



February 2016

REPORT ON

**Foundation Investigation and Design
Landons Bay Highway and Trail Bridges
Thousand Island Parkway
United Counties of Leeds and Grenville, Ontario
W.P. 4091-10-02 (Highway Bridge) and
W.P. 4125-10-01 (Trail Bridge)**

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REPORT



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Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION.....	2
3.0 INVESTIGATION PROCEDURES	5
4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS	8
4.1 Regional Geology and Available Geologic Information.....	8
4.2 Subsurface Conditions.....	8
4.2.1 Causeway Embankment Fill.....	9
4.2.2 Silty Clay to Clay	11
4.2.3 Interlayered Silty Clay and Sandy Silt	11
4.2.4 Sand.....	12
4.2.5 Bedrock.....	12
4.3 Groundwater Conditions	12
5.0 CLOSURE.....	14

PART B – FOUNDATION DESIGN REPORT

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS.....	15
6.1 General.....	15
6.2 Foundation Options	15
6.3 Shallow Foundations	17
6.3.1 Founding Elevations and Preparation Requirements.....	17
6.3.2 Geotechnical Resistance/Reaction	18
6.3.3 Resistance to Lateral Loads.....	18
6.4 Steel H-Pile or Steel Pipe (Tube) Foundations	18
6.4.1 Founding Elevations.....	18
6.4.2 Axial Geotechnical Resistance/Reaction.....	19
6.4.3 Resistance to Lateral Loads.....	19
6.5 Site Coefficient	20
6.6 Lateral Earth Pressures for Design.....	21
6.6.1 Static Considerations	21



6.6.2	Seismic Considerations.....	22
6.7	Approach Embankments	23
6.8	Construction Considerations.....	25
6.8.1	Excavation and Temporary Protection Systems	25
6.8.2	Groundwater Control during Construction.....	26
6.8.3	Obstructions	27
7.0	CLOSURE.....	28

TABLES

Table 1 – Comparison of Replacement Structure Foundation Alternatives

Table 2 – Comparison of EPS Lightweight Fill Alternatives for Settlement Mitigation

DRAWINGS

Drawing 1 – Landons Bay Bridges – Borehole Locations and Soil Strata

FIGURES

Figure 1 – Static Stability Analysis – Existing Embankment – Station 17+635

APPENDICES

APPENDIX A Borehole, Drillhole, and Test Pit Records

Lists of Abbreviations and Symbols

Lithological and Geotechnical Rock Description Terminology

Records of Boreholes 14-1 to 14-4, and 15-3

Test Pit Records 15-1 and 15-2

APPENDIX B Laboratory Test Results

Figure B-1 – Grain Size Distribution – Embankment Fill

Figure B-2 – Grain Size Distribution – Silty Clay to Clay

Figure B-3 – Plasticity Chart – Silty Clay to Clay

Figures B-4 and B-5 – Consolidation Test Results –Silty Clay to Clay

Figure B-6 – Grain Size Distribution – Sandy Silt

Figure B-7 – Grain Size Distribution – Silty Clay

Figure B-8 – Plasticity Chart –Silty Clay

Figure B-9 – Grain Size Distribution – Sand

Point Load Test Worksheet



APPENDIX C Non-Standard Special Provisions and MTO Guidelines

Working Slab

MTO Guidelines for 'Backfill to Structures Adjacent to Rock Embankment Approaches

Expanded Polystyrene Embankment

Groundwater and Surface Water Control

Obstructions

Vibration Monitoring



PART A

FOUNDATION INVESTIGATION REPORT

SITE NOS. 16-103/1 AND 16-103/2

LANDONS BAY HIGHWAY AND TRAIL BRIDGES

THOUSAND ISLAND PARKWAY

UNITED COUNTIES OF LEEDS AND GRENVILLE, ONTARIO

W.P. 4091-10-02 (HIGHWAY BRIDGE) AND 4125-10-01 (TRAIL BRIDGE)



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services associated with numerous culvert and bridge rehabilitations and/or replacements at various locations in the Eastern Region of Ontario as part of the 23 Structures MEGA 3 project.

This foundation investigation report addresses the proposed replacement of the existing highway bridge (South) and pedestrian trail bridge (North) (Site Nos. 16-103/1 and 16-103/2, respectively) at the Thousand Islands Parkway crossing of Landons Bay in the United Counties of Leeds and Grenville, Ontario (WP 4091-10-02 and WP 4125-10-01, respectively).

Initially, the bridge replacements at this site were planned to be undertaken as Design-Build projects. It is now understood that Dillon will be completing the detailed design of the bridge replacements. The purpose of the foundation investigation was to assess the subsurface conditions in the area of the proposed bridge replacements and associated approach embankment areas by excavating test pits, 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 August 2012. In addition, Golder's letter dated April 2, 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 December 2012.



2.0 SITE DESCRIPTION

The Thousand Islands Parkway (TIP) highway and trail bridges at Landons Bay are located in the western portion of the TIP approximately 7.5 km east of the western interchange of Highway 401 and the TIP as shown on the Key Map on Figure 1.

Landons Bay is approximately 450 m wide at the crossing location and contains an island, understood to be referred to as Snake Island, approximately midway across the bay. The existing highway and trail bridges are located in the western side of the bay approximately 50 m west of the island and have an approximate east-west orientation. Causeway embankments which typically have a crest to crest width of slightly less than 30 m have been constructed on either side of the bridges. At the bridge locations, the channel between the causeway embankments is about 15 m wide at normal water level and water flows from north to south.

Based on observations made during an initial site reconnaissance, the causeway embankments are constructed out of granitic rock fill materials containing cobble and boulder sized rock pieces within a matrix of granular fill. Photographs of the fill exposed on the sideslopes of the approach embankment are provided below for reference.





No wing walls are present adjacent to the existing abutment foundations. A surficial layer of concrete has been placed over top of the embankment fill materials in some areas near the existing bridge abutments presumably to provide protection against erosion of the steep fill slopes. Ongoing erosion or sloughing of the fill materials has led to localized undermining of the concrete as shown in the photograph below.



Bedrock outcrops are visible both on the west side of the island located immediately east of the bridges as well as along the western shoreline of the bay.

The existing highway and trail bridges are both approximately 31 m long, 10 m wide single-span truss structures with reinforced concrete decks. The south (highway) structure accommodates one lane of traffic in each direction and has a cantilevered sidewalk on the south side of the truss. The north bridge forms part of a pedestrian trail system and no longer carries highway traffic.

An available design bridge drawing (Drawing D-2617-1 titled Proposed Bridge over Landons Bay – St Lawrence River Road and dated March 1939) indicates the bridges are supported on foundations consisting of 1.8 m wide, reinforced concrete abutment footings with founding elevations of about 248 feet (~75.6 m) perched within the rock fill embankment. This drawing also indicates that:

- The design elevations at the ends of the approach slabs to the bridge decks were approximately 260 feet (~79.2 m);
- The design of the bridges referenced average and high water level elevations within the bay of approximately 245.3 feet (~74.8 m) and 248.8 feet (~75.8 m), respectively;
- The elevation of the base of the channel between the rock fill embankments was to be approximately 238.5 feet (~72.7 m);
- The rock fill embankments in front of the abutments were to incorporate a bench at the high water level; the embankments below this level were to be constructed at 1.5 horizontal to 1 vertical (1.5H;1V) while the embankments above that level were to be constructed at 1 horizontal to 1 vertical; and,
- The bridge drawings indicate that rock fill would be placed above “Muck” materials.

The following provides a discussion of the condition of the existing bridges based on information provided to us by Dillon.



Highway Bridge

The structural steel on the highway bridge varies from good to poor but is generally in good condition with light corrosion of the truss members, floor beams and stringers. Severe corrosion with section loss was noted on the lacing and batten plates on the top chords, and localized section loss on some vertical and diagonal truss members. Water staining on the ballast walls and abutment walls suggest the joints are leaking. The steel roller bearings on the east abutment are corroded and seized, the keeper pins are deformed and the east bearings are in contact with the ballast wall. Concrete had been removed on the east ballast wall to re-establish a gap at the main gussets and allow for bridge expansion. The abutments have map cracking on previous patches, light to medium scaling and delaminations on the abutment walls, and wide cracks, efflorescence, honeycombing. There is very severe spalling/delamination and exposed reinforcing steel on the ballast walls.

Differential movement of the substructure to the superstructure is suggested by the bearing to ballast wall contact described above. This may be a result of sliding or overturning of the existing abutment footings which are understood to be under-designed for overturning.

Trail Bridge

The structural steel trusses on the trail bridge are generally in good condition with light corrosion and failure of the protective coating. Floor beams have light to moderate corrosion, pitting, flaking and breakdown of the protective coating. The end diaphragms have severe corrosion. The deck soffit is in poor condition with delaminations, spalls with exposed reinforcing steel, severe honeycombing, stained cracks and efflorescence stalactites. The abutments have wide stained cracks, efflorescence, scaling and delaminations. A wide crack and possible settlement of the north end of the abutments was noted.



3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out two stages. During the first stage, a preliminary investigation was carried out for a design-build project in May 2014. A second stage of investigation for a detail design was carried out in November 2015. Overall, five boreholes (numbered 14-1 to 14-4, and 15-3) and two test pits (numbered 15-1 and 15-2) were advanced at the locations shown on Drawing 1.

The drilling for the first stage of investigation was carried out using a track-mounted drill rig, supplied and operated by Walker Drilling Ltd. of Utopia, Ontario and the drilling for the second stage of investigation was carried out using a truck-mounted drill rig supplied and operated by George Downing Estate Drilling of Grenville-sur-la-rouge, Québec. The test pits were advanced using a track mounted CAT 314C excavator supplied and operated by Glenn Wright Excavating of Ottawa, Ontario.

A variety of drilling methods including advancing casing using down-the-hole hammer, casing advancer systems and coring were used to try to penetrate through the rock fill embankments. As described in more detail in the table below, refusal to borehole advancement was encountered at the majority of the borehole locations typically as a result of drill equipment being damaged/lost within the borehole due to encountering obstructions (i.e., cobbles and boulders) within the embankment fill or sidewall collapse in some uncased sections of the boreholes. The boreholes at locations BH14-2 to BH14-4 were abandoned prior to penetrating the base of the rock fill. The following table summarizes the drilling methods used and results of the various attempts to advance the boreholes.

Location	Summary of Borehole Advancement
14-1	<p>Attempt 1 – Down-the-hole hammer (DTHH) casing advanced to 5.1 m. Borehole abandoned as casing had sheared off at about 3 m below grade.</p> <p>Attempt 2 – DTHH Casing shoe damaged at 1.5 m depth.</p> <p>Attempt 3 – DTHH advanced to casing refusal at 5.9 m; advanced alternate, smaller hammer to 8.2 m depth; borehole caved at 6.4 m depth when hammer withdrawn. Resume drilling with HW casing until casing shoe worn out at 8.6 m depth. Resume drilling with NW casing to 15 m depth; coring between approximately 12 m and 15 m, until casing shoe worn out. Advanced borehole to termination depth of 45.6 m using BW casing.</p> <p>(Drilled May 12-13 and May 20-28, 2014)</p>
14-2	<p>Attempt 1 – DTHH advanced to 6.4 m. Borehole caved within rock fill upon drill string removal.</p> <p>(Drilled May 13, 2014)</p>
14-3	<p>Attempt 1 – DTHH advanced until casing sheared.</p> <p>Attempt 2 – HW casing advanced in same borehole as Attempt 1 using rotary drilling methods (piece of DTHH casing encountered in HQ core barrel). Borehole abandoned after HW core barrel was sheared off/lost down hole at about 7 m below grade.</p> <p>Attempt 3 – DTHH advanced to 4.3 m without casing. Attempt abandoned due to caving of borehole within rock fill.</p> <p>(Drilled May 14-15, 2014)</p>



Location	Summary of Borehole Advancement
14-4	Attempt 1 – DTTH advanced until casing sheared at 2.8 m. Advanced HW casing to 5.9 m depth at which point casing HW casing shoe and rods sheared off. Attempt 2 – NW casing advanced using a casing advancer to approximately 9.7 m depth at which point casing and casing advancer sheared off within the borehole. (Drilled May 15-16)
15-3	Attempt 1 – Advanced HW casing to 16.8 m depth at which point NW casing was advanced to 35.4 m depth. The borehole was then cored about 3.5 m into the bedrock using NQ-size coring equipment. (Drilled November 10-17, 2015)

As identified above, the boreholes were advanced to depths ranging from 6.4 m to 45.6 m below ground surface. The test pits were advanced to depths of about 4 metres below existing ground surface.

The test pits were advanced to further assess the presence of cobbles and boulders within the existing rock fill embankments and the nature of the granular matrix. The soils exposed on the sides of the test pits were classified by visual and tactile examination. No sampling was carried out during the test pit investigation.

Limited sampling was carried out within the rock fill materials during the borehole investigation; Standard Penetration Test (SPT) sampling was typically attempted where decreased drilling resistances were observed. Within the native overburden soils encountered in Boreholes BH 14-1 and 15-3, soil samples were typically obtained at 0.75 m to 3 m intervals of depth using a 50 mm outside diameter split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586-08a).

During the first phase of investigation, the undrained shear strength of the native, clayey overburden soils were determined using a vane that could be inserted through the BW casing. During the second stage of investigation (i.e., for BH 15-3), MTO 'N' and 'B' vanes were used to determine the undrained shear strengths of the native clayey soils.

A standpipe piezometer was installed in Borehole 15-3 to monitor the groundwater level at the site and for hydraulic conductivity testing. The standpipe consists of a 51 mm diameter rigid PVC pipe with a 3.1 m long slotted screen section, installed within silica sand backfill and sealed by a section of bentonite pellet backfill. The groundwater conditions were observed in the open boreholes/test pits during and immediately following the drilling/excavating operations. The boreholes were backfilled with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended). The test pits were backfilled upon completion of excavating.

The field work was supervised on a full-time basis by a member of Golder's engineering staff who located the boreholes and test pits in the field, observed the drilling and excavating, sampling, and in situ testing operations, and logged the subsurface conditions encountered in the boreholes and test pits. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Ottawa for further examination and laboratory testing. Index and classification tests consisting of water content determinations, grain size distribution analyses and Atterberg Limit testing were carried out on selected soil samples. In addition, diametral point load tests were also conducted on samples of the embankment rock fill materials obtained near the base of the fill in Borehole 14-1 during coring.



FOUNDATION REPORT
SITE NOS. 16-103/1 AND 16-103/2 - LANDONS BAY HIGHWAY AND TRAIL BRIDGES

The groundwater level in the piezometer in Borehole 15-3 was measured on December 3, 2015, at which time a short term constant head pumping test was carried out to assess the hydraulic conductivity and flow rate for the embankment fill.

The borehole and test pit locations and ground surface elevations were surveyed by Golder Associates Ltd. using a Trimble R8 GPS unit. The borehole and test pit locations, including MTM NAD83 northing and easting coordinates, and ground surface elevations referenced to Geodetic datum are summarized in the following table and are shown on Drawing 1.

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole / Test Pit Depth (m)
15-1 ^A	4912481.9	339225.5	78.8	4.0
15-2 ^A	4912492.8	339182.8	78.7	4.0
15-3	4912473.4	339224.9	79.1	38.9
14-1	4912491.2	339184.6	78.8	45.6
14-2	4912482.7	339221.2	78.9	6.4 ^B
14-3	4912475.2	339180.6	79.1	7.0 ^B
14-4	4912465.5	339219.3	79.2	9.7 ^B

Note: A – Indicates a test pit.

B – Refusal to borehole advancement encountered within embankment rock fill materials.



4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology and Available Geologic Information

The site is located within the “Leeds Knobs and Flats” physiographic region which consists predominantly of knobs/outcrops of rock surrounded by deposits of water lain clays, sands and gravels. Bedrock in the immediate vicinity of the site is expected to consist of Precambrian, granitic bedrock.

No site specific information for the Thousand Islands Parkway crossing site at the Landons Bay site was available in the MTO Pavement and Foundations Section’s GEOCRE database. Furthermore, other than identification that the rock fill would be placed over muck materials, no subsurface information was available on the available 1939 bridge drawings.

Information from the Ministry of Environment (MOE) water well database for wells located near the site was collected and reviewed. Review of the well records closest to the site indicates that wells typically encountered red, grey or black granite bedrock at shallow depth although surficial shale and/or clay deposits were identified to depths of up to about 6 m below ground surface at some well locations.

4.2 Subsurface Conditions

As part of the current investigation, boreholes were advanced at 5 locations (designated as 15-3 and 14-1 to 14-4) and test pits were advanced at two locations (designated as 15-1 and 15-2) in the vicinity of the existing Thousand Island Parkway highway and trail bridge structures at Landons Bay underpass. The borehole locations, ground surface elevations and interpreted stratigraphic conditions are shown on Drawing 1.

The subsurface soil and groundwater conditions encountered in the test pits advanced as part of the current investigation are given on the test pit records contained in Tables 1 and 2. The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the current investigation and the results of in situ and laboratory testing are given on the borehole records contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B7 contained in Appendix B.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile on Drawing 1 are inferred from observations of drilling progress and 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 site consist of causeway embankment fill, comprised of granular/rock fill, underlain by an extensive deposit of stiff to very silty clay that extends to approximately 35 to 36 m depth. The silty clay in borehole 14-1 (i.e., the west end of the bridges) is underlain by an interlayered deposit of silty clay and sandy silt to a depth of approximately 41.2 m which in turn is underlain by a deposit of dense to very dense sand. The silty clay in borehole 15-3 (i.e., the east side of the bridge) is underlain by a veneer of sandy silt over granitic bedrock.

A more detailed description of the subsurface conditions encountered in the boreholes and test pits are provided in the following sections.



4.2.1 Causeway Embankment Fill

A surficial asphalt pavement layer, approximately 100 mm in thickness, was encountered at roadway surface at the locations of Boreholes 14-3 and 14-4. A separate geotechnical (pavement) investigation will be completed for this project and further details on the pavement structure fill will be provided in the geotechnical (pavement) investigation report.

The test pits were advanced through the north 'shoulder' of the pedestrian trail, on either side of the bridge. About 240 to 250 mm of granular material was encountered over a concrete deck about 750 to 760 mm in thickness.

The pavement structure fill and concrete deck, where encountered, are underlain by granular, causeway embankment materials. Based on observations of excavating and drilling advancement made, and samples collected during the drilling operations, as well as review of the materials exposed at the surface of the causeway embankments, the causeway embankments are inferred to have been constructed of granitic rock fill materials. In some areas, the down-the-hole hammer and/or drill casing were noted to drop during advancement suggesting the rock fill consists primarily of cobble and boulder sized particles containing voids. In other areas, the rock fill consists of cobbles and boulders within a matrix of sandy gravel. During the test pit investigation, two distinct layers were observed within the upper portion of the rock fill, below the existing concrete deck for the trail bridge; about 1.3 m of silty sand and gravel with cobbles were observed over at least 1.7 m of cobbles and boulders in a matrix of sand and gravel. Numerous cobbles and boulders were observed during the test pit excavations, as shown on the test pit records in Tables 1 and 2, and in the photographs below taken in test pits 15-1 and 15-2, respectively.



As identified previously, boreholes advanced into the embankment fill were often required to be abandoned due to breakage of drill equipment within the boreholes. The boreholes at locations BH14-2, BH14-3 and BH14-4 were terminated within the rock fill materials at depths of between 6.4 m and 9.7 m below ground surface. The rock fill materials extended to depths of approximately 15.1 to 16.2 m, corresponding to elevations of about 63.7 to 63.0 m, at the locations of Boreholes 14-1 and 15-3, respectively.



In order to retrieve samples within the rock fill, Standard Penetration Test (SPT) sampling was attempted sporadically (e.g., in isolated areas when drilling resistances decreased (typically in the lower portion of the rock fill) as well as in areas that had previously been drilled using the down-the-hole hammer). Standard Penetration Test (SPT) “N” values measured within the rock fill that had not previously been disturbed by the DTHH ranged from 5 blows to greater than 50 blows per 0.3 m of penetration, but generally greater than 10 blows per 0.3 m of penetration indicating the embankment fill is typically compact to very dense.

The results from grain size distribution testing completed on three selected samples of the fill material are shown on Figure B-1 in Appendix B; we note that these results represent only the portion of the rock fill materials that were collected within a 50 mm diameter split spoon sampler. The following photograph displays the rock fill materials that were retrieved during coring carried out between a depth of about 12 m and 15 m below ground surface in Borehole 14-1.



Diametral point load testing carried out on samples of the rock fill from this depth interval measured Point Load Index ($Is(50)$) values of between 9.1 and 22.2 MPa suggesting the unconfined compressive strength of the rock pieces varies from about 200 to 500 MPa.

The base of the rock fill encountered in Boreholes 14-1 and 15-3 extends several metres below the base of the bay adjacent to the TIP. This may be a result of the displacement of bay bottom sediments/muck during causeway embankment fill placement. This hypothesis was further demonstrated in borehole 15-3, where organics were encountered in the lower 0.6 m of rock fill. Organic content testing was carried out in one sample of the organic material recovered; a first test was for a whole sample, and a second test was for the fraction of another sample passing through sieve #4 only. The organic contents were measured to be about 10 and 14 percent, respectively, which indicate a relatively high organic content. Natural water contents for these samples were measured to be about 51 and 73 percent, respectively. It is therefore understood that there exists a thin layer or pockets of compressible materials (e.g., muck, organic materials etc.) beneath or within the lower portions of the rock fill.



4.2.2 Silty Clay to Clay

An extensive deposit of silty clay to clay containing trace sand was encountered beneath the causeway embankment fill in Boreholes 15-3 and 14-1. This silty clay to clay deposit was proven to be about 21 to 19 m in thickness, with the base of the deposit observed at depths of approximately 35 to 36 m, corresponding to elevations of about 43.7 to 42.8 m, respectively.

SPT 'N' values measured in the silty clay to clay deposit ranged from 7 to 27 blows per 0.3 m of penetration. In situ shear vane testing carried out where possible within this deposit measured undrained shear strengths that typically ranged from about 95 kPa to 200 kPa and remoulded shear strengths typically ranging between about 30 and 75 kPa, indicating a very stiff consistency. The calculated sensitivity ratios in this deposit generally range between 2 and 4.

The results of grain size distribution testing carried out on six samples of the silty clay to clay are provided on Figure B-2. The results of Atterberg limit testing on nine samples of this deposit measured plastic limits of between 17 and 25 percent and liquid limits of between 33 and 65 percent, as shown on Figure B-3, indicating a clay of generally intermediate to high plasticity. The natural water contents of the silty clay to clay were measured to vary between 21 and 43 percent.

Oedometer consolidation testing was carried out on two samples of the unweathered clay from Borehole 15-3. The results of that testing, which are provided on Figures B-4 and B-5 are summarized in the table below and indicate that this material is overconsolidated, with a preconsolidation pressure of around 650 to 1,000 kPa and overconsolidation ratio of 3.8 to 2.2.

Borehole/Sample Number	Sample Depth/Elevation (m)	Unit Weight (kN/m ³)	σ_p' (kPa)	σ_{vo}' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	Cc	Cr	e _o	OCR
15-3 / 23	19.1 / 60.0	20.0	1000	265	735	0.22	0.007	0.67	3.8
15-3 / 26	23.1 / 56.0	17.9	650	300	350	0.43	0.012	1.15	2.2

Notes:

- σ_p' - Apparent preconsolidation pressure
- σ_{vo}' - Computed existing vertical effective stress
- Cc - Compression index
- Cr - Recompression index
- e_o - Initial void ratio
- OCR - Overconsolidation ratio

4.2.3 Interlayered Silty Clay and Sandy Silt

About 0.5 m of grey sandy silt was encountered below the clay layer in Borehole 15-3, at about elevation 35.4 m.

Below a depth of about 36 m in Borehole 14-1, the silty clay to clay stratum transitions into interlayered deposits of sandy silt to silty clay containing trace sand. These deposits, having a combined thickness of about 5 m, were fully penetrated and proven to extend down to a depth of about 41.2 m corresponding to an elevation of about 37.6 m.



SPT 'N' values measured in the interlayered silty clay and sandy silt deposit in Borehole 14-1 ranged from 8 to 20 blows per 0.3 m of penetration. In situ shear vane testing carried out within these strata measured undrained shear strengths that typically ranged from about 129 kPa to 147 kPa and remoulded shear strengths ranging between about 69 and 103 kPa. Based on the shear vane and SPT results and manual examination of the samples, the silty clay materials are considered to have a very stiff consistency and the sandy silt is considered to be compact.

The results of grain size distribution testing carried out on a sample of the sandy silt and the silty clay are provided on Figures B-6 and B-7, respectively. The results of Atterberg limit testing on one sample of the silty clay measured a plastic limit of about 20 percent and a liquid limit of about 39 percent, as shown on Figure B-8, indicating a soil of intermediate plasticity. The natural water contents of the interlayered silty clay and sandy silt were measured to vary between 20 and 29 percent.

4.2.4 Sand

A sand deposit containing some silt and gravel (locally) was encountered below the interlayered silty clay and sandy silt strata in Borehole 14-1. A seam of silty clay was encountered within the sandy soils in one of the SPT samples.

Standard penetration tests conducted in the sand deposit measured "N" values ranging from 39 to 56 blows per 0.3 m of penetration indicating this stratum is dense to very dense. The results of grain size distribution testing carried out on two samples of the sand are provided on Figure B-9. The measured natural water contents of two samples of the sand range from about 14 to 19 percent.

Borehole 14-1 was terminated within the sand deposit at a depth of 45.6 m below ground surface corresponding to an elevation of approximately 33.2 m. The thickness of the sand deposit was proven to be at least 4.4 m.

4.2.5 Bedrock

Bedrock was encountered beneath the silty clay to clay and veneer of sandy silt in Borehole 15-3, and cored for about 3.5 m depth. The following table summarizes the bedrock surface depth and elevation as encountered at the borehole location.

Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
15-3	79.1	35.4	43.7

The bedrock encountered in the borehole consists of pink, strong granite. The bedrock is slightly weathered to fresh.

The Rock Quality Designation (RQD) values measured on recovered bedrock core samples ranged from about 61 to 100 percent, indicating a fair to excellent quality rock. The RQD values were generally found to increase with depth.

4.3 Groundwater Conditions

Water levels were observed in the boreholes during drilling and test pit operations.



Water levels observed in open boreholes and test pits within the rock fill materials were typically at about 3.8 m to 4 m below ground surface or an elevation of about 75 m. Groundwater levels within the rock fill are expected to be controlled by water levels in the adjacent bay and will be subject to fluctuations both seasonally and as a result of precipitation events. The observed (September 2013) and high water level elevations in the bay are understood to be 74.7 m and 75.7 m, respectively.

The water level measured in the BW casing extending into the sand deposit in Borehole 14-1 was at a depth of about 6.9 m (~Elev. 71.9 m) prior to casing withdrawal and grouting of the borehole.

The groundwater level in the piezometer in Borehole 15-3 was measured on December 3, 2015, at which time hydraulic conductivity testing was also carried out. The piezometer was sealed into the rock fill. The groundwater level in the piezometer and results of the hydraulic conductivity testing is summarized in the table below.

Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Hydraulic Conductivity, k (cm/s)	Date
15-3	79.1	4.8	74.3	6.6×10^{-5}	December 3, 2015

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 Ms. Kim Lesage, P.Eng., and was reviewed by Mr. Fintan Heffernan, P.Eng., the Designated MTO Foundations Contact for Golder for this project.

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PART B

FOUNDATION DESIGN REPORT

SITE NOS. 16-103/1 AND 16-103/2

LANDONS BAY HIGHWAY AND TRAIL BRIDGES

THOUSAND ISLAND PARKWAY

UNITED COUNTIES OF LEEDS AND GRENVILLE, ONTARIO

W.P. 4091-10-02 (HIGHWAY BRIDGE) AND 4125-10-01 (TRAIL BRIDGE)



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation recommendations for the proposed replacement of the existing highway and trail bridges at the Thousand Island Parkway crossing of Landon's Bay. The recommendations are based on interpretation of the factual data obtained from the boreholes and test pits advanced during the current subsurface investigations. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives to carry out the detail design of the foundations for the replacement structures.

Where comments are made on construction, they are provided to highlight those aspects that could affect the detail 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 bridges, which are shown in plan on Drawing 1, consist of single-span, steel truss structures with reinforced concrete decks. The existing bridges are planned to be replaced with two new, single-span permanent bridges; a new two-lane highway bridge located along the same alignment as the existing highway bridge and a one-lane trail bridge located immediately north of the new traffic bridge that will also provide highway traffic access during the traffic bridge construction. A temporary, modular trail bridge is also planned to be constructed to the north of the proposed permanent bridges to maintain trail access during the period of construction of the permanent bridges. Consideration is being given to locating the abutments for the new bridges immediately behind the existing bridge abutment foundations which would result in new bridge spans of approximately 36 m. It is further understood that consideration is being to raising the road grades leading up to the new bridges nominally (for a steel girder bridge) to as much as about 0.4 m if NU concrete girders are utilized.

6.2 Foundation Options

The existing bridges, which are both approximately 31 m long and 10 m wide, are shown in plan on Drawing 1. Based on an available design bridge drawing (Drawing D-2617-1 titled Proposed Bridge over Landons Bay – St Lawrence River Road and dated March 1939), the bridges are single-span steel truss structures with reinforced concrete decks that are supported on foundations consisting of 1.8 m wide, reinforced concrete abutment footings with founding elevations of about 248 feet (~75.6 m) perched within the rock fill embankment.

The existing pavement grade at the highway bridge location is at about Elevation 79.1 m to 79.2 m; the ground surface leading up to the trail bridge is slightly lower. In this area, the Thousand Island Parkway consists of one travelled lane in each direction (i.e., a two-lane highway). The existing highway/causeway embankment surface adjacent to the bridge locations is about 4 m to 4.5 m higher than the observed (September 2013) water level elevation of 74.7 m. It is understood that the seasonal high water level at the site is 75.2 m. Available survey information suggests that the inclination of the sideslopes of the causeway embankments above the water level are variable with some portions of the embankments having inclinations near 1 horizontal to 1 vertical 1H:1V while the overall embankment sideslopes are generally closer to 1.5H:1V.

Based on the proposed bridge geometries and the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments for the new highway and trail bridge structures. 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.



- **Strip/spread footings perched within the approach/causeway embankment:** The existing bridges, which are understood to be approximately 75 years old, are founded on strip/spread footings perched within the granular fill materials that make up the approach embankments and this type of foundation system could be considered for support of the new abutments and associated wing walls/retaining walls at this site. Design geotechnical resistances will be dependent on the composition/density/thickness of rock fill materials present between the underside of footings and the underlying silty clay deposit. Depending on the separation between the new highway and trail bridges, temporary protection systems would be required between these areas during excavation and construction.

It is likely that pockets or layers of compressible organics are present under the rock fill, as found in Borehole 15-3, within the area of proposed grade raise which can lead to some foundation settlement. Settlement mitigation measures may be required for shallow foundations.

Observations made during the field investigations suggest that voids may be present at some locations within the existing fill embankments; however, provision should be made for the construction of concrete working slabs beneath the proposed founding level to fill in any voids that may be present within the existing rock fill embankment, and to provide an acceptable working surface. Based on information provided by Dillon, founding elevations for shallow foundations are proposed to be slightly above the normal water level within the bay. However, localized dewatering within the foundation area may be required to facilitate placement/compaction of the foundation pad materials where the excavations are below the water level existing at the time of foundation construction. Given the coarse nature of the embankment fill and proximity of the excavations to the bay, large groundwater inflows may result and groundwater control measures would be required. Alternatively, consideration could be given to raising founding levels to allow for all, or a portion of, the granular foundation pad to be constructed above the water level.

- **Steel pipe (tube) piles or driven steel H-piles:** Steel pipe (tube) piles or driven steel H-piles are feasible for support of the abutments, and would permit design of conventional abutments. However, the existing rock fill embankments contain numerous cobbles and boulders and, due to the potential for damage or misalignment of the piles, installation of pile foundations through the rock fill is not considered to be practical/feasible without pre-drilling through the rock fill. Temporary liners are also expected to be required within the rock fill to prevent the adjacent fill materials including cobbles and boulders from shifting into the predrilled holes and causing obstructions to pile driving.

Steel pipe or steel H-piles would need to be driven through the silty clay deposit to found within the dense sand or on the bedrock. Higher capacities could be achieved by driving piles to bedrock; in this regard, H-piles would be more likely to penetrate through the sand. Pile driving shoes are recommended to protect the pile tips from damage during driving.

- **Micropile Foundations:** The use of micropile foundations is also considered feasible. Micropiles are typically installed with the use of a down-the-hole hammer system which would aid in installation of the micropiles through the rock fill. However, difficulties similar to those encountered during the foundation investigation (including damage/deflection of the hammer casing) could occur. In addition to installation difficulties, the micropiles would require permanent casings through the rock fill zone to prevent the loss of grout into the rock fill during installation. Furthermore, as bedrock was not encountered on the western



portion of the site during the foundation investigation and is at depth in the eastern portion, the axial resistance of those micropiles would rely predominantly on shaft adhesion (i.e., they would be primarily friction piles). The capacity of the micropiles could be similar to driven piles of the same length but could be lower due to a reduced end-bearing resistance (for the west abutment piles) and smaller pile circumference in comparison to driven piles.

- **Caissons (Drilled Piers):** The installation of caissons (drilled piers) would require pre-drilling and the installation of temporary or permanent liners during caisson construction to control potential groundwater inflow and ground losses in the existing embankment fill. In addition, a stratum suitable to provide end-bearing resistance was not encountered on the western portion of the site within the depth of investigation (i.e., within about 46 m of existing road grade). For these reasons, caissons/drilled piers are not considered a practical option for the support of the bridge structures and are not discussed further in this report.

Based on the above considerations and the available information, both shallow foundations and deep (pile) foundation options are considered feasible for the support of the new bridge abutments. Shallow foundations are preferred from a geotechnical/foundations perspective due to the anticipated difficulties and complexities associated with installing deep foundations through the rock fill and the depth to a suitable bearing stratum for pile foundations.

6.3 Shallow Foundations

6.3.1 Founding Elevations and Preparation Requirements

Shallow foundations are considered feasible for the support of the new bridge abutments, temporary modular bridge, and flared retaining walls.

If shallow foundations are adopted for the replacement structures, spread/strip footings would be founded within the rock fill embankment. Voids may be present within the existing rock fill embankments; therefore, provision should be made for the construction of a levelling mat of lean concrete (i.e., mud slab) beneath the proposed founding level to fill in any voids that may be present within the existing rock fill embankment, and to provide an acceptable working surface.

Based on information provided by Dillon, the founding elevations for shallow foundations are currently proposed to be located at an elevation of around 75.2 m which is slightly above the observed water level within the bay and below the high water level. For this founding elevation, localized dewatering within the foundation area would likely be required to facilitate placement/compaction of the lower portion of the foundation pad materials and mud slab. A permit to take water would be required for excavations below the groundwater level.

The new bridge foundations should be provided with a minimum of 1.5 m of earth cover (i.e., be 1.5 m below the lowest surrounding grade) to provide adequate protection against frost penetration, per Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*). The use of rigid insulation (Styrofoam) could be considered as an alternative to, and/or used in conjunction with, earth cover for frost protection purposes. In this case, the insulation beneath the abutment footings should extend out beyond the edges of the footings a minimum distance of 1.5 m minus the depth of soil cover. The insulation must be provided with sufficient cover to resist buoyancy effects during flood events and should be wrapped in polyethylene sheeting. High density polystyrene rigid foam insulation, such as DOW Highload 100 or equivalent is recommended.



The foundation subgrade materials should be inspected prior to foundation construction, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to check that any softened/loosened soils or other unsuitable material have been removed. A concrete working slab (at least 100 mm thick concrete slab with a compressive strength of 20 MPa) should be placed beneath the proposed founding level to fill in any voids that may be present within the existing rock fill embankment, and to provide an acceptable working surface. A Non-Standard Special Provision (NSSP) for the supply and installation of a working slab should be included in the contract documents and a sample has been provided in Appendix C of this report.

6.3.2 Geotechnical Resistance/Reaction

Strip or spread footings up to 4.5 m in width that are placed on a properly prepared concrete working slab perched within the rock fill approach embankments at elevations of about 75 to 77 m may be designed based on a factored geotechnical resistance of 350 kPa at Ultimate Limit States (ULS) and a geotechnical resistance of 150 kPa at Serviceability Limit States (SLS, for 25 mm of settlement). The geotechnical resistances should be reviewed if the selected footing width or founding elevation differs from those given above.

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 2006)* and its *Commentary*.

In view of the overturning problems with the existing bridge foundations, the new foundations should be designed to resist overturning.

6.3.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade (i.e., concrete working slab) should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on a concrete working slab that is cast on top of rock fill, the coefficient of friction, $\tan \delta$ or $\tan \phi'$, may be taken as follows:

- Cast-in-place footing to concrete working slab: $\tan \delta = 0.6$.
- Cast-in-place concrete working slab to rock fill: $\tan \phi' = 0.65$.

The resistance to lateral loads could be increased by constructing a shear-key at the bottom of the footing. The design of shear keys would require a specific analysis taking into consideration the magnitude of the horizontal loading, the magnitude of the vertical loading, and any variations in the bearing pressure due to overturning moments.

The above values assume that the subgrade materials will not be disturbed by construction activities or groundwater inflow.

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

6.4.1 Founding Elevations

The abutments and associated wingwalls may be supported on steel pipe piles (tube) or steel H-pile piles founded in the dense to very dense sand encountered below the silty clay deposit (west end structures) or on granite bedrock (east end structures).



The pile caps should be constructed at a minimum depth of 1.5 m for frost protection purposes, per OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*). The surface of the dense to very dense sand (with SPT 'N' values ranging from 39 to 56 blows) was encountered at an elevation of about 37.6 m in Borehole 14-1. Based on the available information, an estimated pile tip elevation of 34 m may be used for piles founded within the sand (i.e., west end structures). For the east end structures, where bedrock was encountered below the silty clay to clay deposit in Borehole 15-3, an estimated pile tip elevation of 43.5 m may be used for piles founded on the bedrock.

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 soil deposits. Due to the presence of cobbles and boulders within the approach embankments, it is anticipated that pre-drilling will be required to reduce the potential for driving the piles off alignment, or damaging the pile tips in the very dense soil deposit. It is recommended that temporary liners be used during pre-drilling to support the side walls of the drill hole and minimize loss of ground/movement of cobbles and boulders into the drill hole.

The piles should be reinforced at the tip with driving shoes or bearing points to reduce the potential for damage to the piles during driving, 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/Reaction

For closed-end, concrete filled 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.) driven to the estimated tip elevations provided in Section 6.4.1, the factored axial resistance at ULS may be taken as 900 kN for the western structures (on dense to very dense sand, noting that refusal was not reached during the investigation) and 2,000 kN for the eastern structures (on bedrock). The geotechnical reaction for an individual pile at SLS will not govern and may be higher than the factored geotechnical resistance at ULS. Similar axial resistances may be used in the design for HP 310x110 piles; however for the western structures, the H-piles may penetrate further into the sand if a soil plug does not develop at the tip of the pile.

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.

6.4.3 Resistance to Lateral Loads

Lateral loading can typically be resisted fully or partially by the horizontal component of battered piles; however, the installation of battered/inclined piles (including pre-drilling inclined holes and installation of temporary casings) through the rock fill is not considered practical at this site. For vertical piles, the resistance to lateral loading is derived from the soil in front of the piles. The resistance to lateral loading in front of the piles may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h , is determined based on the equations given below (CFEM, 1992, as noted in Section C6.8.7.1 (Table C6.5) and in Section C6.8.7.3 of the *Commentary to CHBDC*):



For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where: n_h = Constant of subgrade reaction (MPa/m);
 z = Depth (m); and,
 B = Pile diameter/width (m).

For cohesive soils:

$$k_h = \frac{67 S_u}{B}$$

Where: S_u = Undrained shear strength of the soil (kPa); and,
 B = Pile diameter/width (m).

The following values/ranges of n_h and S_u may be assumed in the structural analyses, using the interpreted stratigraphic conditions as shown on the profile on Drawing 1:

Soil Unit	n_h (kPa/m)	S_u (kPa)
Compact Granular Embankment Fill	4,400	--
Very Stiff to Hard Silty Clay	--	150
Dense to Very Dense Sand	11,000	--

The response of a pile/caisson to lateral loads is highly non-linear and methods that assume linear behavior (such as subgrade reaction theory) are only appropriate for static (non-cyclic) loading where maximum caisson/pile deflections are less than 1 percent of the caisson diameter and the pile material is linear. If one or more of these conditions are not satisfied, then it is recommended that the lateral pile analysis be carried out using p-y curves.

Group action for lateral loading should be considered where the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.2, 1982) as follows:

Pile Spacing in direction of Loading (d = Pile Diameter)	Subgrade Reaction Reduction Factor, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above table.

6.5 Site Coefficient

The Site Coefficient, S, for this site in accordance with Section 4.4.6 of the CHBDC may be taken as 1.2, consistent with Soil Profile Type II.



6.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and on any concrete wingwalls/retaining walls will depend on the type and method of placement 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.

6.6.1 Static Considerations

The following recommendations are made concerning the design of the abutment walls and concrete wingwalls or retaining walls. These design recommendations and parameters assume a level backfill/ground surface behind the walls.

- Select, free-draining granular fill meeting the specifications of OPSS.PROV 1010 Granular A or Granular B Type II should be used as backfill directly behind the walls. However, consideration must be given to the potential for migration of finer material into the existing rock fill which could lead to unacceptable settlement at the bridge approaches. In this regard, backfill material in contact with the rock embankment must be properly graded or a proper filter material used. A geotextile should be placed between the new granular backfill and the existing embankment fill. Guidance on this topic is provided in MTO's "Backfill to Structures Adjacent to Rock Embankment Approaches", included in Appendix C.
- A minimum compaction surcharge of 12 kPa should be included for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS.PROV 501. Other surcharge loadings should be accounted for in the design as required.
- As a minimum, the granular fill may be placed either in a zone with the width equal to at least 1.5 m behind the back of the walls (see Case A in Figure C6.20(a) of the *Commentary* to the CHBDC), or within the wedge shaped zone defined by a line drawn at 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the footing (see Case B in Figure C6.20(b) of the *Commentary* to the CHBDC). Alternatively, the granular backfill could be placed to meet the requirements (1.5H:1V slope to back of backfill and associated frost tapers) identified on OPSD 3101.150 (Case C) which would further reduce the potential for differential movements as a result of frost action.
- For Case A, the pressures are based on the existing approach fill materials and the following parameters (unfactored) may be used:

	Existing Fill
Soil unit weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.27
At rest, K_o	0.43



- For Cases B and C, where the pressures are based on OPSS.PROV 1010 Granular A or Granular B Type II fill behind the wall, the following parameters (unfactored) may be assumed:

	Granular A	Granular B Type II
Soil unit weight	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43

Where the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for the geotechnical design. Where the wall support allows lateral yielding of the stem, active earth pressures should be used in the geotechnical design of the wall structure(s). The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary to the CHBDC*.

6.6.2 Seismic Considerations

Seismic loading must be taken into account in accordance with Section 4.6.4 of *CHBDC*, as it can result in increased lateral earth pressures acting on the abutment stem and any associated wing walls/retaining 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 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:

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

Where: K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);

K_{AE} is the seismic active earth pressure coefficient;

γ' is the effective unit weight of the soil (kN/m³);

- taken as soil unit weights given above for new fill materials;
- taken as 21 kN/m³ for the existing embankment fill materials;

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 Table A3.1.7 of the *CHBDC*, this site is located in Seismic Zone 2, and the site-specific zonal acceleration ratio (A) for the Gananoque area is 0.10.

The Site Coefficient (S) may be taken as 1.2, consistent with Soil Profile Type II in accordance with Section 4.4.6 and Table 4.4 of *CHBDC* (2006). Based on the subsurface conditions at the site, a 30 percent amplification of the ground motion is recommended for design, resulting in an increase in the ground surface acceleration to approximately 0.13g.



The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of $A = 0.13$. These coefficients have been determined in accordance with Sections 4.6.4 and C4.6.4 of the *CHBDC* and its *Commentary*.

Seismic Active Pressure Coefficients, K_{AE}

	Case A	Case B	
	Existing Fill	Granular A	Granular B Type II
Yielding Wall	0.31	0.29	0.29
Non-Yielding Wall	0.39	0.40	0.40

Notes: These seismic K_{AE} values include the effect of wall friction, and assume that the back of the wall is vertical and the ground surface behind the wall is flat.

The above K_{AE} values for yielding walls are applicable provided that the wall can move up to $250A$ (mm), where A is the design zonal acceleration ratio of 0.13. This corresponds to displacements of up to approximately 33 mm at this site.

It is noted that for the low zonal acceleration ratio for this site, the seismic K_{AE} values are similar to or less than the static values of K_a and K_o reported above; negative values of $(K_{AE} - K)$ should be not be incorporated into the design.

6.7 Approach Embankments

The replacement Thousand Island Parkway highway and trail bridges at Landons Bay are planned to be constructed at generally the same locations as the existing bridges and, as such, the existing causeway/approach embankments will be utilized. Based on information provided by Dillon, the roadway grade leading up to the replacement bridges is proposed to be raised nominally to about 0.4 m depending on the type of bridge girders used. No widening of the embankments is planned.

6.7.1 Embankment Settlement

The results of the foundation investigation indicate that the causeway/approach embankments are constructed out of granular/rock fill materials that are underlain by an extensive overconsolidated silty clay deposit. A layer of compressible organic material some 0.6 m thick was encountered at the bottom of the rock fill in one of the boreholes advanced in the area of the east embankments. There likely exists a thin layer or some pockets of compressible materials (e.g., muck, organic materials etc.) beneath or within the lower portions of the rock fill in the area of both embankments as a result of bay bottom sediments/muck during causeway embankment fill placement.

Settlement of the existing embankments has likely occurred over time since the original causeway and bridge construction due to both compression and creep of the rock fill. However, given that the existing embankments are understood to have been in place for a period approaching 75 years, it is expected that the magnitude of ongoing creep of the rock fill will be nominal. The underlying very stiff to hard clay is virtually incompressible under the embankment and foundation loading.

The additional loading imposed by any proposed grade raises would result in further settlement of the embankments and layers/pockets of compressible materials. Embankment settlements of less than 40 mm are anticipated to occur based on the proposed grade raise of 0.4 m or less.



The above estimates do not include compression of the new fill materials used to raise the embankment, 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 raise, assuming approximately 98 percent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. For this site, it is anticipated that new fill materials placed to raise the embankment would form the new pavement structure and would be comprised of granular fill. In this case, settlement of the fill itself is expected to occur essentially during embankment construction.

Any topsoil/organic deposits and any softened/loosened soil should be stripped from beneath any areas where approach embankments grade raises are proposed prior to placement of new fill materials.

If these settlements cannot be tolerated, about 0.6 m of lightweight fill materials (such as EPS Geofoam) could be used for the embankment construction, placed below the pavement structure, thereby reducing the stress increase on the compressible organic deposit at depth to a level so that the settlements would be minimized. A Non-Standard Special Provision for the supply and installation of EPS fill should be included in the contract documents and a sample has been provided in Appendix C of this report. A comparison of options for the EPS placement as a method of mitigating the settlements is provided in Table 2 following the text of this report.

Other light weight fill materials could also be considered, such as blast furnace slag or cellular/foamed concrete. However it is considered that, in this case, the unit weights of these materials are not sufficiently low to achieve the needed reductions in the final stress level.

The Geofoam will need to be covered with a concrete slab to protect it from being overstressed by the traffic loads; overstressing of the Geofoam could lead to rutting of the pavement surface. A thickness of 125 millimetres is typical for the protective slab.

A sufficient pavement granular thickness is required to limit the potential for premature icing of the roadway due to the insulating properties of the Geofoam. From that perspective, a minimum of 800 millimetres combined thickness of granular base and subbase should be planned.

A suitable Geofoam type would be EPS22 in accordance with ASTM D6817-02, having a compressive strength at 5% strain of at least 115 kilopascals.

The EPS is potentially soluble in hydrocarbons. To guard against dissolution of the EPS in the case of an accidental release and infiltration of fuel (such as could occur in the case of a collision), it is general practice to cover the outside surface of the EPS with polyethylene sheeting.

A 0.3 metre thick layer of OPSS Granular A would be appropriate as a levelling pad beneath the EPS Geofoam, covered with up to 100 mm of mortar sand.

As an alternative to mitigating the potential settlements using lightweight fill materials, one option would be to allow the settlements to occur. This option would involve allowing the roadway to settle and to accept the short-term impacts of the probable settlements on the roadway performance prior to final paving. After monitoring has indicated that the settlements have completed, it would be planned to pad and overlay the roadway to reinstate the roadway profile.

It is understood that the preferred option is a combined alternative of using EPS at the approaches and for a distance of 12 m in plan from the abutments and to allow the rest of the roadway to settle while monitoring the settlements. A separate technical memorandum has been prepared for this alternative and should be included in the contract documents.



6.7.2 Approach Embankment Slope Stability

Static and seismic (pseudo-static) slope stability analyses were completed of the existing approach embankments as well as an embankment incorporating a 0.5 m grade raise. These analyses were carried out with the commercially available slope stability analysis software, SlopeW (part of the software package, Geo-Studio 2007 Version 7, produced by Geo-Slope International Ltd.). A Morgenstern-Price method was used to determine the factor of safety.

Soil Deposit	Bulk Unit Weight	Effective Friction Angle	Effective Cohesion (kPa)	Undrained Shear Strength (kPa)
Embankment Rock Fill	20 kN/m ³	40°	0	–
Very Stiff Silty Clay (Drained)	18 kN/m ³	27°	5	–
Very Stiff Silty Clay (Undrained)	18 kN/m ³	–		150 kPa

The geometry of the existing approach embankment used in the stability analyses has been based on available survey information recently provided by Dillon and incorporates collected survey data. It is noted that this survey data includes detailed information on the embankment sideslopes above the water line as well as spot elevations of the elevation of the bay/river bed but has limited details on the ground surface profile at the toe of the existing embankments below the water line.

A target minimum Factor of Safety (FOS) of 1.3 is normally used in the design of slopes under static conditions. Stability analyses were carried out for a typical cross-section through the existing embankment (cross-section located to the west of the bridges). These analyses indicate that the FOS against instability under static conditions is greater than 1.3 as shown on Figure 1. The embankment also has a suitable factor of safety (greater than 1.1) against instability under seismic conditions.

In addition, stability analysis was carried out to assess the global stability of the proposed temporary pedestrian bridge foundation configuration. These analyses indicate that the FOS against instability under static conditions is also greater than 1.3.

6.8 Construction Considerations

The following sections identify future construction considerations that should be considered during the design stage, and for which appropriate provisions should be made in the Contract Documents.

6.8.1 Excavation and Temporary Protection Systems

The foundation excavations for spread footings or pile caps would extend to a depth of about 5 m below the Thousand Island Parkway road grade. The excavations would be developed through the existing approach embankment fill materials consisting of compact to very dense granular/rock fill materials.

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 granular fill above the water table would be classified as Type 3 soil based on the OHSA. According to OHSA, temporary excavations (i.e., those that are open for a relatively short time period) above the water table should be made with side slopes no steeper than 1H:1V.



Granular fill 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 (i.e., temporary excavation shoring) will be required. Where shoring is required, the protection system should be designed and constructed in accordance with OPSS.PROV 539. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539.

The selection and design of the protection system will be the responsibility of the Contractor. However, the following comments are provided to aid in the costing and assessment of temporary excavation and protection system options for this site.

- The cobbles and boulders present within the existing approach embankments will pose obstructions to the installation of temporary protection systems. If possible (i.e., enough space is available), consideration could be given to providing sufficient separation between the structures to allow for open cut excavations.
- The installation of a driven, interlocking sheetpile system within the approach embankments is not considered practical due to the potential for the sheetpiles to deflect, become damaged, or 'hang-up' as a result of encountering cobbles and/or boulders.
- It is considered that an interlocking pipe wall system would be suitable for the temporary excavation support at this site, based on the embankment conditions observed in the boreholes and test pits, and following discussions with a foundation contractor. The pipe piles would likely be installed using a downhole hammer and these would have to be installed to sufficient depth to provide the necessary passive resistance for the proposed retained soil heights. s

6.8.2 Groundwater Control during Construction

The water level within the approach embankments is expected to be controlled by the water level within the bay. The observed (September 2013) and high water level elevations in the bay are understood to be 74.7 m and 76.2 m, respectively.

Given the coarse, granular nature of the existing embankment fill materials, significant groundwater seepage is expected to occur into any excavations that extend below the water level within the fill. As identified above, provision should be allowed for the installation of a concrete levelling pad beneath abutment footings (if adopted). As the currently proposed founding level is only slightly above the observed water level at the site, excavations for the levelling pad would extend below the water table. Based on the subsurface soil and groundwater conditions and the result of the short term pumping test in Borehole 15-3, a flow rate in the order of 30,000 to 40,000 litres per day is estimated, and therefore a Permit to Take Water (PTTW) should not be required for this site. However, if excavations are to extend to greater depths below the water level, the dewatering rate may exceed 50,000 litres per day, and therefore a Permit to Take Water (PTTW) may be required in this case.

Based on the site conditions (i.e., presence of granular fill containing cobbles and boulders) and proximity of the excavations to the bay, minor lowering of water levels in the foundation excavations may be carried out by pumping from sumps established below the subgrade level.

A sample NSSP has been included in Appendix C for the control of groundwater and surface water.



6.8.3 Obstructions

The existing embankment fill materials at this site consist of granular fill and cobbles and boulders which could affect the installation of protection systems.

A sample NSSP has been included in Appendix C for advancing past obstructions for protection systems.

6.8.4 Erosion and Scour Protection

The existing granular fill materials that make up the approach embankments 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 in the abutment foreshore areas to protect the foundations/pile caps from being exposed. The rip-rap or granular sheeting should be underlain by a geotextile filter fabric and be consistent with the requirements of OPSS 511 (Rip-Rap, Rock Protection, and Granular Sheeting) and OPSS.PROV 1004 (Aggregates – Miscellaneous), with the type/size of material approved by the hydraulic design engineer.

With the proposed footings being installed behind the existing footings, undermining is not expected. The front slope should be provided with rip-rap founded on a granular bedding.

6.8.5 Vibration Monitoring During Pile Driving

The proposed staged bridge construction is to include construction of the highway bridge structure while the newly constructed trail bridge structure is nearby. It is recommended that vibration monitoring be carried out during installation of driven protection systems to assist in maintaining vibration levels within tolerable ranges for the existing portions of bridges, or for any temporary modular structure used at the site.

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

A sample NSSP has been provided in Appendix C for inclusion in the Contract Documents, to address vibration monitoring during installation of protection systems at this site.



7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Kim Lesage, P.Eng., and was reviewed by Mr. Fintan Heffernan, P.Eng., the Designated MTO Foundations Contact for Golder for this project.

GOLDER ASSOCIATES LTD.

Kim Lesage, P.Eng.
Geotechnical Engineer



Fintan Heffernan, P.Eng.
Designated MTO Foundations Contact



KSL/FJH/ob/ob

\\golder.gds\gal\ottawa\active\2012\1121 - geotechnical\12-1121-0193 dillon mega 3 eastern region\foundations\5 - reports\contract a - landons bay site 16-103\12-1121-0193-1215 rpt-002 final landons bay bridges site 16-103 feb2016.docx



FOUNDATION REPORT
SITE NOS. 16-103/1 AND 16-103/2 - LANDONS BAY HIGHWAY AND TRAIL BRIDGES

Table 1
Comparison of Foundation Alternatives
W.P. 4091-10-02 (Highway Bridge) and 4125-10-01 (Trail Bridge) – Thousand Island Parkway

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Option 1 Shallow Foundations	<ul style="list-style-type: none"> Feasible (preferred option) 	<ul style="list-style-type: none"> Minimizes excavation depth. Avoids difficulties installing deep foundations through rock fill. Reduces potential for significant differential settlement between bridges and approach embankments due to creep or minor additional embankment loading. 	<ul style="list-style-type: none"> Some foundation settlement in comparison to end bearing pile foundations. Construction of concrete levelling pad within existing rock fill will likely require excavation below the water table in pervious materials and associated construction dewatering requirements. Roadway protection (i.e., excavation shoring) likely needed, due to traffic staging. System may be complex and require non-standard design due to presence of cobbles, boulders and/or voids within approach embankment fill. 	<ul style="list-style-type: none"> Lowest cost option. 	<ul style="list-style-type: none"> Generally low risk option.
Option 2 Driven Pile Foundations (Steel Pipe or H-Piles)	<ul style="list-style-type: none"> Feasible but difficulties anticipated installing piles through rock fill. 	<ul style="list-style-type: none"> Negligible foundation settlement. Piles can be spliced to account for current uncertainty in depth to bearing strata. Potential for use of integral or semi-integral abutments. 	<ul style="list-style-type: none"> Difficult installation; requires pre-drilling and temporary casing installation through rock fill. Very long piles required to penetrate clay layer and reach bedrock. Bridge settlement would not conform to embankment settlements. Could result in differential settlement of roadway at ends of bridges. Also requires roadway protection system (same comments as for Option 1). 	<ul style="list-style-type: none"> Moderate to expensive cost. 	<ul style="list-style-type: none"> Medium risk option.



FOUNDATION REPORT
SITE NOS. 16-103/1 AND 16-103/2 - LANDONS BAY HIGHWAY AND TRAIL BRIDGES

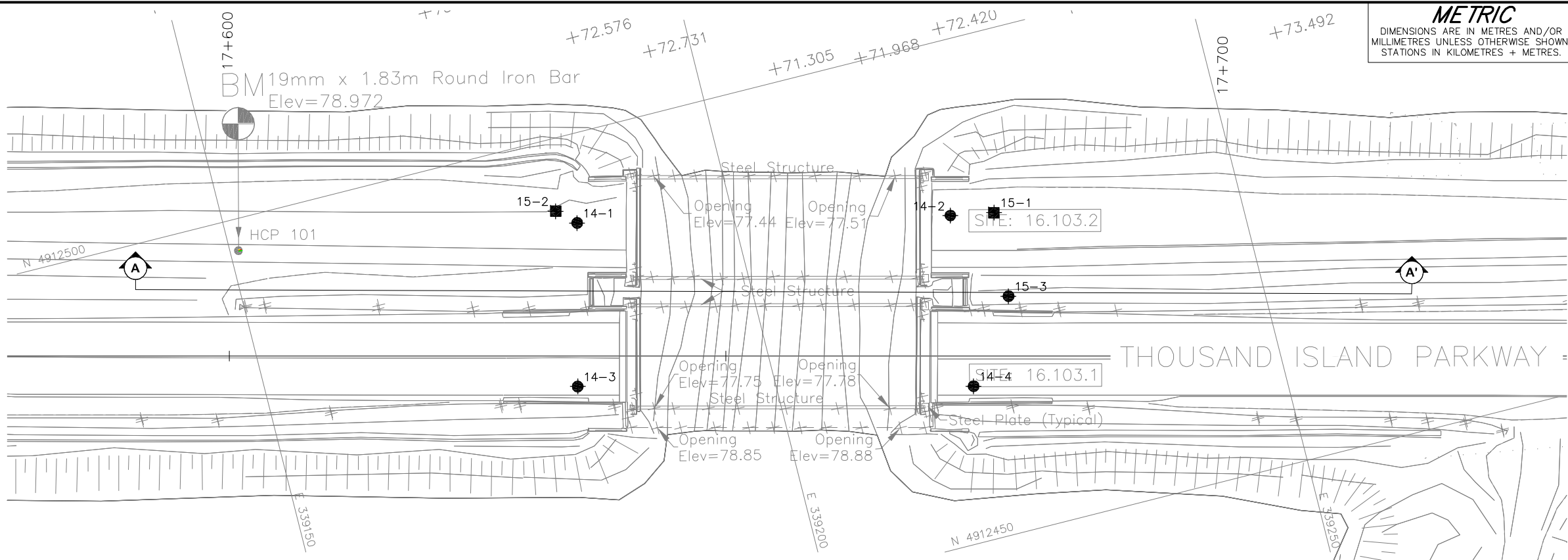
Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Option 3 Micro-Pile Foundations	<ul style="list-style-type: none"> Feasible but would require permanent casings through rock fill for installation. 	<ul style="list-style-type: none"> Down-the-hole hammer Installation methods allow for penetration through rock fill although difficulties could still be encountered. 	<ul style="list-style-type: none"> Permanent casings required to prevent the loss of grout into the rock fill. Axial resistance of the micropiles would rely predominantly on shaft adhesion and, as such, the capacity of the micropiles could be lower than driven piles due to a reduced end-bearing resistance and smaller pile circumference. Does not allow for use of integral abutments. Also requires roadway protection system (same comments as for Option 1). 	<ul style="list-style-type: none"> Moderate to expensive cost. 	<ul style="list-style-type: none"> Medium risk option.
Option 3 Drilled Pier (Caisson) Foundations	<ul style="list-style-type: none"> Not feasible. 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> Very difficult to impractical to install large diameter drilled piers through rock fill embankments. Suitable foundation strata for end-bearing not encountered within 45 m of road surface for east structures. 	<ul style="list-style-type: none"> Most expensive option. 	<ul style="list-style-type: none"> High risk option.



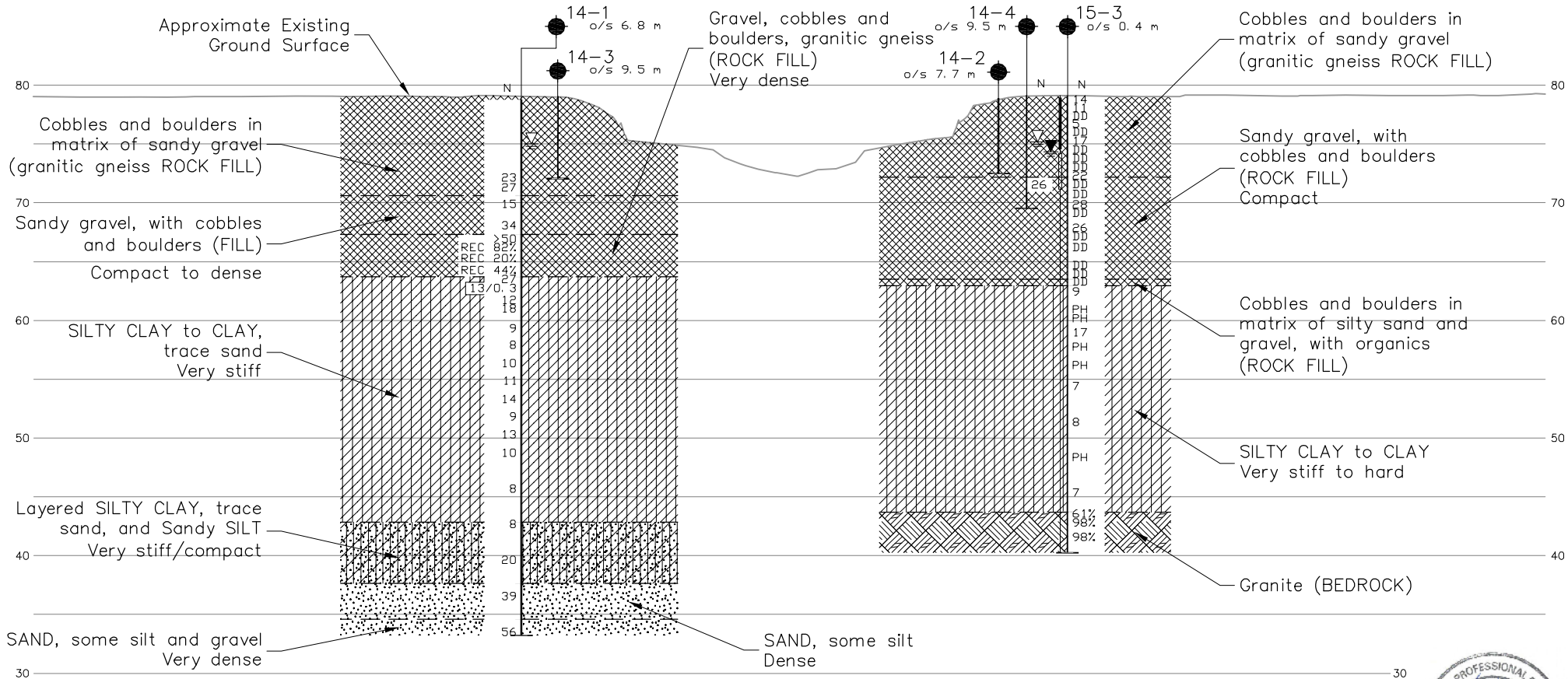
FOUNDATION REPORT
SITE NOS. 16-103/1 AND 16-103/2 - LANDONS BAY HIGHWAY AND TRAIL BRIDGES

Table 2
Comparison of EPS Lightweight Fill Alternatives for Settlement Mitigation
W.P. 4091-10-02 (Highway Bridge) and 4125-10-01 (Trail Bridge) – Thousand Island Parkway

EPS Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
<ul style="list-style-type: none"> Limited amount of EPS: <ul style="list-style-type: none"> 600 mm thick for 8 m 300 mm thick for 4 m 	<ul style="list-style-type: none"> Preferred from a cost/performance overview 	<ul style="list-style-type: none"> Limits settlement within 12 m section Avoids 'bump' at the abutment 	<ul style="list-style-type: none"> Settlement expected beyond 12 m section Padding required in future 	<ul style="list-style-type: none"> Moderate 	<ul style="list-style-type: none"> Uneven settlement beyond 12 m Poor ridability until levelled with padding
<ul style="list-style-type: none"> Full treatment with EPS: <ul style="list-style-type: none"> 1 m thick for 10 m 600 mm thick for 20 m 300 mm thick for 5 m 	<ul style="list-style-type: none"> Preferred technical option 	<ul style="list-style-type: none"> Negligible settlement Little or no maintenance required 	<ul style="list-style-type: none"> Greater excavation limits and EPS thickness required 	<ul style="list-style-type: none"> High 	<ul style="list-style-type: none"> Higher cost
<ul style="list-style-type: none"> No EPS placed 	<ul style="list-style-type: none"> Feasible but with maintenance required 	<ul style="list-style-type: none"> Lower cost 	<ul style="list-style-type: none"> Uneven settlement taking place starting shortly after filling Severe 'bump' at abutment which requires quick remedial action Padding required on more than one occurrence Settlement will cause downdrag on the abutment piles 	<ul style="list-style-type: none"> Low 	<ul style="list-style-type: none"> Uneven pavement Padding required rapidly Safety concerns until padding is provided Monitoring to determine when further padding is required



PLAN
SCALE
0 5 10 m



SECTION A-A'
HORIZ. SCALE
0 5 10 m
VERT. SCALE
0 5 10 m

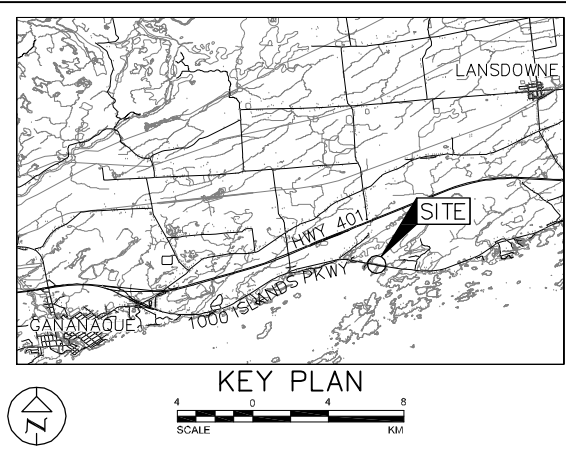
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. 2016-4019
WP No. 4091-10-02 HWY
4125-10-01 TRAIL

LANDONS BAY BRIDGES
BOREHOLE LOCATIONS
AND SOIL STRATA

SHEET
101

Golder Associates Ltd.
OTTAWA ONTARIO, CANADA



LEGEND

- Borehole - Current Investigation
- Testpit - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Total Core Recovery (REC)
- WL upon completion of drilling
- WL in piezometer, measured on December 3, 2015
- Seal
- Piezometer

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
15-1	78.8	4912481.9	339225.5
15-2	78.7	4912492.8	339182.8
15-3	79.1	4912473.4	339224.9
14-1	78.8	4912491.2	339184.6
14-2	78.9	4912482.7	339221.2
14-3	79.1	4912475.2	339180.6
14-4	79.2	4912465.5	339219.3

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 Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete 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 plan provided in digital format by Dillon, drawing file no. TIP Contract A - Base, received August 7, 2014.

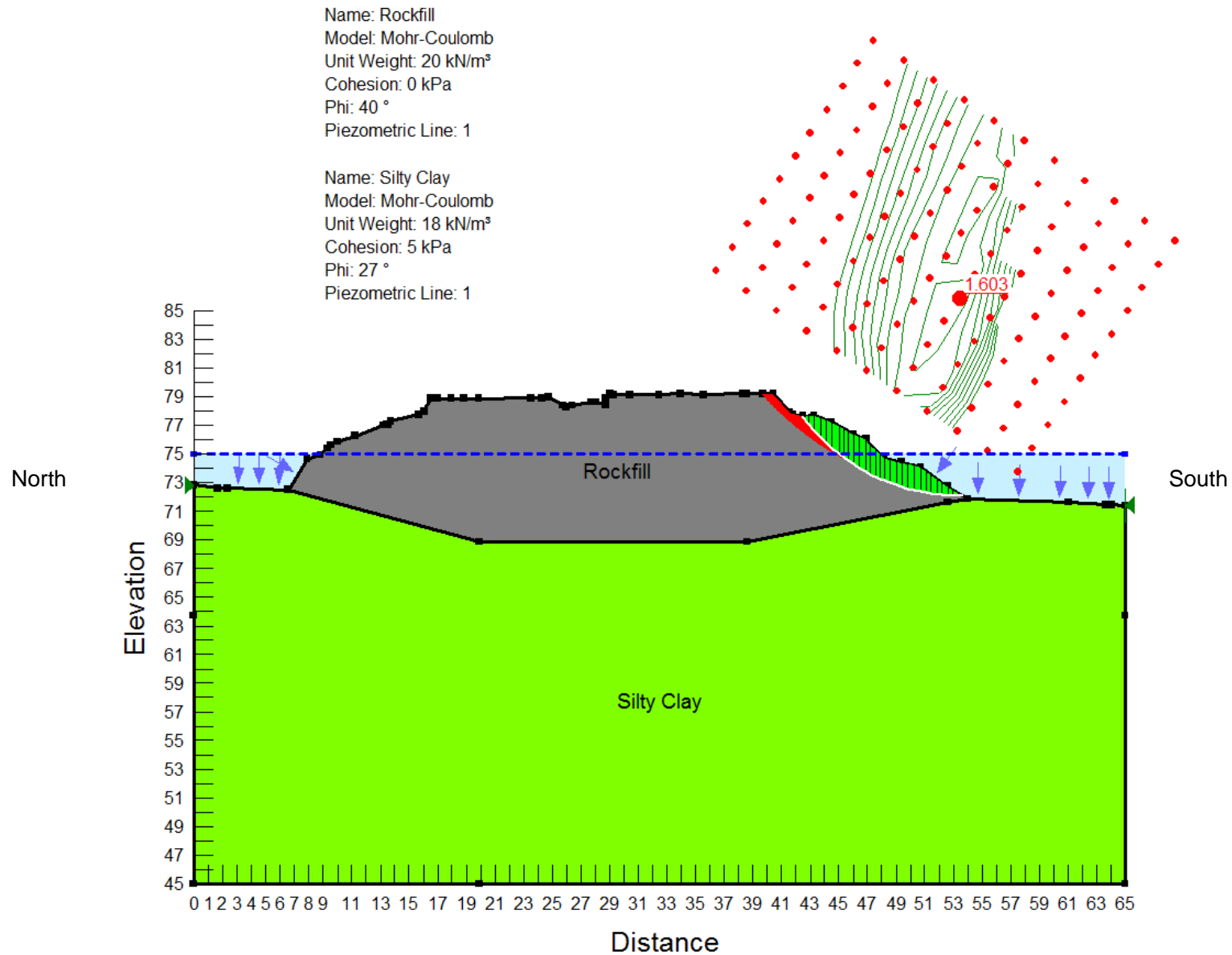


NO.	DATE	BY	REVISION
Geocres No. No.31C-227			
HWY. 1000 Islands Pkwy	PROJECT NO. 12-1121-0193	DIST.	
SUBM'D. KSL	CHKD. KSL	DATE: 2/29/2016	SITE:
DRAWN: JM	CHKD. FJH	APPD. FJH	DWG. 2



Static Stability Analysis – Existing Embankment Station 17+635

Figure 1





APPENDIX A

Borehole, Drillhole, and Test Pit 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		0 to 4		
FS	Foil sample		4 to 10		
RC	Rock core		10 to 30		
SC	Soil core		30 to 50		
SS	Split spoon sampler		over 50		
ST	Slotted tube				
TO	Thin-walled, open				
TP	Thin-walled, piston	(b) Cohesive Soils			
WS	Wash sample	C _u or S _u			
DT	Dual tube sample	Consistency			
DD	Diamond drilling				
II. PENETRATION RESISTANCE			kPa	Psf	
			0 to 12	0 to 250	
			12 to 25	250 to 500	
			25 to 50	500 to 1,000	
			50 to 100	1,000 to 2,000	
			100 to 200	2,000 to 4,000	
Standard Penetration Resistance (SPT), N:		Hard	Over 200	Over 4,000	
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.).		IV. SOIL TESTS			
Dynamic Cone Penetration Resistance (DCPT); N _d :		w	Water content		
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.).		w _p or PL	Plastic limited		
PH: Sampler advanced by hydraulic pressure		w _l or LL	Liquid limit		
PM: Sampler advanced by manual pressure		C	Consolidaiton (oedometer) test		
WH: Sampler advanced by static weight of hammer		CHEM	Chemical analysis (refer to text)		
WR: Sampler advanced by weight of sampler and rod		CID	Consolidated isotropically drained triaxial test ¹		
Cone Penetration Test (CPT):		CIU	Consolidated isotropically undrained triaxial test with porewater pressure measurement ¹		
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.		D _R	Relative density		
		DS	Direct shear test		
		G _s	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 $= \sigma'_p / \sigma'_{vo}$

(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

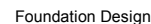
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

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
II -	Parallel To	STR -	Stress Induced
OR -	Orthogonal		



GTA-MTO 001 N:ACTIVE2012\121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IMAGING\PHASE 12\10\12\12\10\193-1210.GPJ GAL-GTA.GDT 01/11/16 JM

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



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



SHEET 3 OF 5

METRIC

ORIGINATED BY HEC

COMPILED BY JJL

CHECKED BY MJK

Continued Next Page




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

GTA-MTO 001 N:ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IMAGING\PHASE 1210\1211210193-1210.GPJ GAL-GTA.GDT 01/11/16 JM



GTA-MTO 001 N:\ACTIVE\2012\121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IMG\INT\PHASE 1210\1211210193-1210.GPJ GAL-GTA.GDT 01/11/16 JM

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

PROJECT 12-1121-0193-1210		RECORD OF BOREHOLE No 14-1				SHEET 5 OF 5		METRIC								
G.W.P. 4091-10-01 & 4125-10-01		LOCATION N 4912491.2 ; E 339184.6				ORIGINATED BY HEC										
DIST _____ HWY 1000 Islands Pkwy		BOREHOLE TYPE Hammer Drill (127 mm ID Well Casing), Rotary Drill NQ Core, Wash Boring				COMPILED BY JJL										
DATUM Geodetic		DATE May 12-28, 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100				
37.6	Layered SILTY CLAY, trace sand, and Sandy SILT Very stiff/compact Grey Wet															
41.2	SAND, some silt Dense Grey Wet															
	- Silty clay layer encountered in sample 24		24	SS	39											0 90 7 3
34.6																
44.2	SAND, some silt and gravel Very dense Grey Wet															
			25	SS	56											13 77 8 2
33.2	END OF BOREHOLE LIMIT OF AVAILABLE CASING															
45.6	<p>NOTES:</p> <p>Attempt 1. Hammer drill/casing advanced until casing sheared at 5.1 m. Borehole abandoned (May 12, 2014).</p> <p>Attempt 2. Hammer drill/casing advanced until casing shoe damaged at 1.5 m depth.</p> <p>Attempt 3. Hammer drill advanced to casing refusal at 5.9 m, advanced alternate hammer drill to 6.7 m without casing, attempted SPTs (May 13, 2014).</p> <p>Resumed drilling with HW casing to depth of 8.6 m (casing shoe worn out) (May 20-28, 2014). Advanced NW casing to 15 m; coring between approximately 12 m and 15 m depth. Advanced borehole to termination depth of 45.6 m by advancing BWV casing.</p> <p>A. Water level in BW Casing at 6.9 m depth (Elev. 71.9 m) on May 27, 2014. Water level in open borehole at a depth of 3.8 m below ground surface (Elev. 75.0 m) upon completion of drilling.</p>															

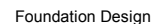
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PROJECT <u>12-1121-0193-1210</u>				RECORD OF BOREHOLE No 14-2				SHEET 1 OF 1				METRIC						
G.W.P. <u>4091-10-01 & 4125-10-01</u>				LOCATION <u>N 4912482.7 ;E 339221.2</u>				ORIGINATED BY <u>HEC</u>										
DIST <u></u> HWY <u>1000 Islands Pkwy</u>				BOREHOLE TYPE <u>Hammer Drill</u>				COMPILED BY <u>JJL</u>										
DATUM <u>Geodetic</u>				DATE <u>May 13, 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			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L			
78.9 0.0	GROUND SURFACE Borehole advanced using down-the-hole hammer system. No sampling carried out. Cobbles and boulders in matrix of sandy gravel (granitic gneiss ROCK FILL) Pink to black							20	40	60	80	100						
							78											
							77											
							76											
							75											
							74											
							73											
72.5 6.4	END OF BOREHOLE NOTE: Attempt 1. Hammer drill advanced to 6.4 m depth, borehole caved on drill string withdrawal. Borehole abandoned (May 13, 2014).																	

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IM\GINT\PHASE 12\10\12\11210193-1210.GPJ GAL-GTA.GDT 01/11/16 JM

PROJECT <u>12-1121-0193-1210</u>		RECORD OF BOREHOLE No 14-3		SHEET 1 OF 1		METRIC											
G.W.P. <u>4091-10-01 & 4125-10-01</u>		LOCATION <u>N 4912475.2 ; E 339180.6</u>		ORIGINATED BY <u>HEC</u>													
DIST <u></u> HWY <u>1000 Islands Pkwy</u>		BOREHOLE TYPE <u>Hammer Drill (127 mm ID Well Casing), Rotary Drill HW Casing, Wash Boring</u>		COMPILED BY <u>JJL</u>													
DATUM <u>Geodetic</u>		DATE <u>May 14-15, 2014</u>		CHECKED BY <u>MJK</u>													
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/m ³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	25 50 75					
79.1 0.0	GROUND SURFACE Borehole advanced using down-the-hole hammer system. No sampling carried out. Cobbles and boulders in matrix of sandy gravel (granitic gneiss ROCK FILL) Pink to black						79										
							78										
							77										
							76										
							75										
							74										
							73										
72.1 7.0	END OF BOREHOLE NOTES: Attempt 1. Advanced borehole using down-the-hole hammer (DTHH) until casing sheared. Attempt 2. Advance HW casing using rotary / washbore drill methods (casing advancer) to 4.9 m depth. DTHH casing encountered in HQ core barrel. Advance HW casing to 7.0 m, core barrel sheared/lost down hole. Borehole abandoned (May 14, 2014). Attempt 3. Hammer drill advanced from surface to 4.3 m depth, abandoned borehole due to caving of borehole (May 15, 2014).																

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IM\GINT\PHASE 12\10\12\10\193-1210.GPJ GAL-GTA.GDT 01/11/16 JM



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

GTA-MTO 001 N:ACTIVE2012\121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IMG\INT\PHASE_12\10\12\12\10\193-12\10.GPJ GAL-GTA.GDT 01/11/16 JM

PROJECT 12-1121-0193-1210		RECORD OF BOREHOLE No 14-4				SHEET 2 OF 2		METRIC																	
G.W.P. 4091-10-01 & 4125-10-01		LOCATION N 4912465.5 ;E 339219.3				ORIGINATED BY HEC																			
DIST _____ HWY 1000 Islands Pkwy		BOREHOLE TYPE Hammer Drill HW Casing, Rotary Drill HW/NW Casing				COMPILED BY JJL																			
DATUM Geodetic		DATE May 15, 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)									
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 </div> <div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div>																		
	<p>END OF BOREHOLE</p> <p>NOTES:</p> <p>Attempt 1. Down-the-hole hammer drill and casing advanced until casing sheared at 2.8 m depth. Advanced HW casing to 5.9 m depth. HW shoe and rods sheared off. Abandoned borehole (May 15, 2014).</p> <p>Attempt 2. NW casing advanced using rotary drill methods (casing advanced) from ground surface to 9.7 m depth. Increased rate of advancement from 7.0 m to 9.7 m depth. NW casing and casing advancer sheared off down-hole at 9.7 m. Borehole abandoned (May 15, 2014).</p> <p>A. Water level at a depth of 4.0 m below ground surface (Elev. 75.1 m) upon completion of drilling.</p>																								

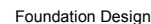
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PROJECT 12-1121-0193-1215		RECORD OF BOREHOLE No 15-3		SHEET 1 OF 5		METRIC											
G.W.P. _____		LOCATION N 4912473.4 ; E 339224.9		ORIGINATED BY HEC													
DIST _____ HWY Landon's Bay		BOREHOLE TYPE Wash Boring, HW/NW Casing, Rotary Drill, NQ Core		COMPILED BY JEM													
DATUM Geodetic		DATE November 10-17, 2015		CHECKED BY KSL													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ kN/m³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	Wp	W	WL	25 50 75					
79.1 0.0	GROUND SURFACE																
0.1	Silty sand, trace organics (TOPSOIL) Dark brown Moist		1	SS	14		79										
	Gravelly sand, some silt, trace clay, contains clay pockets (FILL) Compact Brown Moist		2	SS	11		78									27 51 17 5	
77.6 1.5	Cobbles and boulders in matrix of silty sand and gravel (ROCK FILL) Moist to wet		3	RC	DD		77										
			4	SS	5		76										
			5	RC	DD		75										
			6	SS	17		74										
			7	RC	DD		73										
			8	RC	DD		72										
			9	RC	DD		71										
			10	SS	22		70										
			11	RC	DD												
			12	RC	DD												
			13	SS	28												

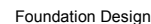
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+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IMG\INTPHASE 1215\1210193-1215.GPJ GAL-GTA.GDT 02/25/16 JEM



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



SHEET 3 OF 5

METRIC

PROJECT 12-1121-0193-1215

G.W.P.

LOCATION

N 4912473.4 :E 339224.9

ORIGINATED BY HEC

DIST

HWY Landon's Bay

BOREHOLE TYPE

Wash Boring, HW/NW Casing, Rotary Drill, NQ Core

COMPILED BY JEM

DATUM Geodetic

DATE _____

November 10-17, 2015

CHECKED BY KSL

[illegible]

Continued Next Page

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

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

INCLINATION: -90° AZIMUTH: --

DRILL RIG: CME 55
DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

DATA-RCK 031 N:ACTIVE20121121 - GEOTECHNICAL12-121-0193 DILLON MEGA 3 EASTERN REGION/SPATIAL IMGINT/PHASE 12151211210193-1215.GPJ GAL-MISS.GDT 02/25/16 JEM

1 : 50



CHECKED: FJH

Golder Associates Ltd.

1931 Robertson Road
Ottawa, Ontario K2H 5B7
Tel: (613) 592-9600
Fax: (613) 592-9601

**TABLE 1 - TEST PIT RECORD****TEST PIT #15-1 (East)**

DATE: 20-Nov-15

PROJECT: Dillon/ Landon's Bay Bridges/Mega 3 Eastern Region
PROJECT No.: 12-1121-0193-1215

EQUIPMENT: CAT 314C

NOTES:

Depth (m)	Elevation (m)	Description	Remarks
0.00	78.84	Crushed Stone (FILL)	
0.08	78.76	Sand and Gravel (FILL)	
		Brown Moist	
0.54	-0.54	Portland Cement Concrete	
1.18	77.58	Silty Sand, some Gravel, with Cobbles (FILL)	
		Brown Moist	
2.30	76.54	Cobbles and Boulders, some Sand and Gravel (FILL)	
		Brown Moist	
3.90	74.94	End of Test Pit	



Notes

- Slide slopes caving at 3.9 m
- Unable to excavate past 3.9 m
- Bottom of test pit wet at 3.9 metres

Golder Associates Ltd.

1931 Robertson Road
Ottawa, Ontario K2H 5B7
Tel: (613) 592-9600
Fax: (613) 592-9601

**TABLE 2 - TEST PIT RECORD****TEST PIT #15-2 (west)**

DATE: 20-Nov-15

PROJECT: Dillon/ Landon's Bay Bridges/Mega 3 Eastern Region
PROJECT No.: 12-1121-0193-1215

EQUIPMENT: CAT 314C

NOTES:

Depth (m)	Elevation (m)	Description	Remarks
0.00	78.72	Crushed Stone and Silty Sand (FILL) Brown Moist	
0.25	78.47	Portland Cement Concrete	
1.00	77.72	Silty Sand, some Gravel, with Cobbles (FILL) Brown Moist	
2.30	76.42	Cobbles and Boulders, some Sand and Gravel (FILL) Brown Moist	
4.00	74.72	End of Test Pit	



Notes -- Test pit walls unstable at 3.30 m.
-- No groundwater infiltration.

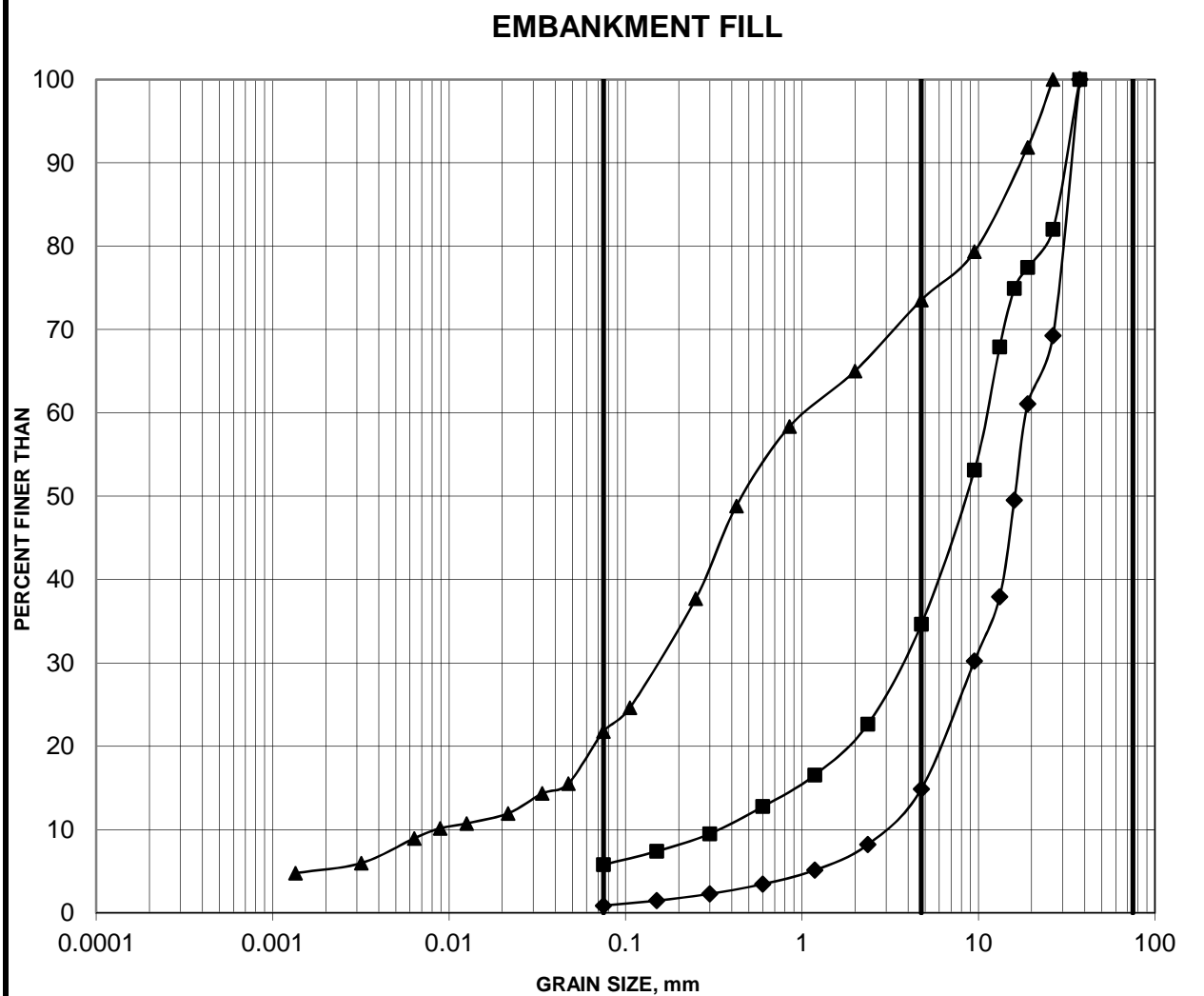


APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

FIGURE B-1

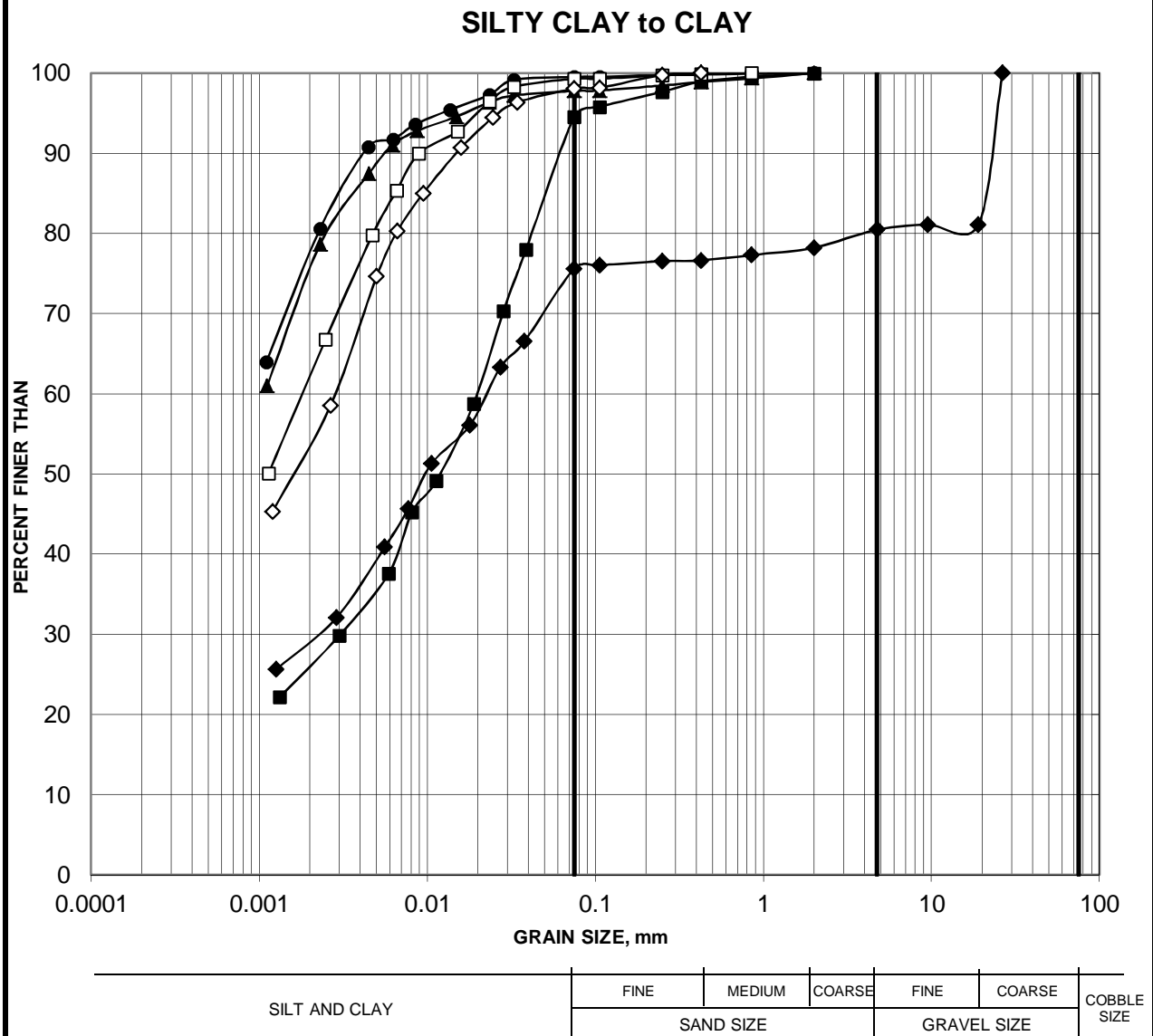


SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

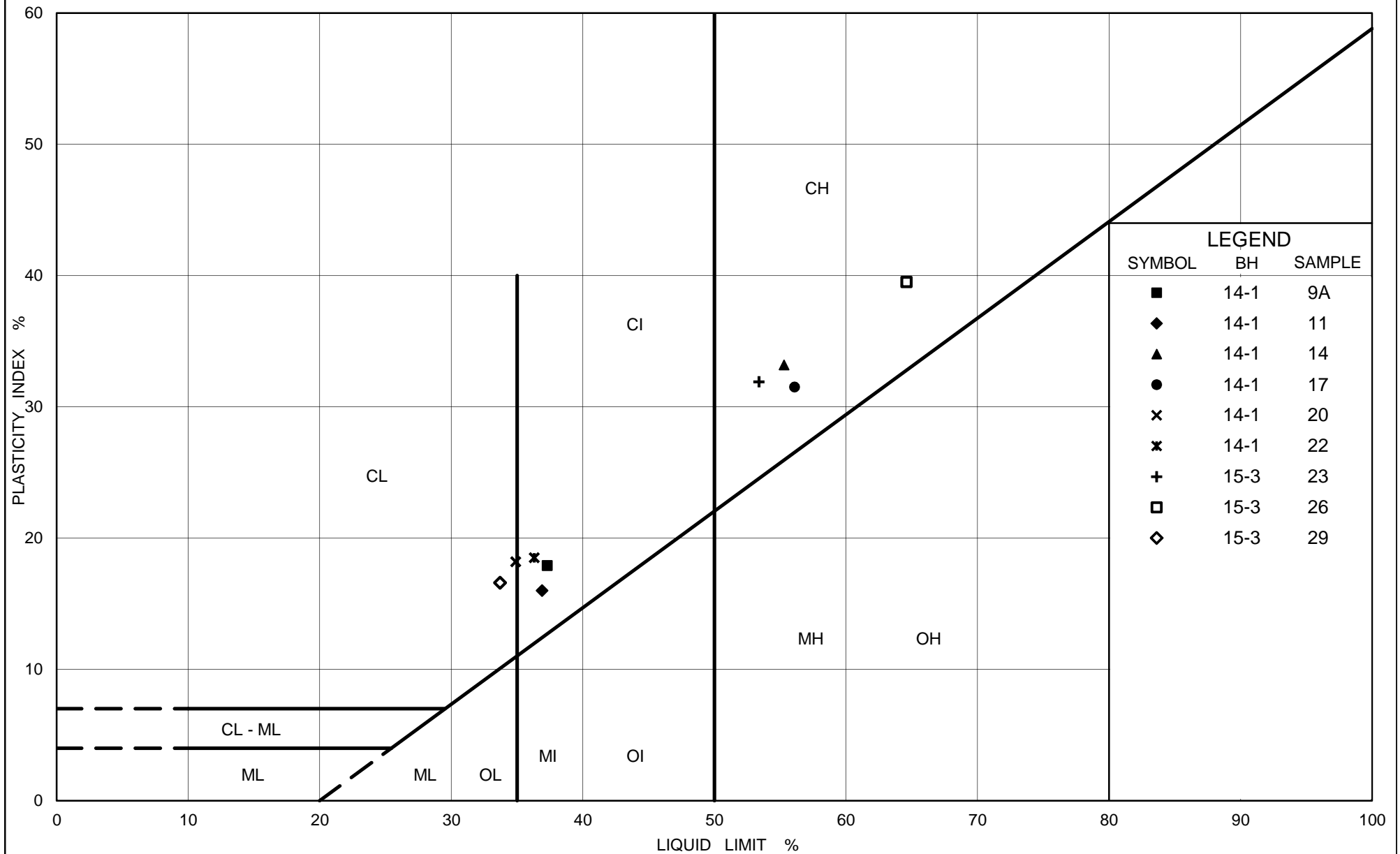
Borehole	Sample	Depth (m)
■ 14-1	4	10.42-11.02
◆ 14-4	1	7.32-7.92
▲ 15-3	2	0.61-1.22

GRAIN SIZE DISTRIBUTION

FIGURE B-2



Borehole	Sample	Depth (m)
—■— 14-1	9A	15.09-15.29
—◆— 14-1	11	16.76-17.37
—▲— 14-1	14	20.57-21.18
—●— 14-1	17	25.15-25.76
—□— 14-1	20	29.72-30.33
—◇— 14-1	22	35.81-35.97



Ministry of Transportation

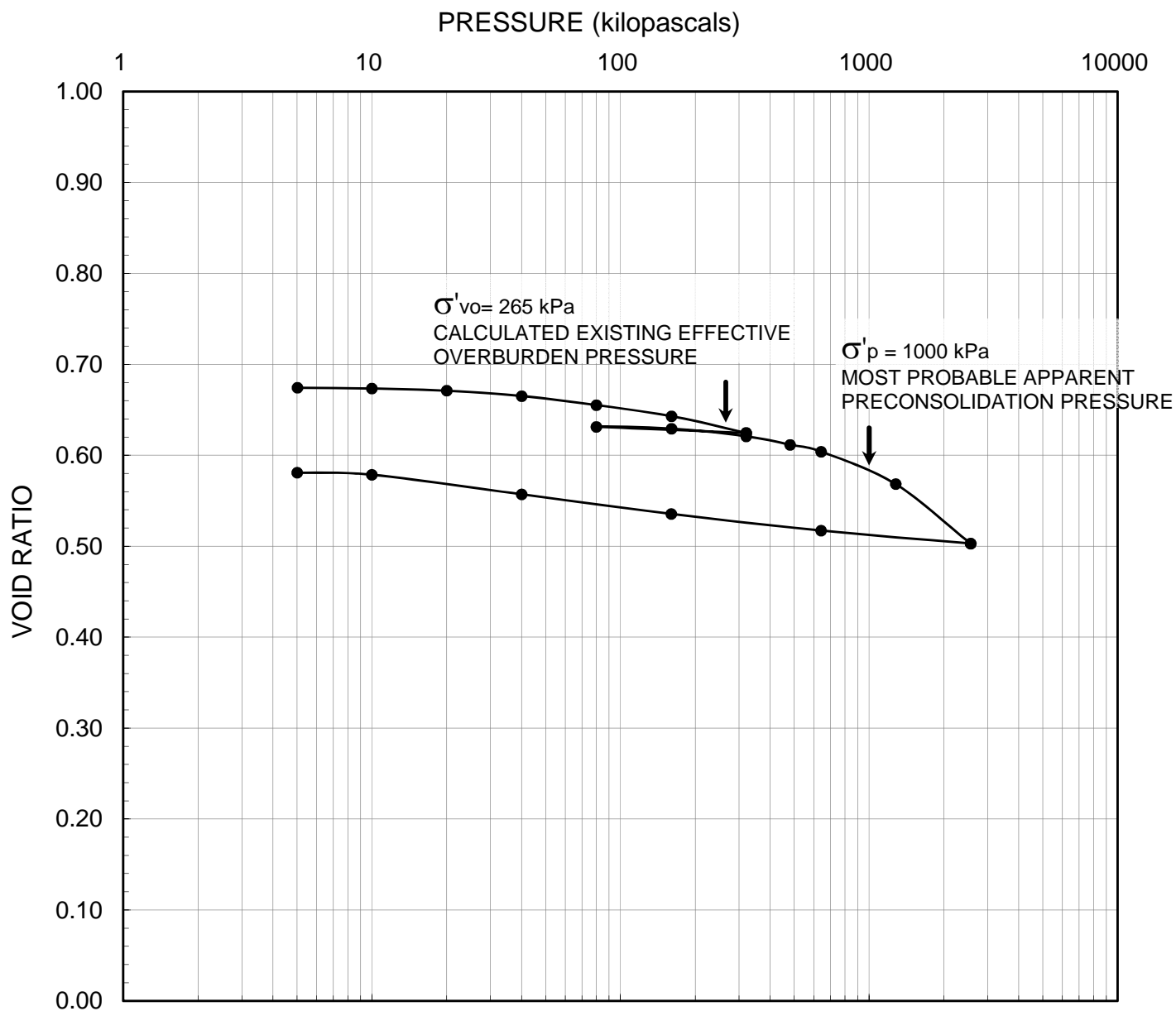
Ontario

PLASTICITY CHART SILTY CLAY to CLAY

FIG No. B-3

Project No. 12-1121-0193/1215

Compiled By : MI Checked By : CNM



LEGEND

Borehole: 15-3	$w_i = 23\%$	$S_o = 95\%$	$\gamma = 20 \text{ kN/m}^3$
Sample: 23	$w_f = 21\%$	$e_o = 0.67$	$G_s = 2.78$
Depth (m): 19.1	$w_l = 53\%$	$C_c = 0.22$	
Elevation (m): 60.0	$w_p = 22\%$	$C_r = 0.0066$	



SCALE	AS SHOWN
DATE	01/12/16
CADD	N/A
ENTERED	MI

TITLE

CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary

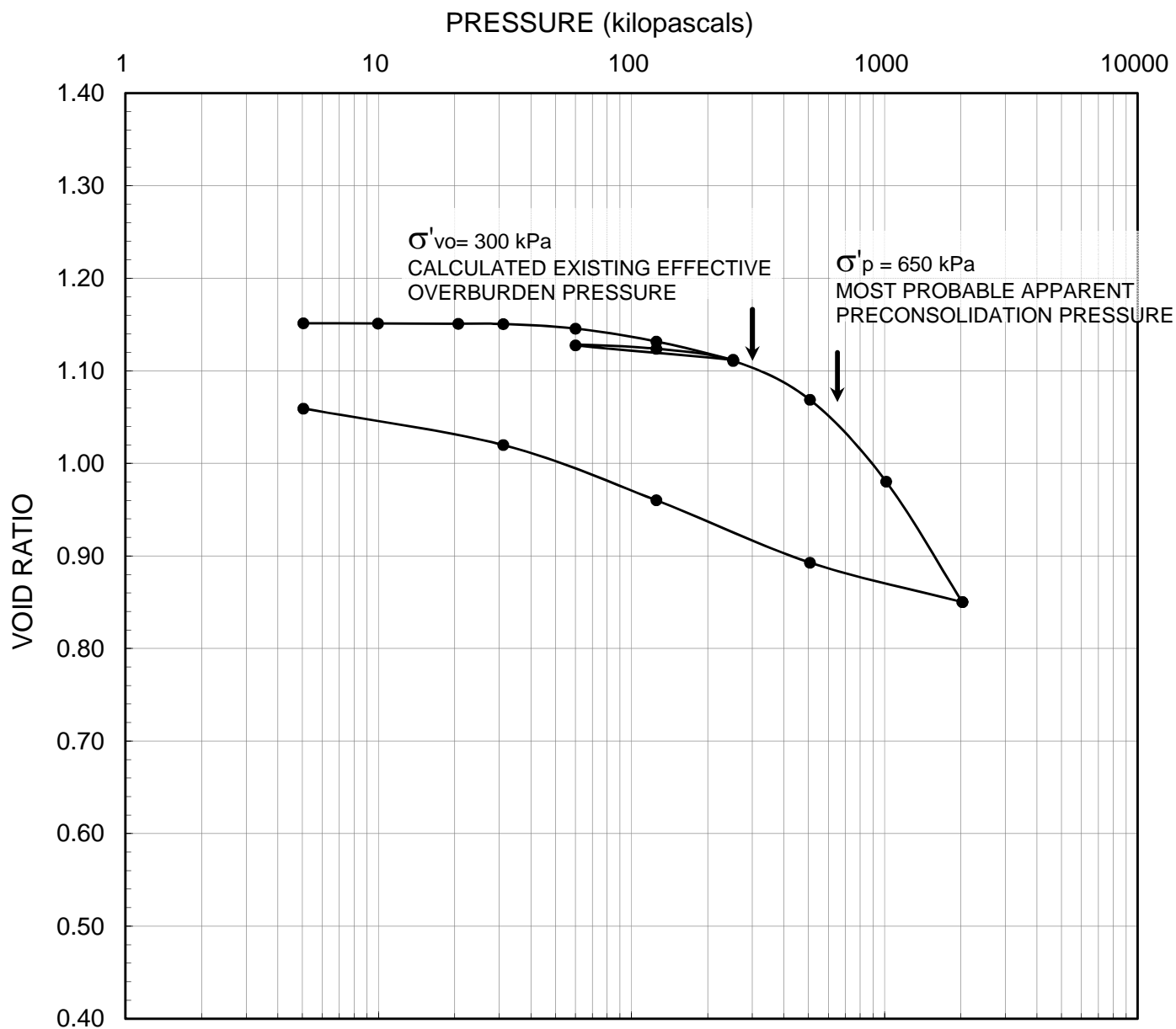
CHECK CNM

PROJECT No. 12-1121-0193 /1215 REV. 1

REVIEW KSL

FIGURE

B-4



LEGEND

Borehole: 15-3	$w_i = 41\%$	$S_o = 100\%$	$\gamma = 17.9 \text{ kN/m}^3$
Sample: 26	$w_f = 39\%$	$e_o = 1.15$	$G_s = 2.78$
Depth (m): 23.1	$w_l = 65\%$	$C_c = 0.43$	
Elevation (m): 56.0	$w_p = 25\%$	$C_r = 0.012$	



SCALE	AS SHOWN
DATE	01/12/16
CADD	N/A
ENTERED	MI

CONSOLIDATION TEST RESULTS

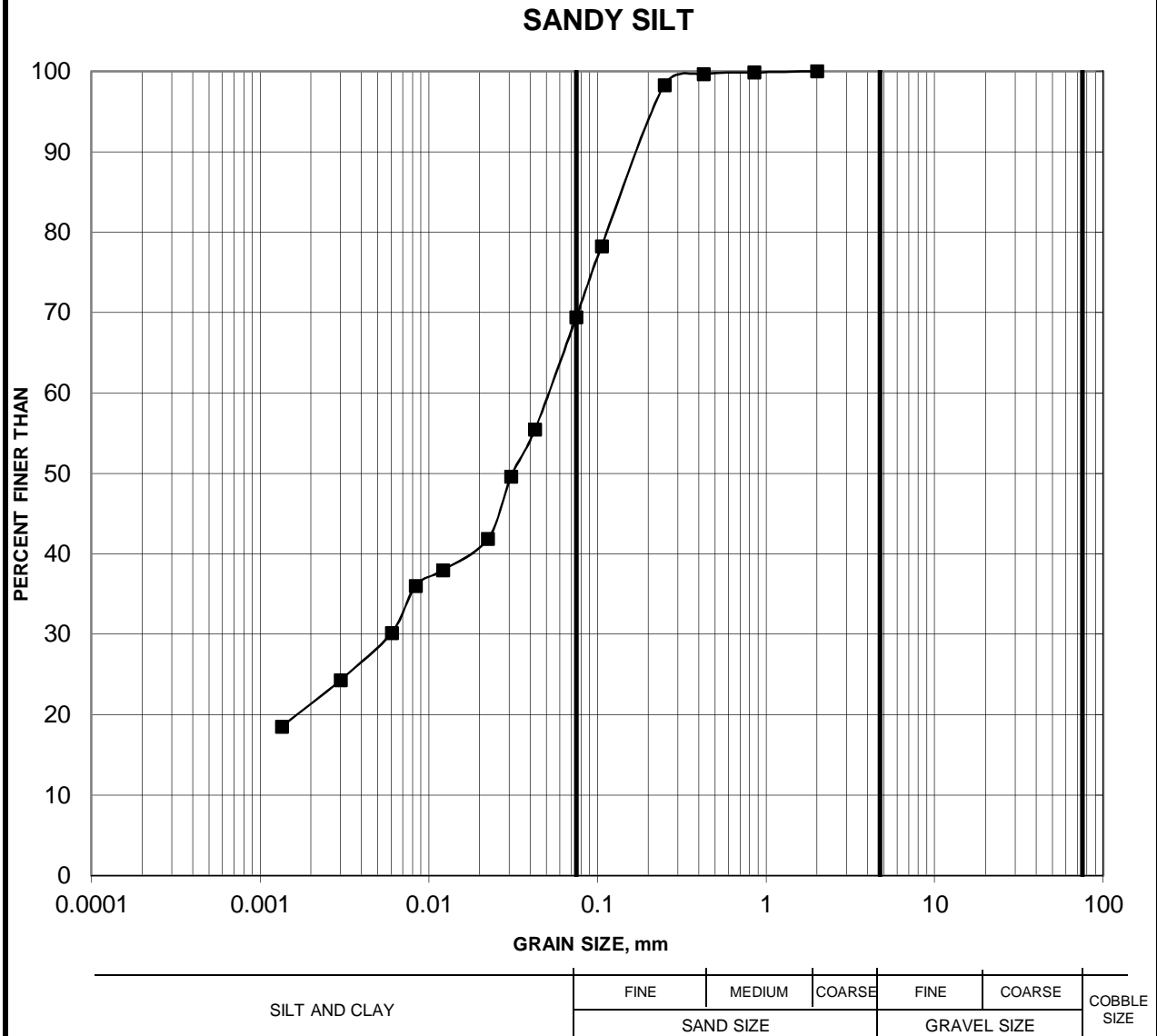
FILE No.	Consolidation summary
PROJECT No.	12-1121-0193-1215
REV.	1

CHECK	CNM
REVIEW	KSL

FIGURE **B-5**

GRAIN SIZE DISTRIBUTION

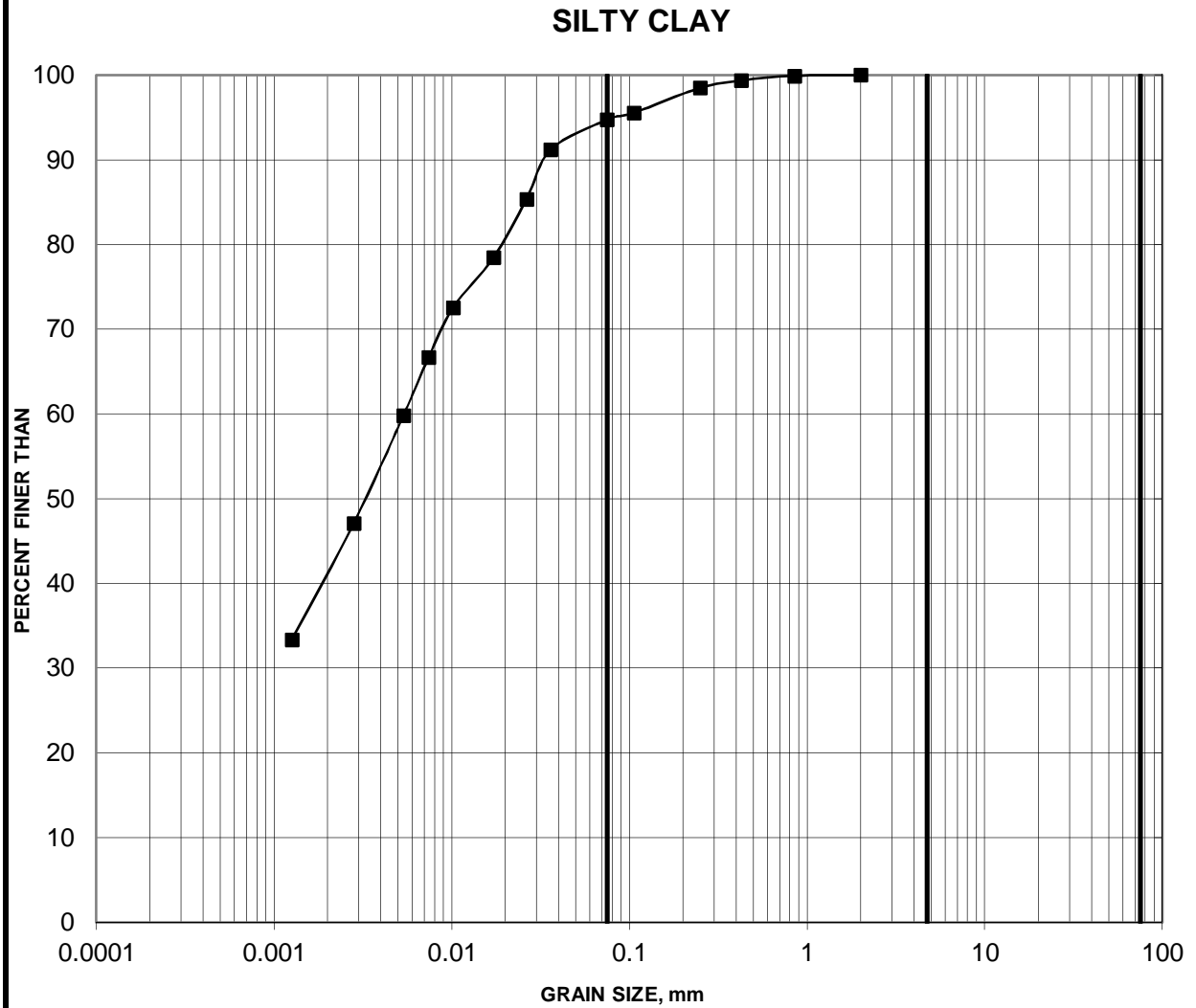
FIGURE B-6



Borehole	Sample	Depth (m)
14-1	22A	35.97-36.42

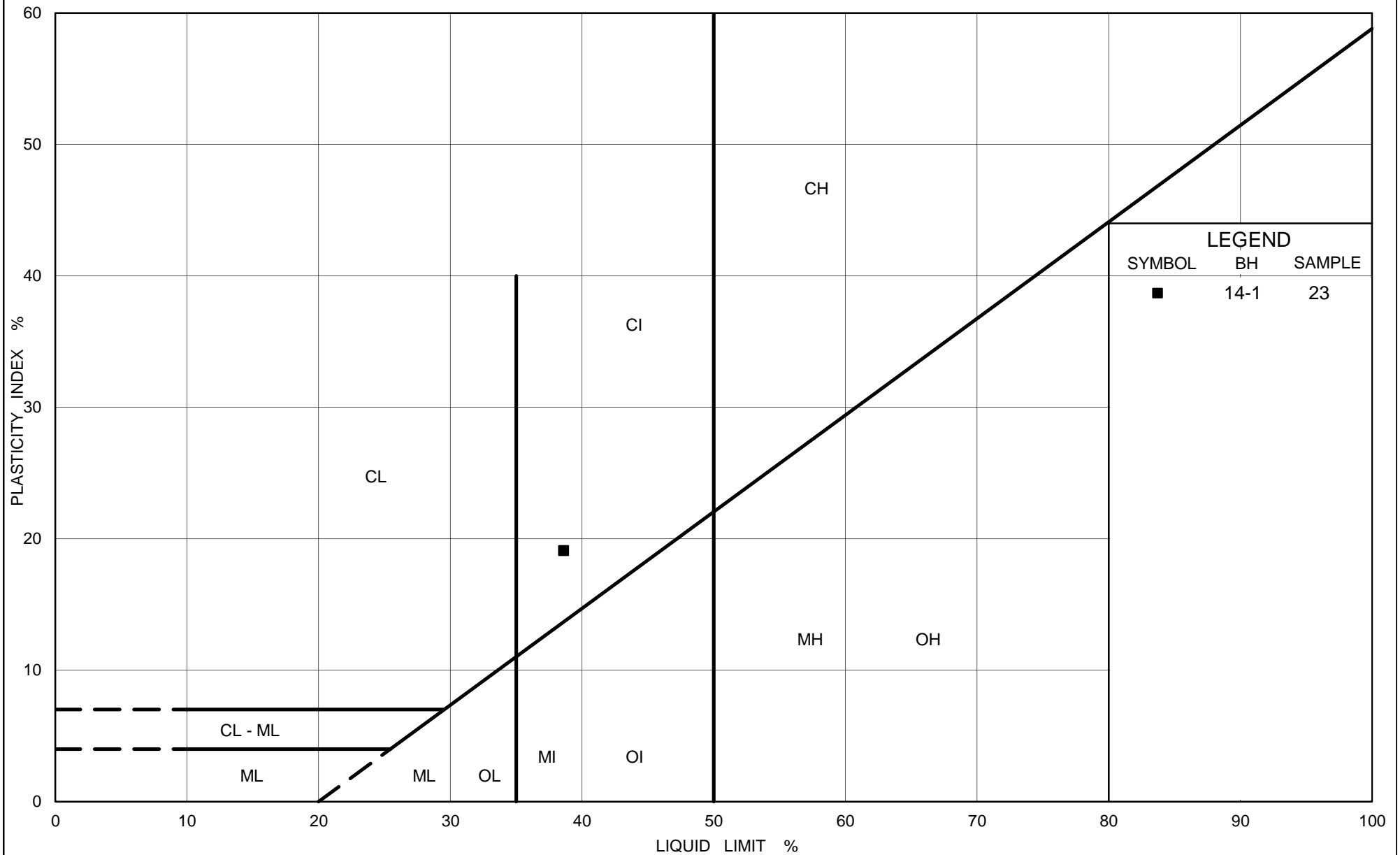
GRAIN SIZE DISTRIBUTION

FIGURE B-7



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

Borehole	Sample	Depth (m)
—■ 14-1	23	38.86-39.47



Ministry of Transportation

Ontario

PLASTICITY CHART SILTY CLAY

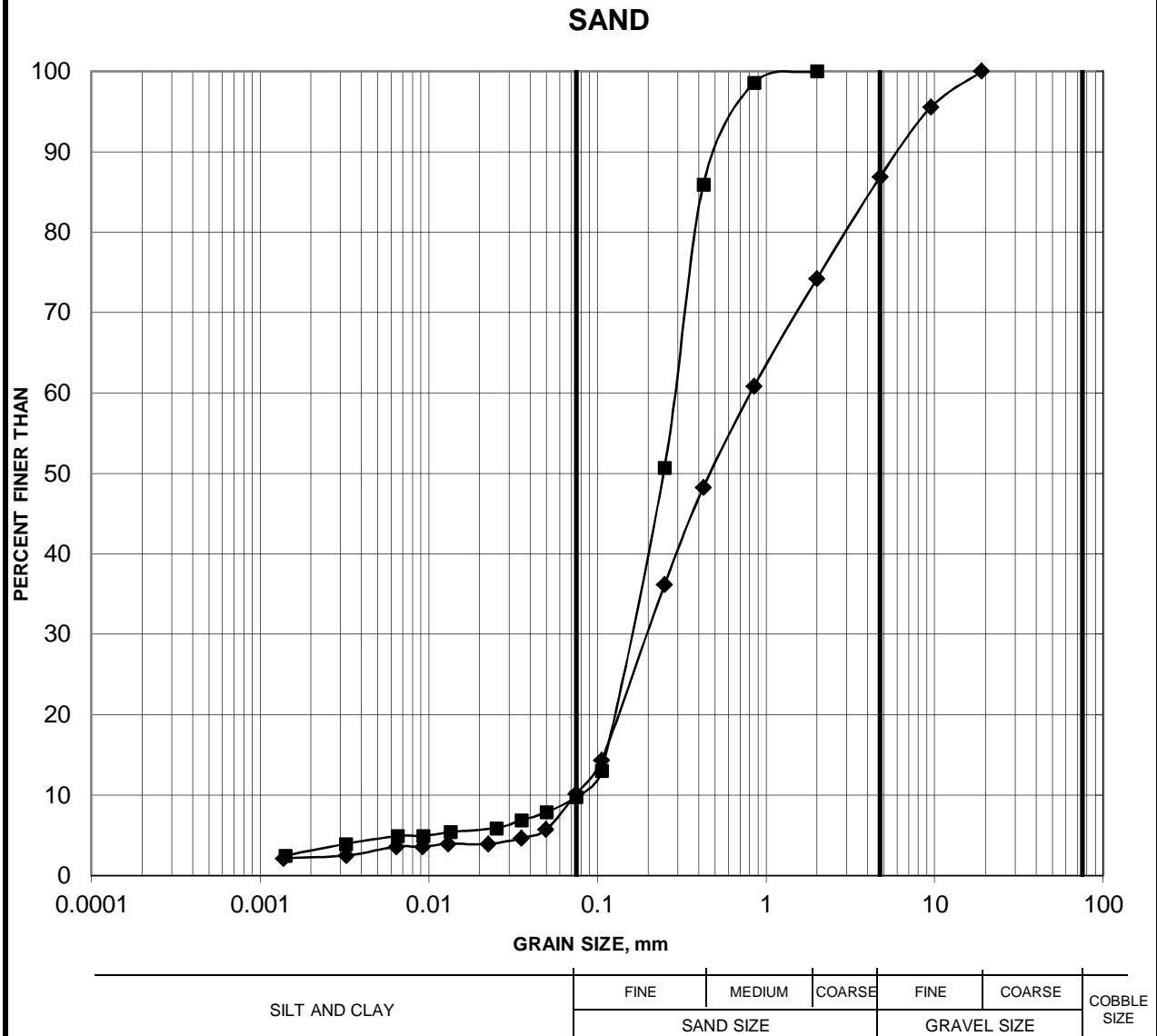
FIG No. B-8

Project No. 12-1121-0193/1215

Compiled By : MI Checked By : CNM

GRAIN SIZE DISTRIBUTION

FIGURE B-9



Borehole	Sample	Depth (m)
—■ 14-1	24	41.91-42.52
—◆ 14-1	25	44.96-45.57

[illegible]

Golder Associates Ltd.



APPENDIX C

Non-Standard Special Provisions and Guidelines

WORKING SLAB – Item No.

Special Provision

1.0 SCOPE

This Special Provision covers the requirements for the supply and placement of a concrete working slab on top of approved subgrade under structure foundations.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction

OPSS 902 Excavating and Backfilling – Structures

3.0 DEFINITIONS – Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS – Not Used

5.0 MATERIALS

Concrete for working slabs shall have a minimum 28-day strength of 20 MPa. The concrete curing requirements of OPSS.PROV 904 shall not apply.

6.0 EQUIPMENT – Not Used

7.0 CONSTRUCTION

7.01 Excavation

Excavation for the working slab shall be according to OPSS 902.

7.02 Protection of Founding Soil

Within four hours following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents.

7.03 Dewatering

Dewatering shall be carried out in accordance with OPSS 902.

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT – Not Used

10.0 BASIS OF PAYMENT

10.01 Working Slab – Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

END OF SECTION

BACKFILL TO STRUCTURES **ADJACENT TO ROCK EMBANKMENT APPROACHES**

November 2002

SUBJECT: BACKFILL TO STRUCTURES

PURPOSE: To provide direction on the type of backfill to structures to be used adjacent to rock embankment approaches throughout the province.

BACKGROUND: The backfill placed in the transitional zone between the approach embankment and the structure (bridges and culverts) must be:

- (1) free draining to avoid buildup of hydrostatic pressures;
- (2) easily compacted, as the material must be compacted using hand operated equipment due to the potential for damage to the structure by heavier equipment; and
- (3) compatible with the adjacent approach embankment material to avoid intermixing
- (4) Non frost susceptible

Rock embankments contain numerous voids into which finer material can migrate under the action of water and/or particle reorientation under sustained loading. The loss of fines can result in unacceptable settlement at the approaches. Backfill material in contact with the rock embankment must be properly graded or a proper filter material used. Geotextile as a separator material for this application is not recommended because the geotextile is susceptible to damage during installation. Introducing a separate soil or rock material with the necessary drainage/filter requirements has practical limitations.

OPSD 3505 indicates the use of maximum 300 mm size rock backfill. While satisfying the requirements of 1, 3 and 4 above, compaction of this material using hand operated equipment is not effective.

Granular B Type II satisfies all four(4) requirements above, and is considered a technically viable, practical and cost-effective alternative material to the 300 mm rock backfill for use as structure backfill adjacent to rock embankment.

Policy:

When the approach embankment is rock, granular material meeting the specification requirements for Granular B Type II(OPSS 1010) is recommended as backfill to structures as an alternative to using rock backfill. For this application, a filter medium between the Granular B Type II and the rock embankment is not required.

On Contracts, the application of Granular B Type II backfill to structures may result in tender items for both Granular B Type I and a small quantity of Granular B Type II. In this instance, the Contract Drawings (by special note) and Quantity Sheets shall clearly define the areas where Granular B Type II is required

Alternatively, as described in CDED 206-2.2.4, when granular material is not readily available or due to high granular costs, rock backfill of maximum 300 mm size may be used as structure backfill adjacent to rock embankment as illustrated in OPSD 3505.

For integral or semi integral abutment piles, rock fill shall not be placed within the active wedge zone.



EXPANDED POLYSTYRENE EMBANKMENT – Item No.

Special Provision

1.0 SCOPE

This special provision covers the requirements for the supply and construction of the rigid expanded polystyrene embankment fill and associated works.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications.

National Standards of Canada

CAN/CGSB - 51.20 M87

ASTM International

ASTM D1621	Test Method for Compressive Properties of Rigid Cellular Plastics
ASTM C203	Test Method for Breaking Load and Flexural Properties of Block Type Thermal Insulation
ASTM C177	Test Method for Steady State Heat Flux Measurements and Thermal Transmission Properties by Means of the Heat Flow Apparatus
ASTM D2842	Test Method for Water Absorption by Rigid Cellular Plastics
ASTM D2863	Test Method for Measuring the Minimum Oxygen Content
ASTM D2126	Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

Ontario Provincial Standards Specification, Construction

OPSS 212	Borrow
OPSS 501	Compacting
OPSS 517	Dewatering Of Pipeline, Utility, and Associated Structure Excavation
OPSS 902	Excavation and Backfilling - Structures

Ontario Provincial Standards Specification, Materials

OPSS 1010	Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
OPSS 1605	Extruded Expanded Polystyrene Pavement Insulation

3.0 DEFINITIONS

For the purpose of this special provision, the following definitions apply:

Rigid Expanded Polystyrene: Moulded rigid blocks produced by a process of pre-expansion, aging and forming of petroleum based raw material.

Rigid Extruded Expanded Polystyrene: Rigid boards made by extrusion of expanded polystyrene beads.

Production Lot: The quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

Quality Verification Engineer: Quality Verification Engineer means an Engineer with a minimum of five (5) years experience related to the design and/or construction of expanded polystyrene systems of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the Contract Documents and issue of certificate(s) of conformance.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Submission Requirements

4.01.01 General

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the shop drawings and method statement signed and sealed by the Quality Verification Engineer that provides full details of materials and construction procedure.

4.01.02 Delivery, Storage, Handling, and Protection

The Contractor shall submit the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the rigid expanded polystyrene manufacturer's requirement.

4.01.03 Construction

The Contractor shall submit full details of the following.

- a) The method of foundation excavation and preparation.
- b) Construction of levelling pad.
- c) The method of placement of expanded polystyrene blocks including temporary ballasting and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer-by-layer basis.
- d) The method and limits of placement of polyethylene sheeting.

- e) The method of placement of 125 mm reinforced concrete pad.
- f) The method of placement of sub-base material.
- g) The method of placement of cover and side backfill.

4.01.04 Quality Verification Engineer

The Contractor shall submit the following.

- a) Details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted at least three weeks prior to the installation of the rigid expanded polystyrene embankments the Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.
- b) To the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation. Upon completion of the Expanded Polystyrene Embankment the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the Expanded Polystyrene Embankment has been constructed in conformance with the installation procedures and specifications of the Contract Documents.

5.0 MATERIALS

5.01 Granular Levelling Pad

The levelling pad shall consist of a Granular "A" material with gradation and physical requirements as specified in OPSS 1010.

5.02 Rigid Expanded Polystyrene

5.02.01 Submission Requirements

The Contractor shall submit the following:

1. A general statement as to the type, composition, and method of production of the material.
2. The manufacturer's name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the rigid expanded polystyrene.
3. Certification of compliance of physical and mechanical properties.

4. An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the rigid expanded polystyrene.
5. The physical and mechanical properties of the rigid expanded polystyrene including:
 - a) Geometry
 - b) Nominal Density
 - c) Compressive Strength
 - d) Flexural Strength
 - e) Thermal Resistance
 - f) Dimensional Stability
 - g) Flammability
 - h) Water Absorption
6. Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
7. A sample of the expanded polystyrene material to the Quality Verification Engineer for review.
8. To the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the Contract Documents.

5.02.02 Production Lots

Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The polystyrene shall be free from defects affecting serviceability.

5.02.03 Detail Requirements

5.02.03.01 General

Material property requirements shall be as shown in Table 1 and as described below.

Table 1:

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear - Flatness - Squareness - Thickness	Mm	1200 x 600 x 300 with tolerances $\pm 1\%$ 10 mm in 3 m $\pm 0.5\%$ -3, +5	
Compressive Strength	kPa (min)	110	ASTM D1621 (Procedure A)

Flexural Strength	kPa (min)	240	ASTM C203
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Thermal Resistance	m ² .°C/W (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	4	ASTM D2842

5.02.03.02 Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel blocks of minimum acceptable dimensions of 1200 mm x 600 mm x 300 mm.

The maximum deviation from the specified linear dimensions shall be $\pm 1\%$. The flatness of the block faces shall be within ± 10 mm of a line formed by a 3 m straight edge.

The maximum difference in corner-to-corner dimensions (squareness) shall be 0.5%. The thickness shall be within -3 to $+5$ mm.

5.02.03.03 Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 110 kPa at a strain of not more than 5%. The maximum permissible permanent stress level should not exceed 30% of the compressive strength of the material at 5% strain.

5.02.03.04 Flexural Strength

The minimum flexural strength of the polystyrene shall be 240 kPa. The flexural strength shall be determined in accordance to ASTM C203, method 1, Procedure B.2.7.4 Dimensional Stability.

5.02.03.05 Dimensional Stability

Dimensional Stability shall be determined in accordance with ASTM D2126, Procedure G. A tolerance of 1.5% shall be satisfied.

5.02.03.06 Thermal Resistance

The thermal resistance shall be 0.7 m².°C/W for a 25 mm thickness using the following equation and using the average value from three specimens:

$$R_{25\text{mm}} = \frac{R_{\text{measured}}}{\text{thickness (mm)}} \times 25$$

The thermal resistance shall be measured in accordance with ASTM C177 or C518.

5.02.03.07 Flammability

The expanded polystyrene shall be classified as to surface burning characteristics in accordance with CAN/ULC - 51022 having a flame spread rating less than 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863

5.02.03.08 Water Absorption

The water absorption as measured by ASTM D2842 shall be limited to 4% by volume.

5.02.03.09 Chemical Resistance

The expanded polystyrene shall be resistant to common inorganic acids and alkalis. A table identifying the chemical resistance as either resistant limited or not resistant shall be submitted.

5.02.03.10 Biological Resistance

The expanded polystyrene shall be resistant to biological degradation caused by organisms or enzymes.

5.02.03.11 Environmental

The expanded polystyrene shall be inert, non-nutritive and highly stable and shall not produce undesirable gases or leachate.

5.03 Polyethylene Sheeting

Polyethylene sheeting shall be a minimum 10 mil thick.

6.0 EQUIPMENT

6.01 Cutting

All cutting of polystyrene materials shall be by electric equipment or by hand.

6.02 Heavy Equipment

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

7.0 CONSTRUCTION

7.01 Supplier Representation

The Contractor shall have on site at the commencement of the work, a representative of the supplier of the rigid expanded polystyrene to advise on recommended construction procedure.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

7.02 Delivery, Storage and Handling

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

7.03 Excavation

Excavation shall be according to OPSS 902.

7.04 Leveling Pad

Place, level and compact a layer of Granular 'A' or Granular 'B' material in accordance with OPSS 501 to within ± 30 mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks. The leveling pad shall not be placed on frozen ground.

7.05 Installation of Blocks

1. The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.
2. Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.
3. A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.
4. Sloping end adjustments at the abutments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.
5. Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.
6. The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.
7. The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction. The proposed method of protection during construction shall be submitted to the Contractor's Quality Verification Engineer for review and to the Contract Administrator for information purposes.
8. Exposed blocks shall be covered immediately to avoid possible burrowing by animals.
9. Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.

10. The top surface and side surfaces of the expanded polystyrene shall be covered with polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.
11. The Contractor shall install the concrete base pad as detailed elsewhere in the Contract.

8.0 QUALITY ASSURANCE

8.01 General

The Contract Administrator may undertake an independent testing program of the expanded polystyrene. Sampling and testing will be carried out in conformance with the relevant test procedure. The physical and thermal property testing identified in Table 1 will be conducted. A recognized testing laboratory accredited by the Standards Council of Canada shall conduct the testing.

8.02 Sampling Frequency

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. As a minimum, three blocks shall be tested.

8.03 Acceptance/Rejection

Failure of any one of the sample blocks to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the blocks shall be at the Contractor's expense.

9.0 MEASUREMENT FOR PAYMENT

9.01 Actual Measurement

Measurement will be by volume in cubic metres measured in its original position and based on cross-sections.

10.0 BASIS OF PAYMENT

10.01 Expanded Polystyrene Embankment - Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

The granular leveling pad and concrete pad shall be paid for with the appropriate tender items as detailed elsewhere in the Contract.

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 bridge 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 abutment areas 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

OBSTRUCTIONS – Item No.

Special Provision

1.0 SCOPE

The embankments at this site are comprised of granular/rock fill and are known to contain cobbles and boulders. Appropriate equipment and construction procedures will be required to penetrate or remove obstructions, such as cobbles and boulders, to permit installation of the protection system.

2.0 BASIS OF PAYMENT

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

VIBRATION MONITORING - Item No.

Special Provision

1.0 SCOPE

This Special Provision covers the requirements for the vibration monitoring during installation of driven protection systems.

2.0 REFERENCES - Not Used

3.0 DEFINITIONS

For the purpose of this specification, the following definitions apply:

Certificate of Conformance means a document issued by the Quality Verification Engineer confirming that the specified components of the Work are in General Conformance with the requirements of the Contract Documents.

Quality Verification Engineer means an Engineer retained by the Contractor qualified to provide the services specified in the Contract Documents. The Engineer shall have a minimum of five (5) years experience in the field of installation of protection systems and vibration monitoring or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the contract.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Submission Requirements

4.01.01 General

The Contractor shall submit details of the vibration monitoring plan to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- a) Qualifications of vibrations monitoring specialist.
- b) Proposed instrumentation.
- c) Proposed location of instruments.
- d) Proposed frequency of readings.
- e) Proposed methods for adjusting piling methods if readings show vibrations exceeding tolerable levels.

The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

4.02.01.02 Monitoring Submissions

The measured results of the vibration monitoring shall be submitted to the Contract Administrator after each section of protection system has been driven and prior to continuing with the subsequent sections. As a minimum, the section number, location, set criteria and driving log must be submitted with vibration monitoring results.

4.01.03 Certificate of Conformance

A completed Certificate of Conformance shall be submitted to the Contract Administrator upon completion of the pile installation work. The Certificate of Conformance shall be sealed and signed by a QVE and shall state that the pile installation work has been carried out in general conformance with the Contract Documents. The Certificate of Conformance shall also certify that the monitoring submissions have been completed as specified.

5.0 MATERIALS - Not Used

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 Monitoring

The Contractor shall take readings during driving of the protection systems. The readings shall be taken and recorded during the entire length of driving.

The section of the protection system furthest from the monitored structure or utility should be driven first to assess the vibration level at the existing structures. If necessary, the Contractor shall alter the driving procedures for the remaining sections of the protection system. The revised procedure shall be submitted to the Contract Administrator for approval prior to driving the remaining sections of the protection system.

The measured vibrations shall not exceed 100 mm/s (peak particle velocity).

The results shall be submitted to the Contract Administrator after each section has been driven and prior to continuing with the subsequent sections. If the vibration monitoring results are acceptable, the Contractor may continue with the next section. If the readings are not within the limits stated above, the Contractor shall alter the driving procedures until the vibrations are within acceptable levels.

8.0 QUALITY ASSURANCE - Not Used

9.0 MEASUREMENT FOR PAYMENT - Not Used

10.0 BASIS OF PAYMENT

10.01 Vibration Monitoring - Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment, and Material to do the work.

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