



February 2016

REPORT ON

**Foundation Investigation and Design
Jones Creek Highway and Trail Bridges
Thousand Island Parkway
United Counties of Leeds and Grenville, Ontario
W.P. 4092-10-02 (Highway Bridge) and
W.P. 4270-11-01 (Trail Bridge)**

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REPORT



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PART A

FOUNDATION INVESTIGATION REPORT

SITE NOS. 16-110/01 AND 16-110/02

JONES CREEK BRIDGES

THOUSAND ISLAND PARKWAY

UNITED COUNTIES OF LEEDS AND GRENVILLE, ONTARIO

W.P. 4092-10-02 (HIGHWAY BRIDGE) AND 4270-11-01 (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) over Jones Creek on the Thousands Island Parkway (Site Nos. 16-110/01 and 16-110/02, respectively) in the United Counties of Leeds and Grenville, Ontario (WP 4092-10-02 and WP 4270-11-01, 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 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, two letters by Golder dated March 2015 and July 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 highway and trail bridges over Jones Creek are located in the northern portion of the Thousand Islands Parkway approximately 2.5 km southwest of the eastern interchange of Highway 401 and the Thousand Islands Parkway. The existing bridges have an approximate southwest-northeast orientation and are located approximately 250 m upstream of the mouth of Jones Creek where it joins with the St. Lawrence River. For clarity, the north (trail) and south (highway) bridges are described herein as having an east-west orientation. The embankments and bridge structures are described relative to the east and west abutments.

The Jones Creek channel at the site is approximately 70 m to 80 m wide. Approach/causeway embankments for a previous bridge structure with an approximate north-south orientation were visible to the south of the existing bridge at the time of the initial site visit. The existing highway and trail bridges are both approximately 31 m long, single-span truss structures with reinforced concrete decks. The original design drawings from 1939 indicate that the structures are each about 11.1 m wide. The south (highway) structure has a cantilevered sidewalk on the south side of the truss. The south bridge has a roadway width of approximately 8.8 m and accommodates one lane of traffic in each direction. The north bridge forms part of a pedestrian trail system and no longer carries highway traffic.

Available bridge drawings (Drawing D2648 Sheets 1 and 2 dated September 1939) indicate the north and south bridges share common foundations consisting of reinforced concrete abutment footings founded at an elevation of 252 feet (~76.8 m). Drawing D2648 Sheet 4, dated May 1940, indicates that the normal creek water level elevation was 244.7 feet (~74.6 m) and that the creek channel bed elevation was approximately 238 feet (~72.5 m). The elevations of the bridge decks are approximately 263.7 feet (~80.4 m). Based on this information, the road surface/top of the approach embankments at the bridge site are in the order of 5 m above the historic normal creek water level.

No site specific information for the Thousand Islands Parkway crossing site at Jones Creek was available in the MTO Pavement and Foundations Section's GEOCRESS database. Furthermore, no subsurface information was available on the 1939/1940 bridge drawings.

The following provides a summary of key observations made during a site visit conducted on April 7, 2014.

North (Trail) Bridge

- Scour/erosion had exposed the front toe of the slope and had thus begun to undermine the abutment foundations on both sides of the river. The leading edges (i.e., upstream portions) of the abutment footings were noted to be lower than the adjacent sections of the footings as shown in the photo below; this is inferred to have been the result of remedial work (i.e., placement of concrete protection/underpinning) carried out on these footings.





- Soil present at and beneath the toe of the abutments typically consists of cobble and boulder sized rock fragments contained in a soil matrix.
- A significant amount of water originating from snowmelt was cascading down the front face of the abutments as well as dripping through the bridge deck.
- The top of the southwest abutment was noted to be cracked and displaced and significant erosion had taken place around the tops of the western abutment wingwalls on both sides of the structure.

South (Highway) Bridge

- Similar to the lower sections of abutment foundations noted on the north (trail) bridge, the south portion of the east abutment was lower than the remaining central section of the abutment footing. It is noted that the ground surface at the toe of the abutment in between the two bridges on the west side of the river is higher than at the ends of the abutments and may represent the original (i.e., pre-erosion) ground surface.
- Erosion had occurred around the south abutment wingwall at the east abutment. Concrete surfacing that appears to have been placed to limit the erosion was undermined in some areas.

Approach Embankments

- The approach embankment sideslopes are relatively steep; available topographic information (photogrammetry files provided by MTO) suggests that portions of these embankments have sideslopes as steep as 35 to 40 degrees (~1.2 horizontal to 1 vertical). Erosion of the sideslopes of the embankment had occurred near the north wingwall at the west abutment, and at both wingwalls at the east abutment. The embankment materials exposed in these areas were comprised of predominantly granular fill containing cobble and boulder-sized rock fragments.
- The toe of the southern side slope of the east approach embankment was noted to be eroded and oversteepened at and immediately above creek level. Rock fill/rip rap materials present beyond (east of) the area of embankment oversteepening extend further into the channel suggesting that the toe of the embankment fill materials may have been eroded by creek action in this area.
- An erosion channel formed by road runoff was also present on the east embankment face.
- Cracking of the pavement was noted at the ends of the approach slabs; significant cracking was not visible in other areas.
- The pavement surface leading up the highway bridge had an uneven appearance suggesting that some settlement of the approach embankments may have occurred.



3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out in July 2014, when six boreholes (designated as Boreholes 14-1 to 14-6) were advanced at the site. Boreholes 14-2, 14-5 and 14-6 were drilled using a GEOPROBE drill rig, supplied and operated by the Strata Drilling Group, which utilized a Symmetrix (down-the-hole air hammer) system to advance a 127 mm diameter casing. Boreholes 14-1, 14-3 and 14-4 were advanced using a CME75 truck-mounted drill rig, supplied and operated by George Downing Estate Drilling of Grenville-sur-la-rouge, Quebec. These boreholes were drilled using a combination of 200 mm inside diameter hollow stem augers and rotary drilling using HQ and NQ sized casing.

Furthermore, an additional borehole (designated as Borehole 15-1) was completed in November 2015 at the site and was advanced using a CME75 truck-mounted drill rig, supplied and operated by George Downing Estate Drilling of Grenville-sur-la-rouge, Quebec. This additional borehole was advanced to assess the subsurface conditions at the end of the proposed retaining wall. This borehole was drilled using a combination of 200 mm inside diameter hollow stem auger drilling and rotary diamond drilling using HW casing.

The boreholes were advanced to depths ranging from 13.7 m to 25 m below ground surface. Soil samples were typically obtained at 0.75 m to 1.5 m intervals of depth using a 50 mm outside diameter split-spoon sampler driven by an automatic hammer in general accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586-08a). Boreholes 14-2, 14-5 and 14-6 were advanced using a down-the-hole hammer to advance the drill string. Following refusal of the down-the-hole hammer at Borehole 14-2, a geoprobe casing was advanced to attempt to recover additional samples and advance the borehole. The geoprobe, and a subsequent Dynamic Cone Penetration Test (DCPT) both met effective refusal at approximately the same depth as the down-the-hole hammer. Boreholes 14-1, 14-3, and 14-4 were advanced through the overburden using hollow stem augers followed by wash boring and coring techniques into the underlying bedrock.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations and a standpipe piezometer was installed in one borehole (Borehole 14-1) to permit monitoring of the groundwater level. The piezometer consists of a 32 mm diameter PVC pipe, with a slotted screen sealed within a sand filter pack at a selected depth interval within the borehole. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was backfilled to the ground surface with bentonite pellets. The piezometer installation details and water level readings are indicated on the borehole record contained in Appendix A. All remaining boreholes were backfilled with bentonite and soil cuttings upon completion, in accordance with Ontario Regulation 903 (as amended).

The field work was supervised on a full-time basis by a member of Golder's engineering staff who located the boreholes in the field, observed the drilling, sampling, and in situ testing operations, and logged the subsurface conditions encountered in the boreholes. 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, organic content testing, grain size distribution analyses and unconfined compressive strength (UCS) tests were carried out on selected soil and bedrock samples.

The borehole locations and ground surface elevations were surveyed by Golder Associates Ltd. using a Trimble R8 GPS unit. The borehole locations, including MTM NAD83 northing and easting coordinates, and ground surface elevations referenced to Geodetic datum are summarized in the following table and are shown on Drawings 1 and 2.



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Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
15-1	4929544.2	360029.5	80.9	14.0
14-1	4929511.4	359984.5	80.1	25.0
14-2	4929476.3	359964.3	80.0	17.6
14-3	4929508.0	360001.7	80.3	23.8
14-4	4929473.3	359980.4	80.3	17.1
14-5	4929461.7	359966.9	80.5	14.3
14-6	4929522.2	360004.3	80.3	13.7*

Note: * Dynamic cone penetration test carried out to depth of 21.2 m in Borehole 14-6.



4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

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 sandstone, dolomitic sandstone or dolostone of the Nepean and March Formations. Precambrian bedrock is also present in close proximity to the site towards the St. Lawrence River; outcrops of Precambrian rock are visible on the northeast bank of the channel at the mouth of Jones Creek. In this regard, the creek channel may have eroded through the sedimentary rock and into the Precambrian strata.

Information from the Ministry of Environment (MOE) water well database in the vicinity of the site was collected and reviewed. Review of the well records indicates that wells located near to but outside of creek channel typically encountered bedrock at shallow depth (i.e., less than 1.5 m below ground surface). Sedimentary bedrock was encountered in wells located to the west/north of the Thousand Islands Parkway while granite bedrock was typically present in the wells to the east/south of the Thousand Islands Parkway.

4.2 Site Stratigraphy

The detailed subsurface stratigraphic and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole and Drillhole sheets contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B9 contained in Appendix B.

An interpreted stratigraphic section projected along the centreline of the proposed bridge alignment is shown on Drawing 1. Stratigraphic cross-sections at the east and west abutment locations are shown on Drawing 2. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the location of the proposed Jones Creek bridge replacements consist of surficial asphaltic concrete layers along the south (highway) bridge and granular fill, overlying embankment fill that contains cobbles and boulders. The fill material is typically underlain by organic silt and peat, overlying silt and sand deposits that are, in turn, underlain by bedrock.

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

4.2.1 Pavement Structure and Embankment Fill

The Thousand Islands Parkway pavement structure was penetrated at Boreholes 14-3 and 14-4. At the borehole locations, the pavement structure consists of about 130 mm asphaltic concrete, overlying about 250 mm of Portland cement concrete. No reinforcement was encountered in the cement concrete at the borehole locations, but it is anticipated that the Portland cement concrete may be part of the approach slab due to the proximity to the bridge structure and thickness of the cement concrete encountered (about 0.3 m). Sand and gravel fill was present beneath the surficial concrete layers at Boreholes 14-3 and 14-4, and at



ground surface at the remaining boreholes locations (Boreholes 15-1, 14-1, 14-2, 14-5, and 14-6). The surficial sand and gravel fill was encountered to depths of up to about 2.1 m below the existing ground surface with thicknesses ranging from 9.6 to 12.2 m.

The results of grain size distribution tests carried out on samples of the surficial pavement structure fill from Borehole 14-2 and 15-1 are shown on Figure B1.

At all borehole locations, the surficial sand and gravel fill was underlain by embankment fill containing sand, gravel, and a significant proportion of cobble and boulder sized particles. Cobble and boulder sized pieces were observed to be part of the embankment fill during the site visit carried out in April 2014. During the current drilling investigation, the presence of cobbles and boulders within the boreholes were inferred based on recovered samples and behaviour of the drill rig during the difficult drilling conditions. Where auger drilling met refusal, washboring techniques were used to penetrate the embankment fill. The fill was encountered to depths ranging from about 9.6 to 12.3 m (Elevations 70.5 to 68.0 m) beneath the west embankment, and 9.8 to 12.2 m (Elevations 70.7 to 67.8 m) beneath the east embankment.

The results of grain size distribution testing carried out on six selected samples obtained from the upper 6 to 7 m of the embankment fill are provided on Figure B2 in Appendix B. Within the lower 1 to 4 m of the fill embankments the fill consists primarily of sandy gravel and fewer cobbles and boulders were inferred to be encountered during drilling. Grain size distribution test results from five selected samples of the lower sandy gravel fill are provided on Figure B3 in Appendix B. The test results shown on the figure do not reflect the cobble/boulder or large gravel content of the material, since the samples were retrieved using a 50 mm outside diameter split-spoon sampler. The measured natural water contents of five selected samples of the embankment fill range from 8 to 29 percent, but are typically between about 10 and 14 percent.

The SPT “N” values measured in the embankment fill were highly variable ranging from 4 to greater than 100 blows per 0.3 m of penetration, indicating the fill is loose to very dense. Effective refusal of the split-spoon sampler (i.e., greater than 50 blows for 75 mm advancement) was encountered at discrete locations within the rock fill in Boreholes 14-1, 14-2, 14-3, and 14-6, and this is inferred to have occurred on cobbles and/or boulders in the fill.

4.2.2 Organic Silt / Peat

Organic soils varying from organic silt to peat were encountered beneath the granular fill at Boreholes 14-1, 14-4, 14-5, and 14-6. The thickness of the deposit was inferred from drilling conditions and the samples recovered, and ranged from about 0.6 m, at Borehole 14-4, to about 2.3 m at Borehole 14-5. No samples of the organic deposit were recovered at Borehole 14-2 and 14-3; however, organics were observed in the cuttings returned to surface during drilling through the soil near the bottom of the granular fill deposit. No organic deposits were found in Borehole 15-1 below the embankment fill.

Typically, the recovered samples of the organic deposit included gravel and cobble-sized particles. The presence of these coarse-grained particles is inferred to be a result of penetration into the organic silt following placement of the overlying granular fill. The coarse grained particles were removed from the samples prior to testing to provide lab test results representative of the fine-grained and organic portion of the samples. The measured natural water contents of three samples of the organic deposit range from 172 to 249 percent. The measured organic contents of the three samples range from 16 to 36 percent.



Most of the SPTs that encountered the organic deposit were advanced across a stratigraphic boundary with another deposit. Therefore, most of the SPT “N” values measured in the organic deposit are not considered to be representative of the organic deposit itself. The exception was at Borehole 14-5 where the SPT “N” value measured within the organic deposit was 8 blows per 0.3 m of penetration.

Where encountered, the organic soils were penetrated at depths ranging from 11.3 to 12.1 m below the existing ground surface (Elevation 69.0 to 68.3 m).

4.2.3 Gravelly Sand

Beneath the embankment fill and/or organic deposit, a layer of gravelly sand was encountered at the boreholes put down near the eastern edge of the existing abutments. At the east abutment (Borehole 14-3), the stratum was encountered directly below the embankment fill and consists of gravelly sand, trace silt, and contains organics. At the west abutment (Borehole 14-4), the stratum was encountered beneath the peat deposit and consists of gravelly silty sand. The gravelly sand stratum ranged from 0.5 to 1.3 m thick.

The results of grain size distribution testing carried out on one sample of the gravelly sand from each of Borehole 14-3 and 14-4 are provided on Figure B4 in Appendix B. The measured natural water contents of two selected samples of the gravelly sand were 19 and 21 percent.

An SPT “N” value measured in the gravelly sand deposit at Borehole 14-3 was 36 blows per 0.3 m of penetration, indicating a dense state of packing.

The gravelly sand was penetrated at depths ranging from 12.6 to 12.8 m below the existing ground surface (Elevation 67.7 to 67.5 m).

4.2.4 Silty Clay/Clayey Silt

Beneath the embankment fill a thin, 200 mm thick, layer of silty clay to clayey silt was encountered at Borehole 15-1.

An SPT “N” value measured in the silty clay/clayey silt deposit at Borehole 15-1 was 16 blows per 0.3 m of penetration, indicating a very stiff consistency.

The silty clay/clayey silt was penetrated at a depth of 11.2 m (Elevation 69.7 m) below the existing ground surface.

The results of a grain size distribution test carried out on one selected sample of the silty clay/clayey silt are given on Figure B9 and are included in Appendix B.

4.2.5 Sand and Silt

Deposits of sand to sandy silt were encountered beneath the embankment fill, organic deposit, silty clay/clayey silt, and/or gravelly sand layer at all borehole locations. These deposits range in composition from sand with interlayers of sandy silt to silt with some sand to sandy silt. At Borehole 14-1, cobbles and/or boulders were inferred to be encountered during drilling at about 15.1 m depth (Elevation 65.0 m).

The thickness of the sand and silt deposits encountered at the borehole locations ranges from about 0.7 m (at Borehole 14-5) to about 10.4 m (at Borehole 14-1), and was at least 2.4 m at borehole 15-1 where it was not fully penetrated. In general, the deposits are thicker near the northern portion of the east approach embankment and thinner near the southern portion of the west approach embankment. The sand and silt



deposits were fully penetrated at Boreholes 14-1, 14-3, 14-4, and 14-5. Drilling and sampling at Borehole 14-6 was terminated within the sand and silt layers following refusal of the down-the-hole-hammer equipment; however, a dynamic cone penetration test was advanced to effective refusal on a very dense stratum.

The results of grain size distribution tests carried out on seven selected samples of the sand and silt deposits are included in Appendix B. Figure B5 shows the results of the testing carried out on three selected samples obtained within the upper portion of the deposits at Boreholes 14-1, 14-6, and 15-1 (from between Elevations 66.0 and 68.0 m) where the deposit is predominantly composed of sand. The results of the testing carried out on four samples that have a higher silt content (obtained between Elevations 63.0 and 67.5 m) from Boreholes 14-1, 14-2, and 14-3 are shown on Figure B6. The results of testing carried out on two samples of sand from the lower portion of the deposits directly above the bedrock are shown on Figure B7.

The measured natural water contents of nine selected samples of the sand to sandy silt range from 15 to 24 percent.

SPT “N” values measured in the sand and silt deposits fall range from 2 to 35 blows per 0.3 m of penetration but were typically greater than about 10 blows per 0.3 m, indicating a compact state of packing.

4.2.6 Bedrock

Bedrock was encountered beneath the sand and silt deposits where they were fully penetrated at Boreholes 14-1, 14-3, 14-4 and 14-5. In Borehole 14-5 the bedrock was completely weathered and was penetrated by augering for a depth of 1.6 m while at the remaining boreholes it was cored for lengths between 3.0 and 3.2 m. At Boreholes 14-2 and 14-6, the depth to bedrock was inferred from Dynamic Penetration Test (DPT) refusal. Figure B8 shows the results of a grain size distribution test on a sample of completely weathered granitic bedrock from Borehole 14-5 at a depth of about 12.7 m. The following table summarizes the bedrock surface depths and elevations as encountered at the three borehole locations.

Borehole Number	Borehole Location	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
14-1	East Abutment (Trail Bridge)	80.1	22.0	58.1
14-2	West Abutment (Trail Bridge)	80.0	17.6 ^a	62.4 ^a
14-3	East Abutment (Highway Bridge)	80.3	20.6	59.7
14-4	West Abutment (Highway Bridge)	80.3	14.1	66.2
14-5	West Embankment	80.5	12.7 ^b	67.8 ^b
14-6	East Embankment	80.3	21.2 ^c	59.1 ^c

Notes: ^a Bedrock surface inferred from refusal of Dynamic Penetration Test, down-the-hole hammer, and geoprobe

^b Completely weathered bedrock (residual soil)

^c Bedrock surface inferred from refusal of Dynamic Penetration Test

The bedrock encountered in the boreholes put down to the west of Jones Creek consists of interlayered fresh to moderately weathered, red-grey to orange-brown, medium strong to very strong granite. At Borehole 14-5, about 1.5 m of silty sand, containing some gravel was encountered at a depth of 12.8 m depth (Elevation 67.7 m) and



was inferred to be completely weathered granite bedrock. Within the 3.0 m of bedrock cored at Borehole 14-4, the fresh granite was interlayered with moderately weathered portions at depths of about 0.1, 0.6, and 1.8 m below the top of the bedrock. The Rock Quality Designation (RQD) values measured on the recovered granite bedrock core samples range from about 65 to 75 percent, indicating fair quality rock.

At the boreholes put down within the east embankment of Jones Creek, bedrock consisting of slightly weathered to fresh, red-grey and green, medium strong to strong monzogabbro (i.e., similar to a granite and part of same family of bedrock) was encountered. At Borehole 14-3, the monzogabbro was moderately altered in portions of the core. The Rock Quality Designation (RQD) values measured on the monzogabbro bedrock core samples range from about 30 to 60 percent, indicating poor to fair quality rock.

Laboratory unconfined compressive strength testing was carried out on selected specimens of the bedrock core. The results of the unconfined compressive strength testing on samples of the bedrock indicate values ranging from 19 to 72 MPa in the monzogabbro, and 173 MPa in the granite.

4.2.7 Groundwater Conditions

During the field investigation, the water levels at the borehole locations were observed to range between 5.4 and 6.1 m depth below the existing ground surface (Elevations 74.3 m to 74.9 m) in the open boreholes. A monitoring well was installed in Borehole 14-1, and the groundwater level measured in the monitoring well was measured on August 10, 2014 at a depth of about 5.4 m below the existing ground surface (Elevation 74.7 m). The water level in Jones Creek was at Elevation 76.5 in October 2014.

It should be noted that groundwater levels in the area are expected to fluctuate with the water level in Jones Creek and are therefore subject to fluctuations both seasonally and with precipitation events.

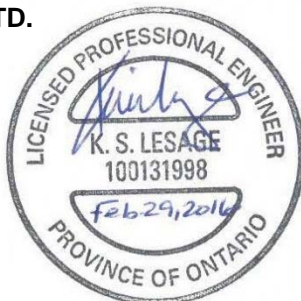


5.0 CLOSURE

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

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PART B

FOUNDATION DESIGN REPORT
SITE NOS. 16-110/01 AND 16-110/02
JONES CREEK BRIDGES
THOUSAND ISLAND PARKWAY
UNITED COUNTIES OF LEEDS AND GRENVILLE, ONTARIO
W.P. 4092-10-02 (HIGHWAY BRIDGE) AND 4270-11-01 (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 Jones Creek. The recommendations are based on interpretation of the factual data obtained from the boreholes 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 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 Drawings 1 and 2, consist of single-span truss structures with reinforced concrete decks that are approximately 31 m long. The south (highway) structure has a cantilevered sidewalk on the south side of the truss. The original design drawings from 1939 indicate that the structures are each about 11.1 m wide. The south (highway) bridge has a roadway width of approximately 8.8 m and accommodates one lane of traffic in each direction. It is understood that the bridge deck was replaced in 1988 and underpinning of the abutment foundations of the north (trail) bridge was carried out in 2012. The north (trail) bridge forms part of a pedestrian trail system and no longer carries highway traffic.

The existing bridges are planned to be replaced with two new, single-span permanent bridges; a new two-lane highway bridge located along the approximately the same alignment as the existing highway bridge and a trail bridge, with both structures being about 31 m in span length. The trail bridge will also provide temporary highway traffic access during the highway bridge construction. Furthermore, 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. Grade raises on the order of 300 to 500 mm are being proposed for the approach embankments to the bridges.

6.2 Existing Foundations

Available bridge drawings from 1939 indicate the north and south bridges share common foundations consisting of 34 m long by 1.8 m wide reinforced concrete abutment footings founded at an elevation of about 76.8 m within and supported on the embankment fills. Based on site observations, the lower front edges of the abutment foundations have been exposed by erosion at several locations and portions of the abutment foundations extend below the design elevation. It is understood that modification to the original foundation configuration (i.e., underpinning or placement of concrete facing in front of the abutment foundations) has been undertaken recently (in 2012) to address foundation performance concerns associated with scour and erosion that has occurred in front of the abutment foundations.

Bridge condition/inspections reports for the existing south (highway) bridge indicate that this structure is exhibiting deterioration with light to moderate pitting, corrosion and flaking of steel members, cracking of concrete, honeycombing and exposed corroded rebar in ballast walls, etc. Rehabilitation of this bridge has been carried out in the past with the replacement of the bearings completed on the west abutment during the 1998



rehabilitation activities. The north (trail) bridge no longer carries highway traffic. There are no bridge condition/inspection reports or information related to previous rehabilitation activities available for this bridge. However, the top of the west abutment wall was noted to be cracked and displaced during the site reconnaissance.

During an April 2014 site inspection, Dillon personnel identified that elastomeric bearings of the highway bridge were in a state of expansion even though the temperature was relatively low (about 7 degrees Celsius). It was also noted that the gusset and shoe plate were in contact with the ballast wall on the east side of the bridge. Further assessment by Dillon identified that the existing abutment configuration resulted in an uplift condition of the heel of the foundation at SLS suggesting that the noted differential movement between the substructure and the superstructure may have been a result of an overturning force on the foundation.

Erosion of the approach embankments has taken place around the existing abutment walls particularly near the crest of the embankment. In addition, a portion of the downstream side of the east approach embankment, located immediately above creek level shows signs of loss of vegetation/sloughing/oversteepening that is inferred to be a result of shallow slope instability and/or creek erosion.

6.3 Foundation Options

Based on the subsurface conditions, only deep foundation options have been considered for the replacement of the Jones Creek bridges, as the use of shallow foundations could lead to unacceptable settlement performance for the structure due to the identified presence of the organic soils beneath/at the base of the existing embankment fill materials. A summary of the advantages and disadvantages associated with each deep foundation option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, constructability and relative costs is provided in Table 1 following the text of this report.

- **Driven steel H-piles:** Steel H-piles driven through the embankment fill and underlying overburden soil to refusal on the underlying bedrock are feasible for support of the replacement bridge structures. Piles driven to bedrock will provide high geotechnical resistances and minimize foundation settlement. The use of steel H-pile foundations would also allow for the construction of integral abutments. Due to the potential for damage or misalignment of the piles resulting from encountering numerous cobbles and boulders within the existing embankment fill materials, installation of some of the piles through the embankment fill will require pre-drilling through the embankment fill.

If the driven piles are utilized, the use of bearing points is recommended to minimize damage while penetrating through the embankment fill (which contains cobbles and boulders) and seating onto the bedrock.

- **Driven steel pipe (tube) piles:** Closed-ended steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutments, and this foundation option would have similar advantages to steel H-piles in terms of minimizing foundation settlement. However, pipe piles are considered to have a higher risk than H-piles for “hanging up” or being deflected away from their vertical or battered orientation if cobbles and/or boulders are encountered and would be more difficult to seat onto the sloping bedrock present at the site.
- **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. The micropiles would require permanent casings through the rock fill zone to prevent the loss of grout into the rock fill during installation. The micropiles could be supported by end-bearing on the competent (unweathered) bedrock.



- **Drilled concrete caissons:** Caissons deriving their support from bearing within the bedrock are also feasible for this site. However, given the anticipated difficulties and mitigation measures required for installation of the drilled caissons described below, the use of drilled caissons are not considered to be practical or cost effective at this site. Caissons would require the use of temporary or permanent liners to mitigate the potential risks of ground loss from within the portions of the granular embankment fill below the water level and the water bearing cohesionless sand and silt layers during construction; establishing a seal between the liner and the bedrock could be problematic due to the strength and sloping nature of the rock surface. In addition, the caissons would have to be socketted into the bedrock and such sockets would have to be advanced by rock coring into the granite or monzogabbro bedrock. The presence of cobbles and boulders may require churn drilling and possibly rock coring techniques to penetrate obstructions where encountered in the embankment fill.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments for the replacement bridge structures on steel H-piles driven to found on the bedrock. For the proposed retaining wall (at southeast corner of new highway bridge) and wing wall (at trail bridge), these structures could be integrated into the existing abutment walls and founded on driven H-piles or be structurally separate from the abutment walls and founded on shallow foundations.

Based on the results of the investigations, steel piles driven to the bedrock surface would be greater than 5 m in length and are therefore considered to be feasible for use in an integral abutment configuration. However, for an integral abutment configuration, the upper portion of each pile is typically cased in a loose sand filled, corrugated steel pipe or within a trench of loose granular soils to provide suitable flexibility of steel H-piles. The corrugated steel pipes or granular trench for the integral abutments would extend below the water level within the existing very loose to very dense sand and gravel fill containing cobbles and boulders and would require either predrilling of larger holes in conjunction with temporary casings (for CSP installation) or excavations below the water table carried out in conjunction with temporary support systems to limit loss of ground into the excavation and protect the adjacent roadways.

Foundations for the temporary trail bridge located north of the existing trail bridge could consist of any of the aforementioned options. Alternatively, given their temporary nature and usage, the temporary structure could be founded on spread footings placed within the existing embankment fill.

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

6.4.1 Founding Elevations

The abutments for the replacement bridges may be supported on steel H-piles or closed-ended pipe (tube) piles driven to found on the bedrock. Based on the geotechnical investigations carried out at the site, the bedrock surface is considered to typically slope down to the northwest from about Elevation 67.7 m to 58.1 m at the borehole locations. Variations in the bedrock elevations should be expected at this site, based on the results of the field investigations and the nature of the Precambrian granitic or monzogabbro bedrock. The following table provides a summary of competent bedrock surface elevations at the boreholes advanced in the vicinity of each foundation unit and can be considered to provide an approximate tip elevation for preliminary design purposes. Piles which encounter fresh/sound bedrock are expected to encounter effective refusal to penetration at or near the bedrock surface. A zone of completely weathered bedrock (residual soil) was inferred to have been encountered at the location of Borehole 14-5 and zones of moderately weathered bedrock were encountered at



other borehole. Additional penetration of the piles into the bedrock may occur where the surface of the bedrock is moderately to completely weathered.

Structure	Foundation Element	Borehole Numbers	Competent Bedrock Surface Elevation (m)
Trail Bridge	East Abutment	14-1	58.1
	West Abutment	14-2	62.4*
Highway Bridge	East Abutment	14-3	59.7
	West Abutment	14-4	66.2

Note: * Bedrock surface inferred from refusal of Dynamic Penetration Test, down-the-hole hammer, and geoprobe; no bedrock coring completed.

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*). In general, the embankment fill encountered at the borehole locations consists of sand and gravel that contains cobble and boulder sized particles. The presence of cobbles and boulders should be considered during pile driving. Steel H-piles reinforced at the tip with rock points (such as Titus Injector Pile Points, or equivalent) should penetrate this layer, provided predrilling is completed to break up larger particles that obstruct pile driving, and continue through the underlying sand and silt layers to the surface of the competent bedrock. In this regard, it is recommended that provision be made to pre-auger to about elevation 68 m, through the embankment fill.

Due to the potential presence of cobbles and boulders within the existing embankment fill and the sloping nature of the bedrock at the site, steel H-piles are preferred over closed-ended steel pipe piles as closed-end pipe piles are considered to pose a higher risk of “hanging up” or being deflected away from their vertical or battered orientation during installation and seating of H-piles on the sloping bedrock is to be more practical than closed end pipe piles. Battered piles should be limited to a batter no greater than 10H:1V. As described above, the piles should be reinforced at the tip with bearing points to improve seating of the piles on the sloping bedrock and to reduce the potential for damage to the piles during driving. If steel pipe piles are used, the piles should be reinforced at the tip with driving shoes (such as Titus Open Cutting Shoes, or equivalent). Non-Standard Special Provisions (NSSPs) for the pre-drilling of deep foundations, H-piles, and the supply and installation of rock points should be included in the contract documents and samples have been provided in Appendix C of this report.

6.4.2 Axial Geotechnical Resistance

For design of HP 310x110 piles driven to the bedrock at estimated tip elevations provided in Section 6.4.1, the factored axial geotechnical resistance at ULS may be taken as 2,000 kN. Serviceability Limit States (SLS) resistances do not apply to piles founded on the bedrock at this site, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. Similar axial resistances may be used in the design of closed-end, concrete-filled, 324 mm diameter steel pipe piles having a minimum wall thickness of 9.5 mm.



Pile installation should be in accordance with OPSS 903 (*Deep Foundations*). 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 Downdrag Load (Negative Skin Friction)

The placement of additional fill and/or replacement of a portion of the existing embankments with fill of greater density than existing materials would raise the effective stress level in the compressible organic silt/peat layer that underlies the existing embankments, producing settlement that could generate downdrag loads on the new abutment piles. The magnitude of the downdrag loads would vary depending on the magnitude of a grade raise and on the sequence of construction.

It is understood that the current road grades are to be maintained and a grade increase of about 300 to 500 mm is envisioned as part of the current design. A portion of the embankment fill will also be excavated to allow for construction of the new abutments. Placement of additional fill to raise the existing grade and/or replacement of the embankments with fill having a greater density than that currently in place could produce settlement in the organic silt and peat layer that would generate downdrag forces, even if current grades are maintained. The surcharge imposed by any grade raise could be reduced through the use of abutment backfill materials that have a density that is lighter than the existing fill materials such as a clean, uniform sand.

Alternatively, if heavier backfill materials are used, consideration may be given to incorporating sufficient lightweight fill materials such as EPS Geofoam, tire-derived aggregate, or blast furnace slag as part of the abutment foundation backfilling and roadway re-grading to provide for a 'no net load increase' (i.e., a portion of the existing embankment may be excavated and replaced with lightweight fill such that there is no net increase in load on the underlying soil). The thickness of lightweight fill that would be required within the final embankments would depend on the type of lightweight fill, the difference in density of the existing and new embankment fill, as well as the overall thickness of fill to be replaced and/or grade raise to the embankment.

If the above options to mitigate settlement in the area of the abutments are not implemented, the structural design of the piles should take into account negative skin friction loads that would develop on the piles above the organic silt/peat layers. Based on an underside of pile cap elevation of about 75.2 m downdrag loads of up to 100 kN can be mobilized in the steel H-Piles from the settling embankment fill materials.

6.4.4 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 existing embankment fill containing cobbles and boulders is not considered practical at this site. Alternatively, the resistance to lateral loading can be derived from the soil in front of the vertical piles, and it may be assumed that this resistance will be nearly the same for vertical and inclined piles as indicated in Section C6.8.7.2 of the Commentary to the CHBDC.

For preliminary assessment purposes, the SLS 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 is the constant of horizontal subgrade reaction, as given below;
 z is the depth (m); and,
 B is the pile diameter/width (m).

For cohesive soils:

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

Where: s_u is the undrained shear strength of the soil (kPa); and,
 B is the pile diameter/width (m).

The following ranges for the values of n_h and s_u may be used in the structural analysis. The ranges in values reflect:

- The variability in the subsurface conditions and the soil properties;
- The approximate nature of the analysis;
- The non-linear nature of the soil behaviour (such that n_h is a function of deflection); and,
- The two extremes of the design; the requirement for flexibility in the case of integral abutments and the requirement for lateral resistance of horizontal loads.

Soil Unit	n_h (MN/m ³)	s_u (kPa)
Sand, gravel, cobbles, boulders (embankment fill) <u>above</u> Elevation 75 m (above groundwater level)	5 to 8	-
Sand, gravel, cobbles, boulders (embankment fill) <u>below</u> Elevation 75 m (below groundwater level)	3 to 5	-
Organic Silt / Peat	-	20
Loose to Compact Sand and Silt Layers	1 to 4	-

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 when 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 lateral subgrade reaction in the direction of loading by a reduction factor as follows:

Pile Spacing in Direction of Loading (d = Pile Diameter)	Reduction Factor
8d	1.0



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Pile Spacing in Direction of Loading (d = Pile Diameter)	Reduction Factor
6d	0.7
4d	0.4
3d	0.25

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

For establishing the ULS factored *structural* resistance, the shear force and bending moment distribution in the piles under factored loading can be established using the procedures and parameters given above for evaluating the SLS response of the pile.

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure theory as outlined in Section C6.8.7 of the *Commentary* to the CHBDC, assuming that it acts over the the pile shaft to a depth equal to six pile diameters below the underside of the pile cap. The ULS geotechnical resistance of the soils can also be estimated using the “Assessed Horizontal Passive Resistance Values for Various Pile Types” provided in the *Commentary* to the CHBDC.

The ULS lateral resistance of a pile group may be estimated as the sum of the individual pile resistances across the face of the pile group, perpendicular to the direction of the applied lateral force.

The ULS resistances obtained using the above parameters represent unfactored values; in accordance with the CHBDC, a resistance factor of 0.5 is to be applied in calculating the horizontal resistance.

6.5 Shallow Foundations for Temporary Bridge

6.5.1 Founding Elevations and Preparation Requirements

If shallow foundations are adopted to support the temporary bridge structure, these could be founded within the existing embankment fill. 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. 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.

The temporary footings 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 foundation subgrade materials should be inspected prior to foundation construction, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to check that softened/loosened soils or other unsuitable material have been removed.



6.5.2 Geotechnical Resistance/Reaction

Strip or spread footings up to 4.0 m in width that are placed on a properly prepared granular foundation pad perched within the rock fill approach embankments at elevations of about 79.0 m may be designed based on a factored geotechnical resistances of 200 kPa at Ultimate Limit States (ULS) and a geotechnical resistance of 180 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 differ from those given above.

These geotechnical resistances are provided for loads applied perpendicular to the surface of the footings and no closer than 0.5 m to the proposed 2H:1V (or flatter) adjacent embankment slope; 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*.

6.5.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade (i.e., granular foundation pad) should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on top of the compacted granular foundation pad, the coefficient of friction, $\tan \delta$ can be taken as 0.6 for design.

6.6 Shallow Foundations for Retaining Walls

6.6.1 Founding Elevations and Preparation Requirements

If shallow foundations are adopted for structures that are structurally separate from the deep foundation supported abutment walls (i.e. southeast retaining wall, trail bridge wing wall, temporary trail bridge), these could be founded within the existing embankment fill. 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. A Non-Standard Special Provision 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.

The footings 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 foundation subgrade materials should be inspected prior to foundation construction, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to check that softened/loosened soils or other unsuitable material have been removed.

6.6.2 Geotechnical Resistance/Reaction

Strip or spread footings up to 4.0 m in width that are placed on a properly prepared granular foundation pad perched within the rock fill approach embankments at elevations of about 75.0 m may be designed based on a factored geotechnical resistances of 200 kPa at Ultimate Limit States (ULS) and a geotechnical resistance of 180 kPa at Serviceability Limit States (SLS, for 25 mm of settlement). In addition to the SLS projected settlements, the proposed grade raises could induce up to an additional 50 mm of settlement unless these are



mitigated through the use of lightweight fill (see Section 6.10.3). The geotechnical resistances should be reviewed if the selected footing width or founding elevation differ from those given above.

These geotechnical resistances are provided for loads applied perpendicular to the surface of the footings and no closer than 0.5 m to the proposed 2H:1V (or flatter) adjacent embankment slope; 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*.

As an alternative to concrete cast-in-place wall for the southeast retaining wall, consideration could be given to an RSS segmental block wall (see Section 6.7 below) which would be able to accommodate the anticipated 50 mm of settlement resulting from the grade raise.

6.6.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade (i.e., granular foundation pad) should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on top of the compacted granular foundation pad, the coefficient of friction, $\tan \delta$ can be taken as 0.6 for design.

6.7 Shallow Foundations for Retained Soil System Wall

6.7.1 Founding Elevations and Preparation Requirements

A Retained Soil System (RSS) wall is considered a feasible option for the proposed retaining wall at the southeast corner of the southeast abutment. Given the limited height of the RSS wall of about 4 m, an interlocking block system with geogrid reinforcement is considered feasible. RSS walls are proprietary systems, designed by the supplier, and for which the designer/supplier must verify the internal stability of the wall. The global stability of the RSS wall was analyzed and found to be acceptable (see Section 6.7.3).

The RSS wall may be designed such that the facing blocks are constructed on a levelling pad. The levelling pad should be constructed using a minimum of 300 mm of Granular A. The levelling pad 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*).

To manage embankment settlements and recognizing that EPS lightweight fill is not a suitable backfill for an RSS wall, the backfill materials for this RSS wall should consist of clear stone with a maximum density of 19 kN/m³. A geotextile filter should be placed between the clear stone backfill and the pavement structure.

The foundation subgrade materials should be inspected prior to foundation construction, in accordance with OPSS 902 (Excavating and Backfilling Structures) to check that softened/loosened soils or other unsuitable material have been removed. 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. A Non-Standard Special Provision 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.



6.7.2 Geotechnical Resistance/Reaction

The bearing of the segmental blocks can be designed using a factored geotechnical resistances of 200 kPa at Ultimate Limit States (ULS) and a geotechnical resistance of 180 kPa at Serviceability Limit States (SLS, for 25 mm of settlement).

6.7.3 Stability

The stability of the proposed RSS wall located at the southeast corner of the southeast abutment was assessed based on the preliminary General Arrangement drawing provided by Dillon (dated October 1014) which shows a slope angle of 2H:1V in front of the wall and a maximum height of about 4 m. The results of the stability analyses indicate that the global stability of the proposed RSS wall should have an adequate minimum factor of safety of at least 1.3 under static conditions. Seismic (pseudo-static) global stability analyses of the proposed RSS wall was also carried out and the results indicate that the RSS wall will have an acceptable factor of safety of at least 1.1 against global failure under seismic conditions.

6.8 Seismic Considerations

The site is located between Brockville and Gananoque, Ontario and according to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio, A , applicable to this site is 0.15.

Below the groundwater table, the soil at the site generally consists of up to about 5 m of loose to dense embankment fill, organic silt / peat, and loose to compact layers of sand and silt overlying granitic bedrock. The SPT “N” values obtained in the submerged embankment fill ranged from 6 to greater than 100 blows per 0.3 m penetration. The SPT “N” values obtained in the sand and silt layers ranged from 4 to 35 blows per 0.3 m penetration.

The Site Coefficient, S , for this site in accordance with Section 4.4.6 of the CHBDC may be taken as 1, consistent with Soil Profile Type I.

A liquefaction assessment was carried out using the subsurface information collected at the site. The methodology used for the assessment is consistent with those outlined in Section C4.6.2 of the *Commentary* to the CHBDC and state-of-practice techniques. The assessment involved comparing the cyclic shear stresses applied to the soil by the design ground motions outlined in the CHBDC, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

The CRR was calculated using the parameter, $(N_1)_{60cs}$, that is based on the SPT “N” blow counts obtained in the field at the boreholes put down as part of the 2014 investigation and corrected for overburden stress, rod length during sampling, hammer energy efficiencies, and fines content.

The results of the assessment indicated a low susceptibility to liquefaction. The soil may be considered non-liquefiable for design purposes.

6.9 Lateral Earth Pressures for Design

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

The following recommendations are made concerning the design of the walls:



- Select free-draining granular fill meeting the specifications of OPSS 1010 Granular A, Granular B Type II or uniform sand 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. Guidance on this topic is provided in MTO's "Backfill to Structures Adjacent to Rock Embankment Approaches", included in Appendix C.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost tapers should be in accordance with OPSD 3101.150, 3190.100, and 3121.150.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Compaction equipment should be used in accordance with MTO's Special Provision 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 width equal to at least 1.5 m behind the back of the abutment stem (Case (a) on Figure C6.20 of the *Commentary* to the CHBDC) or within the wedge-shaped zone defined by a line drawn at 1.5H:1V extending up and back from the rear face of the footing or pile cap (Case (b) on Figure C6.20 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.

6.9.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not seismic) loading conditions. These lateral earth pressures assume that the ground above the wall will be flat, not sloping. If the inclination of the slope above the wall changes then new lateral earth pressures will need to be calculated.

- For Case (a), the pressures are based on the existing approach embankment fill and the following parameters (unfactored) may be used:

Material	Existing Embankment Fill
Soil Unit Weight:	21 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50
Passive, K_p	3.0

- For Cases (b or c), the pressures are based on using engineered granular fill and the following parameters (unfactored) may be used:

Material	Granular A / Granular B Type II	Uniform Sand Fill
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Material	Granular A / Granular B Type II	Uniform Sand Fill
Soil Unit Weight:	22 kN/m ³	18 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47
Passive, K_p	3.7	3.3

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:
 - Rotation of approximately 0.002 about the base of a vertical wall (where the rotation is calculated as the horizontal displacement divided by the height of the wall);
 - Horizontal translation of 0.001 times the height of the wall; or,
 - A combination of both.
- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.
- Where movements are not sufficient to mobilize the full passive resistance, K_p may be determined in accordance with Figure C6.16 of the *Commentary* to the CHBDC based on the amount of displacement.
- Friction between the back of the walls supported on piles and the backfill soil can be taken as 24 degrees for a smooth wall and 28 degrees for a roughened wall.

6.9.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must be taken into account in the design in accordance with Section 4.6.4 of the CHBDC. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the wall. The wall should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given in Section 6.9.1, above, plus the earthquake-induced dynamic earth pressure. The site-specific zonal acceleration ratio (A) for the site is 0.15. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of $A = 0.15$.
- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its *Commentary*, for structures which do not allow lateral yielding, the horizontal seismic coefficient (k_h) used in the calculation of the seismic active pressure coefficient is taken as 1.5 times the zonal acceleration ratio (i.e., $k_h = 0.23$). For structures which allow lateral yielding, (k_h) is taken as 0.5 times the zonal acceleration ratio (i.e., $k_h = 0.08$).



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

Seismic Active Pressure Coefficients, K_{AE}

Material	Case (a)	Case (b or c)	
	Existing Embankment Fill	Granular A / Granular B Type II	Sand Fill
Yielding wall	0.39	0.32	0.36
Non-yielding wall	0.53	0.44	0.49

- 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 0.15. This corresponds to displacements of up to approximately 50 mm at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

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

- Where:
- $\sigma_h(d)$ is the (static plus seismic) lateral earth pressure at depth, d, (kPa);
 - K is the static active earth pressure coefficient, K_a (**to be used for yielding walls**);
 - K is the static at-rest earth pressure coefficient, K_o (**to be used for non-yielding walls**);
 - K_{AE} is the seismic active earth pressure coefficient;
 - γ is the unit weight of the backfill soil (kN/m^3), as given previously;
 - d is the depth below the top of the wall (m); and,
 - H is the total height of the wall (m).

6.10 Approach Embankments

Based on discussion with Dillon, it is understood that the construction staging will first include removal of the existing north (trail) bridge to allow for construction of a temporary highway bridge across the northern portion of the causeway. With traffic detoured on to the temporary bridge, the existing south (highway) bridge abutments and structure will be removed. The new south (highway) bridge will then be constructed along the same alignment as the existing.

It is understood that grade raises on the order of 300 to 500 mm are being proposed for the approach embankments to the bridges. A portion of the existing embankments will be excavated to allow for removal of the existing abutment foundations and construction of the new abutments. Based on the results from the boreholes drilled through the existing embankments, the road structure is generally underlain by embankment fill consisting of sand, gravel, cobbles, and boulders.



Even if current grades are maintained, replacement of the existing embankment fill with fill of greater density than that currently in place could produce settlement in the organic silt and peat layer that would produce settlement of the embankments. The magnitude and rate of settlement would depend on the increase in load imposed by the new fill. These settlements would occur over time following construction of the new embankments and could generate downdrag loads on deep foundations (as described in Section 6.4.3).

6.10.1 General Embankment Construction

It is recommended that any topsoil/organic material or existing loose surficial fill encountered during excavation of the existing embankments be removed prior to placement of embankment fill. The topsoil/organic material or loose fill should be stripped to underlying competent compact to dense granular fill.

The new embankment fill associated with the re-construction of the approach embankments should be placed and compacted in accordance with OPSS.PROV 206 (*Earth Excavation and Grading*) and OPSS 501 (*Compacting*). If additional fill is to be placed on the embankment side slopes, benching of the existing side slopes should be carried out to “key in” the new fill materials, in areas where additional fill is to be placed on the existing embankment side slopes, such as in over steepened areas, in accordance with OPSD 208.010 (*Benching of Earth Slopes*).

6.10.2 Embankment Stability

Static and seismic (pseudo-static) slope stability analyses of the existing embankments 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. The analyses were based on the existing surveyed topographic information provided by the design team and the available subsurface information.

The soil and bedrock stratigraphy between the borehole locations is based on our interpretation of the geological conditions of the area and consequently the actual conditions may vary from that used in our model. The soil parameters used in the analyses were based on in situ and laboratory testing data as well as published correlations and are given in the following table:

Soil Stratum	Bulk Unit Weight (kN/m³)	Friction Angle
Surficial Fill (Pavement Structure)	21	31°
Very dense sand, gravel, cobbles, and boulders (Embankment Fill)	22	36°
Compact to very dense sand and gravel (Embankment Fill)	21	34°
Organic Silt / Peat	16	26°
Sand to Silty Sand	18	30°

A minimum factor of safety of 1.3 is considered acceptable against static, deep-seated embankment instability. The stability of the embankments was also evaluated under seismic loading conditions. The minimum factor of safety value that is typically required against instability under seismic (pseudo-static) conditions is 1.1. The horizontal ground acceleration considered in the pseudo-static slope stability analyses was based on the peak horizontal ground acceleration of 0.15 g, as specified in the CHBDC (see Section 6.8).



Analyses were carried out to assess the stability of the existing embankment slopes in two locations considered to represent the most critical slope configurations: the portion of the downstream side of the east approach embankment within about 30 m of the bridge; and, the portion of the upstream side of the west approach embankment within about 30 m of the bridge. These locations were considered to be the critical portions of the existing embankments based on conditions observed during site visits and topographic survey information provided by Dillon.

6.10.2.1 West Embankment, Upstream Side (Station 19+115)

The results of the stability analyses of the upstream side of the west approach embankment indicate that the existing slope above the creek level is marginally stable (Figure 1) for surficial failure surfaces (i.e., deep seated failure surfaces were found to be stable). Based on discussion with Dillon, it is understood that flattening of this slope by removal of existing fill near the crest of the slope is considered to be feasible once the final road and trail bridges are constructed. The results of the stability analyses indicate that the existing slope would have to be flattened to a final slope angle of about 2H:1V to provide a factor of safety of greater than 1.3 (Figure 2).

Seismic (pseudo-static) slope stability analyses were carried out considering a flattened side slope at the final slope angle of 2H:1V. The results of the analyses indicate that a 2H:1V embankment will have an acceptable factor of safety of at least 1.1 against deep-seated rotational instability under seismic conditions.

6.10.2.2 East Embankment, Downstream Side (Station 19+185)

The results of the stability analyses of the downstream side of the east approach embankment indicate that the existing slope above the creek level is marginally stable (i.e., factor of safety against failure of about 1.0 (Figure 3) as expected based on observations of historical surficial sloughing (i.e. deep seated failure surfaces were found to be stable). Flattening of this slope by removal of existing fill near the crest of the slope (i.e., similar to the west embankment on the upstream side described above) would provide a suitable factor of safety against instability and could be considered; however, this would require shifting the plan alignment of the highway and it is understood that this may not be practical as the road bridge is currently planned to be constructed in the about the same location as the existing bridge. As an alternative, re-grading of the embankment by placement of additional rock fill into the creek channel could be considered to improve the stability.

The results of the stability analyses indicate that a final slope angle of about 2H:1V would be required to provide a factor of safety of greater than 1.3 (Figure 4). If this option is considered, embankment widening would need to be carried out by placement of rock fill, including placement in the water within the creek channel. Benching of the existing embankment side slopes should be carried out to “key in” the new fill materials for the widening, as described in Section 6.10.1.

Seismic (pseudo-static) slope stability analyses carried out considering the placement of additional fill on the downstream side of the east approach embankment to achieve a final slope angle of 2H:1V indicate that the embankment will have an acceptable factor of safety of at least 1.1 against deep-seated rotational instability under seismic conditions.

6.10.2.3 Abutment Slopes, Beneath Bridges

The stability of the proposed abutment slopes beneath the bridge structures was assessed based on the preliminary General Arrangement drawing provided by Dillon (dated October 1014) that shows a slope angle of



3H:1V in front of the abutments. The results of the stability analyses indicate that the proposed slopes should have an adequate minimum factor of safety of at least 1.3 under static conditions (Figure 5).

Seismic (pseudo-static) slope stability analyses of the proposed abutment slopes beneath the bridge structures were also carried out and the results indicate that the proposed abutment slopes beneath the bridges will have an acceptable factor of safety of at least 1.1 against deep-seated rotational instability under seismic conditions.

6.10.3 Settlement

The results of the foundation investigation indicate that the approach embankments are constructed out of granular fill that is underlain by a discontinuous deposit of compressible organic silt and peat. Settlement of the existing embankments due to compression of underlying organic soils and creep of the organic materials and approach embankment fill has likely occurred over time since the original bridge construction.

The original highway grades shown on the 1939 construction drawings were at about Elevation 80.4 m. The surveyed highway grade at the boreholes put down in 2014 along the existing south (highway) bridge were at about Elevation 80.3 m, suggesting that post-construction settlement of the existing bridge structure founded on spread footings above the organic silt and peat deposit may be on the order of at least 0.1 m (however this would not include any previous padding). Furthermore, given that the existing embankments are understood to have been in place for a period approaching 75 years and that the organic soils are relatively thin, it is expected that the magnitude of ongoing creep of the existing embankment will be nominal.

Additional loading imposed by placement of the currently planned 300 to 500 mm increase in the embankment grades and/or replacement of a portion of the existing embankments with fill of greater density would result in further consolidation settlement on the order of 0 to 50 mm of the organic silt/peat layer present beneath the embankments. In addition, a portion of the existing embankment fill will need to be excavated and replaced with new backfill behind the new bridge abutments. As described previously in Sections 6.4.3 and 6.10, consideration should be given to selecting a backfill that has a density that is equal to or less than the existing fill materials to reduce the amount of surcharge imposed on the underlying peat/organics.

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.

If a grade raise is required for any portion of the embankment, consideration could be given to preloading/surcharging any areas of grade raise during the construction period in order to minimize post-construction settlement associated with compression of the organic soil strata and delaying paving the final lift of asphalt.



Alternatively, consideration may be given to the use of lightweight fill materials such as EPS Geofoam, tire-derived aggregate, or blast furnace slag for embankment re-construction to reduce or eliminate the stress increase on the compressible soils to a level that would result in settlements within acceptable tolerances. Considering a 500 mm maximum grade raise, the use of up to 600 mm of EPS lightweight fill would lead to tolerable embankment settlements and no downdrag on deep foundation elements. Further details for this option have been provided in a separate technical memorandum to Dillon. A NSSP 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.

6.11 Construction Considerations

The following sections identify future construction considerations that should be considered at this stage as they may impact the future design and/or require special consideration or non-standard special provisions during construction.

6.11.1 Excavation and Temporary Protection Systems

The excavations for pile caps would extend a minimum of 1.5 m deep (for frost protection purposes) into the existing grade fill at the abutment locations. The pile caps should be constructed at a suitable depth to maintain the required frost protection depth below the road grade as well as any adjacent side slopes. The excavations would be developed through the existing approach embankment fill consisting of very loose to very dense sand, gravel, cobbles, and boulders.

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.

Depending on the separation distances between the new structures, temporary protection systems may be required adjacent to the active and/or detour highway lanes to permit the proposed staged construction of the temporary bridge, the highway bridge, and the pedestrian bridge. This support system should be designed and constructed in accordance with OPSS 539 (*Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539.

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., if 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 a soldier pile and timber lagging system would be suitable for the temporary excavation support at this site, based on the subsurface soil and groundwater conditions. The soldier piles would have to be socketted to sufficient depth to provide the necessary passive resistance for the retained soil height. Pre-drilling and the use of temporary liners may be required for the soldier pile installation. Measures would need to be implemented to mitigate the potential for ground movements behind the support system as a result of displacement of cobbles and boulders during lagging installation. It would also be necessary to control seepage or include measures to mitigate loss of soil particles through the lagging boards.
- Lateral support to the soldier piles could be provided in the form of rakers or temporary anchors.

6.11.2 Groundwater and Surface Water Control

The water level within the approach embankments is expected to be controlled by the water level within the creek. The groundwater level observed in the well installed in Borehole 14-1 was most recently measured on August 10, 2014 at Elevation 74.7 m. The high water level at the site is Elevation 76.5 m, as shown on the General Arrangement drawing provided by Dillon.

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. It is understood that the current design does not include any significant excavations extending below the anticipated water level. However, based on the subsurface soil conditions, it is anticipated that the dewatering rate for any excavations extending below the water level will exceed 50 m³/day, and therefore it would be prudent to obtain a Permit to Take Water (PTTW) for this site in the event of water level fluctuations.

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

Given that only a minor lowering of the water level is anticipated, it is expected that the zone of influence for the dewatering operations would be relatively localized at the structure site. Assuming the dewatering system is properly constructed and operated such that there is no loss of fine soil particles, the dewatering operations are not expected to cause excessive settlement in the embankment fill materials at the site.

Running or flowing of water-bearing cohesionless soil strata could occur during or after pre-augering for pile foundations (if adopted), and basal heave could occur. It is recommended that temporary liners be used during pre-augering to support the side walls of the auger hole and minimize loss of ground.

6.11.3 Obstructions

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

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

The presence of cobbles and boulders in the embankment fill could affect the installation of deep foundations or protection system elements. If driven steel piles are used, pre-augering of the pile locations should be completed to about Elevation 68 m.



6.11.4 Erosion and Scour Protection

As described above, there is evidence of erosion of the existing granular fill materials present immediately in front of the existing bridge abutments. Therefore, the existing granular fill materials that make up the approach embankments at the site are considered 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.

6.11.5 Vibration Monitoring During Pile Driving

The proposed staged bridge construction is to include construction of a temporary traffic/pedestrian bridge while the existing highway bridge structure remains in service, and construction of the new trail bridge following completion of the new highway bridge. It is recommended that vibration monitoring be carried out during installation of piles or driven protection systems to assist in maintaining vibration levels within tolerable ranges for the portions of the bridge in service at the time, or for any temporary modular structure if used at the site. A sample NSSP for vibration monitoring is provided in Appendix C.

A maximum peak particle velocity of 100 mm/sec is recommended at the existing abutments. The piles furthest from the existing structure should be driven first, in order to check the vibration level at the existing structure and, if necessary, alter the installation procedures for the remaining piles.

6.11.6 Ground/Groundwater Control for Deep Foundation Installation

Where pre-augering is required for steel pile installation, the use of temporary or permanent liners will be required to minimize loss of ground through the water-bearing portions of the embankment fill present below the water table. A sample NSSP for groundwater control is provided in Appendix C.



FOUNDATION REPORT

SITE NOS. 16-110/1 AND 16-110/2 – JONES CREEK HIGHWAY AND TRAIL BRIDGES

7.0 CLOSURE

This Foundation Design Report was prepared by Mrs. Kim Lesage, P.Eng., and reviewed by Mr. Michael Snow, P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Fin Heffernan, P.Eng., The Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.

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FOUNDATION REPORT
SITE NOS. 16-110/1 AND 16-110/2 – JONES CREEK HIGHWAY AND TRAIL BRIDGES

Table 1
Comparison of Foundation Alternatives
W.P. 4092-10-02 (Highway Bridge) and 4270-11-01 (Trail Bridge) – Thousand Island Parkway

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-piles driven to bedrock	<ul style="list-style-type: none"> Feasible for support of bridge replacement Preferred option from a foundations perspective 	<ul style="list-style-type: none"> High geotechnical resistances and negligible settlement Preferred foundation option for integral abutment construction 	<ul style="list-style-type: none"> Potential for encountering obstructions (cobbles and/or boulders) during pile driving that could result in some piles “hanging up”, being damaged or deflecting during driving through the embankment fill Temporary excavation support may be required to facilitate removal of the existing abutments Excavation or augering below water table in granular soils required for CSP installation if integral abutments are to be used 	<ul style="list-style-type: none"> Moderate cost 	<ul style="list-style-type: none"> Low risk of driven H-piles “hanging up” in embankment fill Provision for pre-augering to Elevation 68 m prior to pile installation
Steel pipe (tube) piles, driven to found on bedrock	<ul style="list-style-type: none"> Feasible for support of bridge replacement 	<ul style="list-style-type: none"> High geotechnical resistances and negligible settlement 	<ul style="list-style-type: none"> Slightly greater risk than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving; this could result in more piles “hanging up” Temporary excavation support may be required to facilitate removal of the existing abutments 	<ul style="list-style-type: none"> Moderate cost 	<ul style="list-style-type: none"> Moderate risk of pipe piles “hanging up” in embankment fill Provision for pre-augering to Elevation 68 m prior to pile installation
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 end-bearing resistance. Does not allow for use of integral abutments. 	<ul style="list-style-type: none"> Moderate to expensive cost. 	<ul style="list-style-type: none"> Medium risk option.



FOUNDATION REPORT
SITE NOS. 16-110/1 AND 16-110/2 – JONES CREEK HIGHWAY AND TRAIL BRIDGES

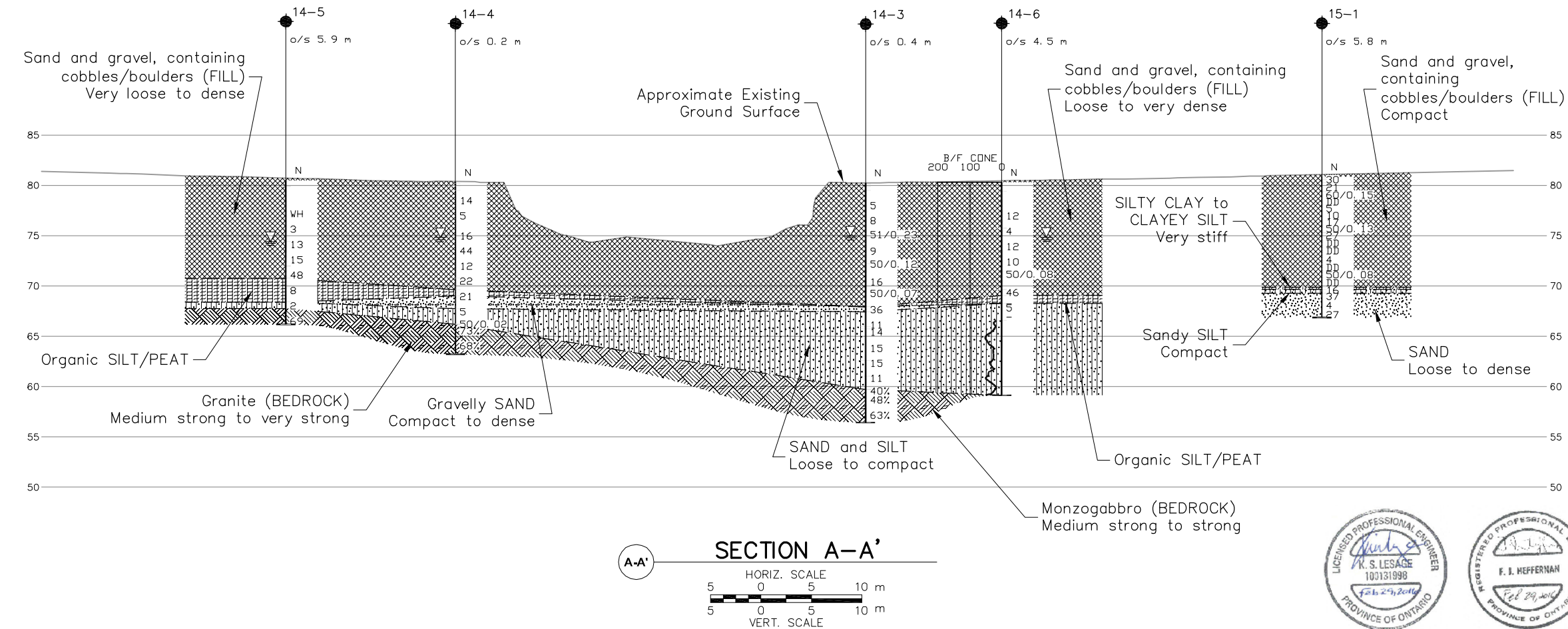
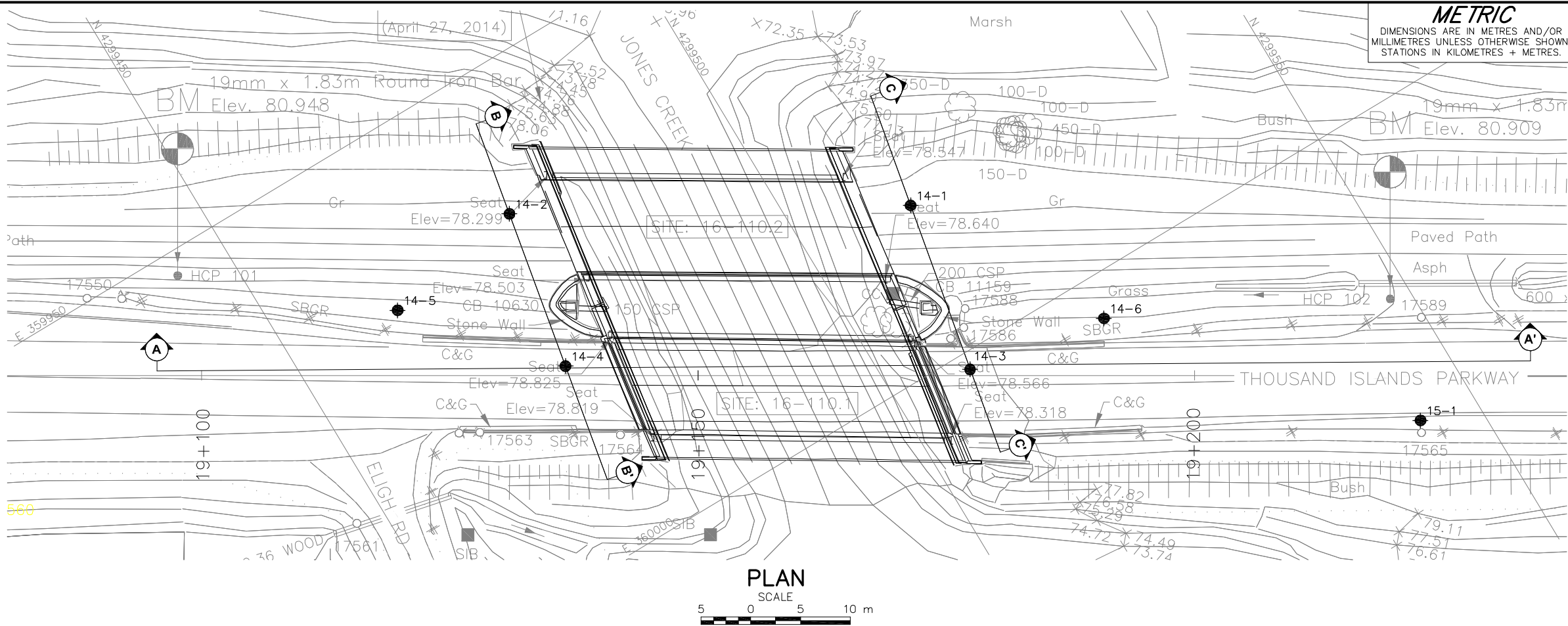
Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Caissons founded on bedrock	<ul style="list-style-type: none"> Feasible but not practical or economical for support of bridge replacement due to expected significant construction related difficulties 	<ul style="list-style-type: none"> Higher capacity than for steel H-piles or pipe piles, so reduced number of deep foundation elements compared to steel piles 	<ul style="list-style-type: none"> Significant caisson length required (at least 20 m at east abutment) Temporary or permanent liners required to control ground and groundwater in water-bearing deposits below the creek level Rock coring, churn drilling or chisel drilling required to form rock sockets in strong to very strong bedrock Running of sand and/or silt in to caisson if liner is not suitably sealed on sloping bedrock Precludes use of integral abutments 	<ul style="list-style-type: none"> Construction of deep caissons more expensive than alternative foundation options 	<ul style="list-style-type: none"> Difficulties advancing liners through embankment fill and maintaining a seal at bedrock surface. Significant length required would result in high foundation construction cost
Spread footings founded within existing embankments	<ul style="list-style-type: none"> Not feasible 		<ul style="list-style-type: none"> Potential for excessive total and differential settlements of the organic silt and peat deposits that underlie the existing embankments 	<ul style="list-style-type: none"> Moderate cost 	<ul style="list-style-type: none"> Significant excavation required to remove or remediate compressible organic deposits



FOUNDATION REPORT
SITE NOS. 16-110/1 AND 16-110/2 – JONES CREEK HIGHWAY AND TRAIL BRIDGES

Table 2
Comparison of EPS Lightweight Fill Alternatives for Settlement Mitigation
W.P. 4092-10-02 (Highway Bridge) and 4270-11-01 (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 rideability 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"> 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

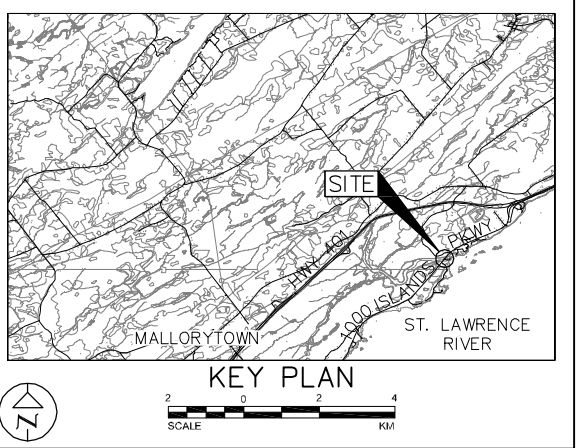


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

HWY T.I.P.
CONT No. 2016-4018
WP No. 4092-10-02 HWY
4270-11-01 TRAIL

SHEET
301

Golder Associates Ltd.
OTTAWA ONTARIO, CANADA



- LEGEND**
- Borehole - Current Investigation
 - N Standard Penetration Test Value
 - 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
 - 100% Rock Quality Designation (RQD)
 - WL in piezometer, measured on August 10, 2014
 - WL upon completion of drilling
 - Seal
 - Piezometer

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
15-1	80.9	4929544.2	360029.5
14-1	80.1	4929511.4	359984.5
14-2	80.0	4929476.3	359964.3
14-3	80.3	4929508.0	360001.7
14-4	80.3	4929473.3	359980.4
14-5	80.5	4929461.7	359966.9
14-6	80.3	4929522.2	360004.3

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. B Plan.dwg, August 7, 2014.








NO.	DATE	BY	REVISION
Geocres No. 31B-87			
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



Golder Associates Ltd.
OTTAWA ONTARIO, CANADA



	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
	WL in piezometer, measured on August 10, 2014
	WL upon completion of drilling
	Seal
	Piezometer

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.

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Base plan provided in digital format by Dillon, drawing file no. B Plan.dwg,
August 7, 2014.

NO.	DATE	BY	REVISION		
Geocres No. 31B-87					
Hwy. 1000 Islands Pkwy	PROJECT NO. 12-1121-0193			DIST.	
SUBM'D. KSL	CHKD. KSL			DATE: 2/29/2016	
DRAWN: JM	CHKD. FJH			APPD. FJH DWG. 3	



HORIZ. SCALE

5 0 5 10 m

5 0 5 10 m

VERT. SCALE



HORIZ. SCALE

5 0 5 10 m

5 0 5 10 m

VERT. SCALE



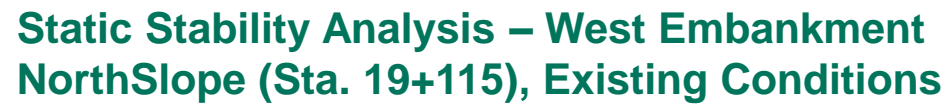
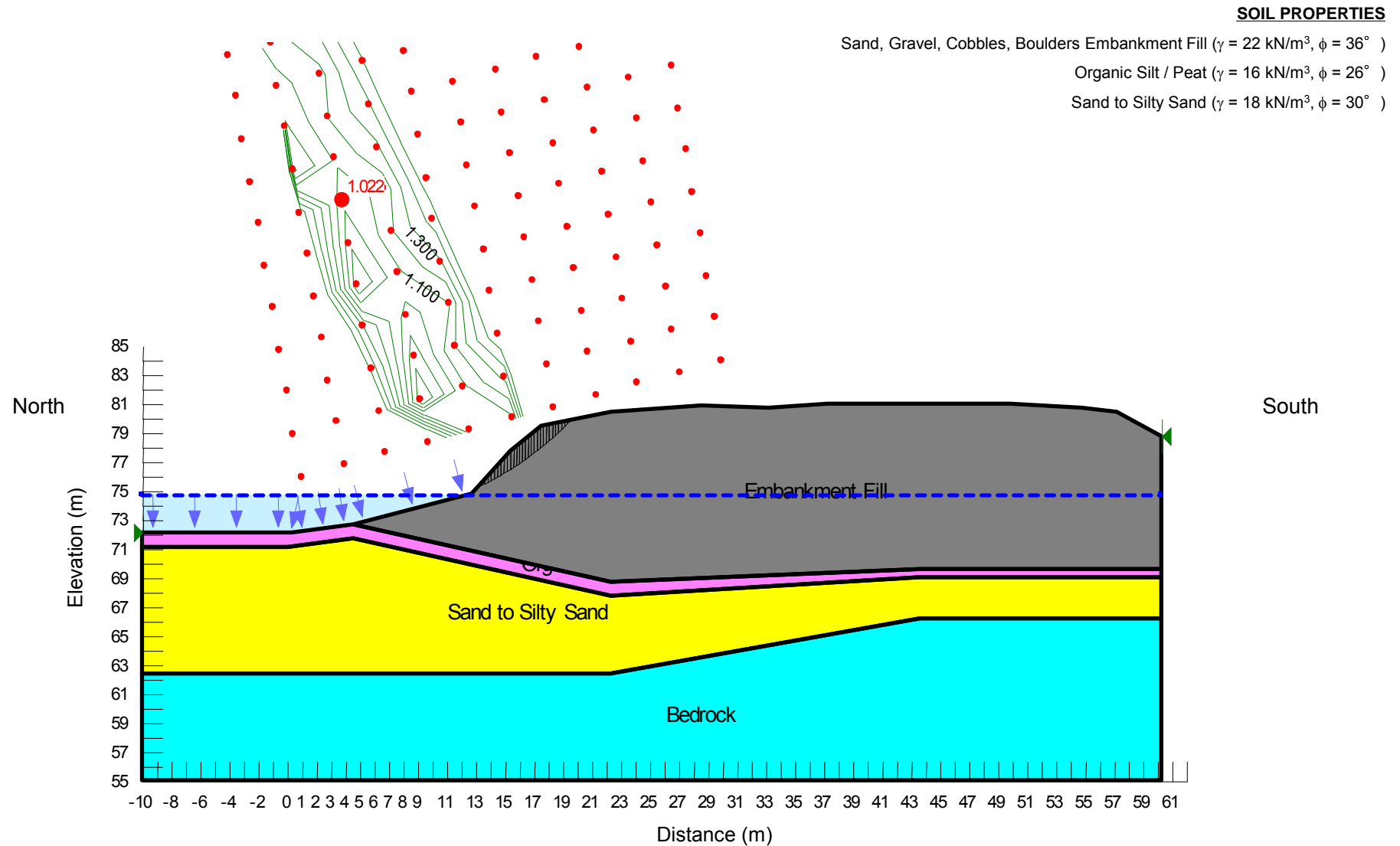


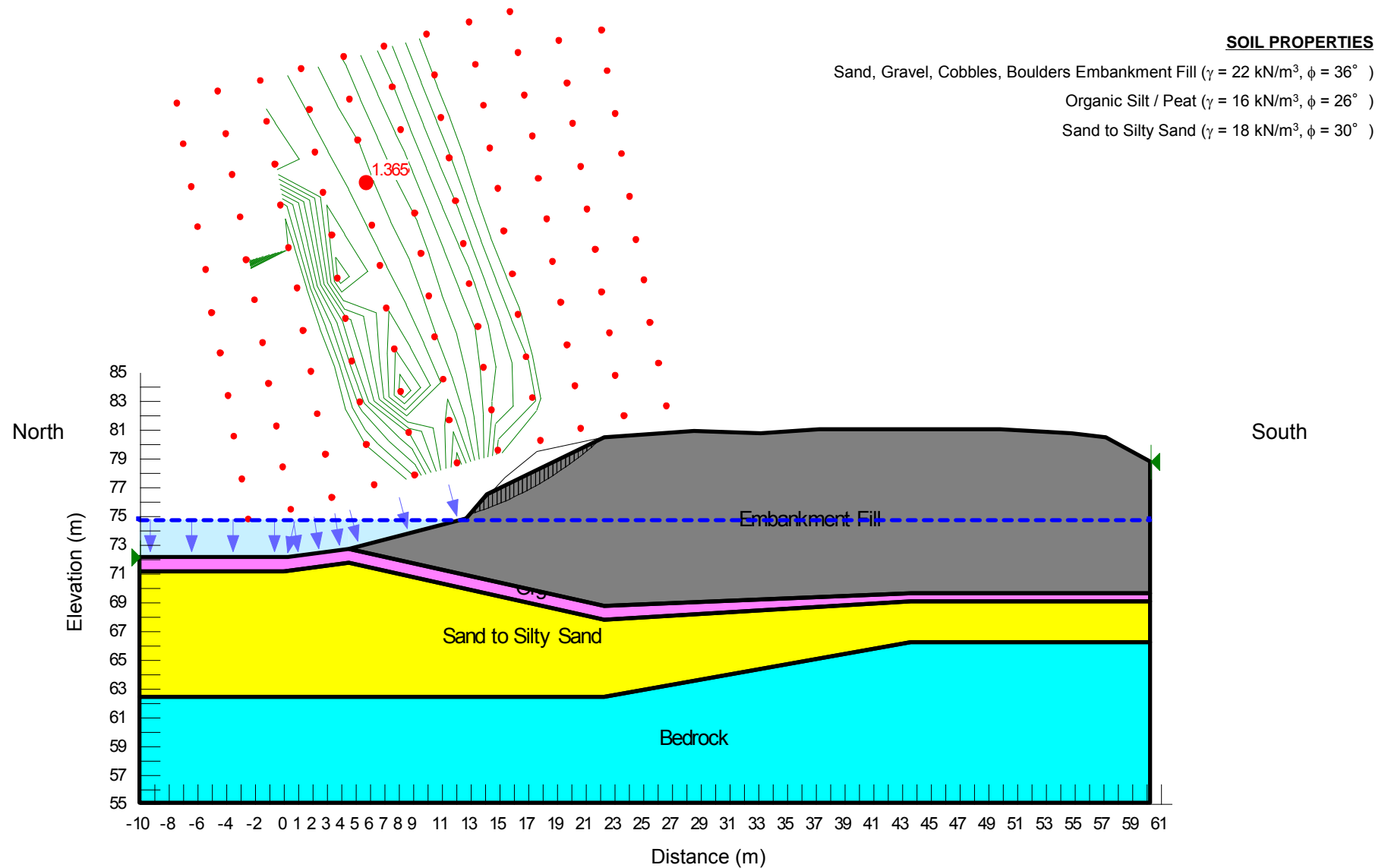
Figure 1





Static Stability Analysis – West Embankment North Slope (Sta. 19+115), 2H:1V Flattened Slope

Figure 2



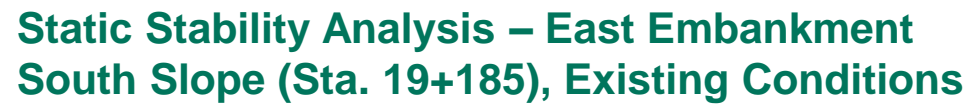
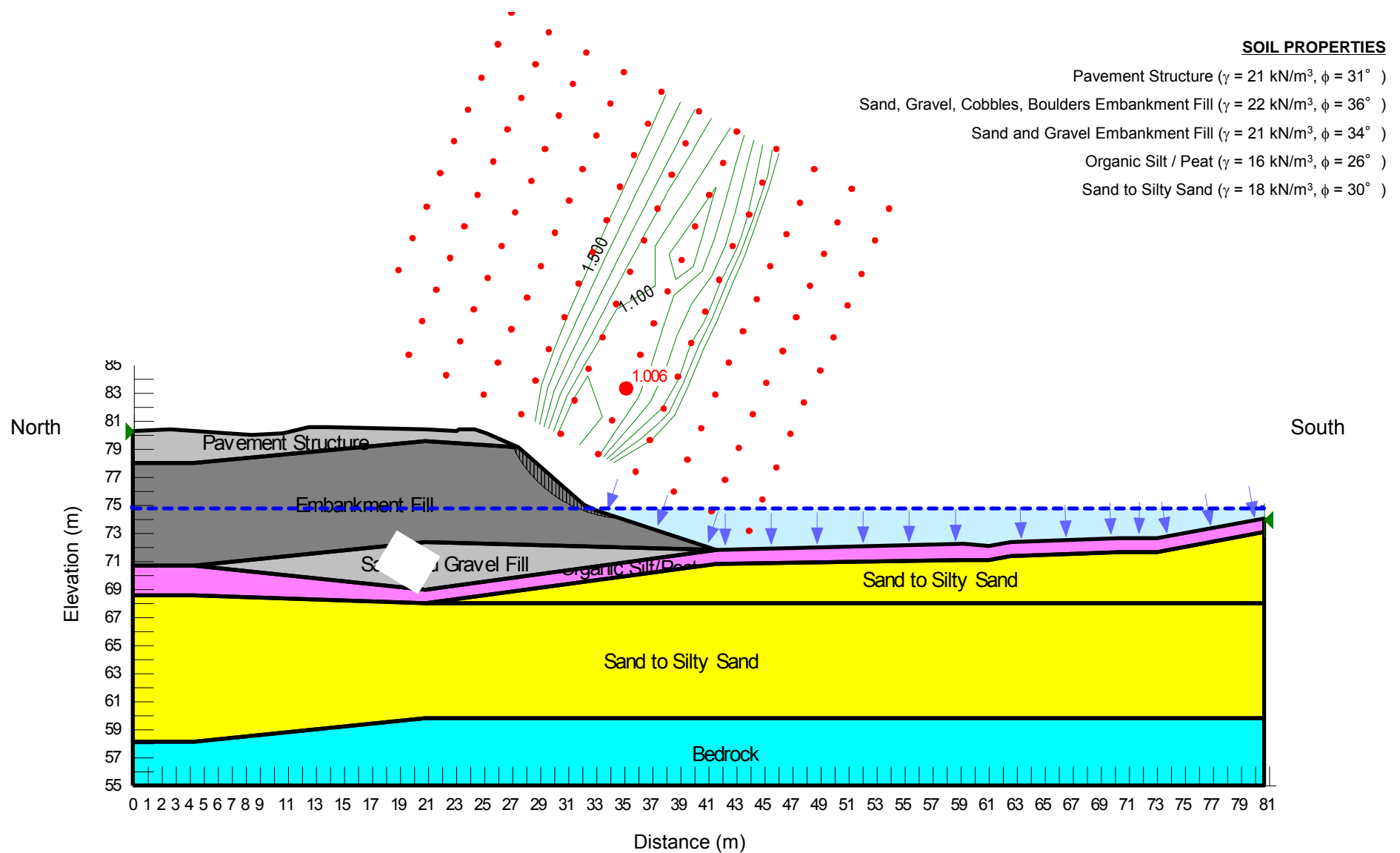


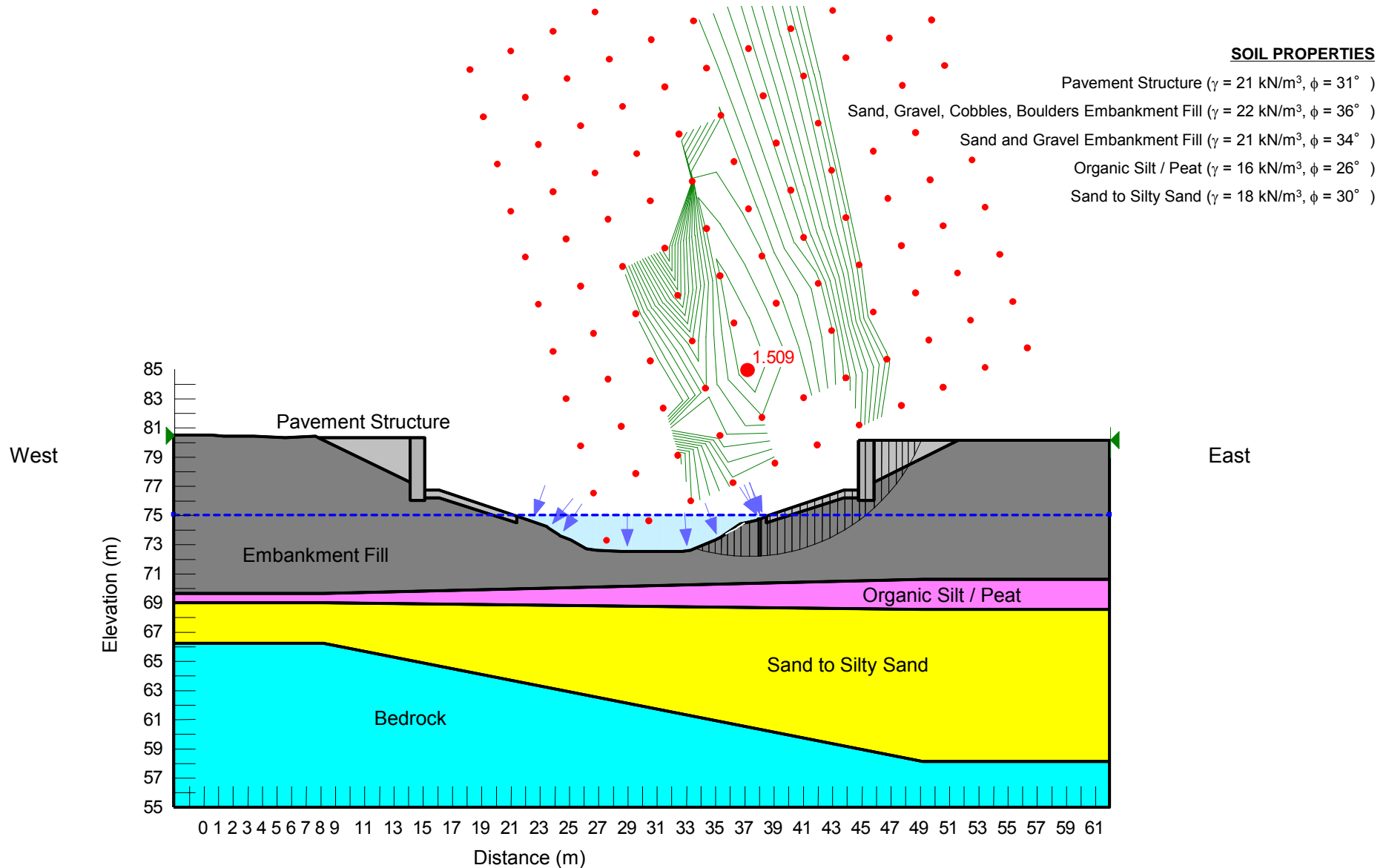
Figure 3





Static Stability Analysis – Abutment Slope East Abutment Slope, Beneath Bridges

Figure 5





APPENDIX A

Records Borehole and Drillhole Sheets

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			<u>Blows/300 mm</u>
DO or DP	Seamless open-ended, driven or pushed tube samplers			<u>Or Blows/ft.</u>
DS	Denison type sample		Very loose	0 to 4
FS	Foil sample		Loose	4 to 10
RC	Rock core		Compact	10 to 30
SC	Soil core		Dense	30 to 50
SS	Split spoon sampler		Very dense	over 50
ST	Slotted tube	(b) Cohesive Soils		
TO	Thin-walled, open	Consistency		C_u or S_u
TP	Thin-walled, piston			
WS	Wash sample		<u>kPa</u>	<u>Psf</u>
DT	Dual tube sample		Very soft	0 to 12
DD	Diamond drilling		Soft	12 to 25
			Firm	25 to 50
			Stiff	50 to 100
			Very stiff	100 to 200
			Hard	Over 200
				Over 4,000
II. PENETRATION RESISTANCE		IV. SOIL TESTS		
Standard Penetration Resistance (SPT), N:		w	Water content	
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split spoon sampler for a distance of 300 mm (12 in.).		w _p or PL	Plastic limited	
Dynamic Cone Penetration Resistance (DCPT); N_d:		w _l or LL	Liquid limit	
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° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).		C	Consolidation (oedometer) test	
PH: Sampler advanced by hydraulic pressure		CHEM	Chemical analysis (refer to text)	
PM: Sampler advanced by manual pressure		CID	Consolidated isotropically drained triaxial test ¹	
WH: Sampler advanced by static weight of hammer		CIU	Consolidated isotropically undrained triaxial test with porewater pressure measurement ¹	
WR: Sampler advanced by weight of sampler and rod		D _R	Relative density	
Cone Penetration Test (CPT):		DS	Direct shear test	
An electronic cone penetrometer with a 60° 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.		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

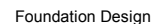
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

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		



SHEET 1 OF 4

METRIC

ORIGINATED BY KE

COMPILED BY JM

CHECKED BY MSS

GTA-MTO 001 N:ACTIVE20121121 - GEOTECHNICAL12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IMG\INT\PHASE_132111211210193-1321.GPJ GAL-GTA.GDT 02/25/16 JM

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

PROJECT 12-1121-0193-1321		RECORD OF BOREHOLE No 14-1		SHEET 2 OF 4		METRIC									
G.W.P. 4092-10-01 & 4270-11-01		LOCATION N 4929511.4 ; E 359984.5		ORIGINATED BY KE											
DIST _____ HWY 1000 Islands Pkwy		BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core		COMPILED BY JM											
DATUM Geodetic		DATE July 17-18, 2014		CHECKED BY MSS											
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa							
--- CONTINUED FROM PREVIOUS PAGE ---															
69.9 10.2	Organic SILT/PEAT, some gravel, with sandy silt, containing cobbles and boulders Dark brown to black Wet		9	SS	33										
68.5 11.6	SAND, some silt, with sandy silt layers Loose to dense Grey Wet		10	SS	4										
			11	SS	35										
65.0 15.1	SAND, some silt, trace gravel, containing cobbles and boulders Compact Grey Wet		12	SS	29										
63.3 16.8	SILT and SAND, trace gravel Dense Grey Wet		13	SS	32										
62.0 18.1	Silty SAND, some gravel Compact Grey Wet		14	SS	28										

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_IMG\INTPHASE 1321\11210193-1321.GPJ GAL-GTA.GDT 02/25/16 JM

PROJECT 12-1121-0193-1321		RECORD OF BOREHOLE No 14-1				SHEET 3 OF 4		METRIC									
G.W.P. 4092-10-01 & 4270-11-01		LOCATION N 4929511.4 ; E 359984.5				ORIGINATED BY KE											
DIST _____ HWY 1000 Islands Pkwy		BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core				COMPILED BY JM											
DATUM Geodetic		DATE July 17-18, 2014				CHECKED BY MSS											
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					
58.8	Silty SAND, some gravel Compact Grey Wet						60										
21.3	SAND, some silt Dense Grey Wet		15	SS	31		59										1 82 16 1
58.1	Monzogabbro (BEDROCK)						58										
22.0	Bedrock cored from depths of 22.0 m to 25.0 m For bedrock coring details refer to Record of Drillhole 14-1		1	RC	REC 100%		57										RQD = 43%
			2	RC	REC 100%		56										RQD = 29%
55.1	END OF BOREHOLE																
25.0	NOTES: 1. Water level in well at a depth of 5.4 m below ground surface (Elev. 74.7 m), measured on August 10, 2014.																

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IM\GINTPHASE 1321\1121\10193-1321.GPJ GAL-GTA.GDT 02/25/16 JM

PROJECT: 12-1121-0193-1321

RECORD OF DRILLHOLE: 14-1

SHEET 4 OF 4

LOCATION: N 4929511.4 ;E 359984.5

DRILLING DATE: July 17-18, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Downing Drilling

GTA-RCK 031 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IMG\INT\PHASE 1321\1211210193-1321.GPJ GAL-MISS GDT 02/25/16 JM

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
						FLUSH RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t CORE AXIS °	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec		WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
							TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr	J8	10 ⁻³ 10 ⁻⁴ 10 ⁻⁵ 10 ⁻⁶	W1 W2 W3 W4 W5 W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
22	Rotary Drill NQ Core	BEDROCK SURFACE		58.11																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															</

UCS =
72.4 MPa


DEPTH SCALE

1 : 50



LOGGED: KE

CHECKED: MSS

PROJECT		12-1121-0193-1321		RECORD OF BOREHOLE No 14-2		SHEET 1 OF 2		METRIC									
G.W.P.		4092-10-01 & 4270-11-01		LOCATION		N 4929476.3 ; E 359964.3		ORIGINATED BY									
DIST		HWY 1000 Islands Pkwy		BOREHOLE TYPE		Geoprobe 127 mm Diam.		COMPILED BY									
DATUM		Geodetic		DATE		July 2-3, 2014		CHECKED BY									
								MSS									
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									
80.0	GROUND SURFACE						20	40	60	80	100						
79.9	Sandy gravel (FILL) Grey Dry		1	GRAB	-												
	Gravel and sand, trace silt (FILL) Brown Moist																
78.5	Cobbles and boulders, some gravel, trace sand (FILL) Compact Grey-brown Moist		2	SS	11												
78.4																	
			3	SS	4												
			4	SS	15												
			5	SS	6												
72.4	Gravel, some sand, trace silt (FILL) Very dense Grey-brown Wet		6	SS	60												
72.3																	
70.9	Sandy gravel, trace silt, containing cobbles, boulders and organic matter (FILL) Compact Grey-brown Wet		7	SS	50/0.16												
70.8																	

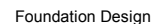
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+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IMG\INTPHASE 1321\11210193-1321.GPJ GAL-GTA.GDT 02/25/16 JM

PROJECT		12-1121-0193-1321		RECORD OF BOREHOLE No 14-2		SHEET 2 OF 2		METRIC									
G.W.P.		4092-10-01 & 4270-11-01		LOCATION		N 4929476.3 ; E 359964.3		ORIGINATED BY		KE							
DIST		HWY 1000 Islands Pkwy		BOREHOLE TYPE		Geoprobe 127 mm Diam.		COMPILED BY		JM							
DATUM		Geodetic		DATE		July 2-3, 2014		CHECKED BY		MSS							
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									WATER CONTENT (%)
--- CONTINUED FROM PREVIOUS PAGE ---								20	40	60	80	100					
67.8	Sandy gravel, trace silt, containing cobbles, boulders and organic matter (FILL) Compact Grey-brown Wet		8	SS	23												69 23 7 1
12.2	SAND and SILT, trace gravel Compact Grey Wet		9	SS	19												8 51 37 4
67.2	Sandy SILT, trace to some clay, containing organic matter Compact Grey Wet		10	GRAB	-												
12.8																	
63.2	SAND, some silt Very loose Grey Wet		12	SS	1												0 82 17 1
62.6	SAND, trace silt and sea shells, with silty clay pockets Red-grey Wet		13	MC	-												
62.4																	
17.6	END OF BOREHOLE DCPT REFUSAL																
NOTES: 1. Water level in open borehole at a depth of 5.7 m below ground surface (Elev. 74.3 m), measured during drilling. 2. Down-the-hole-hammer refusal at 17.6 m. Subsequent DCPT refusal at 17.6 m. Geoprobe advanced to refusal at 17.6 m for sample recovery.																	

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

PROJECT <u>12-1121-0193-1321</u>		RECORD OF BOREHOLE No 14-3		SHEET 2 OF 4		METRIC	
G.W.P. <u>4092-10-01 & 4270-11-01</u>		LOCATION <u>N 4929508.0 ; E 360001.7</u>		ORIGINATED BY <u>KE</u>			
DIST <u></u> HWY <u>1000 Islands Pkwy</u>		BOREHOLE TYPE <u>Wash Boring, HQ/NQ Casing</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>July 16-17, 2014</u>		CHECKED BY <u>MSS</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)			GR	SA	SI	CL	
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× REMOULDED	w _p	w		w _L					
	--- CONTINUED FROM PREVIOUS PAGE ---																			
	Sandy gravel, trace silt, containing cobbles and boulders (FILL) Compact to very dense Grey-brown Wet																			
			7	SS	50/0.07															
68.0																				
12.3	Gravelly SAND, trace silt, with organics Dense Grey-brown Wet		8	SS	36						○						30	62 8 0		
67.5																				
12.8	Silty SAND Compact Grey Wet																			

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

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PROJECT		12-1121-0193-1321		RECORD OF BOREHOLE No 14-3		SHEET 3 OF 4		METRIC							
G.W.P.		4092-10-01 & 4270-11-01		LOCATION		N 4929508.0 ; E 360001.7		ORIGINATED BY							
DIST		HWY 1000 Islands Pkwy		BOREHOLE TYPE		Wash Boring, HQ/NQ Casing		COMPILED BY							
DATUM		Geodetic		DATE		July 16-17, 2014		CHECKED BY							
MSS															
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE		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										
	--- CONTINUED FROM PREVIOUS PAGE ---														
59.7	SAND, some silt Compact Grey Wet														
20.6	Monzogabbro (BEDROCK) Bedrock cored from depths of 20.6 m to 23.8 m For bedrock coring details refer to Record of Drillhole 14-3		1	RC	REC 100%										RQD = 40%
			2	RC	REC 95%										RQD = 48%
			3	RC	REC 100%										RQD = 63%
56.5	END OF BOREHOLE														
23.8	NOTES: 1. Water level in open borehole at a depth of 5.3 m below ground surface (Elev. 75.0 m), measured during drilling.														

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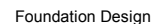
SHEET 4 OF 4

DATUM: Geodetic

DRILLING CONTRACTOR: Downing Drilling

ITA-RCK 031 N:ACTIVE2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL IMGINT\PHASE 1321\1211210193-1321.GPJ GAL-MISS.GDT 02/25/16 JM

LOGGED: KE
CHECKED: MSS



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

PROJECT		12-1121-0193-1321		RECORD OF BOREHOLE No 14-4		SHEET 2 OF 3		METRIC									
G.W.P.		4092-10-01 & 4270-11-01		LOCATION		N 4929473.3 ; E 359980.4		ORIGINATED BY		KE							
DIST		HWY 1000 Islands Pkwy		BOREHOLE TYPE		Power Auger 200 mm Diam. (Hollow Stem)/Wash Boring, HW Casing		COMPILED BY		JM							
DATUM		Geodetic		DATE		July 14-16, 2014		CHECKED BY		MSS							
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									
<div style="display: flex; justify-content: space-between;"> <div> <p>--- CONTINUED FROM PREVIOUS PAGE ---</p> </div> <div> <p>20 40 60 80 100</p> <p>○ UNCONFINED + FIELD VANE</p> <p>● QUICK TRIAXIAL × REMOULDED</p> <p>20 40 60 80 100</p> </div> <div> <p>25 50 75</p> </div> </div>																	
69.6	Silty sand and gravel, containing cobbles and boulders (FILL) Compact Dark brown Wet						70										
10.7	PEAT, trace gravel Black Wet																
69.0	Gravelly Silty SAND Compact Grey Wet		7	SS	21		69								288.8	OC = 36.1%	
11.3																	
67.7	SILT, some sand Loose Brown Wet		8	SS	5		68										
12.6							67										
66.2	Granite (BEDROCK) Completely weathered Orange-red/brown Granite (BEDROCK)		0	SS	50/0.02		66										
14.1	Bedrock cored from depths of 14.1 m to 17.1 m For bedrock coring details refer to Record of Drillhole 14-4		1	RC	REC 100%		65									RQD = 73%	
			2	RC	REC 95%		64									RQD = 68%	
63.2	END OF BOREHOLE																
17.1	NOTES: 1. Water level in open borehole at a depth of 5.5 m below ground surface (Elev. 74.8 m), measured during drilling.																

SHEET 3 OF 3

DATUM: Geodetic

DRILLING CONTRACTOR: Downing Drilling

[illegible]

DEPTH SCALE

1 : 50

LOGGED: KE

CHECKED: MSS

PROJECT <u>12-1121-0193-1321</u>		RECORD OF BOREHOLE No 14-5		SHEET 1 OF 2		METRIC	
G.W.P. <u>4092-10-01 & 4270-11-01</u>		LOCATION <u>N 4929461.7 ; E 359966.9</u>		ORIGINATED BY <u>KE</u>			
DIST <u> </u> HWY <u>1000 Islands Pkwy</u>		BOREHOLE TYPE <u>Geoprobe 127 mm Diam.</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>July 4-7, 2014</u>		CHECKED BY <u>MSS</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								<div><div><div></div><div></div><div></div><div></div><div></div></div></div>							W _p	W	W _L
80.5 0.0	GROUND SURFACE Gravelly sand (FILL) Brown Dry						20	40	60	80	100						
79.9 0.6	Silty sand and gravel, containing cobbles and boulders (FILL) Very loose Brown Dry																
			1	SS	WH												
			2	SS	3												
74.4 6.1	Sandy gravel, trace silt (FILL) Compact Brown Wet		3	SS	13												
			4	SS	15												
72.0 8.5	Silty sand and gravel (FILL) Dense Grey-brown Wet																
			5	SS	48												
70.8 9.8																	

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PROJECT <u>12-1121-0193-1321</u>				RECORD OF BOREHOLE No 14-5				SHEET 2 OF 2				METRIC					
G.W.P. <u>4092-10-01 & 4270-11-01</u>				LOCATION <u>N 4929461.7 ; E 359966.9</u>				ORIGINATED BY <u>KE</u>									
DIST <u></u> HWY <u>1000 Islands Pkwy</u>				BOREHOLE TYPE <u>Geoprobe 127 mm Diam.</u>				COMPILED BY <u>JM</u>									
DATUM <u>Geodetic</u>				DATE <u>July 4-7, 2014</u>				CHECKED BY <u>MSS</u>									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100						
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L					
							20	40	60	80	100	25	50	75			
68.4	Organic SILT/PEAT Dark brown Wet		6	SS	8												
12.1	SILT, some sand Very loose Brown Wet		7	SS	2												
67.8	BEDROCK (Completely weathered) Very dense Orange-red/brown Wet																
12.7			8	SS	69												
66.2	END OF BOREHOLE																
14.3	NOTES: 1. Water level in open borehole at a depth of 6.1 m below ground surface (Elev. 74.4 m), measured during drilling.																

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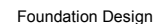
PROJECT <u>12-1121-0193-1321</u>		RECORD OF BOREHOLE No 14-6		SHEET 1 OF 3		METRIC	
G.W.P. <u>4092-10-01 & 4270-11-01</u>		LOCATION <u>N 4929522.2 ; E 360004.3</u>		ORIGINATED BY <u>KE</u>			
DIST <u></u> HWY <u>1000 Islands Pkwy</u>		BOREHOLE TYPE <u>Geoprobe 127 mm Diam.</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>July 8, 2014</u>		CHECKED BY <u>MSS</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIMIT CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	20	40	60	80	100	w _p	w		w _L			
80.3	GROUND SURFACE																			
0.0	Gravelly sand (FILL) Brown Dry																			
79.4																				
0.9	Sandy gravel, trace silt, containing cobbles and boulders (FILL) Compact Grey-brown Dry																			
75.7																				
4.6	Silty sand and gravel, containing cobbles and boulders (FILL) Loose to compact Grey-brown Moist to wet																			


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

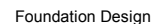
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+3, ×3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT <u>12-1121-0193-1321</u>		RECORD OF BOREHOLE No 14-6		SHEET 3 OF 3		METRIC														
G.W.P. <u>4092-10-01 & 4270-11-01</u>		LOCATION <u>N 4929522.2 ; E 360004.3</u>		ORIGINATED BY <u>KE</u>																
DIST <u></u> HWY <u>1000 Islands Pkwy</u>		BOREHOLE TYPE <u>Geoprobe 127 mm Diam.</u>		COMPILED BY <u>JM</u>																
DATUM <u>Geodetic</u>		DATE <u>July 8, 2014</u>		CHECKED BY <u>MSS</u>																
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa												
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> 25 50 75 </div>								
59.1	Direct Penetration Test					60														
21.2	END OF BOREHOLE DCPT REFUSAL NOTES: 1. Water level in open borehole at a depth of 5.5 m below ground surface (Elev. 74.8 m), measured during drilling.																			

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SHEET 1 OF 2

METRIC

PROJECT 12-1121-0193-1323

G.W.P.	LOCATION	N 4929544.2 ;E 360029.5	ORIGINATED BY	HEC
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DIST _____ HWY 1000 Islands Pkwy BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)/Wash Boring, HW Casing _____ COMPILED BY JEM

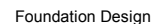
DATUM	Geodetic	DATE	November 9 & 10, 2015	CHECKED BY	KSL
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+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

○ 3% STRAIN AT FAILURE



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



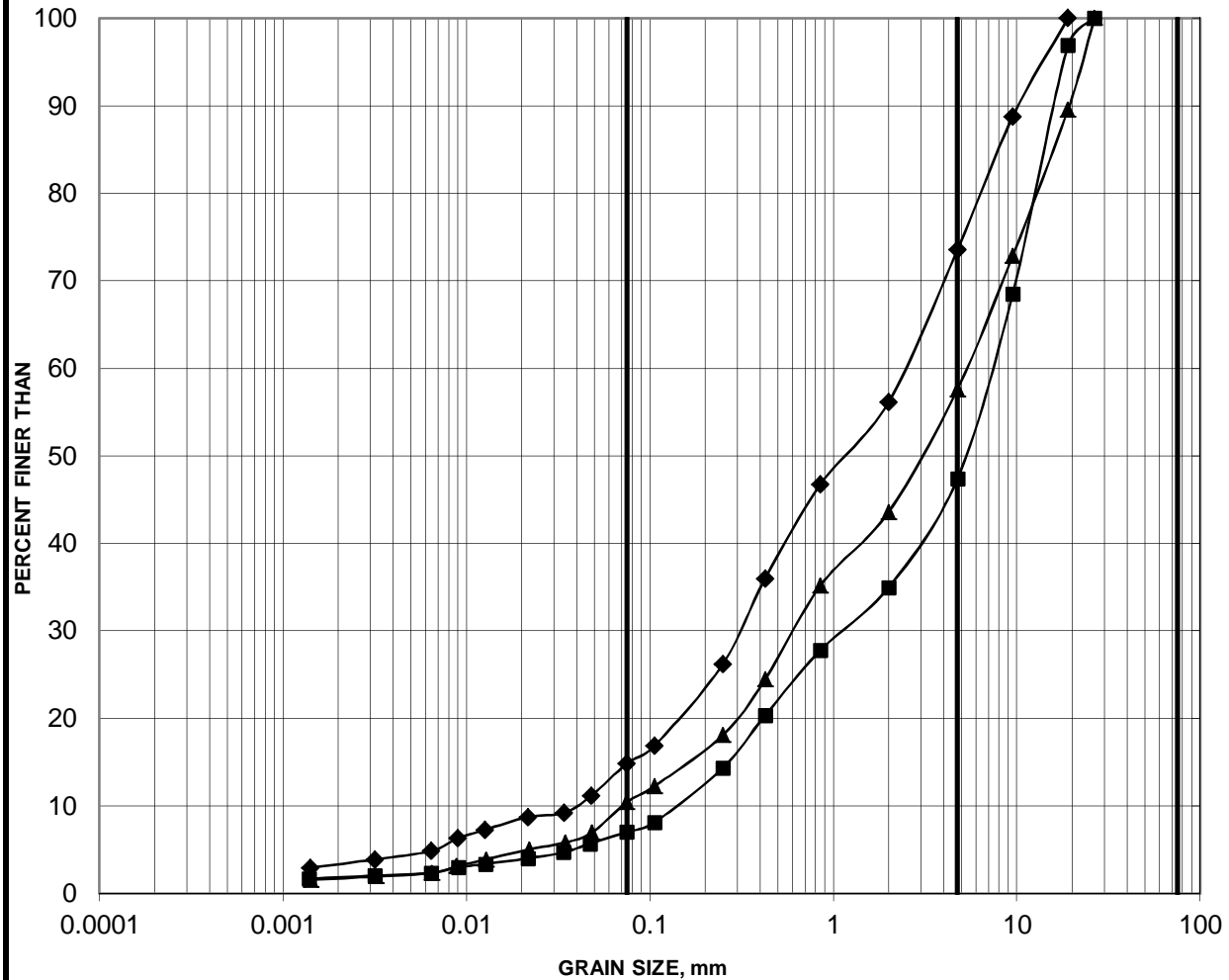
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND AND GRAVEL (PAVEMENT STRUCTURE FILL)



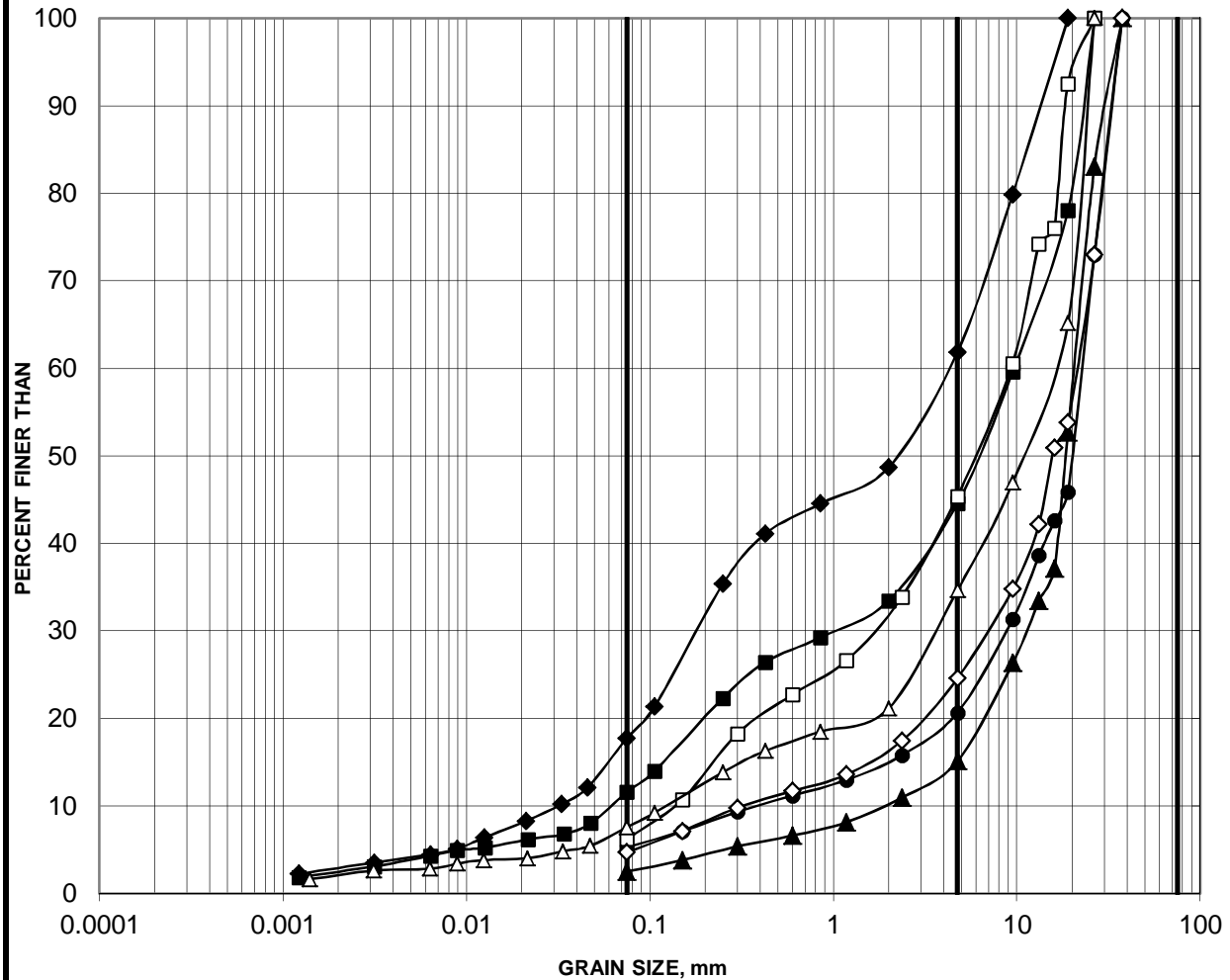
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
■ 14-2	1	0.00-0.91
◆ 15-1	1	0.00-0.61
▲ 15-1	2	0.76-1.37

GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND AND GRAVEL (EMBANKMENT FILL)



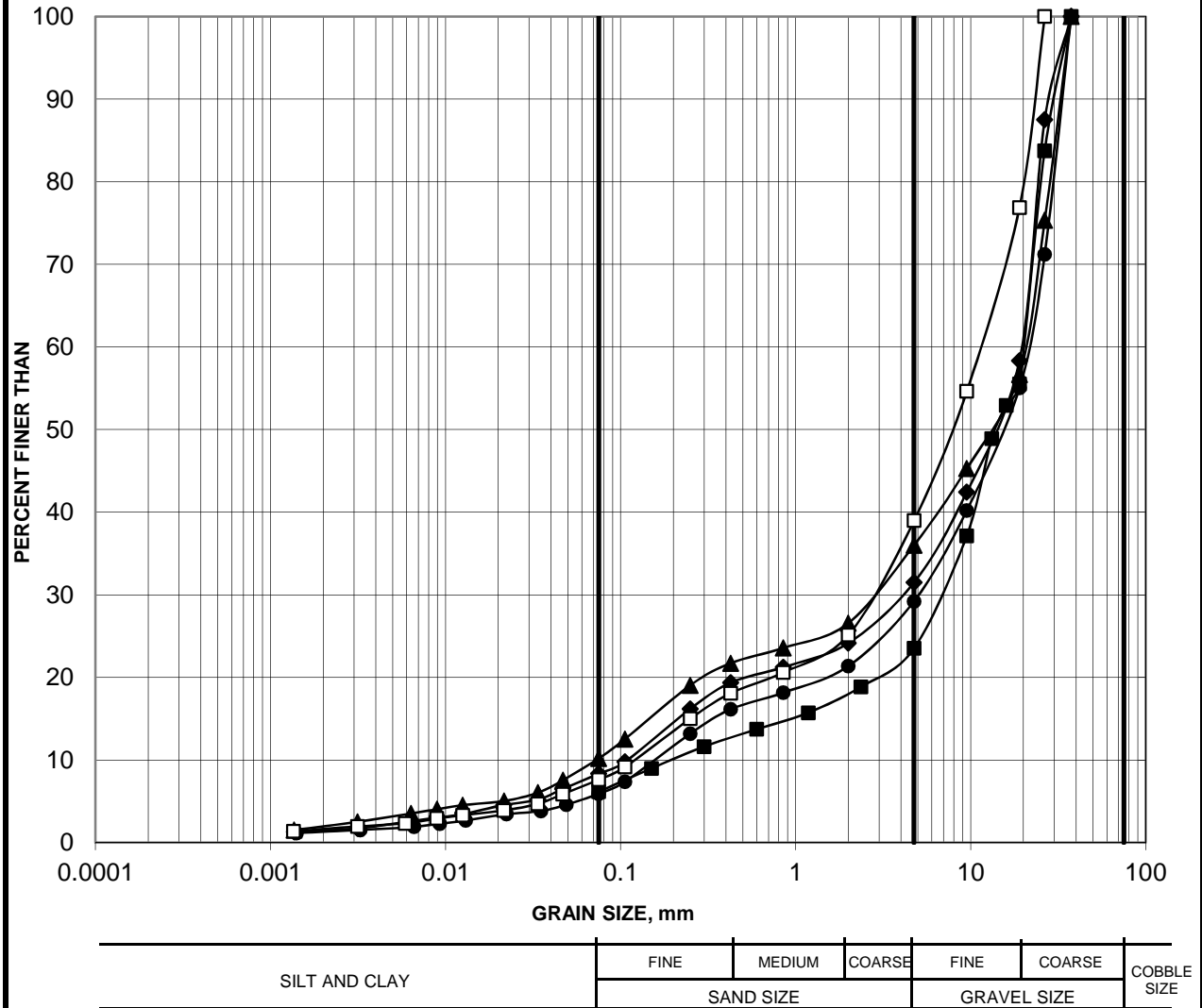
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
14-1	3	3.05-3.66
14-1	6	6.53-7.14
14-2	2	1.52-2.13
14-3	2	3.48-4.09
14-4	4	6.53-7.14
14-6	1	3.05-3.66
15-1	5	2.44-3.05

GRAIN SIZE DISTRIBUTION

FIGURE B3

SANDY GRAVEL (LOWER EMBANKMENT FILL)

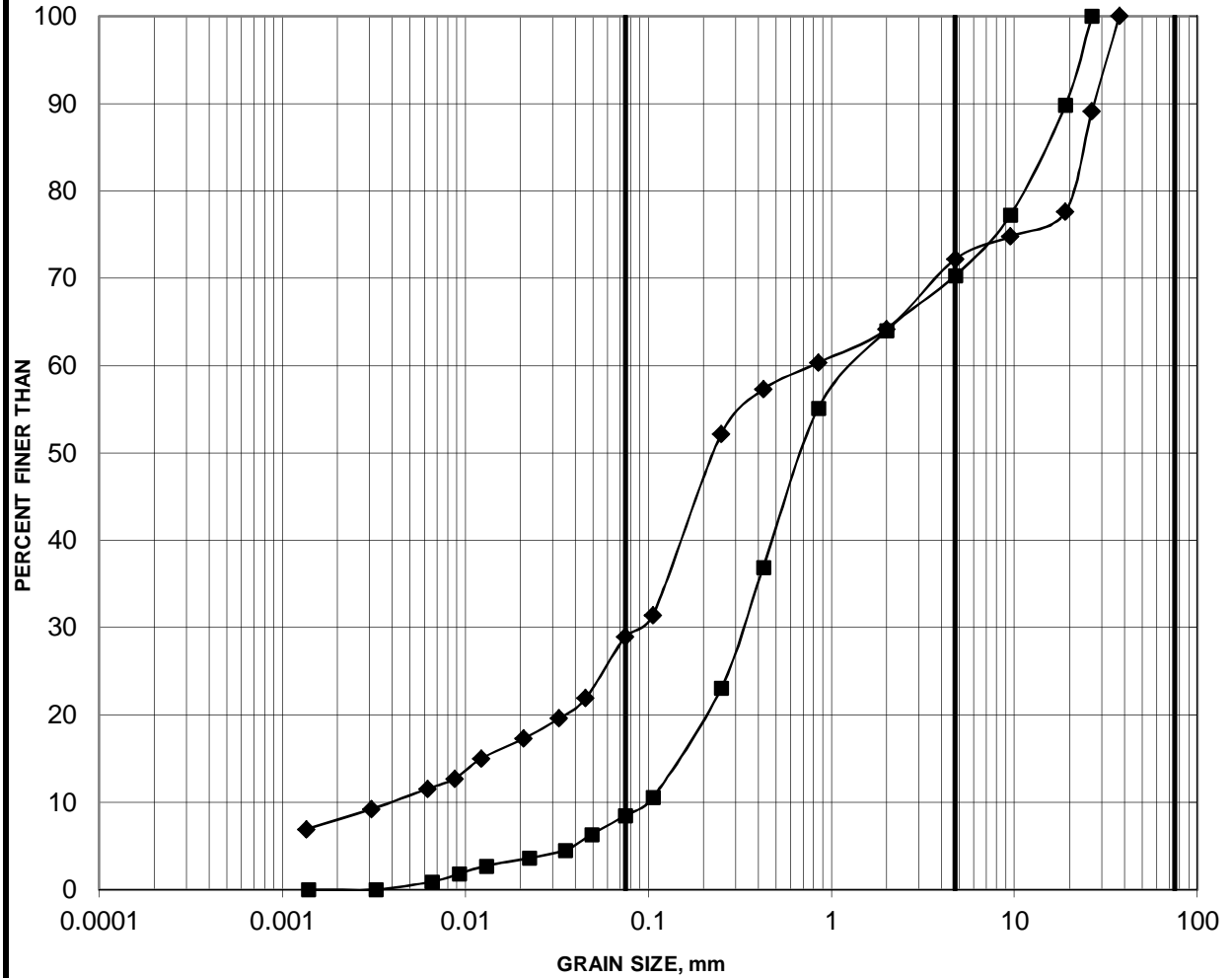


Borehole	Sample	Depth (m)
■ 14-2	6	7.62-8.08
◆ 14-2	8	10.67-11.28
▲ 14-3	5	8.03-8.15
● 14-5	3	6.10-6.71
□ 14-6	6	10.67-11.13

GRAIN SIZE DISTRIBUTION

FIGURE B4

GRAVELLY SAND

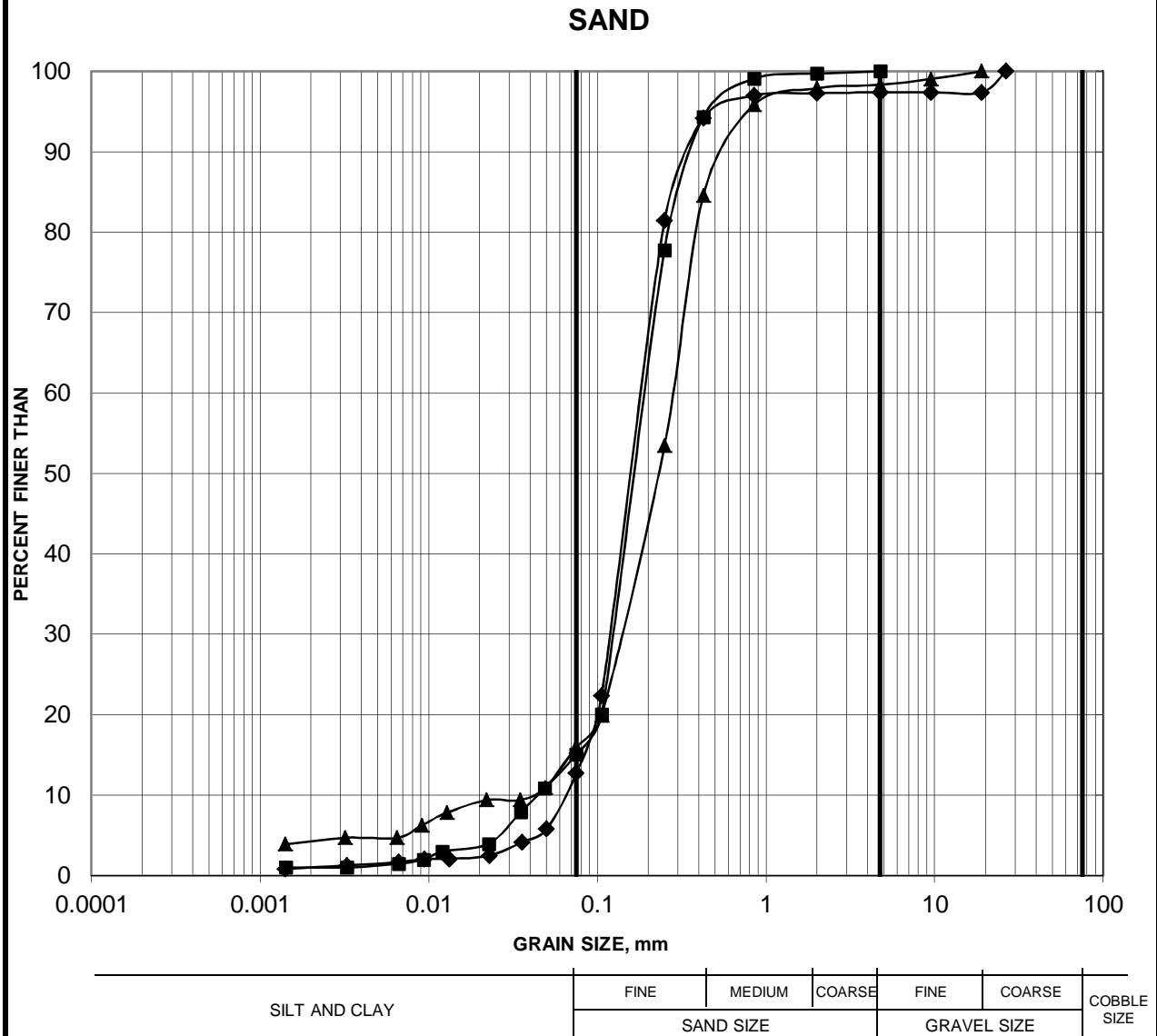


SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
■ 14-3	8	12.32-12.83
◆ 14-4	7A	11.30-11.66

GRAIN SIZE DISTRIBUTION

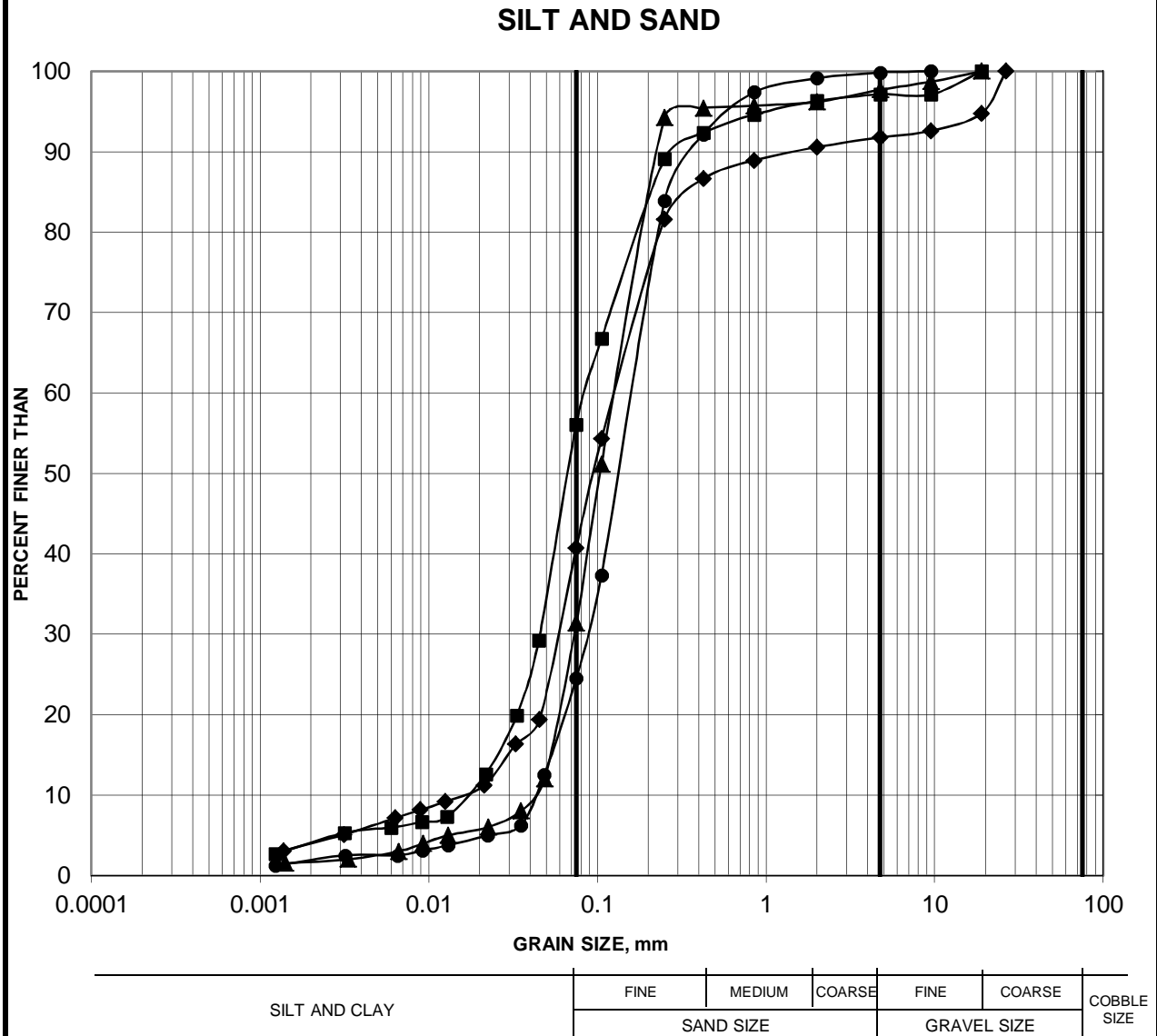
FIGURE B5



Borehole	Sample	Depth (m)
■ 14-1	11	13.67-14.28
◆ 14-6	8	12.20-12.80
▲ 15-1	18	11.59-12.20

GRAIN SIZE DISTRIBUTION

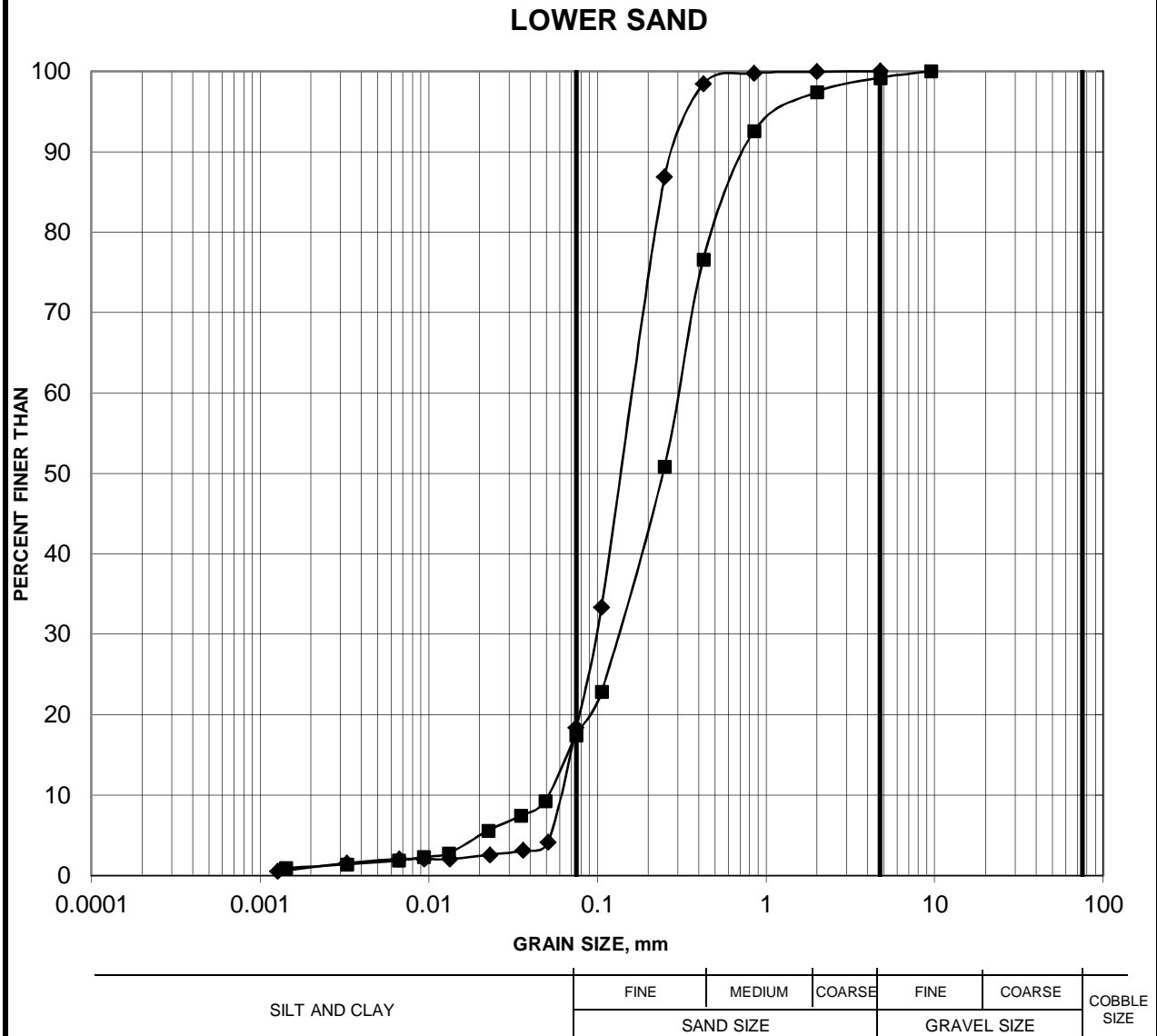
FIGURE B6



Borehole	Sample	Depth (m)
14-1	13A	16.79-17.25
14-2	9	12.20-12.80
14-3	9	13.84-14.30
14-3	11	16.16-16.77

GRAIN SIZE DISTRIBUTION

FIGURE B7

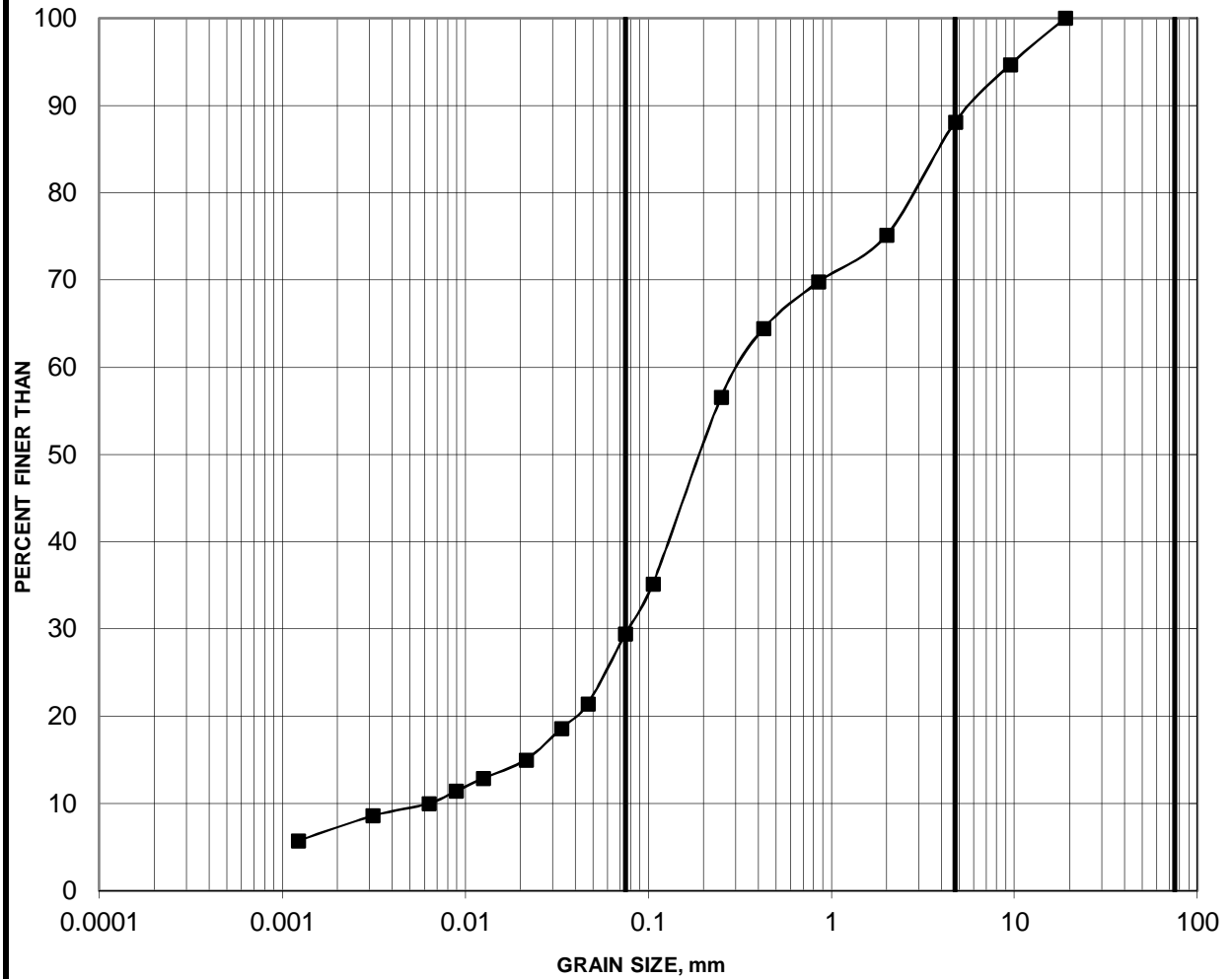


Borehole	Sample	Depth (m)
14-1	15	21.32-21.93
14-2	12	16.77-17.38

GRAIN SIZE DISTRIBUTION

FIGURE B8

BEDROCK (COMPLETELY WEATHERED)

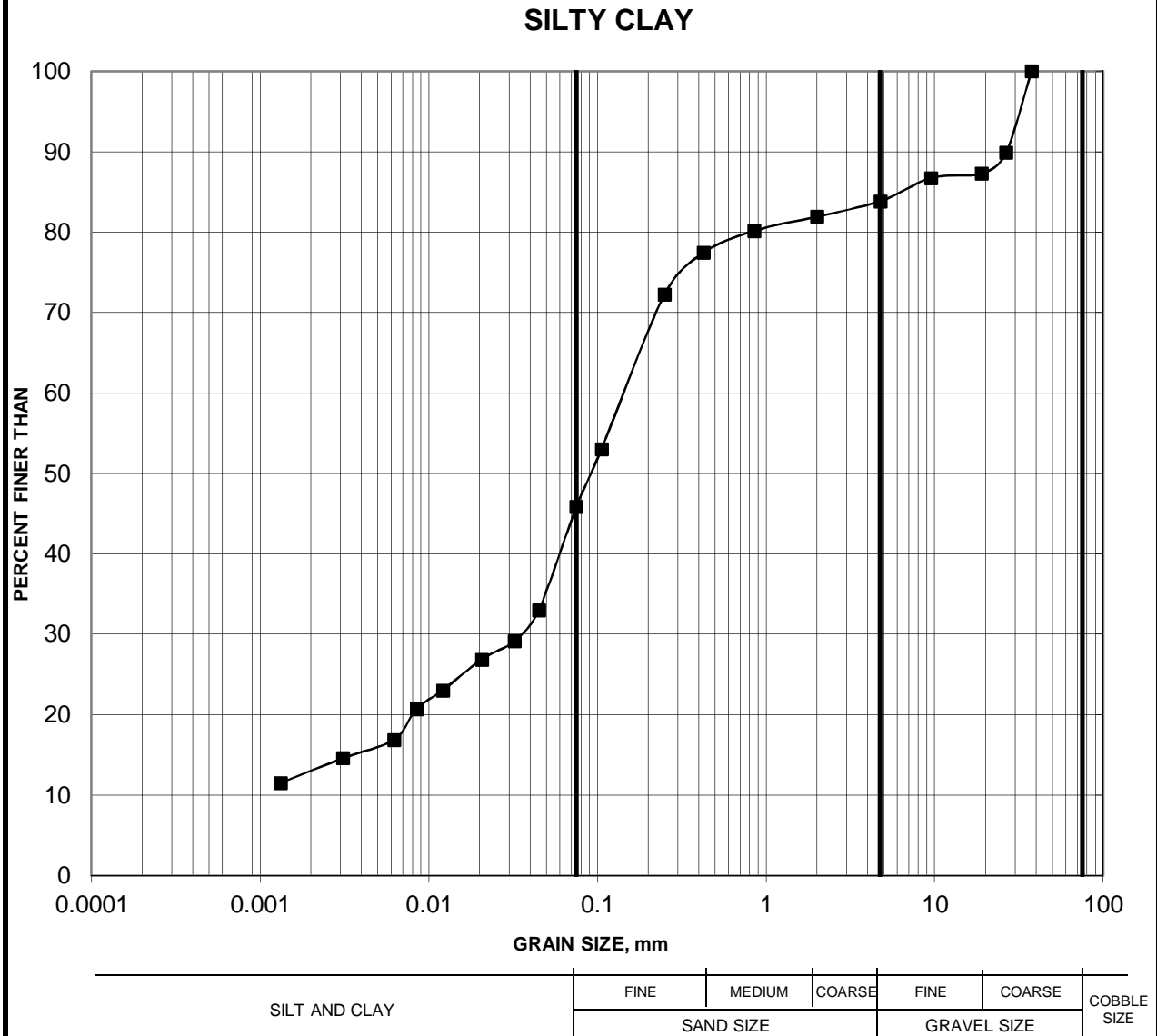


SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
—■ 14-5	7A	12.70-12.80

GRAIN SIZE DISTRIBUTION

FIGURE B9



Borehole	Sample	Depth (m)
—■ 15-1	17	10.98-11.20



APPENDIX C

Non-Standard Special Provisions and Guidelines

DEEP FOUNDATIONS – Item No.

Special Provision

1.0 SCOPE

The embankments at this site are comprised of granular/rock fill, which contain cobbles and boulders. The Contractor is advised that cohesionless soils are susceptible to disturbance under conditions of unbalanced hydrostatic head, and that appropriate equipment and construction procedures will be required for pre-augering through the bouldery fill for installation of steel piles. The Contractor is also advised that appropriate equipment and construction procedures will be required to penetrate or remove obstructions, such as cobbles and boulders, to permit installation of deep foundation elements and shoring elements.

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

H-PILES – HP310 X 110 - Item No.

Non-Standard Special Provision

903.07.02.07.03.03 Driving to Bedrock

Section 903.07.02.07.03.03 of OPSS 903 is deleted and replaced with the following:

When driving piles to bedrock, the Contractor shall adequately seat the pile on bedrock without damaging the pile.

In order to avoid overdriving and possibly damaging the piles when seating onto bedrock, the piles shall be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules. The bedrock elevation shall be recorded. On reaching the required set, the hammer energy shall be reduced to 75 percent of the maximum energy and the pile shall then be re-driven in 2 sets of 10 blows and the penetration recorded after each set of 10 blows. The hammer energy shall then be increased to 100 percent and the pile re-driven for 10 blows and the penetration recorded. A final set of no less than 10 blows per 12 mm of penetration shall be obtained at the maximum hammer energy.

If unrealistic excessive penetration per blow is observed, driving shall be stopped and this excessive penetration immediately reported to the Contract Administrator.

The Quality Verification Engineer shall determine when the hammer energy can be increased and when the driving is complete for each pile.

ROCK POINTS - Item No.

Non-Standard Special Provision

As part of the work under the above tender item, the Contractor shall supply Titus “Rock Injector Design” Pile Points or equivalent on HP 310x110 Piles. Piles will be driven to bedrock.

References

OPSS 906 – Structural Steel
SP903S01

Materials

The pile points shall be of the following:

Product

Manufacturer

HPP-R-12

Titus Steel Company Ltd.
6767 Invader Crescent
Mississauga, Ontario
Tel. 905-564-2446

(Or approved equivalent which includes Oslo Points as per OPSD 3000.201)

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

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.

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 pile installation works and 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 pile installation 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 pile has been driven and prior to continuing with the subsequent piles. As a minimum, the pile 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 each pile. The readings shall be taken and recorded during the entire length of driving and during seating of the pile on the bedrock.

The pile(s) furthest from the monitored structure or utility should be driven first to assess the the vibration level at the existing structures. If necessary, the contractor must alter the pile driving procedures for the remaining piles. The revised procedure shall be submitted to the Contract Administrator for approval prior to driving the remaining piles.

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

The results shall be submitted to the Contract Administrator after each pile has been driven and prior to continuing with the subsequent piles. If the vibration monitoring results are acceptable, the Contractor may continue with the next pile. 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.

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.

Where pre-augering is required for steel pile installation, the use of temporary or permanent liners will be required to minimize loss of ground through the water-bearing portions of the embankment fill present below the water table.

Basis of Payment

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

END OF SECTION

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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