



September 2012

REPORT ON

Foundation Investigation and Design Proposed Bridge Replacement Constant Creek Bridge Highway 132 G.W.P. 4034-05-00

Submitted to:
Genivar Consultants Ltd.
69 Cleak Ave.
Bancroft, Ontario
K0L 1C0

REPORT



A world of
capabilities
delivered locally

Report Number: 11-1121-0130

Geocres Number: 31F-181

Distribution:

2 copy - Genivar Consultants Ltd.

5 copy - Ministry of Transportation, Ontario, Kingston, Ontario

1 copy - MTO Foundations Section, Downsview, Ontario

2 copy - Golder Associates Ltd.





Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION.....	2
3.0 INVESTIGATION PROCEDURES	3
4.0 SITE GEOLOGY AND STRATIGRAPHY	5
4.1 Regional Geological Conditions	5
4.2 Site Stratigraphy	5
4.2.1 West Abutment and Approach Embankment	5
4.2.1.1 Sand Fill.....	5
4.2.1.2 Ice.....	6
4.2.1.3 Alluvium	6
4.2.1.4 Sand / Silt / Gravel.....	6
4.2.1.5 Bedrock	6
4.2.1.6 Groundwater Conditions	7
4.2.2 East Abutment and Approach Embankment	7
4.2.2.1 Ice.....	7
4.2.2.2 Peat / Organics / Topsoil	7
4.2.2.3 Sand / Silt / Gravel.....	8
4.2.2.4 Bedrock	8
4.2.2.5 Groundwater Conditions	9
5.0 CLOSURE.....	10
6.0 ENGINEERING RECOMMENDATIONS.....	11
6.1 General.....	11
6.2 Foundation Options	11
6.3 Steel H-Pile Foundations	12
6.3.1 Axial Geotechnical Resistance.....	12
6.3.2 Resistance to Lateral Loads.....	14
6.3.3 Frost Protection.....	16
6.4 Seismic Site Response Classification.....	16



FOUNDATION INVESTIGATION AND DESIGN REPORT

6.4.1	Liquefaction Assessment	16
6.4.2	Site Coefficient.....	17
6.5	Seismic Hazard Mitigation	17
6.5.1	General	17
6.5.1.1	Vibro-compaction.....	18
6.5.1.2	Dynamic Compaction.....	18
6.5.1.3	Rapid Impact Compactor	19
6.5.1.4	Rammed Aggregate Piers or Geopiers.....	19
6.6	Lateral Earth Pressures for Design.....	19
6.7	Approach Embankment Design and Construction	21
6.7.1	Subgrade Preparation and Embankment Construction	22
6.7.2	Approach Embankment Stability	22
6.7.3	Embankment Settlement.....	23
6.8	Design and Construction Considerations.....	24
6.8.1	Excavations.....	24
6.8.2	Groundwater and Surface Water Control	24
6.8.3	Obstructions.....	25
7.0	CLOSURE.....	26

TABLES

Table 1 – Comparison of Foundation Alternatives

Table 2 – Comparison of Ground Improvement Options

DRAWINGS

Drawing 1 – Highway 132, Constant Creek Bridge – Borehole Locations and Soil Strata

Drawing 2 – Highway 132, Constant Creek Bridge – Borehole Locations and Soil Strata

APPENDICES

APPENDIX A

Record of Boreholes and Record of Drillholes

APPENDIX B

Laboratory Test Results

APPENDIX C

Sample Non-Standard Special Provision



PART A

**Foundation Investigation
Proposed Bridge Replacement
Constant Creek Bridge
Highway 132 Realignment
From 0.2 km East of Highway 41
To 1.6 km East of the Constant Creek Bridge
G.W.P. 4034-05-00**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Genivar Inc. (Genivar) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation for the proposed Constant Creek Bridge replacement as part of the rehabilitation and realignment of Highway 132 from 0.2 km east of highway 41 to 1.6 km east of the constant creek bridge in Dacre, Ontario.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated March 5, 2009 and in Section 5.8 (Foundations Engineering) of the *Technical Proposal* for this assignment. The work was carried out in accordance with Golder's Quality Control Checklist dated March 2012.

Consideration is being given to carrying out this project by design build.



2.0 SITE DESCRIPTION

The existing single span bridge structure at Constant Creek will be replaced with a new single span structure that will be located approximately 50 m to the north of the existing bridge. This new bridge location will provide a straighter alignment of Hwy 132 in this area, which currently curves into and out of the existing bridge location in an 'S turn' configuration.

Through this area, Highway 132 is a two lane undivided highway with a rural cross-section. The existing structure is aligned approximately northwest-southeast. The highway profile grade over the structures is at approximately Elevation 187.5 m to 187.6 m. The existing structure consists of a single-span concrete bridge supported on concrete abutments. Construction drawings for the existing structure are not available and the type of foundation of the existing abutments is unknown.

Constant Creek runs beneath the existing Highway 132 structure with the water level at about Elevation 185.6 m at the time of the geotechnical investigation. Preliminary Design drawings produced by Genivar indicate the water levels to be at about Elevation 185.9 m and 185.0 m in August 2009 and March 2012, respectively.

The existing approach embankments are about 2 m to 3 m high relative to the banks of Constant Creek and have approximately 2H:1V side slopes. No signs of embankment instability were observed.

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.

The proposed bridge replacement will be located along the new Highway 132 alignment approximately 50 m north of the existing bridge as shown on Drawing 1 and will be oriented in an east-west direction.



3.0 INVESTIGATION PROCEDURES

The subsurface investigation for the bridge structure was carried out between February 9 and 20, 2012, at which time six boreholes were advanced at the locations shown on Drawing 1. The borehole locations were selected as follows:

- Two boreholes (numbered 12-1 and 12-6) within the proposed approach embankment footprints, located about 20 m from the proposed abutments.
- Two boreholes (numbered 12-2 and 12-3) and one dynamic cone penetration test (DCPT), located at the proposed west abutment.
- Two boreholes (numbered 12-4 and 12-5), located at the proposed east abutment.

The boreholes and DCPT were advanced using a CME-55 track mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. With the exception of borehole 12-1, the boreholes were advanced using NW size casing. Borehole 12-1 was advanced using 200 mm outside diameter hollow stem augers. The boreholes were advanced for between 3.9 and 11.3 m in the overburden. The hollow stem augers and NW casing prevented soil cave in or sloughing. In addition, the hollow stem augers and NW casing were filled with synthetic drilling mud to ground surface to avoid quick conditions prior to sampling. Soil samples of the overburden were obtained at intervals ranging from 0.6 to 1.5 m of depth, using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures.

Where auger refusal was encountered at foundation elements, the boreholes were advanced between about 3.5 and 5.6 m into the bedrock by using NQ-Size coring equipment. The bedrock was also cored for 2.1 m in Borehole 12-6 at the east approach embankment.

A standpipe piezometer was installed in Borehole 12-1 to monitor the groundwater level at the site. The standpipe consists of a 19 mm diameter rigid PVC pipe with a 0.6 m long slotted section, installed within silica sand backfill and sealed by sections of bentonite pellet backfill. The water level in the standpipe piezometer was frozen near ground surface when measured on February 21, 2012 and is noted on the Record of Borehole sheet in Appendix A. A further reading was taken on April 18, 2012.

The boreholes were backfilled with bentonite pellets mixed with native soil in the overburden and bentonite pellets in the bedrock. The site conditions were restored following completion of the work.

The field work was supervised throughout by members of Golder's technical staff, who located the boreholes, arranged for clearance of underground utilities, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and bedrock 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 and water content testing were carried out on selected soil samples at the Ottawa laboratory. Unconfined compressive strength tests were carried out on selected rock core samples in the Mississauga laboratory. All of the laboratory tests were carried out to MTO and/or ASTM standards as appropriate.

The borehole locations were determined by Golder Associates with respect to existing site features. The ground surface elevations were also determined by Golder Associates and were referenced to the bench marks provided by Genivar (i.e., Control Points B and F). The elevation at Control Point B is understood to be 189.30 m and the elevation at Control Point F is understood to be 187.42 m. These elevations are understood to



FOUNDATION INVESTIGATION AND DESIGN REPORT

be referenced to Geodetic datum. The boreholes and locations, including MTM NAD83 northing and easting coordinates and ground surface/ice surface elevations referenced to Geodetic datum, are summarized in the following table and are shown on the Record of Borehole sheets in Appendix A and on Drawings 1 and 2.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface/ Ice Surface Elevation (m)
12-1	West approach embankment, north side	5025293.9	270333.5	185.7
12-2	West abutment, north side	5025291.1	270345.4	*185.5
12-3	West abutment, south side	5025283.6	270340.3	*185.6
12-4	East abutment, north side	5025287.5	270360.3	*185.6
12-5	East abutment, south side	5025279.3	270357.4	*185.6
12-6	East approach embankment, south side	5025277.3	270370.4	186.4

Note: * Ice surface at the borehole location at the time of drilling.



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 Algonquin Highlands.

The Algonquin Highlands region is characterized by frequent outcrops of granite and other strong Precambrian bedrock, which can extend as high as 160 m above the surrounding land. The thickness of soils over the bedrock can vary greatly over short distances, with many of the valleys between the bedrock outcrops floored with outwashed sand, silt and gravel. Several areas within this region have deeper deposits of glacial till with few bedrock outcrops.¹

4.2 Site Stratigraphy

The borehole locations and ground surface/water level elevations from the recent borehole investigation are shown on Drawings 1 and 2. A soil stratigraphy section projected along the bridge centreline is shown on Drawing 1 and across the abutment foundation areas is shown on Drawing 2. The stratigraphic boundaries shown on the drawings and 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 at the west abutment and approach embankment of the bridge (Boreholes 12-1 to 12-3) generally consist of between about 0.4 m and 0.9 m of alluvium underlain by granular deposits from 8.5 m to greater than 10.7 m thick, consisting of sand to silty sand to sandy silt to silt. The overburden soils are underlain by marble bedrock, which was encountered between about 9.6 m and 11.0 m below ground surface corresponding to Elevations 176.0 m and 174.5 m, respectively. The subsurface conditions encountered at the east abutment and approach embankment of the bridge (Boreholes 12-4 to 12-6) generally consist of between about 0.2 m and 0.3 m of peat and topsoil underlain by granular deposits between about 3.7 m and 5.3 m thick, consisting of sand to silty sand underlain by gravelly sand to sand and gravel. The overburden soils are underlain by marble bedrock, which was encountered between about 3.9 m and 5.8 m below ground surface corresponding to Elevations 182.5 m and 179.8 m, respectively.

One DCPT was advanced within the footprint of the proposed west abutment about 1.8 m south of Borehole 12-2, the results of which are shown on the Record of Borehole sheet for Borehole 12-2 in Appendix A.

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

4.2.1 West Abutment and Approach Embankment

4.2.1.1 Sand Fill

A fill deposit consisting of sand, trace gravel containing organic matter was encountered at ground surface in Borehole 12-1. The sand fill extends to a depth of about 0.2 m below ground surface corresponding to Elevation 185.5 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.



4.2.1.2 Ice

A layer of ice was encountered at the ground surface in the boreholes advanced at the west abutment (Boreholes 12-2 and 12-3). The ice was measured to be between about 0.2 m and 0.4 m thick.

4.2.1.3 Alluvium

Alluvial deposits consisting of black silty sand to sand containing organic matter and occasional peat layers were encountered underlying the fill in Borehole 12-1 and the ice in Boreholes 12-2 and 12-3 at depths ranging from 0.2 m to 0.4 m below the ice surface (Elevation 185.1 m to 185.5 m). The Alluvial deposits extend to depths between about 0.6 m and 1.1 m below ground surface corresponding to Elevation 184.4 m to 185.1 m.

An SPT “N”-value of 0 blows (weight of hammer) per 0.3 m of penetration was measured within the alluvial deposits, indicating a very loose relative density.

The measured water content of two samples of the alluvial deposits are 22 percent and 95 percent and the corresponding organic content measured on the two samples are about 2 percent and 11 percent.

4.2.1.4 Sand / Silt / Gravel

Granular deposits consisting of silt, sandy silt, sand and silt and silty sand were encountered underlying the alluvial deposits at depths ranging between about 0.6 m and 1.1 m below ground surface (Elevation 184.4 m to 185.1 m) at the approach embankment and abutment boreholes. At the west abutment boreholes (12-2 and 12-3) the granular deposits graded to sand and gravel or gravelly silt at depths of about 8.2 m and 8.5 m below ground surface. The granular deposits extend to depths of about 9.6 m and 11.0 m at the abutment boreholes corresponding to Elevations 174.5 m and 176.0 m. At the approach borehole the granular deposit was not fully penetrated to a depth of 11.3 m below ground surface (Elevation 174.4 m).

SPT “N”-values recorded within the fine grained deposits generally range between 2 blows and 9 blows per 0.3 m of penetration, indicating a very loose to loose relative density. Higher SPT “N”-values of 14 and 41 blows per 0.3 m were recorded within the sand and gravel deposits. Refusal to advancement of the drilling equipment was frequently encountered within the sand and gravel deposits at depth due to the presence of cobbles and boulders. Rotary diamond drilling/coring techniques were required to advance the boreholes within these deposits at the west abutment.

Grain size distribution testing was carried out on 31 samples of the granular deposits. The results for the silty sands and sandy silt on the west boreholes are shown on Figure B1, B2 and B3 in Appendix B. These samples were retrieved using a 50 mm diameter sampler and therefore the results do not reflect the cobble and boulder content of the deposits.

The measured water content of the granular deposits ranges from approximately 17 to 41 percent.

4.2.1.5 Bedrock

Bedrock was encountered beneath the granular deposits and cored for 5.4 m to 5.6 m depth, in the abutment boreholes.

The following table summarizes the bedrock surface depths and elevations as encountered at the borehole locations at the abutments where bedrock was cored.



FOUNDATION INVESTIGATION AND DESIGN REPORT

Borehole Number	Existing Ground Surface/Ice Level Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
12-2	185.5	11.0	174.5
12-3	185.6	9.6	176.0

The bedrock encountered in the boreholes typically consists of grey medium to thickly bedded marble with quartz banding. The bedrock is typically fresh, contains zones of fractured, highly weathered rock and is typically medium strong to very strong.

The Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 70 to 100 percent, indicating bedrock of fair to excellent quality. RQD values of 0 were measured within the fractured, highly weathered zones. The discontinuities and bedding observed in the rock core are typically angled approximately 45 degrees to the axis of the drill core.

One unconfined compressive strength test measured a value of about 23 MPa, indicative of weak rock. Results of the UCS test are included on Figure B5 in Appendix B

4.2.1.6 Groundwater Conditions

A piezometer was installed in borehole 12-1 to monitor the groundwater level at the site. This piezometer was sealed into the granular deposit at a depth of about 7 m to 8.5 m. Details of the piezometer installation are shown on the Record of Borehole Sheet in Appendix A. The water level measured in the piezometer is summarized below.

Borehole	Installation	Ground Surface Elevation (m)	Depth to Water Level (m)	Water Level Elevation (m)	Date
12-1	Piezometer	185.7	0.06* 0.16	185.6 185.5	February 21, 2012 April 18, 2012

* Note: Water frozen in monitoring well at depth indicated.

The creek level was at elevation 185.56 on April 18, 2012.

It should be noted that groundwater levels at the site are expected to fluctuate seasonally in the granular soils and are expected to reflect the creek level.

4.2.2 East Abutment and Approach Embankment

4.2.2.1 Ice

A layer of ice was encountered at the ground surface in the boreholes advanced at the east abutment (Boreholes 12-4 and 12-5). The ice was measured to be about 0.2 m thick.

4.2.2.2 Peat / Organics / Topsoil

Organic deposits consisting of peaty organics or silty sand topsoil were encountered at ground surface in Borehole 12-6 and underlying the ice in Boreholes 12-4 and 12-5 at depths of 0.2 m below ground surface (Elevation 185.1 m to 185.4 m). The organic deposits extend to depths between about 0.2 m and 0.5 m below ground surface corresponding to Elevation 185.1 m to 186.2 m.



4.2.2.3 Sand / Silt / Gravel

Granular deposits consisting of sandy silt, silty sand, sand, sand and gravel and gravelly sand containing cobbles and boulders were encountered underlying the organic deposits at depths ranging between about 0.2 m and 0.5 m below ground surface (Elevation 185.1 m to 186.2 m) at the approach embankment and abutment boreholes. The granular deposits extend to depths between about 3.9 m and 5.8 m corresponding to Elevations 182.5 m and 180.3 m.

SPT “N”-values recorded within the granular deposits generally range between 13 blows and 47 blows per 0.3 m of penetration, indicating a compact to dense relative density, however some lower SPT “N”-values between about 2 blows and 4 blows per 0.3 m of penetration were recorded in the upper approximately 1 m of these deposits. Higher SPT “N”-values were recorded within the deposit and at depth are likely parry due to the presence of cobbles and boulders. Refusal to advancement of the drilling equipment was frequently encountered within the sand and gravel and gravelly silt deposits due to the presence of cobbles and boulders. Rotary diamond drilling/coring techniques were required to advance the boreholes within these deposits.

Grain size distribution testing was carried out on 5 samples of the granular deposits, the results of which are shown on Figure B4 in Appendix B. These samples were however retrieved using a 50 mm diameter sampler and therefore the results do not reflect the cobble and boulder content of the deposits.

The measured water content of the granular deposits ranges from approximately 11 to 36 percent.

4.2.2.4 Bedrock

Bedrock was encountered underlying the granular deposits and cored for 3.5 m depth, at abutment boreholes and for 2.1 m at the approach embankment.

The following table summarizes the bedrock surface depths and elevations as encountered at the borehole locations where bedrock was cored.

Borehole Number	Existing Ground Surface/Water Level Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
12-4	185.6	5.3	180.3
12-5	185.6	5.8	179.8
12-6	186.4	3.9	182.5

The bedrock encountered in the boreholes typically consists of grey thickly bedded marble. The bedrock is typically slightly weathered to fresh, and is typically very strong.

The Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 78 to 100 percent, indicating bedrock of good to excellent quality.

One unconfined compressive strength test measured a value of about 46 MPa, indicative of strong rock. Results of the UCS test are included on Figure B4 in Appendix B.



4.2.2.5 *Groundwater Conditions*

The groundwater level was observed during the drilling process. It should be noted that groundwater levels at the site are expected to reflect the creek level in these granular soils.



FOUNDATION INVESTIGATION AND DESIGN REPORT

5.0 CLOSURE

This report was prepared by Mr. Matthew Kelly, P.Eng., and reviewed by Mr. Nicolas LeBlanc, P.Eng. The report was 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.

Matthew Kelly, P. Eng.
Geotechnical Engineer

Nicolas LeBlanc, P.Eng.
Geotechnical Engineer



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



MWK/NRL/FJH/bg/lc

n:\active\2011\1121 - geotechnical\11-1121-0130 genivar hwy 132 renfrew cty\foundation report\2 - final to mto\11-1121-0130-1000 rpt-001 sept 2012.docx



PART B

Foundation Design
Proposed Bridge Replacement
Constant Creek Bridge
Highway 132 Realignment
From 0.2 km East of Highway 41
To 1.6 km East of the Constant Creek Bridge
G.W.P. 4034-05-00



6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides recommendations on the foundation aspects of design for the proposed replacement of the Constant Creek Bridge on Highway 132 as part of the rehabilitation and realignment of Highway 132 from 0.2 km east of highway 41 to 1.6 km east of the Constant Creek Bridge in Dacre, Ontario. The available preliminary drawings at the time of the preparation of this report indicate that the existing single span bridge structure at Constant Creek will be replaced with a new single span structure that will be located approximately 50 m to the north of the existing bridge. This new bridge location will provide a straighter alignment of Hwy 132 in this area, which currently curves into and out of the existing bridge location in an 'S turn' configuration. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site.

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 Foundation Options

Based on the draft Preliminary General Arrangement plans provided by Genivar, it was understood that the realigned Constant Creek bridge structure will consist of a single span CPCI girder, steel plate girder or precast box girder structure with integral abutments and a span of about 30 m. We understand that consideration is being given to increasing the span to 40 m to found outside the creek area and to minimize dewatering. The underside of the pile cap will be at about Elevation 184.5 m on the east side and 183.9 m on the west side. The natural ground surface in the immediate vicinity of the structure site varies from about Elevation 185 m to 187 m. From information recently provided by Genivar to Golder, the proposed Constant Creek bridge has been designed with a deck surface elevation of about 190 m to 191.5 m, resulting in approach embankments up to about 6 m in height. The preliminary General Arrangement drawings identify elevations for the creek bed and water level at about 183.5 m and 185.0 m to 185.9 m, respectively. These water levels are consistent with the water levels encountered during our drilling program in February 2012.

The surficial very loose to loose organics and upper sand deposit encountered at the proposed east abutment and the very loose to loose sandy and silty deposits encountered throughout the depth of the overburden at the proposed west abutment are considered liquefiable, and are therefore not suitable for support of shallow foundations. However, the very loose to loose organics and upper sand deposit encountered at the proposed east abutment extends only to about elevation 184.7 m, and will be completely removed as part of the abutment foundation construction, currently proposed at about elevation 184.5 m. Therefore, soil liquefaction at the east abutment is not of concern with the currently proposed bridge design.

At the west abutment, it is recommended that the foundations be extended below these liquefiable deposits by driving or pre-drilling steel H-piles to bedrock. Steel H-piles are suitable for support of the abutments in a conventional fixed fashion as well as in an integral abutment configuration and it is understood that integral



abutments are the preferred system for the design of this structure. A caisson foundation system is not considered practical at this site due to the presence of the water-bearing granular deposits which would necessitate the use of temporary or permanent liners, the relatively deep overburden deposits at the west abutment and the preference for an integral abutment configuration. The presence of some cobbles and occasional boulders within the lower granular deposits could cause pile advancement issues during construction as discussed in Section 6.3 of this report.

Recommendations for design of steel H-pile foundations are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs and risks/consequences associated with each of the feasible foundation options is presented in Table 1 following the text of this report. From a foundations perspective, based on this comparison, it is considered at this stage of design that steel H-piles are the most practical and the technically preferred foundation solution for this site.

6.3 Steel H-Pile Foundations

Steel H-piles driven through the surficial organic soils and granular deposits to found on the marble bedrock may be used for support of the bridge west abutment.

The sandy and silty soil deposits immediately overlying the bedrock at the west abutment contain some cobbles and boulders. The piles should therefore be provided with Tirus-type bearing points or equivalent to protect the pile tips during driving through the cobbles and boulders in the overburden and seating on the marble bedrock. The piles should be designed to be founded on bedrock. In the event that the piles attain refusal within the bouldery overburden, a lower axial resistance could be considered. Based on past experience with similar soil conditions, only a small number of piles, if any, are expected to reach refusal within the overburden. Although it is an option, pre-drilling the piles at the west abutment is not recommended as it would require the use of casing, which would be more expensive. For a relocated west abutment, the piles would be slightly longer to reach the bedrock.

At the east abutment some of the piles could have difficulty penetrating to depth and could “hang up” in the overburden deposits due to the presence of cobbles and boulders. Pre-drilling of the overburden and bedrock would be required to obtain a 5 m minimum length in either of the optional east abutment locations. Also, 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 hard marble 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.

A Non-Standard Special Provision (NSSP) is provided in Appendix C to alert the contractor to the presence of cobbles and boulders.

6.3.1 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:



FOUNDATION INVESTIGATION AND DESIGN REPORT

Pile Size	Factored ULS Resistance (kN)
HP 310 x 110	2,000
HP 310 x 132	2,400
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 marble 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/Water Level Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
West Abutment	12-2	185.5	11.0	174.5
	12-3	185.6	9.6	176.0
East Abutment	12-4	185.6	5.3	180.3
	12-5	185.6	5.8	179.8

As discussed previously, it is expected that some of the piles at the west abutment may not fully penetrate the cobbly overburden deposits to reach the bedrock surface; these piles could “hang up” at shallower depth in these materials. For these piles, predrilling of the overburden could be considered to allow the piles to reach the bedrock. 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. As a preliminary guideline, for HP 310 x 110 piles founded within the sandy soils near the bedrock surface, a ULS factored geotechnical resistance of 1,800 kN may be used. The axial resistance at SLS for 25 millimetres of settlement would be 1,600 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 for the factored axial resistance. For this situation, the piles should be driven in accordance with Standard SS 103-11, using an ultimate capacity of 3,600 kN per pile.

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.



For the west abutment, 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, these downdrag loads could be up to 250 kN for a HP 360 x 152 steel driven pile, and 195 kN for a HP 310 x 110 steel driven pile.

6.3.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. Alternatively, 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)
West Abutment	PCL ¹ – 177.0	Very loose to loose granular deposits	1.3
	177.0 – 174.5	Compact granular deposits	4.4
East Abutment	PCL ¹ – 183.5	Compact granular deposits	4.4
	183.5 – 179.8	Dense Granular Deposits	11

Note 1: PCL = Pile cap level.

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



FOUNDATION INVESTIGATION AND DESIGN REPORT

Pile Spacing in Direction of Loading (d = Pile Diameter)	Reduction Factor
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., sandy and silty deposits at this site) 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 20 kN/m³;
 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/m³; 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 *preliminary* 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 150 kN at ULS (unfactored), and a maximum lateral resistance of 50 kN at SLS (for 10 mm of horizontal deflection at pile cap level) is recommended for HP 310 x 110 piles in a treated sand and silt deposit.

If the liquefaction susceptible sand and silt layer is not treated or the layer is only partially treated as discussed in Section 6.4, then the soils within and above the liquefiable zone should not be taken into account for lateral resistance. In addition, lateral spreading in the direction of sloping ground will induce additional lateral loads on the deep foundation system, as the liquefiable soils "flow" around the piles during the design earthquake event. These lateral loads involve complex soil-structure-soil interactions, which can only be computed once the detailed design of the foundation system has been established.



6.3.3 Frost Protection

The pile caps should be provided with a minimum of 1.9 m of soil cover for frost protection purposes.

6.4 Seismic Site Response Classification

The site falls within the Western Quebec Seismic Zone (WQSZ) according to the geological survey of Canada. 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, the potential for seismic liquefaction of the overburden therefore needs to be assessed. A seismic Site Coefficient also needs to be assigned, as given in Section 6.4.2., to be used by the structural designer.

6.4.1 Liquefaction Assessment

Seismic liquefaction occurs when earthquake vibrations cause an increase in pore water pressures within the soil. The presence of excess pore water pressure reduces the effective stress between the soil particles, and the soil's frictional resistance to shearing. This phenomenon, which leads to a temporary reduction in the shear strength of the soil, may cause:

- Large lateral movements of even gently sloping ground, referred to as “lateral spreading”;
- Reduced shear resistance (i.e., bearing capacity) of soils which support foundations, as well as reduced resistance to sliding; and,
- Reduced shaft resistance for deep foundations as well as reduced resistance to lateral loading.

In addition, ‘seismic settlements’ may occur once the vibrations and shear stresses have ceased. Seismic settlement is the process whereby the soils stabilize into a denser arrangement after an earthquake, causing potentially large surface settlement.

The following conditions are more prone to experiencing seismic liquefaction:

- Coarse grained soils (i.e., more probable for sands than for silts);
- Soils having a loose state of packing; and,
- Soils located below the groundwater level.

The very loose to loose silt and sand deposit encountered on the west side of Constant Creek is potentially liquefiable. The very loose sand on the east side is of limited depth (1m) and is underlain by compact to dense sand and gravel. An assessment of the liquefaction potential of the very loose silt deposit was carried out using



the simplified procedure based on SPT N_{60} -values from the boreholes. The SPT N-values reported on the borehole records were corrected for overburden stress, rod length during sampling, and hammer energy efficiencies. According to the CHBDC, the site-specific zonal acceleration ratio for Constant Creek is 0.2 for liquefaction analyses.

The results of this assessment suggest that the saturated silt and sand deposit on the west side of Constant Creek would be classified as liquefiable under an earthquake with a magnitude of 5.8. However, the portion of the deposit that will be located below the proposed approach embankment with a height greater than 2 metres will have enough confining pressure to render the loose silts and sands non liquefiable. Therefore, the silt and sand deposit located below the bottom portion of the side slopes will likely liquefy during the design earthquake resulting in slope movements at the toe of the slope that would require repairs following the design earthquake. These small localized failures should not impact the overall stability of the west approach embankment.

Based on the proposed integral abutments for the new bridge which incorporates a slender abutment design (i.e., about 1.7 metres wide), the confining stresses from the approach embankment fills behind and in front of the new west abutment are considered sufficient to render the sands below and surrounding the piles non-liquefiable. Consideration should be given to relocating the wildlife passage on the west side to provide additional confining stresses on this side. Alternatively, additional fill could be placed in front of the west abutment. As a minimum, a 2 m wide (at the top) by 2 m high berm should be placed in front of the west abutment. Therefore, lateral spreading should not occur around the piles during the design earthquake, and no additional lateral loads from soil liquefaction needs to be considered.

The associated ground settlement induced by seismic shaking and potential soil liquefaction identified was further assessed by using the methodologies presented in Tokimatsu and Seed (1984 and 1987)². The estimated total seismic settlement of the ground surface is expected to be in the order of 180 to 230 millimetres post soil liquefaction near the toe of the embankment. The induced differential settlement post liquefaction will be about 50 percent of the total estimated settlement (i.e., 90 to 115 millimetres). Accuracy of the estimated seismic settlements is typically considered to be within 25 to 50% of the actual value. The settlements induced by strong seismic shaking should be considered in addition to static settlements of the embankment anticipated under the dead and live loads.

6.4.2 Site Coefficient

For seismic design purposes, the Site Coefficient, S , for this site in accordance with Section 4.4.6 of the CHBDC may be taken as 1.5, consistent with Soil Profile Type III.

6.5 Seismic Hazard Mitigation

6.5.1 General

The significant presence of liquefiable soils at the west abutment at this site, the extent of potential seismic settlements, and potential for significant lateral spreading, indicate that conventional shallow foundations on the existing soils will not perform well during the design earthquake. Foundations for the proposed structures must

² Tokimatsu, K., and Seed, H. (1987), Evaluation of Settlements in Sands due to Earthquake Shaking, ASCE, Journal of Geotechnical Engineering.



transmit any loads below the liquefiable zones into underlying competent bearing strata to avoid these movements with the use of deep foundations.

As previously mentioned, the slender design of the proposed integral abutments allows the surrounding embankment fill to adequately confine the sand and silt deposit around the piles. Therefore, treatment of the loose sand and silt deposit is not required for satisfactory performance of the foundation system for an integral abutment.

However, a wider abutment design could allow the sands and silts to liquefy around the piles during the design earthquake resulting in large lateral spreading movements, and additional lateral loads on the piles. For the west abutment, a ground improvement program (Schaefer et. al, 1997) could be considered to mitigate the risk of lateral spreading caused by soil liquefaction. Four ground improvement options have been identified for the proposed west approach embankment:

- Vibro-compaction;
- Dynamic compaction;
- Rapid impact compaction (RIC); and,
- Rammed Aggregate Piers or Geopiers.

In addition, differential settlements will be acute at the limits for embankments on “improved ground”. At these locations special measures should be undertaken (e.g., flexible joints or piping) to mitigate the effects of these settlements on buried services, where present.

The ground improvement options described below are intended to reduce the total seismic settlements at, and near the toe of the embankment side slopes to less than 25 millimetres. Consideration could also be given to partial ground improvement (e.g., upper sand/silt only) if higher differential seismic settlements (e.g., 50 to 75 millimetres) can be tolerated for the approach embankment side slopes.

6.5.1.1 *Vibro-compaction*

Vibro-compaction is a method of increasing the in situ density of granular soil by inserting a vibratory probe. The probe is inserted into the ground on a typical spacing in the order of 1.5 to 4 metres. This technique is more suited for clean sands where the fines content is less than 12 to 15 percent and/or where the clay content is less than 3 percent. Depths of treatment with vibro-compaction typically range from 3 to 15 metres. Vibro-compaction has been used on over 2,000 projects in North America. The work is usually completed by specialty geotechnical subcontractors who prepare the detailed design of the probe layout to achieve a required density. This method may not be suitable for this site due to the higher fines content in some portions of the sand and silt deposit.

6.5.1.2 *Dynamic Compaction*

Dynamic compaction is a method of increasing the in situ density of granular soil by dropping a heavy tamper (weight) onto the ground surface. The tamper typically weighs between 5 and 32 tonnes and is dropped from heights of 12 to 25 metres. Depths of treatment with dynamic compaction typically range from 3 to 10 metres. The work is usually completed by specialty geotechnical subcontractors who prepare the detailed design of the treatment layout to achieve a required density. The effectiveness of the method is measured based on the



specified “post-treatment” SPT blow count. With this densification method, a 1.5-metre thick granular cushion must be placed for equipment access. This method may not treat the entire thickness of the loose sand and silt based on the higher fines content of the deposit at depth. However, this method could be considered if a partial densification of the loose deposit is considered.

6.5.1.3 *Rapid Impact Compactor*

The Rapid Impact Compactor (RIC) was originally developed in the early 1990's in conjunction with the British Military as a means of quickly repairing damaged aircraft runways. Dynamic energy is imparted by dropping a 7.5 ton weight from a controlled height onto a patented foot. Energy is transferred to the ground safely and efficiently, since the RIC's foot remains in contact with the ground. Compaction parameters are automatically controlled and monitored from the RIC's cab with an on-board data acquisition system.

The RIC soil improvement would be carried out within the entire footprint of the embankment plus at least 3 metres laterally beyond the footprints. The influence depth of RIC would be typically about 4 metres to 5 metres below the ground surface based on the known site conditions. A performance specification and verification testing program (i.e., SPT or CPT) should be developed for the use of RIC at this site. This method will not treat the entire thickness of the loose deposit based on the higher fines content of the deposit at depth. However, this method could be considered if a partial densification of the loose sand and silt deposit is considered. Similar to the Dynamic Compaction method, a 1 to 1.5 metre thick granular cushion must also be placed.

6.5.1.4 *Rammed Aggregate Piers or Geopiers*

The risk of differential settlements could also be mitigated with the use of properly installed Geopiers or rammed aggregate piers. This technology involves augering holes in a grid pattern below the proposed embankment footprint, followed by ramming of engineered aggregates in the open borehole to create piers of aggregates. With the use of a geogrid reinforced granular mat below the proposed approach embankment, the load would be transferred directly to the Geopiers or stone columns system. The installation methods of Geopiers/stone columns could also densifies the surrounding soils to a limited extent. Due to the loose sand and silts, and the high water level, the installation of the rammed aggregate piers or Geopiers would have to be carried out with the use of a casing. This method is considered feasible, and would be the preferred treatment alternative.

6.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems 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.



FOUNDATION INVESTIGATION AND DESIGN REPORT

- 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 1.9 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 the 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	0.35	0.22
At rest, K_o	0.50	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	0.27	0.27
At rest, K_o	0.43	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 0.2. 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 $A = 0.2$.



- 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., $k_h = 0.3$). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e., $k_h = 0.1$).

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 0.2. This corresponds to displacements of up to approximately 50 mm at this site.
- 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).

6.7 Approach Embankment Design and Construction

It is understood that the proposed approach embankments will be constructed using either rock or earth fill. The use of earth fill for embankment construction generally requires flatter embankment side slopes for stability purposes, but allows the slopes to be vegetated. The use of rock fill can achieve steeper side slopes, but rock fill slopes can generally not sustain the growth of vegetation, unless specialty products are used.



The approach embankments are shown on the preliminary design drawings having side slope profiles of 1.75H:1V at the west approach and 1.5H:1V at the east approach. These slope profiles can only be achieved using rock fill.

Based on the borehole results, the approach embankment subgrade soils will consist of very loose to loose organic or alluvial soils underlain by very loose to dense granular deposits consisting of sand, silt and gravel, underlain by marble bedrock. Cobbles and boulders were also encountered within the sandy deposits.

6.7.1 Subgrade Preparation and Embankment Construction

Based on the borehole results, the approach embankment subgrade soils will consist of very loose to loose organic or alluvial soils underlain by very loose to dense granular deposits consisting of sand, silt and gravel, underlain by marble bedrock.

Any surficial topsoil, organic matter and softened / loosened soils should be stripped from within the limits of the approach embankment filling. All subgrade soils should be proof-rolled prior to fill placement.

Placement of rock or earth fill should be carried out in accordance with the requirements as outlined in the Special Provision 206S03. The rock fill should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Blading, dozing and 'chinking' the rock to form a dense, compact mass will be required to minimise voids and bridging. Side slopes for rock fill embankments should be no steeper than 1.25 horizontal to 1 vertical (1.25H:1V). Earth fill embankments should be no steeper than 2 horizontal to 1 vertical (2H:1V).

In accordance with OPSD 202.010, and as required by MTO, the incorporation of 2 m wide berms (or successive benches) into the uniform side slope profile (i.e. slope flattening) is required wherever rock fill embankments exceed a height of 10 m such that the uninterrupted rock fill slope never exceeds a height of 10 m. Given the maximum approach embankment height is less than 10 m at this site, 2 m wide berms will not be required. For earth fill embankments, 2 m wide berms will be required for embankment heights of 8 m or greater.

Vegetation cover should be established on any earth slopes to protect embankment fill against surficial erosion.

6.7.2 Approach Embankment Stability

Limit equilibrium slope stability analyses were performed using the commercially available program Slide (produced by Rocscience Inc.) and Slope/W (produced by GEO-SLOPE International Ltd.), both employing the Morgenstern-Price method of analyses to check that a minimum factor of safety of 1.3 is achieved against deep-seated, global type failures that would impact the operation of the roadway.

A maximum embankment height of 6.0 m for the approach fill embankments was analyzed for slope global stability. A rock fill embankment was analyzed with a side slope at 1.75H:1V at the west approach and 1.5H:1V at the east approach with no benching. An earth fill embankment was also analyzed with a side slope at 2H:1V at the west approach and 2H:1V at the east approach with no benching. A Factor of Safety of 1.3 or greater was calculated against static deep-seated slope instability for both embankment types (i.e., rock fill or earth fill).

Pseudo-static seismic slope stability analyses for both of the above configurations also indicate that the embankment side slopes will have factors of safety of greater than 1.1 against deep-seated slope instability based on an acceleration of 0.136g. The results do however indicate that some shallow sloughing (with factors of safety less than 1.1) could occur of the embankment side slopes during seismic loading. That sloughing would



FOUNDATION INVESTIGATION AND DESIGN REPORT

not however impair the short term use of the structure and is mainly a maintenance/repair issue. The potential for sloughing of any earth slopes could be reduced by providing well vegetated side slopes.

In addition to the static slope stability analysis and pseudo-static seismic slope stability analysis, the embankment slopes were also analysed for the special conditions immediately after the design earthquake when the sand and silt deposit has liquefied, resulting in a lower residual strength for this soil. The residual strength of the loose sand and silt deposit was determined using the methodologies presented in Seed and Harder (1990)³ for the portion of the deposit located outside of the approach embankment with a height of more than 2 metres. The post-seismic slope stability analyses under residual strength soil conditions indicate that the embankment side slopes will have factors of safety greater than 1.1 against deep-seated slope instability.

The slope stability analyses for the above embankment were carried out using the following parameters:

Material	Bulk Unit Weight (kN/m ³)	Effective Friction Angle (°)	Effective Cohesion (kPa)
Embankment Rock Fill	19	40	-
Embankment Earth Fill	19	32	-
Very Loose to Loose Granular Deposits	18	28	-
Very Loose to Loose Granular Deposits, Post Liquefaction Residual Strength	18	0	10
Compact to Dense Granular Deposits	20	32	-
Bedrock	Impenetrable		

The above slope stability analyses were carried out on the assumption that the subgrade would be adequately prepared, and proper placement of the embankment fill (rock and earth fill).

6.7.3 Embankment Settlement

Settlement analyses were carried out for the new fill approach embankments using the results from the boreholes, in situ field test data (SPT), and laboratory tests to estimate the deformation parameters of the subsoils. A settlement analysis was performed using the commercially available program SETTLE 3D (Version 2.011) produced by Rocscience Inc. For these analyses, the critical sections are assumed to correspond to the greatest new embankment height and/or the subsurface stratigraphy most susceptible to settlement for the approach embankments. The subsurface profiles for the embankment fills described in the previous slope stability section were employed in the analyses. The total net loading on the foundation soils (after stripping, backfilling and embankment fill construction) was estimated to be about 120kPa. The following is a summary of engineering properties used for this settlement analysis.

³ Seed, R.B. and Harder, L.F., Jr. (1990). SPT-based analysis of cyclic pore pressure generation and undrained residual strength. Proceedings of the H.B. Seed Memorial Symposium, Bi-Tech Publishing Ltd., 2: 351-376.



FOUNDATION INVESTIGATION AND DESIGN REPORT

Soil Layer/Deposit	Bulk Unit Weight (kN/m ³)	Poisson Ratio	Modulus of Elasticity, E _s (MPa)
New Embankment Fill	19	-	-
Very Loose to Loose Sandy Deposits	18	0.3	5
Compact to Dense Sandy Deposits	20	0.3	20

As noted previously, the subsoils encountered within the limits of the project site generally consist of sand, silt and gravel deposits, underlain by marble bedrock. Based on the results of the boreholes, settlement of the cohesionless foundation soils is expected to occur during or shortly after construction. Settlement of the new embankments will also occur due to compression of the rockfill itself.

Settlement of the rock fill depends on the type of rock and on the method and sequence of placement and compaction of the fill. Assuming that the rock fill is not end dumped in its final position and is placed in accordance with the requirements as outlined in the Special Provision 206S03, the settlement of the newly placed rock fill, for the up to 6 m high approach embankments, will be about 1 percent of the new effective height of rock fill (i.e. up to about 60 mm at this site). It is anticipated that about 60 percent of this settlement will occur within the first year following construction.

It is predicted that immediate settlement due to compression of the existing cohesionless foundation soil layers will be about 200 mm. The majority of settlements are expected to occur rapidly (i.e. during and shortly after construction) in response to the filling based on the granular nature of the native soils.

6.8 Design and Construction Considerations

6.8.1 Excavations

The excavations for the construction of abutments will extend through the surficial alluvium and organic soils and the very loose to loose near surface granular deposits. 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. When below the water table, these 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.

6.8.2 Groundwater and Surface Water Control

The bridge span length should be chosen to avoid or minimize the amount of dewatering required in Constant Creek.

The groundwater level is considered to be in the range of Elevation 185 m (at about existing ground surface) and highly dependant on the level of Constant Creek. There is a risk that foundation excavations for the pile cap construction will intersect water-bearing alluvial soils or fill materials associated with the former creek meander channel, contributing to higher groundwater inflows into the excavation. If appropriate, groundwater inflows could be minimized with the use of an interlocking steel sheetpile system at the west abutment area. At the east abutment area, the sheeting could get some limited penetration, i.e. based on borehole 12-4 and 12-5 to about 2.5 m depth.



6.8.3 Obstructions

The soils at this site contain cobbles and boulders, which could affect the installation of deep foundations or protection systems. An NSSP should be included in the Contract Documents developed during the detail design stage to identify to the contractor the possible presence of cobbles and/or boulders within the overburden soils.



FOUNDATION INVESTIGATION AND DESIGN REPORT

7.0 CLOSURE

This report was prepared by Mr. Matthew Kelly, P.Eng and reviewed by Mr. Nicolas LeBlanc, P.Eng. 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



MWK/NRL/FJH/bg/lc

n:\active\2011\1121 - geotechnical\11-1121-0130 genivar hwy 132 renfrew cty\foundation report\2 - final to mto\11-1121-0130-1000 rpt-001 sept 2012.docx



FOUNDATION INVESTIGATION AND DESIGN REPORT

Table 1: Comparison of Foundation Alternatives
Constant Creek Bridge Replacement
G.W.P. 4034-05-00

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-piles Founded on Bedrock	<ul style="list-style-type: none">■ Feasible	<ul style="list-style-type: none">■ High bearing resistance■ Negligible settlement■ Integral abutments possible	<ul style="list-style-type: none">■ Possibility of encountering cobbles or boulders during pile driving, and need to pre-auger pile locations or use reduced pile capacity	<ul style="list-style-type: none">■ Least expensive option	<ul style="list-style-type: none">■ Generally low risk option
Caissons Founded on Bedrock	<ul style="list-style-type: none">■ Feasible, not practical	<ul style="list-style-type: none">■ High bearing resistance	<ul style="list-style-type: none">■ High water table will require the use of temporary or permanent liners■ Presence of cobbles and boulders may cause difficulty in drilling large caissons	<ul style="list-style-type: none">■ Most expensive option	<ul style="list-style-type: none">■ Generally high risk option



FOUNDATION INVESTIGATION AND DESIGN REPORT

Table 2: Comparison of Ground Improvement Options
Constant Creek Bridge Replacement
G.W.P. 4034-05-00

Ground Improvement Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Do Nothing	<ul style="list-style-type: none"> Feasible with the use of a narrow integral abutment, and a 2 m wide by 2 m high berm Not feasible with an abutment larger than about 1.7 metres wide 	<ul style="list-style-type: none"> No additional cost No complex construction required No specialized contractor required 	<ul style="list-style-type: none"> Following design earthquake, sloughing at the toe in areas where the embankment height is less than about 2 metres in height With wider embankment, liquefaction around the piles may induce lateral spreading during design earthquake 	<ul style="list-style-type: none"> No additional cost 	<ul style="list-style-type: none"> Generally low risk option
Rammed Aggregate Piers or Geopiers	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Increases stability of embankment slopes Also densifies the sands around the piers during installation 	<ul style="list-style-type: none"> High water table and loose to very loose sandy soil will require the use of a casing to allow their construction Specialized contractor required 	<ul style="list-style-type: none"> Expensive option 	<ul style="list-style-type: none"> Generally moderate risk option



FOUNDATION INVESTIGATION AND DESIGN REPORT

Table 2: Comparison of Ground Improvement Options
Constant Creek Bridge Replacement
G.W.P. 4034-05-00
(continued)

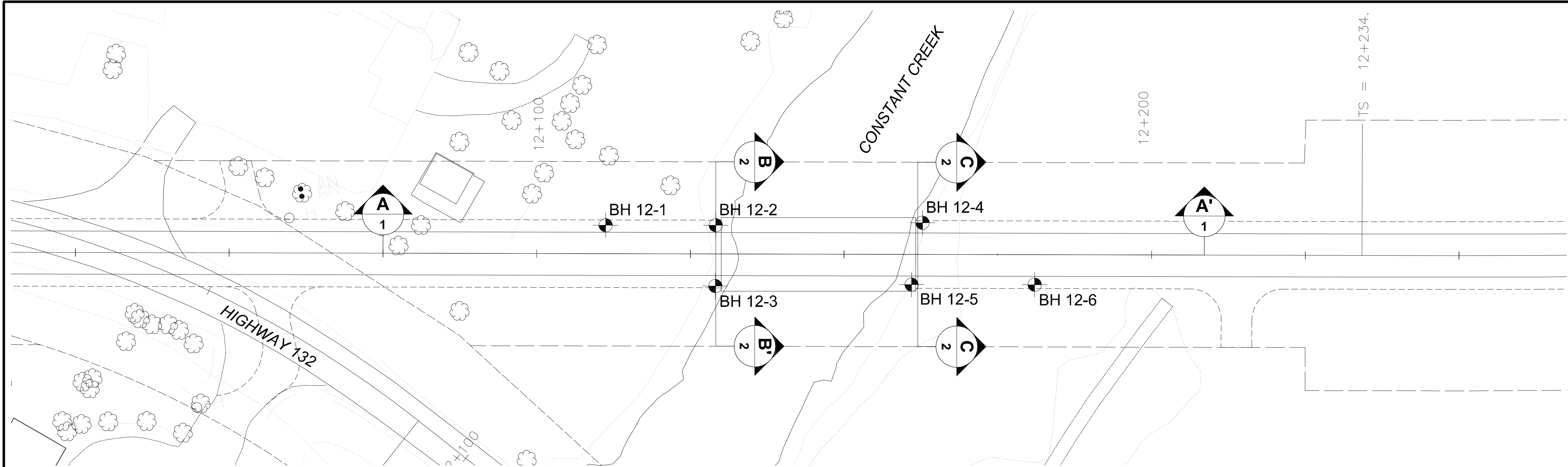
Ground Improvement Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Dynamic Compaction	<ul style="list-style-type: none"> Feasible, not practical 	<ul style="list-style-type: none"> Increases stability of embankment slopes Greater treatment depth than RIC Densifies upper sands to make them non-liquefiable 	<ul style="list-style-type: none"> High water table will require the use of a 1.5 m granular working pad High fines content will limit the depth of treatment. Lower portion of loose deposit may not get treated Specialized contractor required 	<ul style="list-style-type: none"> Moderately expensive option, more expensive than RIC 	<ul style="list-style-type: none"> Generally moderate risk option
Rapid Impact Compactor (RIC)	<ul style="list-style-type: none"> Feasible, not practical 	<ul style="list-style-type: none"> Marginally Increases stability of embankment slopes Densifies upper sands to make them non-liquefiable 	<ul style="list-style-type: none"> High water table will require the use of a 1m granular working pad High fines content will limit the depth of treatment to less than about 4 to 5 metres. Lower portion of loose deposit will not get treated Specialized contractor required 	<ul style="list-style-type: none"> Moderately expensive option, less expensive than Dynamic Compaction 	<ul style="list-style-type: none"> Generally moderate risk option



FOUNDATION INVESTIGATION AND DESIGN REPORT

Table 2: Comparison of Ground Improvement Options
Constant Creek Bridge Replacement
G.W.P. 4034-05-00
(continued)

Ground Improvement Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Vibro-Compaction	<ul style="list-style-type: none">■ Possibility of not being feasible due to the high fines content	<ul style="list-style-type: none">■ Increases stability of embankment slopes■ Densifies sands to make them non-liquefiable.	<ul style="list-style-type: none">■ High water table will require the use of a 1m granular working pad■ May not adequately densify the loose soils due to the high fines content■ Requires further investigation with CPTu to determine its feasibility■ Specialized contractor required	<ul style="list-style-type: none">■ Most Expensive option	<ul style="list-style-type: none">■ Generally high risk option

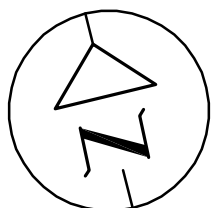


PLAN

SCALE



CONT No.
WP No. 4034-05-00

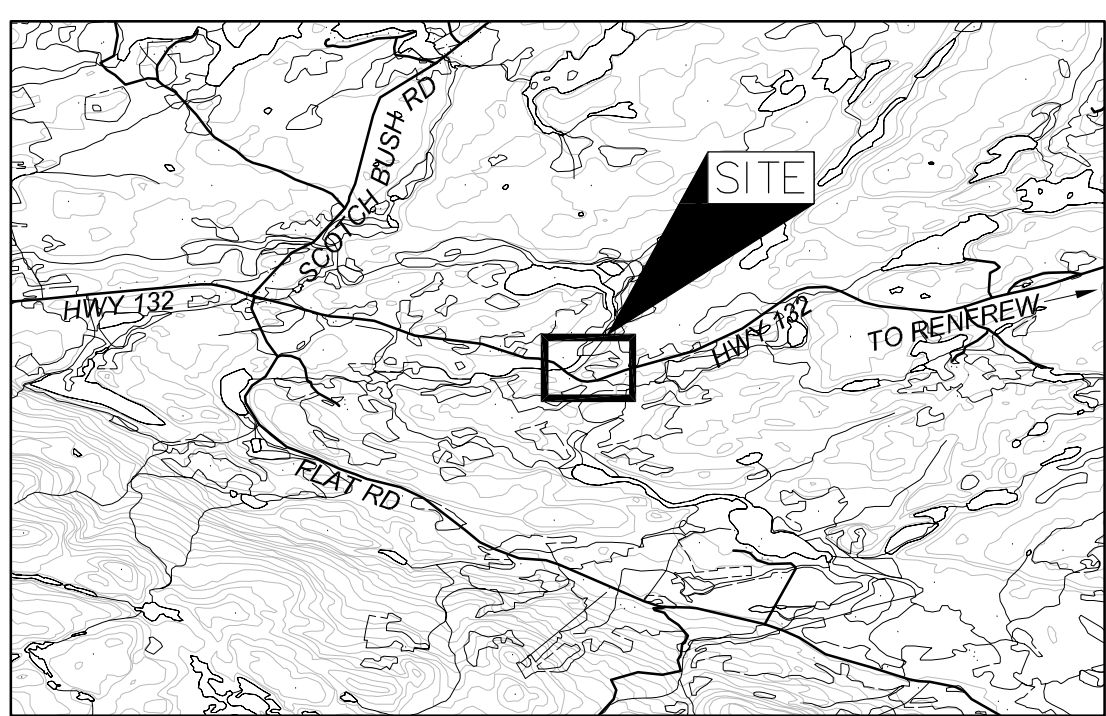


HIGHWAY 132
CONSTANT CREEK BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN

SCALE



LEGEND

- Borehole - Current Investigation
- N Standard Penetration Test Value
- Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation
- Seal
- Piezometer
- WL in piezometer
- WL in open borehole

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
12-1	185.7	5025293.9	270333.5
12-2	185.5	5025291.1	270345.4
12-3	185.6	5025283.6	270340.3
12-4	185.6	5025287.5	270360.3
12-5	185.6	5025279.3	270357.4
12-6	186.4	5025277.3	270370.4

METRIC

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

REFERENCE

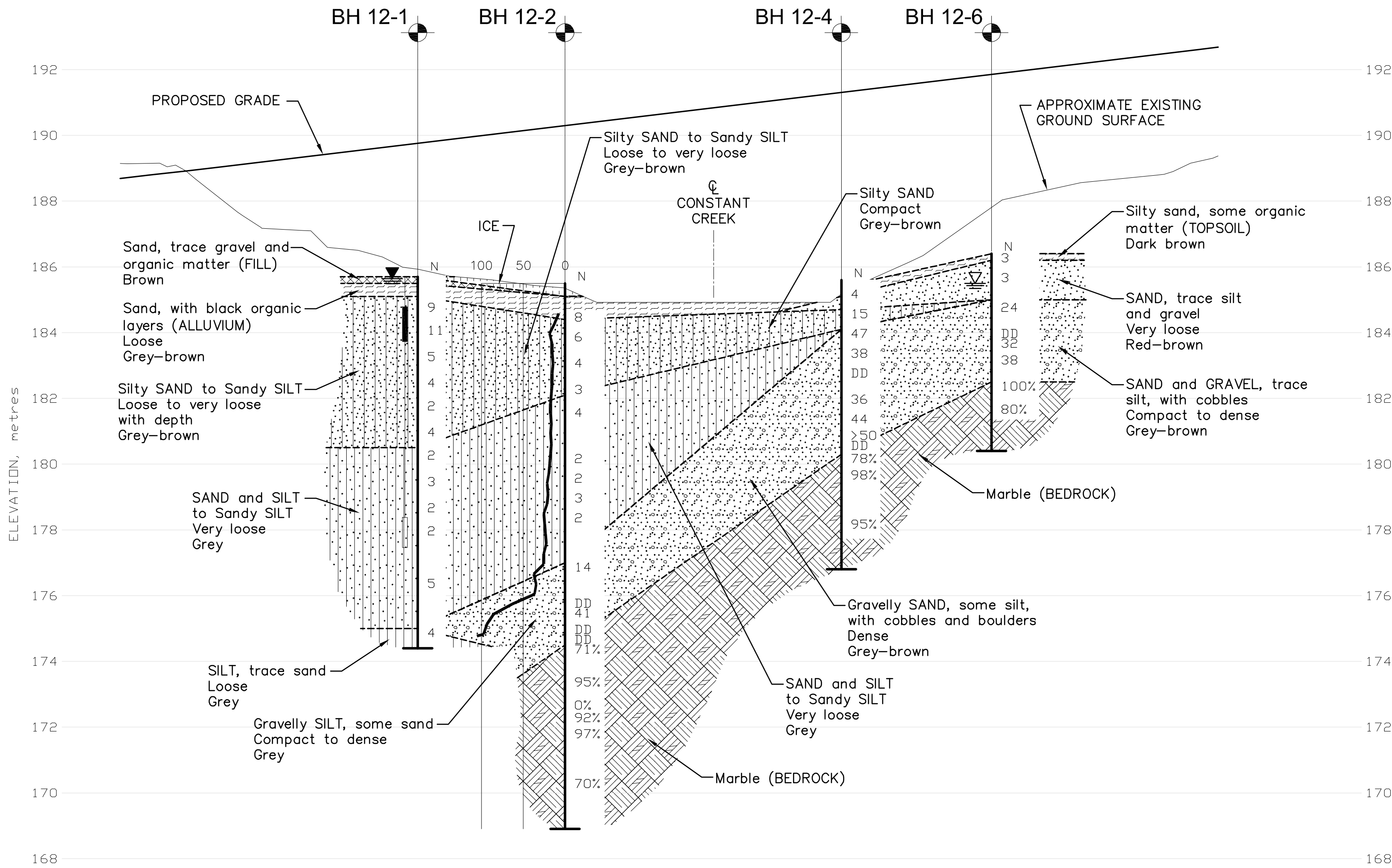
Base plans provided in digital format by Genivar Inc.
Datum: NAD 83, Coordinate System: MTM Zone 9

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.

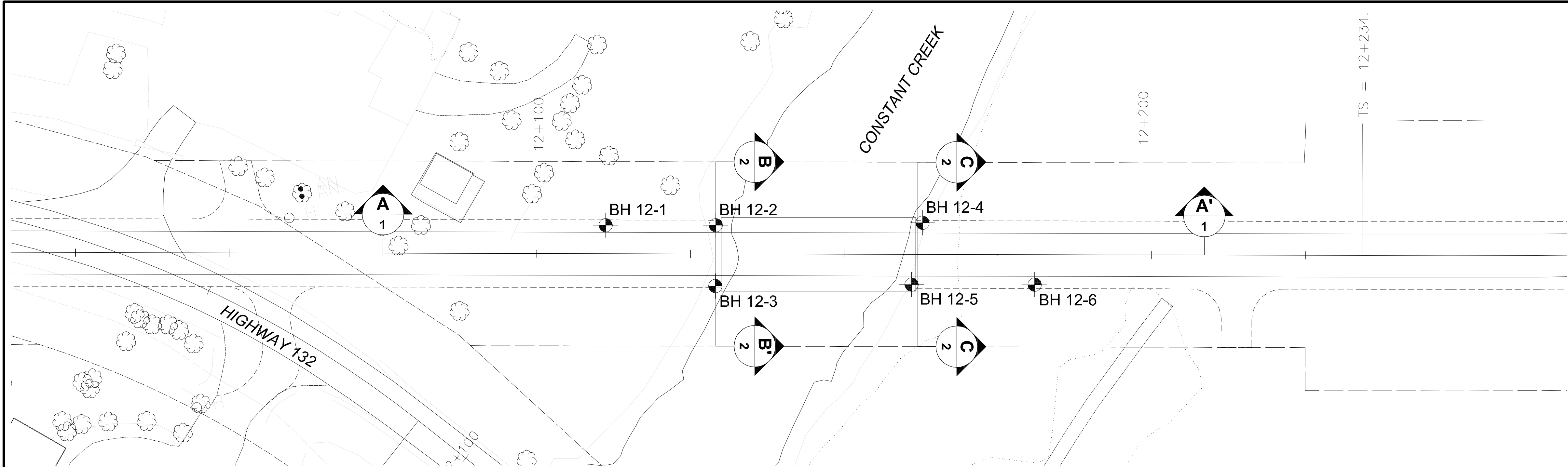


CROSS-SECTION A-A'

HORIZONTAL SCALE



VERTICAL SCALE



PLAN

SCALE

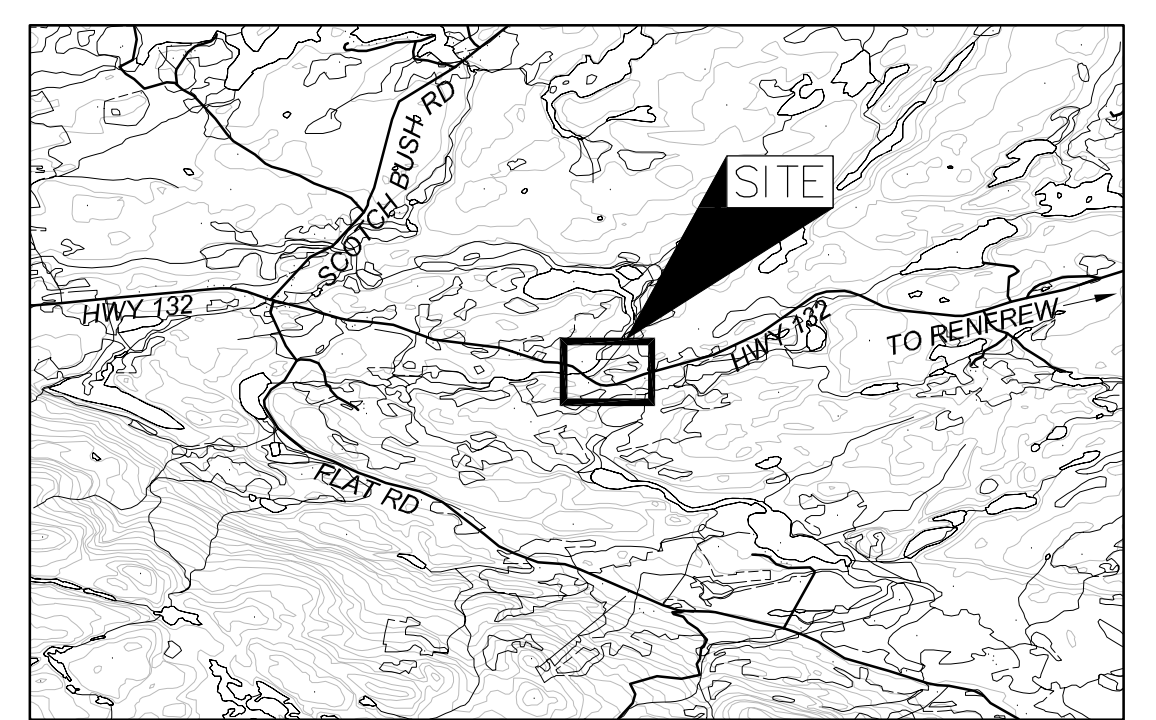


CONT No.
WP No. 4034-05-00

HIGHWAY 132
CONSTANT CREEK BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

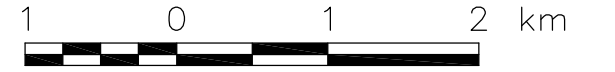


Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN

SCALE



LEGEND

- Borehole - Current Investigation
- N Standard Penetration Test Value
- Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation
- Seal
- Piezometer
- WL in piezometer
- WL in open borehole

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
12-1	185.7	5025293.9	270333.5
12-2	185.5	5025291.1	270345.4
12-3	185.6	5025283.6	270340.3
12-4	185.6	5025287.5	270360.3
12-5	185.6	5025279.3	270357.4
12-6	186.4	5025277.3	270370.4

METRIC

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

REFERENCE

Base plans provided in digital format by Genivar Inc.
Datum: NAD 83, Coordinate System: MTM Zone 9

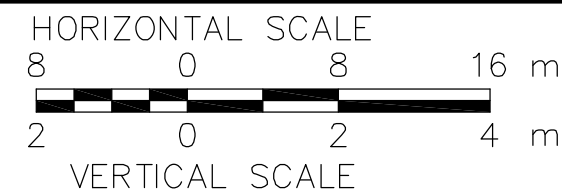
NOTES

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'



CROSS-SECTION C-C'



NO.	DATE	BY	REVISION
Geocres No. 31F-181			
HWY. 132		PROJECT NO. 11-1121-0130	DIST.
SUBM'D. NRL	CHKD. NRL	DATE: MARCH 2012	SITE:
DRAWN: JM	CHKD. NRL	APPD. FJH	DWG. 2



APPENDIX A

Record of Boreholes and Record of Drillholes

PROJECT <u>11-1121-0130</u>		RECORD OF BOREHOLE No 12-1		1 OF 2 METRIC	
G.W.P. <u>4034-05-00</u>		LOCATION <u>N 5025293.9 ; E 270333.5</u>		ORIGINATED BY <u>P.A.H.</u>	
DIST <u> </u> HWY <u>132</u>		BOREHOLE TYPE <u>200 mm Diam. Power Auger (Hollow Stem)</u>		COMPILED BY <u>J.M.</u>	
DATUM <u>Geodetic</u>		DATE <u>February 9, 2012</u>		CHECKED BY <u>N.R.L.</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20	40	60	80	100	w _p	w	w _L		
185.7	GROUND SURFACE																
0.0	Sand, trace gravel and organic matter (FILL)																
185.5	Brown																
0.2	Sand, with black organic layers (ALLUVIUM)		1	GRAB												ORG = 1.9%	
185.1	Loose																
0.6	Grey and brown																
	Silty SAND to Sandy SILT		2	SS	9											0 65 34 1	
	Loose to very loose with depth																
	Grey-brown																
	Saturated																
			3	SS	11											0 50 49 1	
			4	SS	5											0 69 30 1	
			5	SS	4											0 66 33 1	
			6	SS	2											0 75 24 1	
			7	SS	4											0 36 62 2	
180.5																	
5.2	SAND and SILT to Sandy SILT		8	SS	2											0 54 45 1	
	Very loose																
	Grey																
	Saturated																
			9	SS	3											0 43 56 1	
			10	SS	2											0 30 69 1	
			11	SS	2											0 40 59 1	
			12	SS	5											0 40 59 1	

Continued Next Page

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

MIS-MTO-001 1111210130.GPJ GAL-MISS.GDT 09/25/12 JM

PROJECT <u>11-1121-0130</u>		RECORD OF BOREHOLE No 12-1		2 OF 2 METRIC	
G.W.P. <u>4034-05-00</u>		LOCATION <u>N 5025293.9 ; E 270333.5</u>		ORIGINATED BY <u>P.A.H.</u>	
DIST <u> </u> HWY <u>132</u>		BOREHOLE TYPE <u>200 mm Diam. Power Auger (Hollow Stem)</u>		COMPILED BY <u>J.M.</u>	
DATUM <u>Geodetic</u>		DATE <u>February 9, 2012</u>		CHECKED BY <u>N.R.L.</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20	40	60	80	100	25	50	75		
--- CONTINUED FROM PREVIOUS PAGE ---							175										
175.0	SAND and SILT to Sandy SILT Very loose Grey Saturated																
10.7	SILT, trace sand Loose Grey Saturated		13	SS	4												0 5 92 3
174.4																	
11.3	End of Borehole Note: 1. Water level in standpipe frozen at 0.06 m below ground surface on Feb. 21, 2012																

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

PROJECT <u>11-1121-0130</u>		RECORD OF BOREHOLE No 12-2		1 OF 2 METRIC	
G.W.P. <u>4034-05-00</u>		LOCATION <u>N 5025291.1 ; E 270345.4</u>		ORIGINATED BY <u>P.A.H.</u>	
DIST <u> </u> HWY <u>132</u>		BOREHOLE TYPE <u>200 mm Diam. Power Auger (Hollow Stem), Wash Boring, NW Casing</u>		COMPILED BY <u>J.M.</u>	
DATUM <u>Geodetic</u>		DATE <u>February 10-13, 2012</u>		CHECKED BY <u>N.R.L.</u>	





SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					w _p	w	w _L						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED													
185.5	GROUND SURFACE							20	40	60	80	100									
0.0	ICE																				
185.1																					
0.4	Sand, with organic matter, occasional peat layer (ALLUVIUM) Dark grey Saturated		1	GRAB																	
184.4																					
1.1	Silty SAND to Sandy SILT Loose to very loose Grey-brown Saturated		2	SS	8									○				1	62	36	1
			3	SS	6									○				0	62	37	1
			4	SS	4									○				2	72	25	1
182.1			5	SS	3									○				0	36	62	2
3.4	SAND and SILT to Sandy SILT Very loose Grey Saturated																				
			6	SS	4									○				0	38	61	1
			7	SS	2									○				0	48	51	1
			8	SS	2									○				0	51	48	1
			9	SS	3									○				0	44	54	2
			10	SS	2									○				0	39	60	1
177.0																					
8.5	Gravelly SILT, some sand Compact to dense Grey Saturated		11	SS	14									○				37	13	48	2
			12	NQ RC	DD																

Continued Next Page

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

MIS-MTO-001 1111210130.GPJ GAL-MISS.GDT 09/25/12 JM

PROJECT <u>11-1121-0130</u>		RECORD OF BOREHOLE No 12-2		2 OF 2 METRIC	
G.W.P. <u>4034-05-00</u>		LOCATION <u>N 5025291.1 ; E 270345.4</u>		ORIGINATED BY <u>P.A.H.</u>	
DIST <u> </u> HWY <u>132</u>		BOREHOLE TYPE <u>200 mm Diam. Power Auger (Hollow Stem), Wash Boring, NW Casing</u>		COMPILED BY <u>J.M.</u>	
DATUM <u>Geodetic</u>		DATE <u>February 10-13, 2012</u>		CHECKED BY <u>N.R.L.</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED + FIELD VANE								
								● QUICK TRIAXIAL × REMOULDED								
	--- CONTINUED FROM PREVIOUS PAGE ---					20	40	60	80	100	25	50	75			
174.5	Gravelly SILT, some sand Compact to dense Grey Saturated		13	SS	41											
			14	NQ RC	DD											
			15	NQ RC	DD											
11.0	Marble (BEDROCK) Fresh Grey		1	RC	REC 100%										RQD = 71%	
			2	RC	REC 56%										RQD = 95%	
172.8	Marble (BEDROCK) Highly weathered		2	RC	REC 0%										RQD = 0%	
12.7	Marble (BEDROCK) Fresh Grey Medium to thickly bedded Strong		2	RC	REC 100%										RQD = 92%	
172.5	Bedrock cored between 11.0 m and 16.6 m depth. For bedrock coring details refer to Record of Drillhole 12-2.		3	RC	REC 100%										RQD = 97%	
13.0			4	RC	REC 97%										RQD = 70%	
168.9																
16.6	End of Borehole															

PROJECT: 11-1121-0130

RECORD OF DRILLHOLE: 12-2

SHEET 1 OF 1

LOCATION: N 5025291.1 ;E 270345.4

DRILLING DATE: February 10-13, 2012

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE F-FAULT												SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
									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													
									RECOVERY		R.Q.D. %		FRACT. INDEX PER 0.3		DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K _i cm/sec																	
TOTAL CORE %	SOLID CORE %					DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	10 ⁻³																							
11		Continued from Record of Borehole 12-2		174.50																														
		Marble (BEDROCK) Fresh Grey		11.00																				UC = 22.6 MPa										
12					1		0																											
					2		0																											
		Marble (BEDROCK) Highly weathered		172.80																														
13				12.70	2		0																											
		Marble (BEDROCK) Fresh Grey Medium to thickly bedded Strong		172.50																														
				13.00	2		0																											
14																																		
					3		0																											
15																																		
					4		0																											
16																																		
		End of Drillhole		168.90																														
				16.60																														
17																																		
18																																		
19																																		
20																																		
21																																		

DEPTH SCALE

1 : 50



LOGGED: P.A.H.

CHECKED: N.R.L.

MIS-RCK 001 1111210130 (ROCK).GPJ GAL-MISS.GDT 09/25/12 JM


PROJECT <u>11-1121-0130</u>		RECORD OF BOREHOLE No 12-3		1 OF 2 METRIC	
G.W.P. <u>4034-05-00</u>		LOCATION <u>N 5025283.6 ; E 270340.3</u>		ORIGINATED BY <u>P.A.H.</u>	
DIST <u> </u> HWY <u>132</u>		BOREHOLE TYPE <u>Wash Boring, NW Casing</u>		COMPILED BY <u>J.M.</u>	
DATUM <u>Geodetic</u>		DATE <u>February 13-14, 2012</u>		CHECKED BY <u>N.R.L.</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					w _p	w	w _L		
								<div><div></div><div></div><div></div><div></div><div></div></div> <div>20406080100</div>									
185.6	GROUND SURFACE																
0.0	ICE																
185.4																	
0.2	Silty SAND, with organic matter (ALLUVIUM) Saturated		1	SS	WH		185									○ DRG = 11.2%	
184.5																	
1.1	SAND, some silt to Silty SAND Loose Grey-brown Saturated		2	SS	6		184									2 83 14 1	
			3	SS	4											0 64 35 1	
183.3																	
2.3	SILT and SAND to SILT, some sand Very loose Grey Saturated		4	SS	4		183									1 33 65 1	
			5	SS	4		182									0 37 62 1	
			6	SS	3											0 24 75 1	
							181										
			7	SS	3											0 39 59 2	
			8	SS	4		180									0 40 59 1	
			9	SS	5		179									0 33 66 1	
			10	SS	5		178									0 13 85 2	
177.4																	
8.2	SAND and GRAVEL, trace to some silt, with cobbles Compact Grey Saturated		11	SS	19		177										
			11A	NQ RC	DD												
176.0							176										
9.6			1	RC	REC 94%											RQD = 94%	

Continued Next Page

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

MIS-MTO-001 1111210130.GPJ GAL-MISS.GDT 09/25/12 JM

PROJECT 11-1121-0130				RECORD OF BOREHOLE No 12-3				2 OF 2 METRIC											
G.W.P. 4034-05-00				LOCATION N 5025283.6 ; E 270340.3				ORIGINATED BY P.A.H.											
DIST _____ HWY 132				BOREHOLE TYPE Wash Boring, NW Casing				COMPILED BY J.M.											
DATUM Geodetic				DATE February 13-14, 2012				CHECKED BY N.R.L.											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)	
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100							
175.1	Marble (BEDROCK), with quartz banding Fresh Grey		1	RC	REC 94%		175										RQD = 94%		
10.5	Marble (BEDROCK) Highly weathered		12	SS	REC 0%														RQD = 0%
174.8	Marble (BEDROCK), with quartz banding Fresh, grey Medium to thickly bedded Medium strong to very strong Bedrock cored between 9.6 m and 15 m depth. For bedrock coring details refer to Record of Drillhole 12-3.		2	RC	REC 99%														RQD = 93%
10.8			3	RC	REC 100%														RQD = 97%
			4	RC	REC 100%														
170.6	End of Borehole						171												
15.0																			

PROJECT: 11-1121-0130

RECORD OF DRILLHOLE: 12-3

SHEET 1 OF 1

LOCATION: N 5025283.6 ;E 270340.3

DRILLING DATE: February 13-14, 2012

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE F-FAULT				SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec				DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
									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																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
		Continued from Record of Borehole 12-3		176.00																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					

DEPTH SCALE

1 : 50



LOGGED: P.A.H.

CHECKED: N.R.L.

MIS-RCK 001 1111210130 (ROCK).GPJ GAL-MISS.GDT 09/25/12 JM

PROJECT 11-1121-0130			RECORD OF BOREHOLE No 12-4			1 OF 1 METRIC											
G.W.P. 4034-05-00			LOCATION N 5025287.5 ; E 270360.3			ORIGINATED BY P.A.H.											
DIST HWY 132			BOREHOLE TYPE Wash Boring, NW Casing			COMPILED BY J.M.											
DATUM Geodetic			DATE February 16-17, 2012			CHECKED BY N.R.L.											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
185.6	GROUND SURFACE																
0.0	ICE																
185.4																	
0.2	Peaty ORGANIC MATTER Black																
185.1																	
0.5	SAND, trace silt Very loose Yellow-brown Saturated		1	SS	4												
184.7																	
0.9	Silty SAND Compact Grey-brown Saturated		2	SS	15												0 38 61 1
184.1																	
1.5	Gravelly SAND, some silt, with cobbles and boulders Dense Grey-brown Saturated		3	SS	47												
			4	SS	38												31 54 13 2
			5	NQ RC	DD												
			6	SS	36												
			7	SS	44												
			8	SS	>50												
			9	NQ RC	DD												
180.3																	
5.3	Marble (BEDROCK) Fresh Grey Thickly bedded Very Strong		1	RC	REC 100%												RQD = 78%
	Bedrock cored between 5.3 m and 8.8 m depth. For bedrock coring details refer to Record of Drillhole 12-4.		2	RC	REC 98%												RQD = 98%
			3	RC	REC 97%												RQD = 95%
176.8																	
8.8	End of Borehole																

PROJECT: 11-1121-0130

RECORD OF DRILLHOLE: 12-4

SHEET 1 OF 1

LOCATION: N 5025287.5 ;E 270360.3

DRILLING DATE: February 16-17, 2012

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE F-FAULT												SM-SMOOTH		FL-FLEXURED		BC-BROKEN CORE		DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
									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							
									RECOVERY				R.Q.D. %				FRACT. INDEX PER 0.3				DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K _f cm/sec			
TOTAL CORE %				SOLID CORE %								DIP w.r.t. CORE AXIS				TYPE AND SURFACE DESCRIPTION		10 ⁻⁶										
80 60 40 20				80 60 40 20				80 60 40 20				15 10 7.5 5				0 50 100 150		10 ⁻⁵										
6		Continued from Record of Borehole 12-4		180.30																								
		Marble (BEDROCK)		5.30	1	100																						
		Fresh																										
		Grey																										
		Thickly bedded																										
		Very strong																										
7					2	100																						
8					3	80																						
9		End of Drillhole		176.80																								
				8.80																								
10																												
11																												
12																												
13																												
14																												
15																												

DEPTH SCALE

1 : 50



LOGGED: P.A.H.

CHECKED: N.R.L.

MIS-RCK 001 1111210130 (ROCK).GPJ GAL-MISS.GDT 09/25/12 JM

MIS-MTO 001 111210130.GPJ GAL-MISS.GDT 09/25/12 JM

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

PROJECT: 11-1121-0130

RECORD OF DRILLHOLE: 12-5

SHEET 1 OF 1

LOCATION: N 5025279.3 ;E 270357.4

DRILLING DATE: February 15-16, 2012

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE F-FAULT												SM-SMOOTH			FL-FLEXURED			BC-BROKEN CORE			DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
									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										
									RECOVERY		R.Q.D. %		FRACT. INDEX PER 0.3		DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec														
TOTAL CORE %	SOLID CORE %					DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	10 ⁶	10 ⁵	10 ⁴	10 ³	2	4	6																	
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									
80	80	80	80	80	80	5																									
60	60	60	60	60	60	10																									
40	40	40	40	40	40	15																									
20	20	20	20	20	20	20																									

UC = 46.3 MPa

DEPTH SCALE

1 : 50



LOGGED: P.A.H.

CHECKED: N.R.L.

MIS-RCK 001 1111210130 (ROCK).GPJ GAL-MISS.GDT 09/25/12 JM

PROJECT 11-1121-0130			RECORD OF BOREHOLE No 12-6			1 OF 1 METRIC											
G.W.P. 4034-05-00			LOCATION N 5025277.3 ; E 270370.4			ORIGINATED BY P.A.H.											
DIST _____ HWY 132			BOREHOLE TYPE Wash Boring, NW Casing			COMPILED BY J.M.											
DATUM Geodetic			DATE February 17-20, 2012			CHECKED BY N.R.L.											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ kN/m³	GR SA SI CL
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	25 50 75				
186.4	GROUND SURFACE																
0.0 186.2 0.2	Silty sand, some organic matter (TOPSOIL) Dark brown Moist		1	SS	3		186										
	SAND, trace silt and gravel Very loose Red-brown Moist to wet		2	SS	3												9 82 8 1
185.0							185										
1.4	SAND and GRAVEL, trace silt, with cobbles Compact to dense Grey-brown Saturated		3	SS	24												
			4	NQ RC	DD		184										
			5	SS	32												
			6	SS	38		183										
182.5																	
3.9	Marble (BEDROCK) Fresh Grey		1	RC	REC 100%		182										RQD = 100%
181.8																	
4.6 181.5 4.9	Marble (BEDROCK) Slightly weathered Dark grey																
	Marble (BEDROCK) Fresh Dark grey and grey		2	RC	REC 99%		181										RQD = 80%
	Bedrock cored between 3.9 m and 6.0 m depth. For bedrock coring details refer to Record of Drillhole 12-6.																
180.4																	
6.0	End of Borehole																
	Note: 1. Water level in open hole at 0.9 m depth below ground surface on Feb. 20, 2012.																

MIS-MTO-001 1111210130.GPJ GAL-MISS.GDT 09/25/12 JM

PROJECT: 11-1121-0130

RECORD OF DRILLHOLE: 12-6

SHEET 1 OF 1

LOCATION: N 5025277.3 ;E 270370.4

DRILLING DATE: February 17-20, 2012

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE	F-FAULT	SM-SMOOTH	FL-FLEXURED	BC-BROKEN CORE	NOTES WATER LEVELS INSTRUMENTATION
									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		
RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec								
TOTAL CORE %	SOLID CORE %			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION									
80	80	80	80	5	0	10 ⁻⁶	2	4						
60	60	60	60	10	30	10 ⁻⁵								
40	40	40	40	15	60	10 ⁻⁴								
20	20	20	20	20	90	10 ⁻³								
Continued from Record of Borehole 12-6														
4		Marble (BEDROCK)		182.50										
		Fresh		3.90										
		Grey			1		100							
				181.80										
		Marble (BEDROCK)		4.60										
		Slightly weathered												
		Dark grey		181.50										
		Marble (BEDROCK)		4.90										
		Fresh												
		Dark grey and grey			2		100							
6		End of Drillhole		180.40										
		Note:		6.00										
		1. Water level in open hole at												
		0.9 m depth below ground surface												
		on Feb. 20, 2012.												
7														
8														
9														
10														
11														
12														
13														

DEPTH SCALE

1 : 50



LOGGED: P.A.H.

CHECKED: N.R.L.

MIS-RCK 001 1111210130 (ROCK).GPJ GAL-MISS.GDT 09/25/12 JM

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

MIS-MTO 001 111210130.GPJ GAL-MISS.GDT 09/25/12 JM

PROJECT <u>11-1121-0130</u>		RECORD OF BOREHOLE No DCP-1				2 OF 2 METRIC							
G.W.P. <u>4034-05-00</u>		LOCATION <u>N 0.0 ; E 6.0</u>				ORIGINATED BY <u>P.A.H.</u>							
DIST <u> </u> HWY <u>132</u>		BOREHOLE TYPE <u>Dynamic Cone</u>				COMPILED BY <u>J.M.</u>							
DATUM <u>Geodetic</u>		DATE <u>February 13, 2012</u>				CHECKED BY <u>N.R.L.</u>							
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa		W _p	W		
	--- CONTINUED FROM PREVIOUS PAGE ---							<div style="display: flex; justify-content: space-between; border-bottom: 1px solid black; margin-bottom: 5px;"> 20406080100 </div> <div style="display: flex; justify-content: space-between; border-bottom: 1px solid black; margin-bottom: 5px;"> 20406080100 </div> <div style="display: flex; justify-content: space-between; font-size: 0.8em;"> ○ UNCONFINED+FIELD VANE ● QUICK TRIAXIALxREMOULDED </div>					
89.2													
10.8	End of Borehole Refusal												

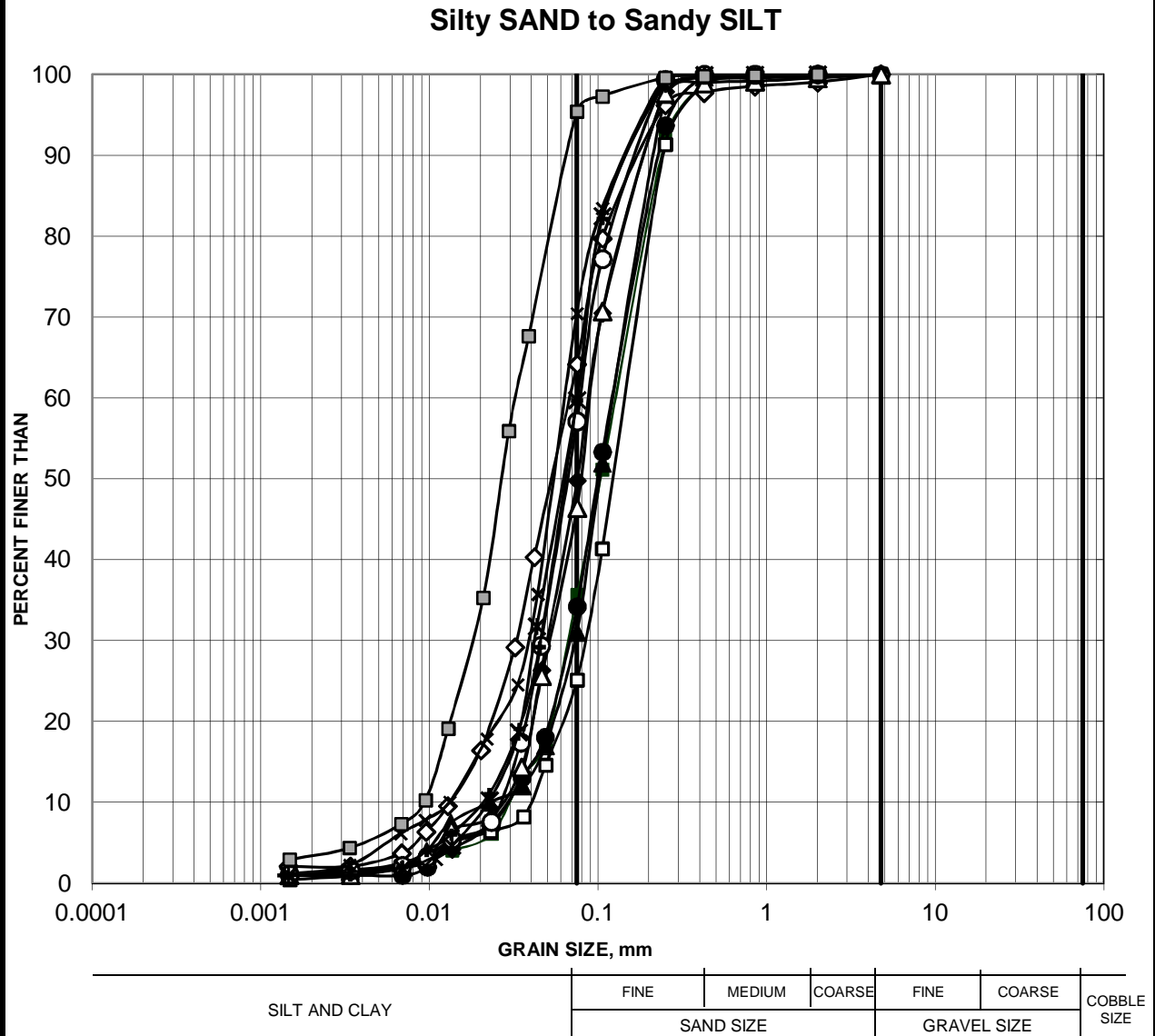


APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

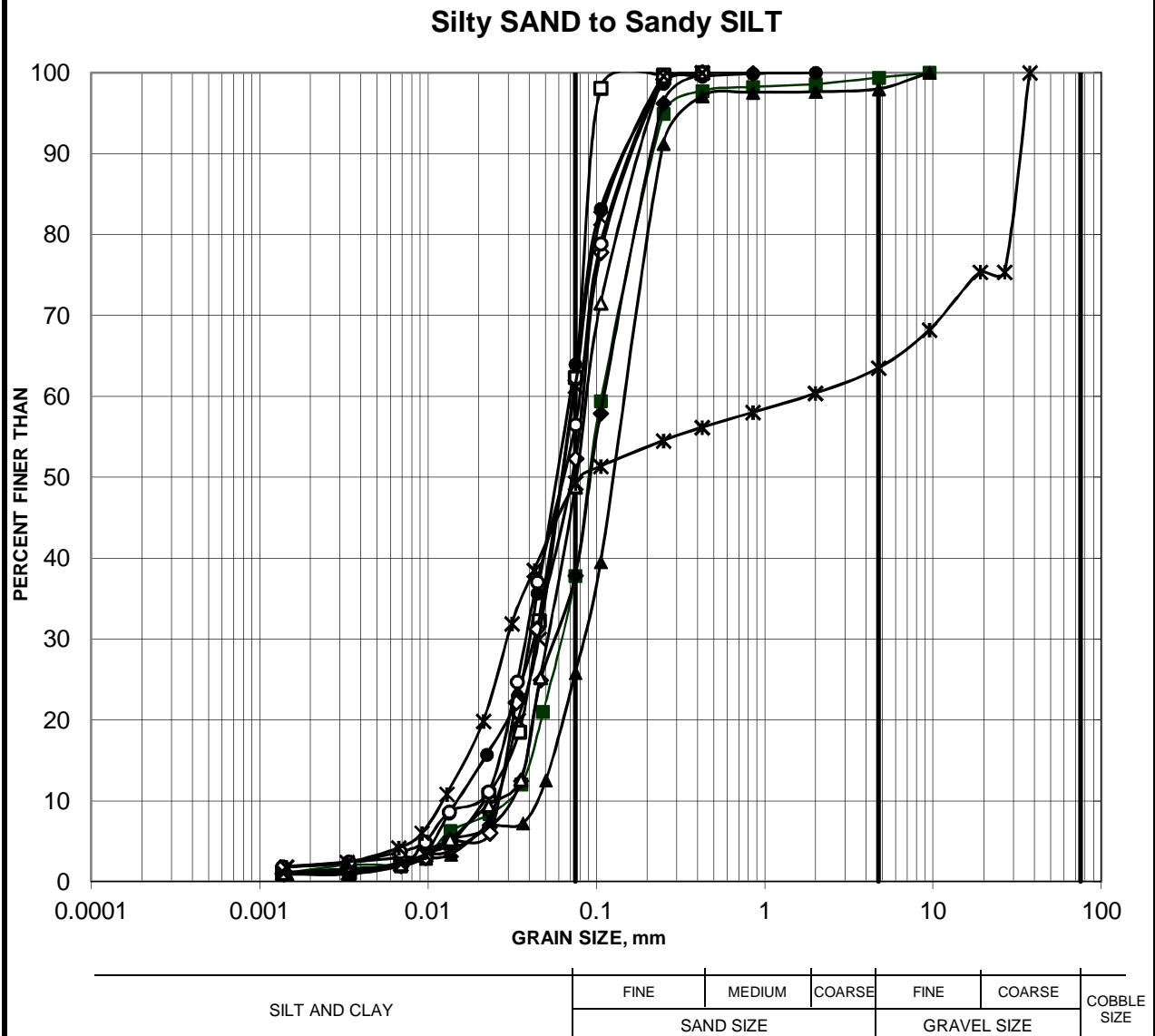
FIGURE B1



Borehole	Sample	Depth (m)
12-1	2	0.76-1.22
12-1	3	1.52-2.13
12-1	4	2.29-2.90
12-1	5	3.05-3.66
12-1	6	3.81-4.42
12-1	7	4.57-5.18
12-1	8	5.34-5.94
12-1	9	6.10-6.71
12-1	10	6.86-7.42
12-1	11	7.62-8.23
12-1	12	9.15-9.76
12-1	13	10.67-11.28

GRAIN SIZE DISTRIBUTION

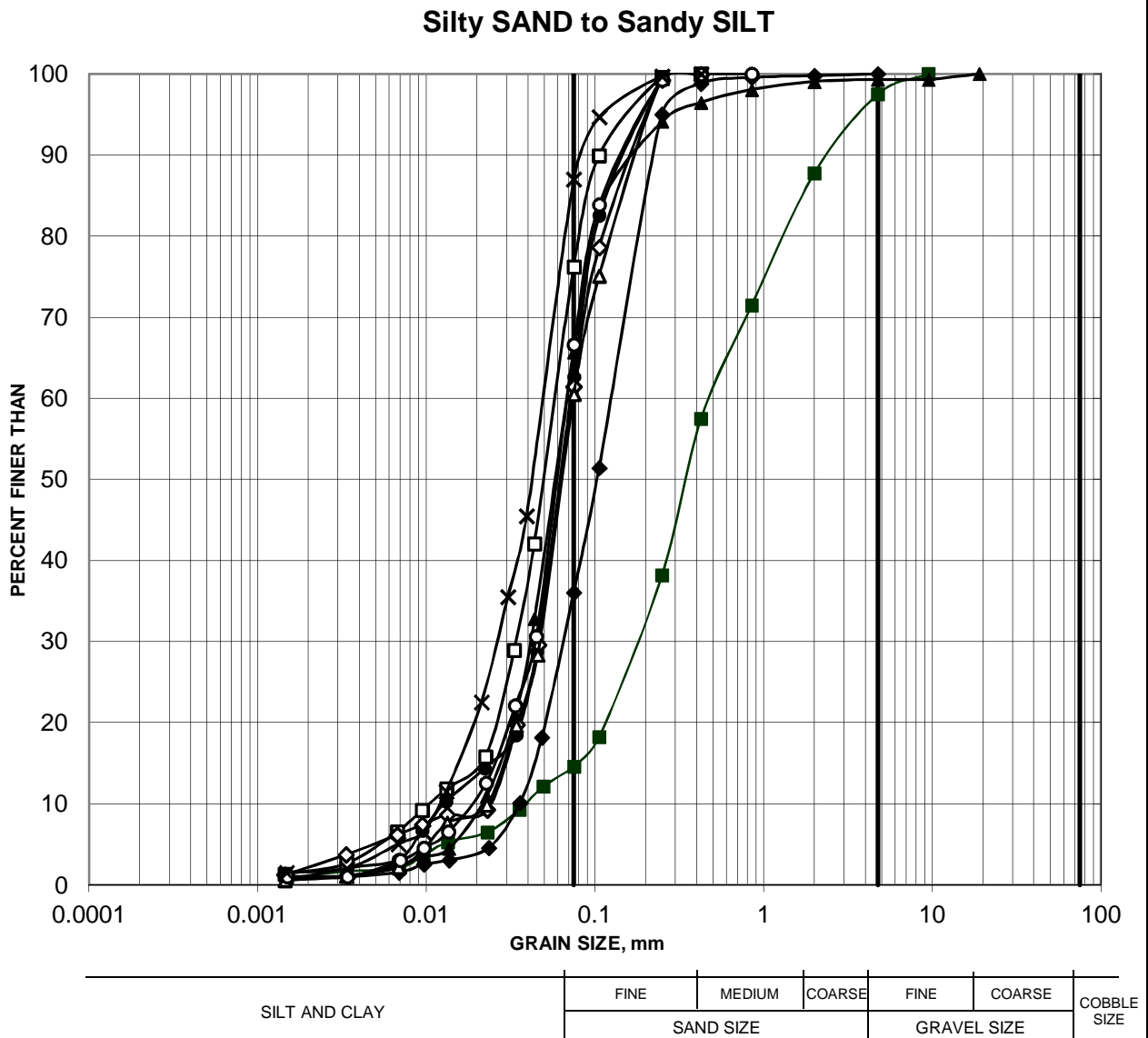
FIGURE B2



Borehole	Sample	Depth (m)
12-2	2B	1.07-1.52
12-2	3	1.52-2.13
12-2	4	2.29-2.90
12-2	5	3.05-3.66
12-2	6	3.81-4.42
12-2	7	5.18-5.79
12-2	8	5.79-6.40
12-2	9	6.40-7.01
12-2	10	7.01-7.62
12-2	11	8.54-9.15

GRAIN SIZE DISTRIBUTION

FIGURE B3

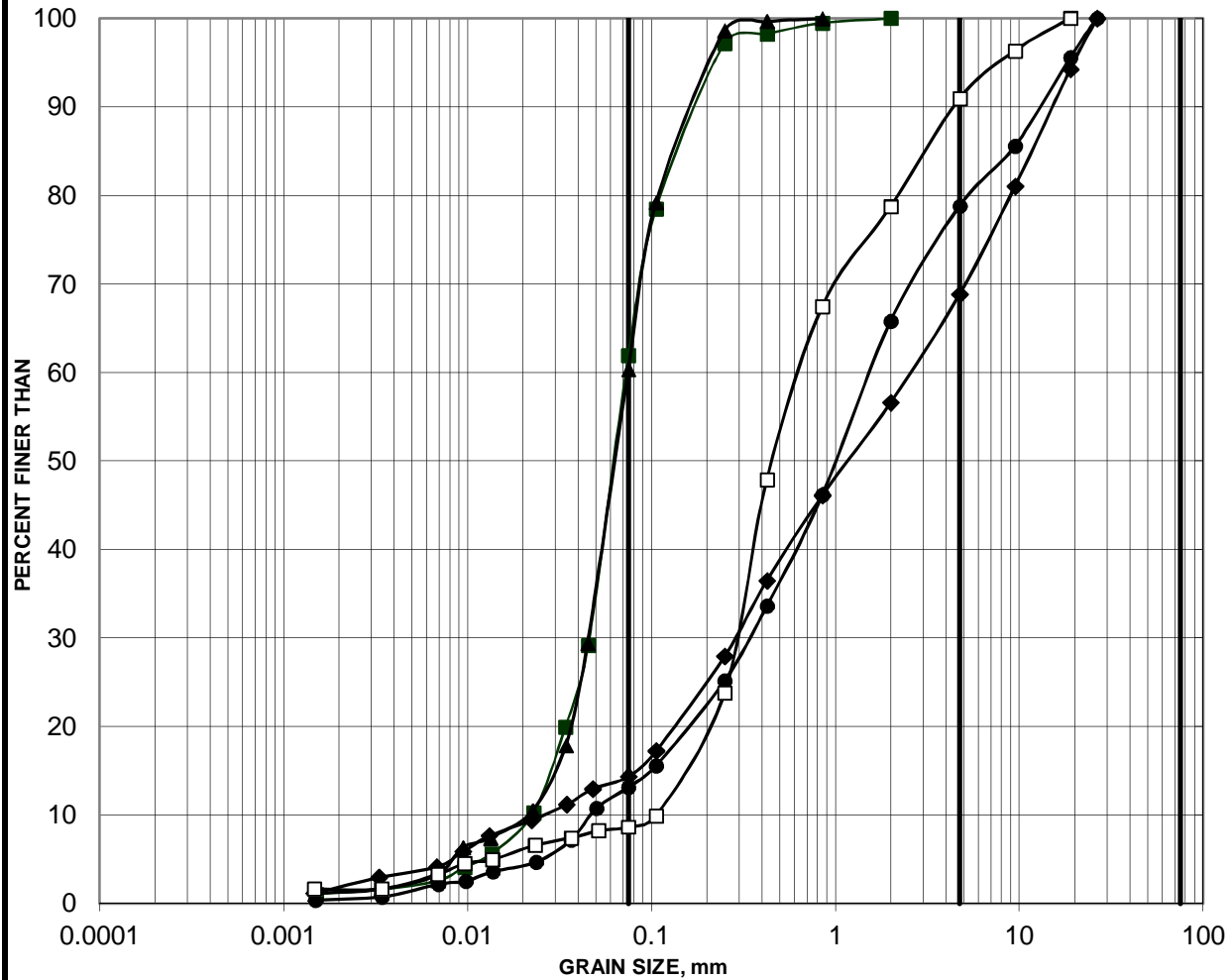


Borehole	Sample	Depth (m)
12-3	2B	1.07-1.52
12-3	3	1.52-2.13
12-3	4	2.29-2.90
12-3	5	3.05-3.66
12-3	6	3.81-4.42
12-3	7	4.57-5.18
12-3	8	5.34-5.95
12-3	9	6.10-6.71
12-3	10	6.86-7.47

GRAIN SIZE DISTRIBUTION

FIGURE B4

Gravelly SAND to SAND and GRAVEL



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

Borehole	Sample	Depth (m)
12-4	2	0.91-1.52
12-4	4	2.13-2.74
12-5	3	1.52-2.13
12-5	6	3.35-3.96
12-6	2	0.61-1.22



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



FOUNDATION INVESTIGATION AND DESIGN REPORT

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

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

