October 2013

REPORT ON

Preliminary Foundation Investigation and Design No Name Creek (Rimington) Culvert Replacement Site No. 11-329c 11.2 km North of Madoc Highway 62, Hastings County, Ontario W.P. 4149-10-01

Submitted to:
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REPORT

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HIGHWAY 62

W.P. 4149-10-01

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin, a member of MMM Group Limited (MRC) 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 foundation investigation conducted for the replacement of the No Name Creek (Rimington) culvert, Site No. 11-329c (WP 4149-10-01) located on Highway 62 about 11.2 km north of Madoc, Ontario.

The purpose of the foundation investigation was to assess the subsurface conditions for the proposed culvert replacement by drilling 4 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 No Name Creek (Rimington) culvert is located on Highway 62 about 11.2 km north of Madoc, Ontario. The existing culvert (Site No. 11-329c) is located at Station 21+325.

The existing culvert is a 3.05 m wide by 1.52 m high double span timber frame structure which is about 21.7 m in length. It is understood that the structure was built in 1957 and is in fair to poor condition. The existing culvert inverts are at about Elevations 229.6 and 229.4 m, at the east and west ends, respectively. The flow in the culvert is from east to west. The depth of water within the culvert was between about 0.1 to 0.2 m at the time of the field investigation. The width of the creek valley is about 8 and 5 m at the inlet and outlet, respectively.

The existing pavement grade at the culvert location is at about Elevation 233 m. In this area, Highway 62 is typically one lane wide in each direction (two-lane highway). However, a speed change lane in the northbound direction is located just north of the culvert site. The existing embankment slopes at the culvert locations are about 2 to 3 m in height and are sloped from about 1.5H:1V.

3.0 INVESTIGATION PROCEDURES

The subsurface investigation was carried out for the culvert replacement between October 26 and November 20, 2012, at which time 4 boreholes (numbered 12-111 to 12-114, inclusive) were advanced at the locations shown on Drawing 1. The boreholes were advanced as follows:

- Boreholes 12-112, 12-113, and 12-114 were advanced using 108 mm inside diameter (I.D.) continuous-flight hollow-stem augers on a truck-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to auger refusal at depths of between about 2.6 and 4.0 m below the existing pavement surface in the overburden. Borehole 12-114 was then cored for 3.2 m into the bedrock using NQ-Size coring equipment.
- Borehole 12-111 was advanced using portable drilling equipment supplied and operated by OGS Inc. of Almonte, Ontario. The borehole was advanced to a depth of 2.1 m below the existing ground surface in the overburden then cored for 3.3 m into the bedrock using NQ-Size coring equipment.

Soil samples in the boreholes were obtained at vertical intervals ranging from 0.60 m to 0.76 m, using a 50-mm outer diameter split-spoon sampler in accordance with Standard Penetration Test procedures.

The water levels in the open boreholes were observed throughout the drilling operations. A standpipe piezometer was installed in Borehole 12-111 to monitor the groundwater level at the site. The standpipe consists of a 32-mm diameter rigid PVC pipe with a 0.9 m long slotted screen section, installed within silica sand backfill and sealed by a section of bentonite pellet backfill. The water level in the standpipe piezometer was measured on November 30, 2012 and July 2, 2013.

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 and water content testing were carried out on selected soil samples at the Ottawa laboratory. Axial point load tests and 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 either determined by Golder Associates in relation to existing site features or surveyed by MRC. The ground surface elevations were surveyed by MRC or determined by MRC from a digital terrain model based on the locations provided by Golder. 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)
12-111	East end of the culvert	4939757.6	223634.0	230.1
12-112	East side of the culvert	4939767.3	223623.4	232.9
12-113	West side of the culvert	4939747.6	223618.9	232.6
12-114	West side of the culvert	4939756.7	223615.1	232.7

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The site is located in the northern portion of the physiographic region known as the Dummer Moraines, and just south of the Georgian Bay fringe, as delineated in The Physiography of Southern Ontario.[1]

The Dummer Moraines is gently sloping southward from about Elevation 244 to 183 m, and is characterized by relatively shallow deposits of glacial till overlying bedrock. The underlying bedrock is typically limestone of the Bobcaygeon and Gull River Formation; however, there is also Precambrian bedrock in the area.[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 B4 contained in Appendix B.

A soil stratigraphy section projected along the centreline of the existing culvert area 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 culvert replacement consist of embankment fill below the roadway and a layered sand and silt deposit and/or glacial till overlying bedrock near the inlet.

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 encountered within the northbound speed change lane of Highway 62 at Borehole 12-112 consists of about 0.1 m of asphaltic concrete overlying 0.3 m of sand and gravel base. The pavement structure encountered within the southbound shoulder at Borehole 12-113 consists of about 0.4 m of sand and gravel base over about 0.4 m of sand subbase. At Borehole 12-114 the pavement structure in the southbound shoulder consists of about 0.8 m of sand and gravel base.

The pavement structure at all of these locations is underlain by embankment fill, which was fully penetrated to depths between about 2.6 and 4.0 m (Elevations 230.0 and 228.9 m, respectively). The embankment fill is between about 1.8 and 3.6 m thick.

The embankment fill generally consists of sand and gravel containing silt and crushed stone. Cobbles also exist within the fill, as do trace amounts of organic matter and weathered bedrock at depth.

Standard Penetration Test (SPT) N values for the embankment fill range from 'weight of hammer' to 33 blows per 0.3 m of penetration, indicating a very loose to compact state of packing.

[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.

The results of grain size distribution testing carried out on samples of the embankment fill are provided on Figure B1 in Appendix B. The embankment fill in the boreholes in the southbound lanes are coarser than the boreholes in the northbound lane. However, the results do not reflect the cobble or full gravel content of the material, since the samples were retrieved using a 50 mm outside diameter sampler.

The measured water content of the fill ranges from approximately 6 to 32 percent where organic matter exists.

4.2.2 Sandy Silt and Gravel and Sand

A layered deposit of sandy silt and gravel and sand was encountered at ground surface at Borehole 12-111. The deposit was fully penetrated to a depth of 1.8 m (Elevation 228.3 m).

Three SPT N values of 3, 6 and 16 blows per 0.3 m of penetration were measured in this deposit indicating a very loose to compact state of packing.

The results of grain size distribution testing on two samples of the deposit are shown on Figure B2 in Appendix B.

The measured natural water contents of three samples of the deposit were about 17, 28 and 34 percent, with the higher values resulting from organic matter.

4.2.3 Gravel Till

The layered deposit at Borehole 12-111 is underlain by a thin deposit of gravel till. The till was fully penetrated to a depth of 2.1 m below the existing ground surface (Elevation 228.0 m) and is about 0.3 m thick.

The glacial till is considered to be a heterogeneous mixture of gravel and cobbles in a matrix of silt and sand. Refusal to advancement of the sampler was encountered on cobbles in the deposit and rotary diamond drilling/coring techniques were required to advance the borehole within the till.

4.2.4 Refusal and Bedrock

Auger refusal was encountered at Elevations 228.9 and 230.0 m at Boreholes 12-112 and 12-113 respectively, which has been inferred to represent the bedrock surface.

Bedrock was encountered beneath the embankment fill at Borehole 12-114 and beneath the till deposit at Borehole 12-111 where it was cored for about 3.2 m and 3.3 m, respectively.

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

Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
12-111	230.1	2.1	228.0
12-112	232.9	*4.0	*228.9
12-113	232.6	*2.6	*230.0
12-114	232.7	2.7	230.0

Note: * Depth and elevation to bedrock inferred from auger refusal.

The bedrock encountered in the cored boreholes consists of grey to greenish black marble on the east side of the culvert and dark green to black granite schist on the west side of the culvert. The bedrock is slightly weathered to fresh and strong to very strong.

The Rock Quality Designation (RQD) values measured on recovered bedrock core samples ranged from about 78 to 100 percent, indicating a good to excellent quality rock. However, one lower RQD value of 15 percent was measured within the upper portion of the bedrock at Borehole 12-114 indicating a very poor quality rock. The discontinuities observed in the rock core were associated with the bedding, joints and fractures of the bedrock. A zone of bedrock with a fracture index of greater than 5 fractures for 0.3 m of core was encountered in the upper portion of the bedrock at Borehole 12-114.

Laboratory axial point load index testing as well as unconfined compressive strength testing was carried out on selected specimens of the bedrock core. The results of the testing are summarized on Figures B3 and B4 in Appendix B. The compressive strengths from the point load index testing for the marble bedrock at Borehole 12-111 range from about 62 to 111 MPa. The results of the unconfined compressive strength test carried out on the marble indicate a value of about 55 MPa. The compressive strengths from the point load index testing for the granite schist at Borehole 12-114 range from about 138 to 273 MPa. The results of the unconfined compressive strength testing on one sample of the granite schist bedrock indicate a value of 53 MPa.

4.2.5 Groundwater Conditions

The groundwater levels measured in the piezometer in Borehole 12-111 are summarized in the table below:

Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date
12-111	230.1	0.1	230.0	November 30, 2012
		0.2	229.9	July 2, 2013

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. 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

NO NAME CREEK (RIMINGTON) CULVERT REPLACEMENT

SITE 11-329C

11.2 KM NORTH OF MADOC

HIGHWAY 62

W.P. 4149-10-01

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 No Name Creek (Rimington) culvert on Highway 62. 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 replacement of the culvert will be along the existing highway and culvert alignment as shown on Drawing 1. The proposed invert level of the existing culvert will be maintained. However, a grade raise of about 0.8 m is planned for the future design of the roadway to meet the current design standards. The Highway 62 embankment will be widened temporarily by about 1 and 4 m on the east and west shoulders, respectively, to accommodate the construction staging of the culvert.

6.2 Foundation Options

The existing No Name Creek (Rimington) culvert is a double-span timber frame structure that was built in 1957. The existing foundation consists of a timber slab likely founded on the native soils or bedrock. The existing culvert inverts are at about Elevations 229.6 and 229.4 m at the east and west ends, respectively. The flow in the culvert is from east to west. The depth of water within the culvert was between about 0.1 and 0.2 m at the time of the field investigation. The width of the creek valley is about 8 and 5 m at the inlet and outlet, respectively.

The existing pavement grade at the culvert location is at about Elevation 233 m. In this area, Highway 62 is typically one lane wide in each direction (i.e., a two-lane highway). However, a speed change lane in the northbound direction is located just north of the culvert site. The existing embankment slopes at the culvert locations are about 2 to 3 m in height and are sloped at about 1.5H:1V and appear stable.

Based on the subsurface conditions, only shallow foundation options have been considered for the replacement of the existing No Name Creek culvert. Deep foundations are not required or recommended as shallow foundations will provide sufficient bearing resistance and acceptable settlement performance for the proposed culvert replacement.

A summary of the advantages and disadvantages associated with each shallow 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.

- **Precast Concrete Box Culvert Founded on the Native Soils/Granular Pad on the Bedrock:** A box culvert could be considered for the culvert replacement provided it is founded on or within the native compact sandy silt and gravel, till or bedrock. The box culvert should be founded below the native soils which contained organic matter (i.e., below Elevation 228.9 m near the culvert inlet). However, if the culvert invert level is maintained, some bedrock removal would likely be required along the west side of the culvert for a box structure, which may require drill and blast procedures given the strong to very strong granite schist at this site. Relatively lower geotechnical resistances will also apply for the native soils or pad of granular fill on the bedrock as opposed to founding on the bedrock itself, with potential for some minimal settlement of the culvert. It is expected that temporary protection systems and/or cofferdams would be required during excavation and construction. A precast culvert would be preferred over a cast-in-place culvert for this option because it would likely be easier and quicker to install, and require less construction time and, therefore, less disruption to traffic.
- **Cast-In-Place/Precast Rigid Frame Open Footing Culvert Founded on the Bedrock:** Given the limited thickness of the overburden at this site and the shallow depth to bedrock, it would likely be more feasible to replace the culvert with a rigid frame open footing culvert founded on the marble and granite schist bedrock. Higher geotechnical resistances will apply for foundations on the bedrock with negligible settlement of the footings. The footings can also be founded on the surface of the bedrock so no bedrock removal would be required. As above, it is expected that a temporary protection system and/or cofferdams would be required during excavation and construction. A cast-in-place culvert would be preferred over a precast culvert for this option because of the variable and shallow depth to bedrock. The footings could be formed and poured directly on the bedrock surface without the need for bedrock removal or a granular bedding layer. A precast open footing culvert supported on cast-in-place variable height footings could also be considered as a viable option for this site.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to replace the culvert with a concrete rigid frame open footing culvert founded on the bedrock.

6.3 Culvert Foundation Options

6.3.1 Precast Concrete Box Culvert

6.3.1.1 Founding Level and Bedding

It is not necessary to found the box culvert at the standard depth for frost protection purposes as box structures are tolerant of small magnitude movements related to freeze-thaw cycles should these occur. The box culvert should, however, be founded below any existing fill and surficial soils containing organic matter.

The bedding and/or leveling pad requirements for a box culvert replacement should be in accordance with Ontario Provincial Standard Specification (OPSS) 422 *(Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut)* for precast concrete box culverts. It is recommended that the box culvert segments be placed on a minimum thickness of 300 mm of granular bedding material meeting OPSS 1010 Granular A or Granular B Type II.

The table below summarizes the recommended founding level for the culvert, assuming a base slab thickness of 300 mm and bedding thickness as described above as well as the depth of native material containing organic matter. Based on these elevations, the box culvert replacement will be typically founded on the compact sandy silt and gravel on the southeast side of the culvert, and a pad of granular fill on the bedrock on the west and northeast sides of the culvert. Based on the borehole results, approximately 1.2 m of bedrock removal would be required on the west side of the culvert to maintain the subgrade level.

Invert Location	Existing Invert Elevation (m)	Box Culvert Founding Elevation (m)	Subgrade Level (m)
East End	229.6	229.3	228.9*
West End	229.4	229.1	228.8

Note: * The elevation of the compact sandy silt and gravel (i.e., below the depth of native soil containing organic matter).

The footing subgrade should be inspected in accordance with OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*).

6.3.1.2 Geotechnical Resistances

For a box culvert founded at the elevations provided in Section 6.3.1.1 and with a span of up to 5.6 m, founded within the compact sandy silt and gravel, a factored geotechnical resistance at Ultimate Limit States (ULS) of 400 kPa and a geotechnical resistance at Serviceability Limit States (SLS, for 25 mm of settlement) of 300 kPa may be used for design purposes. For the section of the box culvert founded on a granular pad on the bedrock, a factored geotechnical resistance at ULS of 500 kPa and an SLS resistance of 300 kPa may be used for design purposes.

These preliminary geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its *Commentary*.

The preliminary geotechnical resistance values provided above will have to be re-evaluated and modified as necessary during detail design.

6.3.2 Cast-in-Place/Precast Rigid Frame Open Footing Culvert Founded on Bedrock

6.3.2.1 Founding Level and Frost Protection Requirements

Strip footings for an open footing culvert replacement, and for any associated concrete wing walls/retaining walls should be founded on the marble or granite schist bedrock.

As per Ontario Provincial Standard Drawing (OPSD) 3090.101 *(Foundation Frost Depths for Southern Ontario)* frost penetration depth in the area is 1.6 m. Therefore, the footings should be provided with a minimum of 1.6 m of earth cover to provide adequate protection against frost penetration. However, this requirement can be waived where the founding level on the bedrock is above the frost depth as the bedrock at this site does not appear to contain any seams of frost susceptible soil.

The table below summarizes the recommended founding level for the proposed open footing replacement culvert based on the bedrock elevations.

Invert Location	Existing Invert Elevation (m)	Open Footing Culvert Founding Elevation (m)
East End	229.6	228.0/228.9*
West End	229.4	230.0**

Notes: * Southeast/northeast sides of the culvert.
** Bedrock elevation above the invert level of the culvert.

The footing subgrade should be inspected in accordance with OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*).

6.3.2.2 Geotechnical Resistance

For footings founded on the bedrock at the elevations provided in Section 6.3.2.1, a factored geotechnical resistance at ULS of 5 MPa may be used for design purposes. SLS resistances do not apply to the design of footings on the marble and granite schist bedrock, because the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

These preliminary geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its *Commentary*.

The preliminary geotechnical resistance values provided above will have to be re-evaluated and modified as necessary during detail design.

6.4 Settlement

It is understood that temporary widenings of about 1 m at the shoulder of the east side of the embankment and about 4 m at the shoulder of the west side of the embankment are required to facilitate the construction staging for the proposed culvert replacement. It is also understood that a grade raise of about 0.8 m is planned in the future for the roadway. The replacement culvert will either be founded on the native sandy silt and gravel soil or on the bedrock. The anticipated total and differential settlements of a box culvert replacement founded on the native soils should be minimal (i.e., less than about 25 and 15 mm, respectively) even with the proposed widening and future grade raise. For an open footing culvert replacement, the footings will be founded on the bedrock; therefore, settlements of the culvert foundations should be negligible even with the proposed widening and future grade raise.

6.5 Culvert Backfill and Erosion Protection

Backfill, cover and construction of the frost taper (backfill transition) for concrete culverts should be completed in accordance with OPSS 902 *(Construction Specification for Excavating and Backfilling - Structures)* and/or OPSD 803.010 *(Backfill and Cover for Concrete Culverts)*. Where frost tapers cannot be accommodated, consideration could be given to the use of high density insulation; however, the details for the transition would be complicated and likely need to extend to greater lengths and depths than if typical frost tapers were used.

Backfill to culvert walls should consist of granular fill meeting the requirements of OPSS 1010 Granular A or Granular B Type II, but with less than 5 per cent passing the No. 200 sieve. The backfill should be placed and compacted in accordance with MTO's Special Provision SP105S21 *(Amendment to OPSS 501).* The fill depth during placement should be maintained equal on both sides of the culvert walls, with one side not exceeding the other by more than 500 mm. The culvert should be designed for the full overburden pressure and live load assuming that the embankment fill has a unit weight of 22 kN/m^3 for Granular A and 21 kN/m^3 for Granular B Type II or select earth fill above and/or surrounding the culvert.

For the box culvert option, where the culvert is founded on the native soils or pad of granular fill on the bedrock, a clay seal or concrete cut-off wall should be provided at the upstream end of the culvert replacement to prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles). Where the culvert is founded on the bedrock (i.e., for the rigid frame open footing culvert option) it is not considered necessary to provide a clay seal or concrete cut-off wall at the upstream end of the culvert.

If gabion baskets or other porous media are adopted for retaining walls adjacent to the replacement culvert, a clay seal or geotextile should also be provided behind the porous gabion baskets to protect the native soils from erosion and scour, and to minimize loss of fine soil particles through voids in the retaining structure.

If the flow velocities are sufficiently high, a provision should be made for scour and erosion protection (suitable non-woven geotextiles and/or rip-rap) at the culvert inlet and outlet. The requirements for and design of erosion protection measures for the culvert inlet should be assessed by the hydraulic design engineer. As a minimum, rip-rap treatment for the culvert outlet should be consistent with the standard Treatment Type A presented in OPSD 810.010 *(Rip-Rap Treatment for Sewer and Culvert Outlets),* with the rip-rap placed up to the toe of slope level, in combination with the cut-off measures noted above.

6.6 Embankment Construction and Stability

It is understood that temporary widenings of about 1 m at the shoulder of the east side of the embankment and about 4 m at the shoulder and up to about 2 m at the toe of the west side of the embankment are required to facilitate the construction staging for the proposed culvert replacement. The widening of the embankment will be about 3 m in height relative to the original ground surface and sloped at about 1.25H:1V for rock fill or 2H:1V for granular fill. Where the toe of the embankment is to be widened on the west side of the culvert, the subsurface conditions are expected to consist of shallow bedrock. However, if encountered, any topsoil, organic matter or softened/loosened soils should be stripped from below the embankment areas.

The fill for the embankment widening areas adjacent to the culvert should be placed and compacted in accordance with MTO's Special Provisions 206S03 and 105S10. Benching of the existing embankment side slopes should be carried out to "key in" the new fill materials for the widening, in accordance with OPSD 208.010. Commonly in embankment widening construction, the fill material cut from the existing embankment side slope for creation of these benches is re-used for the embankment widening below/adjacent to each bench area. Additional fill for construction of the embankment widening above the level of the original ground surface (i.e., above the groundwater level) could consist of clean earth fill, granular fill or rock fill.

Following removal of the fill for the temporary widening, replacement of topsoil and seeding or pegged sod is recommended to reduce surface water erosion on the newly exposed embankment side slopes.

For the soil conditions at the culvert and the embankment height, the embankment will have an adequate factor of safety against both static and seismic slope instability (i.e., greater than 1.3 under static conditions, and 1.1 under seismic conditions).

Settlement of the embankments will occur as a result of compression of the new embankment fill. Provided that the embankment material consists of Select Subgrade Material or clean earth fill, the settlement of the embankment fill itself is expected to be less than 25 mm. The use of granular fill for the new embankment construction would reduce this magnitude of post-construction settlement (likely to less than half that value) since the majority of settlement of these fills will occur during construction.

Where rock fill is used, settlement of the rock fill itself will depend on the type of rock and on the method and sequence of placement and compaction of the fill. Assuming that the rock fill is placed in accordance with the requirements outlined in the SP206S03, the settlement of rock fill in embankments is estimated to be about 1 percent of the embankment height and it is anticipated that the majority of this settlement will occur during the first year following construction.

6.7 Construction Considerations

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

6.7.1 Groundwater and Surface Water Control

Control of the surface water and groundwater will be necessary for the construction of the culvert replacement, to allow excavation and foundation construction to be carried out in dry conditions.

Some groundwater inflow into the excavations should be expected, particularly on the east side of the culvert where the soils consist of sandy silt and gravel or till. It should be possible to handle the groundwater inflow by pumping from well-filtered sumps established in the floor of the excavations, provided that an appropriate cut-off/cofferdam is in place between the culvert foundation excavations and the creek if required. Alternatively, depending on the flow at the time of construction, the surface water flow could be passed through the culvert area by means of a temporary pipe, or diverted by pumping from behind a temporary cofferdam.

Surface water should be directed away from the excavation area, to prevent ponding of water that could result in disturbance and weakening of the foundation subgrade; further discussion on this aspect is provided in Section 6.7.3.

6.7.2 Excavation and Temporary Protection Systems

Temporary excavations for the culvert, up to a depth of about 5 m, will be made through the existing fill, sandy silt and gravel, sand and till. 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 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 fill material, sandy silt and gravel, sand and till 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.

If the above open cut excavation side slopes cannot be accommodated, then temporary protection systems (i.e., temporary excavation shoring) will be required. Where shoring is required, the support system should be designed and constructed in accordance with OPSS 539 *(Temporary Protection Systems)*. The lateral movement of the temporary protection system should meet Performance Level 2 as specified in OPSS 539, provided that any utilities that may be present in the area can tolerate this magnitude of deformation.

It is considered that either a driven, interlocking sheetpile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support at this site, based on the subsurface soil and groundwater conditions. An interlocking sheetpile system would contribute to both ground and groundwater control. For a soldier pile and lagging system, it would be necessary to control seepage or include measures to mitigate loss of soil particles through the lagging boards. The soldier piling and lagging or interlocking steel sheetpiling would be supported against lateral movement using walers, tie backs (into the bedrock) and/or internal struts/braces or socketing into the bedrock.

6.7.3 Subgrade Protection

All embankment fill, topsoil, organics and soft or loose soils should be removed from below the proposed founding elevations and wasted or reused as landscaping fill, as required. Subgrade preparation should be performed and monitored in accordance with OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*). The cleaned excavation base should be inspected prior to pouring the footings for the rigid frame open footing culvert or granular bedding for the box culvert.

6.7.4 Obstructions

Cobbles, which could affect the installation of the protection systems, were encountered in the embankment fill at Boreholes 12-112, 12-113 and 12-114 and the till deposit at Borehole 12-111. Further observation of the presence of cobbles is recommended in the next stage of investigation in support of the detail design.

6.8 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 construction risks. However, at this functional 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:

- Assessment of the variability of any existing fill, surficial soils and bedrock to confirm the founding elevations within the culvert area.
- Observation of the presence of cobbles within the soil deposits, as the presence of such obstructions may affect excavations and the installation of elements of temporary protection systems.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Ms. Susan Trickey, P.Eng. and 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.

GOLDER ASSOCIATES LTD.

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31 OCT 2013
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WAM/SAT/LCC/FJH/bg

n:\active\2012\1121 - geotechnical\12-1121-0099 mrc 22 structures eastern region\foundations\6 - reports\package 1\11-329c no name\12-1121-0099-1110 rpt-001 no name culvert final october 2013.docx

Table 1 – Comparison of Foundation Alternatives

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Relative Costs
Precast box culvert founded on the native soils and a granular pad on the bedrock	▪ Feasible	▪ Shallower excavation depths ▪ Minimal settlement	▪ Lower geotechnical resistances for the box culvert as opposed to founding on bedrock ▪ Would require bedrock removal along the west side of the culvert using drill and blast procedures, which could slow the rate of construction ▪ Groundwater control and temporary protection system required	▪ Conventional excavation and construction techniques and temporary protection system	▪ Higher cost
Cast-in-place or precast open footing culvert founded on the bedrock surface	▪ Feasible, preferred option from foundations perspective	▪ Higher geotechnical resistances as opposed to founding on the native soil ▪ Negligible settlement ▪ Footing can be poured directly on the bedrock surface without any bedrock removal, resulting in a higher rate of construction ▪ Less risk associated with rock excavation and variable rock quantities	▪ Groundwater control and temporary protection system required	▪ Conventional excavation and construction techniques and temporary protection system	▪ Moderate cost

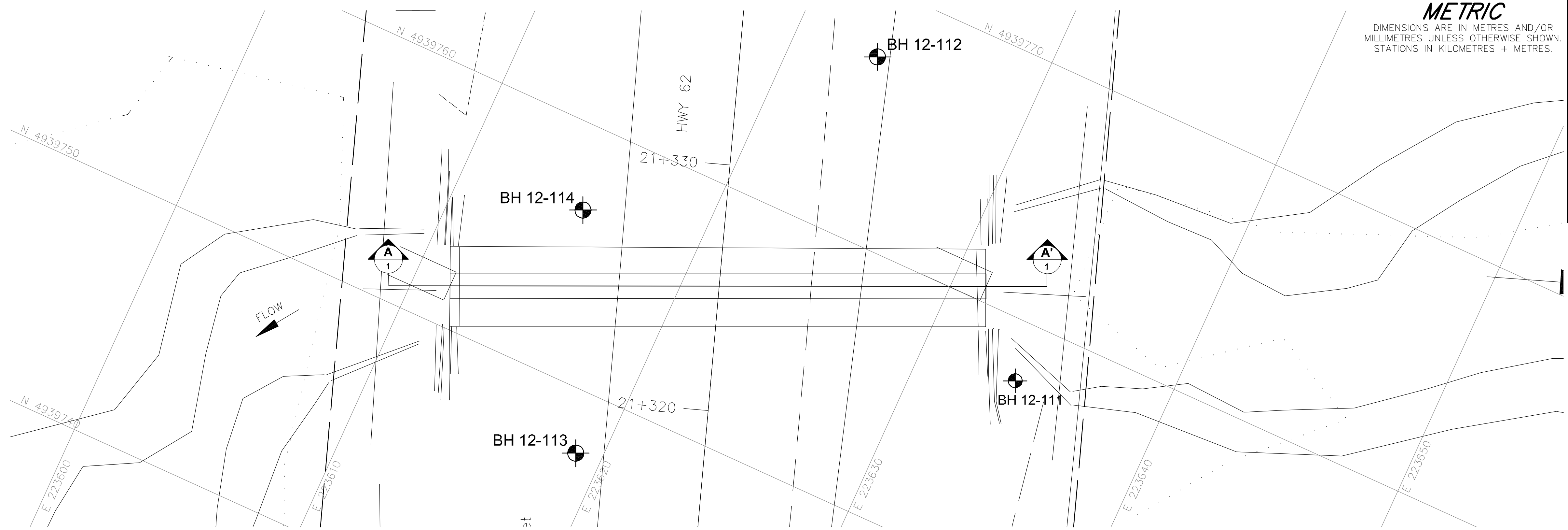
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 4149-10-01

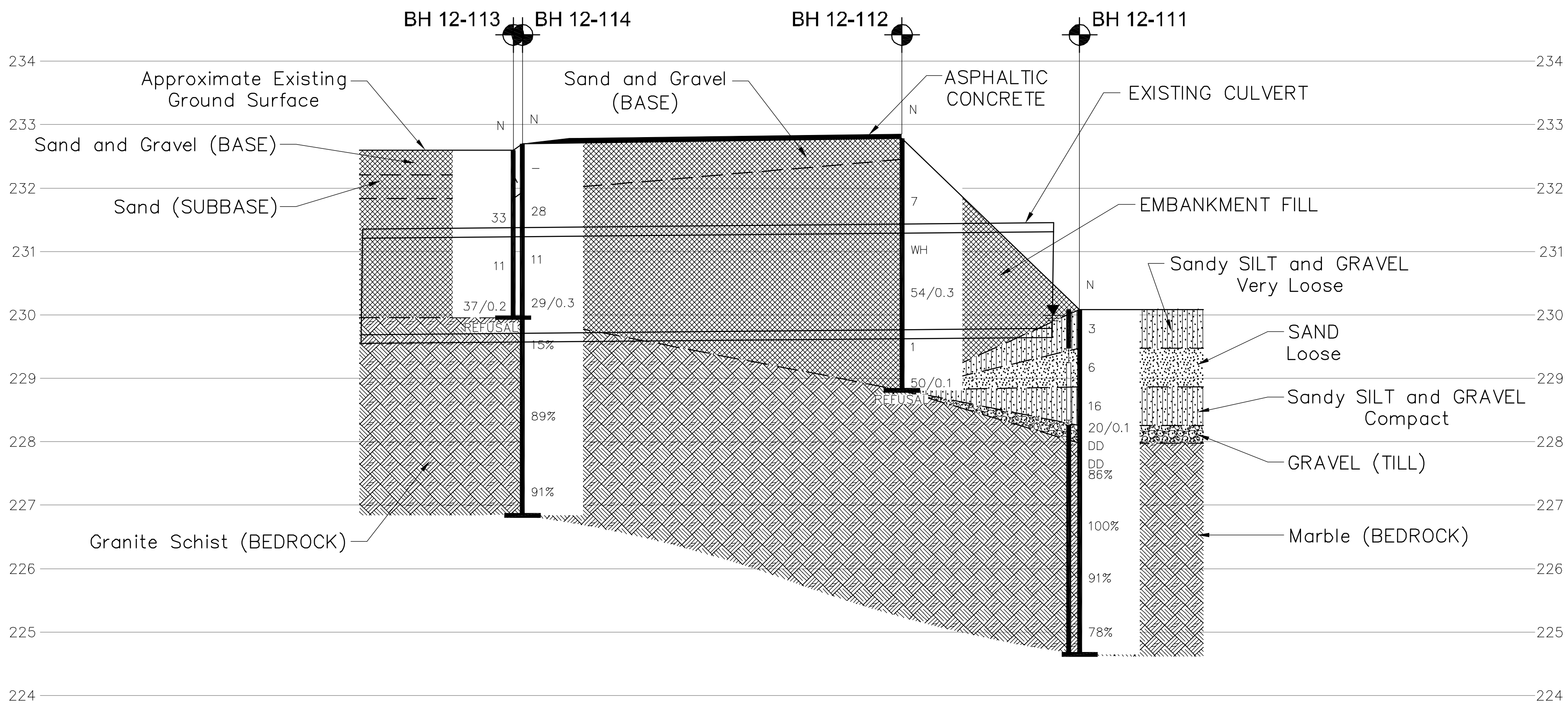
SHEET

HIGHWAY 62-CULVERT REPLACEMENT
NO NAME CREEK (RIMINGTON)
SITE 11-329C
BOREHOLE LOCATIONS AND SOIL STRATA

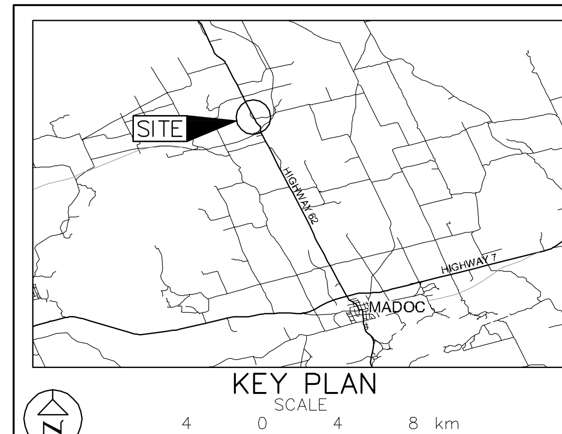
Golder Associates
Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



PLAN
SCALE
2 0 2 4 m



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



KEY PLAN
SCALE
4 0 4 8 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 upon completion of or during drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
12-111	230.1	4939757.6	223634.0
12-112	232.9	4939767.3	223623.5
12-113	232.6	4939747.6	223618.9
12-114	232.7	4939756.7	223615.1

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the 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 11-329-121129.dwg, received November 30, 2012.

L. C. DOYLE 31 Oct 2013 PROVINCE OF ONTARIO LICENSED PROFESSIONAL ENGINEER

F. J. HEFFERNAN 31 OCT 2013 PROVINCE OF ONTARIO LICENSED PROFESSIONAL ENGINEER

NO.	DATE	BY	REVISION

Geocres No. 31C-220			
HWY. 62		PROJECT NO. 12-1121-0099	DIST. EASTERN
SUBM'D. SAT	CHKD. FJH	DATE: Sept. 2013	SITE: 11-329C
DRAWN: JM	CHKD. SAT	APPD. FJH	DWG. 1

APPENDIX A

Borehole and Drillhole Records

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

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

IV. SOIL TESTS

w	Water content
w_p or PL	Plastic limited
w_1 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_1 or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity Index = $(w_1 - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_c	consistency index = $(w_1 - w) / I_p$
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 = $\tan\delta$
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		

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RECORD OF BOREHOLE No 12-111

SHEET 1 OF 2

METRIC

PROJECT 12-1121-0099-1110

G.W.P. 4149-10-01

DIST Eastern HWY 62

DATUM Geodetic

LOCATION N 4939757.6 ;E 223634.0

BOREHOLE TYPE Portable Drill, NW/BW Casing

DATE November 19-20, 2012

ORIGINATED BY DWM

COMPILED BY JM

CHECKED BY SAT

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 25 50 75

UNIT WEIGHT γ kN/m^3

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
230.1	GROUND SURFACE					
0.0	Sandy SILT and GRAVEL, trace clay, containing organic matter; Very loose; Grey-brown; Wet	1	SS	3	230	34 28 32 6
229.5 / 0.6	SAND, some silt, trace to some gravel, containing organic matter; Loose; Grey; Wet	2	SS	6	229	
228.9 / 1.2	Sandy SILT and GRAVEL, trace clay; Compact; Grey; Wet	3	SS	16		34 25 37 4
228.3 / 1.8	GRAVEL, some silt and sand, containing cobbles (TILL); Grey; Wet	4	SS	20/0.1		
		5	RC	DD		
228.0 / 2.1	Marble (BEDROCK); Bedrock cored from depths of 2.1 m to 5.4 m; Ford bedrock coring details refer to Record of Drillhole 12-111	6	RC	DD	228	
		1	RC	REC 97%		RQD = 86%
		2	RC	REC 100%	227	RQD = 100%
		3	RC	REC 100%	226	RQD = 91%
		4	RC	REC 100%	225	RQD = 78%
224.7 / 5.4	END OF BOREHOLE					

NOTES:

1. Water level in well screen at a depth of 0.2 m below ground surface (Elev. 229.9 m), measured on July 2, 2013.

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

RECORD OF DRILLHOLE: 12-111

PROJECT: 12-1121-0099-1110
LOCATION: N 4939757.6 ;E 223634.0
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 19-20, 2012
DRILL RIG: Portable Drill
DRILLING CONTRACTOR: OGS

DATUM: Geodetic

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
		Continued from Record of Borehole 12-111	227.98			
	Portable Drill NQ Core	Marble (BEDROCK) Fresh Grey-greenish black Foliated Fine to medium grained Strong to very strong	2.11	1	100	JN,PL,SM,He FR,ST,SM
3				2	100	JN,CU,SM
4		Healed fracture from 4.3 m to 4.4 m depth		3	100-0	BD,CU,SM JN,CU,SM JN,CU,RO,Fe BD,PL,SM BD,CU,SM
5		Healed fracture from 4.9 m to 5.1 m depth	224.65	4	0	FR,ST,SM JN,CU,SM JN,CU,SM
		END OF DRILLHOLE	5.44			
6						
7						
8						
9						
10						
11						
12						

Other column headers: SYMBOLIC LOG; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER; DIP w.r.t. CORE AXIS; Jr; Ja; HYDRAULIC CONDUCTIVITY K, cm/sec; WEATHERING INDEX (W1–W6); FEATURES; NOTES.

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: DWM
CHECKED: SAT

RECORD OF BOREHOLE No 12-112

SHEET 1 OF 1

METRIC

PROJECT 12-1121-0099-1110
G.W.P. 4149-10-01
LOCATION N 4939767.3 ;E 223623.4
ORIGINATED BY DWM
DIST Eastern HWY 62
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)
COMPILED BY JM
DATUM Geodetic
DATE October 31, 2012
CHECKED BY SAT

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
232.9	GROUND SURFACE									
0.0	ASPHALTIC CONCRETE									
0.1	Sand and gravel, some silt (BASE) Brown									
232.5 / 0.4	Silty sand to sand, trace gravel, containing organic matter (EMBANKMENT FILL) Loose Brown Wet	1	SS	7	232					
231.4 / 1.5	Sandy gravel, some silt, containing cobbles (EMBANKMENT FILL) Very loose Brown Moist to wet	2	SS	WH	231					
		3	SS	54/0.3		○	58	26	14	2
229.9 / 3.1	Sandy silt, trace gravel, containing organic matter (EMBANKMENT FILL) Very loose Brown Wet	4	SS	1	230	○				
229.1	Silty sand and gravel (EMBANKMENT FILL) Very loose Brown Wet	5	SS	50/0.1	229	○				
228.9 / 4.0	END OF BOREHOLE AUGER REFUSAL									

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

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

Golder Associates

RECORD OF BOREHOLE No 12-113

SHEET 1 OF 1 **METRIC**

PROJECT 12-1121-0099-1110

G.W.P. 4149-10-01 LOCATION N 4939747.6 ;E 223618.9 ORIGINATED BY DWM

DIST Eastern HWY 62 BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem) COMPILED BY JM

DATUM Geodetic DATE October 31, 2012 CHECKED BY SAT

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	W (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
232.6	GROUND SURFACE							
0.0	Sand and gravel, some silt (BASE) Brown Dry							
232.2 / 0.4	Sand, some silt, trace gravel (SUBBASE) Brown Dry					232		
231.8 / 0.8	Crushed stone, containing cobbles (EMBANKMENT FILL) Grey-brown to grey Dry	1	SS	33				
		2	SS	11		231		
230.3 / 2.3	Silty sand, some gravel, trace clay (EMBANKMENT FILL) Grey-brown Dry	3	SS	37/0.2			○	15 46 33 6
230.0 / 2.6	END OF BOREHOLE AUGER REFUSAL					230		

NOTES:

1. Open borehole dry upon completion of drilling.

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

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

Golder Associates

RECORD OF BOREHOLE No 12-114

SHEET 1 OF 2

METRIC

PROJECT 12-1121-0099-1110
G.W.P. 4149-10-01
LOCATION N 4939756.7 ;E 223615.1
ORIGINATED BY DWM
DIST Eastern HWY 62
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core
COMPILED BY JM
DATUM Geodetic
DATE October 26, 2012
CHECKED BY SAT

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
232.7	GROUND SURFACE								
0.0	Gravelly sand, trace silt and clay (BASE) Brown Dry	1	GRAB	-			○		37 51 9 3
231.9 / 0.8	Crushed stone, containing cobbles (EMBANKMENT FILL) Compact Grey Dry	2	SS	28		232			
		3	SS	11		231			
230.4 / 2.3	Sand and gravel, some silt, trace weathered bedrock (EMBANKMENT FILL) Brown Moist	4	SS	29/0.3			○		
230.0 / 2.7	Granite Schist (BEDROCK) Bedrock cored from depths 2.7 m to 5.9 m For bedrock coring details refert to Record of Drillhole 12-114	1	RC	REC 100%		230			RQD = 15%
		2	RC	REC 100%		229 / 228			RQD = 89%
		3	RC	REC 100%		227			RQD = 91%
226.8 / 5.9	END OF BOREHOLE								

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

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

RECORD OF DRILLHOLE: 12-114

PROJECT: 12-1121-0099-1110

LOCATION: N 4939756.7 ;E 223615.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 26, 2012

DRILL RIG: CME 75

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	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION
		Continued from Record of Borehole 12-114	229.96			
3	Rotary Drill, NQ Core	Granite Schist (BEDROCK), containing rare lenses of sulphides Slightly weathered to fresh Dark green to black Crystalline Strong to very strong	2.74	1	100	FR FR
4				2	100	FR FR FR,UN,RO
5				3	100	FR,ST,RO
			226.84 5.86			
6		END OF DRILLHOLE				

Column headings: SYMBOLIC LOG; RECOVERY (TOTAL CORE %, SOLID CORE %; 80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX PER (5 10 15 20); DIP w.r.t. CORE AXIS (0 30 60 90); Jr; Ja; HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WEATHERING INDEX (W1 W2 W3 W4 W5 W6); FEATURES; NOTES

Depth scale: 3 4 5 6 7 8 9 10 11 12

DEPTH SCALE

1 : 50

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LOGGED: DWM

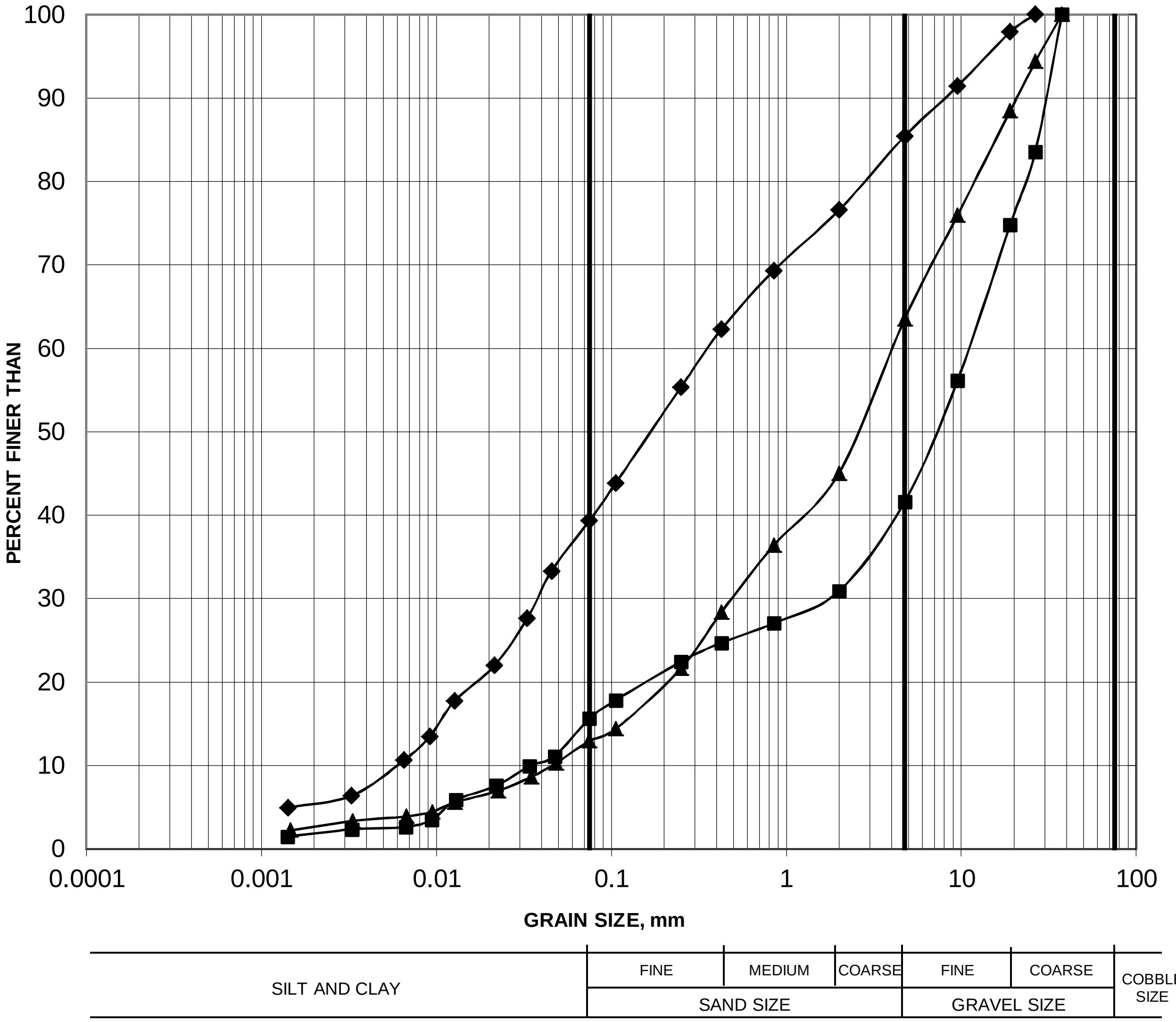
CHECKED: SAT

APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION	FIGURE B1

EMBANKMENT FILL

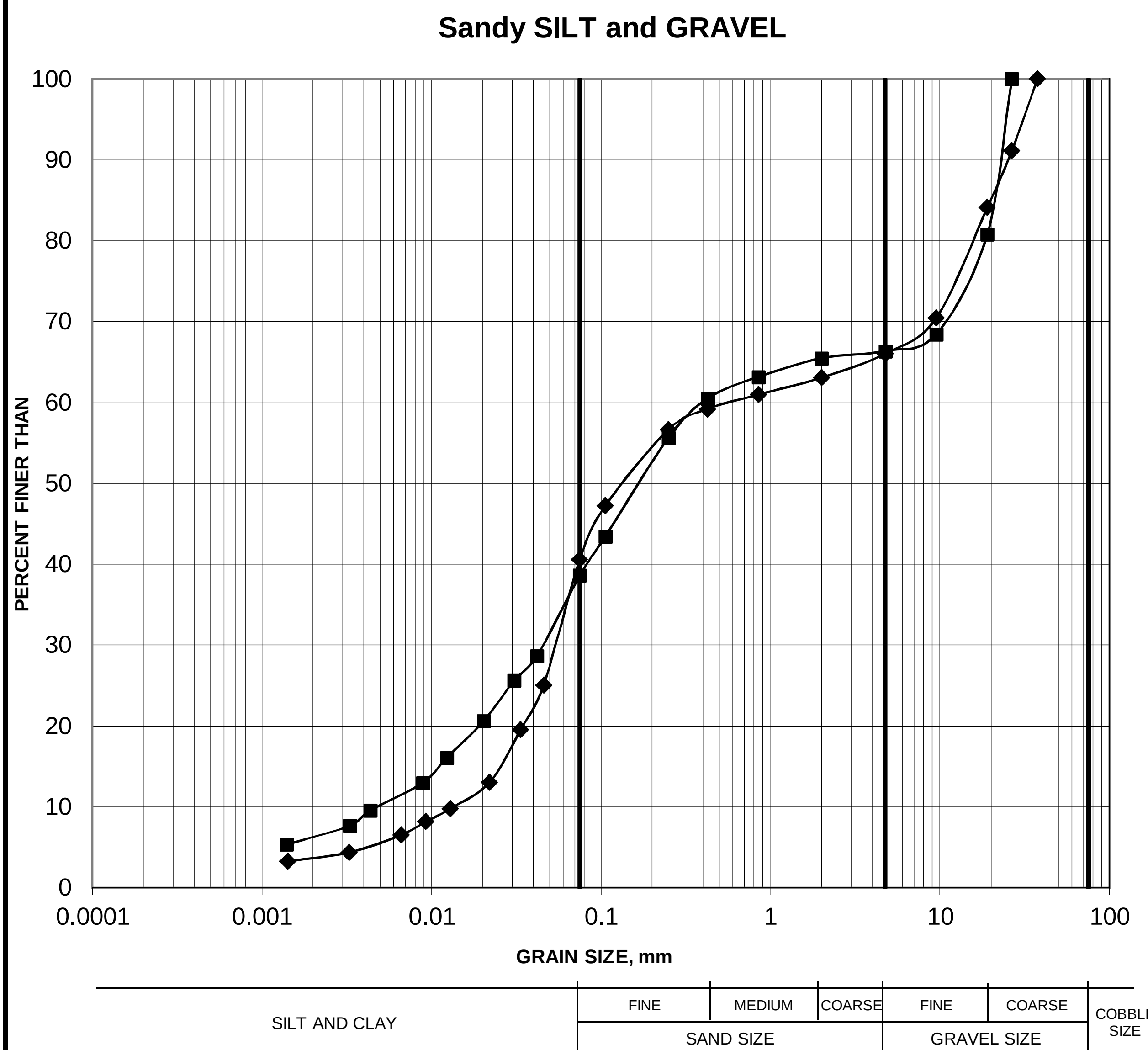


Borehole	Sample	Depth (m)
12-112	3	2.29-2.72
12-113	3	2.29-2.64
12-114	1	0.00-0.76

Project: 12-1121-0099 /1100 **Golder Associates**

Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B2

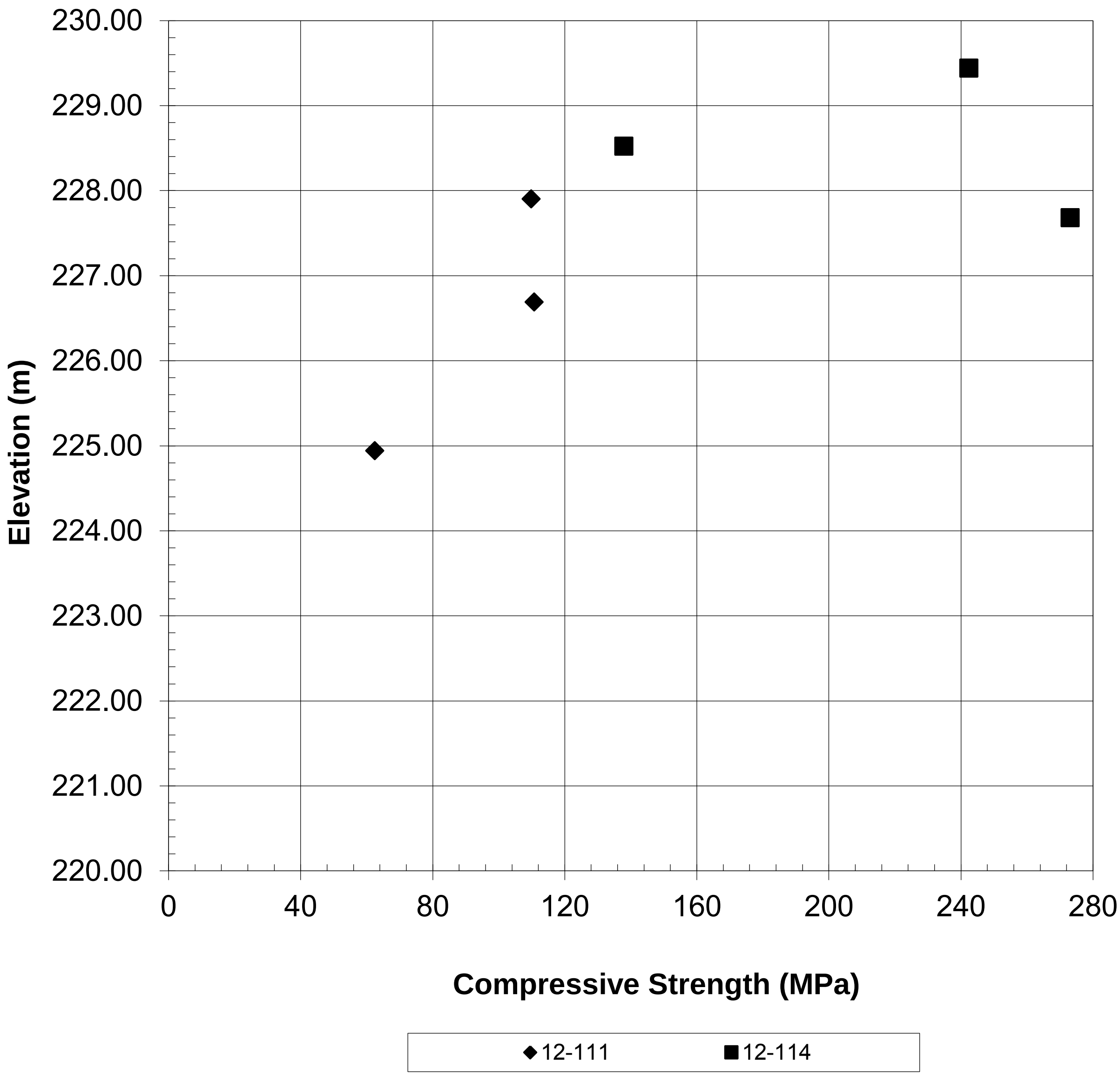


Borehole	Sample	Depth (m)
12-111 (■)	1	0.00-0.61
12-111 (◆)	3	1.22-1.83

Project: 12-1121-0099 /1100 **Golder Associates**

Created by: MI
Checked by: CNM

SUMMARY OF LABORATORY COMPRESSIVE STRENGTH POINT LOAD TESTING	FIGURE B3



Project: 12-1121-0099-1110

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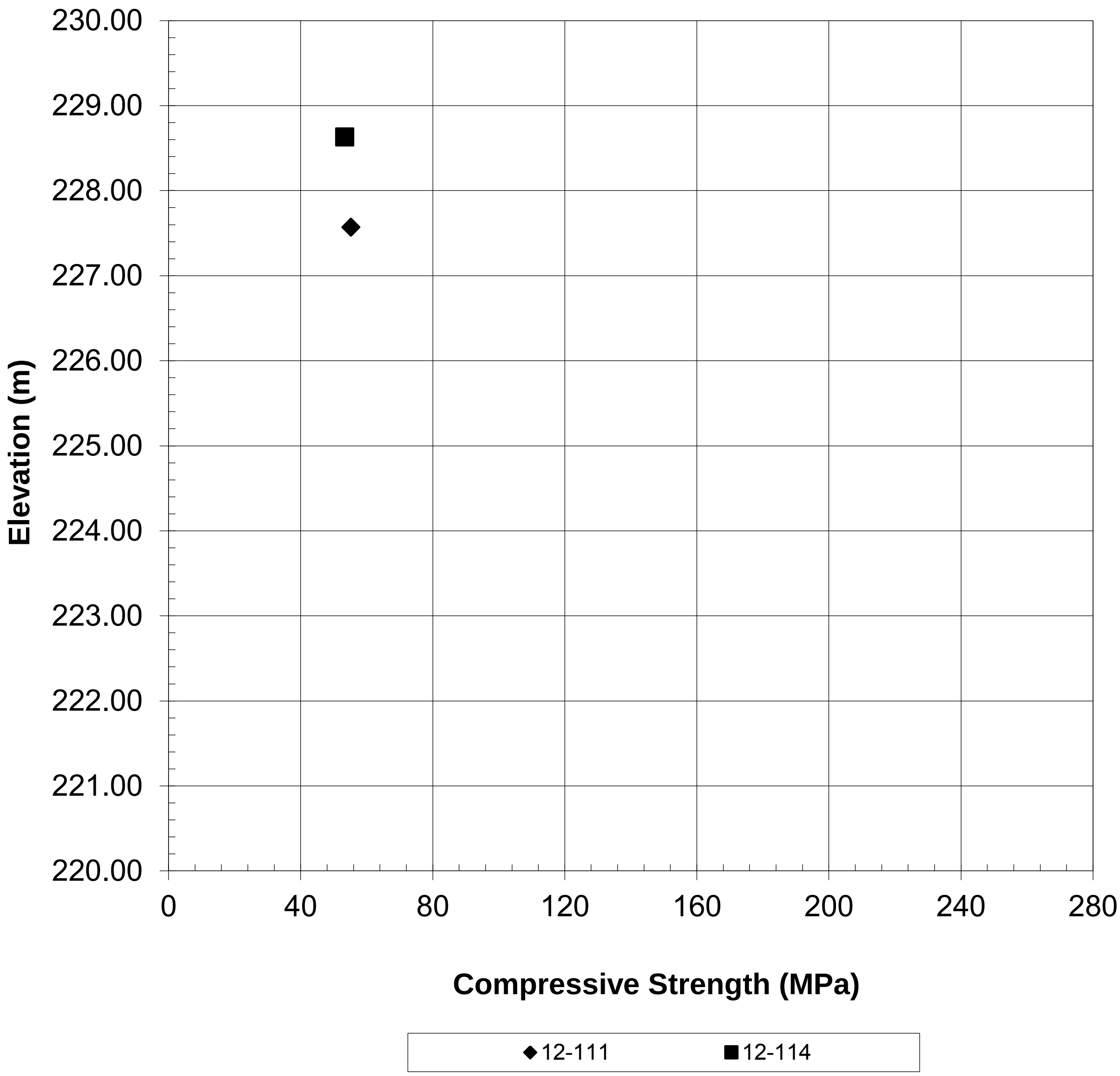
Created by:	SAT
Checked by:	FJH

SUMMARY OF LABORATORY COMPRESSIVE STRENGTH
UNCONFINED COMPRESSION TESTS

FIGURE B4



Project: 12-1121-0099-1110

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Created by:	SAT
Checked by:	FJH

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