



April 2016

REPORT ON

**Foundation Investigation and Design
Un-Named Creek Culvert Replacement
Site No. 3-728c
Highway 417, 130 m West of March Road
Ottawa, Ontario
W.P. 4168-11-01**

Submitted to:
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REPORT



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PART A

**FOUNDATION INVESTIGATION REPORT
UN-NAMED CREEK CULVERT REPLACEMENT
SITE 3-728C
HIGHWAY 417, 130 M WEST OF MARCH ROAD
OTTAWA, ONTARIO
W.P. 4168-11-01**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by MMM Group Ltd. (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the design of bridge and culvert replacements at various locations in the Eastern Region of Ontario as part of the 22 Structures MEGA 2 project.

This report presents the results of the foundation investigation conducted for the replacement of the Un-Named Creek culvert (MTO Structure Site No. 3-728c under W.P. 4168-11-01), located on Highway 417 about 130 m west of March Road in Ottawa, Ontario.

Initially, the culvert replacement at this site was planned to be undertaken as a Design-Build project. It is now understood that MMM will be completing the detailed design of the culvert replacement. The purpose of the foundation investigation was to assess the subsurface conditions for the proposed culvert replacement by drilling boreholes and carrying out in-situ testing and laboratory testing on selected soil samples.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated April 2012. In addition, Golder's letter dated March 11, 2015 described the work plan for additional foundation engineering services for detail design.

The work was carried out in accordance with Golder's Quality Control Plan dated August 2012.



2.0 SITE DESCRIPTION

The Un-Named Creek culvert is located on Highway 417 about 130 m west of March Road in Ottawa, Ontario. The existing culvert (Structure Site No. 3-728c) is located at about Station 18+380 m.

The existing culvert consists of a sectional plate, corrugated steel pipe arch measuring about 3.1 m wide by 2.0 m high. The north portion of the culvert that extends beneath the westbound lanes of Highway 417 was constructed in the 1960s. The culvert was extended to the south beneath the eastbound lanes of Highway 417 in 1993 and has an overall length of about 86 m. It is understood that the culvert has corroded and is perforated at many locations along the spring line with some loss of backfill into the culvert near the mid span. The existing culvert inverts are at about Elevations 111.9 and 112.6 m at the north and south ends, respectively, with flow in the culvert from south to north. The depth of water within the culvert was less than about 200 mm at the time of the field investigation in July, 2014. The width of the water course was estimated to be about 1.0 to 1.5 m.

The existing pavement grade at the culvert location is at about Elevation 118.4 m. In this area, Highway 417 is typically two lanes wide in each direction (i.e., a separated four-lane highway). At the existing culvert crossing, the overall lane configuration also includes two acceleration lanes: one north of the highway, connecting March Road north to Highway 417 west, and one south of the highway (part of the off-ramp), connecting March Road south to Highway 417 east. The existing embankment slopes at the culvert locations are about 4 to 5 m in height and are oriented between about 2 horizontal to 1 vertical and 3 horizontal to 1 vertical (i.e., 2H:1V and 3H:1V). Based on visual observation at the time of the site investigation, the existing embankment slopes appear to be performing satisfactorily.



3.0 INVESTIGATION PROCEDURES

The subsurface investigation for the culvert replacement was carried out in two stages. During the first stage, a preliminary investigation was carried out between July 14 and 16, 2014 for a design build project. A second stage of investigation for a detail design was carried out on December 8, 2015. Overall, five boreholes (numbered 14-321 to 14-324, inclusive, and 15-325) were advanced at the locations shown on Drawing 1. The boreholes were advanced as follows:

- Boreholes 14-321 and 14-324 were advanced near the culvert ends at the toes of the Highway 417 embankments using portable drilling equipment supplied and operated by OGS Inc. of Almonte, Ontario. The boreholes were advanced using near-continuous sampling procedures to depths of up to about 12.2 m below the existing ground surface, to about Elevation 101 m and 10 m below the culvert invert level.
- Boreholes 14-322 and 14-323 were advanced through the existing Highway 417 westbound and eastbound embankments, respectively, using 108 mm inside diameter continuous-flight hollow-stem augers on a truck-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to depths of up to about 18.9 m (Elevation 99 m) below the existing ground surface.
- Borehole 15-325 was advanced within the median of Highway 417 using 108 mm inside diameter continuous-flight hollow-stem augers on a track-mounted drill rig, supplied and operated by CCC Drilling of Ottawa, Ontario. The borehole was advanced to a depth of 11.7 m (Elevation 105.4 m) below the existing ground surface.

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

Where appropriate, the SPT sampling was supplemented with in-situ shear vane testing. An MTO “N”-size vane was used to measure the undrained shear strength of the cohesive soils encountered at Boreholes 14-322, 14-323, and 15-325. To carry out in-situ shear vane testing at Boreholes 14-321 and 14-324, a ‘B’ sized vane was used to accommodate the narrower casing size used with the portable drill rig.

A standpipe piezometer was installed in Borehole 14-321 to monitor the groundwater level at the site. The standpipe consists of a 32 mm diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within silica sand backfill and sealed by a section of bentonite pellet backfill. The water level in the standpipe piezometer was measured on August 13, 2014 and on December 8, 2015.

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

The field work was supervised by members of Golder’s technical and engineering staff, who located the boreholes, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled, and transported to Golder’s laboratories in Ottawa and Mississauga for further examination. Index and classification tests consisting of grain size distribution, Atterberg limits, and water content testing were carried out on selected soil samples. Consolidation testing was carried out on one sample obtained from Borehole 14-322 at about Elevation 106.2 m. All of the laboratory tests were carried out to MTO and/or ASTM standards as appropriate.



FOUNDATION REPORT UN-NAMED CREEK CULVERT REPLACEMENT - HIGHWAY 417

Prior to drilling, the boring locations were staked and surveyed by Golder personnel using a Trimble R8 GPS unit. The boreholes and locations, including MTM NAD83 (Zone 9) northing and easting coordinates and ground surface elevations referenced to Geodetic datum, are summarized in the following table and are shown on Drawing 1.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
14-321	North end of culvert	5019397.0	339451.6	113.2
14-322	Highway 417 westbound lane shoulder on the east side of the culvert	5019378.6	339445.1	118.1
14-323	Highway 417 eastbound S-E off-ramp on the east side of the culvert	5019328.9	339427.4	118.0
14-324	South end of culvert	5019312.8	339422.6	113.7
15-325	Median of HWY 417	5019353.0	339435.4	117.1



4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The study area for this assignment lies within the minor physiographic region known as the Ottawa Valley Clay Plain, as delineated in *The Physiography of Southern Ontario*¹ that lies within the major physiographic region of the Ottawa-St. Lawrence Lowland.

The Ottawa Valley Clay Plain region is characterized by relatively thick deposits of sensitive marine clay, silt and silty clay that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock.² This region is underlain by a series of sedimentary rocks, consisting of sandstones, dolostones, limestones and shales that are, in turn, underlain by igneous and metamorphic bedrock of the Precambrian Shield.

4.2 Site Stratigraphy

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

A soil stratigraphy section projected along the centreline of the existing culvert area is shown on Drawing 1. The stratigraphic boundaries shown on the Record of Borehole sheets and on Drawing 1 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 existing culvert consist of sand and gravel embankment fill overlying sand to silty sand, which is underlain by a deposit of sensitive clayey silt to silty clay followed by glacial till. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

About 100 mm of topsoil was encountered at Boreholes 14-321 and 14-324, which were advanced near the culvert ends at the toes of the Highway 417 embankments.

4.2.2 Pavement Structure and Embankment Fill

The pavement structure within Highway 417 was penetrated within the westbound lane shoulder at Borehole 14-322 and the eastbound S-E on-ramp at Borehole 13-323. At Borehole 14-323, the pavement structure for Highway 417 consists of about 190 mm of asphaltic concrete overlying about 200 mm of gravelly sand base course. At Borehole 14-322, at the S-E ramp, the 760 mm gravelly base course is directly at the ground surface. The granular base is underlain by about 3.0 to 4.2 m of subbase/embankment fill. At these locations, the subbase/embankment fill generally consists of varying compositions of sand and gravel, containing some silt. The lower portion of the embankment fill contains limestone fragments (i.e., rockfill).

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.

² Belanger, J.R. "Urban Geology of Canada's National Capital Area", in *Urban Geology of Canadian Cities*, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White, 1998.



Embankment fill was encountered at surface in the median area, at the location of Borehole 15-325, and generally consists of sand with some silt and pockets of silty clay.

The embankment fill was fully penetrated to depths of about 3.8 to 4.6 m (Elevations 112.5 to 113.5 m) at Boreholes 14-322, 14-323, and 15-325.

Standard Penetration Test (SPT) “N” values measured within the embankment fill range from 8 to 52 blows per 0.3 m of penetration, indicating a loose to very dense state of packing.

The results of grain size distribution testing carried out on three samples of the embankment fill are provided on Figure B1 in Appendix B. The measured water content of selected samples of the embankment fill ranges from approximately 7 to 12 percent.

4.2.3 Sand and Silty Sand

About 1.7 to 3.8 m of sand and silty sand was encountered below the embankment fill or surficial topsoil, where encountered, at the borehole locations. The sand and silty sand was fully penetrated to elevations of about 109.4 to 111.9 m.

The SPT “N” values measured within this material range from 1 to 13 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 seven samples of the sand are provided on Figure B2 in Appendix B. The measured natural water content of selected samples of the sand and silty sand ranges from about 18 to 42 percent.

4.2.4 Clayey Silt to Silty Clay

The sandy soils are underlain by a deposit of grey clayey silt to silty clay. The clayey silt to silty clay was fully penetrated in Boreholes 14-322 to 14-324 to elevations between about 99.7 to 102.9 m with thicknesses between 9 and 10 m. The clayey silt to silty clay was not fully penetrated in Boreholes 14-321 and 15-325 but proven to an elevation of 101.0 m and 105.4 m, respectively.

The measured SPT “N” values within the clayey silt to silty clay deposit range from “weight of rods” to 5 blows per 0.3 m of penetration. In situ vane testing carried out within the deposit measured undrained shear strengths ranging from about 40 near the surface of the clay to 102 kPa at depth. The results of the in-situ testing indicate a firm to very stiff consistency. In situ vane testing carried out on remoulded grey clayey silt to silty clay measured undrained shear strengths generally ranging from 3 to 17 kilopascals, indicating a sensitivity generally ranging from about 3 to 19.

The results of grain size distribution testing carried out on three samples of the clayey silt and seven samples of the silty clay are provided on Figure B3 and B4, respectively. The results of Atterberg limit testing carried out on ten samples of the clayey silt to silty clay indicate plasticity index value between 10 and 24 percent and liquid limit value between 29 and 48, as shown on Figure B5, indicating that the tested samples consist of clayey silt to silty clay of low to intermediate plasticity. The measured natural water content of selected samples of the deposit ranges from 23 to 57 percent. These natural water contents are generally near or above the measured liquid limits.



Oedometer consolidation testing was carried out on one relatively undisturbed sample of the grey clayey silt to silty clay deposit from Borehole 14-322. The results of that testing, which are provided on Figure B6 and are summarized in the table below, indicate that this material (i.e., at the depth and location of the sample) is slightly preconsolidated, with a preconsolidation pressure of about 290 kPa and an overconsolidation ratio of 1.9.

Borehole/Sample Number	Sample Depth/Elevation (m)	Unit Weight (kN/m ³)	$\sigma_{p'}$ (kP)	$\sigma_{vo'}$ (kP)	$\sigma_{p'} - \sigma_{vo'}$ (kPa)	Cc	Cr	e _o	OCR
14-322 / 14	11.9 / 106.2	16.6	295	155	140	1.47	0.010	1.58	1.9

Notes:

$\sigma_{p'}$	-	Apparent preconsolidation pressure	Cr	-	Recompression index
$\sigma_{vo'}$	-	Computed existing vertical effective stress	e _o	-	Initial void ratio
Cc	-	Compression index	OCR	-	Overconsolidation ratio

4.2.5 Gravelly Silty Sand

A layer of gravelly silty sand with some clay was encountered within the silty clay deposit in Borehole 14-323 at a depth of 15.4 m below the existing ground surface, having a thickness of about 0.5 m.

The result of one standard penetration test measured an “N” value of 2 blows per 0.3 m of penetration, indicating a very loose state of packing.

The results of grain size distribution testing carried out on one sample of the deposit are provided on Figure B7. The measured natural water content of one sample of gravelly silty sand was about 14 percent.

4.2.6 Till

Glacial till was encountered below the clayey silt to silty clay deposit in Boreholes 14-322, 14-323 and 14-324, at elevations of 99.7, 101.4, and 102.9, respectively. Refusal to augering or to sampling was encountered at Elevation 101.1 m and 101.5 m in Boreholes 14-323 and 14-324, respectively.

In the area of the site, the glacial till is generally a heterogeneous mixture of gravel and cobbles in a matrix of sand and silt containing a trace to some clay.

The measured SPT “N” values within the till deposit range from “pushed manually” to over 50 blows per 0.3 m of penetration, indicating a very loose to very dense state of packing, although the higher ‘N’ values could reflect the presence of cobbles, rather than the state of packing of the soil matrix.

The results of grain size distribution testing carried out on two samples of the till are provided in Figure B8 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 content of two samples of the till were 11 and 13 percent.

4.2.7 Refusal and Possible Bedrock

Refusal to advancement of the auger and sampler was encountered in/below the till deposit in Boreholes 14-323 and 14-324, respectively. About 50 mm of possible weathered bedrock was encountered within the sampler in portable Borehole 14-324 below the till. The following table summarizes the refusal surface depths and elevations as encountered at the two borehole locations.



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Borehole Number	Existing Ground Surface Elevation (m)	Depth to Refusal (m)	Possible Bedrock Surface Elevation (m)
14-323	118.0	16.9	101.1
14-324	113.7	12.2	101.5

4.2.8 Groundwater Conditions

The groundwater level measured in the standpipe piezometer in Borehole 14-321 is presented in the table below:

Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date
14-321	113.24	0.02	113.2	August 13, 2014
		0.13	113.1	December 8, 2015

The groundwater level in the creek was given as Elevation 112.7 m on March 17, 2014.

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

4.2.9 Groundwater Corrosion Testing

One sample of surface water from the Unnamed Creek culvert site was submitted to Exova (Ottawa) for chemical analysis related to potential corrosion of steel elements and potential sulphate attack on concrete elements. The results of this testing are summarized below.


Sample Number	Sample Date	Chloride (mg/L)	SO ₄ (mg/L)	pH	Resistivity (ohm-cm)
1123431	July 29, 2014	10	14	8.14	1908



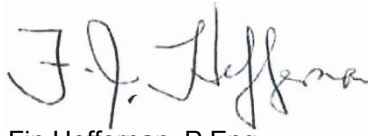
5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Kim Lesage, P.Eng., a geotechnical engineer with Golder. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.

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SG/KSL/FJH/bg/ob

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PART B

FOUNDATION DESIGN REPORT

UN-NAMED CREEK CULVERT REPLACEMENT

SITE 3-728C

HIGHWAY 417, 130 M WEST OF MARCH ROAD

OTTAWA, ONTARIO

W.P. 4168-11-01



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed replacement of the existing culvert on Highway 417 at MTO Structure Site No. 3-728c, about 130 m west of March Road. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the foundations for the replacement structure.

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

The existing culvert is a 3.1 m wide by 2.0 m high, sectional plate, corrugated steel pipe arch. The culvert is about 86 m long. The existing culvert inverts are at about Elevations 111.9 and 112.6 m at the north and south ends, respectively, with flow in the culvert from south to north.

It is understood that the replacement of the culvert will be along the existing culvert alignment. It has been assumed that invert levels of the existing culvert will be maintained for the replacement. It is also understood that the grade of the existing Highway 417 lanes will be maintained (i.e., there will be no grade raise of the highway or associated acceleration lanes) and that the embankments will not be significantly widened.

Several open cut foundation types are considered in the following sections in addition to tunnelling design alternatives. A comparison of the foundation alternatives based on advantages, disadvantages, constructability and relative costs is provided in Table 1 following the text of this report.

6.2 Open Cut Foundation Options

Based on the subsurface conditions, only shallow foundation options have been considered in sufficient detail for design for the replacement of the existing culvert. It is not considered to be a practical or economical option to support the culvert on deep foundations because the shallow subsoils will provide adequate bearing resistance and settlement performance given that the Highway 417 grade will not be raised and only a minor widening may be required. Furthermore, the available subsurface information indicates the bedrock surface to be at least 10 m or more below founding level and would result in relatively long and expensive piles.

A summary of the advantages and disadvantages associated with each shallow foundation option is provided below.

Concrete box culvert founded on the native soils: A pre-cast concrete closed box culvert is considered to be feasible for this site since the foundation loads are distributed over a larger area, resulting in lower foundation stress levels, and therefore reduced settlement magnitudes. It is expected that temporary protection systems and cofferdams would be required during excavation and construction. A precast culvert may be preferred over a cast-in-place culvert for this option because it would likely take less time to install, shortening the period for dewatering and traffic staging.



Rigid frame open footing culvert founded on the native soils: The settlements for a rigid frame open footing culvert would be larger than those for a closed box culvert due to the higher concentration of foundation stresses, and the factored bearing resistance would be insufficient in relation to those higher stresses. In addition, open footing culverts are typically less tolerant of total and differential settlement as compared with box culverts. A rigid frame open box culvert is therefore not considered suitable for the culvert replacement at this site, which is underlain by clay of limited strength.

Relining: Relining the culvert with grout would add additional loading (i.e., the weight of the liner as well as the grout) onto the underlying silt and clay deposits. Relining the culvert may be a suitable option at this site, provided the additional loading applied on the underlying silt and clay deposits is minimized (i.e., to limit the resulting settlements). This option would minimize traffic disruptions; however, a relined culvert may have a shorter design life than a concrete structure. Furthermore, the liner diameter may not be sufficient for the required culvert flow, which we understand is the case. A separate tunnel/culvert would be required to provide the necessary additional flow, which we understand makes this option not feasible.

Based on the above considerations, a closed box culvert is the preferred design option, from a foundation design perspective, due to the greater distribution of the foundation loads which would result in lower foundation stress levels and reduced settlement magnitudes. Recommendations for the box culvert option are therefore presented in the following sections.

6.3 Concrete Box Culvert

6.3.1 Founding Level and Bedding

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

The bedding and/or leveling pad requirements for a box culvert replacement should be in accordance with Ontario Provincial Standard Specification (OPSS) 422 (*Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut*). It is recommended that the box culvert segments be placed on 300 mm of OPSS.PROV 1010 Granular A bedding material. 200 mm of OPSS.PROV 1010 Granular B Type II covered by 100 mm of Granular A may also be used as an alternative. The bedding should extend the full width of the excavation.

The table below summarizes the recommended founding level for the box culvert, assuming a substrate thickness of 400 mm, a culvert base slab thickness of 300 mm and bedding thickness as described above. Based on these elevations, the box culvert replacement will be typically founded on the sand to silty sand, above the firm silty clay.

Invert Location	Proposed Streambed Elevation (m)	Box Culvert Founding Elevation (m)	Subgrade Level (m)
South End (Inlet)	112.5	111.8	111.5
North End (Outlet)	112.2	111.5	111.2



The bedding thickness of the existing pipe arch is not known. If the existing bedding extends beneath the proposed founding elevations, the existing subgrade should be subexcavated to the native soil, and raised to founding level with OPSS.PROV 1010 Granular A.

Where the subgrade consists of sensitive layered clay and silt, the subgrade will be susceptible to disturbance and degradation on exposure to water and construction traffic. As an alternative to the placement of a minimum 300 mm thick layer of Granular A, a 100 mm thick concrete working slab could be placed on the subgrade within the culvert footprint, to protect the subgrade from such degradation. In this case, a 75 mm thick layer of OPSS.PROV 1010 Granular A or concrete fine aggregate meeting the gradation requirements set out in OPSS.PROV 1002 (*Material Specification for Aggregates - Concrete*) should be placed on top of the concrete mat to provide a "levelling pad" for the box culvert replacement. The working slab should be placed within four hours after inspection and approval of the subgrade.

The footing subgrade should be inspected in accordance with OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*). Further discussion regarding subgrade preparation and protection is provided in Section 6.6.3.

6.3.2 Geotechnical Resistances

For a box culvert founded at the elevations provided in Section 6.3.1.1 a factored geotechnical resistance at Ultimate Limit States (ULS) of 225 kPa and a geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement of 170 kPa may be used for design purposes.

These 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)*.

The footing subgrade should be inspected by a Quality Verification Engineer (QVE) in accordance with OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*). Further discussion regarding subgrade preparation and protection is provided in Section 6.6.4.

6.3.3 Resistance to Lateral Loads

The resistance to lateral forces/sliding for the culvert should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The resistance values will also depend on the founding levels/strata.

The culvert may be constructed on a granular pad on the unweathered clay. For this case, the following parameters should be used:

Interface and Loading Condition	Parameter
Concrete – granular pad: short or long term loading	Effective friction angle = 33 degrees
Granular A pad – clay subgrade: short term loading	Undrained cohesion = 40 kPa
Granular A pad – clay subgrade: long term loading	Effective friction angle = 28 degrees

These values are unfactored; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.



6.3.4 Culvert Backfill and Erosion Protection

Backfill and cover for concrete culverts should be completed in accordance with OPSS 422 (*Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut*) or OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*) as well as OPSP 803.010 (*Backfill and Cover for Concrete Culverts*), as appropriate.

Backfill to culvert walls should consist of granular fill meeting the requirements of OPSS.PROV 1010 Granular A or Granular B Type II. The backfill should be placed and compacted in accordance with OPSS.PROV 501 (Compaction). The fill depth during placement should be maintained equal on both sides of the culvert walls, with one side not exceeding the other by more than 500 mm in height. The culvert replacement should be designed for the full overburden pressure and live loads, assuming that the embankment fill has a unit weight of 22 kN/m³ for Granular A and 21 kN/m³ for Granular B Type II or select earth fill above and/or surrounding the culvert. The use of Select Subgrade Material (SSM) is recommended above the culvert to avoid potential settlement resulting from the use of higher unit weight backfill soils than the existing embankment fill material. A unit weight of 19 kN/m³ may be used for SSM.

The performance of the box culvert is dependent on the construction procedures. Therefore it is suggested that a Quality Verification Engineer be retained to verify the foundation and bedding construction procedures.

To prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a clay seal or concrete cut-off wall and headwall/wingwalls should be provided at the upstream and downstream ends of the culvert replacement.

If the flow velocities are sufficiently high, scour and erosion protection (suitable non-woven geotextiles and/or rip-rap) should be provided at the culvert inlet and outlet. The requirements for and design of erosion protection measures for the culvert inlet should be assessed by the hydraulic design engineer. As a minimum, rip-rap treatment for the culvert outlet should be consistent with the standard Treatment Type A presented in OPSP 810.010 (*Rip-Rap Treatment for Sewer and Culvert Outlets*), with the rip-rap placed up to the toe of slope level, in combination with the cut-off measures noted above. Similarly, rip-rap should be provided over the full extent of the clay blanket if adopted, including the drained side slopes and embankment fill slope adjacent to the culvert.

In addition, sediment control such as silt fences and/or erosion control blankets may be required during construction to mitigate migration of fine soil particles into the water courses.

6.4 Lateral Earth Pressures for Culvert Design

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

The following recommendations are made concerning the design of the walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.



FOUNDATION REPORT UN-NAMED CREEK CULVERT REPLACEMENT - HIGHWAY 417

- Select free-draining granular fill meeting the specifications of the provincial version of the Ontario Provincial Standard Specifications (OPSS.PROV) Granular 'A' or Granular 'B' should be used as backfill behind the walls. This fill should be compacted in accordance with OPSS.PROV 501 (Compaction). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost tapers should be in accordance with OPSD 3101.150, 3121.150, and 3190.100.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS.PROV 501 (Compaction). Other surcharge loadings should be accounted for in the design as required.
- The granular fill may be placed either in a zone with a width equal to at least 1.8 m behind the back of the culvert wall (Case I) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the founding elevation (Case II).
- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material:

Soil Unit Weight	20 kN/m ³
Coefficients of Static Lateral Earth Pressure:	
Active, K_a	0.35
At rest, K_o	0.50

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

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

Because the culvert walls do not allow lateral yielding, at-rest earth pressures should be assumed for the geotechnical design. If concrete headwalls or wingwalls are used at the ends of the replacement culvert, active earth pressures should be used in the design.

6.4.1 Seismic Considerations

Seismic (earthquake) loading should be assessed in the design in accordance with Section 4.6.4 of CHBDC, as significant seismic loading would result in increased lateral earth pressures acting on the culvert walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the applicable earthquake-induced dynamic earth pressure.



The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$P = K_o \gamma d + (K_{AE} - K_o) \gamma (H - d)$$

Where: K_o is the static at-rest earth pressure coefficient (K_o);
 K_{AE} is the seismic active earth pressure coefficient;
 γ is the unit weight of the backfill soil (kN/m^3) as given previously;
 d is the depth below the top of the wall (m); and,
 H is the height of the wall above the toe (m).

According to the CHBDC, the site-specific zonal acceleration ratio for the area of the site is 0.2. Based on experience, for the subsurface conditions at this site, a 10 percent amplification of the ground motion could occur, resulting in an increase in the ground surface acceleration to 0.22 g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of $A = 0.22$.

In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, for structures which do not allow lateral yielding (i.e., culvert walls), the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient is taken as 1.5 times the zonal acceleration ratio (i.e., $k_h = 0.33$). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e., $k_h = 0.11$).

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

Seismic Active Pressure Coefficients, K_{AE}

	Case I	Case II	
		Granular 'A'	Granular 'B' Type II
Non-yielding wall	0.65	0.50	0.50

6.5 Estimates of Embankment Settlement

It is understood that the grade of the existing pavement grades will be maintained and that the embankments will not be significantly widened.

Provided that the new embankment fill material consists of Select Subgrade Material or clean earth fill, the settlement due to compression of the embankment fill itself is expected to be less than about 25 mm.

6.6 Gabion Walls

Gabion walls are currently proposed at the north end of the culvert to prevent erosion.



Construction of gabion walls is geotechnically feasible at this site. Gabion walls do not require an embedment depth equivalent to the frost depth provided it is founded on a granular pad of 300 mm compacted thickness and the foundations have adequate embedment to provide a stable structure. Advantages of gabion walls compared to more rigid structures include the ability to accommodate differential settlements, dissipation of the energy of flowing water, and are free-draining provided an adequate filter is placed behind the wall.

Gabion walls can be constructed relatively quickly with minimal equipment and materials. The life expectancy of a gabion wall can be extended by utilizing PVC-coated galvanized steel baskets. Gabion walls are to be constructed in accordance with OPSS.PROV 512.

Gabion walls may be founded directly on a 300 mm thick compacted Granular A pad placed on the embankment fill and sand to silty sand. A factored geotechnical resistance at ULS of 150 kPa and a geotechnical reaction at SLS of 100 kPa may be used for design. The SLS value corresponds to 25 mm of settlement. If required, a granular levelling course approximately 75 mm in thickness may be placed on the founding strata for gabion walls.

Non-woven geotextile is to be placed between the gabions and the backfill in accordance with OPSS.PROV 512, OPSS 1860, and the manufacturer's specifications.

6.7 Construction Considerations

6.7.1 Groundwater and Surface Water Control

The groundwater level was measured at Elevation 113.2 m on August 13, 2014, which is within the sand to silty sand deposit.

Control of the surface water and groundwater would be necessary for the construction of the replacement culvert, to allow excavation and foundation construction to be carried out in dry conditions. Groundwater and surface water control would also be required for grouting of a relined culvert.

Depending on the flow at the time of construction, the surface water flow could be passed through the existing culvert, or diverted by pumping from behind a temporary cofferdam sealed into the silty clay. Surface water should be directed away from the excavation areas to prevent ponding of water that could result in disturbance and weakening of sensitive clay subgrade soils; further discussion on this aspect is provided in Section 6.7.3.

Groundwater inflow into the excavations should be expected. It is not expected that the groundwater inflow to the excavation will be excessive; however some form of groundwater lowering will likely be required in advance of excavation. Otherwise the groundwater inflow will disturb/soften the sandy subgrade soils, where present, and possibly destabilize the side slopes of open cut excavations made through the embankment fill and native sand to silty sand. Pumping from filtered sumps established in the floor of the excavations is unlikely to effectively dewater the excavation due to the fine grained nature of the silty sand. It is therefore considered that pre-pumping from a series of sanded-in vacuum well-points or eductor wells installed into the silty subgrade soils would be required to lower the piezometric level below the excavation level in advance of culvert construction.

It is recommended that coffer dams be constructed around the excavations (on all sides) to bypass the flows around the culvert areas during construction and reduce groundwater flow into the excavation area. The coffer dams should be comprised of interlocking steel sheet piling that penetrates into the underlying clayey silt to silty clay.

A sample Non Standard Special Provision for groundwater and surface water control is provided in Appendix C.



Additional subexcavation or ditches to divert perched water and/or storm water flows around the construction area may also be required to help permit construction and compaction of engineered fill in the “dry”.

6.7.2 Excavation and Temporary Protection Systems

Temporary excavations for the replacement culvert, up to a depth of about 8.3 m, would be made through the existing fill, sand to silty sand, and clayey silt to silty clay (where present). Excavation works must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects. The existing fill above the water table and the clayey silt to silty clay would be classified as Type 3 soil based on the OHSA. According to OHSA, excavations that extend to, or into, Type 3 soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V). The sand to silty sand below the water table would be classified as Type 4 soil based on OSHA, and excavations in these materials should be sloped no steeper than 3H:1V.

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

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

As a further guideline, excavated soils should not be stockpiled adjacent to the crest of the excavation side slopes (or above the protection system) due to the potential to reduce the factor of safety against side slope instability.

6.7.3 Existing Foundations and Subgrade Protection

All embankment fill, topsoil, organics, and soft or loose soils should be removed from below the proposed founding elevations and wasted or reused as landscaping fill, as required. Subgrade preparation should be performed and monitored in accordance with OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*). The cleaned excavation base should be inspected prior to placing granular bedding for the box culvert. As discussed in Sections 6.3.1.1 and 6.3.2.1, the bedding thickness of the existing pipe arch is not known. If the existing bedding extends beneath the proposed founding elevations, the existing subgrade should be subexcavated to the native soil.

The sensitive clayey silt to silty clay subgrade (to be expected at the south end for a box culvert) will be susceptible to disturbance and degradation on exposure to water and construction traffic. It is recommended that a minimum 300 mm thick layer of Ontario Provincial Standard Specification 1010 (*Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material*) Granular A be placed below the base slab on the subgrade to form a bedding layer for the box culvert segments, and to limit the degradation of the sensitive subgrade. The bedding should be placed within four hours after inspection and approval of the subgrade to limit such degradation.



As an alternative to the placement of a minimum 300 mm thick layer of Granular A for a box culvert, a 100 mm thick concrete working slab could be placed on the subgrade within the culvert footprint, to protect the subgrade from such degradation. In this case, a 75 mm thick layer of OPSS.PROV 1010 Granular A or concrete fine aggregate meeting the gradation requirements set out in OPSS.PROV 1002 (*Material Specification for Aggregates – Concrete*) should be placed on top of the concrete mat to provide a “levelling pad” for the box culvert replacement. The working slab should be placed within four hours after inspection and approval of the subgrade.

6.7.4 Corrosion and Cement Type

One surface water sample from Unnamed Creek was submitted to Exova Laboratories for chemical analysis related to potential sulphate attack on concrete and potential corrosion of exposed steel elements. The results of the chemical analysis testing are summarized in Section 4.2.8.

The results indicated a low exposure to sulphate attack, which suggests that concrete made with Type GU Portland cement should be acceptable for buried substructures. The results also indicate an elevated potential for corrosion of exposed ferrous metal, which should be considered in the design of substructures.

6.8 Tunnelling Construction

Construction of the culvert by tunnelling or other trenchless methods has also been considered, recognizing the lesser traffic impacts.

It is understood that the existing invert levels would need to be maintained, or lowered slightly. In order to maintain the flow, it is assumed that a circular replacement culvert, such as would be required to construct the replacement by tunnelling, would be similar in size to the existing (i.e., 3 m inner diameter pipe, which is larger than typical tunnelling size), and that the crown would be at about Elevation 115 m. As a general guideline, trenchless crossings in overburden should only be considered where the cover above the crown of the tunnel/bore would be at least twice the tunnel/bore diameter relative to the ground surface. Lesser amounts of cover could jeopardize the stability of the working face (depending on the method) or lead to ground loss and settlement. The borehole data indicates that the tunnel would be through a mixed face of rock fill and sandy embankment fill, and the underlying native sand to silty sand. Tunnelling of this size would not be a feasible option for the culvert replacement.

Tunnelling may be considered a feasible option if multiple smaller diameter bores/culverts are constructed (rather than one large bore/culvert) or if tunnelling it is used in conjunction with relining. Relining of the existing culvert with a grouted in place metal liner is not a preferred option due mainly to the reduction in size of the existing culvert, which may not be sufficient for the required culvert flow and environmental/fisheries considerations; however if both relining and tunnelling options are adopted in combination, the tunnel diameter can be reduced and thus increasing the cover above the bore, making this option more feasible.

As a preliminary guideline, pipe jacking and horizontal auger boring, or pipe ramming would be feasible construction methods for a smaller diameter pipe (used in combination with relining). In brief, these construction methods involve the following:

- **Pipe Ramming:** Pipe ramming uses a pneumatic ramming tool to hammer a steel casing, in sequential spliced sections, from one pit to the other through the ground. Depending on the length of the installation, the soils inside the pipe can be removed either during or after the installation by augering (most commonly), compressed air, or water jetting. The profile needs to be approximately horizontal. Launching and receiving



pits are required. Pipe ramming is not-steerable, so there is no control over the profile and alignment of the bore once the pipe ramming has started. This method has been used with mixed success in bouldery soils. However, ground surface heaving can be an issue in dense ground.

- **Pipe Jacking and Horizontal Auger Boring:** A pipe jacking operation involves pushing an oversized liner pipe (casing) horizontally into the ground by jacking. The spoil is generally removed from within the casing using an auger boring machine. The cutting head is driven by, and is positioned at, the leading end of an auger string that is established within the casing pipe. The profile needs to be approximately horizontal. Jacking and receiving pits are required. There can be limited ability to steer the casing during jacking. This method is only applicable to construction in the overburden, and may not be feasible in bouldery soils (e.g., rock fill). This method is also not feasible in flowing ground.


Further details on tunnelling would be required if this option is selected.



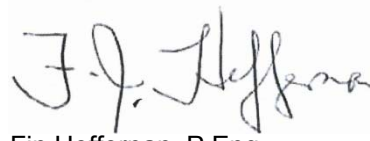
7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Kim Lesage, P.Eng., a geotechnical engineer with Golder. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.

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SG/KSL/FJH/bg/ob

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FOUNDATION REPORT UN-NAMED CREEK CULVERT REPLACEMENT - HIGHWAY 417

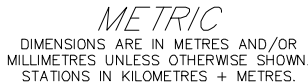
Table 1 – Comparison of Foundation Alternatives

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Option 1 Concrete Box	<ul style="list-style-type: none"> Feasible, preferred option 	<ul style="list-style-type: none"> Potentially shallower excavation depths Foundation loads distributed over a larger area, therefore reducing settlement magnitudes More tolerant of settlement than an open footing culvert Precast sections would be quicker and easier to install (i.e., less disruption to traffic) Long design life 	<ul style="list-style-type: none"> Relatively low geotechnical resistances on the compressible native soils Groundwater control and a temporary protection system are required 	<ul style="list-style-type: none"> Moderate cost 	<ul style="list-style-type: none"> Generally lower risk option
Option 2 Rigid Frame Open Footing	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Desirable option for culvert flow and environmental/fisheries considerations 	<ul style="list-style-type: none"> Deeper excavation depth Larger settlements than those of a box culvert due to the higher concentration of stresses from narrow foundations Groundwater control and a temporary protection system are required 	<ul style="list-style-type: none"> Moderate to high cost 	<ul style="list-style-type: none"> Higher risk option in terms of settlement and performance of culvert and highway embankment
Option 3 Relining	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Will minimize traffic disruption Roadway protection would not be required 	<ul style="list-style-type: none"> Increased loading due to grout and liner could result in culvert settlements Reduction in culvert size may not be desirable for flow and environmental/fisheries considerations May have a shorter design life than a concrete structure 	<ul style="list-style-type: none"> Low to Moderate cost 	<ul style="list-style-type: none"> Low risk option



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Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Option 4 Deep Foundations	<ul style="list-style-type: none">Feasible but not required/practical	<ul style="list-style-type: none">Would not result in culvert settlement	<ul style="list-style-type: none">Would require roadway protection systemWould require piles	<ul style="list-style-type: none">Expensive option	<ul style="list-style-type: none">Low risk option
Option 5 Tunnelling	<ul style="list-style-type: none">Not feasible (unless used in combination with relining or multiple pipes)	<ul style="list-style-type: none">Can be designed to maintain existing stress levels (at the culvert location)Roadway protection may not be required	<ul style="list-style-type: none">Inadequate soil coverSoil conditions are not favorable for trenchless technologies (i.e., presence of rock fill)High likelihood of delaysPotential for sinkholes, which could cause traffic disruptions	<ul style="list-style-type: none">Expensive option	<ul style="list-style-type: none">High risk option



SHEET
44



LEGEND

- | | |
|---|--|
|  | Borehole – Current Investigation |
|  | Seal |
|  | Piezometer |
| N | Standard Penetration Test Value |
| 16 | Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow) |
| PH | Sampler advanced by hydraulic pressure |
| PM | Sampler advanced by manual pressure |
| WR | Sampler advanced by static weight of hammer |
|  | WL in piezometer, measured on Aug. 13, 2014 |

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
14-321	113.2	5019397.0	339451.6
14-322	118.1	5019378.6	339445.1
14-323	118.0	5019328.9	339427.4
14-324	113.7	5019312.8	339422.6
15-325	117.1	5019353.0	339435.4

NOTES

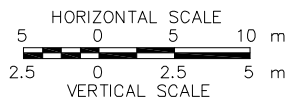
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Preliminary Design Report.

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 nos. Plan-3-728.dwg, received August 7, 2014.



NO.	DATE	BY	REVISION
Geocres No. 31F-191			
HWY. 417		PROJECT NO. 12-1121-0099	DIST. EASTERN
SUBM'D. MJK	CHKD. MJK	DATE: FEB. 2016	SITE: 3-728c
DRAWN: JJJ/JM	CHKD. FJH	APPD. FJH	DWG. 1



APPENDIX A

List of Abbreviations and Symbols
Lithological and Geotechnical Rock Description Terminology
Borehole and Drillhole Records

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE		III. SOIL DESCRIPTION		
AS	Auger sample	(a) Cohesionless Soils		
BS	Block sample	Density Index (Relative Density)		N
CS	Chunk sample			Blows/300 mm
DO or DP	Seamless open-ended, driven or pushed tube samplers			Or Blows/ft.
DS	Denison type sample		Very loose	0 to 4
FS	Foil sample		Loose	4 to 10
RC	Rock core		Compact	10 to 30
SC	Soil core		Dense	30 to 50
SS	Split spoon sampler		Very dense	over 50
ST	Slotted tube	(b) Cohesive Soils		
TO	Thin-walled, open	C _u or S _u		
TP	Thin-walled, piston	Consistency		
WS	Wash sample		kPa	Psf
DT	Dual tube sample		Very soft	0 to 12
DD	Diamond drilling		Soft	12 to 25
			Firm	25 to 50
			Stiff	50 to 100
			Very stiff	100 to 200
			Hard	Over 200
			Over 4,000	
II. PENETRATION RESISTANCE		IV. SOIL TESTS		
Standard Penetration Resistance (SPT), N:		w	Water content	
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.).		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	
PH:	Sampler advanced by hydraulic pressure	D _R	Relative density	
PM:	Sampler advanced by manual pressure	DS	Direct shear test	
WH:	Sampler advanced by static weight of hammer	G _s	Specific gravity	
WR:	Sampler advanced by weight of sampler and rod	M	Sieve analysis for particle size	
Cone Penetration Test (CPT):		MH	Combined sieve and hydrometer (H) analysis	
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.		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 $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

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

Notes:

$$^1 \tau = c' + \sigma' \tan \phi'$$

$$^2 \text{ shear strength} = (\text{compressive strength}) / 2$$

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

<u>Description</u>	<u>Bedding Plane Spacing</u>
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

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

GRAIN SIZE

<u>Term</u>	<u>Size*</u>
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° 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
Il -	Parallel To	STR -	Stress Induced
OR -	Orthogonal		

PROJECT <u>12-1121-0099-1320</u>		RECORD OF BOREHOLE No 14-321		SHEET 1 OF 2		METRIC	
G.W.P. <u>4168-11-01</u>		LOCATION <u>N 5019397.0; E 339451.6</u>		ORIGINATED BY <u>NJ</u>			
DIST <u>Eastern</u> HWY <u>417</u>		BOREHOLE TYPE <u>Portable Drill, NW/BW Casing</u>		COMPILED BY <u>JL</u>			
DATUM <u>Geodetic</u>		DATE <u>July 14, 2014</u>		CHECKED BY <u>MJK</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)			GR	SA	SI	CL
								20	40	60	80	100		W _p	W	W _L				
113.2	GROUND SURFACE																			
0.0	TOPSOIL, with sand, with organics		1	SS	11															
0.1	Brown		2	SS	1															
	SAND, some silt to silty, trace gravel		3	SS	3											2	81			
	Very loose to loose		4	SS	4											14	3			
	Brown to grey		5	SS	6															
	Wet		6	SS	10											1	86			
110.2	SAND, some silt		7	SS	5															
3.0	Compact		8	SS	4															
	Grey		9	SS	4															
	Wet		10	SS	4															
109.4	CLAYEY SILT to SILTY CLAY, some sand, some gravel, with organics		11	SS	3															
3.8	Firm to very stiff		12	SS	3											12	13			
	Grey		13	SS	2											35	40			
	Wet		14	SS	4															
			15	SS	5															
			16	SS	4															

Continued Next Page

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

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PROJECT 12-1121-0099-1320		RECORD OF BOREHOLE No 14-321				SHEET 2 OF 2		METRIC														
G.W.P. 4168-11-01		LOCATION N 5019397.0 ; E 339451.6				ORIGINATED BY NJ																
DIST Eastern HWY 417		BOREHOLE TYPE Portable Drill, NW/BW Casing				COMPILED BY JL																
DATUM Geodetic		DATE July 14, 2014				CHECKED BY MJK																
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa														
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 25 50 75 </div>										
101.6	CLAYEY SILT to SILTY CLAY, some sand, some gravel, with organics Firm to very stiff Grey Wet		17	SS	4		103															
			18	SS	4																	
			19	SS	5		102															
101.6 11.6	SILTY CLAY Stiff Grey Wet		20	SS	4												0 1 45 54					
101.0 12.2	END OF BOREHOLE NOTES: 1. Water level in well at 0.0 m depth below ground surface (Elev. 113.2 m), measured on August 13, 2014.																					

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PROJECT <u>12-1121-0099-1320</u>		RECORD OF BOREHOLE No 14-322		SHEET 2 OF 2		METRIC	
G.W.P. <u>4168-11-01</u>		LOCATION <u>N 5019378.6; E 339445.1</u>		ORIGINATED BY <u>DWM</u>			
DIST <u>Eastern</u> HWY <u>417</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem)</u>		COMPILED BY <u>JL</u>			
DATUM <u>Geodetic</u>		DATE <u>July 14, 2014</u>		CHECKED BY <u>MJK</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20	40	60	80	100	25	50	75		
	--- CONTINUED FROM PREVIOUS PAGE ---						108	X		+							
			13	SS	PM			X		+							
			14	TP	PH												
							106										
								X		+							
								X									
								X		+							
			15	SS	PM			X									
								X		+							
								X									
			16	SS	PM												
								X		+							
								X									
			17	SS	PM												
								X									
								X		+							
99.7							100										
18.4	SAND, some gravel, silt and clay (TILL) Very loose Grey Wet		18	SS	PM												18 49 21 12
99.2																	
18.9	END OF BOREHOLE																

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PROJECT 12-1121-0099-1320		RECORD OF BOREHOLE No 14-323		SHEET 2 OF 2		METRIC														
G.W.P. 4168-11-01		LOCATION N 5019328.9 ; E 339427.4		ORIGINATED BY DWM																
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)		COMPILED BY JL																
DATUM Geodetic		DATE July 16, 2014		CHECKED BY MJK																
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ kN/m³	GR SA SI CL			
							20 40 60 80 100	○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × REMOULDED	W _p	W	W _L	25 50 75							
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100												
	CLAYEY SILT to SILTY CLAY Firm Grey Wet		12	SS	PM		107											0 0 43 57		
							106													
			13	SS	WR		105													
	- Becomes stiff at 13.1 m depth						104													
			14	SS	PM		103													
102.6	Gravelly Silty SAND, some clay Very loose Grey Wet		15	SS	2		102											22 35 27 16		
102.2	SILTY CLAY, trace sand Very loose Grey Wet		16	SS	PM													0 5 45 50		
101.4	SAND (TILL) Very dense Grey Wet		17	SS	50/10 T															
101.1	END OF BOREHOLE AUGER REFUSAL on Possible Bedrock																			

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PROJECT <u>12-1121-0099-1320</u>		RECORD OF BOREHOLE No 14-324		SHEET 1 OF 2		METRIC	
G.W.P. <u>4168-11-01</u>		LOCATION <u>N 5019312.8 ; E 339422.6</u>		ORIGINATED BY <u>NJ</u>			
DIST <u>Eastern</u> HWY <u>417</u>		BOREHOLE TYPE <u>Portable Drill, NW/BW Casing</u>		COMPILED BY <u>JL</u>			
DATUM <u>Geodetic</u>		DATE <u>July 16, 2014</u>		CHECKED BY <u>MJK</u>			

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SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× REMOULDED	W _p	W	W _L			
113.7	GROUND SURFACE																
0.0	TOPSOIL																
0.1	Brown SAND, some silt, trace clay, trace roots Very loose Brown Moist to wet		1	SS	1								○				
			2	SS	1												
			3	SS	2								○		0 71 22 7		
111.9																	
1.8	CLAYEY SILT to SILTY CLAY, trace sand seams Firm to stiff Grey Wet		4	SS	2												
			5	SS	3								○		0 9 55 36		
							×		+								
							×		+								
			6	SS	3												
							×		+								
							×		+								
			7	SS	3								○				
							×		+								
							×		+								
							×			+							
			8	SS	PM												
								×		+							
							×			+							
							×				+						
			9	SS	2								○		0 0 44 56		
							×			+							
103.7							×				+						






Continued Next Page

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

PROJECT		RECORD OF BOREHOLE		No 14-324		SHEET 2 OF 2		METRIC									
G.W.P. 4168-11-01		LOCATION		N 5019312.8 ; E 339422.6		ORIGINATED BY		NJ									
DIST Eastern HWY 417		BOREHOLE TYPE		Portable Drill, NW/BW Casing		COMPILED BY		JL									
DATUM Geodetic		DATE		July 16, 2014		CHECKED BY		MJK									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
10.0	SILTY CLAY Very stiff Grey Wet		10	SS	4												
102.9																	
10.8	SAND, some gravel and silt, trace clay (TILL) Loose to compact Grey Wet		11	SS	11												
			12	SS	6												
101.6	Possible Weathered Bedrock		13	SS	50/0.1												14 50 27 9
12.2	END OF BOREHOLE SAMPLER REFUSAL on Possible Bedrock																

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PROJECT <u>12-1121-0099-1320</u>		RECORD OF BOREHOLE No 15-325		SHEET 1 OF 2		METRIC	
G.W.P. <u>4168-11-01</u>		LOCATION <u>N 5019353.0 ; E 339435.4</u>		ORIGINATED BY <u>DWM</u>			
DIST <u>Eastern</u> HWY <u>417</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem)</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>December 8, 2015</u>		CHECKED BY _____			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE					W _p W W _L				
								● QUICK TRIAXIAL × REMOULDED									
117.1	GROUND SURFACE						20	40	60	80	100						
0.0	Sand, some gravel and silt (FILL)																
116.8	Brown Moist																
0.3	Sand, some silt, trace gravel, contains pockets of silty clay (FILL)																
	Compact to dense Brown Moist to wet		1	SS	16												
			2	SS	21								○				
			3	SS	17												
		4	SS	31								○					
		5	SS	15													
112.5																	
4.6	SAND to Silty SAND, contains organic matter and decomposed wood		6	SS	10								○				
	Loose to compact Black to grey Wet																
111.8																	
5.3	SAND to Silty SAND, contains organic matter		7	SS	7								○				
	Loose Dark grey Wet																
110.9																	
6.3	CLAYEY SILT		8	SS	4								┌─┐				
	Stiff Grey Moist																
110.1																	
7.0	CLAYEY SILT to SILTY CLAY, contains shells																
	Firm Grey Wet		9	SS	PH									┌─┐			
						</											

Continued Next Page

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

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PROJECT <u>12-1121-0099-1320</u>		RECORD OF BOREHOLE No 15-325		SHEET 2 OF 2		METRIC	
G.W.P. <u>4168-11-01</u>		LOCATION <u>N 5019353.0 ; E 339435.4</u>		ORIGINATED BY <u>DWM</u>			
DIST <u>Eastern</u> HWY <u>417</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem)</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>December 8, 2015</u>		CHECKED BY _____			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		GR	SA	SI	CL
-- CONTINUED FROM PREVIOUS PAGE --								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)							
								20	40	60	80	100		25	50	75				
							107	×		+										
								×		+										
			11	SS	PH		106								○					
105.4								×		+										
11.7	END OF BOREHOLE																			

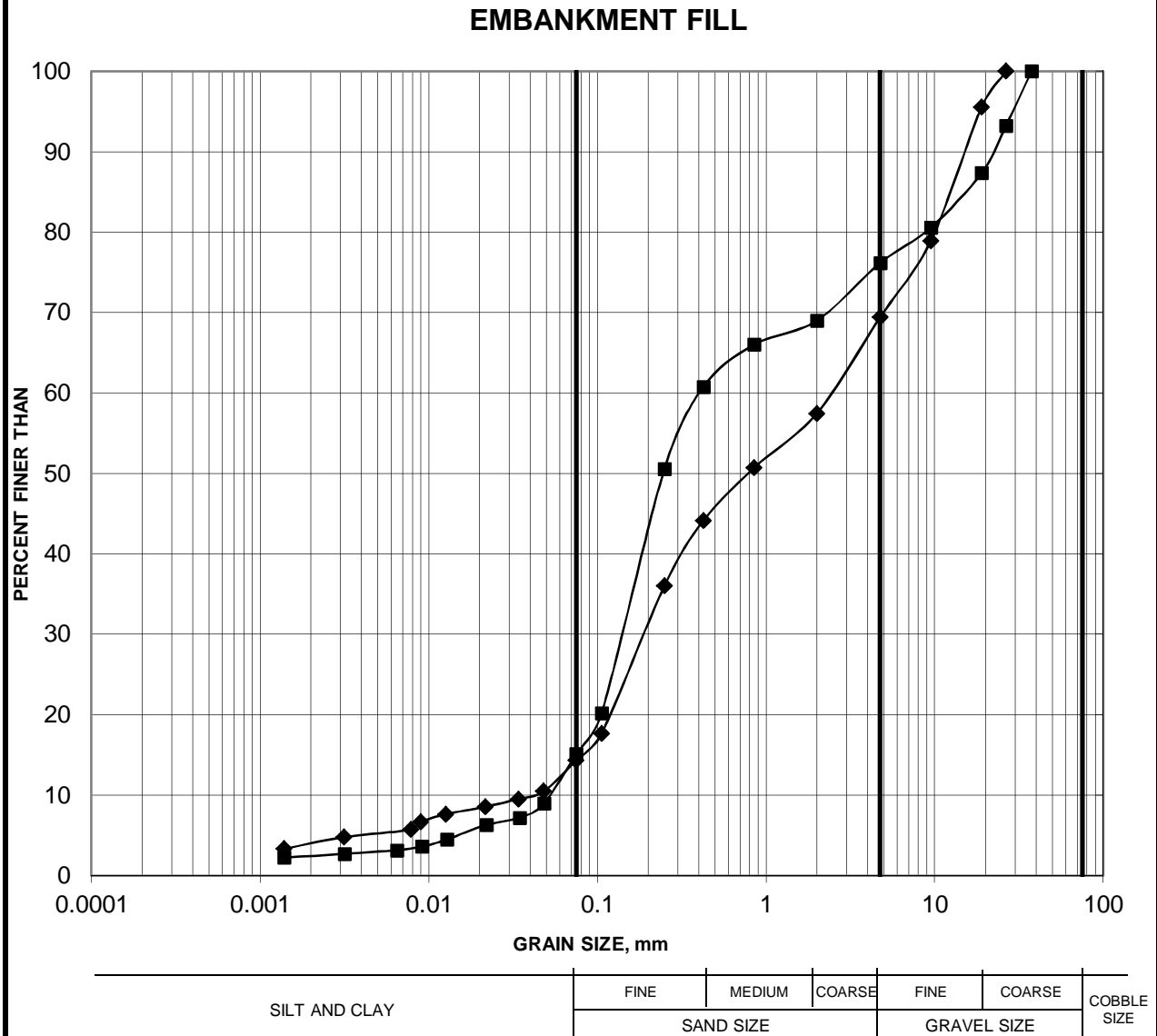


APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

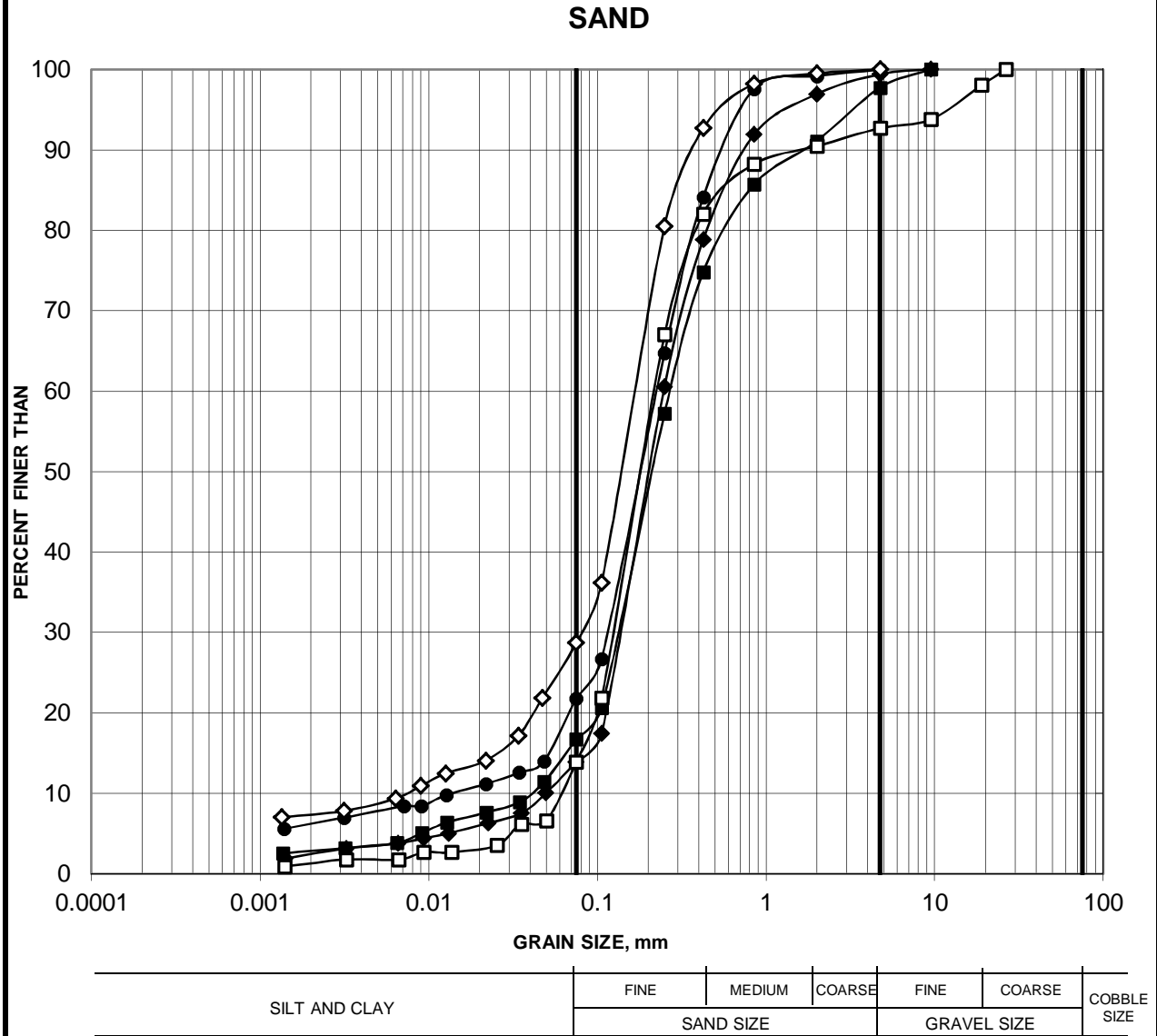
FIGURE B1



Borehole	Sample	Depth (m)
■ 14-322	2	1.52-2.13
◆ 14-323	4	2.29-2.90

GRAIN SIZE DISTRIBUTION

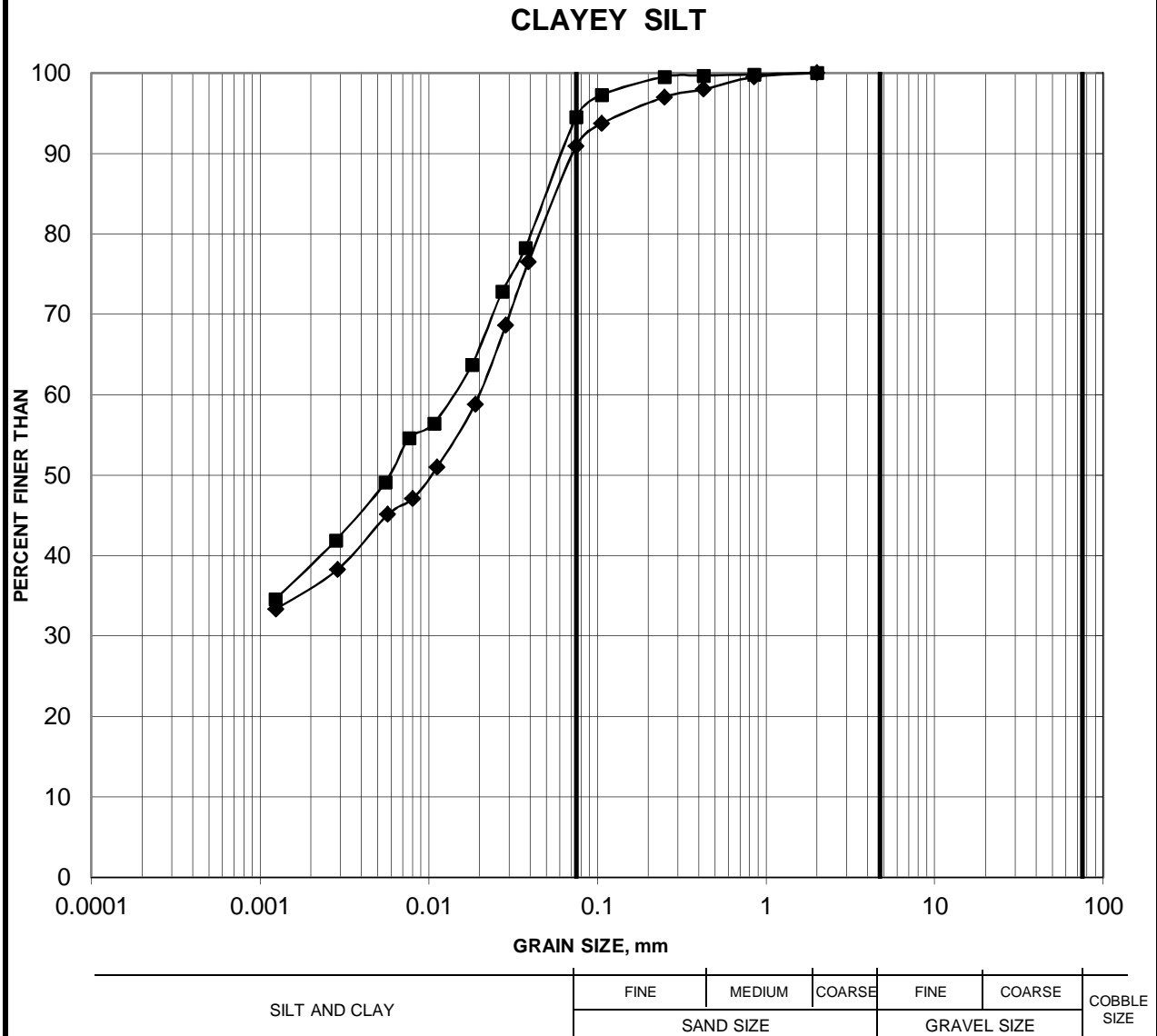
FIGURE B2



Borehole	Sample	Depth (m)
■ 14-321	3	1.22-1.83
◆ 14-321	6	3.05-3.66
● 14-322	10	6.86-7.47
□ 14-323	6	4.57-5.18
◇ 14-324	3	1.22-1.83

GRAIN SIZE DISTRIBUTION

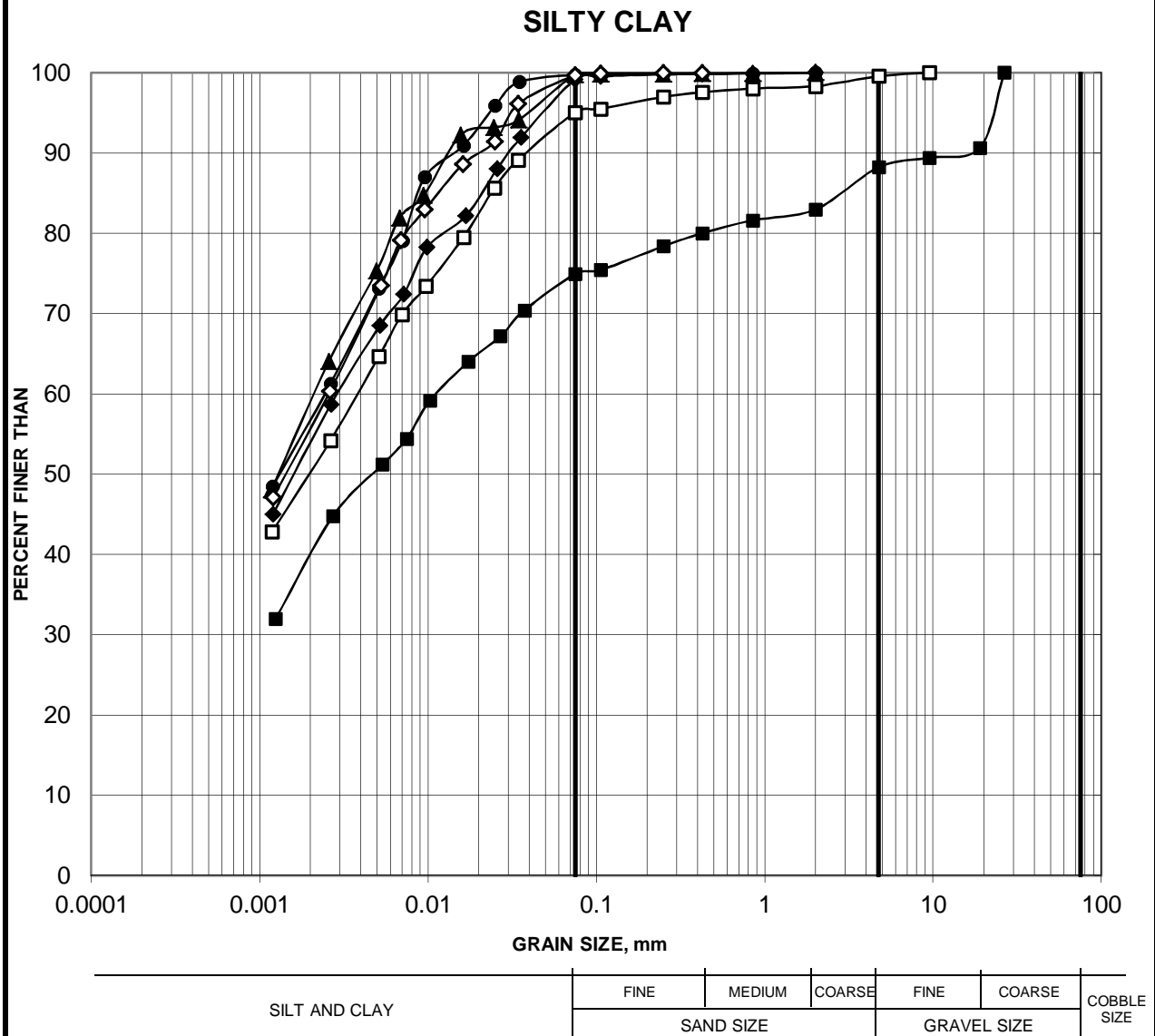
FIGURE B3



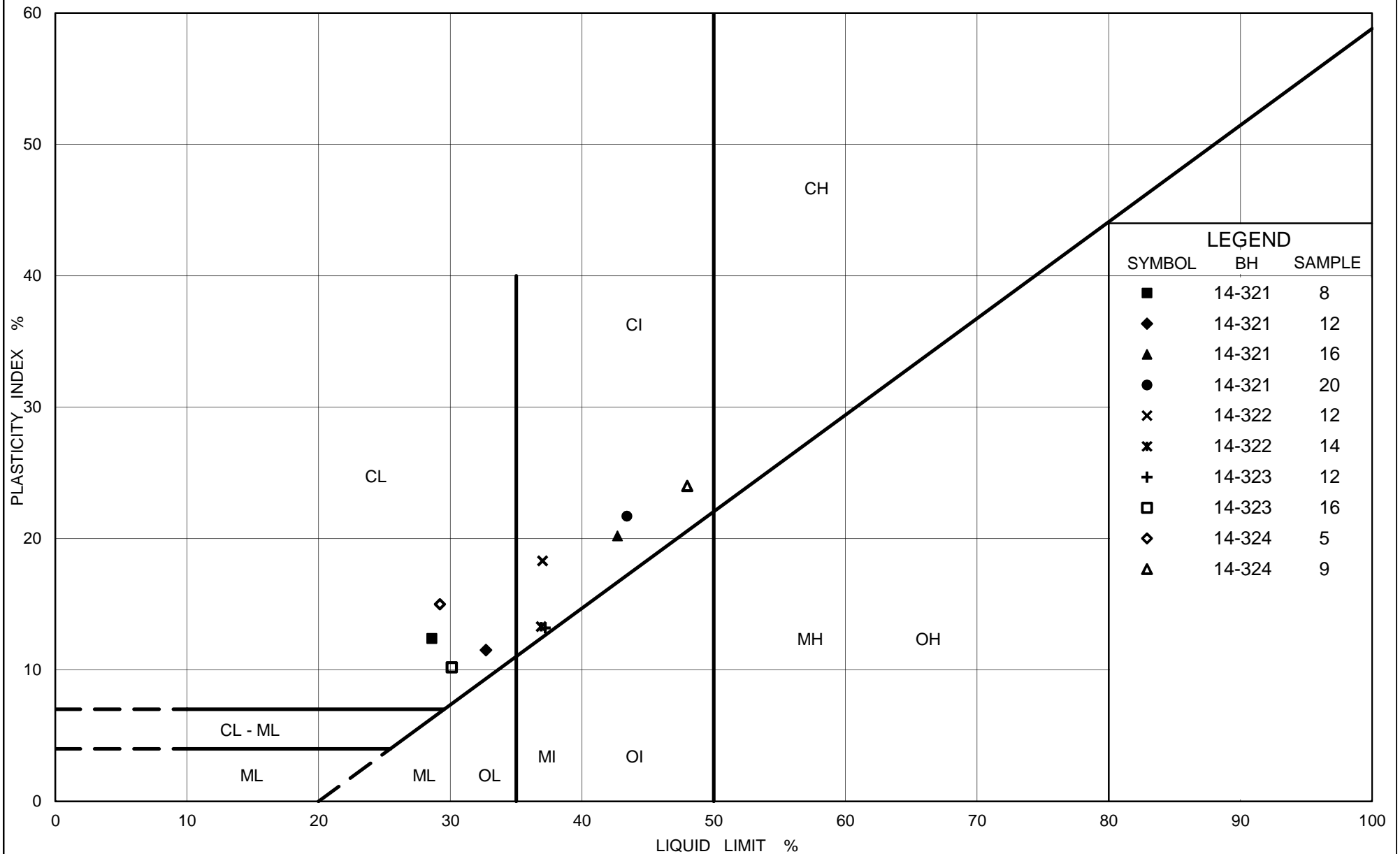
Borehole	Sample	Depth (m)
■ 14-322	12	8.38-8.99
◆ 14-324	5	2.44-3.05

GRAIN SIZE DISTRIBUTION

FIGURE B4



Borehole	Sample	Depth (m)
■ 14-321	12	6.71-7.32
◆ 14-321	20	11.59-12.20
▲ 14-322	14	11.43-12.04
● 14-323	12	10.67-11.28
□ 14-323	16	16.01-16.62
◇ 14-324	9	8.54-9.15



Ontario

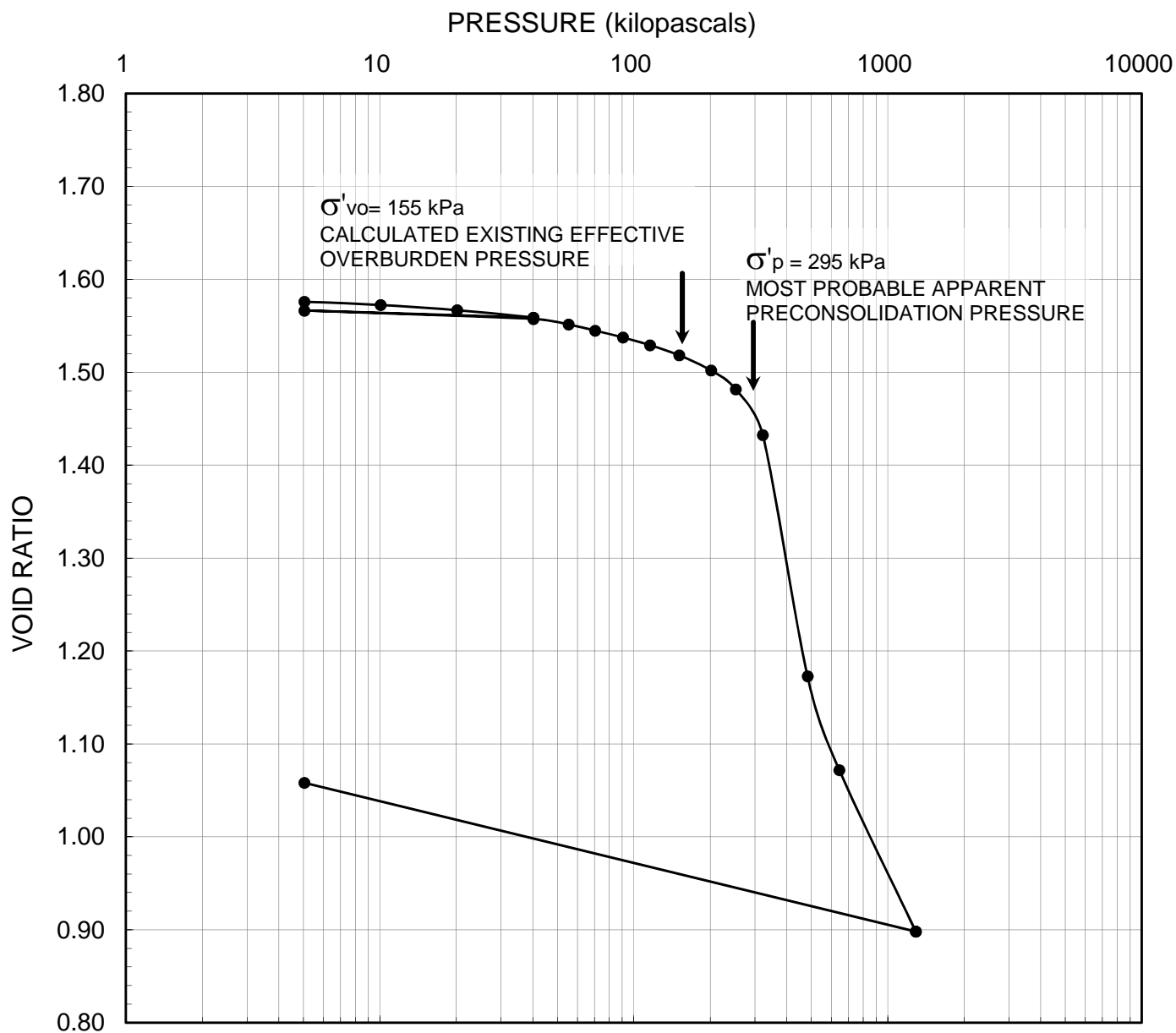
Ministry of Transportation

PLASTICITY CHART CLAYEY SILT to SILTY CLAY

FIG No. B5

Project No. 12-1121-0099/1320

Compiled By : MI Checked By : CNM



LEGEND

Borehole:	14-322	$w_i = 57\%$	$S_o = 100\%$	$\gamma = 16.6 \text{ kN/m}^3$
Sample:	14	$w_f = 38\%$	$e_o = 1.58$	$G_s = 2.78$
Depth (m):	11.9	$w_l = 37\%$	$C_c = 1.47$	
Elevation (m):	106.2	$w_p = 24\%$	$C_r = 0.010$	



**Golder
Associates**

SCALE	AS SHOWN
DATE	11/14/14
CADD	N/A
ENTERED	MI
CHECK	CNM
REVIEW	KSL

TITLE

CONSOLIDATION TEST RESULTS

FILE No.	Consolidation summary
PROJECT No.	12-1121-0099/1320
REV.	0

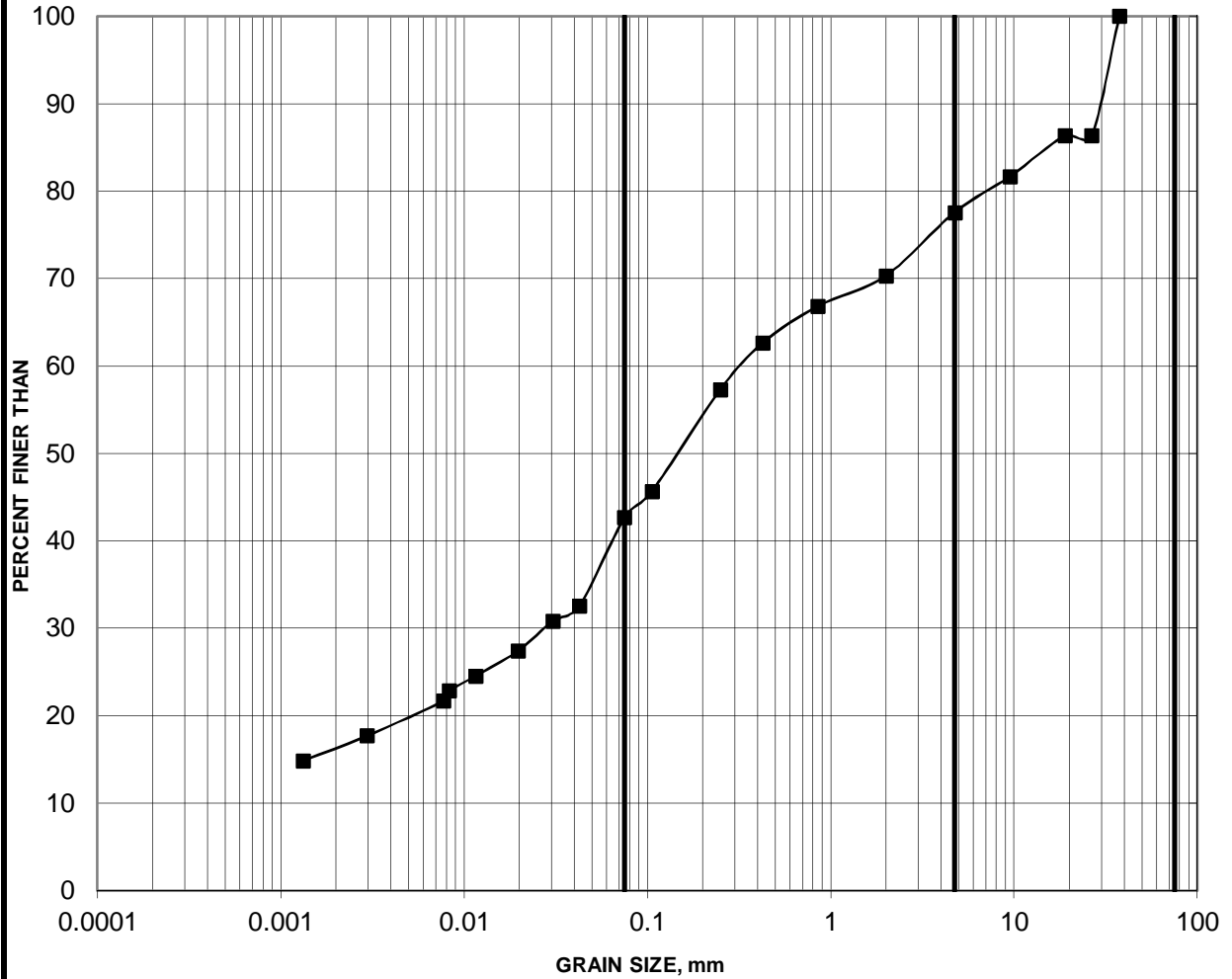
FIGURE

B6

GRAIN SIZE DISTRIBUTION

FIGURE B7

Gravelly Silty SAND

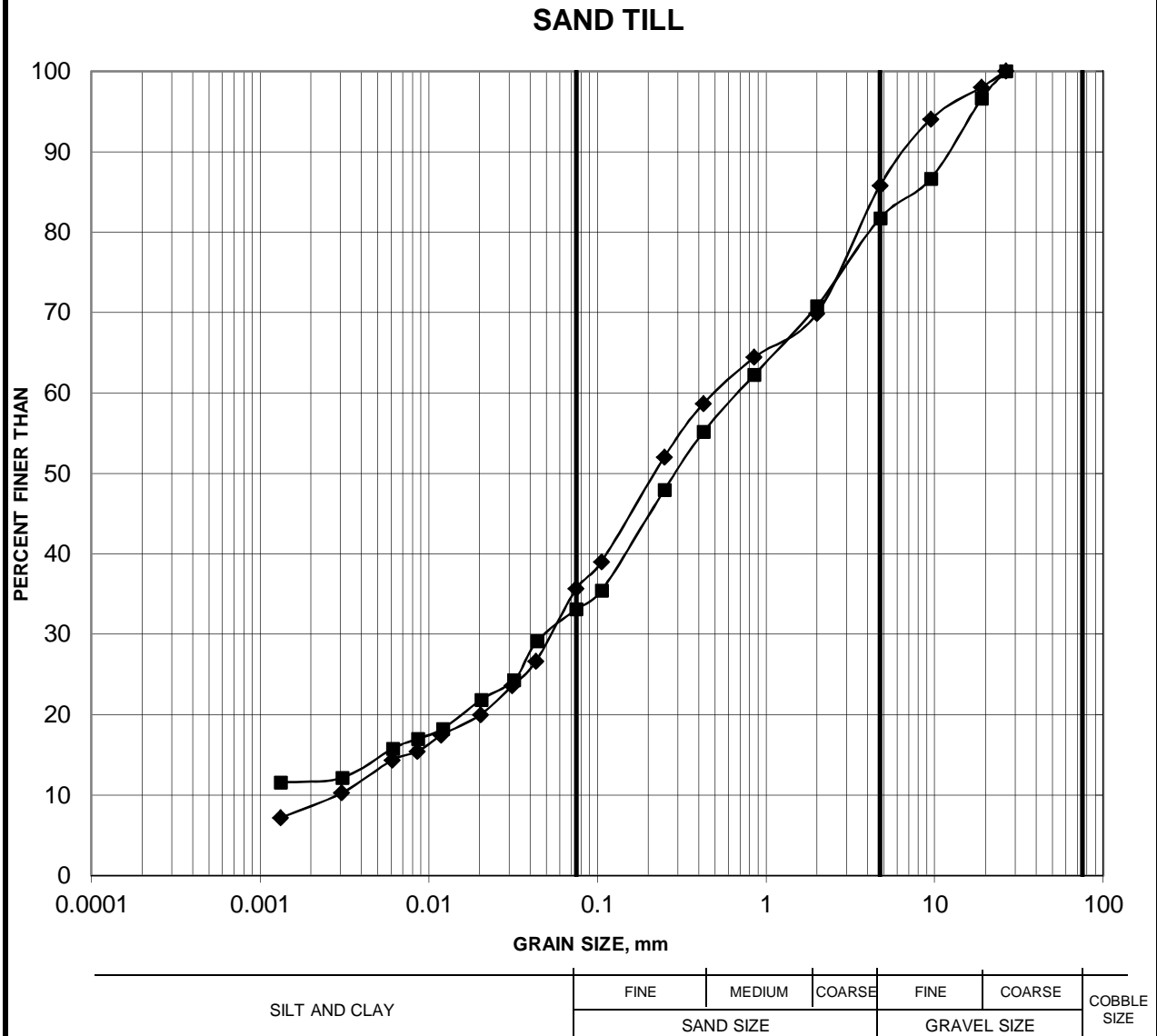


SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
—■ 14-323	15	15.24-15.85

GRAIN SIZE DISTRIBUTION

FIGURE B8



Borehole	Sample	Depth (m)
■ 14-322	18	18.29-18.90
◆ 14-324	12	11.59-12.20



APPENDIX C

Sample Non-Standard Special Provision

GROUND WATER AND SURFACE WATER CONTROL – Item No.

Special Provision

Control of the surface water and groundwater will be necessary for the construction of the culvert replacement to allow excavation and foundation construction to be carried out in dry conditions. The surface water flow could be diverted by pumping from behind a temporary cofferdam(s) or passed through or around the culvert area by means of a temporary pipe. 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 soils.

Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all labour and materials to complete the work.

END OF SECTION

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

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