April 2014

REPORT ON

Preliminary Foundation Investigation and Design
Ouse River Bridge Replacement
Site No. 26-90
8.6 km West of Norwood
Highway 7, Peterborough County, Ontario
W.P. 4127-10-01

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REPORT

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
OUSE RIVER BRIDGE REPLACEMENT
SITE 26-90
8.6 KM WEST OF NORWOOD
HIGHWAY 7
W.P. 4127-10-01

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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 Ouse River Bridge, Site No. 26-90 (WP 4127-10-01) located on Highway 7 about 8.6 km west of Norwood, Ontario.

The purpose of the foundation investigation was to assess the subsurface conditions for the proposed bridge replacement by drilling six boreholes, advancing six hand augerholes, and carrying out in-situ 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 Ouse River Bridge is located on Highway 7 about 8.6 km west of Norwood, Ontario. The existing bridge (Site No. 26-90) is located at about Station 12+490 where Highway 7 consists of a single lane in each direction.

Based on information provided by MMM, the natural ground surface within the lowland floodplain surrounding the Ouse River is relatively flat at about Elevation 196.0 to 196.6 m. The water level in the Ouse River was indicated to be at Elevation 196.7 m in October 2012. At the bridge, the river channel is about 10 m wide, and the water flows from north to south.

The existing bridge is a single-span, cast-in-place rigid frame structure with a width of about 11.5 m and a longitudinal span of about 9.1 m. It is understood that the structure was built in 1935 and is in fair condition. The existing pavement grade is at about Elevation 198.1 m; the approach embankments are therefore approximately 1.5 m to 2 m in height. The existing approach embankment side slopes are oriented at about 2 horizontal to 1 vertical (2H:1V) in the immediate vicinity of the existing bridge and appear to be stable based on visual observation at the time of Golder's field investigation work.

3.0 INVESTIGATION PROCEDURES

The primary subsurface investigation was carried out between April 30 and May 7, 2013, at which time six boreholes (numbered 13-221 to 13-226, inclusive) were put down at the site. A supplementary investigation was carried out during the week of October 21, 2013, at which time six hand augerholes were advanced to the south of the existing Highway 7 alignment. The boreholes and hand augerholes were put down at the locations shown on Drawings 1 and 2.

The boreholes 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 through the overburden to depths between about 7.7 m and 9.9 m below the existing ground surface. Boreholes 13-223, 13-224 and 13-225 were then cored for about 3.0 m to 3.1 m into the bedrock using NQ-size coring equipment.

In the overburden, soil samples in the boreholes were obtained at intervals of about 0.76 m, using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures.

A standpipe piezometer was installed in Borehole 13-226 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 with bentonite pellet backfill. The water level in the standpipe piezometer was measured on June 3, 2013.

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

The supplementary augerholes were put down by hand through the overburden soil at locations near the southern toe of the existing embankment slopes to determine the thickness of the organic silt (alluvial deposits). The hand augerholes were advanced to depths of up to about 1.4 m below the existing ground surface. The hand augerholes were backfilled with native soils to the existing ground surface.

All field work was supervised by a member of Golder's technical staff, who located the boreholes and hand augerholes, supervised the drilling, sampling and in-situ testing operations, logged the boreholes and hand augerholes, 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 for further examination. Index and classification tests consisting of grain size distribution, organic content, Atterberg Limits, and water content testing were carried out on selected soil samples. All of the laboratory tests were carried out to MTO and/or ASTM standards as appropriate.

The borehole locations and ground surface elevations were surveyed by MMM following completion of the drilling operations. The boreholes and their 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 Drawings 1 and 2.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
13-221	Proposed east approach embankment	4913030.9	418500.7	198.1
13-222	Proposed east abutment	4913028.8	418495.9	198.1
13-223	Proposed east abutment	4913024.3	418497.9	198.2
13-224	Proposed west abutment	4913016.6	418478.8	198.1
13-225	Proposed west abutment	4913021.4	418476.5	198.1
13-226	Proposed west approach embankment	4913011.5	418470.0	198.1

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The site is located in the southern portion of the physiographic region known as the Peterborough Drumlin Field, and just north of the Georgian Bay fringe, as delineated in The Physiography of Southern Ontario.[1]

The Peterborough Drumlin Field is characterized by deposits of glacial till overlying bedrock. The underlying bedrock is generally at about Elevation 183 m and typically consists of limestone of the Lindsay and Verulam Formation.[1]

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes and hand augerholes and the results of in situ and laboratory testing are given on the Record of Borehole and Drillhole sheets and the Table of Hand Augerholes contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B8 contained in Appendix B.

The approximate locations of the hand augerholes relative to existing site features and the surveyed borehole locations and ground surface elevations are shown on Drawings 1 and 2. The interpreted stratigraphic conditions along the centreline of the existing bridge and at the proposed abutment locations are shown on Drawings 1 and 2. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic sections included on Drawings 1 and 2 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, organic alluvial deposits, silt, silty clay to clayey silt, sand and gravel and/or glacial till over limestone 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 Highway 7 pavement structure was penetrated within the westbound lanes at Boreholes 13-221, 13-222, and 13-225 and the eastbound lanes at Boreholes 13-223, 13-224 and 13-226. At the borehole locations, the pavement structure consists of about 0.1 m to 0.3 m of asphaltic concrete overlying 0.1 m to 0.3 m of grey crushed stone base. The granular base is underlain by about 1.7 m to 3.5 m of subbase/embankment fill. The subbase/embankment fill generally consists of sand, gravel, and crushed stone with sandy silt layers. Organic matter was also encountered within the embankment fill below about 2.1 m depth at Borehole 13-226.

The fill was fully penetrated to depths between about 2.1 m and 3.7 m (Elevations 196.0 and 194.4 m, respectively).

Standard Penetration Test "N" values measured for the embankment fill range from 4 to 20 blows per 0.3 m of penetration indicating a very loose to compact state of packing.

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 measured water contents of four samples ranged from about 8 to 17 percent.

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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 Organic Silt (Alluvial Deposits)

Layered alluvial soils consisting of organic silt containing varying amounts of sand were encountered below the embankment fill at Boreholes 13-221, 13-222, 13-223 and 13-224, extending to depths ranging from about 2.9 to 3.8 m (Elevations 195.2 m to 194.4 m). Where encountered, the overall deposit ranges in thickness from about 0.8 m to 1.5 m and consists of layers of organic silt and sandy organic silt with variable organic content.

At the hand augerhole locations put down near the southern toe of the existing approach embankments, the organic alluvial deposits were generally encountered at the ground surface and ranged in thickness from about 0.4 m to 0.9 m.

Standard Penetration Test "N" values measured within the organic alluvial deposits range from 'weight of hammer' up to 5 blows per 0.3 m of penetration indicating a very soft to soft consistency.

The result of a grain size distribution test carried out on a sample of the sandy organic silt is provided on Figure B2 in Appendix B. The measured natural water contents of nine samples of the organic alluvial deposits range from about 35 to 138 percent, as measured by dry weight. The measured organic content of five samples ranged from about 4 to 14 percent.

4.2.3 Silt

A discontinuous silt layer containing some clay, and trace amounts of sand and organics, underlies the alluvial organic silt deposit at Boreholes 13-221 and 13-224, and directly underlies the embankment fill in Borehole 13-225. At Borehole 13-221, near the east approach, the silt is about 0.8 m thick. At Boreholes 13-224 and 13-225, near the proposed west abutment location, the thickness of the silt deposit ranges up to about 0.6 m.

Standard Penetration Test "N" values of 1 to 8 blows per 0.3 m of penetration were measured within the silt deposit at Borehole 13-225 indicating a very loose to loose state of packing.

The result of a grain size distribution test carried out on a sample of the silt is provided on Figure B3 in Appendix B. The measured natural water content of the silt sample was about 50 percent.

4.2.4 Silty Clay and Clayey Silt

A deposit of silty clay and clayey silt was encountered beneath the silt and/or organic alluvial deposits at the boreholes located on the east side of the bridge (Boreholes 13-221, 13-222, and 13-223). The deposit was fully penetrated to depths of about 4.9 m to 5.3 m (Elevations 192.9 to 193.2 m) and is about 1.2 m to 1.8 m thick.

Standard Penetration Test "N" values measured in the silty clay and clayey silt were generally 'weight of hammer' to 3 blows per 0.3 m of penetration and indicate a soft consistency.

The results of grain size distribution testing carried out on two samples of the clayey silt portion of the deposit are provided on Figure B4 in Appendix B. The results of Atterberg limit testing carried out samples obtained from the overall deposit gave plasticity index values ranging from 8 to 33 percent and liquid limit values ranging from 26 to 57 percent, as shown on Figure B8, indicating variable plasticity ranging from low to high. The measured natural water content of the samples ranged from 25 to 54 percent.

4.2.5 Silty Sand and Sandy Gravel

The silty clay and clayey silt in Boreholes 13-221, 13-222, 13-223 and the organic alluvial deposit in Borehole 13-224 are underlain by a deposit of silty sand to sand containing variable amounts of gravel. The sandy deposit was fully penetrated to depths of about 5.4 m to 5.9 m (with the deposit base encountered between Elevation 193.7 and 192.3 m) and ranged in thickness from about 0.4 m to 0.6 m. The silty sand in Borehole 13-221 was underlain by a deposit of sand and gravel. The sand and gravel was fully penetrated to a depth of about 7.8 m (Elevation 190.3 m) and has a thickness of about 2.3 m.

Standard Penetration Test "N" values measured in the sand and gravel ranged from 22 to 30 blows per 0.3 m of penetration, indicating a compact state of packing.

The result of grain size distribution testing carried out on one sample of the sand and gravel is provided on Figure B5 in Appendix B. The measured natural water content of one sample of the silty sand was about 30 percent, and that on one sample of the sandy gravel was about 8 percent.

4.2.6 Glacial Till

The upper overburden soil is underlain by a deposit of glacial till. In general, the glacial till is a heterogeneous mixture of gravel and cobbles in a matrix of sand and silt containing trace amounts of clay. The glacial till was fully penetrated in Boreholes 13-222, 13-223, 13-224 and 13-225 to depths between 7.9 and 9.4 m (Elevations 190.2 and 188.7 m). The glacial till in Boreholes 13-221 and 13-226 was not fully penetrated but was proven to a depth of about 9.3 m and 7.7 m (Elevations 188.9 m and 190.4 m). The glacial till had a thickness of about 1.5 m to 4.8 m. The results of grain size distribution testing carried out on twelve samples of the glacial till are provided in Figure B6 in Appendix B. The results do not reflect the cobble or full gravel contents of the material, since the samples were retrieved using a 50 mm outside diameter split-spoon sampler. The measured natural water contents of ten samples of the till ranged from about 7 to 10 percent.

At Boreholes 13-223, 13-224, and 13-225 a stratum of sand to sandy silt and trace gravel was present below or within the lower portion of the glacial till deposit, with the surface of these layers encountered at depths of about 7.3 m to 7.9 m. The thickness of these sandy strata ranged from about 0.6 to 1.1 m. The results of grain size distribution testing carried out on three samples of the sandy stratum within or immediately the till are provided on Figure B7 in Appendix B. The measured natural water contents of tested samples ranged from about 13 to 18 percent.

Standard Penetration Test "N" values measured in the glacial till ranged from 7 to 108 blows per 0.3 m of penetration, indicating a generally compact to very dense state of packing. Standard Penetration Test "N" values measured in the sandy strata ranged from 25 to greater than 100 blows per 0.3 m of penetration, indicating a compact to very dense state of packing similar to that of the overall glacial till deposit.

The glacial till deposit at Borehole 13-222 is underlain by a deposit of cobbles and boulders about 0.6 m thick. The cobbles and boulders were penetrated to a depth of about 9.9 m (Elevation 188.2 m); this zone may also represent weathered bedrock.

4.2.7 Refusal and Bedrock

Bedrock was encountered beneath the glacial till (including sandy strata) at Boreholes 13-223, 13-224, and 13-225, where it was cored for a depth of about 3 m. Auger refusal was encountered at Boreholes 13-221, 13-222, and 13-226; this has been inferred to represent the bedrock surface.

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

Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
13-221	198.1	9.2*	188.9*
13-222	198.1	9.9*	188.2*
13-223	198.2	9.2	189.0
13-224	198.1	8.9	189.2
13-225	198.1	9.4	188.7
13-226	198.1	7.7*	190.4*

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

The bedrock encountered in the boreholes typically consists of fine to coarse grained, grey limestone bedrock with black shale partings and interbeds. The bedrock is slightly weathered to fresh and typically medium strong to strong.

The Rock Quality Designation (RQD) values measured on the recovered bedrock core samples ranged from about 50 to 90 percent, indicating good to excellent quality rock. The discontinuities observed in the rock core were associated with the joints, veins, faults and fractures of the bedrock.

4.2.8 Groundwater Conditions

The groundwater level in the piezometer in Borehole 13-226 was measured on June 3, 2013 and is summarized in the following table:

Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date
13-226	198.1	1.4	196.7	June 3, 2013

The water level in the river was also at Elevation 196.7 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 Foundation Investigation Report was prepared by Mr. Matt Kennedy, P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., a senior geotechnical engineer and Principal 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
OUSE RIVER BRIDGE REPLACEMENT
SITE 26-90
8.6 KM WEST OF NORWOOD
HIGHWAY 7
W.P. 4127-10-01

April 2014
Report No. 12-1121-0099-1220

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 Ouse River Bridge on Highway 7. 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 future stages of design.

Where comments are made on construction, they are provided to highlight those aspects that could affect the preliminary design of the project, and for which special provisions may be required in the terms of reference for the design-build assignment. 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 in plan on Drawings 1 and 2 and consists of a single-span, cast-in-place concrete structure. It is understood that the existing bridge will be replaced with a wider, single-span structure on an alignment shifted up to about 5.0 m to the south. The new bridge structure will be founded on abutments located approximately 16.5 m apart on either side of the river. The proposed pavement grades at the new structure will be about 0.6 m higher than the existing pavement grades. A temporary, modular bridge is to be constructed north of the existing structure to provide an additional travelled lane that will be open during staging operations.

6.2 Foundation Options

The existing Ouse River Bridge is a single-span, rigid frame structure with a reinforced concrete deck. The existing bridge is understood to be in fair condition. Based on the 1934 General Arrangement drawing (Drawing No. 235-11-2, dated August 1, 1934) the existing foundations beneath the abutments are understood to consist of spread footings founded on competent native soils. At the bridge location, the river channel is about 10 m wide and flows from north to south.

The existing pavement grade at the bridge location is at about Elevation 198.1 m. In this area, Highway 7 consists of one travelled lane in each direction (i.e., a two-lane highway). The existing embankment slopes at the culvert locations are about 1.5 m to 2 m in height and are sloped at about 2H:1V.

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

- **Spread footings founded on glacial till:** Spread footings could be considered for support of the replacement structure provided they are founded on or within the compact to dense native till, below the soft silty clay and thin silty sand deposits (at about Elevation 192.3 m) at the east abutment, and below the organic silt to sandy silt deposits (at about Elevation 193.5 m) at the west abutment. Some minimal settlement of the abutment footings (less than about 25 mm) may occur for footings founded on the glacial till. The soils are also water-bearing, and active dewatering would be required to control the ground and groundwater during excavation and construction. It is expected that temporary protection systems and/or cofferdams would be required during excavation and construction.

- **Driven steel H-piles:** Steel H-piles driven to refusal on the limestone bedrock or within the dense to very dense glacial till are feasible for support of the replacement bridge structure, and this option would allow the pile caps to be maintained at a higher elevation than for a spread footing option, thus minimizing excavation depth, protection system requirements and groundwater control requirements, while achieving relatively higher geotechnical resistances and minimizing settlement. Steel H-pile foundations would also allow for the construction of integral abutments. If the piles are driven, the use of driving shoes is recommended to minimize damage while penetrating the glacial till deposit (which contains cobbles and boulders) and seating onto the limestone bedrock.
- **Driven steel pipe (tube) piles:** Steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutments, and this foundation option would have similar advantages to steel H-piles in terms of minimizing excavation depth, protection system requirements and groundwater control requirements. However, pipe piles are considered to have a slightly higher risk than H-piles for "hanging up" or being deflected away from their vertical or battered orientation due to the presence of cobbles and/or boulders within the till deposit.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments for the bridge replacement on steel H-piles driven to found on the bedrock.

6.3 Shallow Foundations

6.3.1 Founding Elevations

If adopted for the replacement structure, spread footings should be founded on the compact to dense glacial till and below any existing fill, compressible organic soil, silt, or soft silty clay. Shallow foundations bearing directly on the underlying limestone bedrock are not considered to be practical due to the significant depth of excavation that would be required.

The following table provides the maximum (highest) founding elevations recommended for preliminary design of footings founded on the compact to dense glacial till deposit. Excavation would be carried out to depths of up to about 5.9 m below the existing roadway grade, and up to about 4.4 m below the measured groundwater level at the site. Dewatering and temporary protection systems/cofferdams would be required to minimize disturbance of the subgrade soils and instability of the excavation side walls, as discussed further in Section 6.6.

Foundation Element	Borehole Numbers	Founding Stratum	Footing Founding Elevation
East Abutment	13-222, 13-223	Compact to very dense till	Below 192.3 m
West Abutment	12-224, 13-225	Dense to very dense till	Below 193.5 m

The footing subgrade should be inspected in accordance with OPSS 902 *(Excavating and Backfilling Structures)* to check that all existing fill, organic deposits, loose silt, and other unsuitable material have been removed. The founding soils will be susceptible to disturbance and should be protected with a concrete working slab (100 mm thick concrete slab with a compressive strength of 20 MPa) if the concrete for the footing is not placed within four hours of the inspection and approval of the subgrade.

To the north of the existing Highway 7 and bridge alignment, the subsurface conditions are anticipated to consist of organic alluvial deposits immediately below the existing ground surface, underlain by silt and soft silty clay that overlie the dense glacial till deposit. Spread footings for the temporary modular bridge to be constructed in this area should also be founded on the compact to dense glacial till or on a compacted granular pad constructed directly on the glacial till.

6.3.2 Geotechnical Resistance

Spread footings placed on the properly prepared glacial till deposit, at or below the design elevations given in the preceding section, should be designed based on preliminary factored geotechnical resistances of 400 kPa at Ultimate Limit States (ULS) and 275 kPa at Serviceability Limit States (SLS, for 25 mm of settlement).

Following removal of the organic and soft silty clay soil that overlies the glacial till or compact silty sand to sand, spread footings for the temporary, modular bridge may be constructed on a compacted granular pad. Preliminary design for this configuration can be completed based on a factored geotechnical resistance at ULS of 500 kPa, and a geotechnical resistance at SLS of 300 kPa, for 25 mm of settlement.

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 later stages of design, based on future additional subsurface investigation at the proposed abutment locations for the permanent or temporary modular structures.

6.4 Driven Steel H-Pile or Steel Pipe (Tube) Pile Foundations

6.4.1 Founding Elevations

The abutments for the replacement bridge, as well as for the temporary modular bridge, may be supported on steel H-piles driven to found on the limestone bedrock or steel pipe (tube) piles founded within the very dense glacial till. Additional borehole investigation will be required during future design stages to confirm the bedrock surface variability within the footprint of the proposed east and west abutments if the final alignment of the permanent structure is modified, and for the temporary modular structure. However, based on the borehole results from the preliminary investigation, and assuming about 0.1 m of penetration into the bedrock to allow for some weathering in the upper portion of the rock, the following pile tip elevations are recommended for preliminary design of steel H-piles:

Foundation Element	Borehole Numbers	Bedrock Surface Elevation	Design Pile Tip Elevation
East Abutment	13-222, 13-223	189.0 to 188.2 m	188.9 to 188.1 m
West Abutment	12-224, 13-225	189.2 to 188.7 m	189.1 to 188.6 m

The pile caps should be constructed at a minimum depth of 1.6 m for frost protection purposes, per OPSD 3090.101 *(Foundation Frost Penetration Depths for Southern Ontario)*.

At the abutment locations, the bedrock surface was encountered at elevations ranging from about 188.2 to 189.2 m. To provide suitable flexibility of steel H-piles for integral abutments, it is understood that the upper portion of the piles would be cased in a sand-filled, corrugated steel pipe (CSP) from about Elevation 194.8 m at the underside of the abutment stem. Based on the results of the current investigation, steel H-piles driven to the bedrock surface at about Elevation 189 m would be greater than 5 m in length and are therefore considered to be feasible for use in an integral abutment configuration.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the till deposit. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of "hanging up" or being deflected away from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip with rock points to improve seating of the piles on the bedrock and to reduce the potential for damage to the piles during driving through soils that contain boulders, in accordance with OPSS 903 *(Deep Foundations)*. If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001.100 Type II *(Steel Tube Pile Driving Shoe)*.

6.4.2 Axial Geotechnical Resistance

For preliminary design of HP 310x110 piles driven to the estimated tip elevations provided in Section 6.4.1, 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 the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. However, it is also noted that based on the presence of cobbles and boulders within the till deposit, and the very dense nature of the till deposit, the steel H-piles or closed-end, concrete-filled, 324 mm diameter steel pipe piles may not reach the bedrock. Provided the piles meet practical refusal in the very dense glacial till, the factored axial resistance at ULS may be taken as 1,600 kN and the geotechnical resistance at SLS may be taken as 1,300 kN.

Pile installation should be in accordance with OPSS 903 *(Deep Foundations)*. The drawings should incorporate the appropriate note stating that the piles should be equipped with bearing points and should be driven to bedrock. For piles driven to refusal on bedrock, 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.

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 maximum peak particle velocity of 100 mm/sec is recommended at the existing abutments. 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 future stages of design in consideration of the additional subsurface investigation that will be carried out at the site.

6.5 Approach Embankments

6.5.1 Subgrade Preparation and Embankment Construction

It is understood that the overall grade of Highway 7 will be raised about 0.6 m to accommodate a required increase in the soffit elevation for hydraulic reasons. In addition, the new highway alignment will be shifted up to about 5.0 m to the south requiring widening of the existing approach embankments.

Based on the results from the boreholes drilled through the existing Highway 7 embankment, the sand and gravel embankment fill is underlain by layered organic silt, layered soft silty clay and clayey silt, sand and gravel, glacial till, and limestone bedrock. The organic silt deposit was encountered in all boreholes put down to the east of the crossing and in one borehole put down within the southern portion of the proposed west abutment area (Borehole 13-224). In addition, the organic deposit was encountered at or near the ground surface within all hand augerholes put down to the south of the existing alignment. The soft silty clay and clayey silt deposit was encountered beneath the organic silt in the boreholes put down to the east of the crossing within the east approach embankment and abutment areas, but was not encountered in the hand augerholes put down to the south of the existing approach embankments.

The organic silt and the underlying silty clay to clayey silt at the east abutment are compressible soils that are expected to experience consolidation settlements under increased load. Within the footprint of the existing approach embankments, the increased load imposed by additional fill placed to accommodate the proposed grade raise would result in additional settlement of these compressible soils on the order of 200 mm. To the south within the footprint of the embankment widening, it is estimated that the new widened embankment loading would result in of settlement on the order of 750 mm in the organic soil and soft clayey soils (where present). Full removal of the existing embankment fill and underlying organic silt and soft silty clay to clayey silt layers would minimize the potential for additional total or differential settlements.

Consideration may be given to the use of lightweight fill materials such as EPS Geofoam for embankment construction within the footprint of the existing approach embankments to reduce the stress increase on the compressible soils to a level that would result in settlements within acceptable tolerances. A portion of the existing embankment may be excavated and replaced with lightweight fill such that there is no net increase in load on the underlying soil. Based on the proposed grade increase of 0.6 m, an estimated 1.0 m thickness of EPS Geofoam would be required within the final embankments. However, the total thickness of embankment fill removal and lightweight fill replacement will be dependent on the type of lightweight fill and total grade raise and should be verified during detail design.

To minimize the required thickness of overexcavation, consideration may be given to preloading and surcharging the proposed approach embankment areas to consolidate the compressible silty clay that underlies parts of the site, provided that the construction period is of significant duration to allow for the majority of the consolidation to occur. The different soil layers beneath the existing embankments and within the widened area to the south of the existing alignment will compress at differing rates depending on the soil composition, the stratum thickness and the previous loading history.

Preload site preparation treatment involves the placement of the permanent loads (typically permanent grade fills) in advance of the completion of the embankment and roadway construction to allow the subgrade soils to compress under the weight of the applied fills, thus reducing the potential post-construction settlement.

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Preloading treatment reduces, but does not entirely eliminate, settlement effects. Some residual, post-treatment settlement is inevitable due to the long-term secondary consolidation of the fine-grained subgrade soils. Surcharging involves the placement of a temporary fill load (in addition to the permanent fill load) to accelerate consolidation of the subgrade soils, and thus accelerate site preparation treatment. The thickness of the applied temporary surcharge varies depending on the degree to which the soil consolidation process (and thus the construction schedule) needs to be accelerated. A trade-off exists between the height or thickness of the applied preload and temporary surcharge fills, and the desired timeframe to complete site preparation treatment; a greater thickness of applied surcharge results in a shorter preload duration, but at increased cost.

Within the existing embankment footprint, the preload/surcharge material may be placed above the existing embankment fill following removal of the existing roadway structure. Within the lowland area to the south of the existing highway alignment, the surficial organic silt and alluvial material should be removed prior to placement of any new embankment fill.

Other methods of ground improvement, including deep soil mixing and rammed aggregate piers, were examined at a conceptual level. A comparison of these and the previously described methods of ground improvement is provided in Table 2 following the text of this report.

Regardless of the selected method of abutment construction (removal of the embankment fill, organic deposits, and soft silty clay to clayey silt; subexcavation of the existing embankment fill and replacement with lightweight fill (where appropriate); or, preload/surcharge treatment of the existing and widened embankment areas), it is recommended that it be carried out below the proposed approach embankments within approximately 25 m of the bridge abutments. Where the organic silt and silty clay to clayey silt layers are to be fully removed, excavation should be carried out to the compact to dense sand and gravel or glacial till. The depth and extent of stripping and/or lightweight fill placement for any embankment construction should be further assessed during future stages of design when additional subsurface information can be obtained.

Any new embankment fill for the approaches should be placed and compacted in accordance with OPSS 206 *(Grading)* and OPSS 501 *(Compacting)*.

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 572 *(Seed and Cover)*.

6.5.2 Global Stability

Provided that the approximately 2.5 m high approach embankment side slopes are maintained no steeper than 2H:1V, and that the surficial organic silt and alluvial material are removed from within the footprint of the widened embankment area to the south of the existing highway alignment, the embankments should have an adequate minimum factor of safety of at least 1.3 under static conditions and 1.1 under design seismic conditions. This minimum factor of safety is considered appropriate for the proposed bridge approach embankments, considering the design requirements and the available field and laboratory testing data.

This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed based on the subsoil conditions encountered during future stages of investigation.

6.5.3 Settlement

Compressible organic and clayey deposits were encountered below the embankment fill at the boreholes put down through the existing approach embankments. Settlement of the existing embankments has likely occurred over time since the original bridge construction. If the existing embankments are left in place, the additional loading imposed by the proposed grade raise would result in further consolidation settlement of the underlying compressible soils; this additional settlement has been estimated to be on the order of 200 mm. In addition, with no subexcavation and assuming the use of conventional earth fill or granular fill for the embankment widening, settlement on the order of 750 mm within the organic soil and soft clayey soils has been estimated under the proposed southward widening area.

As described above, the existing embankment fill, organic deposits, soft silty clay to clayey silt, and any softened/loosened soil should be stripped from the footprint of the proposed approach embankments within approximately 25 m of the bridge abutments. At the west approach, the existing embankment fill, organic silt, and silt should be removed to the underlying glacial till deposit. At the east approach, the soft silty clay and clayey silt should also be removed to the underlying compact to dense sand and gravel or glacial till. The settlement of the sand and glacial till under the grade raise or embankment widening is expected to be less than 25 mm. This settlement is expected to occur relatively quickly during and immediately following construction of the new approach embankments based on the nature of the soils at the site.

Alternatively, within the footprint of the existing approach embankments, partial excavation of the embankment fill and replacement with lightweight fill may be carried out. Provided that a thickness of lightweight fill suitable to minimize the additional vertical load imposed by the grade raise is used, the post-construction settlement could be limited to less than about 25 mm. Embankment construction with lightweight fill is not considered to be feasible for the widened portions of the approach embankments to the south of the existing alignment.

If a preload/surcharge treatment program is carried out, the post-construction settlement of the embankments could be limited to less than 25 mm provided that the construction period is of sufficient duration. As described in Section 6.5.1, the duration required for preload/surcharge treatment will be dependent on the soil composition, the stratum thickness, the previous loading history, and the thickness of surcharge. Because of the limited thickness of the silty clay and clayey silt strata at this site, a surcharge of 1.0 m left in place for a duration of four months should reduce the post-construction settlement to about 25 mm.

The estimated magnitudes of settlement should be reassessed based on the soil and groundwater conditions under the new approach embankments as determined during future stages of design. The above preliminary estimates do not include compression of the embankment fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of fill compression may range from 0.5 to 1 percent of the height of the embankment, assuming approximately 98 percent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur essentially during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time.

6.6 Construction Considerations

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

6.6.1 Excavation and Temporary Protection Systems

If spread footings are adopted for support of the replacement structure, the foundation excavations are expected to extend up to about 6 m below the existing Highway 7 grade or up to about 4.5 m below the floodplain grade into the water-bearing, organic silt, silty clay to clayey silt, and compact to dense sand and glacial till. The excavations for pile caps could be maintained at a higher elevation, behind and above the existing spread footing foundations.

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. The existing fill above the water table would be classified as Type 3 soil, based on the OHSA. According to OHSA excavations in 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, organic silt, silty clay to clayey silt, sand, and glacial 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.

At this preliminary stage, it is anticipated that temporary protection systems would likely be required to facilitate the excavation to foundation or pile cap level for the new abutments and construction of the approach embankments. The temporary excavation 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; however, where excavations are within the zone of influence of existing or new footings while those footings support structures that are in service, it is recommended that the lateral movement of the protection system meet Performance Level 1b as specified in OPSS 539.

It is considered that an interlocking sheetpile system would aid in groundwater control at this site, although the presence of cobbles or boulders may impact the depth that sheet piling can be driven and the effectiveness of the system.

6.6.2 Groundwater Control

Depending on the river flow at the time of construction, the surface water flow could be passed through the site by means of a temporary pipe, or diverted by pumping from behind a temporary cofferdam. 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.

The excavations for the bridge replacement will extend into the water-bearing fill, organic silt, silty clay to clayey silt, sand and glacial till. The excavation walls will require shoring and groundwater control by means of dewatering from wells or wellpoints within or outside the excavation for foundation excavations (whether footings or pile caps). Penetration of interlocking sheetpile walls into the dense sandy silt till and sand and gravel sand may be difficult due to the presence of cobbles and boulders. The excavation walls may be supported by driven steel soldier piles and lagging. In areas where the subgrade soil consists of relatively clean sand and gravel, the groundwater level mat be lowered beneath the base of the excavation by pumping from wellpoints established below the subgrade level. It is anticipated that this method may be used at the east abutment, but further field investigation at the design stage would be required to confirm feasibility.

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.

6.6.3 Subgrade Protection

If the abutments are to be founded on shallow spread footings, the glacial till that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade.

6.6.4 Vibration Monitoring During Pile Driving

The proposed staged construction is to include partial construction of the new the structure while a portion of the existing structure remains in service. It is recommended that vibration monitoring be carried out during installation of piles or driven protection systems to assist in maintaining vibration levels within tolerable ranges for the existing portions of the bridge, or for any temporary modular structure if used at the site.

A maximum peak particle velocity of 100 mm/sec is recommended at the existing abutments. 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

The glacial till at this site should be expected to contain cobbles and boulders, which could affect the installation of deep foundations and/or protection systems. Further observation of the frequency of such obstructions is recommended in the next stage of investigation in support of the detail design.

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 river banks to protect the foundations/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

Additional boreholes will be required to support final design, to provide additional information within the proposed widened portions of the alignment and 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 and surficial soils to confirm the founding elevation for spread footings within each abutment area, if shallow foundations are selected for the replacement structure.
- Confirmation of the bedrock surface elevation within the proposed abutment area, and the thickness of any weathered, fractured or otherwise weakened zone at the top of the bedrock, to confirm the founding elevation for piled foundations, particularly for the widening area to the south.
- Assessment of the depth and extent of stripping of fill, organic silt, and clayey deposits within the footprint of the existing and widened approach embankments, and/or the settlement properties of the soft clayey deposits to refine the settlement estimates and preload/surcharge period estimates.

- Observation of the presence of cobbles and/or boulders within the soil deposits as they may affect excavations and the installation of driven steel H-pile or pipe pile foundations, or elements of temporary protection systems.
- Further assessment of the groundwater level and permeability of the site soils to refine dewatering estimates.
- Assessment of the existing soil and groundwater conditions at the proposed foundation locations for the temporary modular bridge (i.e., a minimum of one borehole at each abutment location).

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Matt Kennedy, P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., a senior geotechnical engineer and Principal 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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April 17, 2014
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Geotechnical Engineer

Lisa Coyne, P.Eng.
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REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
17 APR 2014
PROVINCE OF ONTARIO

Fintan Heffernan, P.Eng.
Designated MTO Contact

WAM/MJK/LCC/FJH/bg

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Table 1 – Comparison of Foundation Alternatives

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-piles driven to bedrock	▪ Feasible for support of bridge replacement ▪ Preferred option from a foundations perspective	▪ Abutment pile caps could be maintained higher than for footings, reducing depth of excavation and temporary excavation support requirements ▪ Higher geotechnical resistances and negligible settlement ▪ Allows for integral abutment construction	▪ Potential for encountering obstructions (cobbles and/or boulders) during pile driving that could result in piles “hanging up” in the glacial till deposit and lower geotechnical resistances ▪ Some groundwater control would still be required	▪ Moderate cost	▪ Low risk of driven H-piles “hanging up” in glacial till
Steel pipe (tube) piles, driven to found in bedrock	▪ Feasible for support of bridge replacement	▪ Abutment pile caps could be maintained higher than footings, reducing depth of excavation and temporary protection system ▪ Higher geotechnical resistances and negligible settlement ▪ Allows for semi-integral abutment configuration	▪ Slightly greater risk than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving; this could result in piles “hanging up” and lower geotechnical resistances ▪ Some groundwater control would still be required	▪ Moderate cost	▪ Moderate risk of pipe piles “hanging up” in glacial till

Table 1 – Comparison of Foundation Alternatives (Continued)

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread/strip footings on compact to dense sand and gravel or glacial till	▪ Because of dewatering concerns, spread footings may not be feasible	▪ Existing structure supported on shallow foundations, and has performed reasonably ▪ Allows for semi-integral abutments	▪ Historic settlement data not available ▪ Significant excavations to a depth of up to about 4.5 m below the existing floodplain grade, or more than 6 m below the Highway 7 grade; would require temporary protection systems ▪ Significant groundwater control requirements ▪ Lower geotechnical resistances as compared with deep foundations; potential for approximately 25 mm of settlement ▪ Precludes use of integral abutments; potentially greater maintenance required	▪ Less expensive than deep foundations although bridge maintenance costs may be higher due to non-integral abutment configuration ▪ Additional costs required for dewatering may eliminate any cost differential	▪ Risk of instability of existing embankment slopes without appropriate temporary protection measures and dewatering during excavation to significant depth

Table 2 – Comparison of Methods of Ground Improvement

Ground Improvement Method	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Full excavation of embankment fill and underlying organic silt and silty clay	▪ Feasible for replacement/ improvement of embankment fill	▪ No specialized construction equipment required ▪ Reduced potential for differential settlement between existing embankment and widened embankment	▪ Excavations would extend up to about 3 m below the groundwater and flood plain elevation; groundwater control and temporary protection measures would be required	▪ Low relative cost	▪ Risk of unstable excavation side slopes at depths below the existing groundwater level
Use of lightweight fill (i.e., EPS Geofoam)	▪ Feasible for replacement/ improvement of existing embankment fill	▪ Excavations within existing embankment footprint would be of limited depth and would not extend below the groundwater table ▪ Portions of the existing embankment fill can be left in place	▪ Excavations to remove the organic silt within the widened portion of the embankment would still be required; groundwater control would be required ▪ Different treatment methods required beneath existing embankment and within widened portion	▪ Low relative cost ▪ Higher material cost, but less excavation required	▪ Risk of unstable excavation side slopes in widened areas at depths below the existing groundwater level

Table 2 – Comparison of Methods of Ground Improvement (Continued)

Ground Improvement Method	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Preload and surcharge (1.0 thickness) for period of 4 months	▪ Feasible for improvement of existing and widened embankment footprint	▪ Minimizes requirement for over-excavation within embankment footprint ▪ Method is achievable within anticipated construction timeframe ▪ Similar treatment may be applied to both existing and widened portion of the embankment reducing potential for differential settlements	▪ Method is time-dependent and will require a suitable duration for surcharge treatment	▪ Low relative cost ▪ Higher material cost, but significantly less excavation required	▪ Some risk of instability of existing embankment slopes without appropriate temporary protection measures and dewatering in widened area during excavation to significant depth
Rammed aggregate piers	▪ Potentially feasible, depending on access and equipment availability	▪ Uniform treatment of improved area	▪ Potential construction schedule implications depending on site access and equipment availability ▪ High mobilization and set-up costs	▪ Moderate to High relative cost	▪ Additional design/consideration required
Deep soil mixing	▪ Potentially feasible, depending on access and equipment availability	▪ Uniform treatment of improved area tailored to site conditions	▪ Potential construction schedule implications depending on site access and equipment availability ▪ High mobilization and set-up costs	▪ High relative cost	▪ Additional design/consideration required

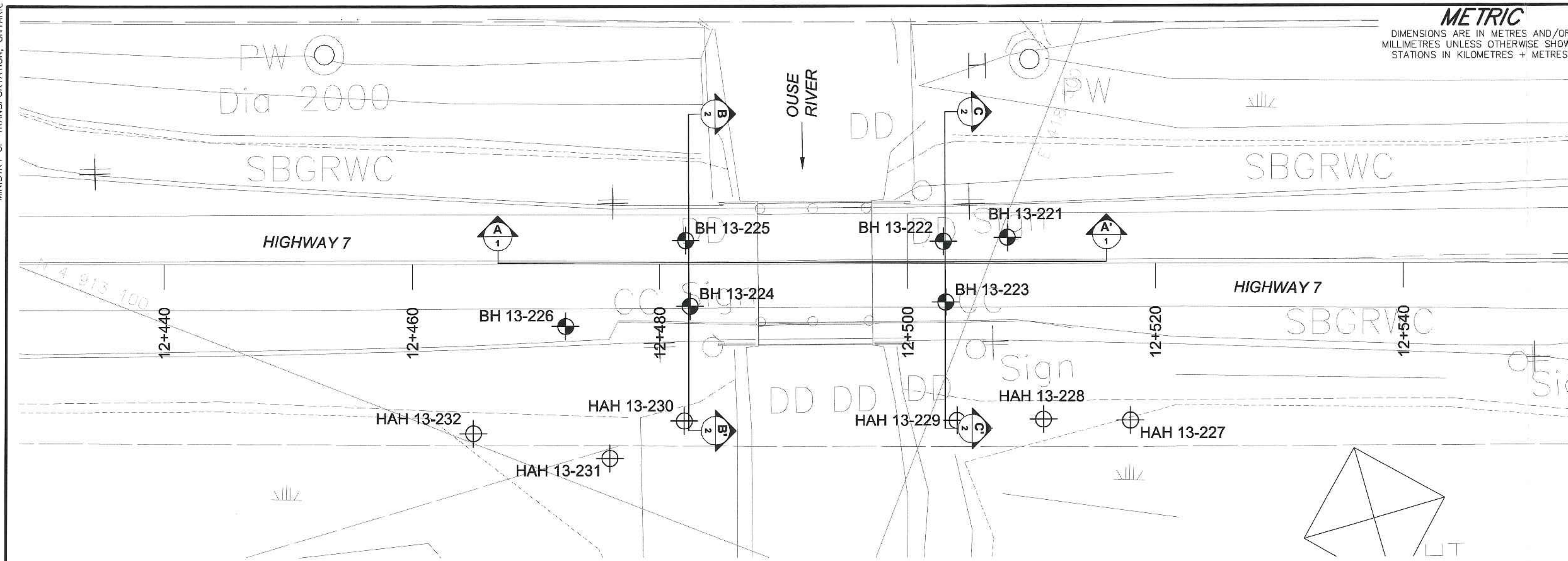
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 4127-10-01

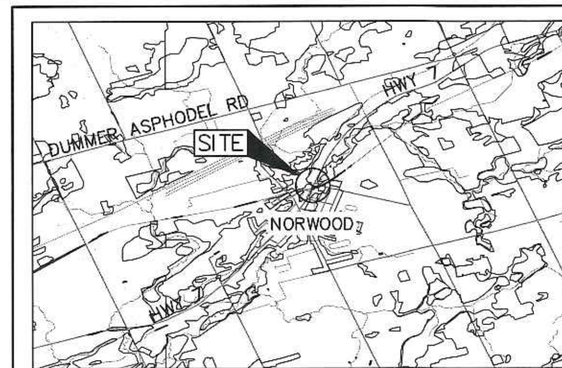
SHEET

OUSE RIVER BRIDGE
SITE 26-90
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



PLAN
SCALE
4 0 4 8 m



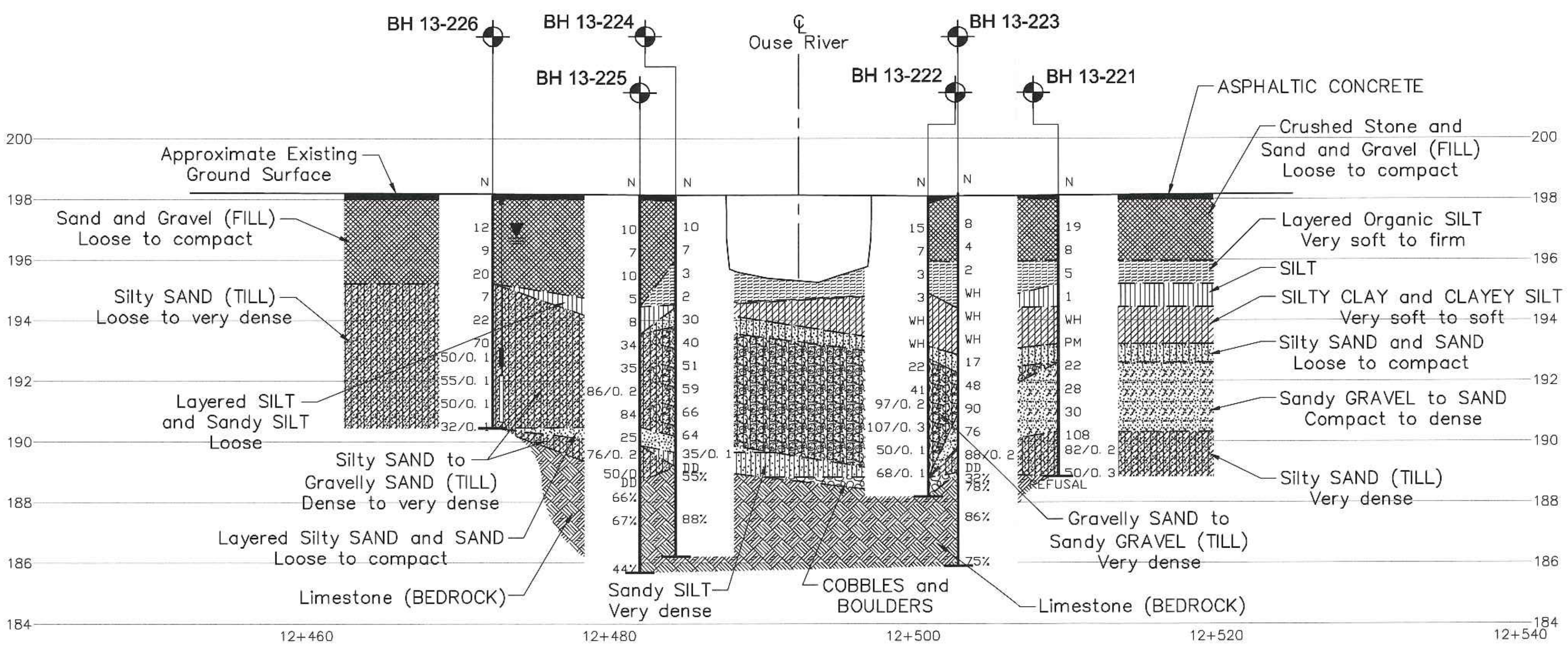
KEY PLAN
SCALE
1 0 1 2 km

LEGEND

	Borehole – Current Investigation
	Hand Augerhole – 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, measured June 3, 2013
	WL upon completion of or during drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
12–221	198.1	4913030.9	418500.7
12–222	198.1	4913028.8	418495.9
12–223	198.2	4913024.3	418497.9
12–224	198.1	4913016.6	418478.8
12–225	198.1	4913021.4	418476.5
12–226	198.1	4913011.5	418470.0



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

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 26-90.dwg, received March 27, 2013.

NO.	DATE	BY	REVISION

Geocres No. 31C-222

HWY. 7	PROJECT NO. 12-1121-0099-1220		DIST. Eastern
SUBM'D. SAT	CHKD. FJH	DATE: Mar. 2014	SITE: 26-90
DRAWN: JM	CHKD. SAT	APPD. FJH	DWG 1

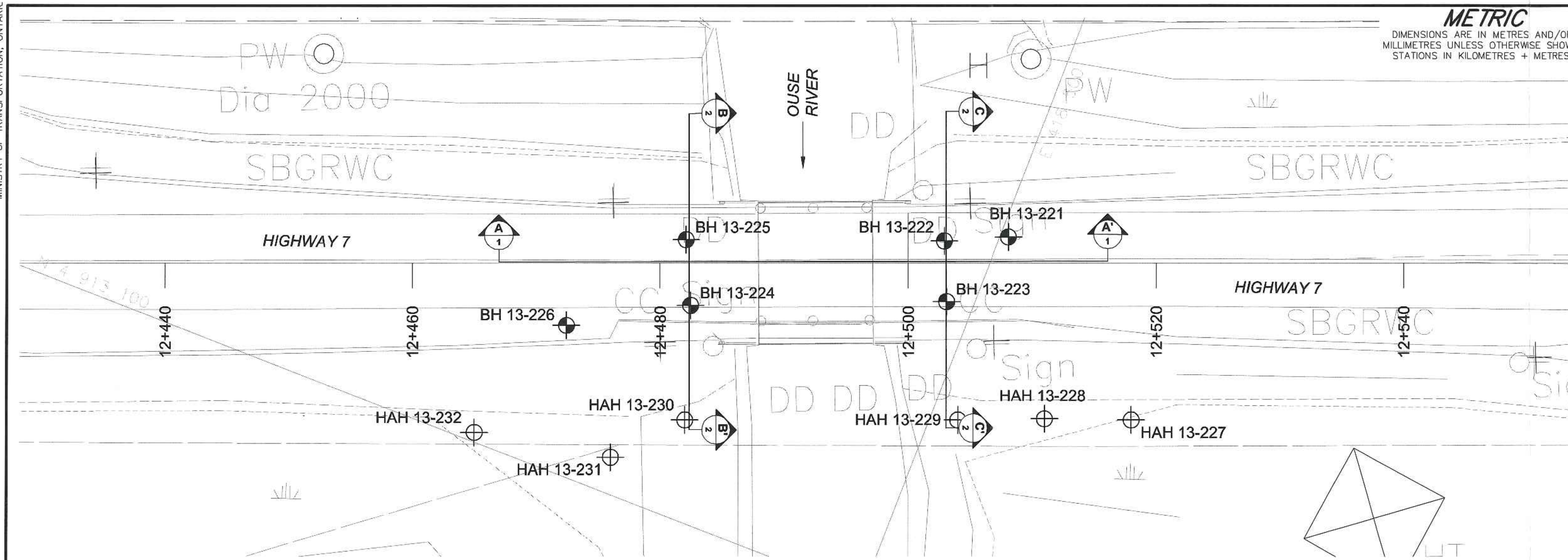
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 4127-10-01

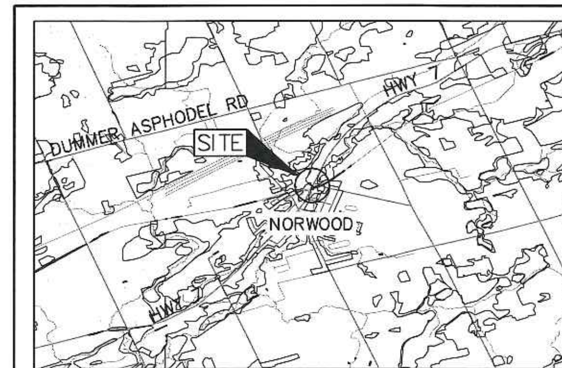
SHEET

OUSE RIVER BRIDGE
SITE 26-90
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates
Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



PLAN
SCALE
4 0 4 8 m



KEY PLAN
SCALE
1 0 1 2 km

LEGEND

- Borehole – Current Investigation
- Hand Augerhole – 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, measured June 3, 2013
- WL upon completion of or during drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
12-221	198.1	4913030.9	418500.7
12-222	198.1	4913028.8	418495.9
12-223	198.2	4913024.3	418497.9
12-224	198.1	4913016.6	418478.8
12-225	198.1	4913021.4	418476.5
12-226	198.1	4913011.5	418470.0

NOTES

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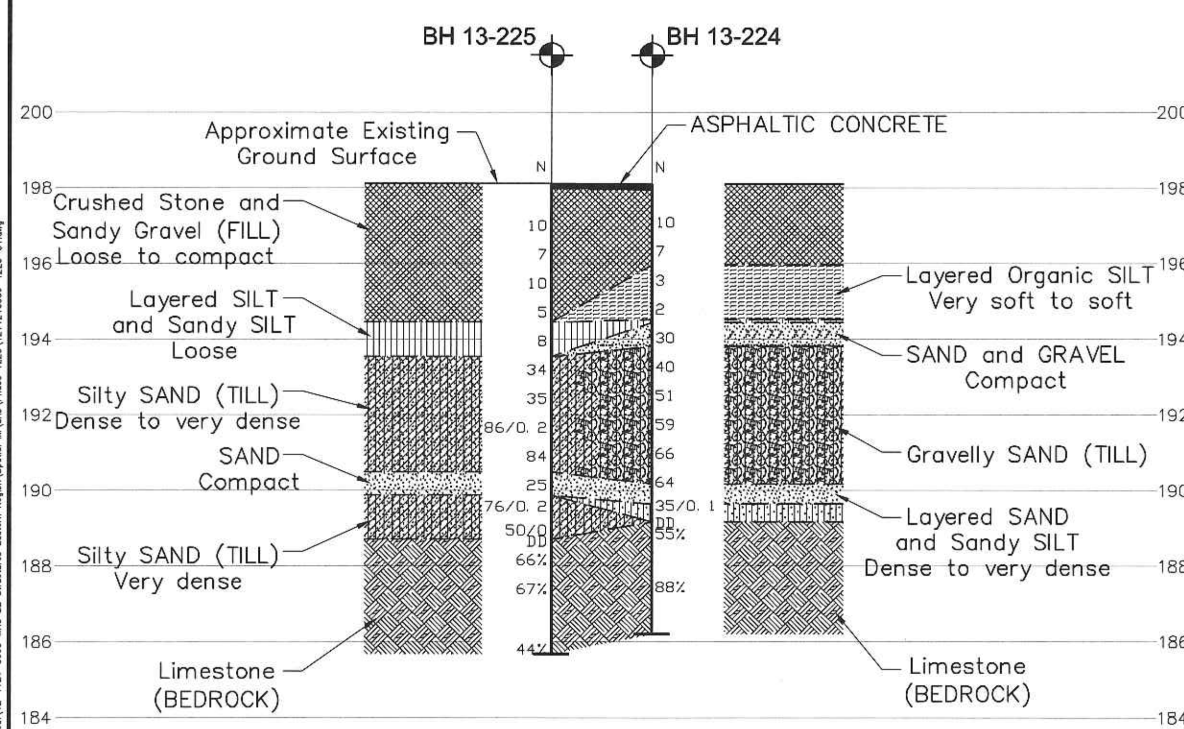
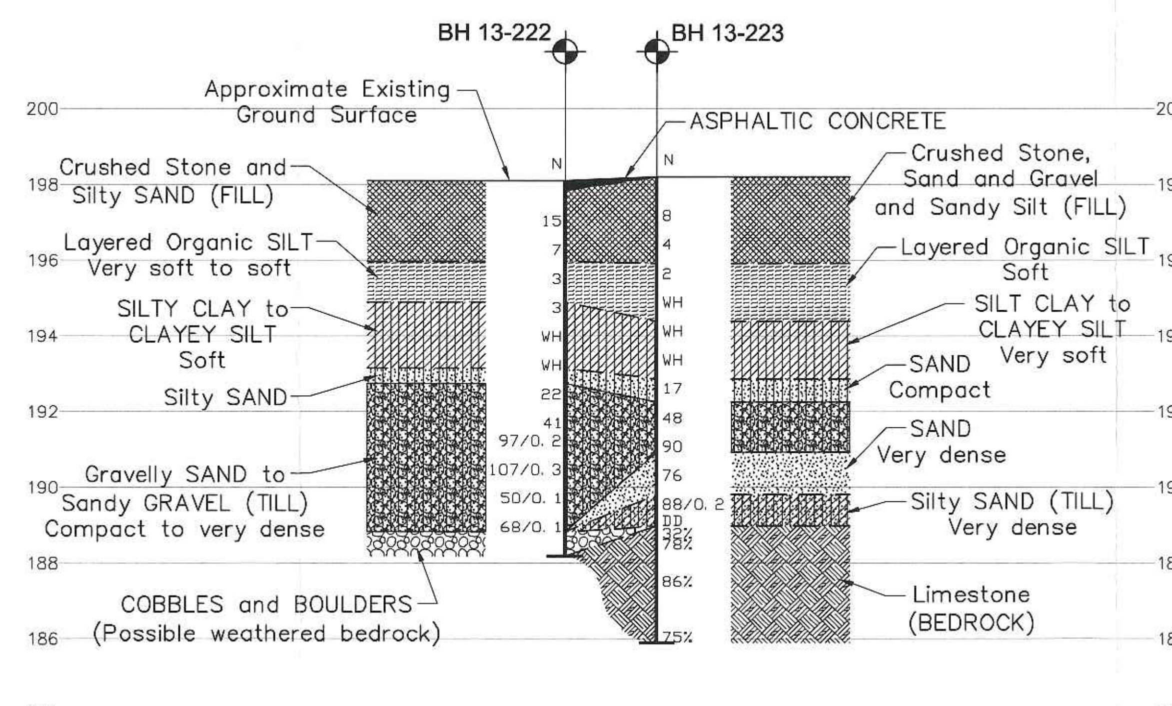


CROSS-SECTION B-B'
HORIZONTAL SCALE 4 0 4 8 m
VERTICAL SCALE 2 0 2 4 m



CROSS-SECTION C-C'
HORIZONTAL SCALE 4 0 4 8 m
VERTICAL SCALE 2 0 2 4 m

REFERENCE

Base plans provided in digital format by MMM Group Limited, drawing file no. B-PLAN 26-90.dwg, received March 27, 2013.

NO.	DATE	BY	REVISION

Geocres No. 31C-222

HWY. 7		PROJECT NO. 12-1121-0099-1220	DIST. Eastern
SUBM'D. SAT	CHKD. FJH	DATE: Mar. 2014	SITE: 26-90
DRAWN: JM	CHKD. SAT	APPD. FJH	DWG 2

APPENDIX A

Borehole 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

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
ll -	Parallel To	STR -	Stress Induced
OR -	Orthogonal		

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RECORD OF BOREHOLE No 13-221

SHEET 1 OF 1 **METRIC**

PROJECT 12-1121-0099-1220

G.W.P. 4127-10-01 LOCATION N 4913030.9 ;E 418500.7 ORIGINATED BY HEC

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

DATUM Geodetic DATE April 30, 2013 CHECKED BY SAT

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
198.1	GROUND SURFACE				
0.0	ASPHALTIC CONCRETE				
0.2	Crushed stone (FILL) Grey / Sand and gravel (FILL) Brown				
197.4 / 0.7	Crushed stone, trace silt (FILL) Compact Grey Moist	1	SS	19	
196.7 / 1.4	Sand and gravel, some silt, trace clay (FILL) Loose Grey Wet	2	SS	8	39 39 16 6
196.0 / 2.1	Sandy Organic SILT Firm Dark brown Wet	3	SS	5	
195.4	Organic SILT Soft Brown Wet				
195.2 / 2.9	SILT, some clay, trace sand, trace to some organic matter and roots Very loose Brown to grey Wet	4	SS	1	
194.4 / 3.7	SILTY CLAY, trace roots Soft Grey Wet	5	SS	WH	
		6	SS	PM	
193.2 / 4.9	Silty SAND Very loose Brown Wet				
192.6 / 5.5	SAND and GRAVEL, trace silt Compact Brown Wet	7	SS	22	
		8	SS	28	49 44 6 1
		9	SS	30	
190.3 / 7.8	Silty SAND, some gravel, trace clay, with cobbles (TILL) Very dense Grey-brown to grey Wet	10	SS	108	17 46 30 7
		11	SS	82/0.2	
		12	SS	50/0.3	
188.9 / 9.3	END OF BOREHOLE SAMPLER REFUSAL				

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (198–189); 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 (138.3); UNIT WEIGHT γ kN/m³

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

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RECORD OF BOREHOLE No 13-222

SHEET 1 OF 2 — METRIC

PROJECT 12-1121-0099-1220
G.W.P. 4127-10-01 — LOCATION N 4913028.8 ;E 418495.9 — ORIGINATED BY HEC
DIST Eastern HWY 7 — BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem) — COMPILED BY JM
DATUM Geodetic — DATE May 1, 2013 — CHECKED BY SAT

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS (GR SA SI CL)
198.1	GROUND SURFACE					
0.0	ASPHALTIC CONCRETE					
197.9 / 0.3	Crushed stone, trace to some silt (FILL) Compact Grey Moist	1	SS	15		
196.6 / 1.5	Silty sand, trace gravel (FILL) Loose Dark grey Moist to wet	2	SS	7		
196.0 / 2.1	Sandy Organic SILT Soft Dark brown Wet	3	SS	3	ORG = 11.7	
195.4 / 2.7	Organic SILT Soft Dark brown Wet				122.5; ORG = 13.7	
194.9 / 3.2	CLAYEY SILT, trace sand Soft Grey Wet	4	SS	3		
		5	SS	WH		0 3 71 26
		6	SS	WH		
193.2 / 5.0	Silty SAND Very loose Brown Wet					
192.7 / 5.4	Gravelly SAND to Sandy GRAVEL, trace silt and clay, with cobbles and boulders (TILL) Compact to very dense Brown Wet	7	SS	22		41 52 6 1
		8	SS	41		
		9	SS	97/0.2		
		10	SS	107/0.3		51 38 8 3
		11	SS	50/0.1		
		12	SS	68/0.1		
188.8 / 9.3	COBBLES and BOULDERS or possible weathered BEDROCK					
188.2						

Continued Next Page

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

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PROJECT 12-1121-0099-1220

RECORD OF BOREHOLE No 13-222 SHEET 2 OF 2 **METRIC**

G.W.P. 4127-10-01 LOCATION N 4913028.8 ;E 418495.9 ORIGINATED BY HEC

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

DATUM Geodetic DATE May 1, 2013 CHECKED BY SAT

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION — CONTINUED FROM PREVIOUS PAGE —	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
9.9	END OF BOREHOLE										

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

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RECORD OF BOREHOLE No 13-223

SHEET 1 OF 3

METRIC

PROJECT	12-1121-0099-1220
G.W.P.	4127-10-01
DIST	Eastern
HWY	7
DATUM	Geodetic
LOCATION	N 4913024.3 ;E 418497.9
BOREHOLE TYPE	Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core
DATE	May 7, 2013
ORIGINATED BY	HEC
COMPILED BY	JM
CHECKED BY	SAT

ELEV / DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
198.2	GROUND SURFACE					
0.0	Crushed stone (FILL), Grey					
0.3	Sand and gravel (FILL), Brown					
	Sandy silt, some clay (FILL), Loose, Grey, Moist	1	SS	8		2 31 52 15
197.0 / 1.2	Crushed stone (FILL), Loose, Grey, Moist					
196.7 / 1.5	Sandy silt, some clay and gravel (FILL), Very loose to loose, Brown, Wet	2	SS	4		
195.9	Organic SILT, Soft, Dark brown, Wet	3	SS	2		0 25 65 10
195.7 / 2.5						
195.3 / 2.9	Organic SILT, some sand and clay, Soft, Grey, Wet					
	Sandy Organic SILT, trace to some shells and roots, Very soft, Brown to grey-brown, Wet	4	SS	WH	ORG = 5.6	
194.4 / 3.8	SILTY CLAY, trace to some organic matter, Very soft, Grey, Wet	5	SS	WH		
193.9 / 4.3	CLAYEY SILT, trace sand, Very soft, Grey to brown, Wet	6	SS	WH		0 3 72 25
192.9 / 5.3	SAND, some gravel, Compact, Brown, Wet	7	SS	17		
192.3 / 5.9	Gravelly SAND, some silt, trace clay, with cobbles and boulders (TILL), Very dense, Brown to grey, Wet	8	SS	48		41 44 12 3
		9	SS	90		
190.9 / 7.3	SAND, trace silt and gravel, with cobbles and boulders, Very dense, Brown, Wet	10	SS	76		5 87 7 1
189.8 / 8.4	Silty SAND, some gravel, trace clay, with cobbles and boulders (TILL), Very dense, Grey, Wet	11	SS	88/0.2		
		12	RC	DD		
189.0 / 9.2		1	RC	REC 100%		RQD = 32%
		2	RC	REC 100%		RQD = 78%

Continued Next Page

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

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

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PROJECT 12-1121-0099-1220

RECORD OF BOREHOLE No 13-223

SHEET 2 OF 3

METRIC

G.W.P. 4127-10-01 LOCATION N 4913024.3 ;E 418497.9 ORIGINATED BY HEC

DIST Eastern HWY 7 BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core COMPILED BY JM

DATUM Geodetic DATE May 7, 2013 CHECKED BY SAT

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) (25 50 75)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —										
	Limestone (BEDROCK) Bedrock cored from depths of 9.2 m to 12.3 m For bedrock coring details refer to Record of Drillhole 13-223		2	RC	REC 100%		188				RQD = 78%
			3	RC	REC 100%		187				RQD = 86%
185.9			4	RC	REC 100%		186				RQD = 75%
12.3	END OF BOREHOLE										

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

RECORD OF DRILLHOLE: 13-223

PROJECT: 12-1121-0099-1220

LOCATION: N 4913024.3 ;E 418497.9

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: May 7, 2013

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	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES
		Continued from Record of Borehole 13-223		188.98				
	Rotary Drill NQ Core	LIMESTONE Slightly weathered to fresh, thinly to medium bedded, fine grained, non-porous, medium strong to strong, grey, nodular, with black shale partings and laminate to thin interbeds		9.22	1			
10					2		JN	
		- Broken core from 10.5 m to 10.6 m						
11					3		JN JN	
12				185.91 12.29	4			
		END OF DRILLHOLE						
13								
14								
15								
16								
17								
18								
19								

Column headings also printed: RECOVERY (TOTAL CORE %, SOLID CORE %), R.Q.D. %, FRACT. INDEX PER, DIP w.r.t. CORE AXIS, Jr, Ja, HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}), WEATH-ERING INDEX (W1 W2 W3 W4 W5 W6), FEATURES

DEPTH SCALE

1 : 50

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LOGGED: HEC

CHECKED: SAT

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RECORD OF BOREHOLE No 13-224

SHEET 1 OF 3

METRIC

PROJECT 12-1121-0099-1220

G.W.P. 4127-10-01

DIST Eastern HWY 7

DATUM Geodetic

LOCATION N 4913016.6 ;E 418478.8

BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core

DATE May 6, 2013

ORIGINATED BY HEC

COMPILED BY JM

CHECKED BY SAT

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
198.1	GROUND SURFACE							
0.0	ASPHALTIC CONCRETE				198			
	Crushed stone (FILL) Grey							
197.7 0.4	Sand and gravel (FILL) Compact Brown Moist	1	SS	10				
197.0 1.1	Crushed stone (FILL) Loose Grey-brown Moist to wet	2	SS	7	197			
196.0 2.1	Sandy Organic SILT Soft Dark brown to black Wet	3	SS	3	196	○	ORG = 4.2	
195.5 2.6	Organic SILT Soft Dark brown Wet					○	ORG = 6.4	
195.1 3.1	Sandy Organic SILT, trace to some shells Very loose Dark brown to black Wet	4	SS	2	195	○		
194.5 3.7	SILT, some clay, trace sand Very loose Brown Wet							
193.8 4.3	SAND and GRAVEL Compact to dense Brown Wet	5	SS	30	194			
	Gravelly SAND, some silt, trace clay, with cobbles and boulders (TILL) Dense to very dense Grey-brown to grey Wet	6	SS	40	193	○		29 47 19 5
		7	SS	51				
		8	SS	59	192			
		9	SS	66	191	○		35 37 23 5
190.2 7.9	Silty SAND, trace gravel and clay Very dense Grey-brown Wet	10	SS	64	190	○		4 61 31 4
189.9		11	SS	35/0.1		○		
189.6 8.5	SAND Compact Brown Wet	12	RC	DD				
189.2 8.9	Sandy SILT, trace gravel and clay, occasional cobbles Very dense Grey Wet	1	RC	REC 98%	189			RQD = 55%

Continued Next Page

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

GTA-MTO 001 1211210099.GPJ GAL-GTA.GDT 02/13/14 JM

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RECORD OF BOREHOLE No 13-224

METRIC

PROJECT 12-1121-0099-1220			
G.W.P. 4127-10-01	LOCATION N 4913016.6 ;E 418478.8		ORIGINATED BY HEC
DIST Eastern	HWY 7	BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core	COMPILED BY JM
DATUM Geodetic	DATE May 6, 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* ---										
	Limestone (BEDROCK) Bedrock cored from depths of 8.9 m to 11.9 m For bedrock coring details refer to Record of Drillhole 13-224		1	RC	REC 98%		188				RQD = 55%
			2	RC	REC 100%		187				RQD = 88%
186.2 11.9	END OF BOREHOLE										

GTA-MTO 001 1211210099.GPJ GAL-GTA.GDT 02/13/14 JM

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

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

PROJECT: 12-1121-0099-1220

RECORD OF DRILLHOLE: 13-224

LOCATION: N 4913016.6 ;E 418478.8

DRILLING DATE: May 6, 2013

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

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.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES
		Continued from Record of Borehole 13-224	189.18			
9	Rotary Drill NQ Core	LIMESTONE Slightly weathered to fresh, thinly to medium bedded, fine to coarse grained, non-porous, medium strong, grey, nodular, with black shale partings and laminate to thin interbeds - Broken core from 9.0 m to 9.1 m - Broken core from 9.2 m to 9.3 m	8.92	1	JN JN JN JN JN JN JN	
10						
11				2	JN JN JN JN JN JN	
12		END OF DRILLHOLE	186.21 11.89			
13						
14						
15						
16						
17						
18						

Column headers also include: SYMBOLIC LOG; FLUSH RETURN; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER; DIP w.r.t. CORE AXIS; Jr; Ja; HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WEATHERING INDEX (W1–W6); FEATURES.

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: HEC

CHECKED: SAT

RECORD OF BOREHOLE No 13-225

SHEET 1 OF 3

METRIC

PROJECT 12-1121-0099-1220			
G.W.P. 4127-10-01	LOCATION N 4913021.4 ;E 418476.5	ORIGINATED BY HEC	
DIST Eastern HWY 7	BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core	COMPILED BY JM	
DATUM Geodetic	DATE May 2, 2013	CHECKED BY SAT	

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

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR SA SI CL / REMARKS
198.1	GROUND SURFACE					
0.0	ASPHALTIC CONCRETE				198	
0.2	Crushed stone (FILL) Grey / Crushed sand and gravel (FILL) Grey-brown Moist	1	SS	10	197	
196.7 / 1.4	Sandy gravel, trace silt (FILL) Loose to compact Grey-brown Moist to wet	2	SS	7	196	
		3	SS	10		57 35 7 1
195.2 / 2.9	Crushed stone, trace silt (FILL) Loose Grey Wet	4	SS	5	195	
194.4 / 3.7	SILT, some clay, trace sand, trace roots Loose Grey-brown Wet	5	SS	8	194	0 7 83 12
193.8 / 4.3	Sandy SILT, some gravel Compact Brown Wet					
193.5 / 4.6	Silty SAND, some gravel, trace clay to Gravelly SAND, some silt, trace clay (TILL) Dense to very dense Grey-brown to grey Wet	6	SS	34	193	
		7	SS	35		14 42 39 5
		8	SS	86/0.2	192	
		9	SS	84	191	38 39 20 3
190.5 / 7.6	SAND, some silt, trace gravel Compact Grey-brown wet	10	SS	25	190	8 73 17 2
189.9 / 8.2	Silty SAND, some gravel, trace clay, with cobbles (TILL) Very dense Grey to grey-brown Wet	11	SS	76/0.2		12 50 33 5
		~~12~~	~~SS~~	~~50/0~~	189	
		13	RC	DD		
188.7 / 9.4		1	RC	REC 100%		RQD = 66%

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 13-225

METRIC

PROJECT 12-1121-0099-1220			
G.W.P. 4127-10-01	LOCATION N 4913021.4 ;E 418476.5		ORIGINATED BY HEC
DIST Eastern	HWY 7	BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core	COMPILED BY JM
DATUM Geodetic	DATE May 2, 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 ---										
	Limestone (BEDROCK) Bedrock cored from depths of 9.4 m to 12.4 m For bedrock coring details refer to Record of Drillhole 13-225		1	RC	REC 100%		188				RQD = 66%
			2	RC	REC 100%		187				RQD = 67%
185.7			3	RC	REC 100%		186				RQD = 44%
12.4	END OF BOREHOLE										

Legend: 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

PROJECT: 12-1121-0099-1220

RECORD OF DRILLHOLE: 13-225

LOCATION: N 4913021.4 ;E 418476.5

DRILLING DATE: May 2, 2013

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

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	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES
		Continued from Record of Borehole 13-225		188.71				
10	Rotary Drill, NQ Core	LIMESTONE Slightly weathered to fresh, thinly to medium bedded, fine to coarse grained, non-porous, medium strong, grey, nodular, with black shale partings and laminated to thin interbeds - Broken core from 9.5 m to 9.6 m - Broken core from 9.7 m to 9.8 m		9.40	1		JN JN	
11					2		JN JN JN JN	
12				185.69 12.42	3		JN JN JN JN JN JN	
		END OF DRILLHOLE						
13								
14								
15								
16								
17								
18								
19								

Other column headings: RECOVERY (TOTAL CORE %, SOLID CORE %), R.Q.D. %, FRACT. INDEX PER 0.25 m, DIP w.r.t. CORE AXIS, Jr, Ja, HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}), WEATHERING INDEX (W1 W2 W3 W4 W5 W6), FEATURES

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: HEC

CHECKED: SAT

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

RECORD OF BOREHOLE No 13-226

SHEET 1 OF 1

METRIC

PROJECT 12-1121-0099-1220

G.W.P. 4127-10-01 LOCATION N 4913011.5 ;E 418470.0 ORIGINATED BY HEC

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

DATUM Geodetic DATE May 3, 2013 CHECKED BY SAT

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
198.1	GROUND SURFACE
0.0	ASPHALTIC CONCRETE
0.1	Sand and gravel, trace silt and clay (FILL) Compact to loose Brown Moist to wet
196.0 / 2.1	Sand and gravel, trace to some organics (FILL) Compact Grey Wet
195.2 / 2.9	Silty SAND, some gravel, trace clay, with cobbles (TILL) Loose to very dense Grey Wet
190.4 / 7.7	END OF BOREHOLE

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	12				
2	SS	9				
3	SS	20				
4	SS	7	23	44	25	8
5	SS	22				
6	SS	70	19	43	32	6
7	SS	50/0.1	22	39	33	6
8	SS	55/0.1				
9	SS	50/0.1				
10	SS	32/0.1				

Elevation scale: 198, 197, 196, 195, 194, 193, 192, 191

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

NOTES:

1. Water level in well screen at a depth of 1.4 m below ground surface (Elev. 196.7 m), measured on June 3, 2013.

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

TABLE A1

RECORD OF HAND AUGERHOLES

Hand Augerhole Number	Depth (metres)	Description
13-227	0.00 – 0.15	Water
	0.15 – 0.75	Fibrous organic material, brown, wet
	0.75 – 1.00	Silty sand, fine-grained, compact, brown, moist to wet
	1.00	End of Hand Augerhole
13-228	0.00 – 0.19	Water
	0.19 – 0.70	Fibrous organic material, brown, wet
	0.70 – 1.00	Silty sand, fine-grained, compact, brown, moist to wet
	1.00	End of Hand Augerhole
13-229	0.00 – 0.35	Sandy topsoil, contains organic matter, brown
	0.35 – 0.95	Silty sand, fine-grained, compact, brown, moist to wet
	0.95 – 1.20	Silty sand, fine-grained, compact, grey, moist to wet
	1.20	End of Hand Augerhole
13-230	0.00 – 0.45	Fibrous organic material, brown, wet
	0.45 – 1.15	Sand with gravel, trace clay, coarse-grained, compact, brown, wet
	1.15	End of Hand Augerhole
13-231	0.00 – 0.10	Water
	0.10 – 0.88	Fibrous organic material, brown, wet
	0.88 – 1.35	Silty sand and gravel, trace clay, compact, brown, wet
	1.35	End of Hand Augerhole
13-232	0.00 – 0.09	Water
	0.09 – 0.14	Silty sand and gravel, compact, brown, wet
	0.14 – 0.44	Fibrous organic material, brown, wet
	0.44 – 1.10	Silty sand and gravel, trace clay, contains trace to some organic matter, brown, wet
	1.10 – 1.40	Silty sand, fine-grained, compact, grey, moist to wet
	1.40	End of Hand Augerhole

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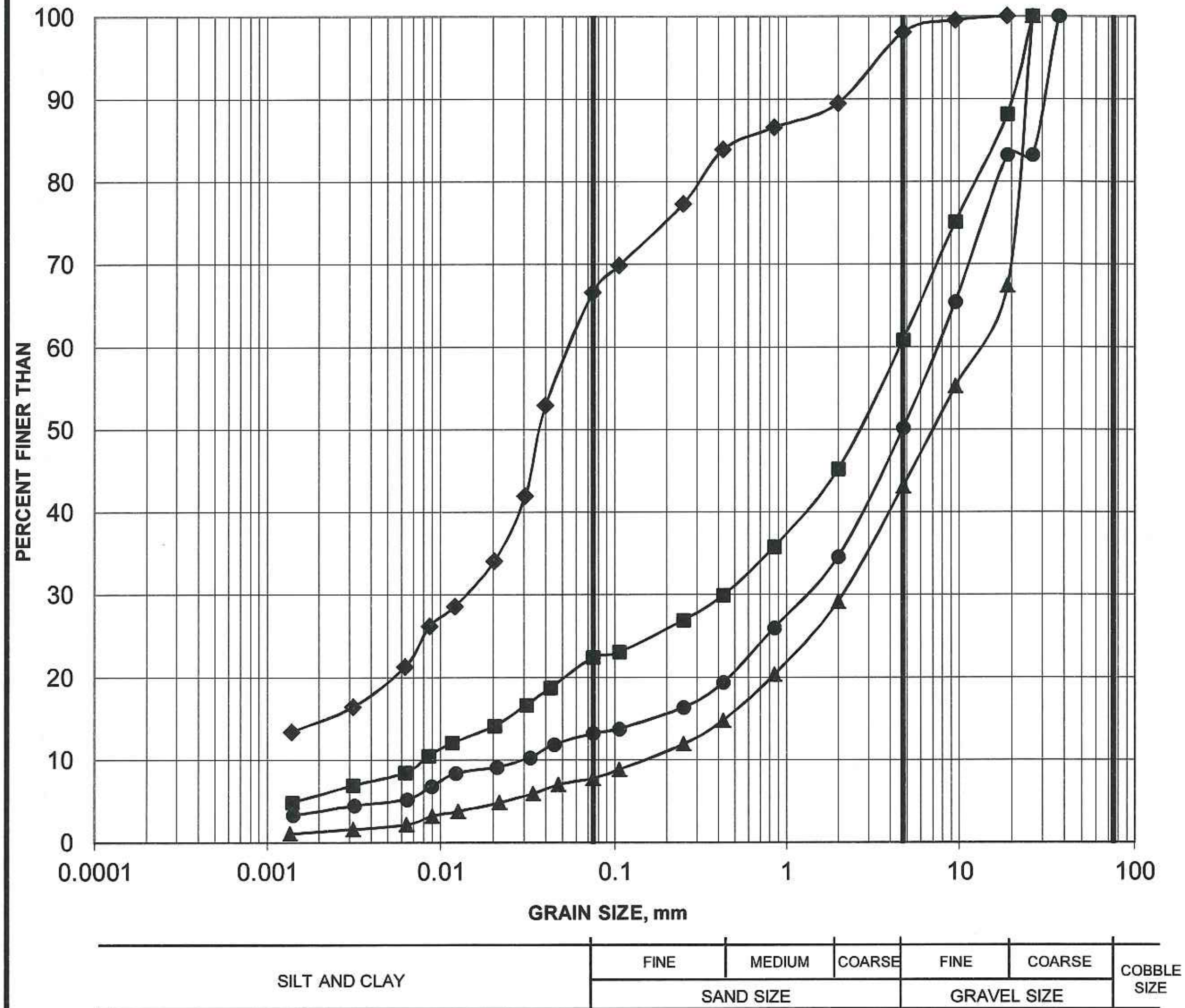
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION	FIGURE B1

EMBANKMENT FILL



Borehole	Sample	Depth (m)
13-221	2	1.52-2.13
13-223	1	0.76-1.37
13-225	3	2.29-2.90
13-226	2	1.52-2.13

Created by: MI
Checked by: CNM

Project: 12-1121-0099/1220

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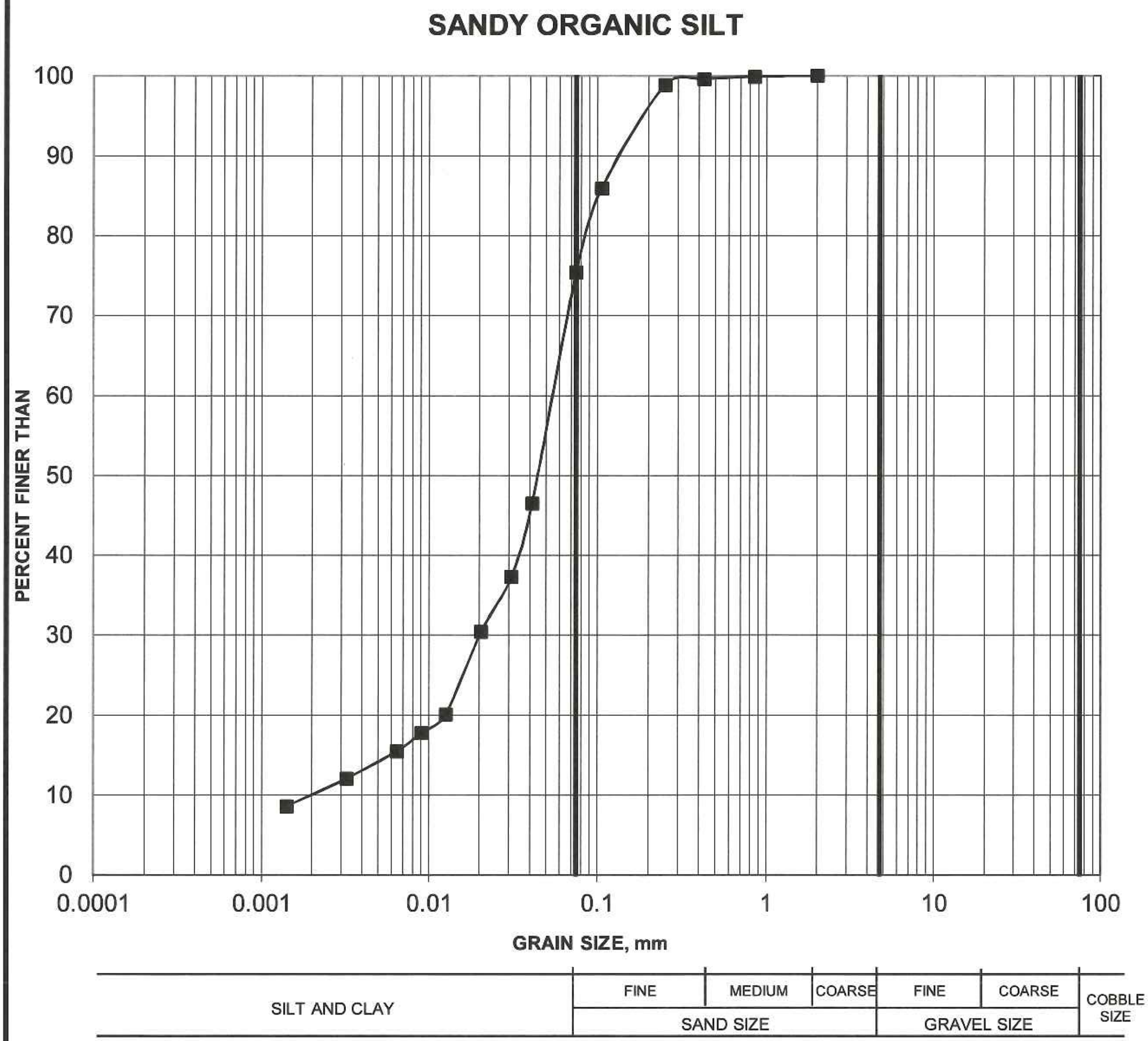
GRAIN SIZE DISTRIBUTION

FIGURE B2



Borehole	Sample	Depth (m)
13-223	3A	2.44-2.90

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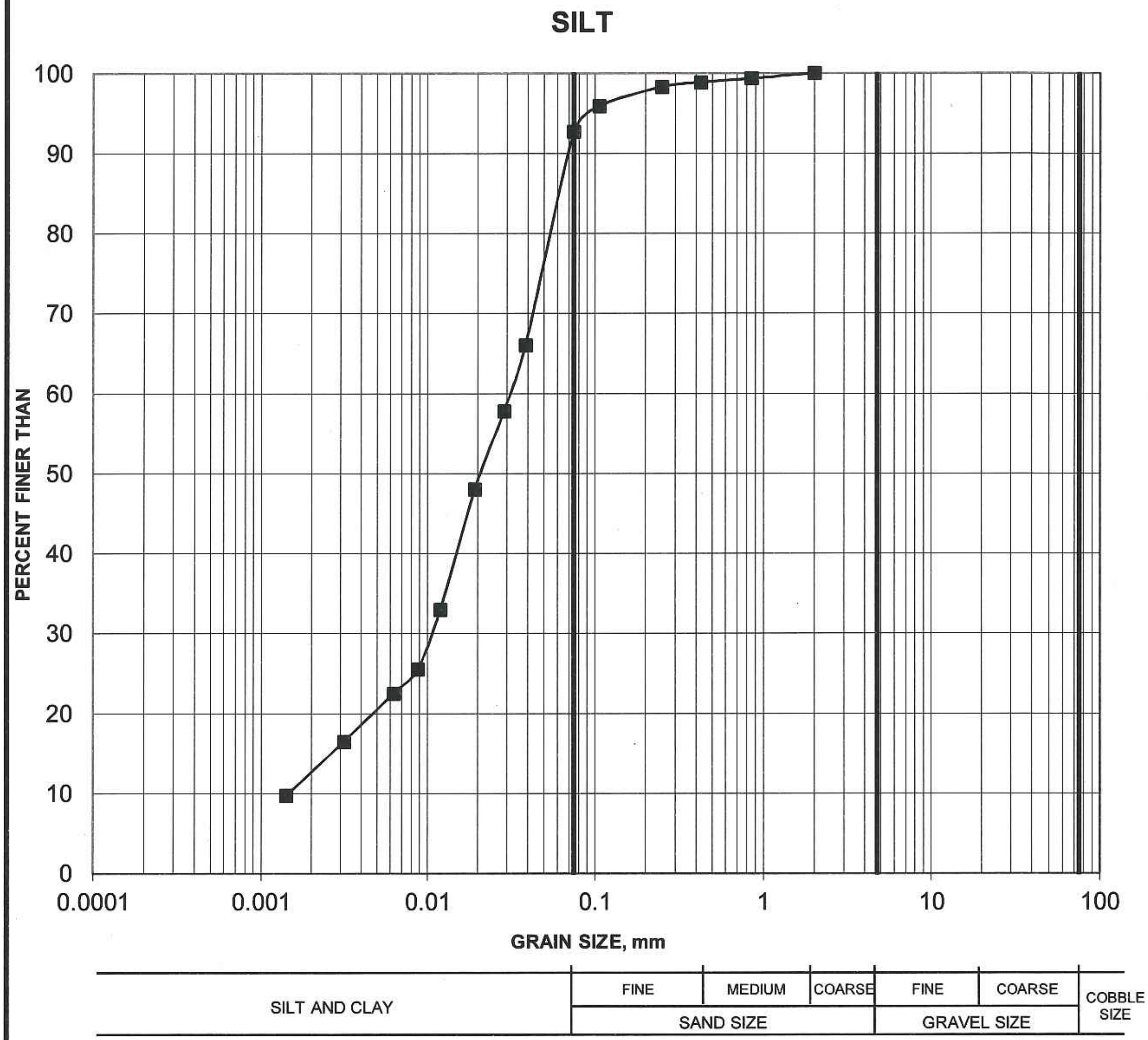
Created by: MI

Checked by: CNM

GRAIN SIZE DISTRIBUTION

FIGURE B3



Borehole	Sample	Depth (m)
13-225	5	3.81-4.42

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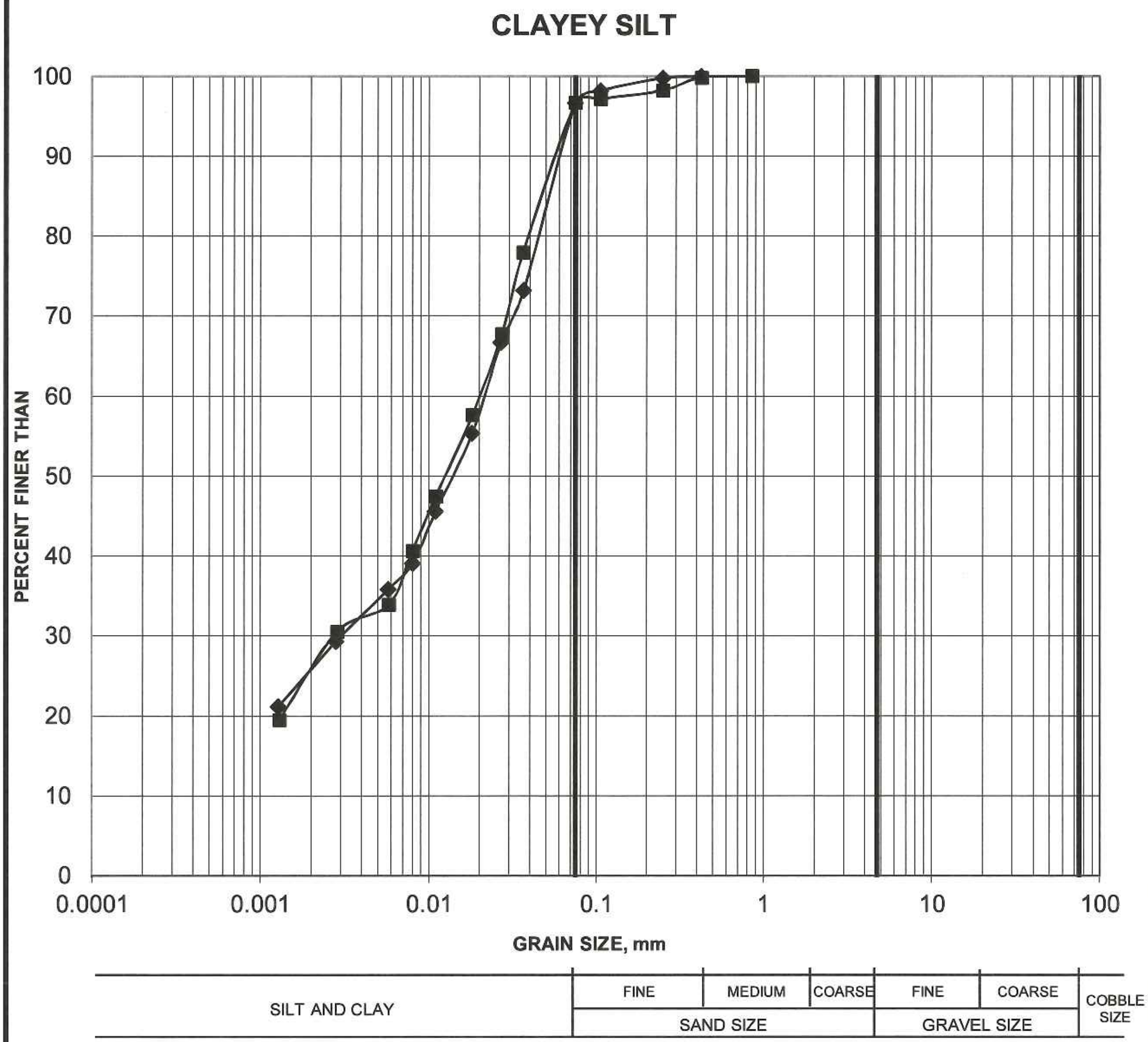
Created by: MI

Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B4

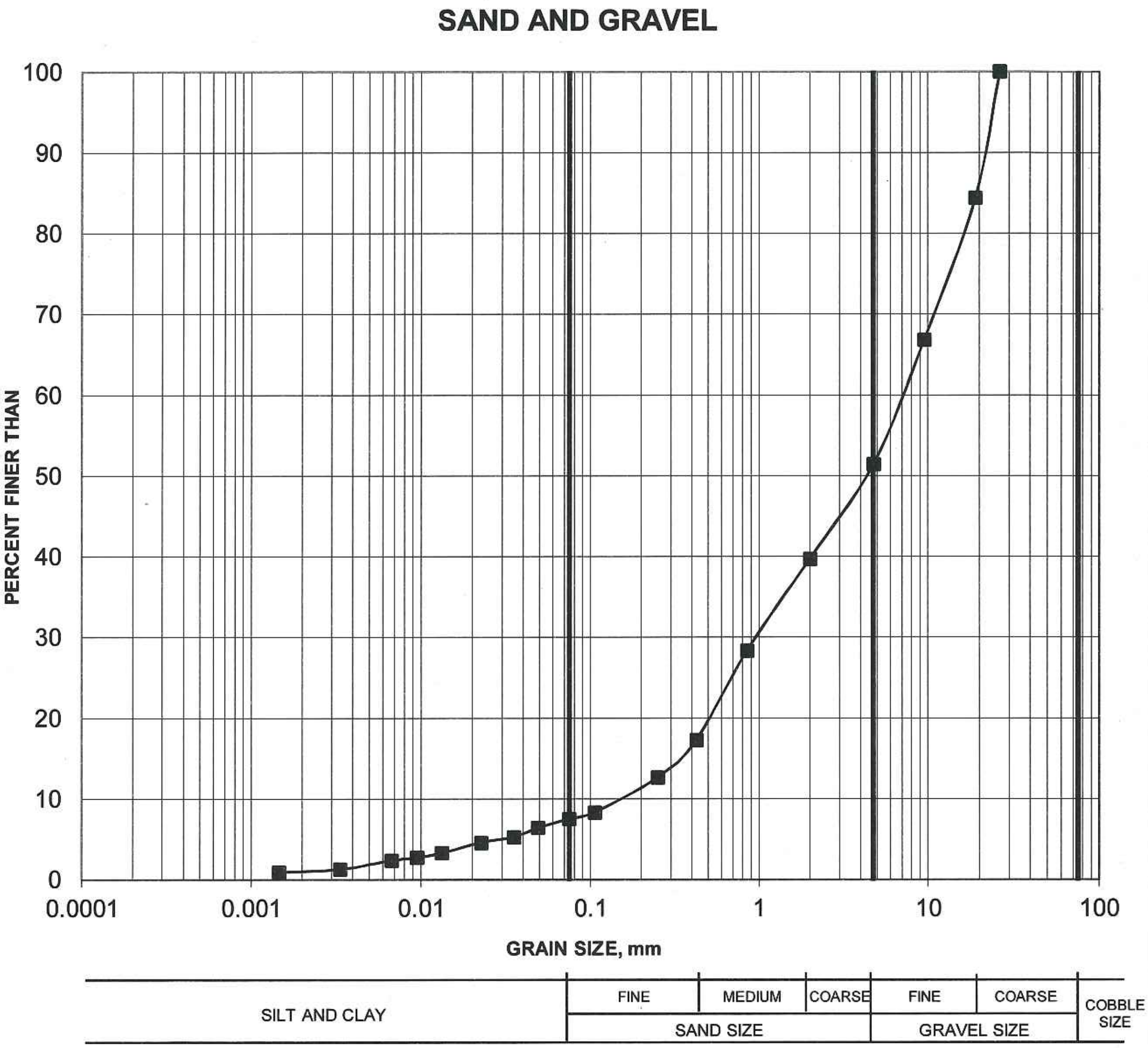


Borehole	Sample	Depth (m)
13-222	5	3.81-4.42
13-223	6	4.57-4.95

Project: 12-1121-0099/1220 **Golder Associates**

Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B5



Borehole	Sample	Depth (m)
13-221	8	6.10-6.71

Project: 12-1121-0099/1220

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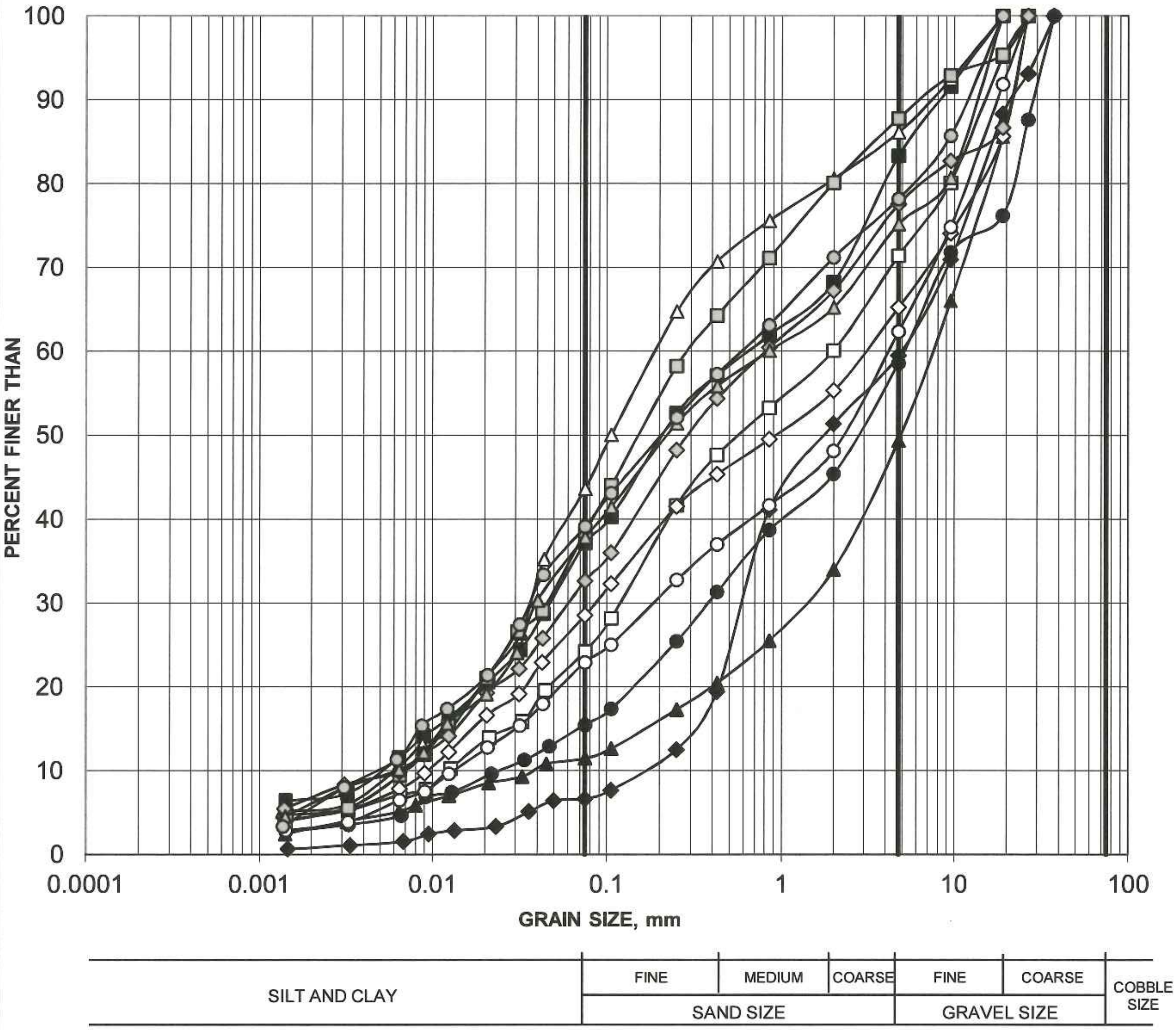
Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B6

GLACIAL TILL



Borehole	Sample	Depth (m)
13-221	10	7.62-8.18
13-222	7	5.34-5.95
13-222	10	7.62-7.90
13-223	8	6.10-6.71
13-224	6	4.57-5.18
13-224	9	6.86-7.47
13-225	7	5.34-5.95
13-225	9	6.86-7.47
13-225	11	8.38-8.63
13-226	4	3.05-3.66
13-226	6	4.57-4.73
13-226	7	5.34-5.95

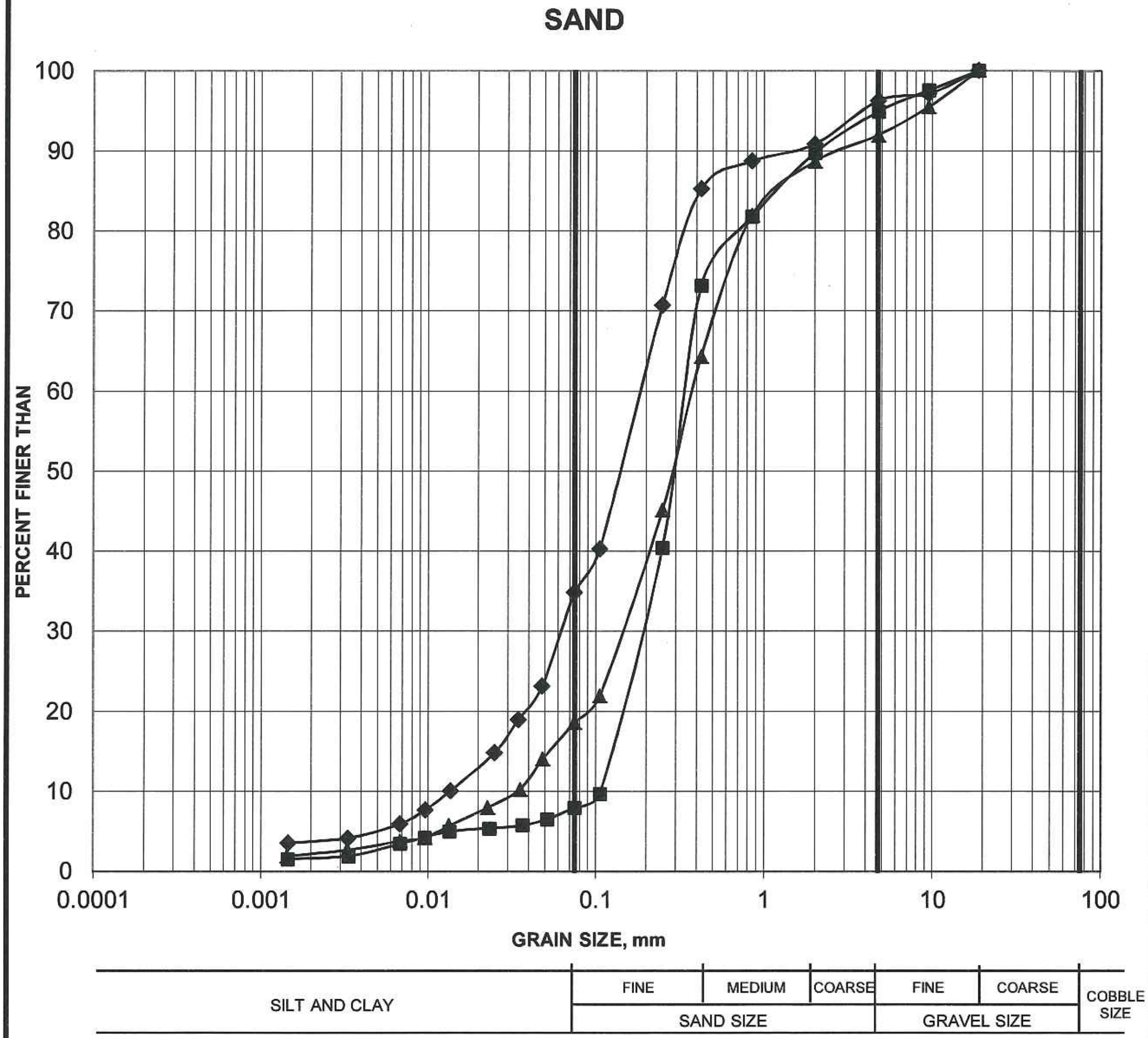
Created by: MI

Checked by: CNM

Project: 12-1121-0099/1220

Golder Associates

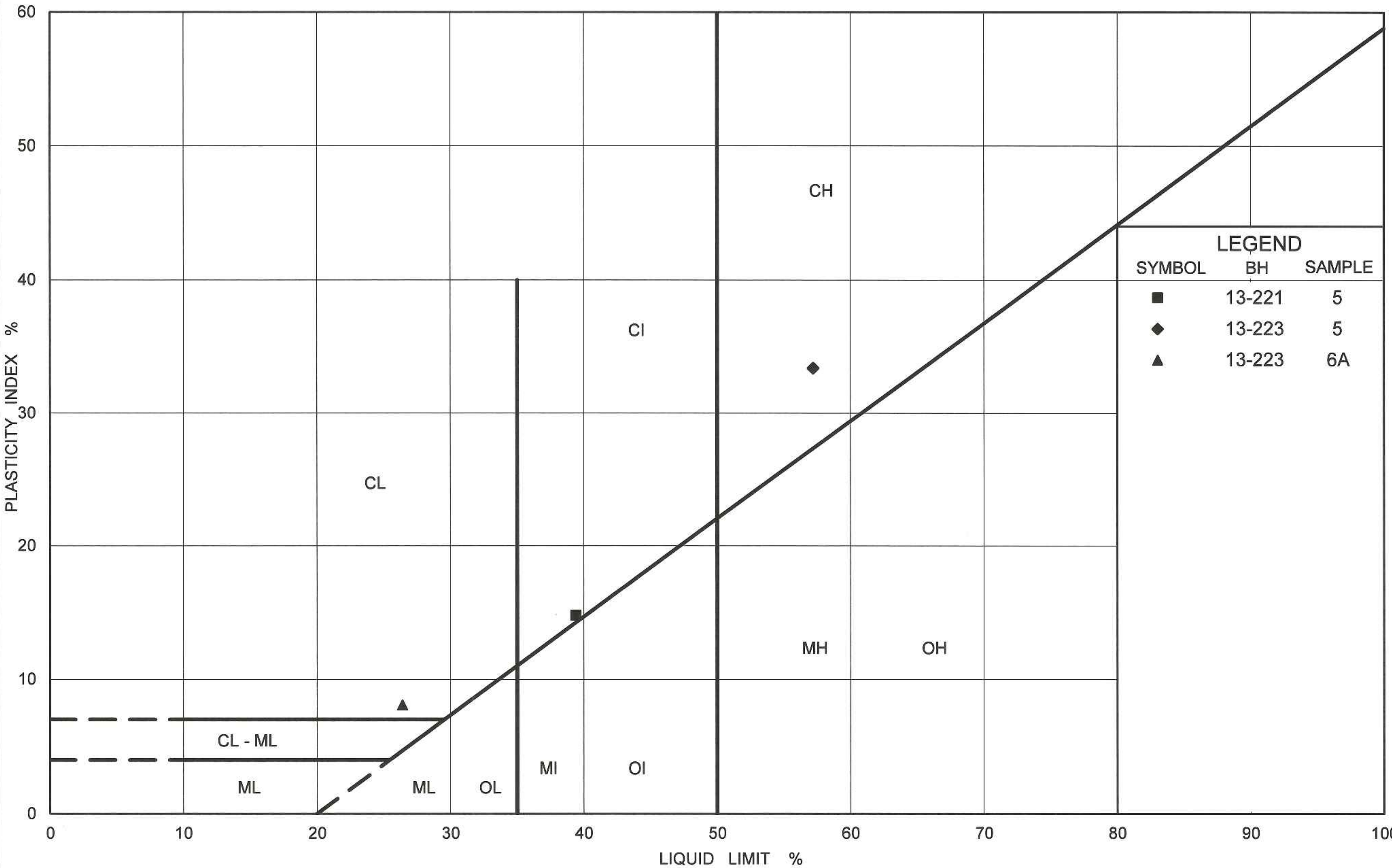
GRAIN SIZE DISTRIBUTION	FIGURE B7



Borehole	Sample	Depth (m)
13-223	10	7.62-8.20
13-224	10A	7.93-8.23
13-225	10	7.62-8.23

Project: 12-1121-0099/1220 **Golder Associates**

Created by: MI
Checked by: CNM



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PLASTICITY CHART

FIG No. B8

Project No. 12-1121-0099/1220

Compiled By : MI Checked By : CNM