REPORT

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April 2014

REPORT ON

Preliminary Foundation Investigation and Design Eel's Creek Bridge Replacement Site No. 26-118 40 m South of Haultain Road Highway 28, Peterborough County, Ontario W.P. 4126-10-01

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W.P. 4126-10-01

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by MMM Group Ltd. (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the Design-Build of seven culvert replacements and two bridge replacements at various locations in the Eastern Region of Ontario as part of the 22 Structures MEGA 2 project. This report presents the results of the preliminary foundation investigation conducted for the replacement of the Eel's Creek bridge, Site No. 26-118 (WP 4126-10-01), located on Highway 28 about 40 m south of Haultain Road in Peterborough County, Ontario.

The purpose of the foundation investigation was to assess the subsurface conditions for the proposed bridge replacement by drilling 5 boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated April 2012. The work was carried out in accordance with Golder's Quality Control Plan dated August 2012.

2.0 SITE DESCRIPTION

The Eel's Creek Bridge is located on Highway 28, about 40 m south of Haultain Road in Peterborough County, Ontario. The existing bridge (Site No. 26-118) is located at about Station 21+230.

The existing bridge consists of a ten span timber deck supported by timber abutments and pier bents. Each pier bent consists of a wood pier cap supported on 15 timber piles. The existing structure is aligned approximately north-south, about 47.5 m in length and 11.6 m in width. It is understood that the structure was built in 1952 and is presently in poor condition. There is also an active snowmobile trail under the bridge on the north side of the structure.

The natural ground surface within the lowland floodplain of Eel's Creek at the toe of the existing bridge embankments is at about Elevation 249.7 m. It is understood that the water level in the creek, as measured by MMM in October 2012, was at about Elevation 248.3 m.

Highway 28 in this area is a two-lane undivided highway with a rural cross-section. In the area of the bridge, Highway 28 has been constructed on embankments that are between about 4.5 and 7.5 m in height, with the pavement surface between about Elevations 255.2 at the south end and 254.1 m at the north end. The highway embankment side slopes are oriented between about 1.5 horizontal to 1 vertical and 2 horizontal to 1 vertical (i.e., 1.5H:1V and 2H:1V). Based on visual observation at the time of the site investigation, the existing embankment slopes appear to be performing satisfactorily.

3.0 INVESTIGATION PROCEDURES

The subsurface investigation for the proposed bridge replacement was carried out between May 14 to 16 and September 20 to 26, 2013, and at which time five boreholes (numbered 13-211 to 13-215, inclusive) were advanced at the locations shown on Drawing 1. The boreholes were advanced as follows:

- Boreholes 13-211, 13-213, 13-214 and 13-215 were advanced with 108 mm inside diameter continuous-flight hollow-stem augers and/or wash boring using NW casing with a track-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to depths between about 13.1 and 20.7 m, below the existing pavement/ground surface in the overburden. The boreholes were then cored between about 3.0 and 3.5 m into the bedrock using NQ-size coring equipment.
- Borehole 13-212 was advanced using portable drilling equipment supplied and operated by OGS Inc. of Almonte, Ontario. The borehole was advanced to 17.0 m at which time a dynamic cone penetration test was driven to refusal at a depth of about 17.4 m below the existing ground surface in the overburden.

Soil samples in the boreholes were obtained at vertical intervals of about 0.60 to 1.52 m, using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test procedures.

A standpipe piezometer was installed in Borehole 13-211 to monitor the groundwater level at the site. The standpipe consists of a 32 mm diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within silica sand backfill and sealed by a section of bentonite pellet backfill.

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

The field work was supervised by a member of Golder's technical staff, who located the boreholes, 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, organic content 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 in relation to existing site features. The ground surface elevations were also surveyed by Golder to an established benchmark provided by MMM consisting of round iron bar on the west side of the Highway 28, just north of Haultain Road at Station 21+296.7, labelled BM 253. The elevation of the benchmark is understood to be Elevation 253.25 m. The boreholes and locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to Geodetic datum, are summarized in the following table and are shown on Drawing 1.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
13-211	Proposed north abutment (at the toe of the northeast side of the existing approach embankment)	4942710.4	413290.0	249.7
13-212	Proposed south abutment (at the toe of the southeast side of the existing approach embankment)	4942692.0	413309.3	249.7
13-213	Within the northbound lane of Highway 28 through the existing south approach embankment	4942672.3	413315.9	255.4
13-214	Within the northbound lane of Highway 28 through the existing north approach embankment	4942723.7	413273.9	253.7
13-215	Proposed north abutment along the northern bank of Eel's Creek	4942734.6	413288.6	249.4

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The site is located in the physiographic region known as the Georgian Bay fringe, just north of the Dummer moraines, as delineated in The Physiography of Southern Ontario.[1]

The Georgian Bay fringe is characterized by shallow deposits of glacial till and bare rock knobs and ridges. The underlying bedrock in the area is typically Precambrian.[1]

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole and Drillhole sheets contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B6 contained in Appendix B.

A soil stratigraphy section projected along the centreline of the proposed bridge alignment is shown on Drawing 1. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the location of the proposed bridge replacement consist of the embankment fill, sand, organic sand and silt overlying granite bedrock.

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

4.2.1 Pavement Structure and Embankment Fill

The pavement structure within the highway was penetrated within the northbound lane at Boreholes 13-213 and 13-214. At the borehole locations, the pavement structure consists of about 0.2 m of asphaltic concrete overlying about 0.2 m of crushed stone base. The granular base is underlain by about 4.2 to 7.0 m of subbase/embankment fill. The subbase/embankment fill generally consists of sand and gravel to sand, with trace to some silt. Cobbles and boulders were also encountered within the embankment fill.

The embankment fill was fully penetrated to depths of about 4.6 and 7.4 m (Elevations 249.1 and 248.0 m) at Boreholes 13-214 and 13-213, respectively.

Standard Penetration Test (SPT) N values measured for the embankment fill range from 5 to 53 blows per 0.3 m of penetration indicating a loose to very dense state of packing. Refusal to advancement of the sampler was frequently encountered, on cobbles and boulders in the deposit and in some instances rotary diamond drilling techniques were required to advance the boreholes within the embankment fill.

The results of grain size distribution testing carried out on four samples of the embankment fill are provided on Figure B1 in Appendix B. The results do not reflect the cobble or boulder content of the material, because the samples were retrieved using a 50 mm outside diameter sampler. The measured water contents of the samples varied from approximately 3 to 18 percent.

Approximately 0.8 m of silty sand fill was encountered at ground surface at Borehole 13-211.

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[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.

4.2.2 Topsoil and Upper Sandy Layers

About 0.2 m of topsoil was encountered at ground surface at Borehole 13-212.

The embankment fill is underlain by about 1.6 m of organic sand at Borehole 13-213 and 2.3 m of sand with some silt and trace organic matter at Borehole 13-214. About 1.5 m of silty sand with trace organic matter was encountered at ground surface at Borehole 13-215.

SPT N values in these materials ranged from about 2 to 17 blows per 0.3 m of penetration indicating a very loose to compact relative density.

The results of grain size distribution testing carried out on samples of these materials are provided on Figure B2 in Appendix B. The measured natural water contents of several samples of these materials range from about 36 to 55 percent. The measured organic contents of two samples of these materials were about 3 and 11 percent.

4.2.3 Silty Sand to Sand

The fill, topsoil and upper sandy layers are underlain by a deposit of silty sand to sand with trace to some silt. Wood was also encountered within the deposit at some locations. The deposit was fully penetrated to depths between 6.1 to 15.2 m (Elevations 240.2 to 244.7 m) and is about 2.1 to 9.6 m thick.

The measured SPT N values range from 1 to 10 blows per 0.3 m of penetration, indicating a very loose to compact state of packing for the deposit.

The results of a grain size distribution carried out on several samples of the silty sand to sand are provided on Figure B3 in Appendix B. The measured natural water contents of these samples ranged from about 19 to 44 percent.

4.2.4 Organic Sand, Silty Sand and Silt

The silty sand to sand is underlain by a deposit of organic sand, silty sand and silt. Wood and roots were also encountered within the deposit. The deposit was fully penetrated to depths between 13.1 to 20.7 m (Elevations 232.3 to 240.6 m) and is about 4.1 to 7.6 m thick.

The measured SPT N values range from 1 to 17 blows per 0.3 m of penetration, indicating a very loose to compact state of packing for the deposit.

The results of a grain size distribution carried out on several samples of the deposit are provided on Figure B4 in Appendix B. The measured natural water contents of these samples ranged from about 37 to 103 percent. The measured organic contents of several samples of the deposit ranged from about 3 to 18 percent.

4.2.5 Lower Silty Sand, Sand and Gravel, Cobbles and Boulders

The organic sand at Borehole 13-215 is underlain by about 0.4 m of silty sand. One SPT value of 5 blows per 0.3 m of penetration was measured in the silty sand, indicating a loose state of packing.

The silty sand is underlain by and 0.5 m of sand and gravel containing cobbles. About 0.6 m of boulders were also encountered beneath the sand and gravel layer at Borehole 13-215.

The results of grain size distribution testing carried out on a sample of the sand and gravel are provided on Figure B5 in Appendix B. The results do not reflect the cobble, boulder or coarse gravel content of the material, because the samples were retrieved using a 50 mm outside diameter sampler. The measured natural water content of the sample was about 8 percent.

The organic silt at Borehole13-211 is underlain by an approximately 1.4 m layer of cobbles and boulders. Rotary diamond drilling techniques were required to advance the borehole through the cobbles and boulders at this location.

4.2.6 Refusal and Bedrock

Refusal to dynamic cone penetration was encountered at Elevation 232.3 m at Boreholes 13-212, which has been inferred to represent the bedrock surface. Bedrock was encountered beneath the cobbles and boulders at Borehole 13-211, the organic silty sand/sand at Boreholes 13-214 and 13-213, and beneath the sand and gravel at Borehole 13-215 where it was cored for depths between about 3.0 and 3.5 m. The following table summarizes the bedrock surface depths and elevations as encountered at the borehole locations.

Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
13-211	249.7	15.4	234.3
13-212	249.7	17.4*	232.3*
13-213	255.4	20.7	234.7
13-214	253.7	13.1	240.6
13-215	249.4	14.8	234.6

Note: * Depth and elevation to bedrock inferred from refusal to dynamic cone penetration.

The bedrock encountered in the cored boreholes typically consists of grey, blue, and pink granite bedrock. The bedrock is generally slightly weathered to fresh, fine to medium grained, crystalline, non-porous, and medium strong to strong.

The Rock Quality Designation (RQD) values measured on the recovered bedrock core samples typically ranged from about 84 to 100 percent, indicating good to excellent quality rock. A lower RQD value of 50 percent was measured in the upper portion of the bedrock at Borehole 13-215 indicating a poor quality rock. The discontinuities observed in the rock core were associated with the joints, veins, faults and fractures of the bedrock.

Laboratory unconfined compressive strength testing was carried out on two selected specimens of the bedrock core. The results of the testing are summarized on Figure B6 in Appendix B. The results of the unconfined compressive strength testing on indicate values of 48 and 58 MPa.

4.2.7 Groundwater Conditions

The groundwater condition observed upon completion of drilling at Boreholes 13-212 was at about Elevation 248.2 m. The groundwater level was not established at Boreholes 13-213, 13-214 and 13-215.

The groundwater levels measured in the piezometer in Borehole 13-211 are summarized in the table below:

Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date
13-211	249.7	1.2	248.5	June 3, 2013
		1.3	248.4	September 23, 2013

The water level in Eel's Creek was measured by MMM at Elevation 248.3 m in October 2012.

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

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Ms. Susan Trickey, P.Eng., and Mr. Matt Kennedy, P.Eng., and was reviewed by Ms. Lisa Coyne, P.Eng., a Principal and geotechnical engineer with Golder. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
EEL'S CREEK BRIDGE REPLACEMENT
SITE 26-118
HIGHWAY 28, 40 M SOUTH OF HAULTAIN ROAD
W.P. 4126-10-01

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6.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed replacement of the existing Eel's Creek Bridge on Highway 28. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this preliminary subsurface investigation. The discussion and preliminary recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the foundations for the replacement structure. Further investigation and analysis will be required during detail design.

Where comments are made on construction, they are provided to highlight those aspects that could affect the preliminary design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The existing bridge is shown on Drawing 1. As part of this assignment, various highway alignments and bridge locations have been considered for the replacement. It is understood that the preferred recommended alternative for the proposed two-lane replacement bridge includes an alignment shift to the east of the existing highway. The foundation investigations were undertaken based on this understanding, in consultation with the MTO. The foundation recommendations are limited to alignment alternatives to the east of the existing highway and structure, unless otherwise specified. Where discussion is provided that considers alignments within or to the west of the existing alignment, the recommendations are based on extrapolation of the factual subsurface data and should be considered preliminary and functional in nature only.

For highway alignments to the east of the existing, the east half of the bridge would only be constructed initially and the existing structure would be demolished prior to construction of the west half of the bridge. The span length of the proposed replacement bridge structure will depend on the selected structure location and highway alignment alternative, but is expected to be approximately 40 m.

6.2 Foundation Options

The existing Eel's Creek bridge consists of a ten-span timber deck supported by timber abutments and pier bents. Each pier bent was to consist of a wood pier cap supported on eight timber piles, which was subsequently increased to 15 timber piles. Based on the original design drawings, the piles are understood to range in length from about 10.6 to 13.7 m; however, the actual as-built lengths may vary. The founding depth of the timber piles is unknown but existing information suggests that the piles are likely founded within the very loose to compact sandy overburden soils (i.e., they are not end-bearing on the bedrock). The existing structure is aligned approximately north-south, and is about 47.5 m in length and 11.6 m in width. It is understood that the structure was built in 1952 and is presently in poor condition. There is also an active snowmobile trail under the bridge on the north side of the structure.

The natural ground surface within the lowland floodplain of Eel's Creek at the toe of the existing bridge embankments is at about Elevations 249.7 m. It is understood that the water level in the creek, as measured by MMM in October 2012, was at about Elevation 248.3 m.

Highway 28 in this area is a two-lane undivided highway with a rural cross-section. In the area of the bridge, Highway 28 has been constructed on embankments that are between about 4.5 and 7.5 m in height, with the pavement surface between about Elevations 254.1 and 255.2 m. The highway embankment side slopes are oriented between about 1.5 and 2 horizontal to 1 vertical (1.5H:1V to 2H:1V).

Based on the subsurface conditions, only deep foundation options have been considered for the replacement of the existing Eel's Creek bridge and construction of a temporary bridge (if this option is chosen), as shallow foundations would not provide sufficient bearing resistances or acceptable settlement for the structure. The recommendations for deep foundation design of a temporary bridge constructed adjacent to the existing highway alignment, and for the final replacement bridge will be similar in nature and are discussed in further detail below. A summary of the advantages and disadvantages associated with each deep foundation option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, constructability and relative costs is provided in Table 1 following the text of this report.

- **Driven steel H-piles:** Steel H-piles driven to refusal on the granite bedrock could be considered as a deep foundation option for the support of the bridge replacement. This option would provide high geotechnical resistances and minimal post-construction settlements for the new structure. If the piles are driven, the use of driving shoes is recommended to minimize damage while driving through the cobbles and boulders at depth above the granite bedrock and to provide fixity at the bedrock surface. This construction option could result in significant vibrations that could influence the timber piles of the existing bridge structure, especially while driving through the cobbles and boulders. Given the proximity of the new structure foundations to the existing timber structure, the energy delivered during driving of the new piles would have to be carefully controlled to limit vibrations transmitted to the existing structure and supporting timber friction piles.

- **Socketted steel pipe piles:** Socketted steel pipe piles installed using the down-the-hole hammer method could also be considered as a deep foundation option for support of the abutments. This foundation option would have similar advantages to steel H-piles in terms of high geotechnical resistances and minimal settlements. The vibrations associated with this type of pile installation would be lower than those expected for typical driven H-pile construction, even during penetration through the cobbles and boulders encountered at depth above the granite bedrock, thereby reducing the potential impacts on the existing timber bridge structure.

In general, the geotechnical recommendations on foundation design for a temporary structure will be similar to those provided for the replacement structure. However, it is important to note that the geotechnical investigation carried out at the site included boreholes within and to the east of the existing highway alignment only. No boreholes were put down to the west of the existing bridge and highway alignment. The recommendations provided in the following sections are limited to structures founded within and to the east of the existing highway and bridge alignment. Recommendations provided on alternatives to the west of the existing alignment are provided for discussion purposes only and should be confirmed by additional subsurface investigation should a westward option be selected.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments for the bridge replacement or temporary structure on steel pipe piles installed using the down-the-hole hammer method. However, support of the abutments on steel H-piles driven to found on the bedrock is also considered to be feasible, provided that the vibrations transmitted to the existing timber structure are monitored and kept to suitably low levels to minimize any detrimental effects on the existing structure.

6.3 Socketed Steel Pipe Pile Foundations

6.3.1 Founding Elevations

The abutments for the replacement bridge or temporary structure may be supported on steel H-piles driven to found on the granite bedrock or on steel pipe piles installed to found on the bedrock then socketed into the granite bedrock using the down-the-hole hammer method. It is recommended that the steel pipe piles be socketed 1.2 m (i.e., three pile diameters) into the bedrock for axial resistance considerations. Steel H-piles may be driven to a nominal embedment below the bedrock surface, provided they are reinforced with suitable driving shoes. All new piles should be at least 5 m away from the piled foundations of the existing bridge structure. Additional borehole investigation will be required at the detail design stage to confirm the bedrock surface variability within the footprint of the proposed abutment locations. However, based on the results from the preliminary investigation that included boreholes put down within and to the east of the existing highway alignment, the following socket founding elevations are recommended for preliminary design:

Foundation Element	Borehole Number	Bedrock Surface Elevation (m)	Design Socket Founding Elevation (m)
North Abutment	13-211	234.3	233.1
	13-215	234.0	232.8
South Abutment	13-212	232.3*	231.1*
	13-213	234.7	233.5

Note: * Bedrock surface elevation and design socket founding elevation inferred from refusal to dynamic cone penetration.

The pile caps should be constructed at a minimum depth of 1.7 m for frost protection purposes, per Ontario Provincial Standard Drawing (OPSD) 3090.101 *(Foundation Frost Penetration Depths for Southern Ontario).*

H-piles should be reinforced at the tip with rock point driving shoes to improve seating of the piles on the bedrock and to reduce the potential for damage to the piles during driving through the overlying cobbles and boulders, in accordance with Ontario Provincial Standard Specification (OPSS) 903 *(Deep Foundations).* To ensure adequate penetration into the hard and locally steeply sloping bedrock to provide fixity, a Titus HD Rock Injector rock point (or equivalent) driving shoe should be used.

6.3.2 Axial Geotechnical Resistance

For preliminary design of steel pipe piles, concrete-filled, 406 mm diameter piles installed to the bedrock surface elevations provided in Section 6.3.1, then socketed 1.2 m (i.e., three pile diameters) into the granite bedrock have been assumed. The preliminary foundation design recommendations have been based on the side-wall (shaft) resistance of the rock socket and a factored geotechnical resistance at Ultimate Limit States (ULS) of 1,500 kPa. For a 406 mm diameter pile, this would equate to a factored geotechnical resistance at ULS of 2,000 kN. The ULS resistance considers the RQD values recorded for the bedrock as well as the compressive strength data for the rock core. This value is applicable provided that the socket is within competent bedrock and that the side wall of the socket is cleaned of any smeared material.

For preliminary design of HP 310x110 piles driven to the estimated bedrock surface elevations provided in Section 6.3.1, the factored axial resistance at ULS may be taken as 2,000 kN.

Serviceability Limit States (SLS) resistances do not apply to piles founded on or socketed in the granitic bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial resistance at ULS.

Pile installation should be in accordance with OPSS 903 *(Deep Foundations)*.

For steel H-piles, the required hammer energy is expected to be relatively low through the loose sand and silty sand deposits. Significantly more energy will be required to penetrate the cobbles and boulders that overlie the bedrock. For piles driven to refusal on bedrock, and as described in OPSS 903, 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.

However, installation of steel H-piles driven from surface could result in vibrations that would influence the timber piles of the existing bridge structure (which are likely founded within the sandy overburden soils). Significant vibrations could result in settlement and/or other damage to the existing structure. The energy delivered to the H-piles would have to be carefully controlled to limit vibration transmitted to the existing timber friction piles.

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing structure are maintained below tolerable levels. A vibration monitoring program that limits the peak particle velocity at the existing structure should be developed at the detail design stage in consultation with a structural engineer. The piles furthest from the existing structure should be driven first, in order to check the vibration level at the existing structure and, if necessary, alter the pile driving procedures for the remaining piles.

The preliminary geotechnical resistances provided above will have to be re-evaluated and modified as necessary during detail design in consideration of the additional subsurface investigation that will be carried out at the site.

6.3.3 Downdrag Load (Negative Skin Friction)

If the very loose to loose sand deposit that underlies the embankment fill liquefies during a seismic event, post-seismic reconsolidation settlement of the liquefied soil would occur. The resulting downward movement of the overlying embankment fill would cause additional loading on the abutment piles and would result in downdrag forces that should be considered in design.

The magnitude of the down drag load would depend on the length of pile embedded in the embankment fill. Assuming that the pile caps are constructed at a depth of 1.7 m below the existing Highway 28 grade (to accommodate frost protection requirements), the resultant unfactored down drag load may be taken as about 525 kN for a single HP 310 x 110 pile, or about 550 kN for a single 406 mm diameter pipe pile. The magnitude of the down drag forces would decrease with shorter pile length embedded in the embankment fill (i.e., deeper pile caps). The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the Canadian Highway Bridge Design Code (CHBDC).

6.4 Approach Embankments

It is understood that construction of new approach embankments will be required for the permanent replacement bridge or temporary structure and that these embankments will be between about 4 and 7 m in height. Standard embankment construction (i.e., embankments with typical side slopes oriented at 2H:1V for earth fill, or 1.25H:1V for rock fill) is proposed along the east side of the new bridge approach embankments.

However, during construction of a bridge structure (temporary or permanent) to the east of the existing bridge, a temporary retaining wall consisting of a Reinforced Soil System (RSS) wall or Wire Wall would be required to retain the adjacent, existing approach embankment fill while the existing bridge is in place at the south end of the new bridge.

6.4.1 General Embankment Construction

It is recommended that all topsoil/organic soil, soil containing organic matter or existing surficial fill materials be stripped from the footprint of the approaches for the realigned or widened Highway 28 embankments.

The embankment fill for the bridge replacement should be placed and compacted in accordance with OPSS 206 *(Grading)* and OPSS 501 *(Compacting)*. Benching of the existing Highway 28 embankment side slopes should be carried out to "key in" the new fill materials in areas where the embankment is widened, in accordance with OPSD 208.010 (*Benching of Earth Slopes*).

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS 804 *(Seed and Cover)*.

Provided the embankment heights are maintained between about 4 and 7 m and are constructed using suitable compacted rock fill, the new embankments should have an adequate factor of safety against both static and seismic slope instability (i.e., greater than 1.3 under static conditions, and greater than 1.1 under seismic conditions). These values were calculated assuming an effective friction angle of 45° for the rock fill and 30° for the native sandy soils. The side slopes of the embankment should be sloped no steeper than 1.25H:1V.

Settlement of the embankments will occur as a result of compression of the underlying very loose to compact sandy soils as well as compression of the new embankment fill. The settlement of the native sandy soils will be elastic in nature and should therefore occur during construction. Settlement of the rock fill itself will depend on the type of rock fill and on the method and sequence of placement and compaction of the fill. Assuming the rock fill is placed in accordance with the requirements outlined in SP206S03, the settlement of the rock fill embankments is estimated to be about 1 percent of the embankment height and it is anticipated that the majority of the settlement will occur during the first year following construction.

This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed based on the subsoil conditions encountered within the proposed approach embankment footprints during detail design.

6.5 Retaining Wall

6.5.1 Reinforced Soil System or Temporary Wire Walls

As mentioned previously, it is understood that a temporary retaining wall may be required at the south end of a new bridge structure constructed to the east of the existing bridge, where a new abutment would be closer to the creek bank than the existing abutment (i.e., additional fill must be placed on/adjacent to the existing embankment behind the new abutment wall where no fill currently exists).

A combination of an RSS/temporary wire wall and a sloped embankment could be used at the south end of the bridge. It is also understood that the wall would be up to about 6 m in height. The use of a temporary RSS wall or wire wall is considered feasible from a foundations perspective for this site.

It is recommended that all topsoil/organic soil, soil containing organic matter or existing surficial fill materials be stripped from the footprint of the proposed wall. The wall should then be founded on a minimum 0.3 m thick pad of engineered fill consisting of OPSS Granular B Type II placed in maximum 300 mm thick lifts and compacted to at least 95 percent of its standard Proctor maximum dry density in accordance with SP105S10. The proprietary wall supplier should confirm the adequacy of the suggested engineered fill pad prior to construction.

A factored geotechnical resistance at ULS of 150 kPa and a geotechnical resistance at SLS of 100 kPa may be used for the design of the wall.

With appropriate subgrade preparation and proper placement of the granular soils for construction of the wall (such as OPSS Granular B Type II), the approximately 6 m high wall should have an adequate factor of safety against both static and seismic slope instability (i.e., greater than 1.3 under static conditions, and greater than 1.1 under seismic conditions).

Settlement of the wall will occur as a result of compression of the underlying very loose to compact sandy soils. The settlement of the native sandy soils will be elastic in nature and should therefore occur during construction. It is expected that the wall will be able to accommodate the resulting total and differential settlements of the native soils and that slip joints within the wall will not be required.

This preliminary assessment of the geotechnical resistances, stability and settlement of the temporary RSS wall or wire wall provided above will have to be re-evaluated and modified as necessary during detail design.

6.6 Future Design and Construction Considerations

The following sections identify future construction issues that should be considered during the preliminary design stage as they may impact the planning and preliminary design.

6.6.1 Seismic Considerations

The site is located near Peterborough Ontario and according to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio, A, applicable to this site is 0.05. The corresponding acceleration related seismic zone, Z_a, is 1.

The soils at this site consist of very loose to compact sandy soils below the water table; therefore, these soils could potentially be liquefiable. The following provides a general description of seismic liquefaction and its potential impacts on foundations and embankments.

Seismic liquefaction occurs when earthquake induced vibrations cause an increase in pore water pressure within the soil. The presence of excess pore water pressures 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'. This strength loss can also result in instability of slopes, approach embankments, and retaining structures (i.e., deep-seated shear failure through the underlying soil);
- Reduced shear resistance (i.e., bearing capacity) of soils which support foundations, as well as reduced resistance to sliding of shallow foundations; 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 settlements. If seismic settlements occur, down drag-loads would also be induced on deep foundation elements; the design of the foundations would have to consider this additional load, discussed in Section 6.3.3, which would result in the requirement for higher capacity piles or a higher number of piles.

Therefore, the potential for seismic liquefaction of this deposit and its impact on the foundation design should be taken into consideration during detailed design.

6.6.2 Excavation and Temporary Protection Systems

Depending on the local topography, the foundation excavations for pile caps are expected to extend to depths ranging from about 1.7 to 3.0 m below the existing ground surface at the creek bank (i.e., up to about 7 to 10 m below the existing Highway 28 grade) into the water-bearing, very loose to loose sandy deposits.

Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. Excavation works must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects. The existing fill and overburden sandy deposits above the water table would be classified as Type 3 soil based on the OHSA. According to OHSA, excavations that extend to, or into, Type 3 soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V). The sandy deposits below the water table would be classified as Type 4 soil based on OSHA, and excavations in these materials should be sloped no steeper than 3H:1V. However, if active dewatering is used for the sandy soils (i.e., assuming that the groundwater level in the soils is lowered to about 0.5 m below the maximum depth of excavation) these soils may be classified as Type 3, according to OHSA and temporary excavation side slopes could be made at 1H:1V.

At this preliminary stage, it is anticipated that temporary protection system would likely be required to facilitate the excavation to foundation level of the pile caps for the new abutments and potentially for the removal of the existing bridge. Where shoring is required, the protection system should be designed and constructed in accordance with OPSS 539 *(Temporary Protection Systems)*. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539.

It is considered that soldier pile and lagging or an interlocking sheetpile system would be feasible at this site. The use of an interlocking sheetpile system has an advantage over soldier pile and lagging in that it would aid in groundwater control at this site, although the presence of cobbles or boulders within the embankment may impact the depth that sheetpiling can be driven and the effectiveness of the system. Therefore, the preferred method of shoring north of the creek, where cobbles and boulders were encountered in the embankment fill, would be soldier piles and lagging. Pre-augering through the embankment fill may also be required prior to the installation of the soldier piles for the protection system to limit the vibration impacts on the existing bridge structure. Interlocking sheetpiling is considered to be a feasible shoring option south of the creek, where the sheetpiling would extend through the native, very loose to compact, sandy deposits that contain cobbles, but where boulders aren't expected.

The soldier pile and lagging or sheetpiling would be supported against lateral movement using walers, tie backs and/or internal struts/braces. To reduce potential disturbance of the existing bridge, sheetpiling or pre-augered soldier piles should be installed at a minimum offset of 2 m from the piled foundations of the existing structure. Where soldier piles are to be driven, the protection system should be installed at a minimum offset of 5 m from the existing piled foundations.

6.6.3 Groundwater Control

The excavation for the bridge abutments will extend into the water-bearing sandy deposits. It is anticipated that the use of interlocking sheetpile walls (cofferdams), with dewatering from wells or wellpoints within or outside the cofferdams, will be appropriate to control the excavation sides and groundwater for foundation excavations for the pile caps. Based on the subsurface soil and groundwater conditions, it is anticipated that the dewatering rate will exceed 50 m^3/day, and therefore, a Permit to Take Water (PTTW) will be required for this site.

Surface water should be directed away from the excavation areas, to prevent ponding of water that could result in disturbance and weakening of the subgrade.

6.6.4 Vibration Monitoring During Pile Installation

Due to the founding conditions of the existing bridge (supported on friction timber piles) and the planned staged construction and proximity to the existing structure, vibration monitoring is recommended during pile installation to assist in maintaining vibration levels within tolerable ranges for the for the existing portions of the bridge.

A maximum peak particle velocity of 50 mm/sec is recommended at the existing structure foundations. The piles furthest from the existing structure should be driven first, in order to check the vibration level at the existing structure and, if necessary, alter the installation procedures for the remaining piles.

6.6.5 Obstructions

Cobbles and boulders were encountered both within the embankment fill and at depth above the bedrock surface, which could affect the installation of the protection systems and pile foundations. Further observation is recommended in the next stage of investigation in support of the Design-Build.

6.6.6 Erosion and Scour Protection

The near-surface soils at the site are expected to be susceptible to erosion and scour under the design flood/flow velocities. The requirements for design of erosion/scour protection should be assessed by the hydraulic design engineer. As a minimum, it is recommended that erosion protection (e.g., rip-rap or granular sheeting) be provided on the creek banks up to the high water level to protect the pile caps from being exposed. The rip-rap should be consistent with the standard R-10 classification or granular sheeting classification in accordance with OPSS 1004 (Aggregates) but should be approved by the hydraulic design engineer.

6.7 Recommendations for Further Work in Detail Design

The design-build proponent will be responsible for the detail design and assessing additional requirements for investigations to suit the final design and mitigating any identified constructions risks. However, at this functional/preliminary design stage, it is anticipated that additional boreholes will be required during the design-build stage of investigation, to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided in this report, as follows:

- Abutments:
  - Confirmation of the bedrock surface elevation within the proposed abutment area, to confirm the founding elevation for deep foundations.
  - Observation of the presence of cobbles and/or boulders within the soil deposits, to assess the presence of such obstructions as they may affect excavations and the installation of elements of temporary protection systems and deep foundations.
  - Further assessment of the potential for liquefaction at the site and its impacts on the deep foundations.
  - Further assessment of the groundwater level and permeability of the site soils to refine dewatering estimates.
- Approach embankments:
  - Assessment of the depth and extent of stripping of topsoil/organic soils and fill materials within the footprint of the approach embankments.
  - Further assessment of the thickness and elastic compression properties of any loose sandy soils within the footprint of the approach embankments, to confirm the settlement estimates.
  - Further assessment of the potential for liquefaction at the site and its impacts on embankment stability.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Ms. Susan Trickey, P.Eng., and Mr. Matt Kennedy, P.Eng., with technical input from Mr. Murty Devata, P.Eng. It was reviewed by Ms. Lisa Coyne, P.Eng., a Principal and geotechnical engineer with Golder. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.

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SAT/MJK/LCC/FJH/bg

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Table 1 – Comparison of Foundation Alternatives

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Relative Costs
Steel H-piles driven to bedrock	▪ Feasible for support of bridge replacement	▪ High geotechnical resistances and negligible settlement	▪ Some groundwater control would still be required ▪ Potential for encountering obstructions (cobbles and/or boulders) during pile driving that could result in piles "hanging up" and lower geotechnical resistances ▪ Potential need for pre-augering through the cobbles and boulders ▪ Could result in higher vibrations during pile driving (than the pipe piles) which could impact the existing bridge structure	▪ Conventional construction methods for H-pile foundations	▪ Lower relative cost compared with pipe pile option
Socketed steel pipe piles installed in the bedrock	▪ Feasible for support of bridge replacement ▪ Preferred option from a foundations perspective	▪ High geotechnical resistances and negligible settlement ▪ Installation using the down-the-hole hammer method would result in lower vibrations	▪ Requires specialized equipment for penetrating through the cobbles and boulders and for forming the rock socket	▪ Rock socketing would be required using the down-the-hole hammer method	▪ Costs for steel pipe piles slightly higher than for steel H-piles

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Relative Costs
Temporary bridge to the east of existing alignment	▪ Feasible for temporary bypass of existing bridge area	▪ Allows reconstruction of permanent bridge on existing alignment	▪ Requires additional deep foundations in addition to those for permanent bridge	▪ Foundation construction would include same risks as those for permanent structure	▪ Significant additional cost associated with temporary deep foundations
Temporary bridge to the west of existing alignment	▪ Feasible for temporary bypass of existing bridge area	▪ Allows reconstruction of permanent bridge on existing alignment	▪ Requires additional deep foundations in addition to those for permanent bridge ▪ Subsurface geometry and condition of existing heritage foundation unknown and may conflict with proposed work	▪ Foundation construction would include same risks as those for permanent structure	▪ Significant additional cost associated with temporary deep foundations

MINISTRY OF TRANSPORTATION, ONTARIO

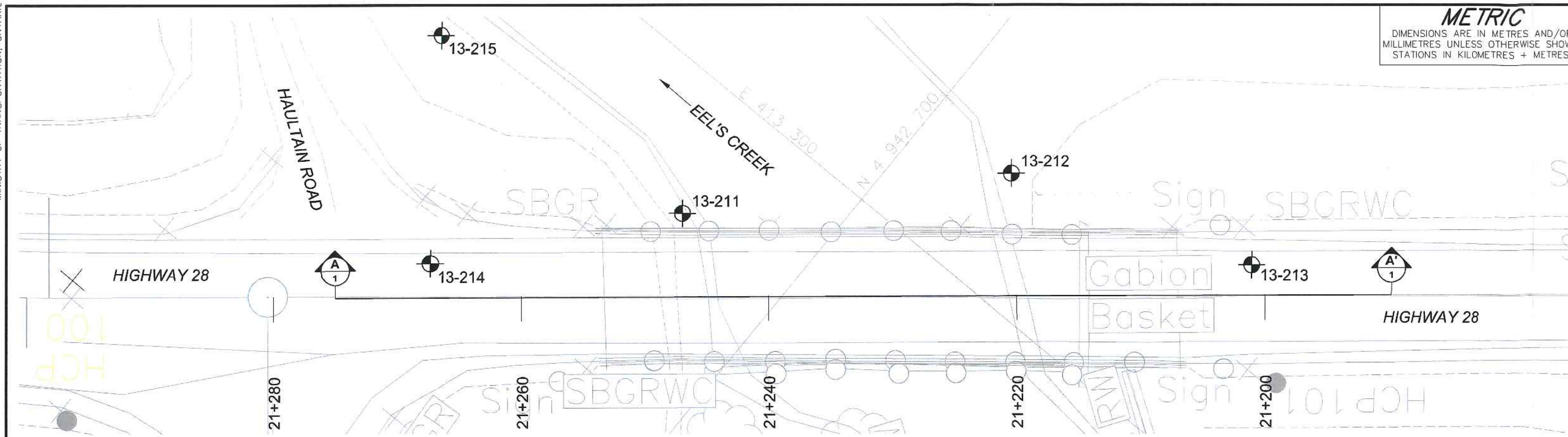
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 4126-10-01

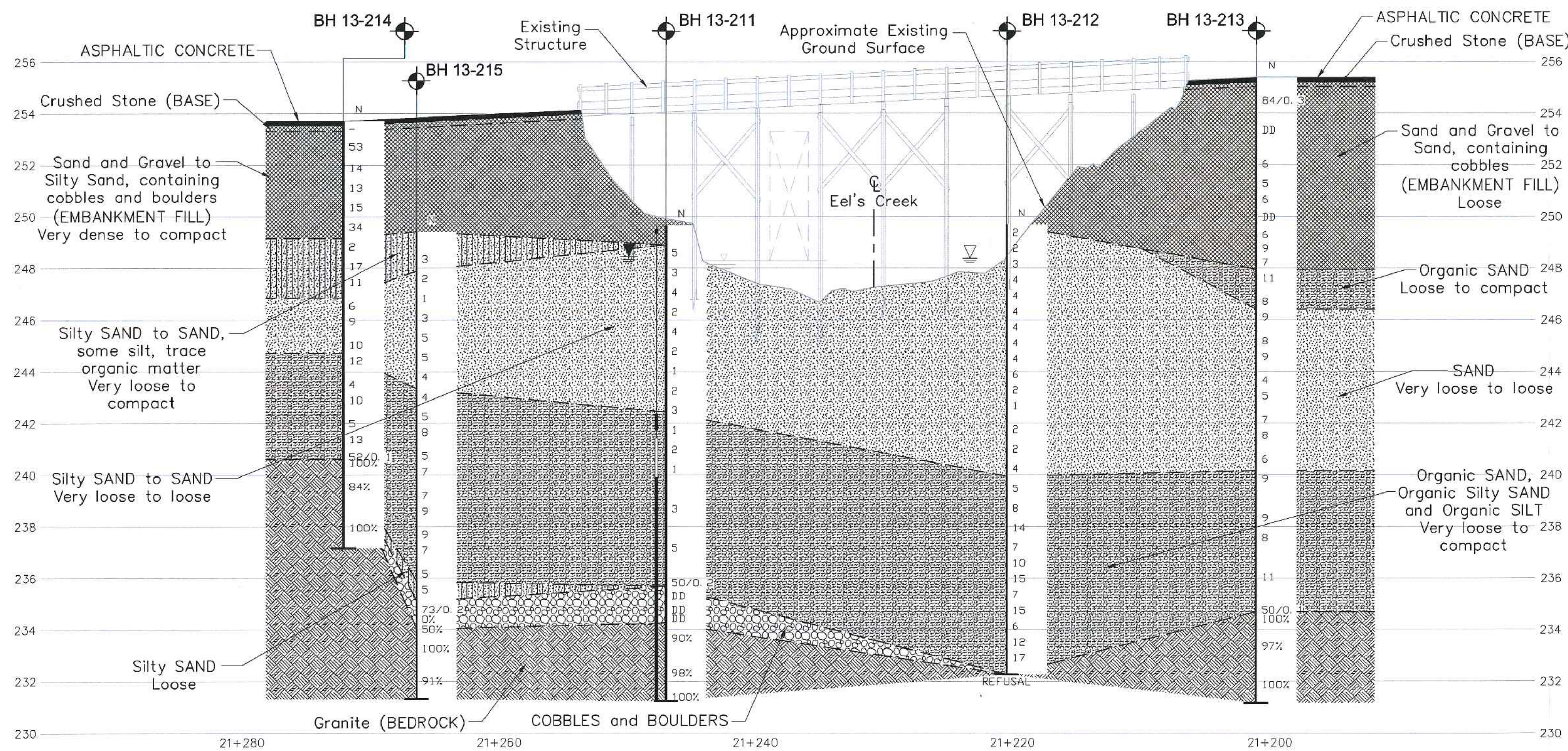
HIGHWAY 28-BRIDGE REPLACEMENT
EEL'S CREEK
SITE 26-118
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

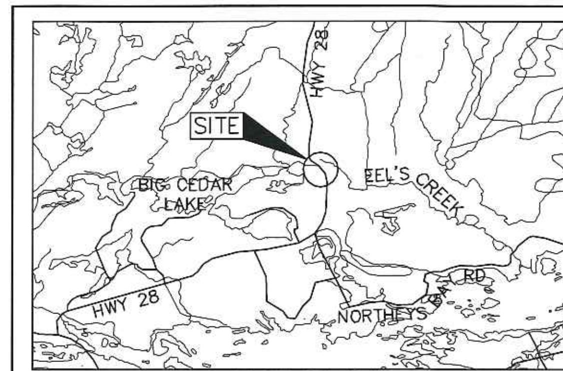
Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



PLAN
SCALE
4 0 4 8 m



PROFILE
A-A' 1
HORIZONTAL SCALE
4 0 4 8 m
2 0 2 4 m
VERTICAL SCALE



KEY PLAN
SCALE
2 0 2 4 km

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation
- WL in piezometer
- WL upon completion of or during drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
12-211	249.7	4942710.4	413290.0
12-212	249.7	4942692.0	413309.3
12-213	255.4	4942672.3	413315.9
12-214	253.7	4942723.7	413273.9
12-215	249.4	4942734.6	413288.6

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.

REFERENCE

Base plans provided in digital format by MMM Group Limited, drawing file no. B-PLAN 26-118.dwg, received July 15, 2013.

NO.	DATE	BY	REVISION

Geocres No. 31D-567			
HWY. 28		PROJECT NO. 12-1121-0099-1210	DIST. Eastern
SUBM'D. SAT	CHKD. FJH	DATE: Jan. 2014	SITE: 26-118
DRAWN: JM	CHKD. MJK	APPD. FJH	DWG. 1

PLOT DATE: February 27, 2014

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO or DP	Seamless open-ended, driven or pushed tube samplers
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split spoon sampler
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample
DT	Dual tube sample
DD	Diamond drilling

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive an uncased 50 mm (2 in.) diameter, 60^0 cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Cone Penetration Test (CPT):

An electronic cone penetrometer with a 60^0 conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	**N** Blows/300 mm Or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

C_u or S_u

Consistency	kPa	Psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	Over 200	Over 4,000

IV. SOIL TESTS

w	Water content
w_p or PL	Plastic limited
w_l or LL	Liquid limit
C	Consolidaiton (oedometer) test
CHEM	Chemical analysis (refer to text)
CID	Consolidated isotropically drained triaxial test[1]
CIU	Consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	Relative density
DS	Direct shear test
Gs	Specific gravity
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	Organic content test
SO_4	Concentration of water-soluble sulphates
UC	Unconfined compression test
UU	Unconsolidated undrained triaxial test
V	Field vane test (LV-laboratory vane test)
γ	Unit weight

Note: [1] Tests which are anisotropically consolidated prior shear are shown as CAD, CAU.

LIST OF SYMBOLS

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

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$ x or log x	logarithm of x to base 10
g	acceleration due to gravity
t	time
FOS	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes:
[1] $\tau = c' + \sigma' \tan \phi'$
[2] shear strength = (compressive strength) / 2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of rock material weathering
Faintly Weathered: weathering limited to the surface of major discontinuities.
Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.
Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable
Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.
Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very Thickly Bedded	> 2 m
Thickly Bedded	0.6 m to 2m
Medium Bedded	0.2 m to 0.6 m
Thinly Bedded	60 mm to 0.2 m
Very Thinly Bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly Laminated	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very Wide	> 3 m
Wide	1 – 3 m
Moderately Close	0.3 – 1 m
Close	50 – 300 mm
Very Close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 – 60 mm
Medium Grained	60 microns – 2mm
Fine Grained	2 – 60 microns
Very Fine Grained	< 2 microns

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

CORE CONDITION

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

Solid Core Recovery (SCR)
The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

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

DISCONTINUITY DATA

Fracture Index
A count of the number of discontinuities (physical separations) in the rock core, including naturally occurring fractures but not including mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90^0 angle is horizontal.

Description and Notes
An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

BD -	Bedding	PY -	Pyrite
FO -	Foliation/Schistosity	Ca -	Calcite
CL -	Clean	PO -	Polished
SH -	Shear Plane/Zone	K -	Slickensided
VN -	Vein	SM -	Smooth
FLT -	Fault	RO -	Ridged/Rough
CO -	Contact	ST -	Stepped
JN -	Joint	PL -	Planar
FR -	Fracture	IR -	Irregular
MB -	Mechanical Break	UN -	Undulating
BR -	Broken Rock	CU -	Curved
BL -	Blast Induced	TCA -	To Core Axis
II -	Parallel To	STR -	Stress Induced
OR -	Orthogonal		

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 13-211

SHEET 1 OF 3 — **METRIC**

PROJECT 12-1121-0099-1210
G.W.P. 4126-10-01
DIST Eastern HWY 28
DATUM Geodetic
LOCATION N 4942710.4 ;E 413290.0
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core
DATE May 15-16, 2013
ORIGINATED BY HEC
COMPILED BY JM
CHECKED BY SAT

ELEV DEPTH	DESCRIPTION
249.7	GROUND SURFACE
0.0	Silty sand, trace gravel, cobbles and roots (FILL) Brown
248.9 / 0.8	Silty SAND, trace clay and shells; Loose; Grey-brown; Moist to wet
247.9 / 1.8	SAND, some silt, trace wood; Loose; Dark grey; Wet
246.7 / 3.1	SAND, trace silt; Very loose; Grey-brown; Wet
242.5 / 7.2	Organic SAND, some silt and sand seams, trace wood and rootlets; Very loose; Dark grey; Wet
236.9 / 12.8	Organic SILT, some sand, trace wood and rootlets; Very loose; Grey to grey-brown; Wet
235.7 / 14.0	COBBLES and BOULDERS

SAMPLE NUMBER	TYPE	"N" VALUES	Water content (%)	Unit weight / Remarks	GR	SA	SI	CL
1	SS	5	○		0	50	47	3
2	SS	3						
3	SS	4	○		2	85	12	1
4	SS	2						
5	SS	4	○		0	96	4	0
6	SS	2						
7	SS	1	○		0	96	4	0
8	SS	2						
9	SS	3	○		0	99	1	0
10	SS	1	○	ORG = 8.3				
11	SS	2						
12	SS	1						
13	SS	3	○	ORG = 7.4				
14	SS	5						
15	SS	50/0.2	○	ORG = 5.4				
16	RC	DD						
17	RC	DD						

ELEVATION SCALE: 249, 248, 247, 246, 245, 244, 243, 242, 241, 240, 239, 238, 237, 236, 235

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

Continued Next Page

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

RECORD OF BOREHOLE No 13-211

METRIC

PROJECT 12-1121-0099-1210

G.W.P. 4126-10-01 LOCATION N 4942710.4 ;E 413290.0 ORIGINATED BY HEC

DIST Eastern HWY 28 BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core COMPILED BY JM

DATUM Geodetic DATE May 15-16, 2013 CHECKED BY SAT

ELEV DEPTH	SOIL PROFILE — DESCRIPTION (--- CONTINUED FROM PREVIOUS PAGE ---)	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
234.3	COBBLES and BOULDERS		18	RC	DD						
15.4	Granite (BEDROCK) Bedrock cored from depths of 15.4 m to 18.4 m For bedrock coring details refer to Record of Drillhole 13-211		1	RC	REC 100%		234				RQD = 90%
			2	RC	REC 100%		233 232				RQD = 98%
231.3			3	RC	REC 100%						RQD = 100%
18.4	END OF BOREHOLE										

NOTES:

1. Water level in well screen at a depth of 1.2 m below ground surface (Elev. 248.5 m), measured on June 3, 2013.
2. Water level in well screen at a depth of 1.3 m below ground surface (Elev. 248.4 m), measured on September 23, 2013.

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

Golder Associates

RECORD OF BOREHOLE No 13-212

SHEET 2 OF 2

METRIC

PROJECT 12-1121-0099-1210			
G.W.P. 4126-10-01	LOCATION N 4942692.0 ;E 413309.3		ORIGINATED BY DG
DIST Eastern	HWY 28	BOREHOLE TYPE Portable Drill, NW/BW Casing	COMPILED BY JM
DATUM Geodetic	DATE May 14-15, 2013		CHECKED BY SAT

SOIL PROFILE — CONTINUED FROM PREVIOUS PAGE —

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20, 40, 60, 80, 100

SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED: 20, 40, 60, 80, 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%): 25, 50, 75

ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
234.5										
15.2	Organic SILT, some sand, trace wood and rootlets Loose to compact Dark brown Wet		24	SS	6		234			
			25	SS	12					
			26	SS	17		233	102.9		0 28 (72)
232.6	END OF BOREHOLE									
232.3 17.4	DYNAMIC CONE PENETRATION TEST REFUSAL									

NOTES:

1. Water level in open borehole at a depth of 1.3 m below ground surface (Elev. 248.4 m), measured during drilling.

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

RECORD OF BOREHOLE No 13-213

SHEET 1 OF 3 — METRIC

PROJECT 12-1121-0099-1210
G.W.P. 4126-10-01 — LOCATION N 4942672.3 ;E 413315.9 — ORIGINATED BY DG
DIST Eastern HWY 28 — BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring NW Casing, Rotary Drill NQ Core — COMPILED BY JM
DATUM Geodetic — DATE September 24-25, 2013 — CHECKED BY SAT

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
255.4	GROUND SURFACE									
0.0	ASPHALTIC CONCRETE									
0.4	Crushed stone (BASE) Grey									
	Sand and gravel, some silt to sand, trace to some gravel and silt, with cobbles and boulders (EMBANKMENT FILL) Loose Brown Moist to wet	1	SS	84/0.3	○		38	49	12	1
		1A	RC	DD						
		2	SS	6						
		3	SS	5						
		4	SS	6						
		4A	RC	DD						
		5	SS	6						
		6	SS	9	○		3	89	8	0
		7	SS	7						
248.0 / 7.4	Organic SAND, some silt, trace gravel, trace roots Loose to compact Dark brown Wet	8	SS	11	○	ORG = 11				
		9	SS	8	○		4	84	(12)	
246.4 / 9.0	SAND, trace silt, trace wood Loose Grey-brown Wet	10	SS	9						
		11	SS	8						
		12	SS	9						
		13	SS	4						
		14	SS	5	○		0	95	4	1
		15	SS	7						
		16	SS	8						
		17	SS	6						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (255–241); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; REMARKS & GRAIN SIZE DISTRIBUTION (%).

Continued Next Page

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

PROJECT 12-1121-0099-1210	RECORD OF BOREHOLE No 13-213	SHEET 2 OF 3	METRIC
G.W.P. 4126-10-01	LOCATION N 4942672.3 ;E 413315.9		ORIGINATED BY DG
DIST Eastern HWY 28	BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring NW Casing, Rotary Drill NQ Core		COMPILED BY JM
DATUM Geodetic	DATE September 24-25, 2013		CHECKED BY SAT

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
240.2 / 15.2	Organic SAND, trace to some silt, trace wood Loose to compact Dark brown Wet		18	SS	9		240				
							239				
			19	SS	9				○	ORG = 3.4	
							238				
			20	SS	8						
							237				
			21	SS	11		236		○		0 92 (8)
							235				
234.7			22	SS	50/0.1						
20.7	Granite (BEDROCK) Bedrock cored from depths of 20.7 m to 24.2 m For bedrock coring details refer to Record of Drillhole 13-213		1	RC	REC 100%						RQD = 100%
							234				
			2	RC	REC 100%		233				RQD = 97%
			3	RC	REC 100%		232				RQD = 100%
231.2 / 24.2	End of Borehole										

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

PROJECT: 12-1121-0099-1210

LOCATION: N 4942672.3 ;E 413315.9

INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: 13-213

DRILLING DATE: September 24-25, 2013

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

NOTE: For abbreviations, symbols and descriptions refer to **LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY**

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	WEATHERING INDEX	FEATURES	NOTES
		Continued from Record of Borehole 13-213		234.70														
21	Rotary Drill NQ Core	Granite (BEDROCK) Fresh Foliated Light grey Felsic Fine to medium grained Strong		20.70	1	100												
22					2	100												
23–24					3	100												
				231.17 24.23														
		End of Drillhole																
25																		
26																		
27																		
28																		
29																		
30																		
31																		
32																		
33																		
34																		
35																		

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: DG

CHECKED: SAT

GTA-RCK 031 1211210099.GPJ GAL-MISS.GDT 02/13/14 JM

RECORD OF BOREHOLE No 13-214

SHEET 1 OF 3 **METRIC**

PROJECT 12-1121-0099-1210 | LOCATION N 4942723.7 ;E 413273.9 | ORIGINATED BY DG

G.W.P. 4126-10-01 | BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring NW Casing, Rotary Drill NQ Core | COMPILED BY JM

DIST Eastern HWY 28 | DATE September 20-23, 2013 | CHECKED BY SAT

DATUM Geodetic

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%): 25 50 75

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
253.7	GROUND SURFACE						
0.0	ASPHALTIC CONCRETE						
	Crushed stone (BASE) Grey Moist	1	GRAB	-			
0.4	Sand and gravel, some silt, with cobbles and boulders (EMBANKMENT FILL) Very dense to compact Brown Moist	2	SS	53	253		34 51 14 1
		3	SS	14	252		
		4	SS	13	251		
		5	SS	15			50 38 10 2
		6	SS	34	250		
249.1 / 4.6	SAND, some silt, trace wood Very loose to compact Dark brown Wet	7	SS	2	249		
		8	SS	17	248	ORG = 3.3	
		9	SS	11	247		
246.8 / 6.9	SAND, trace silt Loose Grey-brown Wet	10	SS	6			
		11	SS	9	246		
		12	SS	10	245		
244.7 / 9.0	Organic SAND, some silt, trace wood Loose to compact Dark brown Wet	13	SS	12	244		
		14	SS	4			1 80 (20)
		15	SS	10	243	ORG = 3.4	
242.3 / 11.4	Organic Silty SAND, trace wood Loose to compact Dark brown Wet	16	SS	5	242		0 57 (43)
		17	SS	13	241	ORG = 4.1	
		18	SS	52/0.1			
240.6 / 13.1	Granite (BEDROCK)	1	RC	REC 100%			RQD = 100%
	Bedrock cored from depths of 13.1 m to 16.6 m. For bedrock coring details refer to Record of Drillhole 13-214	2	RC	REC 100%	240 / 239		RQD = 84%

Continued Next Page

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

RECORD OF BOREHOLE No 13-214

SHEET 2 OF 3 **METRIC**

PROJECT 12-1121-0099-1210
G.W.P. 4126-10-01
LOCATION N 4942723.7 ;E 413273.9
ORIGINATED BY DG
DIST Eastern HWY 28
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring NW Casing, Rotary Drill NQ Core
COMPILED BY JM
DATUM Geodetic
DATE September 20-23, 2013
CHECKED BY SAT

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Granite (BEDROCK) Bedrock cored from depths of 13.1 m to 16.6 m For bedrock coring details refer to Record of Drillhole 13-214		3	RC	REC 100%		238				RQD = 100%
237.1 16.6	End of Borehole										

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

PROJECT: 12-1121-0099-1210

RECORD OF DRILLHOLE: 13-214

LOCATION: N 4942723.7 ;E 413273.9

DRILLING DATE: September 20-23, 2013

DATUM: Geodetic

DRILL RIG: CME 55

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Marathon Drilling

NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES
		Continued from Record of Borehole 13-214	240.61				
	Rotary Drill, NQ Core	Granite (BEDROCK) Slightly weathered to fresh Foliated Light grey Fine to medium grained Crystalline Non-porous Strong	13.11	1	100		
14				2	100	FO,RO FO,RO	
15–16				3	50		
			237.16 16.56				
		End of Drillhole					

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: DG

CHECKED: SAT

Golder Associates

RECORD OF BOREHOLE No 13-215

SHEET 1 OF 3 **METRIC**

PROJECT 12-1121-0099-1210
G.W.P. 4126-10-01
DIST Eastern HWY 28
DATUM Geodetic
LOCATION N 4942734.6 ;E 413288.6
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring NW Casing, Rotary Drill NQ Core
DATE September 26, 2013
ORIGINATED BY DG
COMPILED BY JM
CHECKED BY SAT

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	UNIT WEIGHT / REMARKS	GR	SA	SI	CL
249.4	GROUND SURFACE									
0.0	Silty SAND, trace organic matter, Very loose, Dark brown, Wet	1	SS	3	○		1	65	34	0
247.9 / 1.5	Silty SAND, Very loose, Grey, Wet	2	SS	2						
247.1 / 2.3	SAND, trace silt, Very loose to loose, Grey-brown, Wet	3	SS	1						
		4	SS	3						
		5	SS	5	○		0	98	2	0
		6	SS	5						
		7	SS	4						
243.3 / 6.1	Organic SAND, some silt, trace wood and roots, Loose, Dark brown, Wet	8	SS	4	○		6	85	(9)	
		9	SS	5						
		10	SS	8	○	ORG = 9.5				
		11	SS	5						
		12	SS	7	○		0	86	(14)	
		13	SS	7						
		14	SS	9						
		15	SS	9						
		16	SS	7	○	ORG = 11.3				
		17	SS	5						
235.8 / 13.6	Silty SAND, Loose, Grey, Wet	18	SS	5						
235.1 / 14.3		19	SS	73/0.2	○		59	36	(5)	
234.6 / 14.8										

Continued Next Page

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 13-215

SHEET 2 OF 3 **METRIC**

PROJECT 12-1121-0099-1210
G.W.P. 4126-10-01 LOCATION N 4942734.6 ;E 413288.6 ORIGINATED BY DG
DIST Eastern HWY 28 BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Wash Boring NW Casing, Rotary Drill NQ Core COMPILED BY JM
DATUM Geodetic DATE September 26, 2013 CHECKED BY SAT

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
234.0 15.4	SAND and GRAVEL, trace silt, with cobbles and boulders Dense Grey Wet BOULDER		1	RC	~~REC 100%~~		234				RQD = 50%
	Granite (BEDROCK) Bedrock cored from depths of 14.8 m to 18.1 m For bedrock coring details refer to Record of Drillhole 13-215		2	RC	REC 100%		233				RQD = 100%
			3	RC	REC 100%		232				RQD = 91%
231.3 18.1	End of Borehole										

Plot scales: Dynamic cone 20 40 60 80 100; Shear strength ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; Water content 25 50 75

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

PROJECT: 12-1121-0099-1210

RECORD OF DRILLHOLE: 13-215

LOCATION: N 4942734.6 ;E 413288.6

DRILLING DATE: September 26, 2013

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES
		Continued from Record of Borehole 13-215	234.03				
16	Rotary Drill, NQ Core	Granite (BEDROCK) Fresh Dark grey and blue Fine to medium grained Crystalline Non-porous Strong	15.40	1 2	100 100	VN	
17–18	Rotary Drill, NQ Core	Granite (BEDROCK), with small weathered veins (~1 mm thick) Fresh Pink Fine to medium grained Crystalline Non-porous Medium strong to strong	232.74 16.69	3	100	VN VN VN VN	
		End of Drillhole	231.33 18.10				

Depth scale: 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30

GTA-RCK 031 1211210099.GPJ GAL-MISS.GDT 02/13/14 JM

DEPTH SCALE

1 : 75

Golder Associates

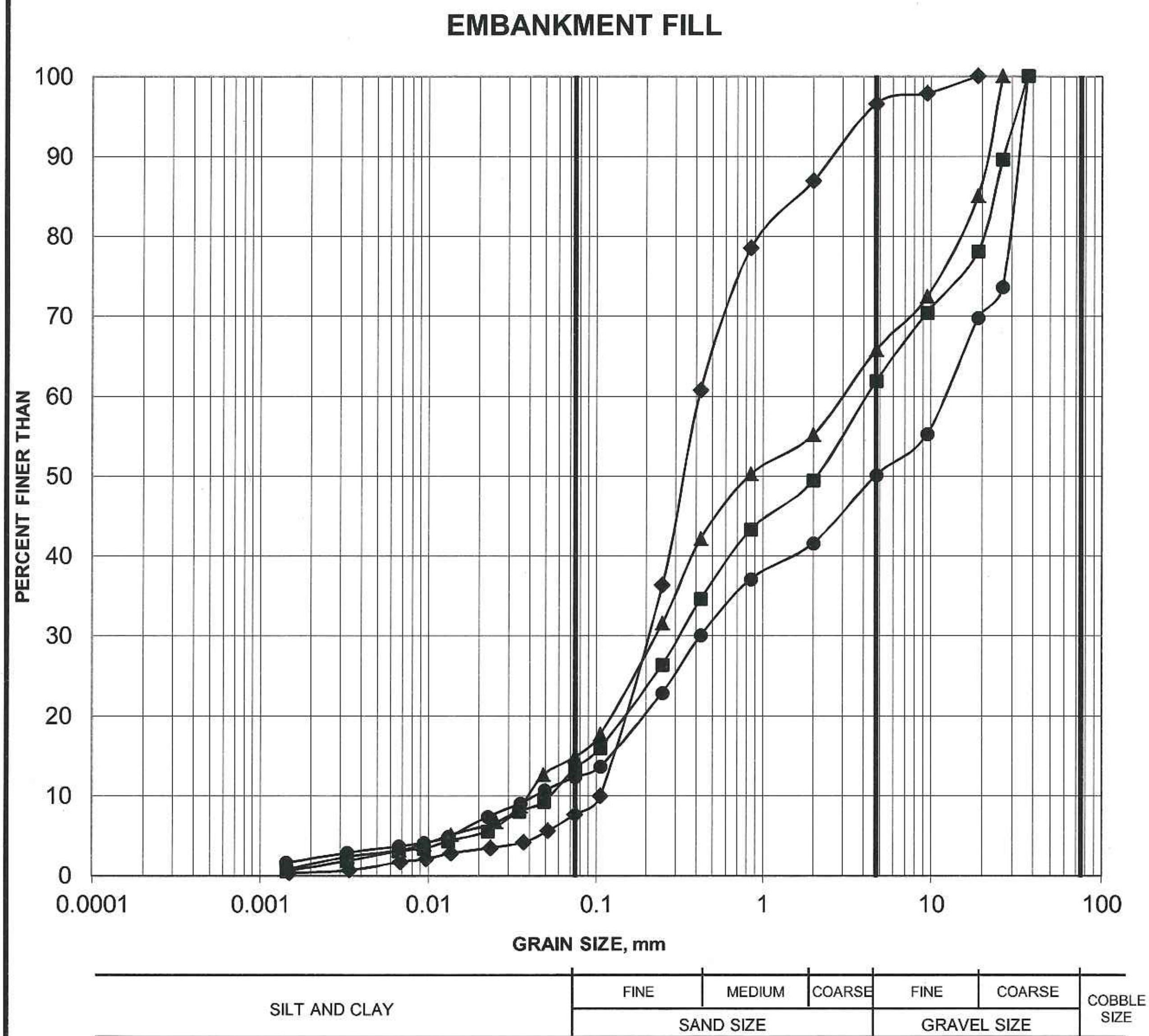
LOGGED: DG

CHECKED: SAT

APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION	FIGURE B1



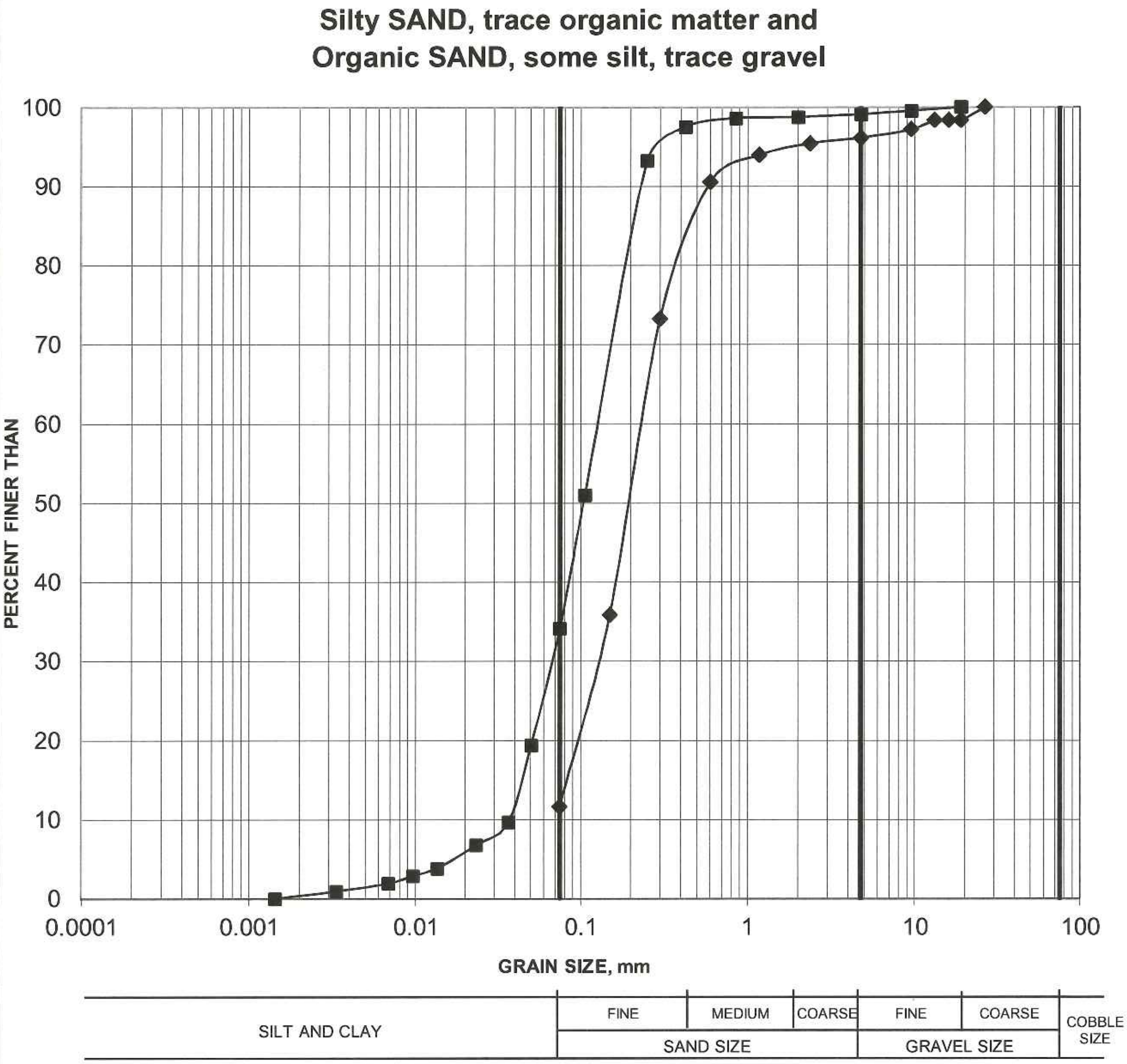
Borehole	Sample	Depth (m)
13-213	1	0.76-1.04
13-213	6	6.40-6.86
13-214	2	0.76-1.24
13-214	5	3.05-3.66

Project: 12-1121-0099/1210 **Golder Associates**

Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION

FIGURE B2



Borehole	Sample	Depth (m)
13-213	9	8.38-8.99
13-215	1 *	0.76-1.37

* - All material retained on the 2.0 mm sieve is organic.

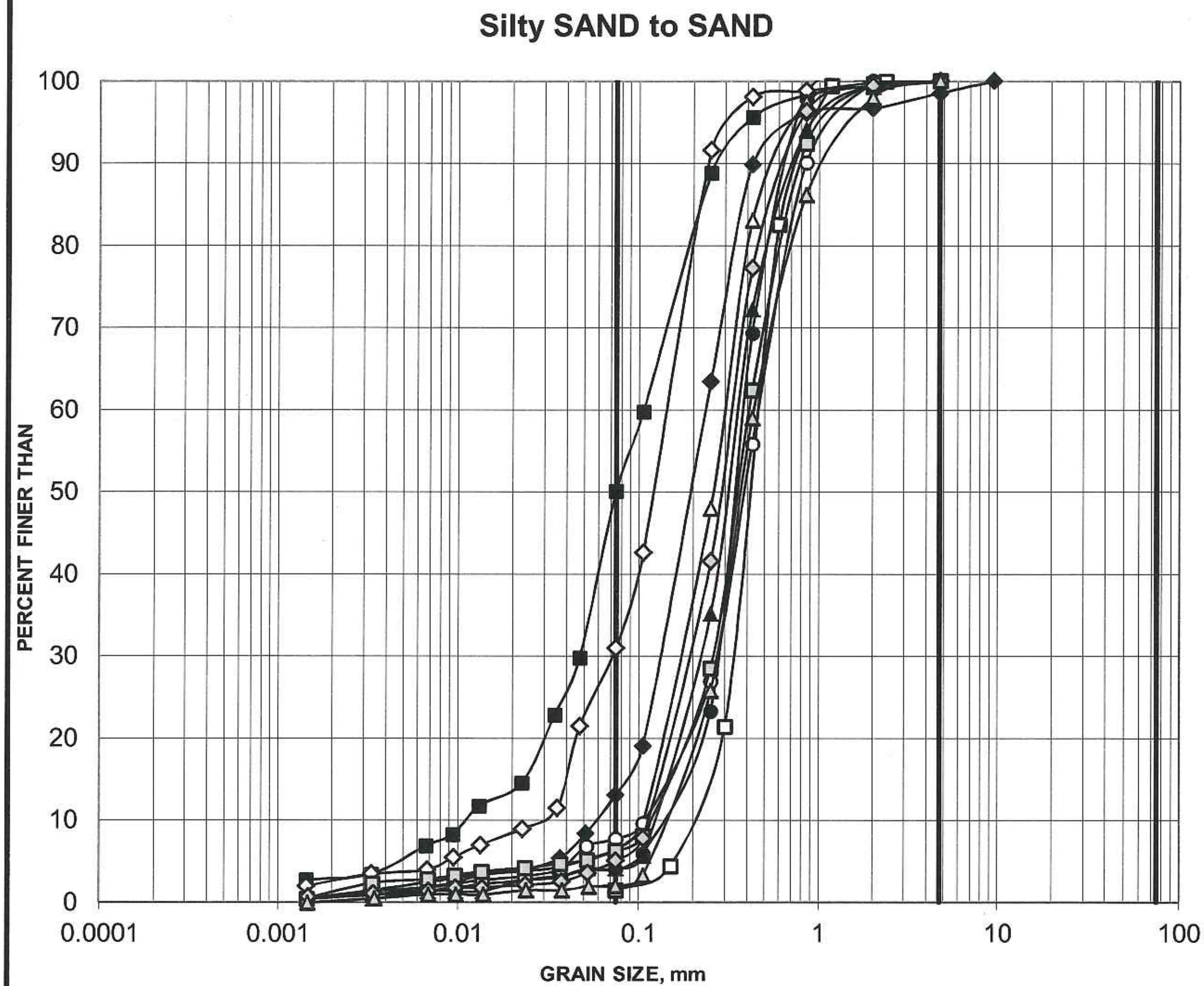
Project: 12-1121-0099/1210

Golder Associates

Created by: MI

Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B3



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

Borehole	Sample	Depth (m)
13-211	1	0.76-1.37
13-211	3	2.29-2.90
13-211	5	3.81-4.42
13-211	7	5.34-5.95
13-211	9	6.86-7.24
13-212	2	0.61-1.22
13-212	5	2.44-3.05
13-212	8	4.27-4.88
13-212	12	6.71-7.32
13-213	14	12.04-12.65
13-215	5	3.81-4.42

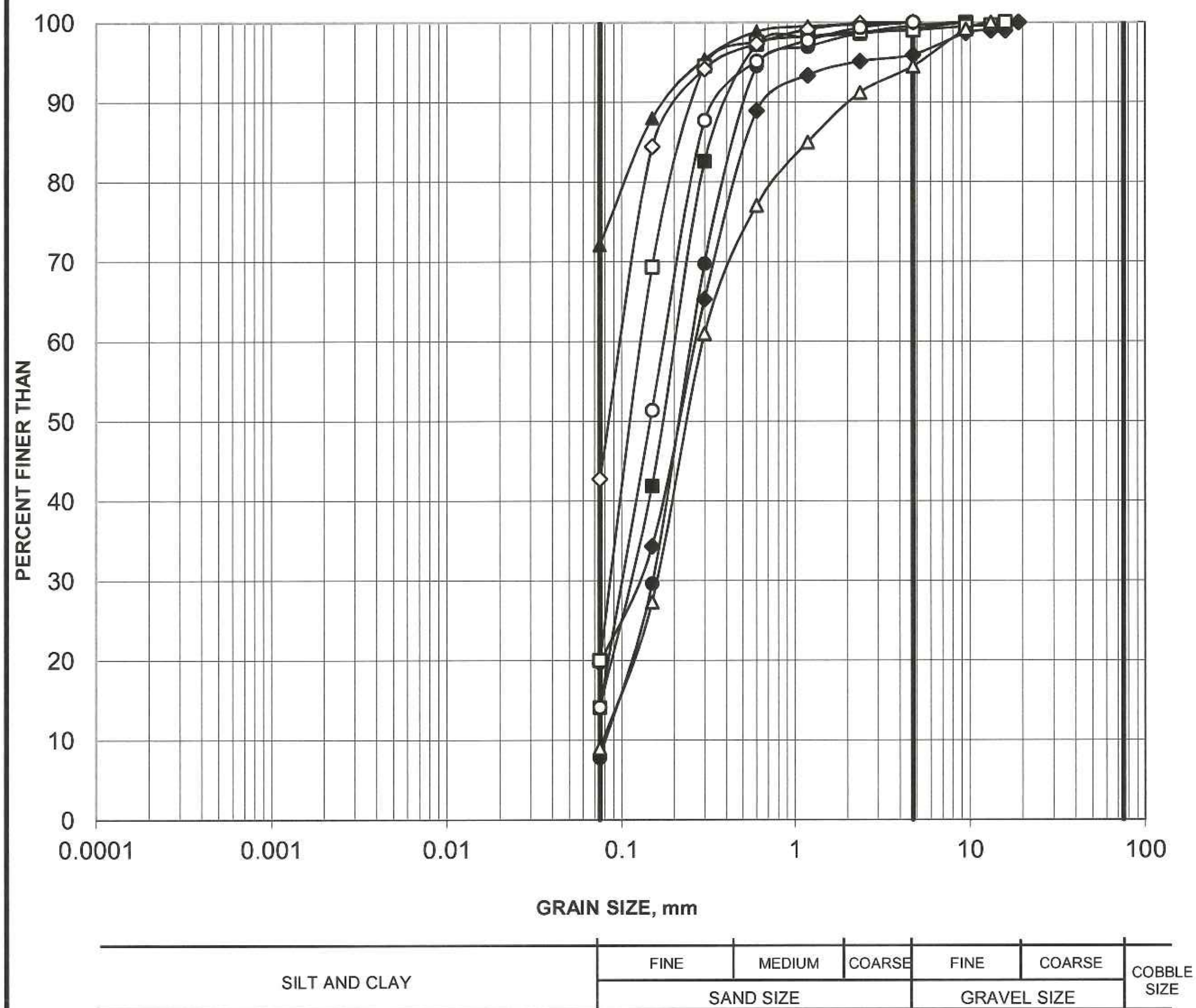
Project: 12-1121-0099/1210

Golder Associates

Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B4

Organic SAND, Silty SAND and SILT



Borehole	Sample	Depth (m)
13-212	17 *	10.67-11.28
13-212	20 *	12.80-13.41
13-212	26 *	16.46-17.07
13-213	21	19.05-19.66
13-214	14	9.91-10.52
13-214	16	11.43-12.04
13-215	8 ŧ	6.10-6.71
13-215	12	8.99-9.60

* - Sample contains pieces of wood and rootlet fibres.

ŧ - All material retained on the 4.75mm sieve is organic.

Project: 12-1121-0099/1210

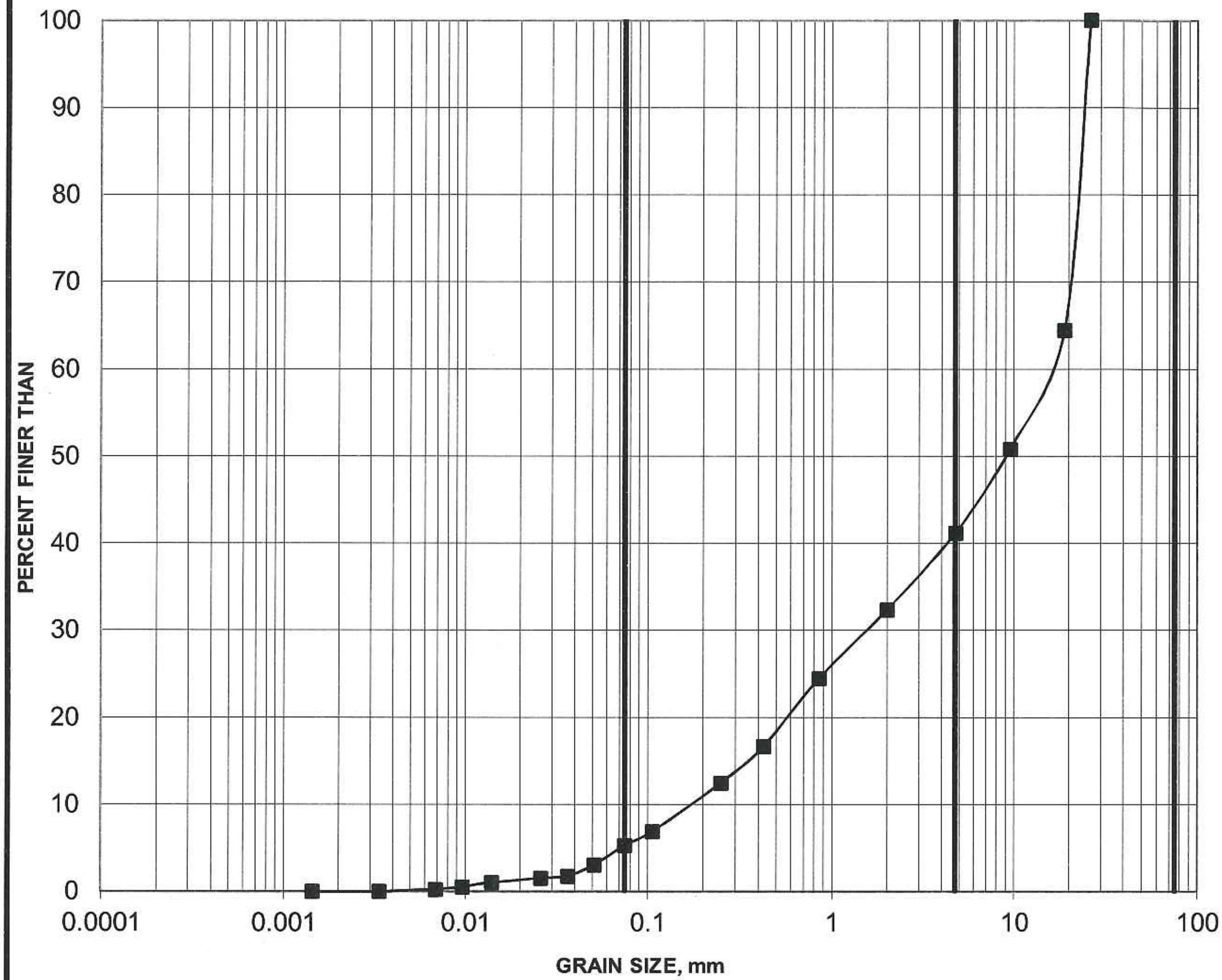
Golder Associates

Created by: MI

Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B5

SAND and GRAVEL



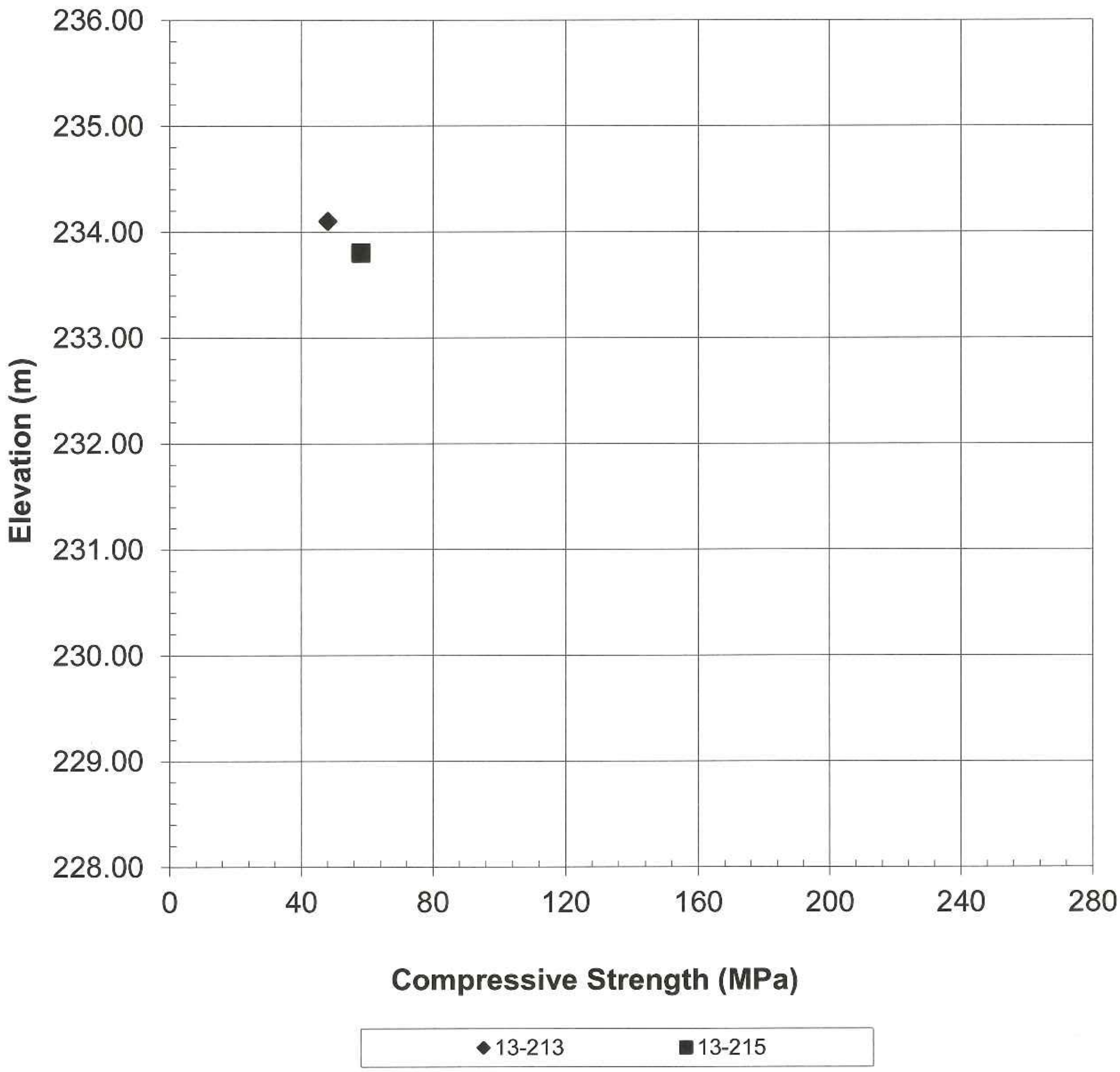
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
13-215	19	14.48-14.81

Project: 12-1121-0099/1210 **Golder Associates**

Created by: MI
Checked by: CNM

SUMMARY OF LABORATORY COMPRESSIVE STRENGTH UNCONFINED COMPRESSION TESTS	FIGURE B6



Project: 12-1121-0099-1210

Golder Associates

Created by:	SAT
Checked by:	FJH