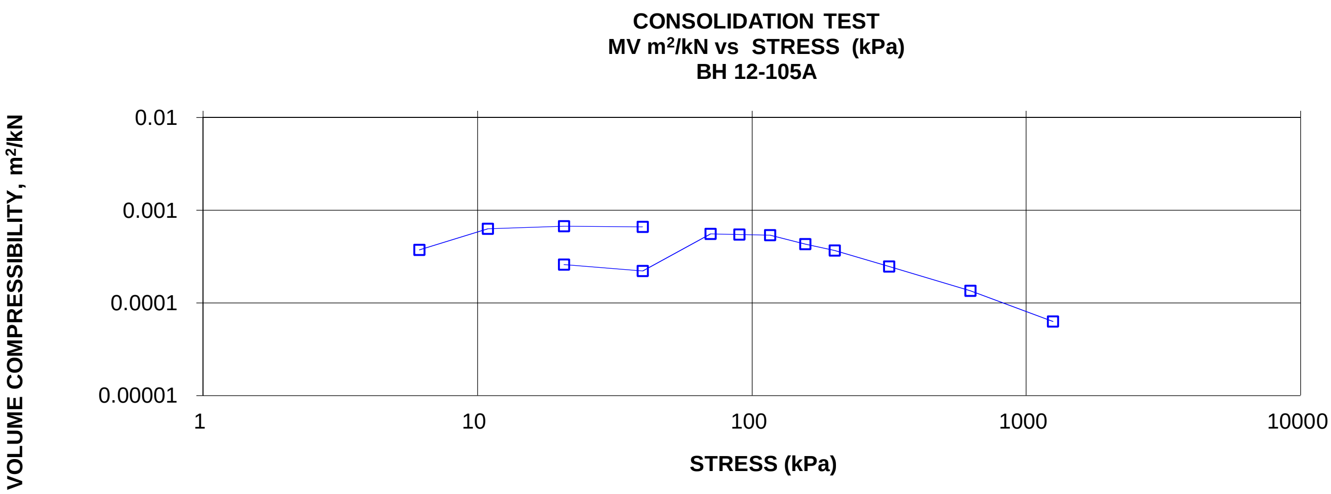
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

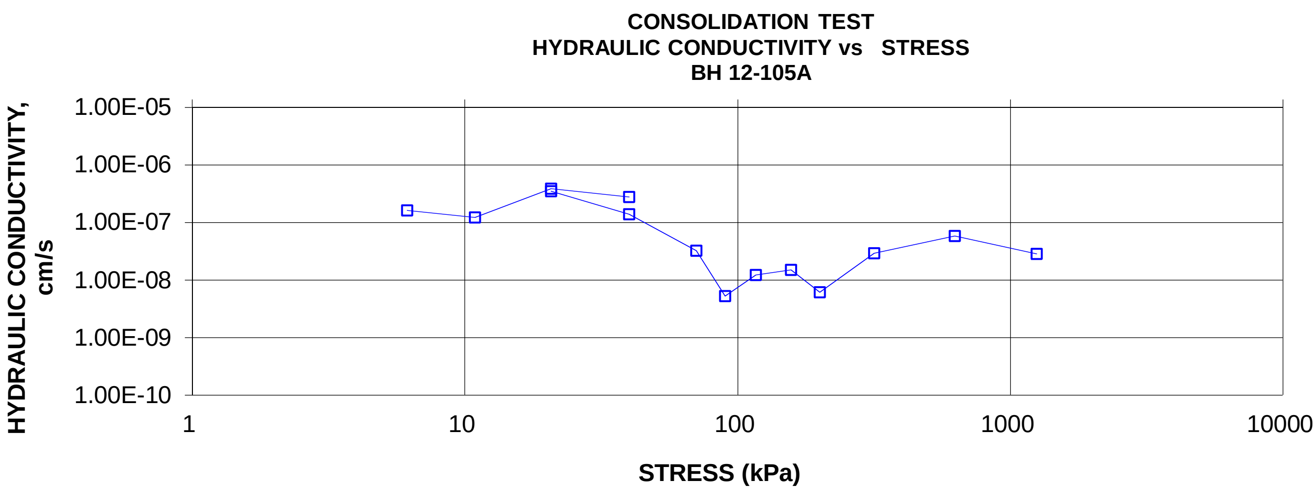
Sample Height, cm	2.09	Unit Weight, kN/m^3	20.37
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	16.22
Area, cm^2	31.71	Specific Gravity, measured	2.76
Volume, cm^3	66.12	Solids Height, cm	1.249
Water Content, %	25.60	Volume of Solids, cm^3	39.62
Wet Mass, g	137.33	Volume of Voids, cm^3	26.51
Dry Mass, g	109.34		

Prepared By: LFG **Golder Associates** Checked By: CNM

CONSOLIDATION TEST SUMMARY	FIGURE A8

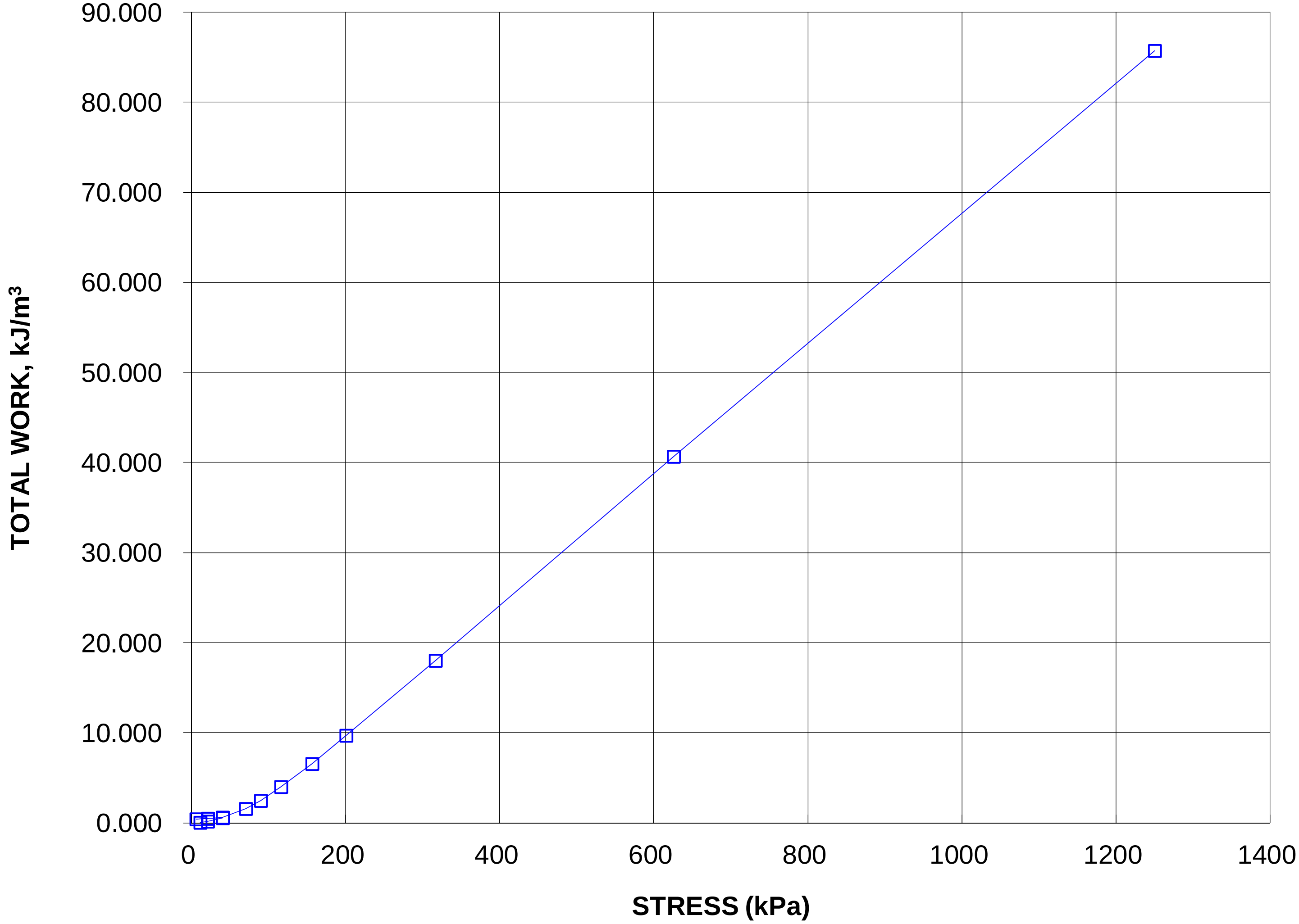






Project No. 11-1111-0017
Prepared By: LFG

CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs STRESS
BH 12-105A



Golder Associates

Checked By: CNM

CONSOLIDATION TEST
TOTAL WORK VS STRESS

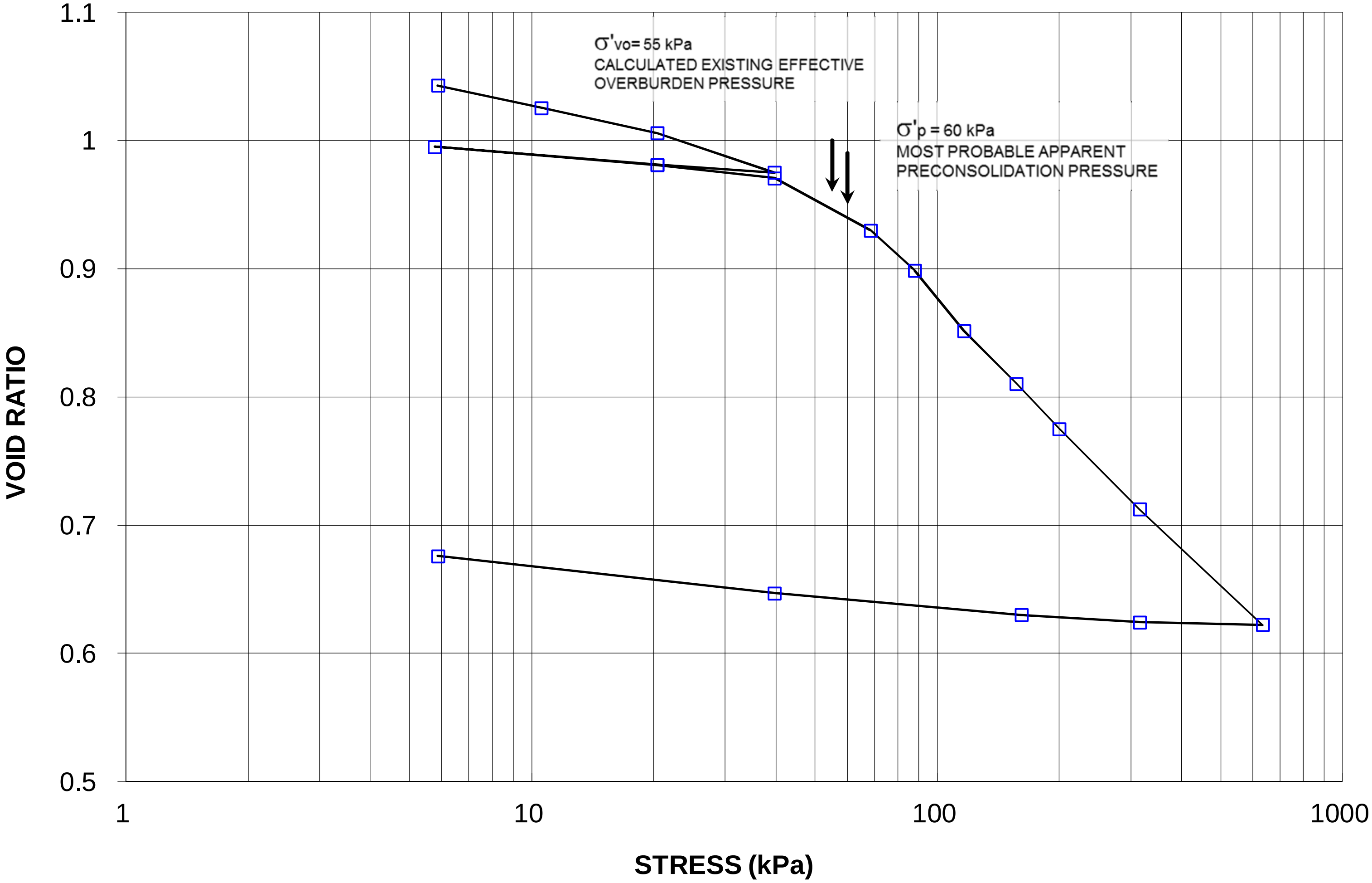
FIGURE A9

Project No. 11-1111-0017
Prepared By: LFG

Golder Associates

Checked By: CNM

CONSOLIDATION TEST
VOID RATIO vs STRESS
BH 12-107



CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE A10

CONSOLIDATION TEST SUMMARY	FIGURE A11

SAMPLE IDENTIFICATION

Project Number	11-1111-0017	Sample Number	-
Borehole Number	12-107	Sample Depth, m	4.4-5.0

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	1/14/2013		
Date Completed	2/03/2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.89	Unit Weight, kN/m^3	18.53
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	13.24
Area, cm^2	31.69	Specific Gravity, measured	2.76
Volume, cm^3	59.80	Solids Height, cm	0.923
Water Content, %	39.99	Volume of Solids, cm^3	29.25
Wet Mass, g	113.01	Volume of Voids, cm^3	30.55
Dry Mass, g	80.73	Degree of Saturation, %	105.7

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.887	1.044	1.887				
5.87	1.886	1.043	1.886	154	4.90E-03	1.35E-04	6.50E-08
10.56	1.870	1.025	1.878	265	2.82E-03	1.81E-03	5.00E-07
20.41	1.851	1.006	1.860	240	3.06E-03	9.79E-04	2.93E-07
39.68	1.823	0.975	1.837	482	1.48E-03	7.78E-04	1.13E-07
20.41	1.829	0.981	1.826				
5.77	1.842	0.995	1.835				
20.41	1.828	0.981	1.835	178	4.01E-03	4.78E-04	1.88E-07
39.68	1.819	0.971	1.824	190	3.71E-03	2.56E-04	9.30E-08
68.46	1.781	0.930	1.800	1325	5.18E-04	6.94E-04	3.53E-08
87.85	1.753	0.899	1.767	10297	6.43E-05	7.87E-04	4.96E-09
116.33	1.709	0.852	1.731	5881	1.08E-04	8.07E-04	8.54E-09
156.56	1.671	0.810	1.690	4914	1.23E-04	5.03E-04	6.07E-09
200.02	1.638	0.775	1.655	14519	4.00E-05	3.98E-04	1.56E-09
315.53	1.581	0.712	1.609	913	6.01E-04	2.66E-04	1.57E-08
633.82	1.498	0.622	1.539	144	3.49E-03	1.38E-04	4.72E-08
315.53	1.499	0.624	1.498				
161.40	1.505	0.630	1.502				
39.68	1.520	0.647	1.512				
5.87	1.547	0.676	1.534				

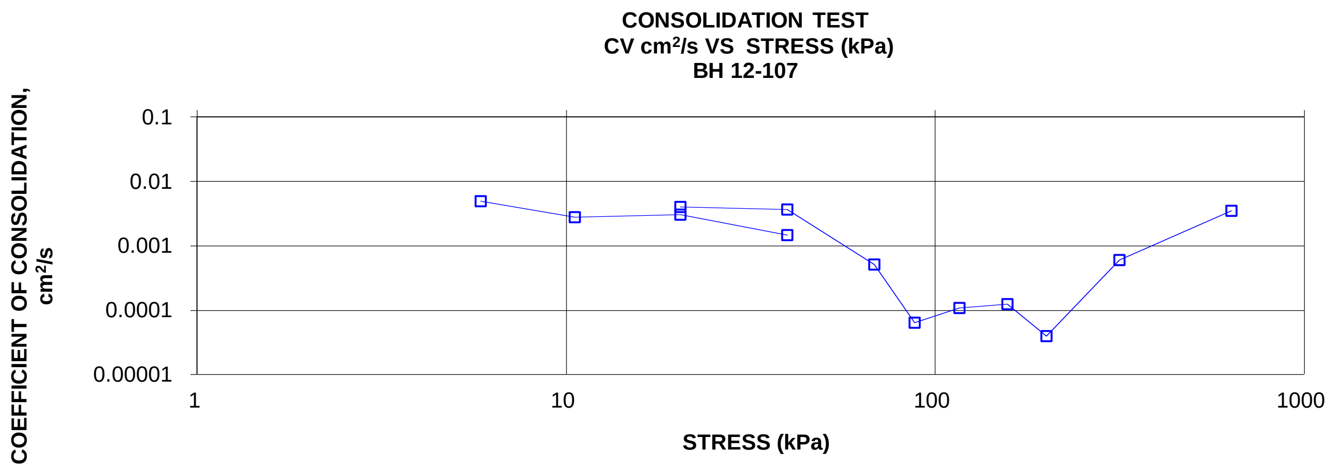
Note:
k calculated using cv based on t_{90} values.

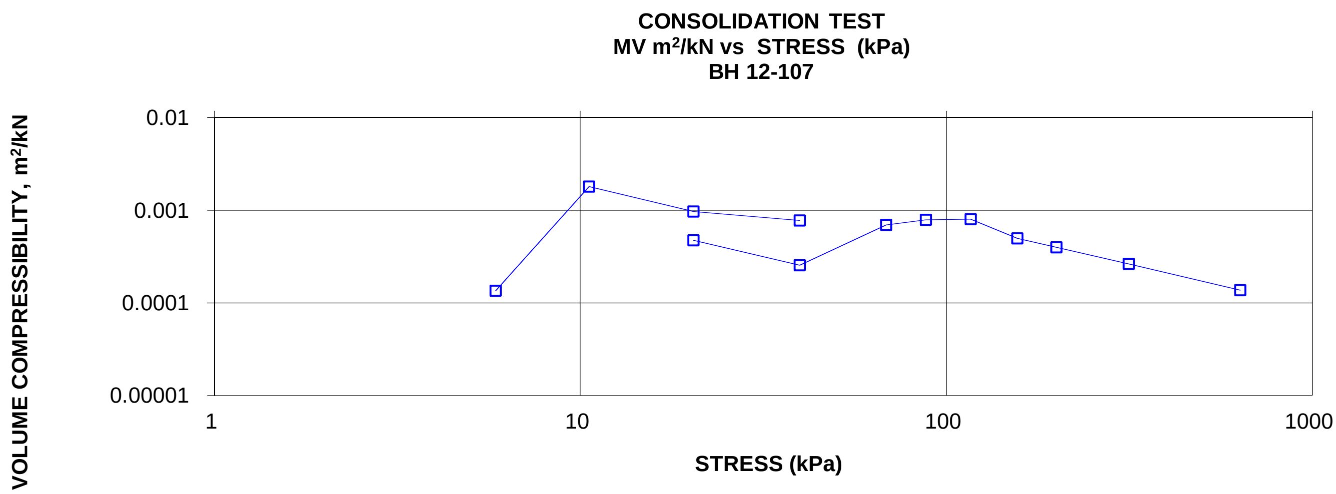
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

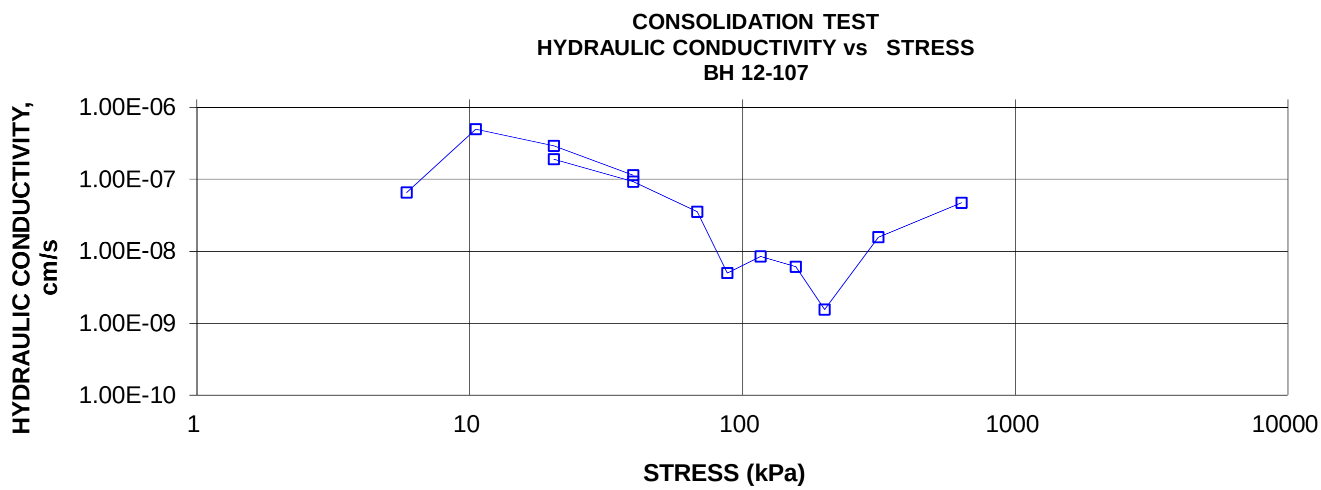
Sample Height, cm	1.55	Unit Weight, kN/m^3	20.69
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	16.15
Area, cm^2	31.69	Specific Gravity, measured	2.76
Volume, cm^3	49.02	Solids Height, cm	0.923
Water Content, %	28.11	Volume of Solids, cm^3	29.25
Wet Mass, g	103.42	Volume of Voids, cm^3	19.77
Dry Mass, g	80.73		

Prepared By: LFG **Golder Associates** Checked By: CNM

CONSOLIDATION TEST SUMMARY	FIGURE A12







Project No. 11-1111-0017

Prepared By: LFG

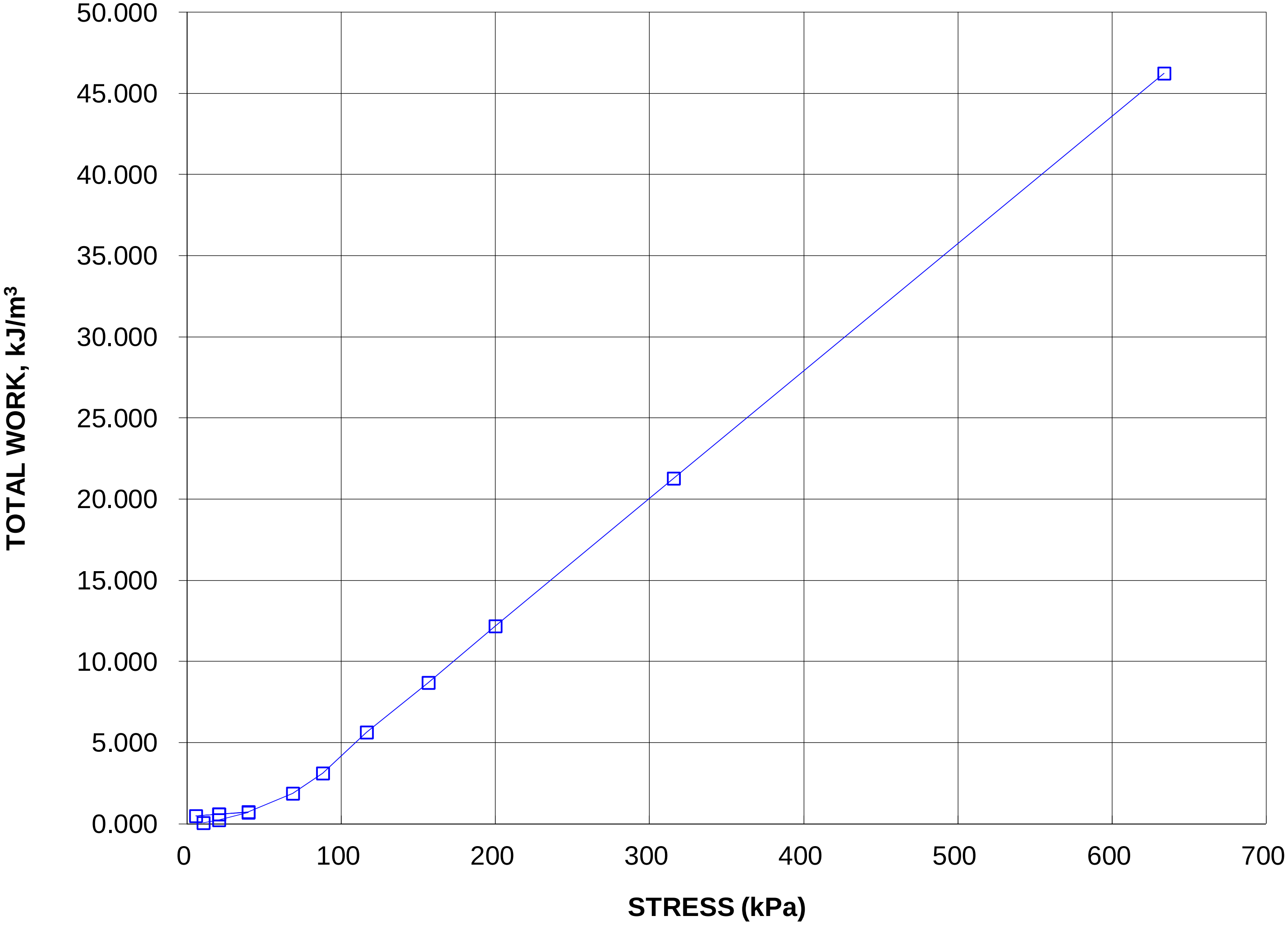
Golder Associates

Checked By: CNM

Project No. 11-1111-0017
Prepared By: LFG

Golder Associates

CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs STRESS
BH 12-107



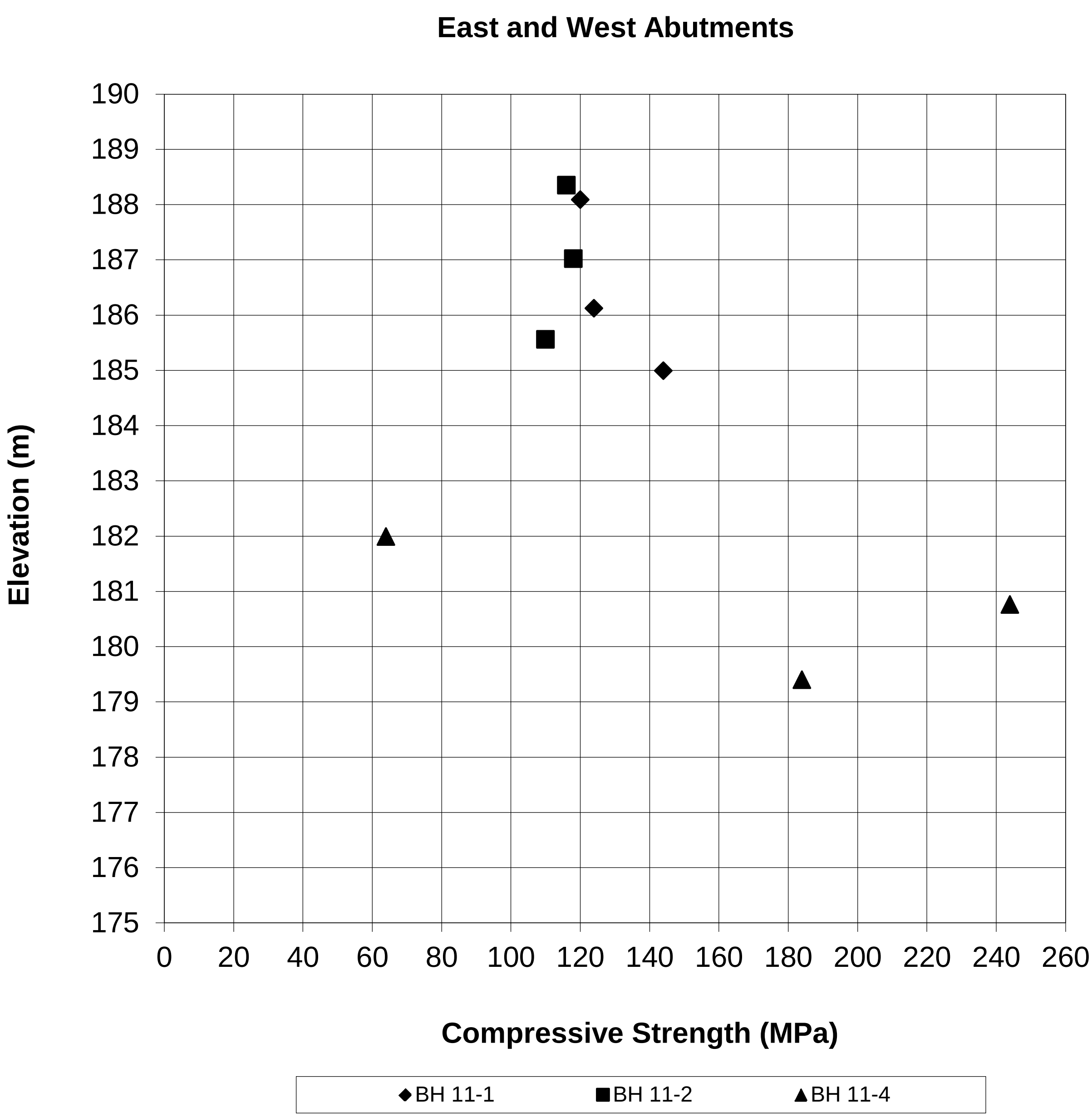
Checked By: CNM

CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE A13

SUMMARY OF LABORATORY COMPRESSIVE STRENGTH
POINT LOAD TESTING

FIGURE A14

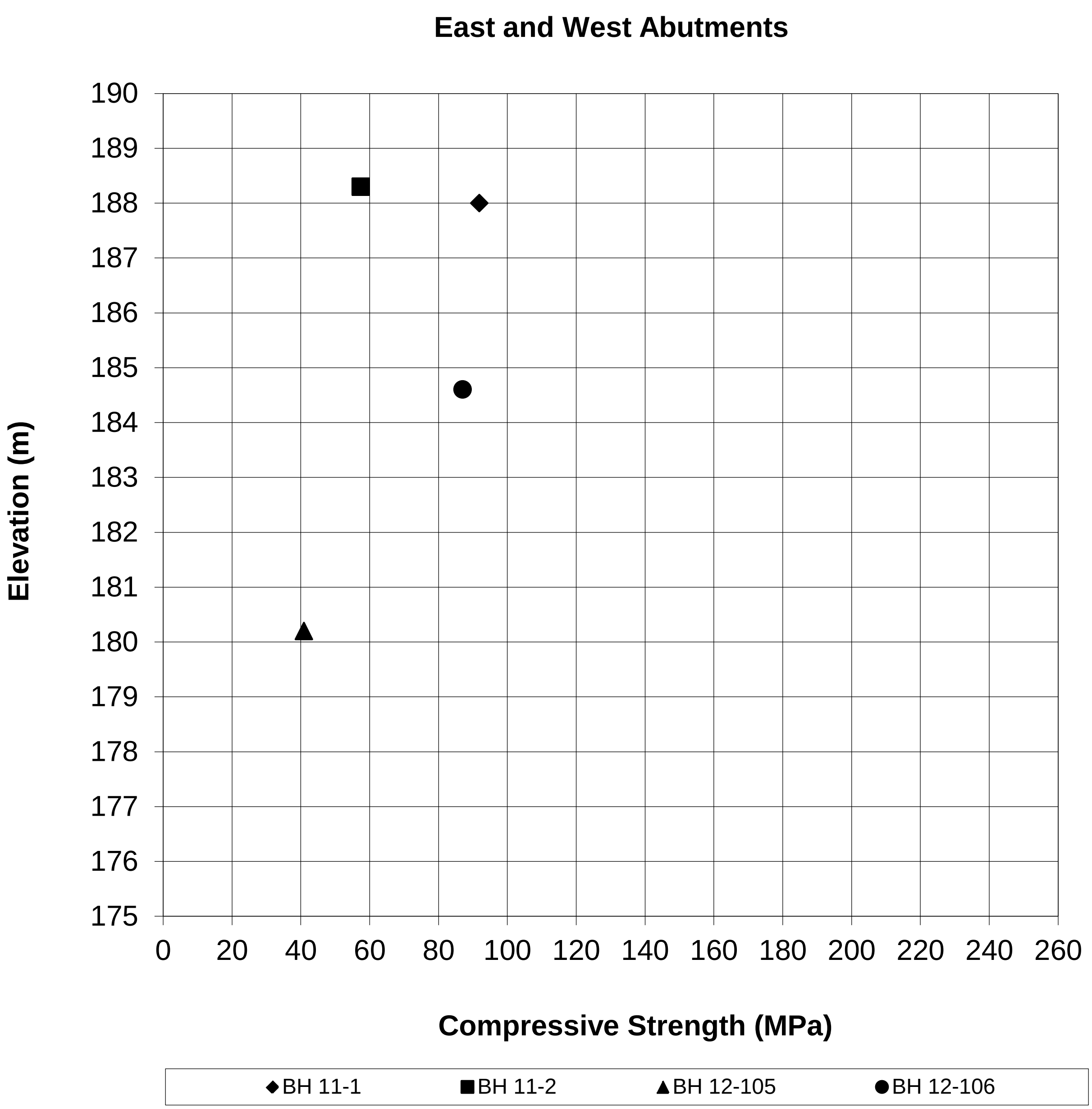


Project: 11-1111-0017 (1000)

Golder Associates

Created by:	NRL
Checked by:	FJH

SUMMARY OF LABORATORY COMPRESSIVE STRENGTH UNCONFINED COMPRESSION TESTS	FIGURE A15



Project: 11-1111-0017 (1000)

Golder Associates

Created by:	NRL
Checked by:	FJH

APPENDIX B

Record of Boreholes and Drillholes and Laboratory Test Results Preliminary Investigation (2011)

RECORD OF BOREHOLE No 11-1

 METRIC

PROJECT 11-1111-0017-1000
G.W.P. 4034-09-00 LOCATION N 4955513.0 ;E 270261.3 ORIGINATED BY P.A.H.
DIST HWY 7 BOREHOLE TYPE Wash Boring, NW Casing COMPILED BY J.M.
DATUM Geodetic DATE December 8-9, 2011 CHECKED BY N.R.L.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.7	GROUND SURFACE					
191.5 / 0.0	ASPHALTIC CONCRETE					
0.2	Crushed sand and gravel (FILL) Brown	1	GRAB			
191.1 / 0.6	Sand, gravel and cobbles (FILL) Grey-brown Moist				191	
190.8		2	SS	44		
190.6	Organic silty sand (FILL) Dark brown Moist					
1.1	ROCKFILL, some sandy silt and gravel infill Moist	3	NQ RC	DD	190	
189.2 / 2.5	Organic Silty SAND to organic CLAYEY SILT Black Moist to wet	4	SS	5	189	
188.4		5	SS	WR		
	COBBLE					
3.4	Mica Schist (BEDROCK), with thin beds of granite Fresh Dark Grey Bedrock cored between 3.4 m and 7.3 m depth. For bedrock coring details refer to Record of Drillhole 11-1.	1	RC	REC 100%	188	RQD = 100%
		2	RC	REC 100%	187, 186	RQD = 97%
		3	RC	REC 100%	185	RQD = 93%
184.4 / 7.3	End of Borehole					

Column headings also include: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³.

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

PROJECT: 11-1111-0017-1000

RECORD OF DRILLHOLE: 11-1

SHEET 1 OF 1

LOCATION: N 4955513.0 ;E 270261.3

DRILLING DATE: December 8-9, 2011

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole 11-1		188.30												
4	Rotary Drill NQ Core	Mica Schist (BEDROCK), with thin bands of granite Fresh Dark Grey		3.40	1		75 - 0									UC = 91.9 MPa
5					2		0									
6–7				184.40	3		0									
		End of Drillhole		7.30												

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: P.A.H.

CHECKED: N.R.L.

MIS-RCK 001 1111110017-1200 (ROCK).GPJ GAL-MISS.GDT 05/29/13 JM

RECORD OF BOREHOLE No 11-2

 METRIC

PROJECT 11-1111-0017-1000			
G.W.P. 4034-09-00	LOCATION N 4955527.7 ;E 270303.9	ORIGINATED BY P.A.H.	
DIST	HWY 7	BOREHOLE TYPE 200 mm Diam. Power Auger (Hollow Stem)	COMPILED BY J.M.
DATUM Geodetic	DATE December 9-12, 2011	CHECKED BY N.R.L.	

Columns: SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (25, 50, 75); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.4	GROUND SURFACE						
0.0	ASPHALTIC CONCRETE						
191.2 0.2	Crushed sand and gravel (FILL) Brown	1	GRAB		191		
190.9 0.5	Sand and gravel (FILL) Grey-brown Very moist						
190.4 1.0	SILTY CLAY, with clayey silt layers Very stiff Grey-brown Moist	2	SS	5	190		
189.9 1.5	Gravelly Sandy SILT and cobbles (TILL) Compact to dense Grey-brown to grey Very moist	3	SS	16		○	21 31 42 6
		4	SS	35	189		
188.6 2.8	Granitic Gneiss (BEDROCK), with interbeds of mica schist Fresh to slightly weathered Light grey to green Bedrock cored between 2.8 m and 7.0 m depth. For bedrock coring details refer to Record of Drillhole 11-2.	1	RC	REC 99%	188		RQD = 86%
		2	RC	REC 100%	187, 186		RQD = 98%
		3	RC	REC 100%	185		RQD = 100%
184.4 7.0	End of Borehole						

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

PROJECT: 11-1111-0017-1000

RECORD OF DRILLHOLE: 11-2

SHEET 1 OF 1

LOCATION: N 4955527.7 ;E 270303.9

DRILLING DATE: December 9-12, 2011

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole 11-2		188.60												
3	Rotary Drill NQ Core	Granitic Gneiss (BEDROCK), with interbeds of mica schist Fresh to slightly weathered Light grey to green		2.80	1		100									UC = 57.4 MPa
4–5					2		100									
6				184.40 7.00	3		100									
7		End of Drillhole														

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: P.A.H.

CHECKED: N.R.L.

MIS-RCK 001 1111110017-1200 (ROCK).GPJ GAL-MISS.GDT 05/29/13 JM

RECORD OF BOREHOLE No 11-3

 METRIC

PROJECT 11-1111-0017-1000
G.W.P. 4034-09-00 LOCATION N 4955498.3 ;E 270267.1 ORIGINATED BY P.A.H.
DIST HWY 7 BOREHOLE TYPE 200 mm Diam. Power Auger (Hollow Stem) COMPILED BY J.M.
DATUM Geodetic DATE December 12-13, 2011 CHECKED BY N.R.L.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
190.5	GROUND SURFACE										
0.0	TOPSOIL										
0.1	Fine sand (FILL) Brown				190						
190.1 0.4	Layered fine sand, silty sand and sandy silt, trace organic matter (FILL) Very loose Brown Moist	1	SS	3		○		0	42	41	17
189.1 1.4	Organic fine sand (TOPSOIL) Dark brown Wet				189						
188.8 1.7	Fine SAND, trace to some silt Very loose Brown Wet	2	SS	2							
188.1 2.4	Organic Silty SAND to organic CLAYEY SILT Dark brown to black Wet	3	SS	2	188						
		4	SS	2	187	○	Org=5.0%				
		5	SS	2		○	Org=6.9%				
		6	SS	2	186	147.3 ○	Org=16.7%				
185.2	SAND Grey Wet										
185.0 5.5	SILTY CLAY, trace organic mottling Soft Grey Wet	7	SS	1	185	H○		2	36	41	21
		8	SS		184						
		9	TP	PM	183						
181.3 9.2	SAND and GRAVEL, with cobbles and boulders Very dense Grey Wet	10	SS	50/0.1	182						
		11	NQ RC	DD	181						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75
REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

RECORD OF BOREHOLE No 11-3

 METRIC

PROJECT 11-1111-0017-1000

G.W.P. 4034-09-00

LOCATION N 4955498.3 ;E 270267.1

ORIGINATED BY P.A.H.

DIST HWY 7

BOREHOLE TYPE 200 mm Diam. Power Auger (Hollow Stem)

COMPILED BY J.M.

DATUM Geodetic

DATE December 12-13, 2011

CHECKED BY N.R.L.

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER (SAMPLES)	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SAND and GRAVEL, with cobbles and boulders Very dense Grey Wet		12	NQ RC	DD						
			13	SS	110		180				
179.7 10.8	End of Borehole Sampler Refusal Note: 1. Water level in Standpipe at 1.5 m depth (Elev. 189.0 m) on Jan. 15, 2013.										

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 11-4

 METRIC

PROJECT 11-1111-0017-1000
G.W.P. 4034-09-00
LOCATION N 4955513.5 ;E 270308.7
ORIGINATED BY P.A.H.
DIST HWY 7
BOREHOLE TYPE 200 mm Diam. Power Auger (Hollow Stem)
COMPILED BY J.M.
DATUM Geodetic
DATE December 14, 2011
CHECKED BY N.R.L.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	PLASTIC LIMIT / WATER CONTENT / LIQUID LIMIT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
190.7	GROUND SURFACE							
0.0	Sand and gravel, trace silt (FILL) Brown							
190.3 0.4	Silty sand and clayey silt, with organic matter (FILL) Very loose Brown and grey-brown Very moist to wet	1	SS	2	190			
189.0 1.7	Organic SILTY CLAY to organic CLAYEY SILT, with occasional silty sand layers Black Wet	2	SS	2	189			
		3	SS	2	188	plotted	Org=8.5%	
		4	SS	WH		112.5	Org=12.8%	
187.0 3.7	SILTY CLAY Soft Grey Wet	5	SS	WH	187	plotted		0 31 46 23
		6	SS	PH	186			
					185			
		7	SS	3		plotted		0 9 74 17
184.4 6.3	Silty SAND, some gravel Compact Grey-brown to grey with depth Saturated	8	SS	10	184			
		9	SS	9	183			
182.2 8.5	Mica Schist (BEDROCK) Fresh Dark grey	1	RC	REC 100%	182			RQD = 98%
181.1 9.6		2	RC		181			RQD = 100%

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
WATER CONTENT (%) 25 50 75

Continued Next Page

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

Golder Associates

PROJECT 11-1111-0017-1000	**RECORD OF BOREHOLE No 11-4**	2 OF 2	**METRIC**
G.W.P. 4034-09-00	LOCATION N 4955513.5 ;E 270308.7		ORIGINATED BY P.A.H.
DIST ___ HWY 7	BOREHOLE TYPE 200 mm Diam. Power Auger (Hollow Stem)		COMPILED BY J.M.
DATUM Geodetic	DATE December 14, 2011		CHECKED BY N.R.L.

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L / WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Quartz Mica Schist (BEDROCK), with thin to medium thick felsic granitic bands Fresh Light grey to white Bedrock cored between 8.5 m and 11.4 m depth. For bedrock coring details refer to Record of Drillhole 11-4.		2	RC	REC 100%		180				RQD = 100%
179.3 11.4	End of Borehole										

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

MIS-MTO 001 1111110017-1200.GPJ GAL-MISS.GDT 05/29/13 JM

PROJECT: 11-1111-0017-1000

RECORD OF DRILLHOLE: 11-4

LOCATION: N 4955513.5 ;E 270308.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: December 14, 2011

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	FLUSH % RETURN
		Continued from Record of Borehole 11-4	182.20		
9	Rotary Drill, NQ Core	Mica Schist (BEDROCK) Fresh Dark grey	8.50	1	100
10		Quartz Mica Schist (BEDROCK), with thin to medium thick felsic granitic bands Fresh Light grey to white	181.10 9.60	2	100
11					
		End of Drillhole	179.30 11.40		

Other column headings: SYMBOLIC LOG; PENETRATION RATE (m/min); COLOUR; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.3; DISCONTINUITY DATA (DIP w.r.t. CORE AXIS; TYPE AND SURFACE DESCRIPTION); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); DIAMETRAL POINT LOAD INDEX (MPa); NOTES WATER LEVELS INSTRUMENTATION

DEPTH SCALE

1 : 75

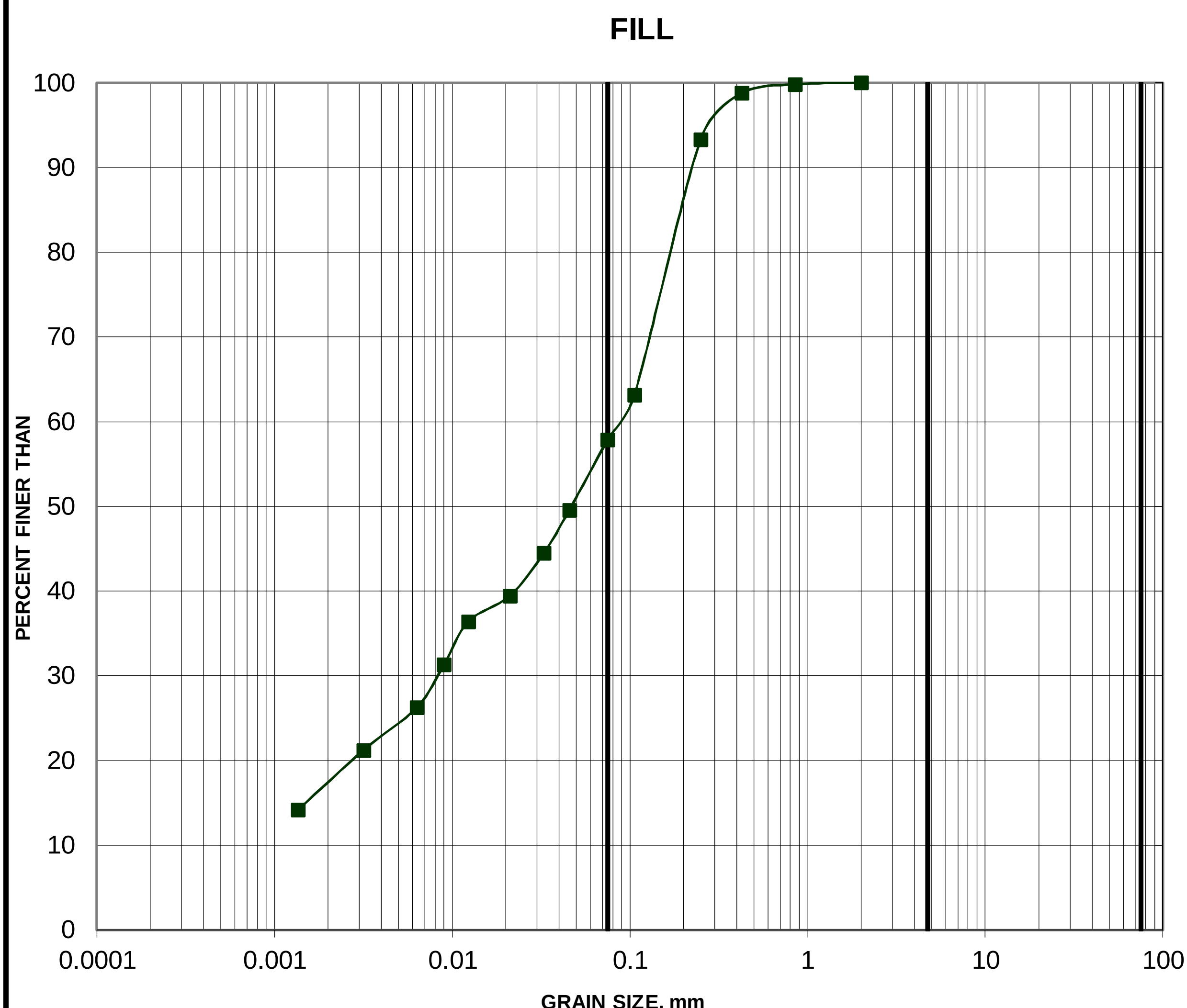
Golder Associates

LOGGED: P.A.H.

CHECKED: N.R.L.

GRAIN SIZE DISTRIBUTION

FIGURE B1



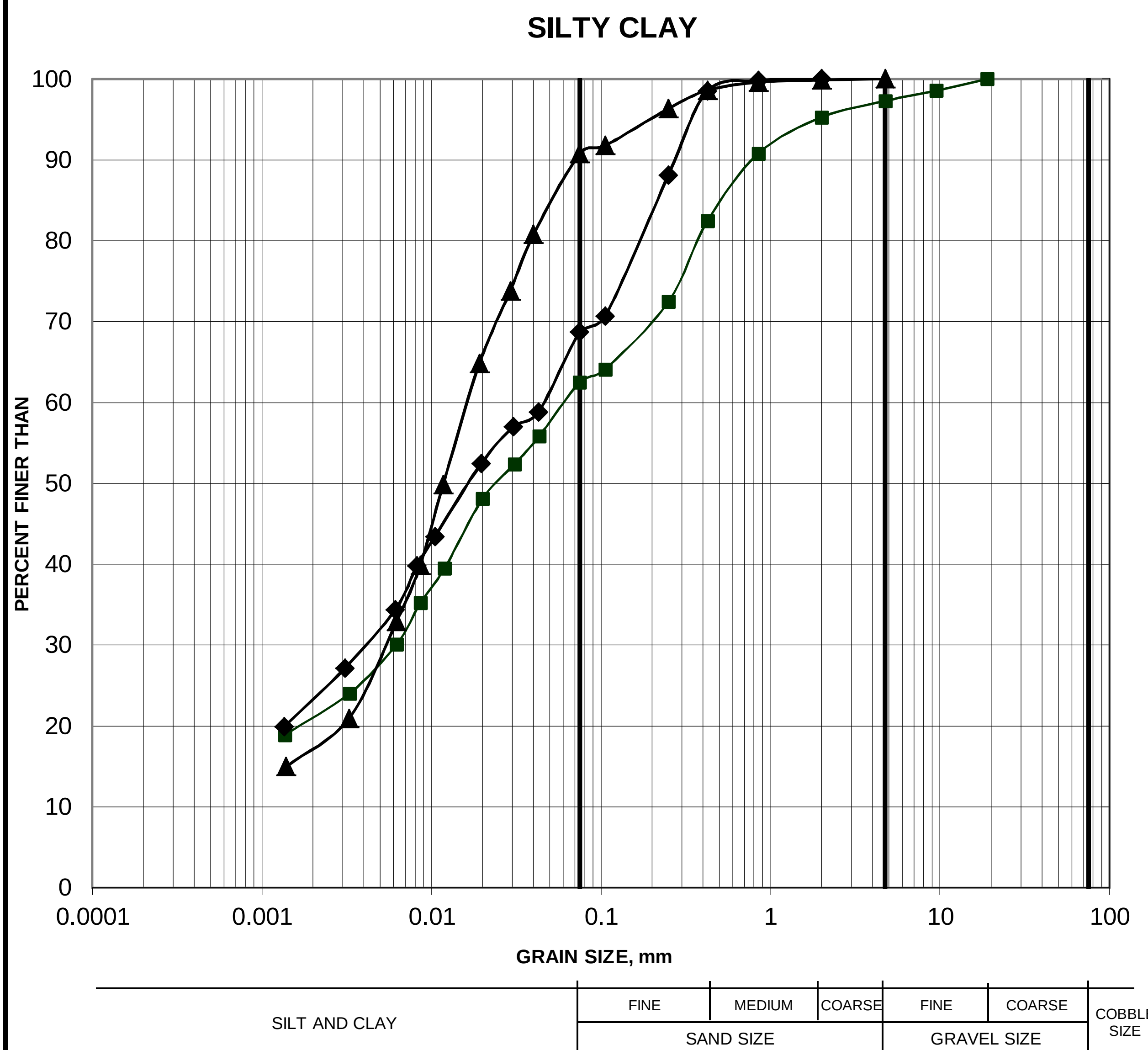
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
11-3	1	0.76-1.37

Created by:	CW
Checked by:	CNM

Project: 11-1111-0017/1300 **Golder Associates**

GRAIN SIZE DISTRIBUTION	FIGURE B2

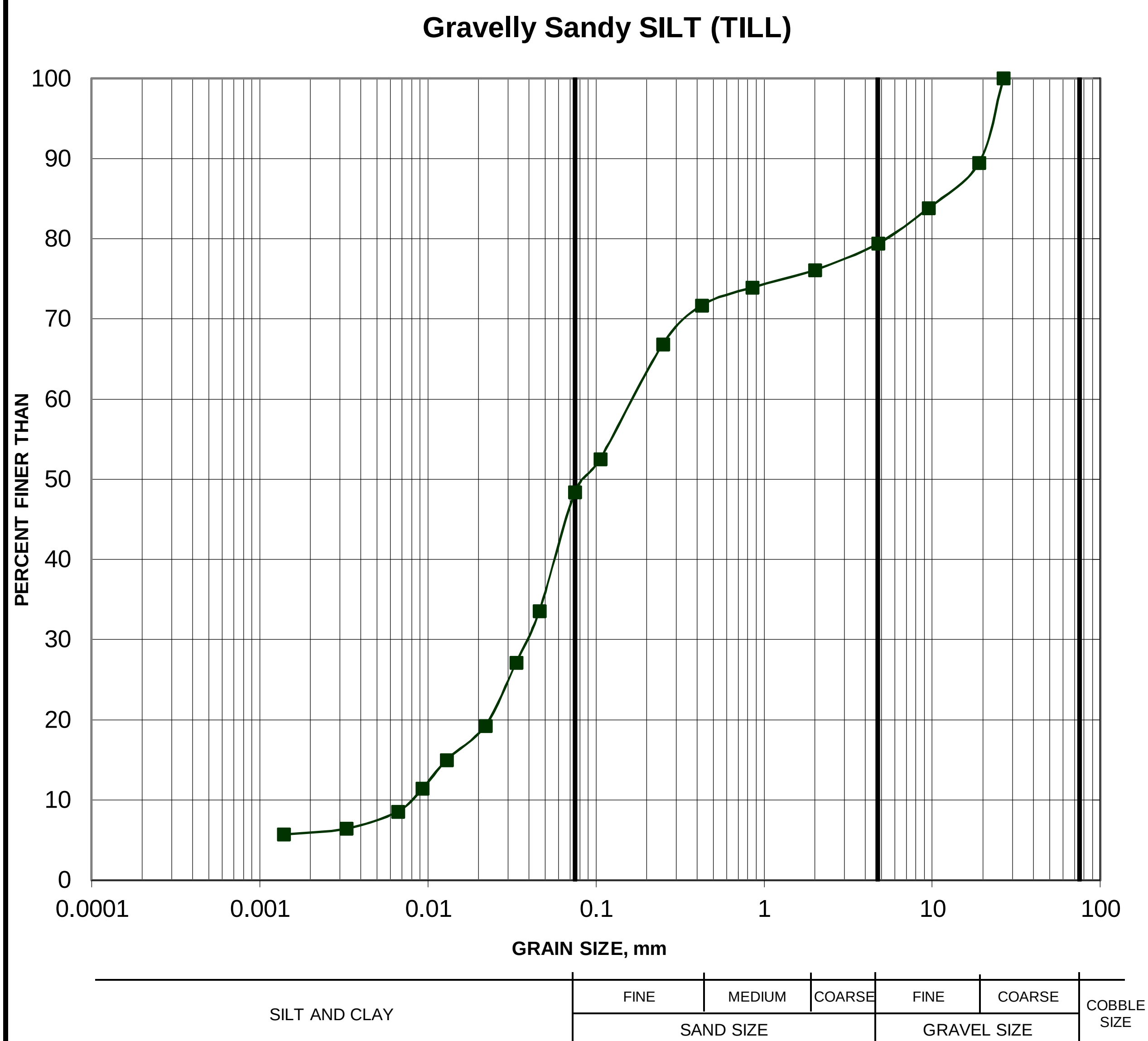


Borehole	Sample	Depth (m)
11-3	7	5.49-5.95
11-4	5	3.66-4.27
11-4	7	5.95-6.25

Project: 11-1111-0017/1300 **Golder Associates**

Created by:	CW
Checked by:	CNM

GRAIN SIZE DISTRIBUTION	FIGURE B3



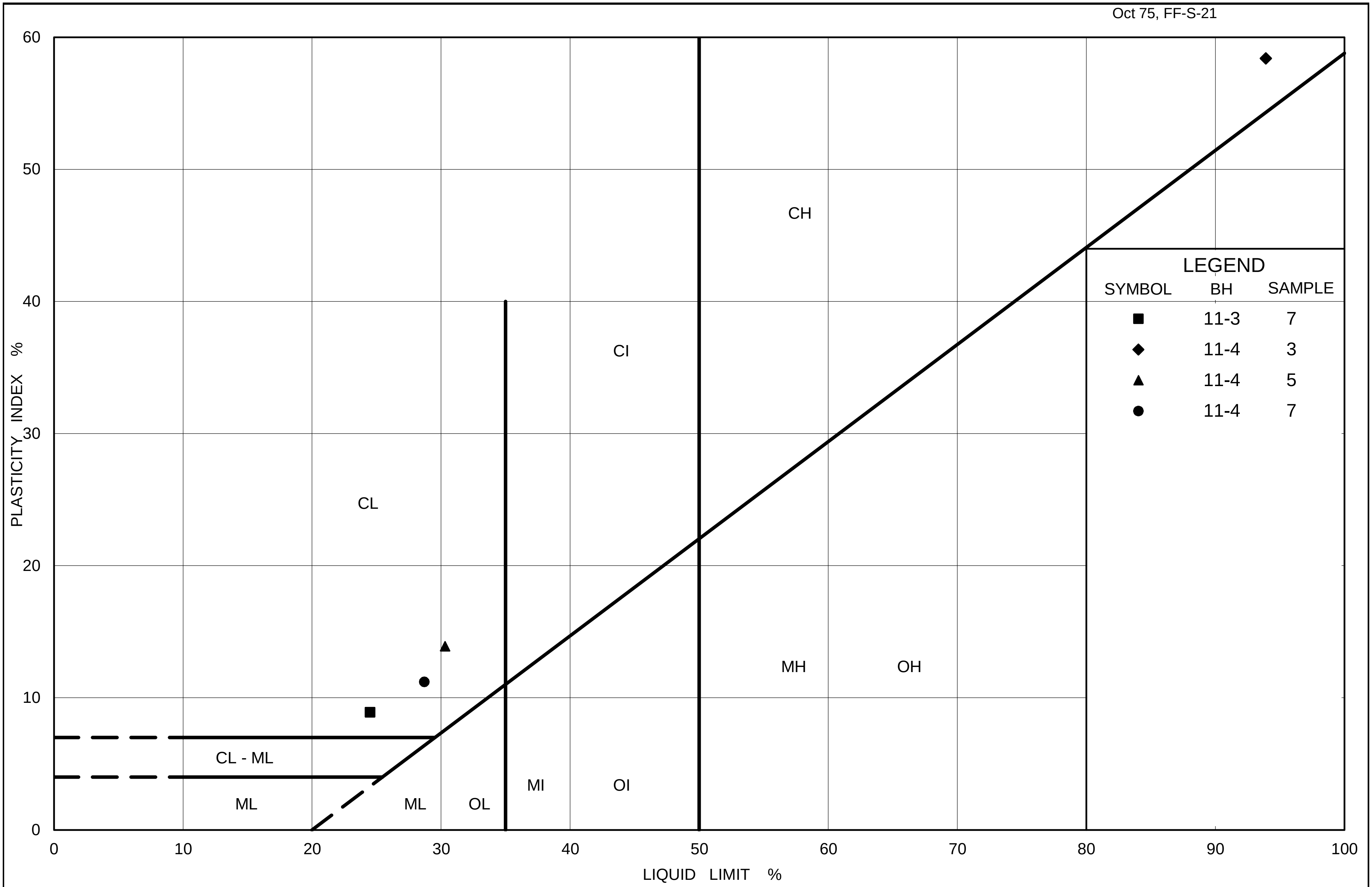
Borehole	Sample	Depth (m)
11-2	3	1.52-2.13

Project: 11-1111-0017/1300

Golder Associates

Created by: CW

Checked by: CNM



Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Clay and Organic Silty Clay

FIG No. B4

Project No. 11-1111-0017/1000

Compiled By : CW   Checked By : CNM

APPENDIX C

Sample Non-Standard Special Provision

BOULDERS/COBBLES DURING PILE OR SHORING INSTALLATION - Item No.

Non-Standard Special Provision

The overburden soils at the site include sandy deposits containing cobbles and boulders.

Appropriate equipment and procedures will be required to penetrate/remove cobbles/boulders that are encountered during pile driving or shoring installation.

BASIS OF PAYMENT

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

GROUNDWATER CONTROL - Item No.

Non-Standard Special Provision

SCOPE

Foundations for the foundations of the new abutments will require excavations to extend below the water level. Cohesionless soils (sands and silts) that are present below the groundwater table will slough, run, boil or cave into the excavation unless appropriate groundwater controls are in place. The Contractor is to design and install an appropriate dewatering system for the foundations to enable construction in dry conditions, and prevent disturbance to the founding soils.

REFERENCES

OPSS 518 Construction Specification for Control of Water from Dewatering Operations

SUBMISSION AND DESIGN REQUIREMENTS

A dewatering plan providing written details and shop drawings for the proposed dewatering system shall be submitted to the Contract Administrator for information purposes. This dewatering plan shall be submitted to the Contract Administrator a minimum of ten business days prior to commencing dewatering operations. The Contractor shall reference borehole logs included in the contract documents as a guide in determining requirements.

CONSTRUCTION

DEWATERING SYSTEM

The Contractor is responsible for the design, installation, operation, and maintenance of temporary dewatering systems to lower the groundwater level in the underlying sands and silts to at least 0.5 m below the bottom of the excavations to allow excavation, foundation subgrade preparation, and foundation construction to be carried out in a safe condition.

Water pumped from the system should be discharged in a manner that is not injurious to public health or safety, to property, to the environment, or to any part of the work already completed or under construction.

BASIS OF PAYMENT

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

H-PILES – HP310 X 110 - Item No.

Non-Standard Special Provision

903.07.02.07.03.03 Driving to Bedrock

Section 903.07.02.07.03.03 of OPSS 903 is deleted and replaced with the following:

When driving piles to bedrock, the Contractor shall adequately seat the pile on bedrock without damaging the pile.

In order to avoid overdriving and possibly damaging the piles when seating onto bedrock, the piles shall be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules. The bedrock elevation shall be recorded. On reaching the required set, the hammer energy shall be reduced to 75 percent of the maximum energy and the pile shall then be re-driven in 2 sets of 10 blows and the penetration recorded after each set of 10 blows. The hammer energy shall then be increased to 100 percent and the pile re-driven for 10 blows and the penetration recorded. A final set of no less than 10 blows per 12 mm of penetration shall be obtained at the maximum hammer energy.

If unrealistic excessive penetration per blow is observed, driving shall be stopped and this excessive penetration immediately reported to the Contract Administrator.

The Quality Verification Engineer shall determine when the hammer energy can be increased and when the driving is complete for each pile.

BASIS OF PAYMENT

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

CONCRETE WORKING SLAB – Item No.

Non-Standard Special Provision

1.0 SCOPE

This Special Provision covers the requirements for the supply and placement of a concrete working slab under the Matheson Boulevard overpass abutment widening foundations.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction

OPSS 902 Excavating and Backfilling - Structures

3.0 DEFINITIONS - Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS - Not Used

5.0 MATERIALS

Concrete for working slabs shall have a minimum 28 day strength of 20 MPa.

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 Excavation

Excavation for the working slab shall be according to OPSS 902.

7.02 Protection of Founding Soil

Following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents.

7.03 Protection of Founding Bedrock

The surface of the founding rock shall be exposed and cleaned and any loose or fractured parts removed so that sound rock is exposed. The working slab shall be placed on the exposed cleaned sound founding rock surface as specified in the Contract Documents.

The thickness of the mass concrete pad shall depend on the slope and irregularities in the exposed founding rock surface. A nominal thickness and a footprint plan view area has been specified on the Contract Documents

7.04 Dewatering

Dewatering shall be carried out in accordance with OPSS 902.

8.0 QUALITY ASSURANCE - Not Used

9.0 MEASUREMENT FOR PAYMENT – Not Used

10.0 BASIS OF PAYMENT

10.01 Working Slab - Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

END OF SECTION

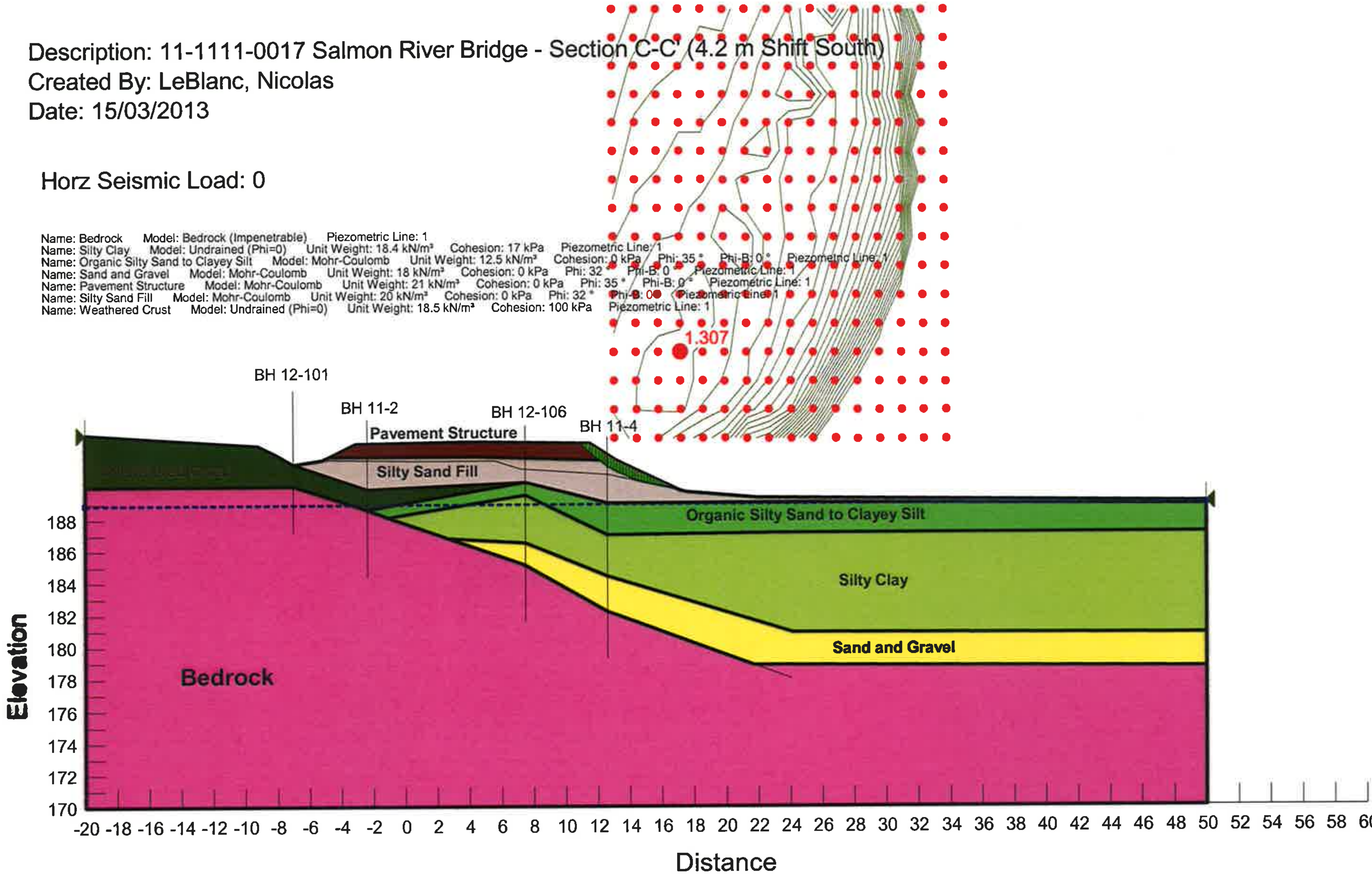
APPENDIX D

Sample Slope/W Output at Cross-Section C-C' (Station 22+720)

Description: 11-1111-0017 Salmon River Bridge - Section C-C' (4.2 m Shift South)
Created By: LeBlanc, Nicolas
Date: 15/03/2013

Horz Seismic Load: 0

Name: Bedrock Model: Bedrock (Impenetrable) Piezometric Line: 1
Name: Silty Clay Model: Undrained (Phi=0) Unit Weight: 18.4 kN/m³ Cohesion: 17 kPa Piezometric Line: 1
Name: Organic Silty Sand to Clayey Silt Model: Mohr-Coulomb Unit Weight: 12.5 kN/m³ Cohesion: 0 kPa Phi: 35 ° Phi-B: 0 ° Piezometric Line: 1
Name: Sand and Gravel Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 32 ° Phi-B: 0 ° Piezometric Line: 1
Name: Pavement Structure Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 35 ° Phi-B: 0 ° Piezometric Line: 1
Name: Silty Sand Fill Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 32 ° Phi-B: 0 ° Piezometric Line: 1
Name: Weathered Crust Model: Undrained (Phi=0) Unit Weight: 18.5 kN/m³ Cohesion: 100 kPa Piezometric Line: 1



Directory: N:\Active\2011\1111 -Mississauga\11-1111-0017-1000 Salmon River\Foundations - Detailed Design (Ph 3000)\Analysis\Settlement and Slope Analysis\
File Name: 11-1111-0017 C-C' 4.2m South Shift 1 March 2013.gsz

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
32 Steacie Drive
Kanata, Ontario, K2K 2A9
Canada
T: +1 (613) 592 9600

Golder Associates