July 2017

REPORT

DRAFT REPORT ON

Foundation Investigation and Design Mississippi River Bridge Highway 17, Site 3-003 West Carleton Township, Ontario W.P. 4121-10-01

Submitted to:
Dillon Consulting Limited
130 Dufferin Avenue, Suite 1400
London, Ontario
N6A 5R2

Geocres Number:

Report Number: 12-1121-0193-1130

Distribution:

1 copy - Ministry of Transportation, Ontario, Kingston
1 copy - Ministry of Transportation, Ontario, Downsview
1 copy - Dillon Consulting Limited
1 copy - Golder Associates Ltd.

Table of Contents

TABLES

DRAWINGS

APPENDICES

PART A

FOUNDATION INVESTIGATION REPORT
MISSISSIPPI RIVER BRIDGE
HIGHWAY 17, SITE NO. 3-003
WEST CARLETON TOWNSHIP, ONTARIO
W.P. 4121-10-01

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 17 Bridge (Site No. 3-003) over the Mississippi River which is located in the West Carleton Township approximately 9 km southeast of Arnprior, Ontario (W.P. 4121-10-01).

The purpose of this foundation investigation was to assess the subsurface conditions in the area of the proposed bridge and associated approach embankment areas to provide information for the detailed design of the bridge replacement. The foundation investigation included drilling boreholes as well as carrying out in-situ testing and laboratory testing on selected soil samples.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated August 2012. In addition, Golder's letter dated June 28, 2016 described the work plan for additional foundation and 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 Mississippi River Bridge is located on Highway 17, about 3.1 km northwest of the intersection with Kinburn Side Road, and approximately 9 km southeast of Arnprior, Ontario. The bridge has a northwest-southeast orientation. Foundations for a previous bridge with an east-west orientation are still visible, located to the north of the current bridge. The west abutment of the previous bridge was located immediately to the north of the current bridge, while the eastern abutment and pier foundation (still visible) are located approximately 50 to 60 m to the north of the current bridge.

The existing bridge consists of a 102 m long, 3-span truss bridge with a reinforced concrete deck slab supported on steel stringers and floor beams as shown in the photo below. The bridge has a roadway width of approximately 10 m and accommodates one lane of traffic in each direction. The existing pavement grade at the bridge location is at about Elevation 87 m. Available design drawings indicate the bridge piers and abutments are supported on about 15 m long untreated timber friction piles. The approach embankments to the bridge are in the order of 3 to 6 m high.

Available information indicates that the site is underlain by an extensive deposit of sensitive marine clay up to 60 m thick, over limestone bedrock.

At the crossing location, the Mississippi River floodplain is approximately 450 m wide and the river is approximately 80 m to 90 m wide. The ground surface elevation in the tableland areas beyond the floodplain is just over 90 m. The floodplain on the east bank of the river is at an elevation of approximately 83 m and contains an environmentally sensitive (Class 1) wetland environment. The southeastern approach embankment to the bridge cuts across the northern extent of this wetland. Marsh vegetation is also present in some portions of the floodplain on the northwest side of the river.

The Mississippi River flows from southwest to northeast at the site. The existing bridge embankment slopes are at approximately 2H: 1V (horizontal: vertical) or flatter, with some isolated portions sloped at 1H:1V, they are performing well, and appear to be stable.

It is understood that the only underground utility that exists at the site is a Bell Canada conduit aligned along the toe of the northeast embankment, parallel the bridge.

No information is available on the condition of the timber piles at the base of each foundation unit.

It is understood that the new bridge will be a three-span structure with the new abutments located exterior of the existing abutments and the new piers located interior of the existing piers. This will result in a slightly longer structure than the current bridge. A slight grade raise is proposed, up to approximately 0.7 m for the approach embankment. The new embankment side slopes are planned to be constructed at 2H: 1V. The new structure is planned to be constructed in a single stage using a full closure of the highway, and traffic will be re-routed.

3.0 INVESTIGATION PROCEDURES

The subsurface investigation for the bridge replacement was carried out in two stages. During the first stage, between August 15 and 30, 2016, ten boreholes were advanced. A second stage of investigation was carried out on October 27, 2016 to advance two boreholes at the toe of the embankments. Overall, eight main boreholes (numbered 16-101 to 16-108) were advanced at the locations shown on Drawing 1, with multiple attempts at four borehole locations, as described in the table below. The boreholes were advanced to depths ranging from 5.5 to 72.7 m below the existing ground surface.

The following table summarizes the drilling methods, borehole depth, and the number of attempts to advance the boreholes to target depths/layers and/or retrieve soil samples.

Borehole	Summary of Borehole Advancement
16-101	Advanced by HQ rotary drill wash boring to 29.9 m, NW wash boring to 69.1 m, NQ coring to 72.7 m.
16-101A	Attempt 2 at Borehole 16-101 to obtain additional samples of the fill and to better define the fill thickness. Advanced using a 200 mm inside diameter (I.D.) continuous-flight hollow-stem augers to 5.8 m.
16-102	Advanced by PW coring through asphalt and concrete, PW wash boring to 11.6 m, HW wash boring to 69.0 m, NQ coring to 71.9 m.
16-103	Attempt 1 – Advanced by PW coring through asphalt and concrete. Encountered diagonal I-beam below bridge deck. Attempt 2 – Moved 0.5 m west. PW coring through asphalt and concrete, PW wash boring to 13.1 m, HW wash boring to 68.2 m.
16-104	Advanced using a 200 mm inside diameter (I.D.) continuous-flight hollow-stem augers to 9.2 m, HW wash boring to 29.8 m, NW wash boring to 67.4 m, NQ coring to 71.2 m.
16-104A	Attempt 2 at Borehole 16-104 to better define the fill thickness. Advanced using a 200 mm I.D. continuous-flight hollow-stem augers to 5.9 m.
16-104B	Attempt 3 at Borehole 16-104 to better define the fill thickness. Advanced using a 200 mm I.D. continuous-flight hollow-stem augers to 5.8 m.
16-105	Advanced using a 200 mm I.D. continuous-flight hollow-stem augers to 15.2 m.
16-106	Advanced using a 200 mm I.D. continuous-flight hollow-stem augers to 15.2 m.
16-107	Advanced in a 50 mm diameter open hole by continuous split-spoon to 5.5 m.
16-108	Attempt 1 – NW wash boring to 1.8 m, split spoon tip fell down the hole. Attempt 2 – Moved 1 m south. NW wash boring to 1.8 m, split spoon tip feel down the hole. Attempt 3 – Moved 0.5 m south. NW wash boring to 4.9 m.
16-108A	Attempt 4 at Borehole 16-108 to define thickness of organic layer. Advanced in a 50 mm diameter open hole by continuous split-spoon to 6.7 m.

Boreholes 16-101 to 16-106, inclusive, 16-101A, 16-104A, and 16-104B were advanced using a truck-mounted drill rig, supplied and operated by George Downing Estate Drilling (Downing) of Hawkesbury, Ontario. Borehole 16-108 was advanced using portable drilling equipment also supplied and operated by Downing. Lastly, boreholes 16-107 and 16-108A were advanced using portable drilling equipment, supplied and operated by OGS Inc. of Almonte, Ontario.

Soil samples in the boreholes were obtained at vertical intervals ranging from about 0.6 to 6.0 m of depth using a 50 mm outer diameter (O.D.) split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedures. In-situ vane testing, using an MTO “N”-size vane, was carried out to measure the undrained shear strength of the cohesive soils encountered at the site. Twelve relatively undisturbed, 73 mm diameter thin walled Shelby tube samples of clay were retrieved using a piston sampler.

Monitoring wells were installed in boreholes 16-105, 16-106, and 16-108 to monitor groundwater levels at the site. The wells consist of 50 mm diameter rigid PVC pipes with a 1.5 m long slotted screen section. The standpipes were installed with silica sand and sealed with bentonite pellet backfill. Groundwater conditions were observed in the open boreholes during drilling operations. The groundwater levels in the monitoring wells were measured on October 27 of 2016 and May 10 of 2017.

PVC casings were installed and grouted in two boreholes to allow for Vertical Seismic Profiling (VSP) testing, as follows:

- Borehole 16-101 was backfilled with grout from 72.7 to 41.8 m depth and bentonite from 41.8 to 33.1 m depth. A 64 mm diameter rigid PVC standpipe was then installed from 33.1 m depth to the ground surface. The standpipe was backfilled with grout from 33.1 to 1.1 m depth. The remaining 1.1 m depth was backfilled with silica sand followed by the placement of a flushmount well cap surrounded by a cold asphalt mix.
- Borehole 16-104 was backfilled with grout from 71.2 to 33.6 m depth. A 64 mm diameter rigid PVC standpipe was then installed from 33.6 m depth to the ground surface. The standpipe was backfilled with grout from 33.6 to 1.2 m depth. The installation was backfilled with bentonite from 1.2 to 0.6 m depth and silica sand from 0.6 to 0.2 m depth followed by the placement of a flushmount well cap surrounded by a cold asphalt mix.

VSP surveys were carried out in the casings installed in boreholes 16-101 and 16-104 on August 29, 2016.

The other boreholes were generally backfilled with grout and/or bentonite pellets, mixed with native soils.

Artesian flow conditions at about the bedrock level were sealed with grout. The site conditions were restored following completion of the work.

The field work was supervised on a full-time basis by members of Golder’s engineering staff who located the boreholes in the field, observed the drilling, sampling, performed in situ testing operations, and logged the subsurface conditions encountered in the boreholes. The samples were identified in the field, placed in labelled containers and transported to Golder’s laboratories in Ottawa and Mississauga for further examination and testing. Index and classification tests consisting of water content, organic content, grain size distribution and Atterberg Limit testing were carried out on selected soil samples at Golder’s Ottawa laboratory. Oedometer consolidation testing was carried out on four samples of the clay from boreholes 16-04. In addition, unconfined compressive strength (UCS) testing was carried out on four samples of the bedrock. The oedometer and UCS testing were carried out at the Golder’s Mississauga laboratory, in addition to a small number of water content and Atterberg Limit tests. All the laboratory tests were carried out to MTO and/or ASTM standards as appropriate.

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 Drawing 1.

Borehole Number	Borehole Location with respect to Bridge Structure	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
16-101	East Abutment	5027536.9	323963.2	87.0	72.7
16-101A	East Abutment	5027533.0	323967.0	87.0	5.8
16-102	East Pier	5027554.1	323946.8	87.3	71.9
16-103	West Pier	5027594.4	323903.0	87.3	68.2
16-104	West Abutment	5027611.2	323887.7	87.0	71.2
16-104A	West Abutment	5027615.3	323883.6	87.0	6.0
16-104B	West Abutment	5027616.1	323882.9	87.0	5.8
16-105	East Approach	5027515.2	323983.1	86.9	15.2
16-106	West Approach	5027624.5	323875.1	86.9	15.2
16-107	Southwest Embankment Toe	5027521.8	323951.8	83.6	5.5
16-108	Southeast Embankment Toe	5027603.0	323882.0	83.8	4.9
16-108A	Southeast Embankment Toe	5027601.9	323883.1	83.8	6.7

Note 1: Boreholes for the east and west piers were advanced on the inside (closer to the shore) of the existing piers. It is understood that the new piers will be placed interior (further from the shore) of the existing piers. From a foundations perspective, the boreholes would likely be within approximately 7 to 8 m from the new pier locations and should be acceptable for the design of deep foundations at the new pier locations.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology and Available Geologic Information

The site is located within the 'Ottawa Valley Clay Plain' minor physiographic region, as delineated in *The Physiography of Southern Ontario*[1], that lie within the major physiographic region of the Ottawa-St. Lawrence Lowland. This physiographic region is underlain by a series of sedimentary rocks, consisting of sandstones, dolostones, limestones and shales that are, in turn, underlain by igneous and metamorphic bedrock of the Precambrian Shield.

The Ottawa Valley Clay Plain region is characterized by relatively thick deposits of sensitive marine clay, silt and silty clay that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock.[1]

The 1952 design drawings for the bridge contain "rudimentary boring data" associated with 4 boreholes drilled to depths of about 23 to 30 m below ground/river level adjacent to the bridge abutment and piers. The boring data suggests that the subsurface conditions at the site consist typically consist of surficial clay or loam overlying deep deposits of soft clay to silt that extend to the maximum depth of the boreholes (corresponding to elevations of approximately 53 and 61 m at the north and south abutments, respectively). The deep silt/clay deposits were not penetrated and reported to contain numerous springs.

No existing information was available for the site from Golder's in-house files or from MTO Pavement and Foundations Section's GEOCRES database.

4.2 Subsurface Conditions

As part of the current investigation, boreholes were advanced at 8 main locations in the vicinity of the existing Highway 17 crossing over the Mississippi River. The borehole locations, ground surface elevations, and interpreted stratigraphic conditions are shown on Drawing 1.

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the current investigation and the results of in situ and laboratory testing are given on the borehole records contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B11 and Table B1 contained in Appendix B.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile on Drawing 1 are inferred from observations of drilling progress and from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the site consist of up to 6 m of embankment fill underlain by an extensive deposit of firm to very stiff clay to a depth of approximately 66 m to 69 m. The silty clay deposit is underlain by a relatively thin transition layer of silt and clay with some sand over limestone bedrock.

---

[1] Chapman, L.J. and D.F. Putnam. *The Physiography of Southern Ontario*, Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,000.

Although the available boring data from the 1952 design drawings indicated a 'soft' material, they only included a description of the type of material encountered and did not include other information (e.g., Standard Penetration Tests, in situ vane testing, laboratory testing data etc.) that could be used to further define the strength or other engineering characteristics of the site materials. Based on the results of our current investigation, the clay deposit at the site was found to be firm to very stiff, generally increasing in strength with depth.

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

4.2.1 Pavement, Topsoil Fill, and Embankment Fill

A surficial asphalt pavement layer, ranging in thickness from 10 to 30 mm on the bridge deck, 80 mm on the approach slab, and 200 mm at the embankments, was encountered at the borehole locations. Under the asphalt layer, the concrete approach slab and bridge deck, where encountered, range in thickness from 190 to 270 mm.

Beneath the asphalt and concrete, embankment fill was encountered, extending to depths of about 4.8 to 6.4 m below the existing ground surface (i.e. Elevation 80.6 to 82.1 m). The embankment fill consists mainly of sand and gravel with variable amounts of silt. Silty clay fill was encountered at about 4.7 m depth at borehole 16-101, near the ground surface at borehole 16-105, and below a thin layer of sand and gravel fill at borehole 16-106. Traces of organic matter and cobbles were also noted in the embankment fill. Standard Penetration Test (SPT) "N" values measured within the embankment fill ranged from 2 to 42 blows per 0.3 m of penetration, but more generally between 3 and 26 blows per 0.3 m of penetration, indicating that the embankment fill is typically loose to compact.

A layer of topsoil fill having a thickness of about 100 mm was encountered at boreholes 16-107 and 16-108A.

The results of organic content testing carried out on one sample of the organic fill material are provided in Table B1 and indicate an organic content of about 17 percent. The results from grain size distribution testing completed on 6 selected samples of the sand and gravel fill material are shown on Figure B1 in Appendix B. These results represent only the portion of the fill materials that were collected within a 50 mm diameter split spoon sampler. The results of Atterberg limit testing on two samples of cohesive fill indicate plasticity index values of about 9 and 15 percent and liquid limit values of about 22 and 31 percent, as shown on Figure B2, indicating a silty clay fill of low plasticity. The natural water contents of the fill material were measured to vary between 3 and 70 percent, with the higher water contents measured in samples containing clay and organic matter.

4.2.2 Organics

Deposits of organic silt and clay were encountered beneath the embankment fill within boreholes 16-101, 16-103, 16-104A, 16-105, 16-106, and 16-108A. The organic deposits have thicknesses of between 0.3 to 3.2 m at the borehole locations.

The results of organic content testing carried out on 8 samples of the organic material are provided in Table B1 and indicate organic contents ranging from about 9 to 13 percent. The results of Atterberg limit testing on four samples of the organic material indicate plasticity index values of about 29 to 52 percent and liquid limit values of about 65 to 109 percent, as shown on Figure B3. The results from the laboratory testing indicate that the organic layers range from organic clay to organic silt with a high plasticity. The natural water contents of the organic samples were measured to vary between about 59 and 126 percent.

4.2.3 Sensitive Silty Clay to Clay

An extensive deposit of silty clay to clay was encountered at the site. The silty clay deposit was proven to be about 58 m to 62 m in thickness, with the base of the deposit observed at depths of approximately 67 to 69 m below the existing ground surface (i.e. Elevation 18.0 to 19.7 m). Much of the silty clay to clay deposit contained black organic mottling. Some silty sand layers were also noted.

The upper 0.6 to 2.3 m of the silty clay to clay has been weathered to a grey-brown crust at all of the borehole locations except 16-102 and 16-103, where the clay deposit is under that Mississippi River and not subject to the same weathering effects.

The bottom 1 to 4 m of the silty clay to clay deposit is sandy or contains traces to some sand. Such variations are common in the composition of the Champlain Sea clay.

SPT "N" values measured in the extensive silty clay deposit ranged from "weight of rods" to 14 blows per 0.3 m of penetration, but generally less than 3 and increasing with depth. In situ shear vane testing carried out within this deposit measured undrained shear strengths ranged from about 31 kPa to more than 96 kPa, indicating that the deposit has a firm to very stiff consistency. Below the stiffer weathered crust (where encountered), the shear strength generally increased with depth somewhat linearly from about 40 kPa (firm) at about Elevation 77 m to in excess of 96 kPa (very stiff) at about Elevation 40m. Remoulded shear strengths measured in the deposit ranged from about 6 to 35 kPa. The calculated sensitivity ratios in this deposit generally range between 2 and 7.

The results of grain size distribution testing carried out on 6 selected samples of the silty clay are provided on Figures B4 and B5. The results of Atterberg limit testing on 17 samples of this deposit indicate plasticity index values of between 15 and 60 percent and liquid limit values of between 33 and 88 percent, indicating that the deposit ranges from a clay of high plasticity (Figure B61) to a silty clay of low to intermediate plasticity (Figure B6b). The result of one Atterberg limit test was non plastic (see Figure B6b), is likely representing a sample taken within a silty sand layer within the clay deposit. The natural water contents of the silty clay to clay were measured to range from about 24 to 91 percent and was generally near or above the measured liquid limit values.

Oedometer consolidation testing was carried out on four samples of silty clay to clay from Borehole 16-104. The results of that testing, which are provided on Figures B7 to B10, are summarized in the table below and indicate that this material is normally consolidated to overconsolidated, with a preconsolidation pressure of about 125 to 270 kPa and overconsolidation ratio of 1.0 to 1.3.

Borehole/Sample Number	Sample Depth/Elevation (m)	Unit Weight (kN/m^3)	σ_P' (kP)	σ_{VO}' (kP)	σ_P' - σ_{vo}' (kPa)	Cc	Cr	e_o	OCR
16-104 / 10	9.5 / 77.5	17.4	125	125	0	0.35	0.019	1.09	1.0
16-104 / 11	12.0 / 75.0	14.8	165	140	25	3.36	0.032	2.38	1.18
16-104 / 13	18.0 / 69.0	15.4	180	170	10	1.97	0.031	2.00	1.06
16-104 / 15	24.0 / 63.0	16.4	270	210	60	1.34	0.012	1.55	1.29

Notes:

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

4.2.4 Sandy Silt

A thin veneer of sandy silt, with some gravel was noted between the silty clay and bedrock in Borehole 16-101, no more than 160 mm in thickness. This is likely the same sandier layer of silt and clay that was encountered in the other three deep boreholes, however more difficult to identify with accuracy due to the limited sampling intervals that were used at depth.

4.2.5 Refusal and Bedrock

Refusal to sampler advancement was encountered in Borehole 16-103 at about 68.2 m depth (Elevation 19.1 m).

Bedrock was encountered beneath the thick clay deposit (and sandy silt in Borehole 16-101) at depths ranging from about 67.4 m to 69.1 m (i.e., Elevation 19.7 to 17.9 m). The bedrock was cored between 2.9 and 3.8 m using a NQ drill bit and rods.

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

Borehole Number	Borehole Location with respect to Bridge Structure	Existing Ground Surface Elevation (m)	Depth to Bedrock/Refusal (m)	Bedrock Surface/Refusal Elevation (m)
16-101	East Abutment	87.0	69.1	17.9
16-102	East Pier	87.3	69.0	18.3
16-103	West Pier	87.3	68.2[1]	19.1[1]
16-104	West Abutment	87.0	67.4	19.7

Note 1: Refusal to sampler advancement.

Based on the bedrock surface elevations obtained where bedrock was proven, it is likely that the refusal to sampler advancement was encountered at the surface of the bedrock in Borehole 16-103.

The bedrock encountered in these boreholes consist of fresh, thinly to medium bedded, grey, fine to medium grained, non-porous limestone with occasional shale partings. The Rock Quality Designation (RQD) values measured on recovered bedrock core samples ranged from about 96 to 100 percent, indicating an excellent quality rock.

Results of unconfined compressive strength testing carried out on four bedrock core samples are presented in Figure B11. The results range from about 45 to 74 MPa and indicate a medium strong to strong bedrock.

A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

4.3 Groundwater Conditions

Water levels were observed in the boreholes during drilling operations. Open borehole water levels ranged from about 0.8 to 4.1 m below the existing ground surface.

Standpipe piezometers were installed in boreholes 16-105, 16-106, and 16-108 at bottom depths of 15.1 m, 15.1 m, and 4.8 m below ground surface, respectively. All three standpipe piezometers were installed with the screened section within the silty clay to clay deposit. The water levels were measured in the piezometers on October 27 of 2016 and on May 10 of 2017. The observations are summarized in the following table:

Borehole	Ground Surface Elevation (m)	Approximate Depth to Screen (m)	October 27, 2016		May 10, 2017	
			Water Level Depth (m)	Water Level Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)
16-105	86.9	15.1	3.1	83.8	2.9	84.0
16-106	86.9	15.1	3.0	83.9	1.3	85.6
16-108	83.8	4.8	0.4	83.4	N/A[1]	N/A[1]

Note 1: The piezometer was not accessible because the borehole location was below the river level at the time of measurement.

Artesian conditions were encountered at boreholes 16-101 and 16-104 during the drilling operations. The water level elevations and depth of casing at the time of artesian conditions are listed in the following table.

Borehole	Ground Surface Elevation (m)	Depth of Casing (m)	Water Level Above Ground Surface (m)	Water Level Elevation (m)	Date
16-101	86.9	69.1	4.1	91.0	August 18, 2016
16-104	86.9	67.5	4.4	91.3	August 24, 2016

These combined groundwater level data indicate an apparent upward hydraulic gradient.

It is expected that these water levels will be subject to fluctuations both seasonally and as a result of precipitation events.

5.0 CLOSURE

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

GOLDER ASSOCIATES LTD.

Kim Lesage, P.Eng.
Geotechnical Engineer

Fintan Heffernan, P.Eng.
Designated MTO Foundations Contact

SN/SG/KSL/MSD/FJH/mvrd
n:\active\2012\1121 - geotechnical\12-1121-0193 dillon mega 3 eastern region\foundations\5 - reports\contract f - mississippi river bridge\12-1121-0193-1130 site 3-003 draft july 2017 fidr.docx

PART B

FOUNDATION DESIGN REPORT
MISSISSIPPI RIVER BRIDGE
HIGHWAY 17, SITE NO. 3-003
WEST CARLETON TOWNSHIP, ONTARIO
W.P. 4121-10-01

6.0 FOUNDATION ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed replacement of the existing Mississippi River Bridge (MTO Structure Site No. 3-003) on Highway 17 near Arnprior, Ontario. 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 only to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the detail design of the foundations for the replacement structure.

The foundation design report, discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. 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 the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Golder Associates Ltd. (Golder) has been retained by Dillon Consulting Limited (Dillon) on behalf of the MTO to provide recommendations on the foundation aspects for the design of the replacement structure of the Mississippi River Bridge on Highway 17 near Arnprior, Ontario.

The existing bridge consists of a three-span truss structure with reinforced concrete slab supported on steel stringers and floorbeams. The existing bridge was constructed in 1954 and is founded on about 15 m long untreated timber friction piles. The existing bridge is understood to be in poor condition. It currently carries two lanes of Highway 17 traffic over the Mississippi River and is approximately 12 m wide and 102 m long. The existing pavement grade at the bridge location is at about Elevation 87 m. The existing embankment slopes at the bridge location are about 3 to 6 m in height and are sloped at about 2H:1V or flatter, with some isolated portions sloped at 1H:1V. At the bridge location, the river channel is about 80 to 90 m wide and flows from southwest to northeast.

Consideration is being given to replacing the existing bridge with a three-span structure. It is understood that the abutments for the new bridge will likely be placed immediately behind the existing bridge abutment foundations, along the same alignment, which would result in a slightly longer bridge (i.e., approximately 110 m long). Consideration is currently being given to integral and semi-integral abutments. It is further understood that consideration is being given to raising the approach road grades as much as 0.7 m.It is understood that the bridge is to be designed in accordance with the current Canadian Highway Bridge Design Code CSA-S6-14 (CHBDC).

6.2 Seismic Design

6.2.1 Importance Category

The CHBDC states that the seismic hazard values associated with the design earthquakes should be those established for the National Building Code of Canada (NBCC) by the Geological Survey of Canada (GSC). The GSC has developed a new set of seismic hazard maps (referred to as the 5th generation seismic hazard maps) that were made available for public use in December 2015.

In accordance with Section 4.4.2 of the CHBDC, we understand that the proposed bridge structure has an importance category of 'other' bridge.

6.2.2 Seismic Site Classification

Vertical Seismic Profiling (VSP) geophysical testing was carried out near the east and west abutment areas to evaluate the average shear wave velocity of the upper 30 m of soil at the site. The shear wave velocities measured are presented in a technical memorandum (see results in Appendix C) and indicate that the average shear wave velocity in the upper 30 m of the subsurface stratigraphy at the two locations are in general agreement, ranging from about 173 m/s to 140 m/s, to the east and west of the bridge respectively. The measured shear wave velocity at 30 m depth (from ground surface) is 200 m/s or greater. Based on these results and using Table 4.1 of the CHBDC, it is considered that a Site Class of E would be applicable for the design of the structure.

6.2.3 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the CHBDC and based on the location of the bridge (latitude 45.388 and longitude -76.256), the following are the reference Site Class C (reference) peak seismic hazard values based on the 5th generation seismic hazard maps published by the GSC.

Seismic Hazard Values for Reference Ground Condition Site Class C

Seismic Hazard Values	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.240
PGV (m/s)	0.170
Sa (0.2) (g)	0.375
Sa (0.5) (g)	0.204
Sa (1.0) (g)	0.103
Sa (2.0) (g)	0.050
Sa (5.0) (g)	0.013
Sa (10.0) (g)	0.0049

The values given above are for the reference ground condition Site Class C and must be modified to the site-specific seismic site classification given in Section 6.2.2 (Site Class E) in accordance with Section 4.4.3.3 of the CHBDC. As indicated in Section 4.4.3.3 of the CHBDC the value of PGA_{ref} for use with Tables 4.2 to 4.9 shall be taken as 80 percent of the PGA for Site Class C where Sa(0.2)/PGA is less than 2.0. Based on this requirement a PGA_{ref} value of 0.192 for the 2,475 year return was used.

The corresponding site-specific seismic hazard values given in the table below can be used for design.

Seismic Hazard Values for Reference Ground Condition Site Class E

Seismic Hazard Values	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.306
PGV (m/s)	0.315
Sa (0.2) (g)	0.477
Sa (0.5) (g)	0.378
Sa (1.0) (g)	0.220
Sa (2.0) (g)	0.115
Sa (5.0) (g)	0.032
Sa (10.0) (g)	0.011

6.3 Bridge Foundation Options

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

- **Driven steel H-piles:** Steel H-piles driven to refusal on the limestone bedrock are feasible for support of the replacement bridge structure. Steel H-pile foundations would also allow for the construction of integral abutments. It is recommended that any battered piles (which will be more prone to sliding on the level bedrock) be provided with suitable driving points. With a grade raise in the area of the abutments, significant downdrag forces could act on the piles if settlements are not mitigated.

- **Driven steel pipe (tube) piles:** Steel tube piles driven to refusal on the limestone bedrock are feasible for support of the abutments and piers, where applicable, and would allow for the construction of integral and semi-integral abutments. If battered piles are required, which will be prone to sliding on the surface of the level bedrock, they will require driving shoes. With a grade raise in the area of the abutments, significant downdrag forces could act on the piles if settlements are not mitigated.

- **Caissons (Drilled Piers):** Caissons socketed into the bedrock are considered to be technically feasible for support of the replacement bridge structure. Caissons would likely be significantly more expensive than steel H-piles due to the lengths required at this site. The use of a liner or casing would be required in order to advance the caissons with minimal loss of ground, since the overburden materials would not stand un-supported. It is also recommended that the casings be left in-place as a permanent component of the caissons. Similarly to steel H-piles, a grade raise at the site could lead to even greater downdrag forces on caissons if the settlements are not mitigated.

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

6.3.1 Feasibility of Integral Abutments

As outlined in MTO's report SO-96-01, integral abutment bridges are single span or multiple span continuous deck type bridges with a movement system composed primarily of abutments on flexible integral foundations and approach slabs, in lieu of movable deck expansion joints and expansion bearings at the abutments. The feasibility of integral abutments is influenced by a number of factors including geometry and subsurface conditions. The primary criterion is the need to support the abutments on relatively flexible piles. Where the load bearing stratum is near the surface or where the use of short piles or caissons (less than 5 m in length) is planned, the site is not considered suitable for integral abutment bridges. Geometric constraints on the use of integral abutments are also applicable and include: overall bridge length less than 150 m; skew angle less than 35º; and abutment wall heights less than 6 m without a retained soil system.

From a foundation perspective, integral abutments are considered feasible at this location.

6.3.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the CHBDC and its Commentary, the proposed bridge structure and foundation system may be classified as having medium to large traffic volumes and its performance as having potential impacts on other transportation corridors, hence having a "typical" consequence level associated with exceeding limits states design. Given the level of foundation investigation completed to date as presented in Sections 3.0 and 4.0, in comparison to the degree of site understanding in Section 6.5 of CHBDC, the level of confidence for design is considered to be a "typical degree of site and prediction model understanding." Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ, and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Tables 6.1 and 6.2 of the CHBDC have been used for design.

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

6.3.3.1 Founding Elevations

The abutments and associated wingwalls for the replacement bridge, and piers for a three-span structure, may be supported on steel H-piles or steel pipe piles driven to found on the limestone bedrock. Based on the borehole results from the investigation, the following pile tip elevations are recommended for design of piles:

Foundation Element	Borehole Numbers	Bedrock Surface / Pile Tip Elevation
East Abutment	16-101	17.9
East pier	16-102	18.3
West pier	16-103	19.1
West Abutment	16-104	19.7

The pile caps should be constructed at a minimum depth of 1.8 m for frost protection purposes, per OPSD 3090.101 *(Foundation Frost Penetration Depths for Southern Ontario)*.

All piles (particularly battered piles) should be equipped with suitable driving points (such as Titus Standard 'H' Bearing Pile Points for H-piles or Titus Open Cutting Shoe for pipe piles, or equivalent) to ensure adequate seating of the piles on the bedrock. If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001.100 Type II *(Steel Tube Pile Driving Shoe).*

6.3.3.2 Axial Geotechnical Resistance

For design of HP 310 x 110 piles driven to bedrock at the estimated tip elevations provided in Section 6.3.3.1, the factored axial resistance at ULS may be taken as 2,000 kN. For closed-end, concrete-filled, 324 mm diameter steel pipe piles (with 13 mm wall thickness) driven to bedrock, the factored axial resistance at ULS may be taken as 2,000 kN. Serviceability Limit States (SLS) resistances do not apply to piles founded on the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

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

It should be noted that steel tube piles are large displacement piles compared to steel H-piles and therefore would remould the sensitive clay during driving.

6.3.3.3 Downdrag Load (Negative Skin Friction)

The placement of earth fill for any embankment widening (expected to be minimal) or grade raise (up to 0.7 m for Option 1 only) will raise the effective stress level in the silty clay to clay deposit which underlies the site. This increase in stress could lead to elevated settlement of the underlying silty clay deposit and corresponding downdrag loads on the piles at the abutments, which could in turn reduce the available capacity.

These downdrag loads (or negative skin friction) will need to be taken into account during the design of the piles supporting the bridge abutments. No downdrag loads would be expected at the piers.

The downdrag loads could vary depending on the selected embankment fill material, on the sequence of construction, and on the underside of pile cap elevation. Assuming an underside of the pile cap of about Elevation 82 m, the unfactored downdrag load acting on a single HP 310 x 110 pile, over the length of pile within the portion of the compressible soils that would experience settlement, is estimated to be up to 700 kN. Similary, the unfactored downdrag load acting on a single 324 mm diameter steel pipe piles is estimated to be up to 600 kN.

The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.11.4.10 of the CHBDC.

If the predicted downdrag loads cannot be accommodated structurally, downdrag loads on the piles can be eliminated by preventing post-construction settlement of the surrounding soils. Further discussion of the embankment settlement mitigation options are provided in Section 6.5.2.

6.3.3.4 Lateral Geotechnical Resistance

If integral abutments are selected for the replacement structure, there will be a requirement for the piles to move sufficiently to accommodate the thermal movements of the bridge. To accommodate the movements associated with integral abutments, a sand-filled corrugated steel pipe (CSP), 0.6 m in diameter and 3 m in length, is typically provided extending below the underside of the pile cap. A Non Standard Special Provision for the supply and installation of CSP's should be included in the contract documents and a sample has been included in Appendix D of this report. The grading of the sand backfill in the CSP is also given in the NSSP.

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure theory, assuming that it acts over the the pile shaft to a depth equal to six pile diameters below the underside of the pile cap and an equivalent width equal to three pile diameters.

The ULS lateral resistance of a pile group may be estimated as the sum of the individual pile resistances across the width of the pile group in plan, perpendicular to the direction of the applied lateral force, and a depth of six pile diameters below the underside of the pile cap.

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.

For design of the structure, the SLS geotechnical response of the soil in front of the piles under lateral loading may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the equation given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (3rd Edition). It may be assumed that this resistance will be nearly the same for vertical and inclined piles.

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:

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

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, the soil properties and the approximate nature of the analysis and the non-linear nature of the soil behaviour (such that k_h is a function of deflection).

Location (Borehole)	Elevation (m)	Soil Type	n_h (MN/m^3)	s_u (kPa)
East Abutment (16-101)	80.3 – 82.2[1]	New Compacted Fill	6.6	-
	79.2 – 80.3	Stiff Weathered Silty Clay to Clay Crust	-	75
	60.0 – 79.2	Firm to Stiff Silty Clay to Clay	-	40 to 60
	50.0 – 60.0	Stiff Silty Clay to Clay	-	60 to 80
	39.3 – 50.0	Stiff Silty Clay to Clay	-	80 to 100
	17.9 – 39.3	Stiff to Very Stiff Silty Clay to Clay	-	100
	17.9	Bedrock	-	-

Location (Borehole)	Elevation (m)	Soil Type	n_h (MN/m^3)	s_u (kPa)
East Pier (16-102)	65.0 – 73.5[2]	Firm to Stiff Silty Clay to Clay	-	40 to 60
	51.0 – 65.0	Stiff Silty Clay to Clay	-	60 to 80
	39.0 – 51.0	Stiff Silty Clay to Clay	-	80 to 100
	18.3 – 39.0	Stiff to Very Stiff Silty Clay to Clay	-	100
	18.3	Bedrock	-	-
West Pier (16-103)	68.0 – 76.5[2]	Firm to Stiff Silty Clay to Clay	-	40 to 60
	45.0 – 68.0	Stiff to Firm Silty Clay to Clay	-	60 to 80
	18.3 – 45.0	Stiff Silty Clay to Clay	-	100
	19.1	Probable Bedrock	-	-
East Abutment (16-104)	81.7 – 82.2[1]	New Compacted Fill	6.6	-
	79.4 – 81.7	Stiff Weathered Silty Clay to Clay Crust	-	75
	63.0 – 79.4	Firm to Stiff Silty Clay to Clay	-	40 to 60
	39.4 – 63.0	Stiff Silty Clay to Clay	-	60 to 80
	19.7 – 39.3	Stiff to Very Stiff Silty Clay to Clay	-	100
	19.7	Bedrock	-	-

Note 1: Underside of abutment elevation.
Note 2: Top of footing elevation.

6.3.4 Cast-in-Place Concrete Caissons

6.3.4.1 Founding Elevations

Cast-in-place concrete caissons, supported on or socketed nominally (i.e., about 1 m) into the limestone bedrock, could be used for support of the abutments and piers. Based on the borehole results from the investigation, the following 'bottom of shaft' elevations are recommended for design of concrete caissons:

Foundation Element	Borehole Numbers	Bedrock Surface / Bottom of Shaft Elevation
East Abutment	16-101	17.9
East pier	16-102	18.3
West pier	16-103	19.1
West Abutment	16-104	19.7

The native marine (Champlain Sea) clay at this site is a sensitive soil. The disturbed clay could "flow" into the auger hole during drilled shaft installation if left unsupported. The use of a permanent casing will be required in order to advance the drilled shafts with minimal loss of ground. For caissons of this length, it should not be planned to remove the casing during concreting.

Additionally, it will be difficult to clean the bedrock surface, even with the use of liners, unless the liner is nominally socketed into the bedrock; once disturbed, the sensitive clay soils, as well as the sandy material at depth, could flow under the casings, at the interface with the bedrock.

The caisson excavations may need to be cleaned using methods such as airlifting prior to concreting, and tremie concreting techniques may be required for placing concrete.

Given these conditions, the minimum recommended caisson diameter is 0.9 m. However diameters of up to 1.5 m can be constructed using locally available equipment.

The pile caps should be constructed at a minimum depth of 1.8 m for frost protection purposes, per OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario).

6.3.4.2 Axial Geotechnical Resistance

Caissons socketed nominally into the bedrock should be designed based on end-bearing resistance, and a factored geotechnical resistance at ULS of 5 MPa should be used. Serviceability Limit States resistance does not apply, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. The expected settlements for caissons sized in accordance with the above resistance should be negligible.

6.3.4.3 Downdrag Load (Negative Skin Friction)

The placement of embankment fill (for conventional embankment construction) will raise the effective stress level in the silty clay deposit, leading to some consolidation of the deposit. As discussed previously in Section 6.3.3.3, this condition will similarly result in downdrag forces on caissons. The unfactored downdrag load acting on a single 0.9 metre or 1.5 metre diameter caisson over the length of caisson within the portion of the compressible soils that would experience settlement is estimated at ranging from 1,600 to 2,700 kilonewtons (depending on the underside of pile cap level and caisson diameter). The structural capacity of the caissons must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the CHBDC. The assumptions and methods used in assessing that downdrag force or of mitigating it are the same as those described in Section 6.3.3.3 of this report with respect to steel H-piles.

6.3.4.4 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons, and the reductions due to group effects, may be determined as per Section 6.3.3.4.

6.3.5 Lateral Soil-Structure Interaction Springs

The foundation lateral soil-structure interaction springs required for the static and dynamic analyses of the bridge abutments and piers will be computed once the structure type, foundation type, and pile layout is confirmed. This information will be included in the final report.

6.4 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 abutment walls. These design recommendations and parameters assume a level backfill/ground surface behind the walls.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 *(Aggregates)* Granular A or Granular B Type II, should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 *(Compacting)*. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 *(Walls, Abutment, Backfill, Minimum Granular Requirement),* OPSD 3121.150 *(Walls, Retaining, Backfill, Minimum Granular Requirement)*, and 3190.100 *(Walls, Retaining and Abutment, Wall Drain).*

- 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 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand-operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.

- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.8 m behind the back of the wall (Case (a) on Figure C6.20 of the *Commentary* to the CHBDC). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing or pile cap (Case (b) on Figure C6.20 of the *Commentary* to the CHBDC).

6.4.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) 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 proposed embankment fill and the following parameters (unfactored) may be used assuming the use of earth fill or Select Subgrade Material (SSM):

Material	Earth Fill or SSM
Soil Unit Weight:	20 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 Case (b), the pressures are based on using engineered granular fill and the following parameters (unfactored) may be used:

Material	Granular A	Granular B Type II
Soil Unit Weight:	22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o Passive, K_p	 0.27 0.43 3.7	 0.27 0.43 3.7

- Where the wall support 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 the wall support and superstructure allow lateral yielding, active earth pressures should be used for the geotechnical design of the structure. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the Commentary to the CHBDC.

6.4.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must be taken into account in the design in accordance with Section 4.6 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 6.4.1, above, plus the earthquake-induced dynamic earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 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 1.0 times the PGA. For structures which allow lateral yielding, (k_h) is taken as 0.5 times the PGA.
- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case (a) and Case (b)) 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}

Wall Type	Design Earthquake	Site PGA	K_{AE} for Granular A	K_{AE} for Granular B Type II	K_{AE} for SSM
Yielding Wall	**2,475-Yr**	0.306	0.36	0.36	0.44
Non-Yielding Wall	**2,475-Yr**	0.306	0.48	0.48	0.58

- 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:

$$\sigma_h(z) = K_a\,\gamma\,z + (K_{AE} - K_a)\,\gamma\,(H\text{-}z)\text{, yielding walls}$$

$$\sigma_h(z) = K_o\,\gamma\,z + (K_{AE} - K_a)\,\gamma\,(H\text{-}z)\text{, non-yielding walls}$$

Where: $\sigma_h(d)$ is the (static plus seismic) lateral earth pressure at depth, z, (kPa);

K_a is the static active earth pressure coefficient;

K_o is the static at-rest earth pressure coefficient;

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

γ is the unit weight of the backfill soil (kN/m^3), as given previously;

z is the depth below the top of the wall (m); and,

H is the total height of the wall (m).

6.4.3 Considerations for EPS Light Weight Embankment Fill

As discussed below in Section 6.5.2 of this report, the use of expanded polystyrene (EPS) Geofoam light weight embankment fill is proposed as an alternative for mitigating the potential roadway settlements of the existing approach embankments due to compression of the underlying clay deposits. Further details on the placement of the EPS backfill are provided in Section 6.5.2, however, based on the proposed grade increase of up to 0.7 m in the area of the existing approach embankments, an estimated 1.2 m thickness of EPS would be required within the core of the final embankments, with potential greater thickness below the widened embankment. The thickness of the EPS below the widened embankments will be determined when the General Arrangement drawing is available and will be included in the final report.

In regards to the lateral earth pressures, the low unit weight and relatively high mechanical strength characteristics of the EPS blocks (in comparison to soil) will alter the design lateral earth pressures. For design purposes, the EPS could be assumed to have a unit weight of 1 kN/m^3; this low unit weight could be considered in the calculation of the vertical stress level in the underlying granular backfill, and thus the horizontal lateral pressure applied to the abutment wall. Furthermore, because the EPS blocks would hold a vertical face without support, the lateral earth pressure applied by the EPS itself could be quite minor, resulting only from the resistance to lateral expansion of the material under vertical loading (i.e., from the ‘poisson’ effect), which is however difficult to quantify (and highly dependent on how tightly fitting the EPS blocks are placed against the abutment). It is therefore considered that the lateral earth pressures from the EPS can be neglected. Where the backfill is relied upon to provide passive resistance to the abutment, the contribution of the EPS itself should also be neglected, but the effect of the lower unit weight and lower vertical stress level must be considered in assessing the passive resistance from the underlying backfill

6.5 Approach Embankments

6.5.1 Subgrade Preparation and Embankment Construction

It is understood that the overall grade of Highway 17 may be raised up to 0.7 m.

The subsurface conditions at the site consist of sand and gravel layered with silty clay embankment fill and/or organic deposits underlain by layered compressible silty clay to clay, and limestone bedrock. The organic deposits were encountered in boreholes on both side of the river, with the thickest layer (3.2 m) at the toe of the embankment slope of the southwest quadrant of the bridge. The compressible silty clay to clay deposit was encountered beneath the fill and organic deposits at all of the borehole locations.

- It is recommended, and considered quite feasible, to remove the organic deposits below the footprint of any potential embankment widening, prior to placement of the new embankment fill. However, to ensure global stability, the excavation should be carried out in strips with 10 m maximum width and keyed into the existing slope. Backfilling should be carried out simultaneously to ensure stability.
- Full removal of the existing embankment and underlying organic deposits would materially reduce the potential for additional settlement of these deposits due to any grade raise fill (for Option 1 only). However, given that these organic soils have been buried beneath the existing fills for over 60 years, the additional potential compression from these soils is considered to be modest and removal of the alluvium is therefore not justified. The fill subgrade should nonetheless be proof-rolled to identify any particularly compressible subgrade areas and to compact the remaining soils.
- Compression of and longer-term decomposition of the organic content within the alluvium could lead to additional ground surface settlement. However, given that these organic soils have been buried beneath the existing median fills for approximately 60 years, the additional potential compression from these soils is considered to be modest and removal of the alluvium is therefore not justified.

Any new embankment fill for the approach embankments should be placed and compacted in accordance with OPSS.PROV 206 *(Grading)* and OPSS.PROV 501 *(Compacting)*. Benching of the existing embankment side slopes should be carried out to “key in” the new fill materials in areas where the embankment is widened, in accordance with OPSD 208.010 *(Benching of Earth Slopes).*

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS.PROV 804 *(Seed and Cover).*

6.5.2 Settlement

Settlement of the existing embankments has occurred over time since the existing bridge construction in 1954. The additional loading imposed by the proposed grade raise and embankment widening would result in further consolidation settlement of the underlying compressible soils. Notwithstanding the relatively limited amount of additional load that the proposed filling will apply to the underlying subgrade, the clay deposit which underlies this site is compressible and significant settlements are expected. Based on the indicated grade raise and the assessed existing stress level and preconsolidation pressure profile within the silty clay deposit, the calculated primary consolidation settlements are estimated to be in the order of 0.7 to 1.0 m. Even with no grade raise, some ongoing consolidation settlements are expected to occur with the widening of the embankment.

As shown in the photo below, the existing approach embankments have settled and resulted in differential settlement of the roadway with respect to the bridge.

To mitigate these anticipated settlements, consideration may be given to the use of lightweight fill materials (such as EPS Geofoam) for embankment construction, within the footprint of the approach embankments, to reduce the stress increase on the compressible soils to a level that would result in settlements within acceptable tolerances. A portion of the existing embankment core may be subexcavated and replaced with lightweight fill such that there is no net increase in load on the underlying soil. Based on the proposed grade increase of 0.7 m, an estimated 1.2 m thickness of EPS Geofoam would be required within the final embankments. However, the total thickness of embankment fill removal and lightweight fill replacement will be dependent on the type of lightweight fill and total grade raise, and should be verified. Provided that 1.2 m of EPS Geofoam lightweight fill is used below the pavement structure, the post-construction settlement could be limited to less than about 25 mm.

An NSSP for the supply and installation of EPS fill should be included in the contract documents and a sample has been provided in Appendix D of this report.

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

The EPS Geofoam blocks would need to be protected with a concrete slab at pavement subgrade level, to distribute the traffic loads. A thickness of 125 mm is typical for the protective slab. A sufficient pavement granular thickness is also required above the EPS to limit the potential for premature icing of the roadway due to the insulating properties of the Geofoam. From that perspective, a minimum of 900 millimetres combined thickness of granular base and subbase should be planned.

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

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

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

As an alternative to mitigating the potential settlements using EPS lightweight fill, and if time allows, the embankments could be preloaded, the settlements allowed to occur (and monitored), and then the bridge and pavements constructed once the settlements were complete (or sufficient settlement had occurred). A temporary surcharge above the proposed roadway level would need to be placed for the preload period, to apply a stress equivalent to the future pavement weight and also to accelerate the settlements. The temporary surcharge above the design roadway level would need to be placed for the preload period to:

1) Apply a stress equivalent to the ‘design’ level, after accounting for the future pavement weight and for any potential groundwater level lowering (not anticipated at this site);
2) To potentially accelerate the settlements; and,
3) To reduce the potential for post-construction ‘creep’ settlements which could occur in the long term.

The magnitude of the surcharge would therefore depend on:

1) The pavement design;
2) The duration of preloading that is desired; and,

6.5.3 Global Stability

Provided that the approximately 6 m high approach embankment side slopes are maintained no steeper than 2H:1V, and that the surficial organic deposits are removed from within the footprint of the widened embankment areas to the south and north of the existing highway alignment, the embankments should have an adequate minimum factor of safety of at least 1.3 under static conditions.

6.6 Construction Considerations

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

6.6.1 Excavation and Temporary Protection Systems

It is assumed that the underside of pile cap level for the new abutments will be at about Elevation 82 m and that the pile cap level for the piers will be below the river bed. The excavations for the pile caps will extend through the existing embankment fills, through the organic silt and clay, and into the firm to very stiff silty clay to clay.

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. It is anticipated that the excavations will experience potentially significant groundwater inflow since the pile cap levels will likely be below the measured groundwater level and also below the normal Mississippi River water level. The water-bearing organic silt and clay deposits would be classified as Type 4 soils in accordance with the OHSA and the excavations would have to be made with side slopes no steeper than 3 horizontal to 1 vertical (3H:1V) unless groundwater control measures are put in place. The embankment fills and underlying silty clay to clay would be classified as Type 3 soils and, according to OHSA, temporary excavations 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, and excavations in these materials should be sloped no steeper than 3H:1V.

It is important that soil not be stock piled adjacent to the excavations. The weight of that soil could lead to shearing of the underlying clay and basal instability of shored excavations or deep-seated instability of open cut side slopes.

6.6.2 Groundwater and Surface Water Control

As discussed above, the excavations will extend below the measured groundwater level as well as below the normal level of the Mississippi River. Relatively significant groundwater inflow may be experienced from the embankment fill (below the ground water level) and organic deposits, which are considered to have a relatively high hydraulic conductivity. Excavations will therefore require groundwater control. Excavations behind the existing abutments, and around the piers (if required for structure removal) would also need to deal with flow from the river.

It is therefore expected that the excavations will need to be separated from the river using a sheet pile coffer dam. That sheet piling will need to extend into the underlying silty clay to clay deposit.

If the sheet piling were to fully surround the excavation, and not just separate the excavation from the river (i.e., 4-sided coffer dam), the rate of groundwater inflow to the excavation would be greatly reduced.

An NSSP should be included in the contract documents, alerting the contractor to this issue, and a sample Non-Standard Special Provision (NSSP) is included in Appendix D.

Based on the subsurface soil and groundwater conditions, it is anticipated that the dewatering rate will exceed 50 m^3/day, and therefore a Permit to Take Water (PTTW) will be required for this site.

6.6.3 Subgrade Protection

Within the excavations, the subgrade will likely be wet and sensitive to disturbance, which could impact on trafficability with the excavation. In that case, the subgrade should be protected with a working pad of granular fill and a Class II woven geotextile (per OPSS 1860) should be placed on the subgrade to protect it. This requirement can be addressed either with a note on the General Arrangement drawing, or with an NSSP.

6.6.4 Erosion and Scour Protection

The existing organic deposits and granular fill materials that make up the approach embankments at the site are expected to be susceptible to erosion and scour under the design flood/flow velocities. The requirements for design of erosion/scour protection should be assessed by the hydraulic design engineer. As a minimum, it is recommended that erosion protection (e.g., rip-rap or granular sheeting) be provided on the river banks 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.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Kim Lesage, P.Eng., and was reviewed by Mr. Murty Devata, P.Eng., a senior consultant with Golder. Mr. Fintan Heffernan, P.Eng., the Designated MTO Foundations Contact for Golder for this project, conducted an independent quality review of the report.

GOLDER ASSOCIATES LTD.

Kim Lesage, P.Eng.
Geotechnical Engineer

Fintan Heffernan, P.Eng.
Designated MTO Foundations Contact

SN/SG/KSL/MSD/FJH/mvrd
n:\active\2012\1121 - geotechnical\12-1121-0193 dillon mega 3 eastern region\foundations\5 - reports\contract f - mississippi river bridge\12-1121-0193-1130 site 3-003 draft july 2017 fidr.docx

Table 1
Comparison of Foundation Alternatives

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Option 1 Steel H-piles driven to bedrock	▪ Feasible for support of bridge replacement. ▪ Preferred option from a foundations perspective.	▪ High geotechnical resistances. ▪ Negligible foundation settlement. ▪ Piles can be spliced to account for extensive depth. ▪ Allows for integral or semi-intergral abutment construction.	▪ Very long piles required to penetrate clay layer and reach bedrock. ▪ Possibility of battered piles sliding along bedrock if not provided with suitable driving points.	▪ Moderate cost.	▪ Low to medium risk option.
Option 2 Steel pipe (tube) piles, driven to found in bedrock	▪ Feasible for support of bridge replacement.	▪ High geotechnical resistances. ▪ Negligible foundation settlement. ▪ Piles can be spliced to account for extensive depth. ▪ Allows for semi-integral abutment construction and possibly integral abutment construction.	▪ Relatively large displacement pile compared to steel H-piles. ▪ Very long piles required to penetrate clay layer and reach bedrock. ▪ Possibility of battered piles sliding along bedrock if not provided with suitable driving points.	▪ Moderate to expensive cost.	▪ Medium risk option.
Option 3 Drilled Pier (Caisson) Foundations	▪ Feasible for support of bridge replacement.	▪ Higher geotechnical resistances. ▪ Negligible foundation settlement.	▪ Very long caissons required to penetrate clay layer and reach bedrock. ▪ Permanent casings required to construct caissons. ▪ Coring or churn drilling may be required to form nominal socket in bedrock.	▪ Most expensive option.	▪ Higher risk option.

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

CONT No.
WP No.4121–10–01

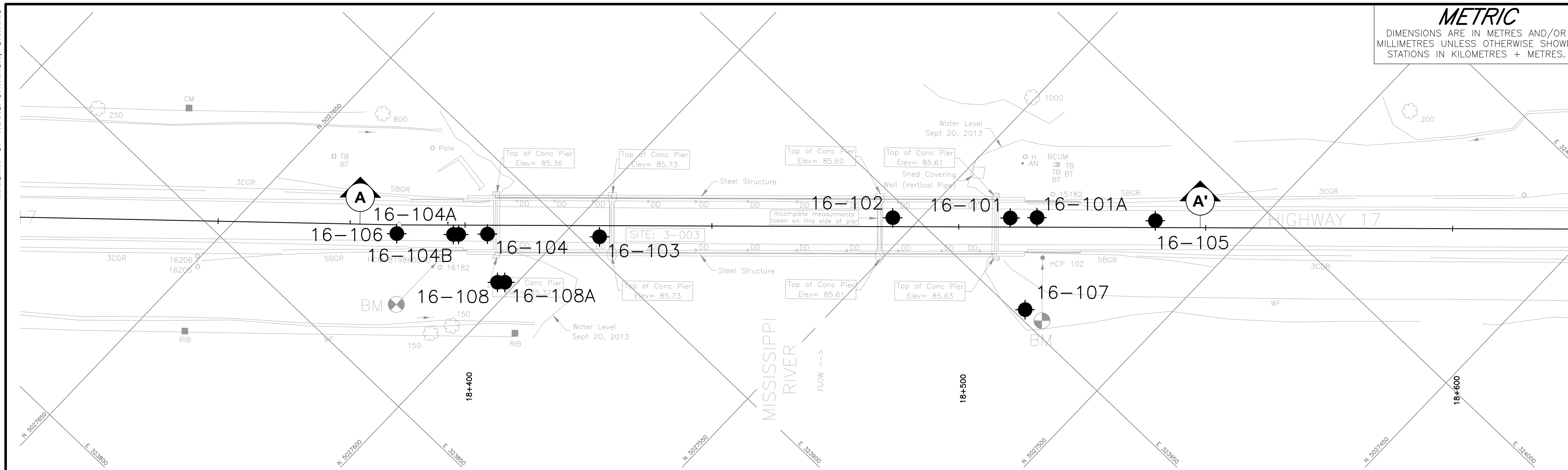
SHEET

BRIDGE REHABILITATION

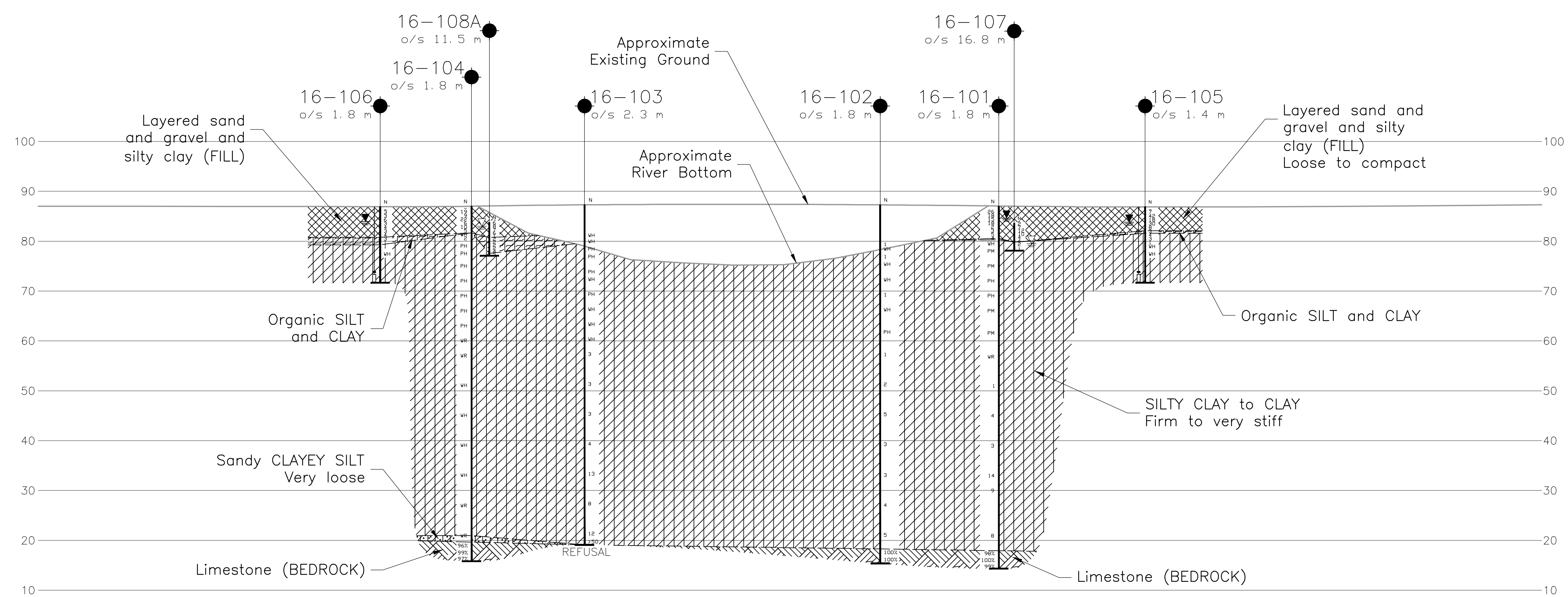
HIGHWAY 17 BRIDGE AT MISSISSIPPI RIVER

BOREHOLE LOCATIONS AND SOIL STRATA

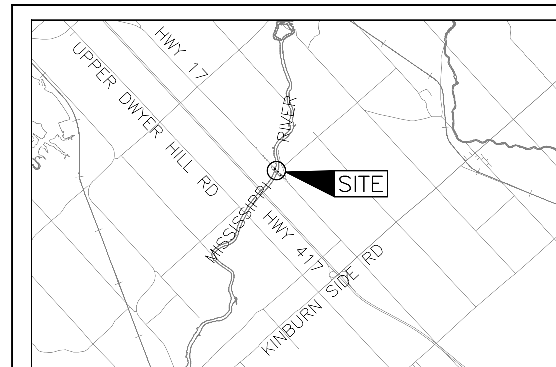
Golder Associates Ltd.
OTTAWA ONTARIO, CANADA



PLAN
SCALE
10 0 10 20 m



PROFILE A–A'
HORIZ. SCALE
10 0 10 20 m
5 0 5 10 m
VERT. SCALE



KEY PLAN
SCALE 2 0 2 4 KM

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 Oct. 27, 2016
- WL upon completion of drilling
- Seal
- Piezometer

DRAFT

BOREHOLE CO–ORDINATES

No.	ELEVATION	NORTHING	EASTING
16–101	87.0	5027536.865	323963.182
16–101A	87.0	5027533.02	323967.0176
16–102	87.3	5027554.112	323946.775
16–103	87.3	5027594.376	323902.96
16–104	87.0	5027611.18	323887.698
16–104A	87.0	5027615.334	323883.5994
16–104B	87.0	5027616.092	323882.8728
16–105	86.9	5027515.239	323983.077
16–106	86.9	5027624.503	323875.098
16–107	83.6	5027521.848	323951.84
16–108	83.8	5027602.95	323882.038
16–108A	83.8	5027601.86	323883.0684

NOTES

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

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

REFERENCE

Base plan provided in digital format by Dillion, drawing file no. 4121 Base.dwg received October 11, 2013.

NO.	DATE	BY	REVISION

Geocres No.

HWY. 17		PROJECT NO. 12–1121–0193–1130	DIST.
SUBM'D. KSL	CHKD. KSL	DATE: 5/11/2017	SITE: 3–003
DRAWN: JM	CHKD. KSL	APPD. FJH	DWG. 1

APPENDIX A

Borehole and Drillhole Records

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

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

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
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (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 (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

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

Notes:
1. $\tau = c' + \sigma' \tan \phi'$
2. shear strength = (compressive strength)/2

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to “A” size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)
A electronic cone penetrometer with a 60° conical tip and a project 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 (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index Relative Density	N Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils
Consistency

	c_u, s_u kPa	c_u, s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
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 (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

WEATHERINGS STATE

Fresh: no visible sign of 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 and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
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	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)

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 varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

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

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

Golder Associates

RECORD OF BOREHOLE No 16-101

METRIC

PROJECT	12-1121-0193-1130		
G.W.P.	4121-10-01	LOCATION	N 5027536.9 ;E 323963.2
DIST	Eastern HWY 17	BOREHOLE TYPE	Rotary Drill (Hollow Stem)/HW Casing, NQ Core
DATUM	Geodetic	DATE	August 15-18, 2016
ORIGINATED BY	RI	COMPILED BY	ZS
CHECKED BY			

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	UNIT WEIGHT	GR	SA	SI	CL
87.0	GROUND SURFACE									
0.0	ASPHALTIC CONCRETE									
86.7	PORTLAND CEMENT CONCRETE									
0.3	Gravelly silty sand, contains wood fragments and cobbles (FILL) Compact Brown Wet	1	SS	26						
		2	SS	18						
		3	SS	15						
		4	RC	DD						
84.0 / 3.1	Sand and gravel, some silt (FILL) Compact to very loose Brown Moist to wet	5	SS	18			36	53	8	3
		6	SS	2						
82.3 / 4.7	Silty clay, trace sand and gravel, contains rootlets (FILL) Dark grey-brown Wet	7	SS	3						
		8	SS	5						
80.6 / 6.4	Organic SILTY CLAY, trace sand, trace rootlets Dark grey-brown to black Wet	9	SS	4		OC = 10.2%				
80.3 / 6.7	SILTY CLAY to CLAY (Weathered Crust) Stiff to firm Grey with black organic mottling Wet	10	SS	WH						
79.2 / 7.8	SILTY CLAY to CLAY Stiff to firm Grey with black organic mottling Wet	11	SS	PM						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEVATION SCALE: 86, 85, 84, 83, 82, 81, 80, 79, 78

Continued Next Page

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 16-101

SHEET 2 OF 9

METRIC

PROJECT 12-1121-0193-1130
G.W.P. 4121-10-01
DIST Eastern HWY 17
DATUM Geodetic

LOCATION N 5027536.9 ;E 323963.2
BOREHOLE TYPE Rotary Drill (Hollow Stem)/HW Casing, NQ Core
DATE August 15-18, 2016

ORIGINATED BY RI
COMPILED BY ZS
CHECKED BY

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

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
	SILTY CLAY to CLAY Stiff to firm Grey with black organic mottling Wet					76				
			12	SS	PM	75	0	1	15	84
						74				
						73				
			13	TP	PH	72				
						71				
						70				
			14	SS	PH	69				
						68				

Continued Next Page

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

RECORD OF BOREHOLE No 16-101

SHEET 3 OF 9

METRIC

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01 LOCATION N 5027536.9 ;E 323963.2 ORIGINATED BY RI

DIST Eastern HWY 17 BOREHOLE TYPE Rotary Drill (Hollow Stem)/HW Casing, NQ Core COMPILED BY ZS

DATUM Geodetic DATE August 15-18, 2016 CHECKED BY

SOIL PROFILE: ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SILTY CLAY to CLAY Stiff to firm Grey with black organic mottling Wet										
			15	SS	PM		66				
							65				
							64				
							63				
							62				
			16	SS	PM						
							61				
							60				
							59				
							58				

Continued Next Page

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

RECORD OF BOREHOLE No 16-101

METRIC

PROJECT	12-1121-0193-1130				
G.W.P.	4121-10-01	LOCATION	N 5027536.9 ;E 323963.2	ORIGINATED BY	RI
DIST	Eastern HWY 17	BOREHOLE TYPE	Rotary Drill (Hollow Stem)/HW Casing, NQ Core	COMPILED BY	ZS
DATUM	Geodetic	DATE	August 15-18, 2016	CHECKED BY	

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY to CLAY Stiff to firm Grey with black organic mottling Wet	17	SS	WR				
					56			
					55			
					54			
					53			
					52			
		18	SS	1	51			
					50			
					49			
					48			

Continued Next Page

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 16-101

SHEET 5 OF 9 **METRIC**

PROJECT 12-1121-0193-1130
G.W.P. 4121-10-01
LOCATION N 5027536.9 ;E 323963.2
ORIGINATED BY RI
DIST Eastern HWY 17
BOREHOLE TYPE Rotary Drill (Hollow Stem)/HW Casing, NQ Core
COMPILED BY ZS
DATUM Geodetic
DATE August 15-18, 2016
CHECKED BY

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L | WATER CONTENT (%) 25 50 75 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	SILTY CLAY to CLAY Stiff to firm Grey with black organic mottling Wet					46
			19	SS	4	45
						44, 43, 42, 41, 40
39.3 47.7	SILTY CLAY to CLAY Very stiff Grey Wet		20	SS	3	39
						38

Field vane readings near elev. 44.5: × ~30, + ~92; × ~31, + ~88. Near elev. 38.5: >96+, >96+.

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IM\GINT\PHASE 1130\1211210193-1130.GPJ GAL-GTA.GDT 5/1/17

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 16-101

METRIC

PROJECT	12-1121-0193-1130				
G.W.P.	4121-10-01	LOCATION	N 5027536.9 ;E 323963.2	ORIGINATED BY	RI
DIST	Eastern HWY 17	BOREHOLE TYPE	Rotary Drill (Hollow Stem)/HW Casing, NQ Core	COMPILED BY	ZS
DATUM	Geodetic	DATE	August 15-18, 2016	CHECKED BY	

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY to CLAY Very stiff Grey Wet										
							36				
							35				
							34				
			21	SS	14		33				
							32				
							31				
			22	SS	9		30				
							29				
							28				

Dynamic cone plot scale: 20 40 60 80 100. Shear strength legend: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × REMOULDED; scale 20 40 60 80 100. Plastic limit W_P, natural moisture content W, liquid limit W_L; water content scale 25 50 75.

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IM\GINT\PHASE 1130\1211210193-1130.GPJ GAL-GTA.GDT 5/1/17

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 16-101

SHEET 7 OF 9

METRIC

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01

LOCATION N 5027536.9 ;E 323963.2

ORIGINATED BY RI

DIST Eastern HWY 17

BOREHOLE TYPE Rotary Drill (Hollow Stem)/HW Casing, NQ Core

COMPILED BY ZS

DATUM Geodetic

DATE August 15-18, 2016

CHECKED BY

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY to CLAY Very stiff Grey Wet				
21.3 65.8	SILTY CLAY, trace sand, contains thick laminations to very thin beds of sandy silt Very stiff Grey Wet	23	SS	8	
18.0 69.1	Sandy SILT, some gravel to gravelly Grey Wet	1	RC	REC 99%	RQD = 98%

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

Elevation scale: 26, 25, 24, 23, 22, 21, 20, 19, 18

DRAFT

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 16-101

METRIC

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01 LOCATION N 5027536.9 ;E 323963.2 ORIGINATED BY RI

DIST Eastern HWY 17 BOREHOLE TYPE Rotary Drill (Hollow Stem)/HW Casing, NQ Core COMPILED BY ZS

DATUM Geodetic DATE August 15-18, 2016 CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Limestone (BEDROCK)		1	RC							
	Bedrock cored from depths of 69.1 m to 72.7 m For bedrock coring details refer to Record of Drillhole 16-101		2	RC	REC 100%		16				RQD = 100%
			3	RC	REC 100%		15				RQD = 99%
14.3 72.7	END OF BOREHOLE										

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

RECORD OF DRILLHOLE: 16-101

PROJECT: 12-1121-0193-1130
LOCATION: N 5027536.9 ;E 323963.2
INCLINATION: -90° AZIMUTH: ---
DRILLING DATE: August 15-18, 2016
DRILL RIG: CME 75
DRILLING CONTRACTOR: Downing Drilling
DATUM: Geodetic

NOTE: For abbreviations, symbols and descriptions refer to **LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY**

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN
		BEDROCK SURFACE	17.92		
70–72	Rotary Drill, NQ Core	Limestone (BEDROCK), occasional black shale partings Fresh Thinly to medium bedded Grey Fine to medium grained Non-porous	69.08	1	100
				2	100
			14.32	3	100
		END OF DRILLHOLE	72.68		

NOTES:

1. A creosote odour was observed in fill at 2.3 to 3.1 m depth.

2. Artesian flow conditions were observed 4.1 m above ground surface within casing installed to 69.1 m depth and 0.5 m above ground surface within casing installed to 65.8 m depth prior to sealing borehole.

Column headings: SYMBOLIC LOG; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER; DISCONTINUITY DATA (DIP w.r.t. CORE AXIS; TYPE AND SURFACE DESCRIPTION; Jr; Ja); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WEATHERING INDEX (W1 W2 W3 W4 W5 W6); FEATURES

Depth scale: 70, 71, 72, 73, 74, 75, 76, 77, 78, 79

DRAFT

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: RI
CHECKED:

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 16-101A

SHEET 1 OF 1 **METRIC**

PROJECT 12-1121-0193-1130
G.W.P. 4121-10-01
LOCATION N 5027533.0 ;E 323967.0
ORIGINATED BY RI
DIST Eastern HWY 17
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)
COMPILED BY ZS
DATUM Geodetic
DATE August 25, 2016
CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
87.0	GROUND SURFACE													
0.0	For soil stratigraphy from 0.0 m to 0.8 m see Record of Borehole 16-101													
86.2 0.8	Sand and gravel, some silt (FILL) Dense to compact Brown Moist		1	SS	38		86							
			2	SS	45		85		o		42	41	13	4
			3	SS	14		84							
			4	SS	10									
83.2 3.8	Sand, some gravel and silt (FILL) Loose Brown to grey Wet		5	SS	9		83		o		13	77	8	2
82.3 4.7	CLAYEY SILT to SILTY CLAY, some sand, trace organic matter (Weathered Crust) Very stiff Dark grey-brown Moist		6	SS	3		82							
81.8 5.2	SILTY CLAY to CLAY, trace sand, contains rootlets (Weathered Crust) Very stiff Grey-brown Wet		7	SS	9									
81.2 5.8	END OF BOREHOLE													

Plot legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100. PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75. REMARKS & GRAIN SIZE DISTRIBUTION (%).

DRAFT

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

RECORD OF BOREHOLE No 16-102

SHEET 1 OF 9 **METRIC**

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01 LOCATION N 5027554.1 ;E 323946.8 ORIGINATED BY DG

DIST Eastern HWY 17 BOREHOLE TYPE Wash Boring, PW/HW Casing, Rotary Drill, NQ Core COMPILED BY ZS

DATUM Geodetic DATE August 15, 2016 CHECKED BY

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
87.3	BRIDGE DECK									
0.0	ASPHALTIC CONCRETE									
87.1 / 0.2	PORTLAND CEMENT CONCRETE					87				
	AIR					86, 85, 84				
83.1 / 4.2	WATER					83, 82, 81, 80				
79.8 / 7.5	Probable Silty Clay (no sample recovered)	1	SS	1		79				
78.8 / 8.5	SILTY CLAY to CLAY, trace sand Grey with black organic mottling Wet Stiff	2	SS	WH		78				

Continued Next Page

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

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 16-102

SHEET 2 OF 9 **METRIC**

PROJECT 12-1121-0193-1130 | LOCATION N 5027554.1 ;E 323946.8 | ORIGINATED BY DG

G.W.P. 4121-10-01 | BOREHOLE TYPE Wash Boring, PW/HW Casing, Rotary Drill, NQ Core | COMPILED BY ZS

DIST Eastern HWY 17 | DATE August 15, 2016 | CHECKED BY

DATUM Geodetic

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS
	SILTY CLAY to CLAY, trace sand Grey with black organic mottling Wet Stiff	3	SS	1	77	
76.6 10.7	SILTY CLAY to CLAY Firm to stiff Grey with black organic mottling Wet	4	SS	WH	76, 75, 74, 73	
		5	SS	WH	72, 71, 70	
69.6 17.7	SILTY CLAY to CLAY Stiff Grey with black organic mottling Wet	6	SS	1	69, 68	

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL; STRAT PLOT

DRAFT

Continued Next Page

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

RECORD OF BOREHOLE No 16-102

SHEET 3 OF 9

METRIC

PROJECT 12-1121-0193-1130			
G.W.P. 4121-10-01	LOCATION N 5027554.1 ;E 323946.8	ORIGINATED BY DG
DIST Eastern HWY 17	BOREHOLE TYPE Wash Boring, PW/HW Casing, Rotary Drill, NQ Core	COMPILED BY ZS
DATUM Geodetic	DATE August 15, 2016	CHECKED BY

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

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
	--- CONTINUED FROM PREVIOUS PAGE ---						
	SILTY CLAY to CLAY Stiff Grey with black organic mottling Wet						67
			7	SS	WH		66
							65
							64
							63
							62
			8	TP	PH		61
							60
							59
							58

Continued Next Page

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

RECORD OF BOREHOLE No 16-102

METRIC

PROJECT 12-1121-0193-1130
G.W.P. 4121-10-01
LOCATION N 5027554.1 ;E 323946.8
ORIGINATED BY DG
DIST Eastern HWY 17
BOREHOLE TYPE Wash Boring, PW/HW Casing, Rotary Drill, NQ Core
COMPILED BY ZS
DATUM Geodetic
DATE August 15, 2016
CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---													
	SILTY CLAY to CLAY Stiff Grey with black organic mottling Wet		9	SS	1		57							
							56							
							55							
							54							
							53							
							52							
			10	SS	2						0	1	67	32
							51							
							50							
							49							
							48							

Legend: ○ UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × REMOULDED. DCPT scale: 20 40 60 80 100. Plastic limit W_P, natural moisture content W, liquid limit W_L; water content scale: 25 50 75. REMARKS & GRAIN SIZE DISTRIBUTION (%).

Continued Next Page

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

RECORD OF BOREHOLE No 16-102

METRIC

PROJECT 12-1121-0193-1130			
G.W.P. 4121-10-01	LOCATION N 5027554.1 ;E 323946.8	ORIGINATED BY DG
DIST Eastern HWY 17	BOREHOLE TYPE Wash Boring, PW/HW Casing, Rotary Drill, NQ Core	COMPILED BY ZS
DATUM Geodetic	DATE August 15, 2016	CHECKED BY

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY to CLAY Stiff Grey with black organic mottling Wet						47				
							46				
			11	SS	5		45				
								× +			
								× +			
							44				
							43				
							42				
							41				
							40				
			12	SS	3						
39.0 48.3	SILTY CLAY to CLAY Stiff to very stiff Grey Wet						39				
								>96+			
								>96+			
							38				

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) 25 50 75

DRAFT

Continued Next Page

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

RECORD OF BOREHOLE No 16-102

SHEET 6 OF 9 **METRIC**

PROJECT 12-1121-0193-1130
G.W.P. 4121-10-01
LOCATION N 5027554.1 ;E 323946.8
ORIGINATED BY DG
DIST Eastern HWY 17
BOREHOLE TYPE Wash Boring, PW/HW Casing, Rotary Drill, NQ Core
COMPILED BY ZS
DATUM Geodetic
DATE August 15, 2016
CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SILTY CLAY to CLAY Stiff to very stiff Grey Wet						37				
							36				
							35				
							34				
			13	SS	3		33	+	○ I		
							32				
							31				
							30				
							29				
							28				

Continued Next Page

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

RECORD OF BOREHOLE No 16-102

SHEET 7 OF 9

METRIC

PROJECT 12-1121-0193-1130
G.W.P. 4121-10-01
DIST Eastern HWY 17
DATUM Geodetic
LOCATION N 5027554.1 ;E 323946.8
BOREHOLE TYPE Wash Boring, PW/HW Casing, Rotary Drill, NQ Core
DATE August 15, 2016
ORIGINATED BY DG
COMPILED BY ZS
CHECKED BY

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY to CLAY Stiff to very stiff Grey Wet	14	SS	4	27		
					26		
					25		
					24		
					23		
22.3 65.0	SILTY CLAY, some sand Very loose Grey Wet				22		
		15	SS	5	21	o (approx. 26)	0 15 34 51
					20		
					19		
18.3 69.0	Limestone (BEDROCK) Bedrock cored from depths of 69.0 m to 71.9 m For bedrock coring details refer to Record of Drillhole 16-102	1	RC	REC 100%	18		RQD = 100%

Continued Next Page

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

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

Golder Associates

RECORD OF BOREHOLE No 16-102

SHEET 8 OF 9 **METRIC**

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01 LOCATION N 5027554.1 ;E 323946.8 ORIGINATED BY DG

DIST Eastern HWY 17 BOREHOLE TYPE Wash Boring, PW/HW Casing, Rotary Drill, NQ Core COMPILED BY ZS

DATUM Geodetic DATE August 15, 2016 CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L / WATER CONTENT (%) (25 50 75)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Limestone (BEDROCK) Bedrock cored from depths of 69.0 m to 71.9 m For bedrock coring details refer to Record of Drillhole 16-102		1	RC	REC 100%		17				
			2	RC	REC 100%		16				RQD = 100%
15.4 71.9	END OF BOREHOLE										

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

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IM\GINT\PHASE 1130\1211210193-1130.GPJ GAL-GTA.GDT 5/1/17

PROJECT: 12-1121-0193-1130

RECORD OF DRILLHOLE: 16-102

LOCATION: N 5027554.1 ;E 323946.8

DRILLING DATE: August 15, 2016

DATUM: Geodetic

DRILL RIG: CME 75

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Downing Drilling

NOTE: For abbreviations, symbols and descriptions refer to **LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY**

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	WEATHERING INDEX	FEATURES
		BEDROCK SURFACE		18.28													
	Rotary Drill NQ Core	Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Fine grained Non-porous		69.02													
70					1	100											
71					2	100											
				15.37													
72		END OF DRILLHOLE		71.93													
73																	
74																	
75																	
76																	
77																	
78																	
79																	

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: DG

CHECKED:

Golder Associates

RECORD OF BOREHOLE No 16-103

SHEET 1 OF 7 **METRIC**

PROJECT	12-1121-0193-1130		
G.W.P.	4121-10-01	LOCATION	N 5027594.4 ;E 323903.0
DIST	Eastern HWY 17	BOREHOLE TYPE	Wash Boring, PW/HW Casing, Rotary Drill
DATUM	Geodetic	DATE	August 18-19, 2016
ORIGINATED BY	DG	COMPILED BY	ZS
CHECKED BY			

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

ELEV / DEPTH	DESCRIPTION
87.3	BRIDGE DECK
0.0	ASPHALTIC CONCRETE
87.0 / 0.3	PORTLAND CEMENT CONCRETE
	AIR
83.1 / 4.2	WATER
81.5 / 5.8	Organic SILT Very loose Wet
79.7 / 7.6	SILTY CLAY to CLAY, contains silty sand layers Firm Grey Wet

NUMBER	TYPE	"N" VALUES	Approx. Elevation	Water content / Remarks
1	SS	WH	81.0	126.2 ; OC = 10.7%
2	SS	WH	80.0	111.4
3	TP	PH	78.5	

Elevation scale: 87, 86, 85, 84, 83, 82, 81, 80, 79, 78

Field vane (+) about 40 kPa and remoulded (×) below 20 kPa, plotted at about El. 79.1 and El. 77.6.

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 16-103

METRIC

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01 LOCATION N 5027594.4 ;E 323903.0 ORIGINATED BY DG

DIST Eastern HWY 17 BOREHOLE TYPE Wash Boring, PW/HW Casing, Rotary Drill COMPILED BY ZS

DATUM Geodetic DATE August 18-19, 2016 CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	--- CONTINUED FROM PREVIOUS PAGE ---				
	SILTY CLAY to CLAY, contains silty sand layers Firm Grey Wet	4	TP	PH	77
76.4 10.9	SILTY CLAY to CLAY Firm Grey with black organic mottling Wet				76
					75
		5	TP	PH	74
					73
		6	SS	WH	
71.8 15.5	SILTY CLAY to CLAY Stiff Grey with black organic mottling Wet				72
					71
					70
		7	TP	PH	69
					68

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

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 16-103

METRIC

PROJECT 12-1121-0193-1130			
G.W.P. 4121-10-01	LOCATION N 5027594.4 ;E 323903.0	ORIGINATED BY DG	
DIST Eastern HWY 17	BOREHOLE TYPE Wash Boring, PW/HW Casing, Rotary Drill	COMPILED BY ZS	
DATUM Geodetic	DATE August 18-19, 2016	CHECKED BY	

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

--- CONTINUED FROM PREVIOUS PAGE ---

SILTY CLAY to CLAY
Stiff
Grey with black organic mottling
Wet

NUMBER	TYPE	"N" VALUES
8	SS	WH
9	SS	WH
10	SS	WH
11	SS	3

Elevation scale: 67, 66, 65, 64, 63, 62, 61, 60, 59, 58

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 16-103

METRIC

PROJECT 12-1121-0193-1130
G.W.P. 4121-10-01
DIST Eastern HWY 17
DATUM Geodetic
LOCATION N 5027594.4 ;E 323903.0
BOREHOLE TYPE Wash Boring, PW/HW Casing, Rotary Drill
DATE August 18-19, 2016
ORIGINATED BY DG
COMPILED BY ZS
CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) (25 50 75)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SILTY CLAY to CLAY Stiff Grey with black organic mottling Wet						57				
							56				
							55				
							54				
							53				
							52				
			12	SS	3						
							51				
							50				
							49				
							48				

Continued Next Page

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

RECORD OF BOREHOLE No 16-103

SHEET 5 OF 7

METRIC

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01

LOCATION N 5027594.4 ;E 323903.0

ORIGINATED BY DG

DIST Eastern HWY 17

BOREHOLE TYPE Wash Boring, PW/HW Casing, Rotary Drill

COMPILED BY ZS

DATUM Geodetic

DATE August 18-19, 2016

CHECKED BY

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY to CLAY Stiff Grey with black organic mottling Wet						47				
							46				
			13	SS	3						
45.1 42.2	SILTY CLAY to CLAY Very stiff Grey Wet						45	>96+			
								>96+			
							44				
							43				
							42				
							41				
							40				
			14	SS	4				o		
							39				
							38				

Legend: PLASTIC LIMIT W_P; NATURAL MOISTURE CONTENT W; LIQUID LIMIT W_L. SHEAR STRENGTH: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × REMOULDED.

Continued Next Page

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

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

RECORD OF BOREHOLE No 16-103

METRIC

PROJECT 12-1121-0193-1130			
G.W.P. 4121-10-01	LOCATION N 5027594.4 ;E 323903.0	ORIGINATED BY DG	
DIST Eastern HWY 17	BOREHOLE TYPE Wash Boring, PW/HW Casing, Rotary Drill	COMPILED BY ZS	
DATUM Geodetic	DATE August 18-19, 2016	CHECKED BY	

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY to CLAY Very stiff Grey Wet						37				
							36				
							35				
							34				
			15	SS	13		33				
							32				
							31				
							30				
							29				
							28				
			16	SS	8						

Shear strength legend: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED. Plot scales: 20 40 60 80 100. Water content: PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; 25 50 75.

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 16-103

SHEET 7 OF 7

METRIC

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01 LOCATION N 5027594.4 ;E 323903.0 ORIGINATED BY DG

DIST Eastern HWY 17 BOREHOLE TYPE Wash Boring, PW/HW Casing, Rotary Drill COMPILED BY ZS

DATUM Geodetic DATE August 18-19, 2016 CHECKED BY

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	PLASTIC LIMIT / WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY to CLAY Very stiff Grey Wet					
21.4 65.9	SILT and CLAY, some sand Compact Grey Wet	17	SS	12		16 47 37
19.1 68.2	END OF BOREHOLE SAMPLER REFUSAL	18	SS	>50		

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

Elevation scale: 27, 26, 25, 24, 23, 22, 21, 20

DRAFT

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

Golder Associates

RECORD OF BOREHOLE No 16-104

SHEET 1 OF 9 **METRIC**

PROJECT 12-1121-0193-1130
G.W.P. 4121-10-01
LOCATION N 5027611.2 ;E 323887.7
ORIGINATED BY RI
DIST Eastern HWY 17
BOREHOLE TYPE Power Auger (Hollow Stem)/HW/NW Casing, Rotary Drill, NQ Core
COMPILED BY ZS
DATUM Geodetic
DATE August 19, 2016
CHECKED BY

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
87.0	GROUND SURFACE				
0.0	ASPHALTIC CONCRETE				
86.7	PORTLAND CEMENT CONCRETE	A	GRAB	-	
0.4	Sand and gravel, some silt (FILL) Compact to loose Brown Moist	1	SS	19	
		2	SS	9	
		3	SS	22	41 49 7 3
		4	SS	6	
83.2 3.8	Sandy gravel, some silt (FILL) Dark brown Wet	5	SS	10	48 32 13 7
82.4 4.6	Silty sand, trace organic matter (FILL) Loose to very loose Dark brown Wet	6	SS	5	
81.7 5.3	SILTY CLAY to CLAY, contains rootlets and thin silty sand layers (Weathered Crust) Stiff to firm Dark grey brown Wet	7	SS	WH	
		8	SS	2	
79.4 7.6	SILTY CLAY to CLAY Stiff to firm Grey with black organic mottling Wet	9	TP	PH	
		10	TP	PH	

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (86, 85, 84, 83, 82, 81, 80, 79, 78); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³.

DRAFT

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 16-104

SHEET 2 OF 9

METRIC

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01

LOCATION N 5027611.2 ;E 323887.7

ORIGINATED BY RI

DIST Eastern HWY 17

BOREHOLE TYPE Power Auger (Hollow Stem)/HW/NW Casing, Rotary Drill, NQ Core

COMPILED BY ZS

DATUM Geodetic

DATE August 19, 2016

CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	--- CONTINUED FROM PREVIOUS PAGE ---				
	SILTY CLAY to CLAY Stiff to firm Grey with black organic mottling Wet				76
75.4 11.6	SILTY CLAY to CLAY Firm to stiff Grey with black organic mottling Wet	11	TP	PH	75
					74
					73
		12	TP	PH	72
					71
					70
		13	TP	PH	69
					68

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

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

Foundation Design

RECORD OF BOREHOLE No 16-104

SHEET 3 OF 9 — **METRIC**

PROJECT	12-1121-0193-1130		
G.W.P.	4121-10-01	LOCATION	N 5027611.2 ;E 323887.7
DIST	Eastern HWY 17	BOREHOLE TYPE	Power Auger (Hollow Stem)/HW/NW Casing, Rotary Drill, NQ Core
DATUM	Geodetic	DATE	August 19, 2016

ORIGINATED BY RI — COMPILED BY ZS — CHECKED BY

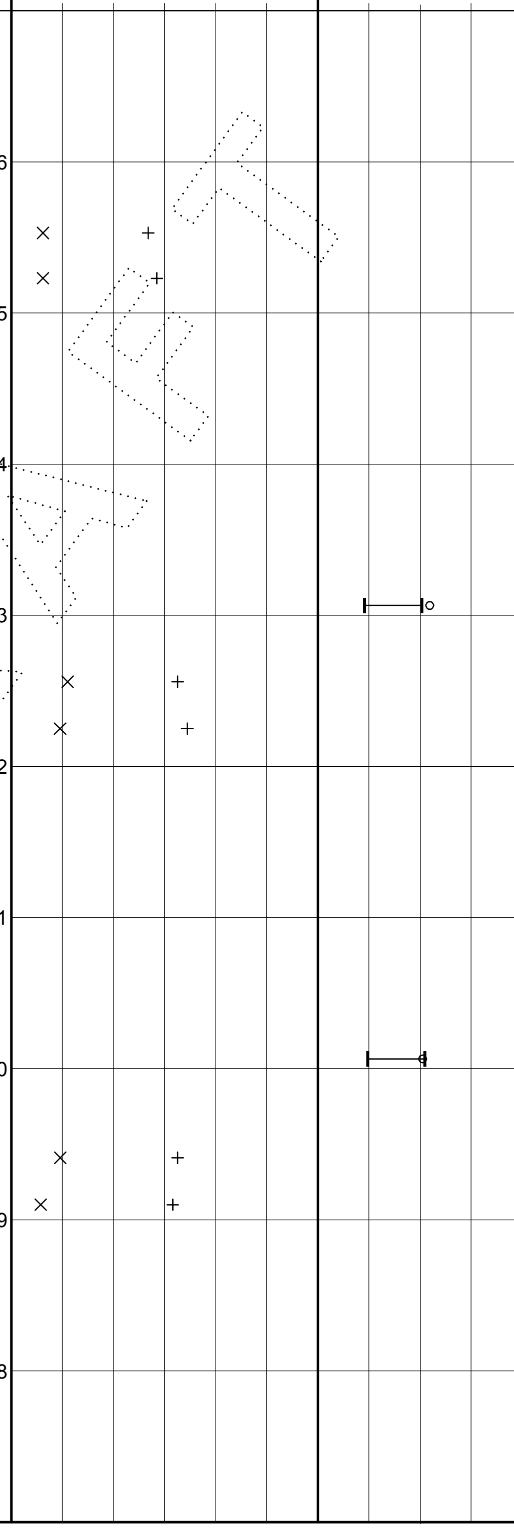
Column headings: SOIL PROFILE (ELEV / DEPTH; DESCRIPTION; STRAT PLOT) | SAMPLES (NUMBER; TYPE; "N" VALUES) | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60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

--- CONTINUED FROM PREVIOUS PAGE ---

SILTY CLAY to CLAY
Firm to stiff
Grey with black organic mottling
Wet

NUMBER	TYPE	"N" VALUES
14	TP	PH
15	TP	PH
16	SS	WR
17	SS	WR

Elevation scale: 66, 65, 64, 63, 62, 61, 60, 59, 58



Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 16-104

SHEET 4 OF 9

METRIC

PROJECT 12-1121-0193-1130			
G.W.P. 4121-10-01	LOCATION N 5027611.2 ;E 323887.7		ORIGINATED BY RI
DIST Eastern HWY 17	BOREHOLE TYPE Power Auger (Hollow Stem)/HW/NW Casing, Rotary Drill, NQ Core		COMPILED BY ZS
DATUM Geodetic	DATE August 19, 2016		CHECKED BY

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

--- CONTINUED FROM PREVIOUS PAGE ---

SILTY CLAY to CLAY
Firm to stiff
Grey with black organic mottling
Wet

NUMBER	TYPE	"N" VALUES
18	SS	WH

Elevation scale: 56, 55, 54, 53, 52, 51, 50, 49, 48

Continued Next Page

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

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

Foundation Design

Golder Associates

RECORD OF BOREHOLE No 16-104

METRIC

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01

LOCATION N 5027611.2 ;E 323887.7

ORIGINATED BY RI

DIST Eastern HWY 17

BOREHOLE TYPE Power Auger (Hollow Stem)/HW/NW Casing, Rotary Drill, NQ Core

COMPILED BY ZS

DATUM Geodetic

DATE August 19, 2016

CHECKED BY

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	GR	SA	SI	CL
	SILTY CLAY to CLAY Firm to stiff Grey with black organic mottling Wet				46						
		19	SS	WH	45						
						× (remoulded ~24); + (field vane ~80)					
						× (remoulded ~33); + (field vane ~80)					
					44						
					43						
					42						
					41						
					40						
39.4 47.6	SILTY CLAY to CLAY Very stiff Grey Wet	20	SS	WH	39		o (~47)	0	1	40	59
						>96 +					
						>96 +					
					38						

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

DRAFT

Golder Associates

RECORD OF BOREHOLE No 16-104

METRIC

PROJECT	12-1121-0193-1130		
G.W.P.	4121-10-01	LOCATION	N 5027611.2 ;E 323887.7
DIST	Eastern	HWY	17
BOREHOLE TYPE	Power Auger (Hollow Stem)/HW/NW Casing, Rotary Drill, NQ Core		
DATUM	Geodetic	DATE	August 19, 2016
ORIGINATED BY	RI	COMPILED BY	ZS
CHECKED BY			

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa (FIELD VANE)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY to CLAY Very stiff Grey Wet						36				
							35				
							34				
			21	SS	WH		33	>96+ >96+			
							32				
							31				
							30				
29.1 57.9	SILTY CLAY to CLAY, trace sand Very stiff Grey Wet						29				
							28				

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 16-104

SHEET 7 OF 9

METRIC

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01 | LOCATION N 5027611.2 ;E 323887.7 | ORIGINATED BY RI

DIST Eastern HWY 17 | BOREHOLE TYPE Power Auger (Hollow Stem)/HW/NW Casing, Rotary Drill, NQ Core | COMPILED BY ZS

DATUM Geodetic | DATE August 19, 2016 | CHECKED BY

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY to CLAY, trace sand Very stiff Grey Wet	22	SS	WR		
20.9 66.1	Sandy CLAYEY SILT Very loose Grey Wet	23	SS	WR	○	0 30 36 34
19.7 67.4	Limestone (BEDROCK) Bedrock cored from depths of 67.4 m to 71.2 m For bedrock coring details refer to Record of Drillhole 16-104	1	RC	REC 99%		RQD = 96%
		2	RC	REC 100%		RQD = 99%

Elevation scale: 26, 25, 24, 23, 22, 21, 20, 19, 18

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75

UNIT WEIGHT γ kN/m^3

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 16-104

SHEET 8 OF 9

METRIC

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01 LOCATION N 5027611.2 ;E 323887.7 ORIGINATED BY RI

DIST Eastern HWY 17 BOREHOLE TYPE Power Auger (Hollow Stem)/HW/NW Casing, Rotary Drill, NQ Core COMPILED BY ZS

DATUM Geodetic DATE August 19, 2016 CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (25 50 75)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Limestone (BEDROCK)		2	RC							
	Bedrock cored from depths of 67.4 m to 71.2 m For bedrock coring details refer to Record of Drillhole 16-104		3	RC	REC 100%		16				RQD = 97%
15.8 71.2	END OF BOREHOLE										

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

RECORD OF DRILLHOLE: 16-104

PROJECT: 12-1121-0193-1130
LOCATION: N 5027611.2 ;E 323887.7
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: August 19, 2016
DRILL RIG: CME 75
DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

NOTE: For abbreviations, symbols and descriptions refer to **LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY**

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	WEATHERING INDEX	FEATURES
		BEDROCK SURFACE		19.65													
68–71	Rotary Drill, NQ Core	Limestone (BEDROCK), with occasional black shale partings Fresh Thinly to medium bedded Grey Fine to medium grained Non-porous		67.35	1	100											
					2	100											
				15.83	3	100											
		END OF DRILLHOLE NOTES: 1. Artesian flow conditions were observed 4.4 m above ground surface within casing installed to 67.5 m depth prior to sealing borehole.		71.17													

Depth scale: 68, 69, 70, 71, 72, 73, 74, 75, 76, 77

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: RI
CHECKED:

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 16-104A

SHEET 1 OF 1

METRIC

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01 | LOCATION N 5027615.3 ;E 323883.6 | ORIGINATED BY RI

DIST Eastern HWY 17 | BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem) | COMPILED BY ZS

DATUM Geodetic | DATE August 25, 2016 | CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
87.0	GROUND SURFACE										
0.0	For soil stratigraphy from 0.0 m to 4.6 m see Record of Borehole 16-104										
							86				
							85				
							84				
							83				
82.4 / 4.7	Silty clay, some sand, trace gravel, trace organic matter (FILL) Dark brown Moist		1	SS	8		82		~25 (o)		OC = 4.4%
81.8 / 5.2	CLAY, trace organic matter Dark brown Moist										
81.1 / 6.0	SILTY CLAY to CLAY, contains rootlets and thin silty sand layers (Weathered Crust) Very stiff Grey-brown Moist		2	SS	5						
	END OF BOREHOLE										

Dynamic cone penetration resistance plot scale: 20 40 60 80 100. Shear strength legend: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED. Water content: PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; scale 25 50 75.

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 16-104B

SHEET 1 OF 1 **METRIC**

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01 LOCATION N 5027616.1 ;E 323882.9 ORIGINATED BY RI

DIST Eastern HWY 17 BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem) COMPILED BY ZS

DATUM Geodetic DATE August 25, 2016 CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
87.0	GROUND SURFACE				
0.0	For soil stratigraphy from 0.0 m to 3.1 m see Record of Borehole 16-104				86, 85
84.0 3.1	Silty sand, some gravel (FILL) Loose Brown Moist	1	SS	4	84
83.2 3.8	Gravelly silty sand (FILL) Dense Brown to dark grey Wet	2	SS	33	83
82.4 4.6	CLAY, some sand, trace gravel, contains organic matter, rootlets and wood Dark grey Moist	3	SS	8	82
81.8 5.2	SILTY CLAY to CLAY, contains very thin silty sand layers (Weathered Crust) Very stiff Dark grey-brown Moist	4	SS	14	
81.2 5.8	END OF BOREHOLE				

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

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

Golder Associates

RECORD OF BOREHOLE No 16-105

SHEET 1 OF 2

METRIC

PROJECT 12-1121-0193-1130
W.P. 4121-10-01
LOCATION N 5027515.2 ;E 323983.1
ORIGINATED BY DG
DIST Eastern HWY 17
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)
COMPILED BY ZS
DATUM Geodetic
DATE August 23-24, 2016
CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
86.9	GROUND SURFACE							
0.0 / 86.7	ASPHALTIC CONCRETE							
	Gravelly sand (FILL) Brown Moist	1	GRAB	-				
0.4	Sand and gravel (FILL) Grey-brown Moist	2	GRAB	-				
86.0 / 0.9	Silty clay (FILL) Grey-brown Moist	3	SS	7	86			
85.1 / 1.8	Sand, some silt and gravel (FILL) Dense to compact Brown Most	4	SS	42	85			
		5	SS	18	84			8 77 11 4
		6	SS	30				
83.2 / 3.7	Sandy silt (FILL) Loose Grey Wet	7	SS	8	83			
82.1 / 4.8	Organic SILT Very loose Dark grey-brown Moist	8	SS	3	82	109	OC = 11.0%	
81.6 / 5.3	SILTY CLAY to CLAY (Weathered Crust) Stiff Dark grey with black organic mottling Moist	9	SS	4	81			
		10	SS	3				
80.0 / 6.9	SILTY CLAY to CLAY Firm Grey Wet	11	SS	1	80			
		12	SS	WH	79			
					78			
		13	SS	WH	77			

Dynamic cone penetration resistance plot scale: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 16-105

SHEET 2 OF 2 | METRIC

PROJECT 12-1121-0193-1130			
W.P. 4121-10-01	LOCATION N 5027515.2 ;E 323983.1		ORIGINATED BY DG
DIST Eastern HWY 17	BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)		COMPILED BY ZS
DATUM Geodetic	DATE August 23-24, 2016		CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SILTY CLAY to CLAY Firm Grey Wet						76 75 74 73 72				
71.7 15.2	END OF BOREHOLE NOTES: 1. Water level in well screen at a depth of 3.1 m below ground surface (Elev. 83.8 m), measured on October 27, 2016.										

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L — WATER CONTENT (%) 25 50 75

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

RECORD OF BOREHOLE No 16-106

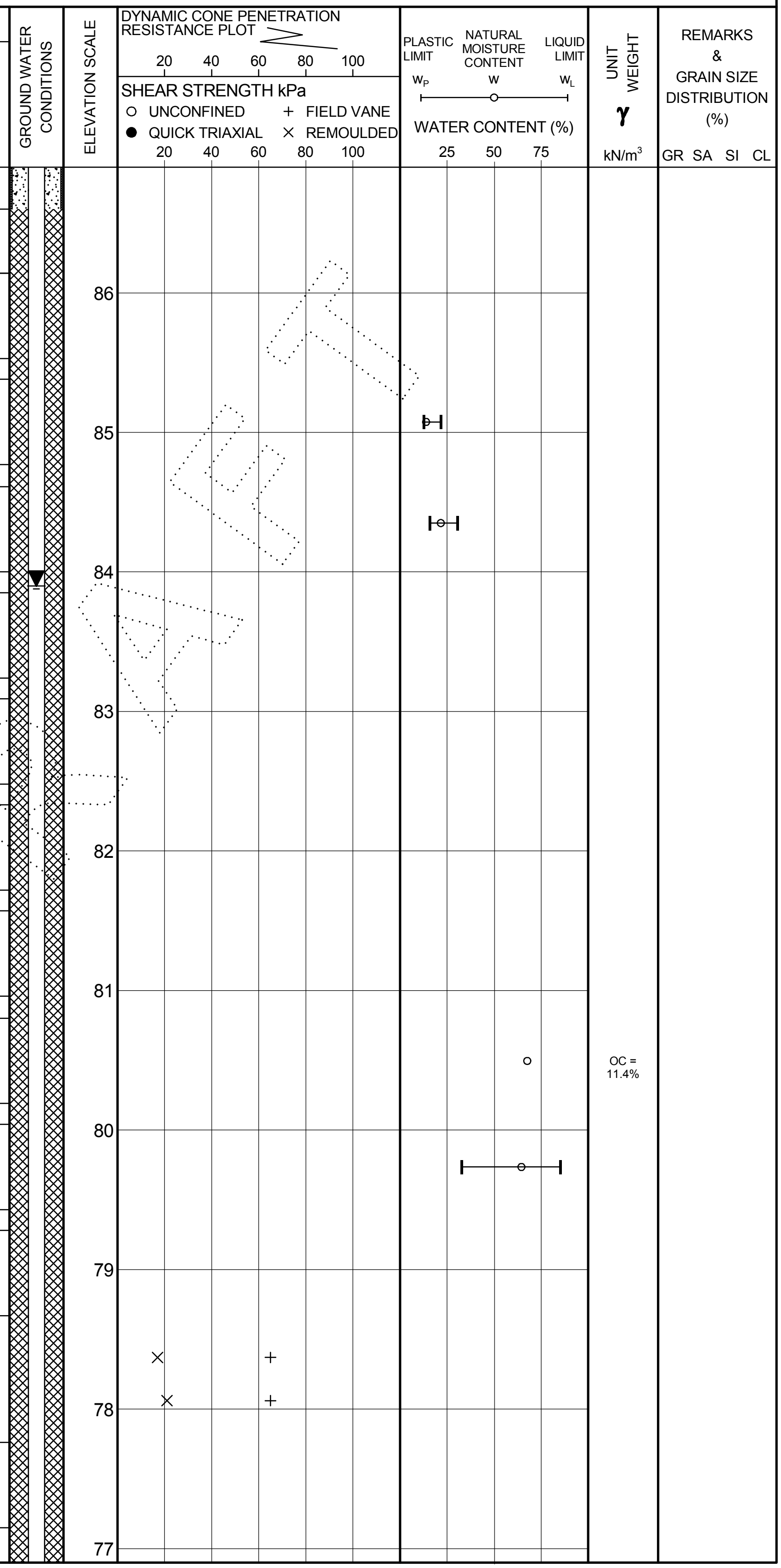
SHEET 1 OF 2 **METRIC**

PROJECT 12-1121-0193-1130
G.W.P. 4121-10-01
DIST Eastern HWY 17
DATUM Geodetic

LOCATION N 5027624.5 ;E 323875.1
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)
DATE August 24, 2016

ORIGINATED BY DG
COMPILED BY ZS
CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS
86.9	GROUND SURFACE				
0.0	ASPHALTIC CONCRETE				
86.7 / 0.2	Sand and gravel (FILL) Very loose Grey-brown Moist	1	GRAB	-	
85.9 / 1.0	Silty clay, some sand and gravel (FILL) Grey-brown Moist	2	SS	3	
		3	SS	3	
		4	SS	6	
84.1 / 2.8	Silty clay (FILL) Dark grey with black organic mottling Moist	5	SS	3	
		6	SS	4	
		7	SS	3	
81.8 / 5.1	Silty clay, contains sand seams (FILL) Grey-brown to grey Moist	8	SS	4	
80.8 / 6.1	Organic CLAY, contains rootlets Stiff Dark grey Moist to wet	9	SS	3	OC = 11.4%
		10	SS	1	
79.3 / 7.6	SILTY CLAY to CLAY, trace organic matter (Weathered Crust) Dark grey to grey	11	SS	1	
78.8 / 8.2	SILTY CLAY to CLAY Firm to stiff Grey Wet				
		12	SS	WH	



Continued Next Page

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 16-106

SHEET 2 OF 2 **METRIC**

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01 LOCATION N 5027624.5 ;E 323875.1 ORIGINATED BY DG

DIST Eastern HWY 17 BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem) COMPILED BY ZS

DATUM Geodetic DATE August 24, 2016 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (25, 50, 75)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SILTY CLAY to CLAY Firm to stiff Grey Wet						76 75 74 73 72				
71.7 15.2	END OF BOREHOLE NOTES: 1. Water level in well screen at a depth of 3.0 m below ground surface (Elev. 83.9 m), measured on October 27, 2016.										

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

Foundation Design

RECORD OF BOREHOLE No 16-107

SHEET 1 OF 1 **METRIC**

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01 LOCATION N 5027521.8 ;E 323951.8 ORIGINATED BY KM

DIST Eastern HWY 17 BOREHOLE TYPE Portable Drill, Continuous Sampling COMPILED BY JM

DATUM Geodetic DATE October 27, 2016 CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
83.6	GROUND SURFACE					
0.0	Sandy silt, contains rootlets (TOPSOIL)					
83.5 0.1	Dark brown Moist	1	SS	2		
83.0 0.6	Sand, trace gravel (FILL) Very loose Brown Moist					83
	SILTY CLAY to CLAY, trace organic matter (WEATHERED CRUST) Stiff Grey-brown Wet	2	SS	7		
		3	SS	12		82
		4	SS	10		
81.2 2.4	SILTY CLAY to CLAY, trace organic matter Firm to stiff Grey Wet	5	SS	4		81
80.6 3.1	SILTY CLAY, with sand seams Firm to stiff Grey Wet	6	SS	4		80
79.8 3.8	SILTY CLAY to CLAY Firm Grey Wet	7	SS	2	∇	79
		8	SS	1		
78.1 5.5	END OF BOREHOLE					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%) 25 50 75

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

NOTES:

1. Water level in open borehole at a depth of 4.1 m below ground surface (Elev. 79.5 m), measured during drilling.

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 16-108

SHEET 1 OF 1 **METRIC**

PROJECT 12-1121-0193-1130
G.W.P. 4121-10-01 LOCATION N 5027603.0 ;E 323882.0 ORIGINATED BY DG
DIST Eastern HWY 17 BOREHOLE TYPE Wash Boring, NW Casing COMPILED BY ZS
DATUM Geodetic DATE August 30, 2016 CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS
83.8	GROUND SURFACE				
0.0	Organic silt (FILL)				
0.1	Silty clay, contains rootlets (FILL) Loose Grey-brown Moist	1	SS	6	
83.2 / 0.6	Sand (FILL) Compact Grey Wet	2	SS	16	
82.6 / 1.2	SILTY CLAY to CLAY (WEATHERED CRUST) Stiff Grey-brown Moist	3	SS	7	
82.0 / 1.8	SILTY CLAY to CLAY, contains organic matter Firm Grey with black organic mottling	4	SS	5	
		5	SS	2	
80.5 / 3.4	Organic SILT Loose to very loose Dark grey Wet	6	SS	1	
		7	SS	4	OC = 12.8%
		8	SS	2	OC = 8.6%
78.9 / 4.9	END OF BOREHOLE				

NOTES:

1. Water level in well screen at a depth of 0.4 m below ground surface (Elev. 83.4 m), measured on October 27, 2016.

Elevation scale: 83, 82, 81, 80, 79

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75
UNIT WEIGHT γ kN/m³
GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

117.8

+ 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_IM\GINT\PHASE 1130\1211210193-1130.GPJ GAL-GTA.GDT 5/1/17

Foundation Design

Golder Associates

RECORD OF BOREHOLE No 16-108A

SHEET 1 OF 1

METRIC

PROJECT 12-1121-0193-1130

G.W.P. 4121-10-01

DIST Eastern HWY 17

DATUM Geodetic

LOCATION N 5027601.9 ;E 323883.1

BOREHOLE TYPE Portable Drill

DATE October 27, 2016

ORIGINATED BY KM

COMPILED BY JM

CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³ / WATER CONTENT
83.8	GROUND SURFACE					
0.0	Sandy silt (TOPSOIL) Dark brown Moist					OC = 16.9%
83.5 / 0.3	Sand and silt, trace organic matter, with gravel (FILL) Loose Brown Moist	1	SS	6		
	Silty clay, trace gravel and organic matter, with sand (FILL) Grey-brown Moist	2	SS	8	▽	
82.6 / 1.2	SILTY CLAY, trace organic matter (WEATHERED CRUST) Firm Grey-brown Wet	3	SS	7		
		4	SS	6		
81.4 / 2.4	CLAY, trace sand and organic matter Firm Grey Wet	5	SS	1		
80.8 / 3.1	Organic SILT, trace sand, contains wood fragments Very soft to soft Dark brown Wet	6	SS	2		OC = 9.2%
		7	SS	2		
		8	SS	2		
		9	SS	2		OC = 10.6%
		10	SS	4		
77.6 / 6.3	SILTY CLAY to CLAY Firm to stiff Grey Wet	11	SS	5		
77.1 / 6.7	END OF BOREHOLE					

Elevation scale: 83, 82, 81, 80, 79, 78

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

NOTES:

1. Water level in open borehole at a depth of 0.8 m below ground surface (Elev. 83.0 m), measured during drilling.

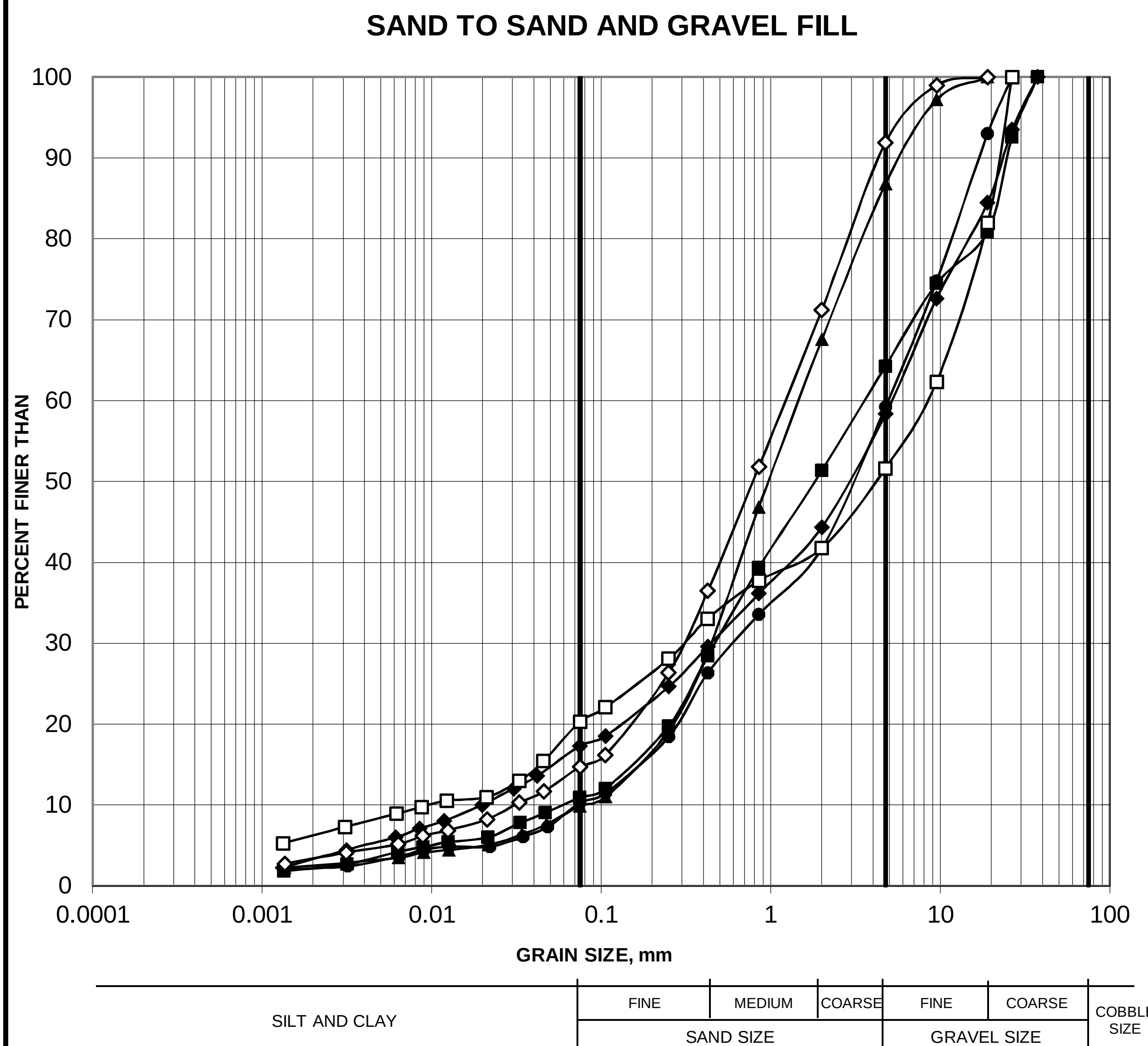
2. Three attempts were taken for this borehole due to equipment malfunction.

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

APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION	FIGURE B1

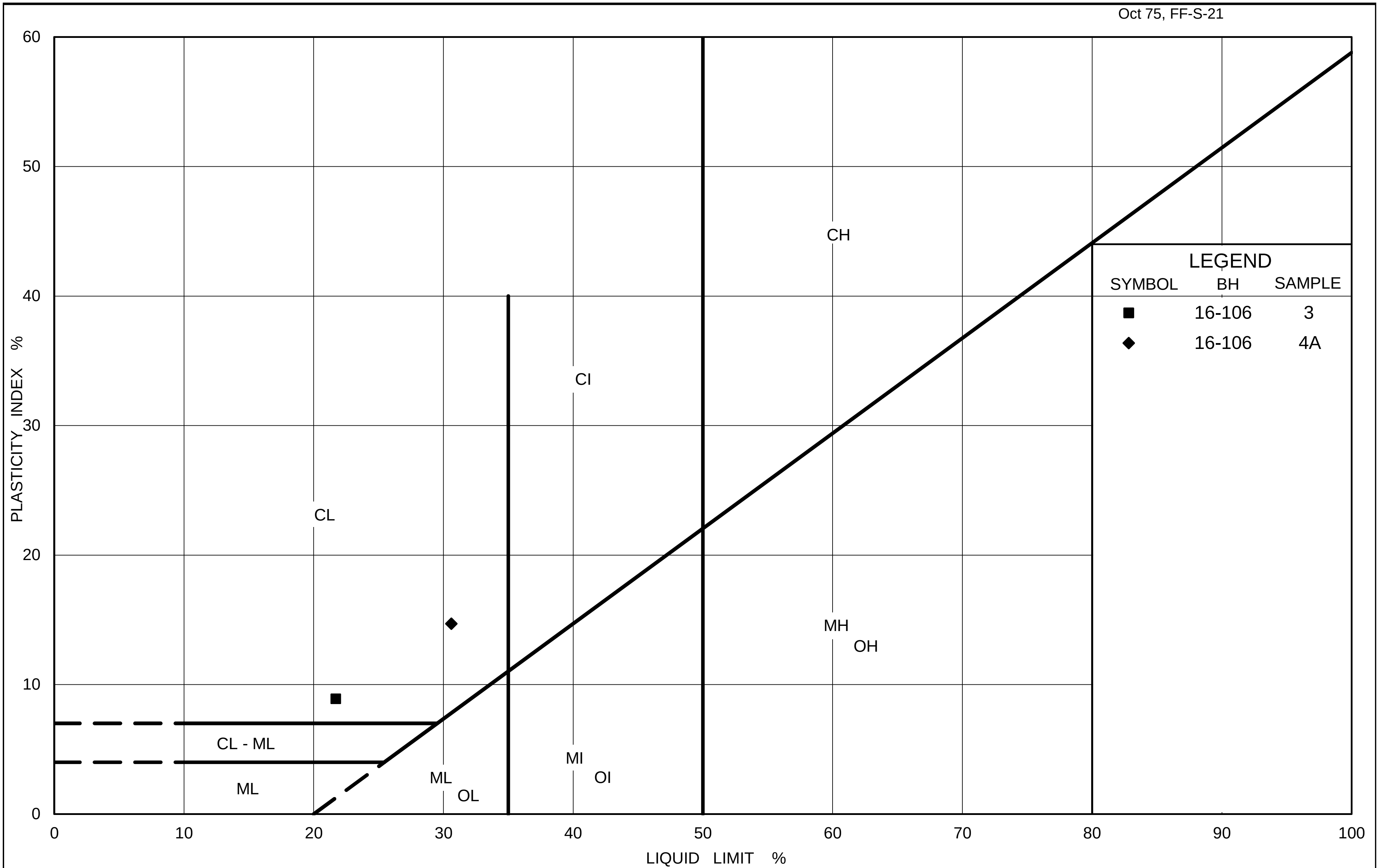


Borehole	Sample	Depth (m)
16-101	5	3.05-3.66
16-101A	2	1.52-2.13
16-101A	5	3.81-4.42
16-104	3	2.29-2.90
16-104	5	3.81-4.42
16-105	5	2.29-2.90

Created by: CW
Checked by: CNM

Project: 12-1121-0193 /1130

Golder Associates



Ministry of Transportation

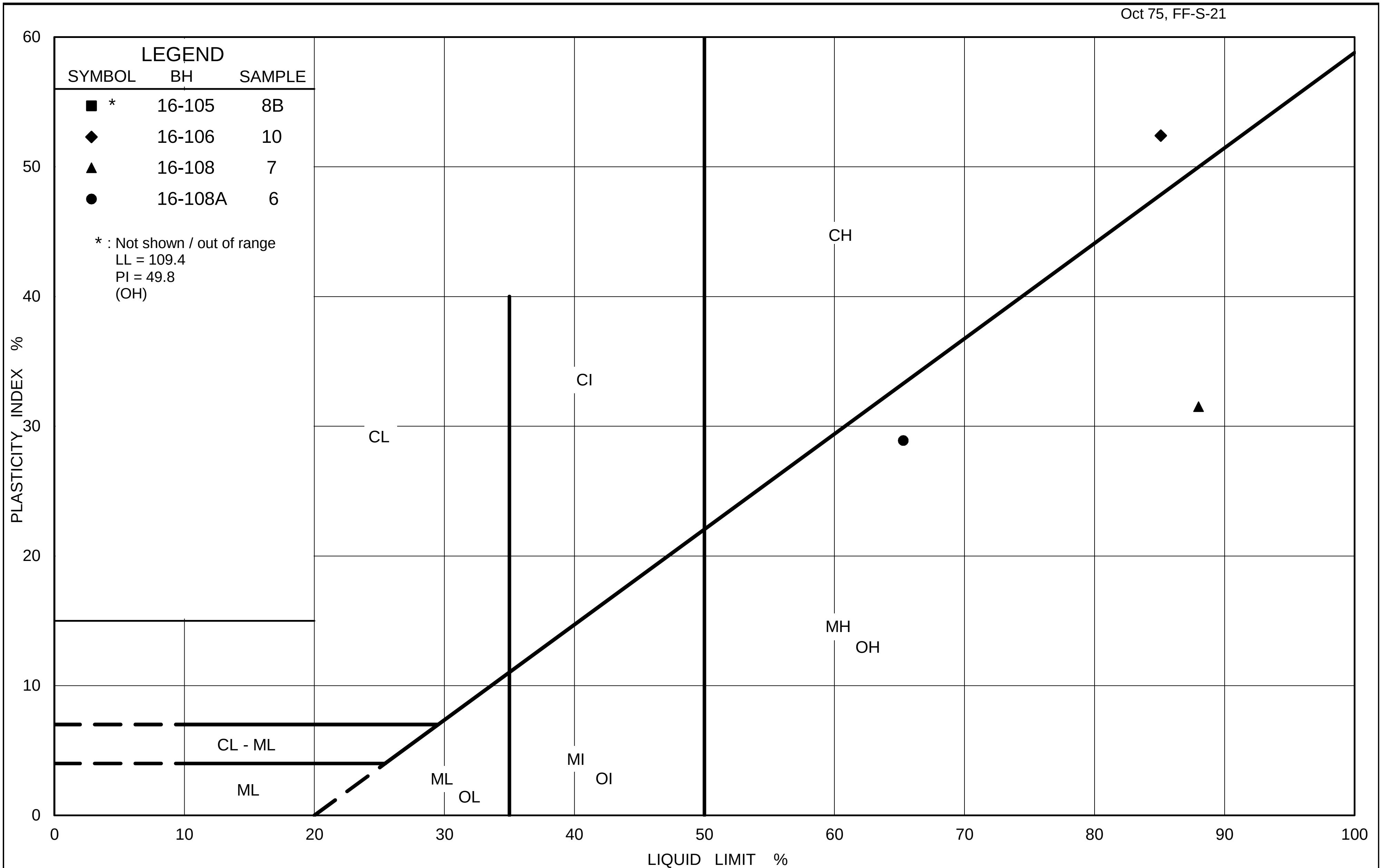
Ontario

PLASTICITY CHART
SILTY CLAY (FILL)

FIG No. B2

Project No. 12-1121-0193 /1130

Compiled By : CW   Checked By : CNM



Ministry of Transportation

Ontario

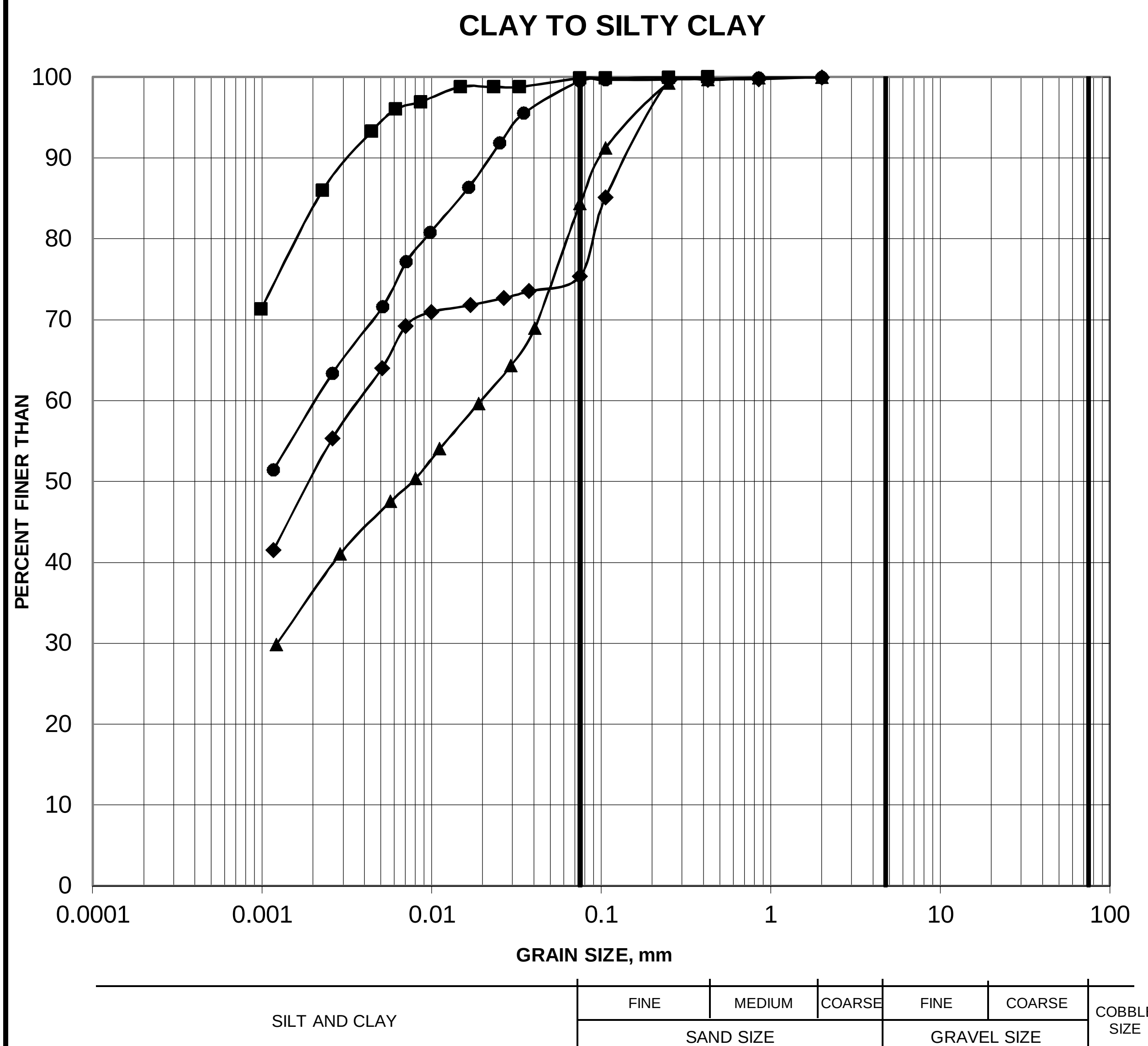
PLASTICITY CHART
ORGANICS

FIG No. B3

Project No. 12-1121-0193 /1130

Compiled By : CW Checked By : CNM

GRAIN SIZE DISTRIBUTION	FIGURE B4



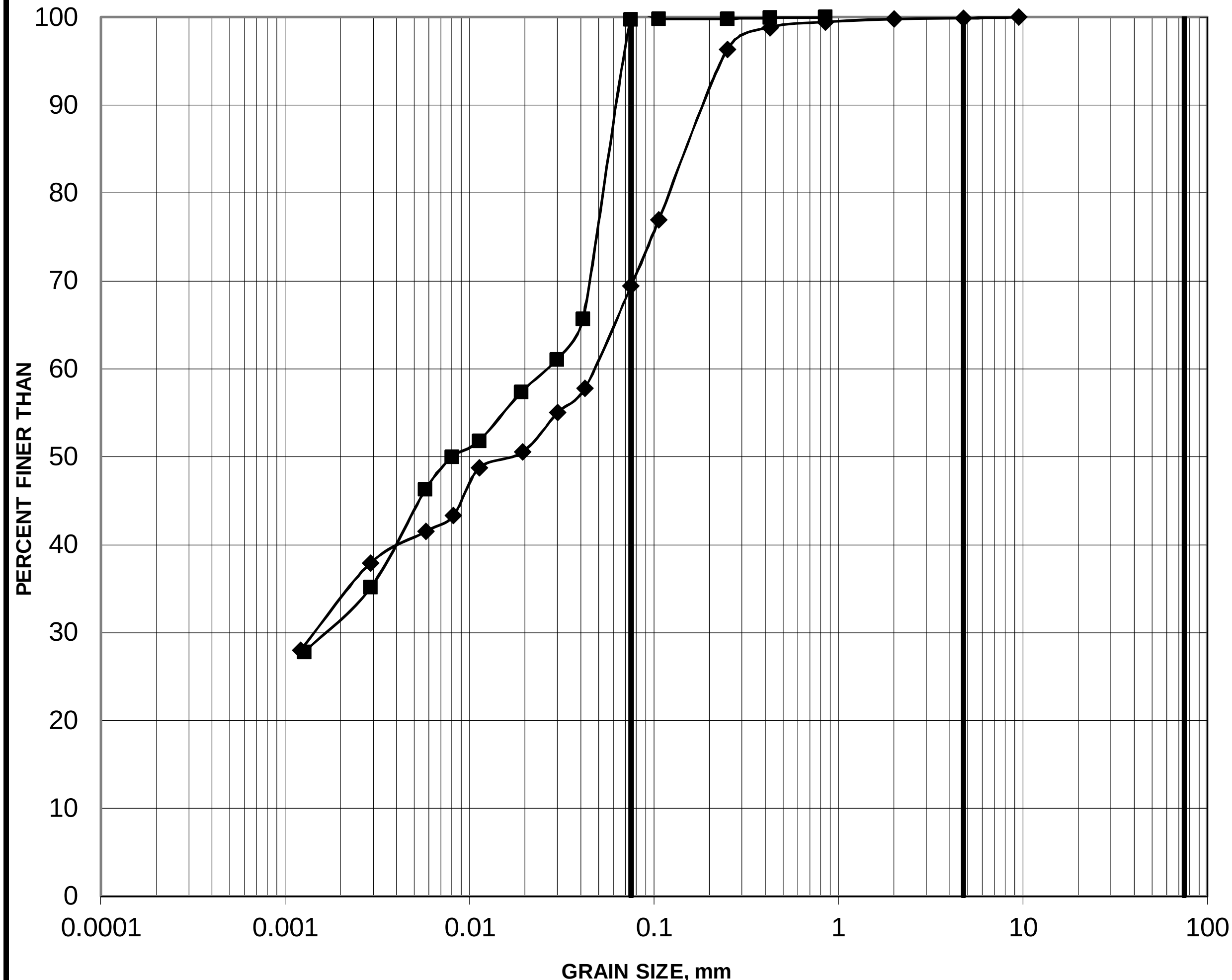
Borehole	Sample	Depth (m)
16-101	12	11.67-12.28
16-102	15	65.90-66.50
16-103	17B	65.91-66.21
16-104	20	47.57-48.19

Created by: CW
Checked by: CNM

Project: 12-1121-0193 /1130 **Golder Associates**

GRAIN SIZE DISTRIBUTION	FIGURE B5

CLAYEY SILT

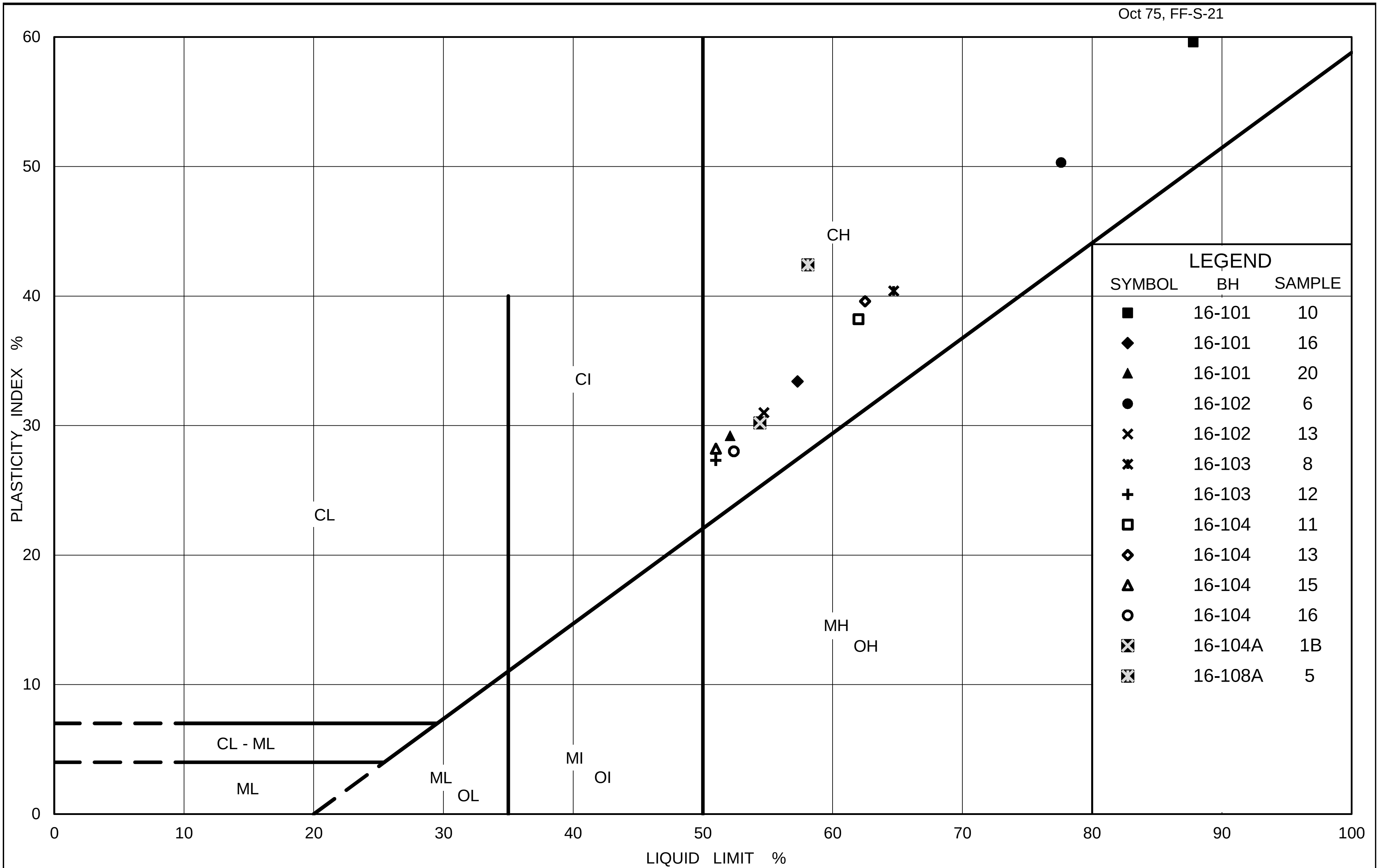


SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
16-102	10	35.70-36.30
16-104	23B	66.04-66.35

Created by:	CW
Checked by:	CNM

Project: 12-1121-0193 /1130 **Golder Associates**



Ministry of Transportation

Ontario

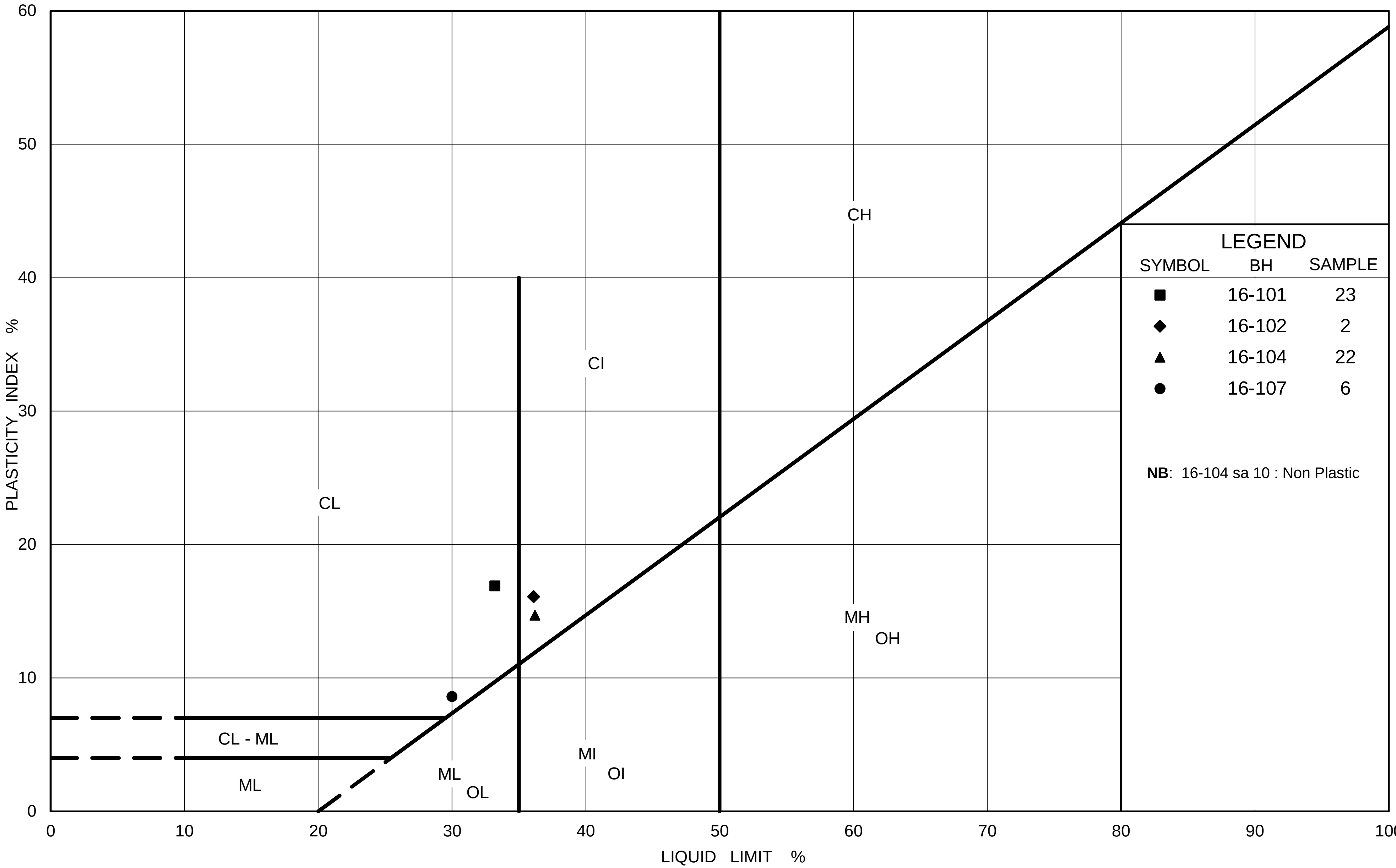
PLASTICITY CHART
CLAY

FIG No. B6a

Project No. 12-1121-0193 /1130

Compiled By : CW Checked By : CNM

Oct 75, FF-S-21



Ministry of Transportation

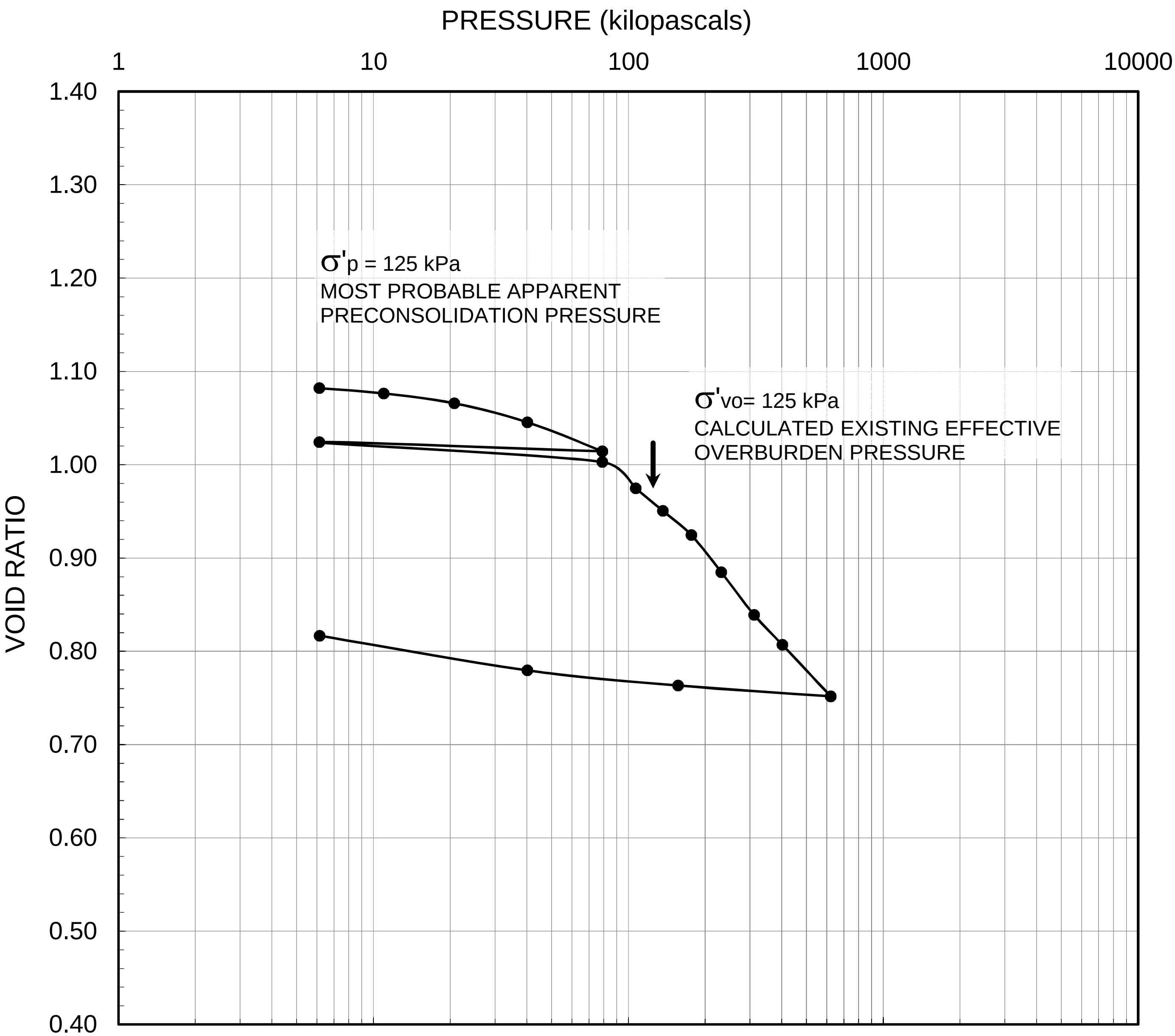
Ontario

PLASTICITY CHART
SILTY CLAY

FIG No. B6b

Project No. 12-1121-0193 /1130

Compiled By : CW   Checked By : CNM

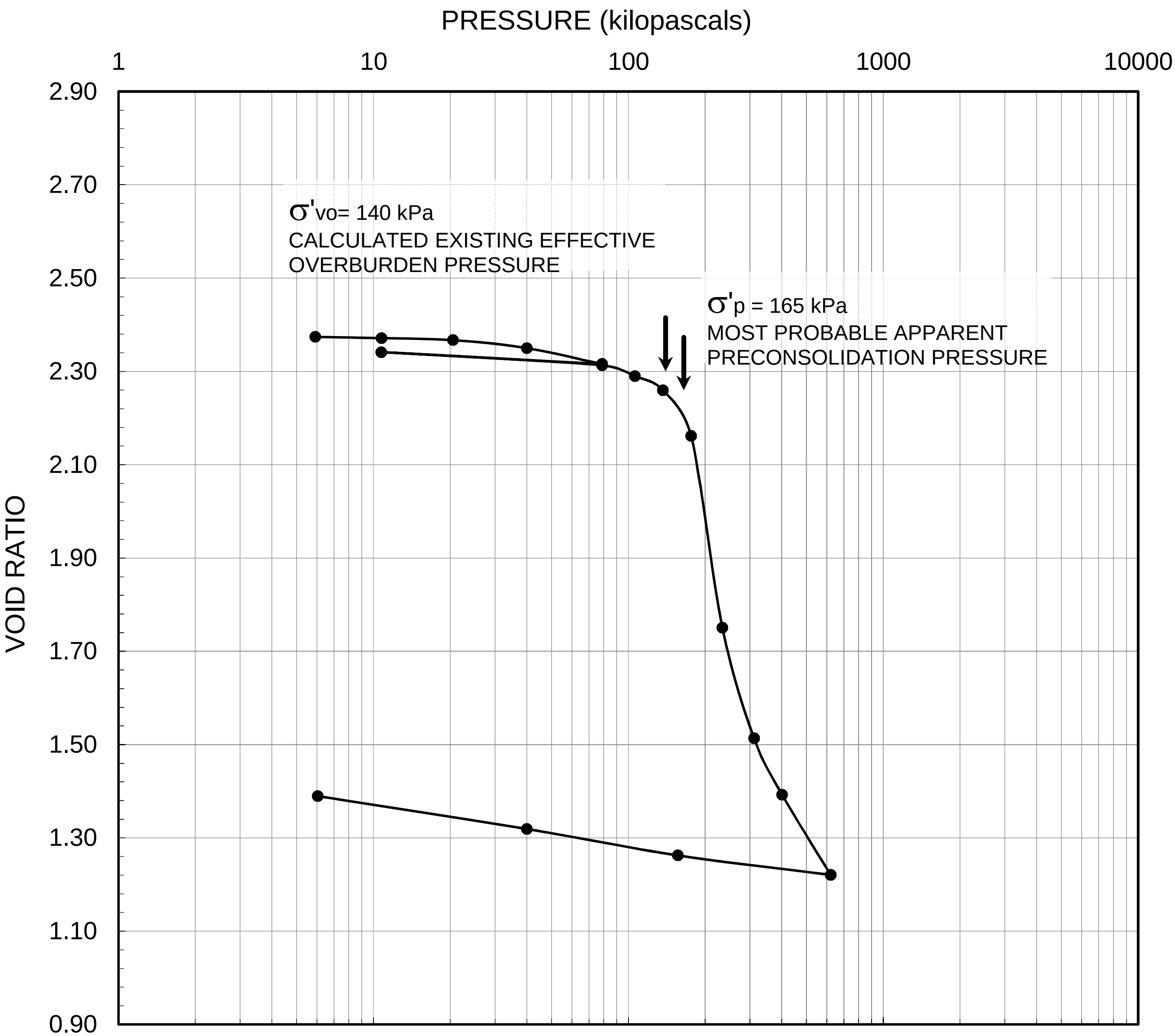


LEGEND

Borehole:	16-104	w_i = 41%	S_o = 100%	γ = 17.4 kN/m³
Sample:	10	w_f = 31%	e_o = 1.09	G_s = 2.62
Depth (m):	9.5	w_l =	C_c = 0.35	
Elevation (m):	77.5	w_p =	C_r = 0.019	

Golder Associates

FILE No. Consolidation summary	SCALE AS SHOWN	TITLE	
PROJECT No. 12-1121-0193/1130 REV. 1	DATE 05/03/17	CONSOLIDATION TEST RESULTS	
	CADD N/A		
	ENTERED CW		
	CHECK CNM		
	REVIEW KSL		FIGURE B7

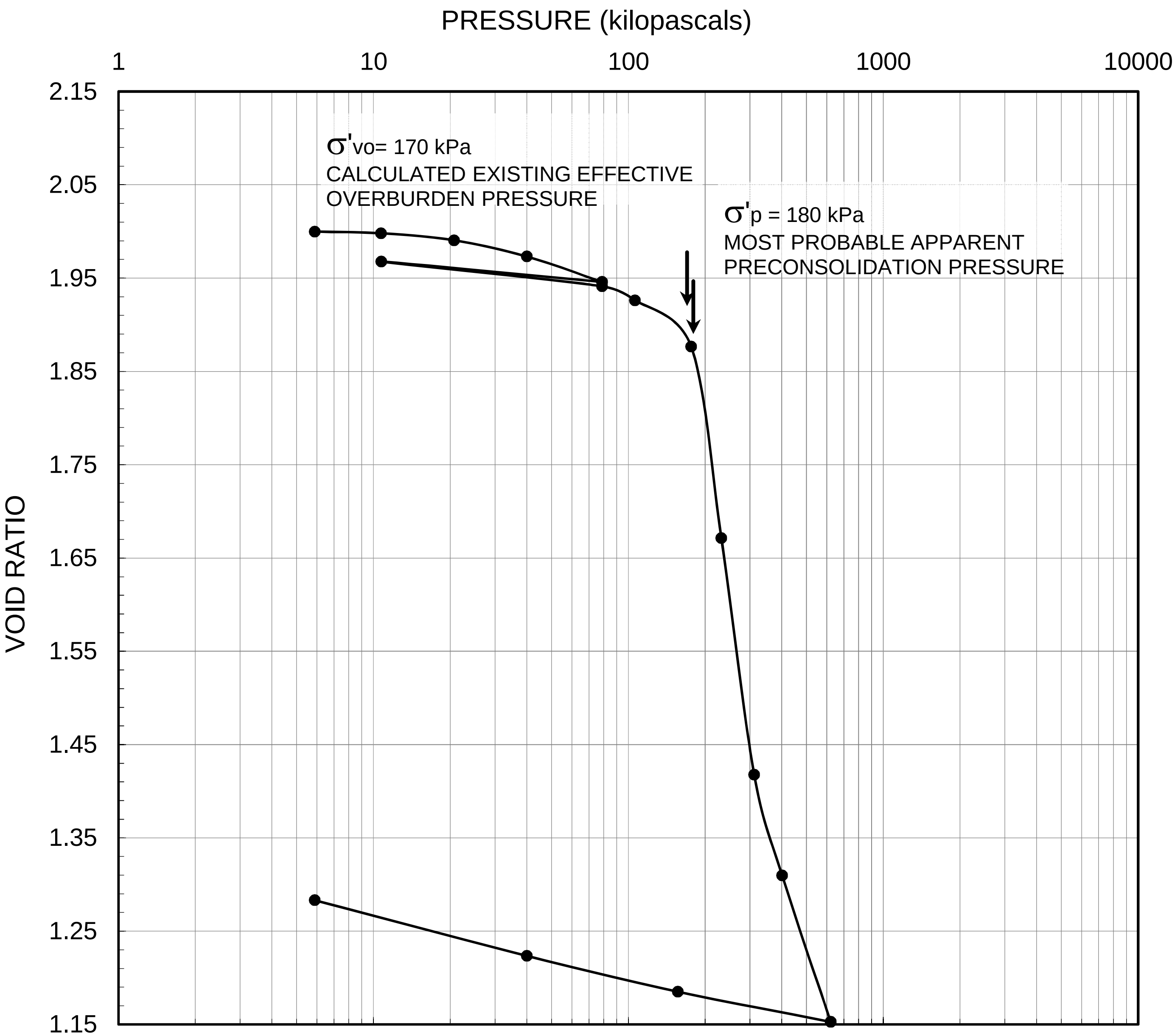


LEGEND

Borehole:	16-104	w_i = 87%	S_o = 100%	γ = 14.8 kN/m³
Sample:	11	w_f = 53%	e_o = 2.38	G_s = 2.73
Depth (m):	12.0	w_l = 62%	C_c = 3.36	
Elevation (m):	75.0	w_p = 24%	C_r = 0.032	

Golder Associates

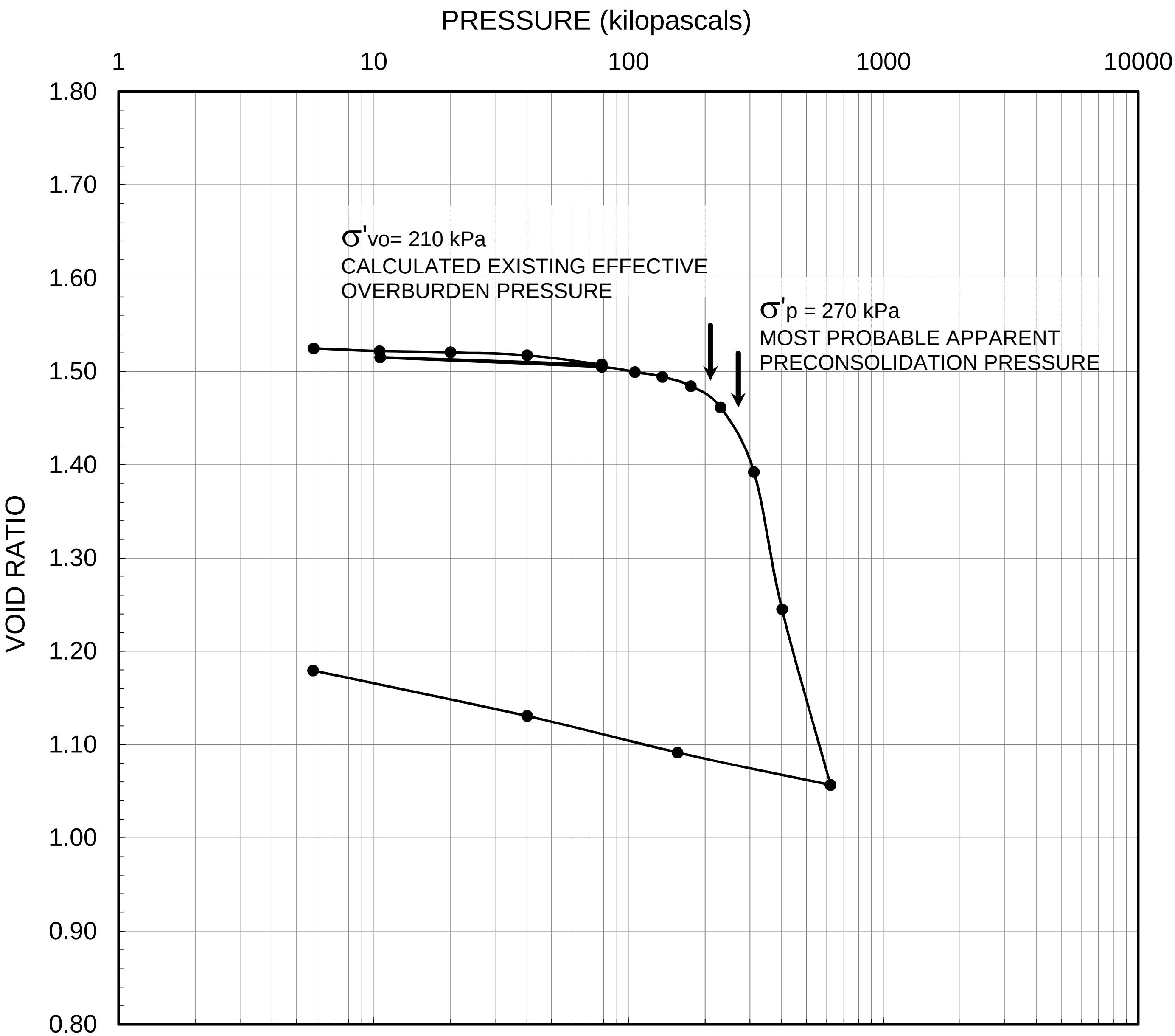
SCALE	AS SHOWN	TITLE	CONSOLIDATION TEST RESULTS
DATE	05/03/17		
CADD	N/A		
ENTERED	CW		
FILE No. Consolidation summary	CHECK	CNM	
PROJECT No. 12-1121-0193/1130 REV. 1	REVIEW	KSL	FIGURE B8



LEGEND

Borehole:	16-104	w_i	= 73%	S_o	= 100%	γ	= 15.4 kN/m³
Sample:	13	w_f	= 47%	e_o	= 2.00	G_s	= 2.72
Depth (m):	18.0	w_l	= 63%	C_c	= 1.97		
Elevation (m):	69.0	w_p	= 23%	C_r	= 0.031		

Golder Associates	SCALE: AS SHOWN	TITLE
	DATE: 05/03/17	**CONSOLIDATION TEST RESULTS**
	CADD: N/A	
	ENTERED: CW	
FILE No.: Consolidation summary	CHECK: CNM	
PROJECT No.: 12-1121-0193/1130 REV.: 1	REVIEW: KSL	FIGURE **B9**



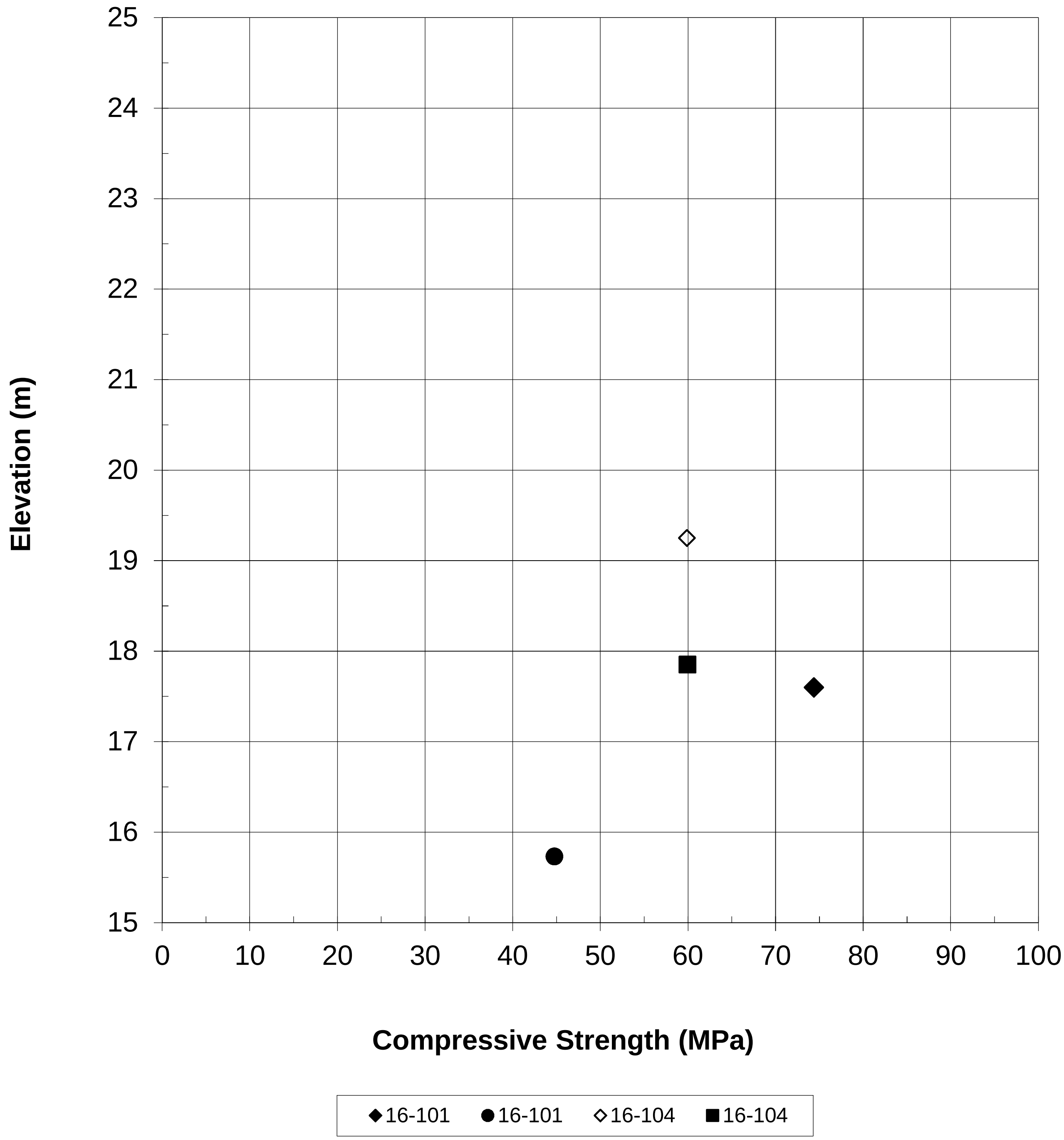
LEGEND

Borehole:	16-104	w_i = 55%	S_o = 98%	γ = 16.4 kN/m³
Sample:	15	w_f = 43%	e_o = 1.55	G_s = 2.76
Depth (m):	24.0	w_l = 51%	C_c = 1.34	
Elevation (m):	63.0	w_p = 23%	C_r = 0.012	

Golder Associates

SCALE	AS SHOWN	TITLE	
DATE	05/03/17		
CADD	N/A	**CONSOLIDATION TEST RESULTS**	
ENTERED	CW		
FILE No.	Consolidation summary	CHECK	CNM
PROJECT No.	12-1121-0193/1130	REV. 1	REVIEW KSL
FIGURE	B10		

SUMMARY OF LABORATORY COMPRESSIVE STRENGTH UNCONFINED COMPRESSION TESTS	FIGURE B11



Project: 12-1121-0193 /1130 **Golder Associates**

Created by: MI
Checked by: CNM

TABLE B1

SUMMARY OF ORGANIC CONTENT BY PERCENTAGE

PROJECT NUMBER : 12-1121-0193 /1130

PROJECT NAME : Dillon Mega 3 Eastern Region - Mississipi River Bridge

DATE TESTED : 10-Apr-17

Borehole No.	Sample No.	Depth (m)	Water Content (%)	Organic Content (%)
16-101	9B	6.40-6.71	72.4%	10.2%
16-103	1	5.79-6.39	126.2%	10.7%
16-104A	1B	4.72-5.18	26.2%	4.3%
16-105	8B	4.80-5.18	59.0%	11.0%
16-106	9	6.10-6.71	67.6%	11.4%
16-108A	1A	0.00-0.08	70.3%	16.9%
16-108A	6	3.05-3.66	85.5%	9.2%
16-108A	9	4.88-5.49	98.9%	10.6%
16-108B	7	3.66-4.27	116.2%	12.8%
16-108B	8	4.27-4.88	94.2%	8.6%

Golder Associates Ltd.

Checked by : CNM .

APPENDIX C

Vertical Seismic Profiling Test Results

TECHNICAL MEMORANDUM

DATE October 06, 2016 **PROJECT No.** 12-1121-0193

TO Kim Lesage
Golder Associates

FROM Stephane Sol, Christopher Phillips **EMAIL** ssol@golder.com, cphillips@golder.com

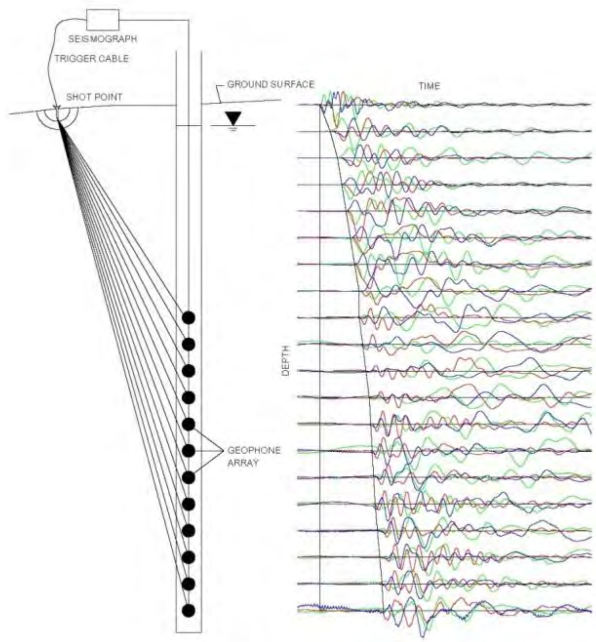
VERTICAL SEISMIC PROFILING TEST RESULTS
HWY 17 BRIDGE, KINBURN, ON

This memorandum presents the results of Vertical Seismic Profiling (VSP) testing carried out at the two locations located on HWY 17 just north of Kinburn Side Rd, Ottawa. VSP testing was carried out on August 29, 2016. Borehole 16-101 and 16-102 were drilled on both sides of the bridge to a depth of 72.68 m and 71.17 m respectively and then cased to 30 metres with a PVC pipe grouted in place. Both boreholes consisted of approximately 5 to 6 metres of silty sand over silty clay down to the bottom of the PVC casing (30 m).

Methodology

For the VSP method, seismic energy is generated at the ground surface by an active seismic source and recorded by a geophone located in a nearby borehole at a known depth. The active seismic source can be either compression or shear wave. The time required for the energy to travel from the source to the receiver (geophone) provides a measurement of the average compression or shear-wave seismic velocity of the medium between the source and the receiver. Data obtained from different geophone depths are used to calculate a detailed vertical seismic velocity profile of the subsurface in the immediate vicinity of the test borehole.

The high resolution results of a VSP survey are often used for earthquake engineering site classification, as per the 2010 National Building Code of Canada.



Example 1: Layout and resulting time traces from a VSP survey.

Fieldwork

The fieldwork was carried out on August 29, 2016, by personnel from the Golder Mississauga and Ottawa offices.

Both compression and shear-wave seismic sources were used and both were located 2 m from the borehole. The seismic source for the compression wave test consisted of a 9.9 kilogram sledge hammer vertically impacted on a metal plate. The seismic source for the shear-wave test consisted of a 2.4 metre long, 150 millimetre by 150 millimetre wooden beam, weighted by a vehicle and horizontally struck with a 9.9 kilogram sledge hammer on alternate ends of the beam to induce polarized shear waves. The shear source was coupled to the ground surface by parking a vehicle on top of it. Test measurements started at ground surface and were recorded in the borehole with a 3-component receiver spaced at 0.5-metre intervals below the ground surface to a maximum depth of the casing (9.77 m).

The seismic records collected for each source location were stacked a minimum of five times to minimize the effects of ambient background seismic noise on the collected data. The data was sampled at 0.020833 millisecond intervals and a total time window of 0.5 seconds was collected for each seismic shot.

Data Processing

Processing of the VSP test results consisted of the following main steps:

1) Combination of seismic records to present seismic traces for all depth intervals on a single plot for each seismic source and for each component;

2) Low Pass Filtering of data to remove spurious high frequency noise;

3) First break picking of the compression and shear-wave arrivals; and,

4) Calculation of the average compression and shear-wave velocity to each tested depth interval.

Processing of the VSP data was completed using the SeisImager/SW software package (Geometrics Inc.). The seismic records for borehole 16-101 are presented on the following two plots and show the first break picks of the compression wave (Figure 1) and shear wave arrivals (Figure 2) overlaid on the seismic waveform traces recorded at the different geophone depths for Borehole 16-101. The seismic records for borehole 16-104 are presented on the following two plots and show the first break picks of the compression wave (Figure 3) and shear wave arrivals (Figure 4) overlaid on the seismic waveform traces recorded at the different geophone depths for Borehole 16-104. The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.

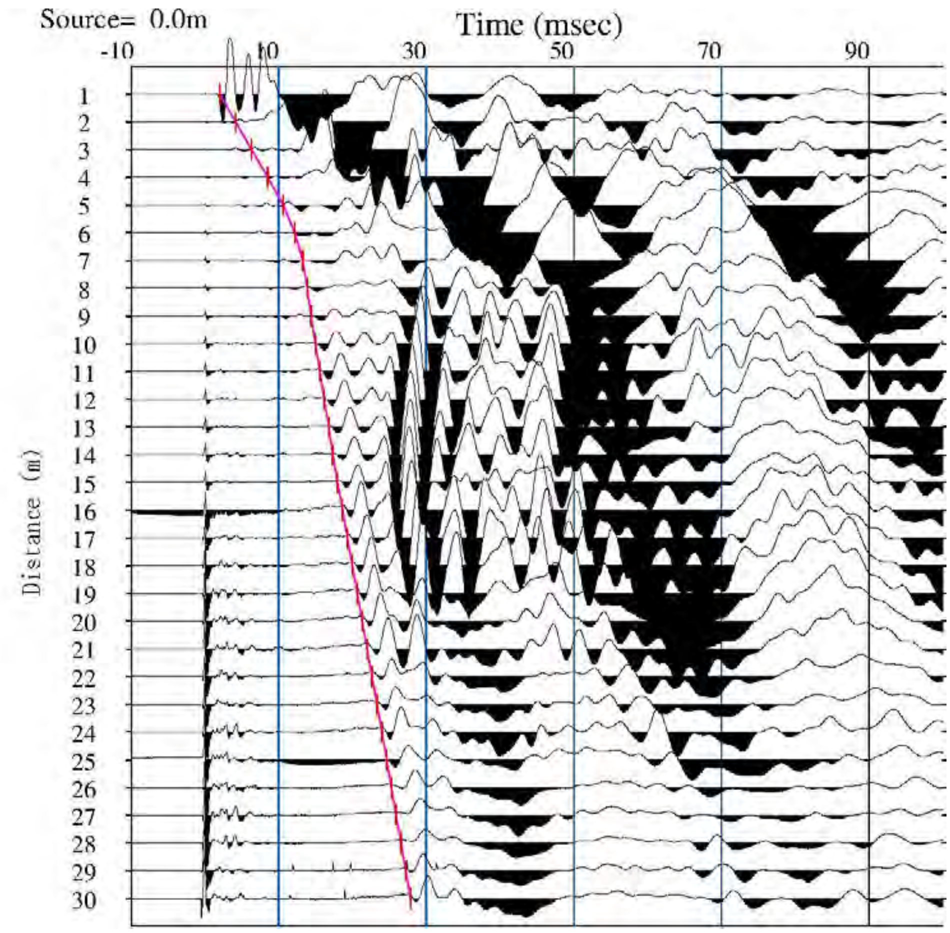


Figure 1: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 16-101.

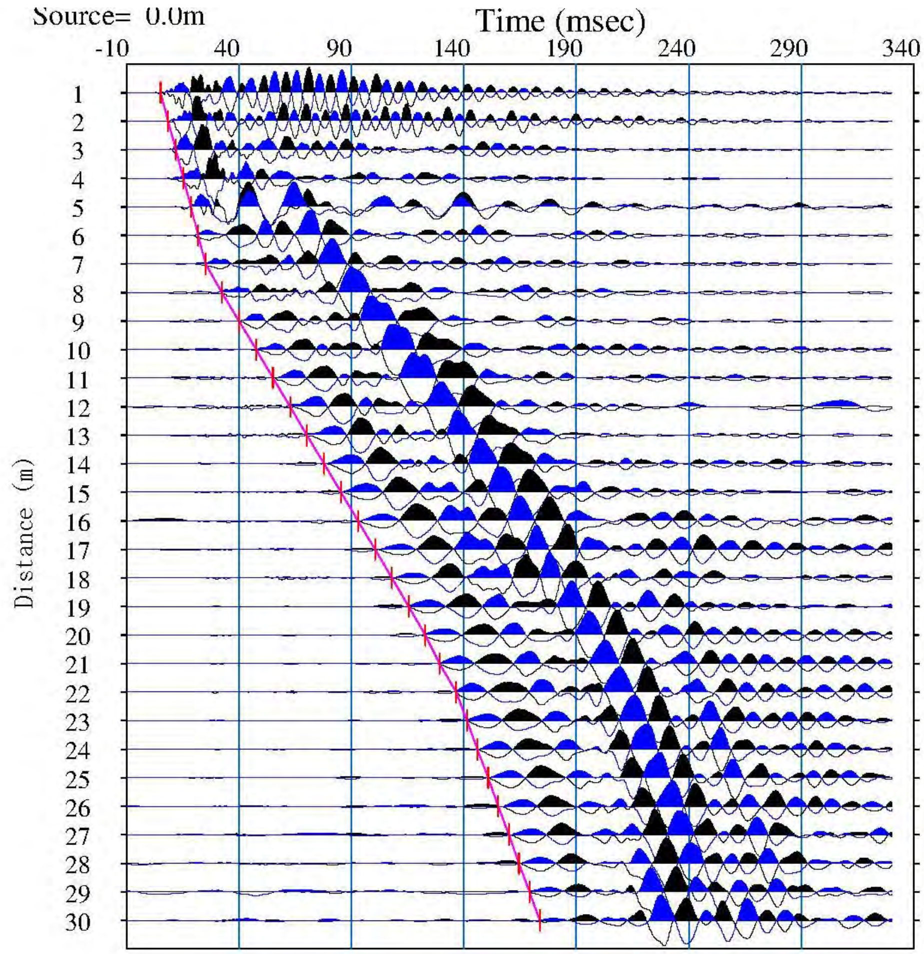


Figure 2: First break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 16-101.

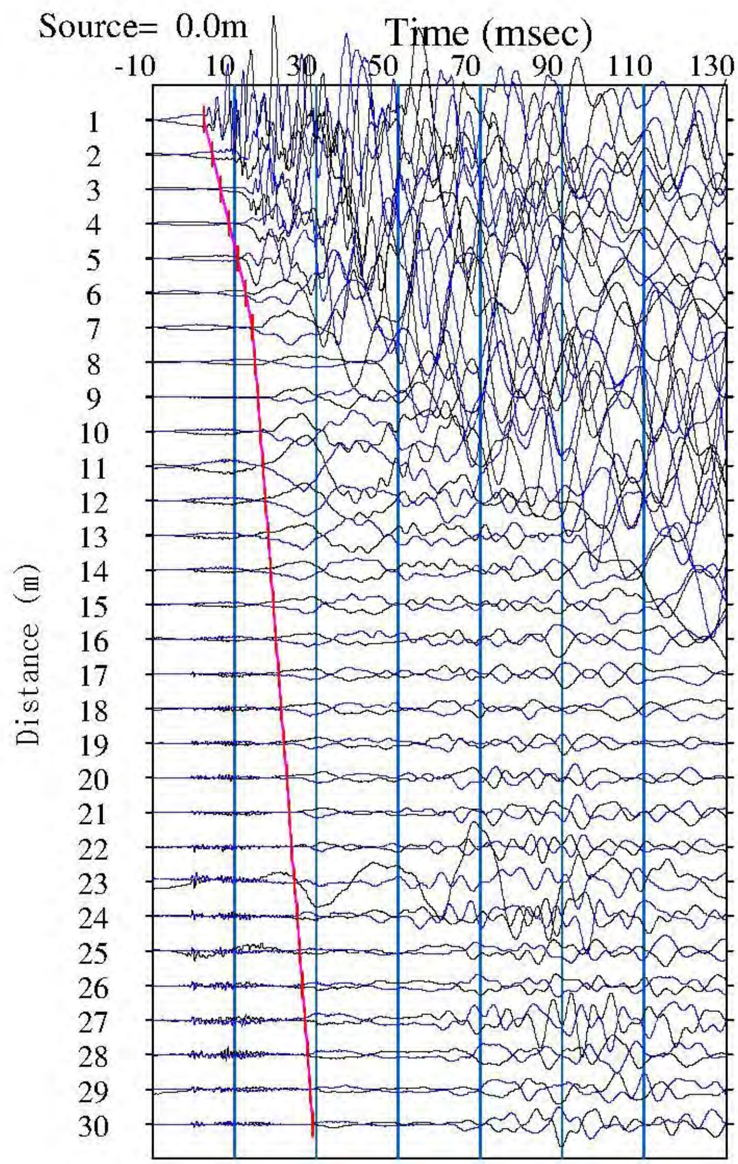


Figure 3: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 16-104.

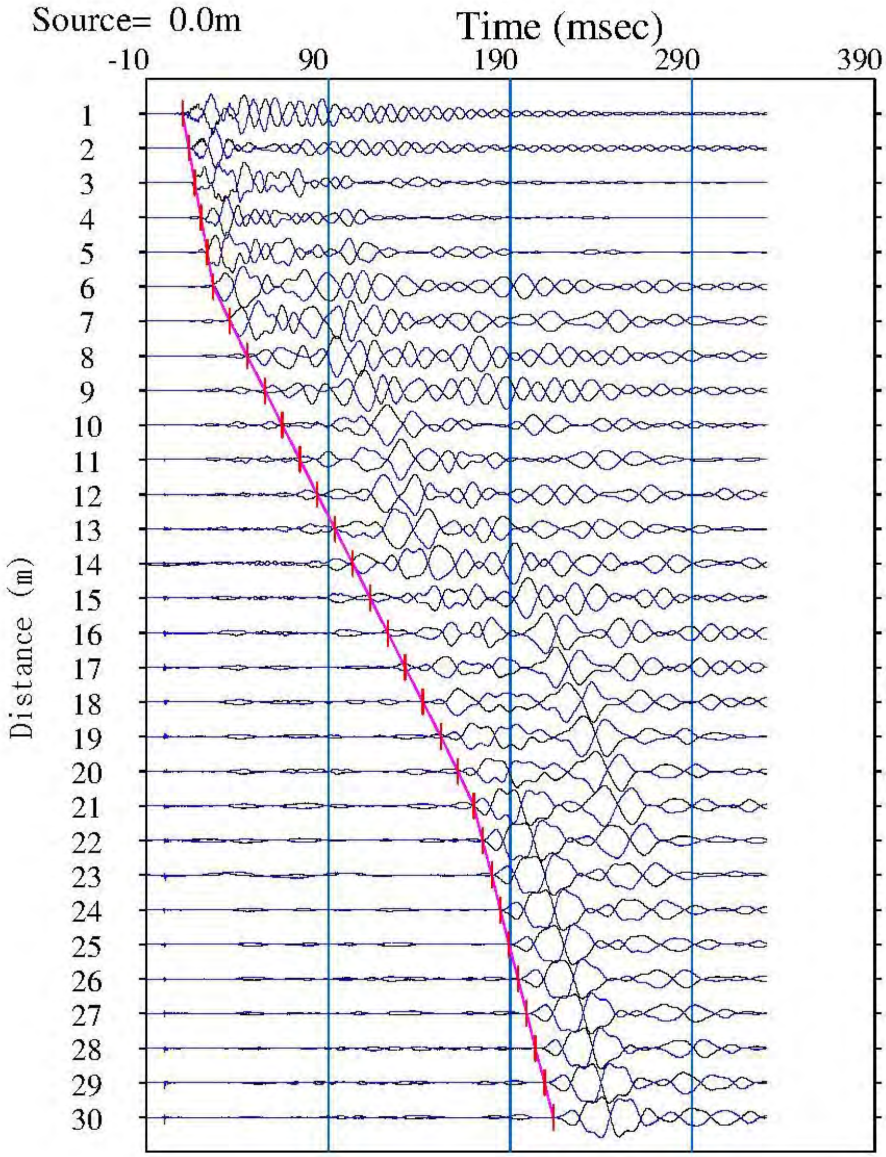


Figure 4: First break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 16-104.

Results

The VSP results for boreholes 16-101 and 16-104 are summarized in Tables 1 and 2, respectively. The shear wave and compression wave layer velocities were calculated by best fitting a theoretical travel time model to the field data. The depths presented on the table are relative to ground surface.

The estimated dynamic engineering moduli, based on the calculated wave velocities, are also presented in Tables 1 and 2. The engineering moduli were calculated using an estimated bulk density, based on the borehole log. For the overburden down to 30 metres, the bulk density of 1,750 kg/m^3 was used.

The average shear wave velocity from ground surface to a depth of 30 metres was measured to be 173 metres per second at borehole 16-101 and 140 metres per second at borehole 16-104.

Limitations

This technical memorandum, which specifically includes all tables, figures and attachments, is based on data and information collected by Golder Associates Ltd. and is based solely on the conditions of the properties at the time of the work, supplemented by historical information and data obtained by Golder Associates Ltd. as described in this memo.

Golder Associates Ltd. has relied in good faith on all information provided and does not accept responsibility for any deficiency, misstatements, or inaccuracies contained in the reports as a result of omissions, misinterpretation, or fraudulent acts of the persons contacted or errors or omissions in the reviewed documentation.

The services performed, as described in this memo, were conducted in a manner consistent with that level of care and skill normally exercised by other members of the engineering and science professions currently practicing under similar conditions, subject to the time limits and financial and physical constraints applicable to the services.

Any use which a third party makes of this memo, or any reliance on, or decisions to be made based on it, are the responsibilities of such third parties. Golder Associates Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this memo.

The findings and conclusions of this memo are valid only as of the date of this memo. If new information is discovered in future work, including excavations, borings, or other studies, Golder Associates Ltd. should be requested to re-evaluate the conclusions of this memo, and to provide amendments as required.

Closure

We trust that these results meet your current needs. If you have any questions or require clarification, please contact the undersigned at your convenience.

GOLDER ASSOCIATES LTD.

Stephane Sol, Ph.D., P.Geo
Senior Geophysicist

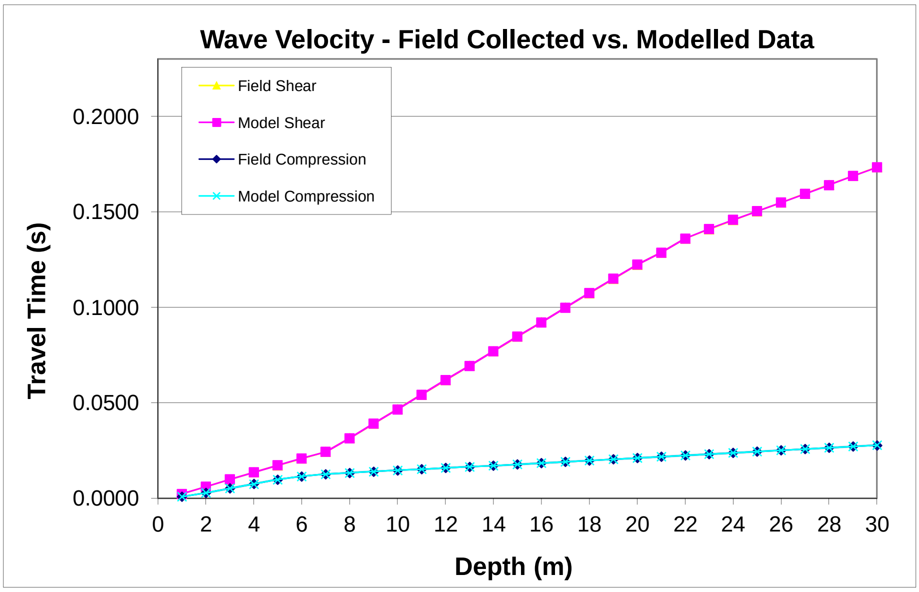
Christopher Phillips, M.Sc., P.Geo
Principal, Senior Geophysicist

SS/CRP/jl
\\golder.gds\gal\ottawa\active\2012\1121 - geotechnical\12-1121-0193 dillon mega 3 eastern region\geophysics\12-1121-0193 vsp ottawa\report\12-1121-0193 tech memo 2016oct06 vsp.docx

Attachment: Table 1 – Shear Wave Velocity Profile at BH-16-101
Table 2 – Shear Wave Velocity Profile at BH-16-104

TABLE 1
SHEAR WAVE VELOCITY PROFILE AT BH 16-101

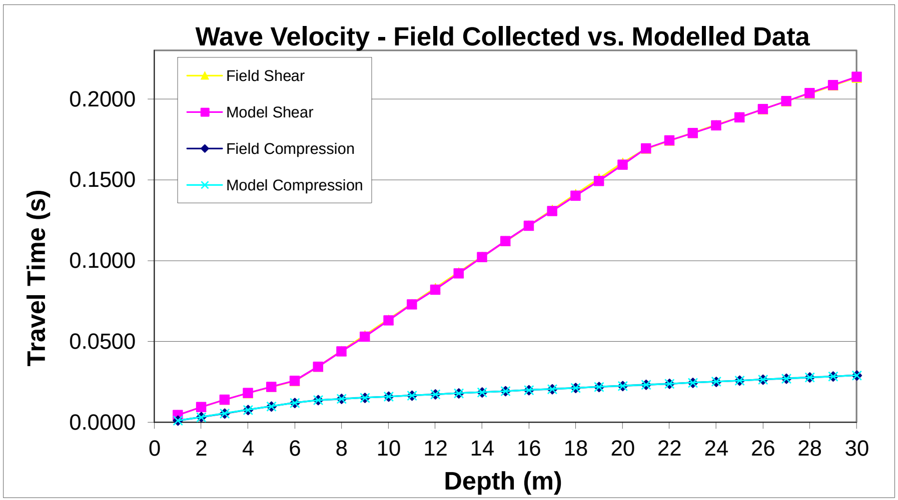
Layer Depth (m)					Dynamic Engineering Properties			
Top	Bottom	Compressional Wave (m/s)	Shear Wave (m/s)	Estimated Bulk Density (kg/m^3)	Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0	1	1140	430	1750	0.42	324	917	1843
1	2	490	270	1750	0.28	128	327	250
2	3	440	260	1750	0.23	118	291	181
3	4	430	270	1750	0.17	128	300	153
4	5	430	270	1750	0.17	128	300	153
5	6	600	280	1750	0.36	137	373	447
6	7	850	290	1750	0.43	147	422	1068
7	8	1460	140	1750	0.50	34	103	3685
8	9	1500	130	1750	0.50	30	89	3898
9	10	1600	135	1750	0.50	32	95	4437
10	11	1600	130	1750	0.50	30	89	4441
11	12	1600	130	1750	0.50	30	89	4441
12	13	1620	135	1750	0.50	32	95	4550
13	14	1630	130	1750	0.50	30	89	4610
14	15	1490	130	1750	0.50	30	88	3846
15	16	1490	135	1750	0.50	32	95	3843
16	17	1490	130	1750	0.50	30	88	3846
17	18	1490	130	1750	0.50	30	88	3846
18	19	1490	135	1750	0.50	32	95	3843
19	20	1490	135	1750	0.50	32	95	3843
20	21	1490	160	1750	0.49	45	134	3825
21	22	1500	135	1750	0.50	32	95	3895
22	23	1490	200	1750	0.49	70	209	3792
23	24	1500	210	1750	0.49	77	230	3835
24	25	1500	220	1750	0.49	85	252	3825
25	26	1520	220	1750	0.49	85	252	3930
26	27	1520	220	1750	0.49	85	252	3930
27	28	1520	220	1750	0.49	85	252	3930
28	29	1520	210	1750	0.49	77	230	3940
29	30	1520	220	1750	0.49	85	252	3930



Notes
1. Depth Presented relative to ground surface.
2. This Table to be analyzed in conjunction with the accompanying report.

TABLE 2
SHEAR WAVE VELOCITY PROFILE AT BH 16-104

Layer Depth (m)				Estimated Bulk Density	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave (m/s)	Shear Wave (m/s)	(kg/m^3)	Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0	1	880	225	1750	0.47	89	260	1237
1	2	480	200	1750	0.39	70	195	310
2	3	430	220	1750	0.32	85	224	211
3	4	460	240	1750	0.31	101	265	236
4	5	450	260	1750	0.25	118	296	197
5	6	470	270	1750	0.25	128	320	216
6	7	580	115	1750	0.48	23	68	558
7	8	1350	105	1750	0.50	19	58	3164
8	9	1380	110	1750	0.50	21	63	3304
9	10	1440	100	1750	0.50	18	52	3605
10	11	1440	100	1750	0.50	18	52	3605
11	12	1500	110	1750	0.50	21	63	3909
12	13	1500	100	1750	0.50	18	52	3914
13	14	1500	100	1750	0.50	18	52	3914
14	15	1500	100	1750	0.50	18	52	3914
15	16	1500	105	1750	0.50	19	58	3912
16	17	1550	110	1750	0.50	21	63	4176
17	18	1530	105	1750	0.50	19	58	4071
18	19	1530	110	1750	0.50	21	63	4068
19	20	1530	100	1750	0.50	18	52	4073
20	21	1530	100	1750	0.50	18	52	4073
21	22	1540	200	1750	0.49	70	209	4057
22	23	1550	220	1750	0.49	85	252	4091
23	24	1550	210	1750	0.49	77	230	4101
24	25	1550	200	1750	0.49	70	209	4111
25	26	1550	200	1750	0.49	70	209	4111
26	27	1550	200	1750	0.49	70	209	4111
27	28	1550	200	1750	0.49	70	209	4111
28	29	1550	200	1750	0.49	70	209	4111
29	30	1550	200	1750	0.49	70	209	4111



Notes
1. Depth Presented relative to ground surface.
2. This Table to be analyzed in conjunction with the accompanying report.

APPENDIX D

Non-Standard Special Provisions

CSP for Integral Abutments
Rigid Polystyrene Embankment
Dewatering

<u>CSP FOR INTEGRAL ABUTMENTS</u> - Item No.

Special Provision

<u>Scope</u>

This specification covers the requirements for the installation of the CSP's, including sand fill and polystyrene sheets, at the integral abutments.

<u>References</u>

This specification refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction:

OPSS 906	Construction Specification for Structural Steel for Bridges
OPSS.PROV 909	Construction Specification for Prestressed Concrete - Precast Girders

Ontario Provincial Standard Specifications, Material:

OPSS 1605	Material Specification for Extruded Expanded Polystyrene Pavement Insulation
OPSS 1801	Material Specification for Corrugated Steel Pipe (CSP) Products

Canadian Standards Association Standards:

CSA G164-M Hot Dip Galvanizing of Irregularly Shaped Articles

Ministry of Transportation Publications

MTO Manual of Designated Sources of Materials

<u>Definitions</u>

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

Abutment Stem: means the cast-in-place concrete component of the abutment placed over the top of the piles and forming the bearing seat for the girders.

CSP: means helical corrugated steel pipe.

Design Engineer: means the Engineer who produces the design and/or working drawings, and who has a minimum of five (5) years in the design and/or construction of bridges.

<u>Submission and Design Requirements</u>

Submissions

All submissions shall bear the seal and signature of the Design Engineer.

At least two weeks prior to commencement of installation of the abutment, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times.

Working Drawing Requirements

Working drawings shall include at least the following:

1. Layout and Elevations of the CSP's;
2. Source of the sand fill, and description of placing method and equipment;
3. Location and details of all temporary bracing, including permanent and temporary spacers, for the piles, CSP's and abutment stems;
4. Detailed construction sequence for the work, including installation and removal of the temporary bracing.

Design Requirements

The Contractor shall be responsible for the complete detailed design of the construction sequence for the work, including the installation and removal of all temporary bracing. The general sequence of construction shall be as shown on the Contract drawings.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including temporary and permanent spacers, required to maintain the piles, CSP's, abutment stems and girders in their specified positions through all stages of construction until concrete in deck has reached a compressive strength of 25 MPa. All temporary bracing, except spacers identified as permanent on the Contract drawings, shall be removed.

Temporary bracing for prestressed, precast girders shall meet the requirements of OPSS.PROV 909. Temporary bracing for structural steel girders shall meet the requirements of OPSS 906.

<u>Material</u>

Corrugated Steel Pipe

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM # 4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

Permanent Spacers and Associated Hardware

Permanent spacers and associated hardware left in place shall not consist of wood and corrodible material.

Sand Fill

The sand fill for backfilling the CSP's shall meet the gradation requirements of Table 1 below:

Table 1 - Sand Fill Gradation Requirements

MTO Sieve Designation		Percentage Passing by Mass
2 mm	# 10	100 %
600 μm	# 30	80 % to 100 %
425 μm	# 40	40 % to 80 %
250 μm	# 60	5 % to 25 %
150 μm	# 100	0 % to 6 %

Expanded Extruded Polystyrene

Expanded extruded polystyrene shall be in accordance with OPSS 1605, and shall be from a supplier listed under DSM # 3.30.30.

<u>Construction</u>

General

The sequence of construction for installing the concrete pads, CSP's, sand fill and abutment stems, including the installation and removal of the temporary bracing, shall be in accordance with the working drawings.

The Contractor shall not proceed with the abutment backfill above the level of the bottom of the CSP's without written permission from the Contract Administrator.

Corrugated Steel Pipe

CSP's shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSP's will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSP's shall be in accordance with the manufacturer's recommendations. Damaged CSP's shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSP's shall be repaired by two coats of zinc-rich paint.

The Contractor shall set the CSP over each pile in the abutment into the concrete pad, following the batter of the pile, while the concrete in the concrete pad is still plastic. The CSP's shall extend at least 150 mm into the concrete pad.

The Contractor shall ensure the full perimeter of the tops of all CSP's at each abutment are at the elevation shown on the working drawings.

After the CSP's have been set, the Contractor shall take all measures necessary to prevent the ingress of water, backfill and debris into the CSP's.

Sand Fill

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand fill itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP's.

After the sand fill has been placed to the top of each CSP, the Contractor shall take all measures necessary to prevent the ingress of water and other liquids into the sand fill until after the concrete in the abutment stem has been placed and cured.

Expanded Extruded Polystyrene

The expanded extruded polystyrene sheets shall completely cover the area under the abutment stem as shown on the Contract drawings. The sheets shall be placed in one piece for the width of the abutment stem, with butt joints perpendicular to the centre-line of abutment bearings. The minimum length of sheet shall be 500 mm.

Joints between sheets within 500 mm of a pile centre-line will not be permitted. At each pile location, a minimum 1000 mm long sheet shall be centred on the pile and a 500 mm diameter hole neatly cut in the sheet so as to fit over the pile in one piece.

The Contractor shall adjust the backfill to ensure full and uniform contact of the sheets with the backfill and the full perimeter of the tops of the CSP's. The vertical step at joints between sheets shall not exceed 5 mm.

The Contractor shall protect the sheets from damage during installation of the reinforcing for the abutment stem, and shall secure the sheets from "floating" during placing of the concrete in the abutment stem. Only hardware approved by the Owner shall be used to secure the sheets. All hardware used to secure the sheets shall be installed so as not to project above the top surface of the sheets into the abutment stem.

Temporary Bracing

Temporary bracing shall be installed and removed in accordance with the working drawings.

The temporary bracing shall not distort, nor pierce the walls of, the CSP's. Welding to the CSP's will not be permitted.

Concrete anchors shall be removed and the holes filled with non-shrink grout.

Tolerances

The CSP's at each pile shall be constructed to the following tolerances:

Criteria	Tolerance
Maximum deviation of CSP from pile centroid.	± 25 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified Elevation.	± 10 mm

Quality Assurance

Prior to placing the CSP's, the Contractor shall establish reference points at each abutment and determine the location of the centroid of each pile in the abutment with respect to these reference points. The Contractor shall maintain the reference points until written permission to proceed with the backfill above the level of the bottom of the CSP's has been given by the Contract Administrator.

Measurement for Payment

There will be no measurement for this item.

Basis of Payment

Payment at the contract price for the above items shall be full compensation for all labour, equipment and material required to do the work.

DEWATERING – Item No.

Special Provision

SCOPE

The work under this item includes the design, installation, operation, maintenance and removal of temporary dewatering systems to facilitate the construction of the foundations for the Mississippi River bridge replacement on Highway 17.

Construction of the bridge replacement will require excavation into granular embankment fills, organic deposits and silty clay to clay below the groundwater level at the site and below the water level in the Mississippi River. The cohesionless soils below the groundwater table will be subjected to conditions of unbalanced hydrostatic head and can slough, boil and cave in during temporary excavation work.

REFERENCES

OPSS 902	Construction Specification for Excavation and Backfill – Structures
OPSS 518	Construction Specification for Control of Water from Dewatering Operations

SUBMISSION AND DESIGN REQUIREMENTS

Written details for the proposed dewatering system shall be submitted to the Contract Administrator for information purposes a minimum of ten business days prior to commencing dewatering operations. The Contractor shall reference borehole logs included in the contract documents as a guide in determining requirements.

CONSTRUCTION

Dewatering System

The Contractor is responsible for the design, installation, operation and maintenance of an adequate dewatering system to lower the groundwater level to at least 0.3 m below the required excavation level for the bridge replacement, to allow excavation, foundation subgrade preparation and foundation construction in dry conditions.

Water pumped from trenches shall be redirected into the watercourse downstream of the work area in a manner that is not injurious to public health or safety, to property, to the environment or to any part of the work already completed or under construction.

Operation

A continuous dewatering operation shall be provided to facilitate the construction at all times during the work. All components of the dewatering system shall be maintained in an effective, functioning and stable condition at all times during the work. Notwithstanding the above, the work shall be completed in accordance with the environmental and operational constraints specified elsewhere in the contract.

Restoration

All equipment and materials placed shall be removed from the right-of-way upon the completion of the work and all areas disturbed as part of this work shall be restored to their preconstruction conditions, unless specified otherwise.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and material to do the work.

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

Non-Standard Special Provision

1.0 SCOPE

This special provision covers the requirements for the supply and construction of the expanded polystyrene embankment fill, including foundation preparation, excavation, leveling pad, polyethylene sheeting and associated works as shown on the contract drawings.

2.0 REFERENCES

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

National Standards of Canada

CAN/CGSB - 51.20 M87 Thermal Insulation, Polystyrene, Boards and Pipe Covering

ASTM

ASTM D6817 Standard Specification for Rigid Cellular Polystyrene Geofoam

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 Plastics

ASTM D2126 Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

OPSS - Ontario Provincial Standard Specification

OPSS.PROV 212	Construction Specification for Earth Borrow
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS 902	Construction Specification for Excavating and Backfilling - Structures
OPSS.PROV 1010	Material Specification for Aggregates-Base, Subbase, Select Subgrade, and Backfill Material
OPSS 1605	Material Specification for Extruded Expanded Polystyrene Pavement Insulation
OPSS 1860	Material Specification for Geotextiles

3.0 SUBSURFACE CONDITIONS

The subsurface conditions at the site are described in the geotechnical investigation reports for this Contract.

4.0 DEFINITIONS

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

Rigid Expanded Polystyrene

Molded rigid blocks produced by a process of pre-expansion, aging and forming of a petroleum based raw material.

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: 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.

5.0 QUALIFICATION

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.

6.0 SUBMISSION AND DESIGN REQUIREMENTS

6.1 Submission of Shop Drawings

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

6.2 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 requirements.

6.3 Construction

The contractor shall submit full details of the following.

a) The method of foundation excavation and preparation.

b) Construction of granular leveling pad.

c) The method of placement of expanded polystyrene including temporary ballasting (if required) 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 protective concrete slab.

f) The method of placement of subbase material.

g) The method of placement of side slope cover.

7. MATERIALS

7.1 Granular Leveling Pad

The leveling pad shall consist of a Granular ‘A’ or Granular ‘B’ Type II material with gradation and physical requirements as specified in OPSS 1010.

7.2 Rigid Expanded Polystyrene

7.2.1 General

7.2.1.1 The Contractor shall submit:

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 expanded polystyrene.

5. The physical and mechanical properties of the rigid expanded polystyrene including:

    1. Geometry
    2. Nominal Density
    3. Compressive Strength
    4. Flexural Strength
    5. Dimensional Stability
    6. Oxygen Index
    7. 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 Contract Administrator for review.

8. To the Contract Administrator, a Certificate of Conformance sealed and signed by the Quality Verification Engineer. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents. Certificate to be submitted a minimum of one week prior to commencement of work under this item.

7.2.1.2 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.

7.2.2 Detail Requirements

The polystyrene shall meet the requirements for EPS22, as defined by ASTM D6817-02, as follows:

TABLE 1 – MATERIAL PROPERTIES

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear - Flatness - Squareness - Thickness	Mm	1200 x 600 x 100 ± 0.5%	
Compressive Strength at 5% strain	kPa (min)	115	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	276	ASTM C203
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	4	ASTM D2842

7.2.2.1 Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel sheets bundled into minimum acceptable dimensions of 1200 mm x 600 mm x 100 mm.

The maximum deviation from the specified linear dimensions, flatness, squareness and thickness shall be ± 0.5%.

7.2.2.2 Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 115 kPa at a strain of not more than 5%. The maximum design permanent stress level must not exceed 30% of the compressive strength of the material at 5% strain.

7.2.2.3 Flexural Strength

The minimum flexural strength of the polystyrene shall be 276 kPa. The flexural strength shall be determined in accordance to ASTM C203, Method 1, Procedure B.2.7.4 Dimensional Stability.

7.2.2.4 Dimensional Stability

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

7.2.2.5 Flammability

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

7.2.2.6 Water Absorption

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

7.2.2.7 Chemical Resistance

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

7.2.2.8 Biological Resistance

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

7.2.2.9 Environmental

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

8.0 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.

9.0 CONSTRUCTION

9.1 Foundation Excavation

Foundation excavation shall be carried out to the design elevations shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with Granular 'A' or Granular 'B' material.

9.2 Leveling Pad

Place, level and compact a layer of Granular 'A' or Granular 'B' Type II 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.

9.3 Installation of Polystyrene

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 90º 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 a maximum joint 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 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.

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 10 mil 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 side slope of the rigid expanded polystyrene embankment shall be covered with fill material as detailed elsewhere in this contract.

12) The Contractor shall submit 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 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.

13) The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer, a minimum of one week prior to the 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.***

10. EQUIPMENT

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

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.

11. QUALITY ASSURANCE

11.1 Quality Assurance

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.

11.2 Sampling and Testing

11.2.1 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 may be conducted. The testing shall be conducted by a recognized testing laboratory accredited by the Standards Council of Canada.

11.2.2 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, one (1) block shall be tested for the full suite of tests and three (3) blocks shall be tested for compressive strength.

11.2.3 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.

12.0 Measurement for Payment

12.1 Actual Measurement

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

13.0 Payment

13.1 Basis of Payment

The granular leveling pad shall be included in the work and shall not be measured for separate payment.

Payment at the contract price for the above tender item shall be full compensation for all labour, materials, and equipment to do the work as described above and no extra payments will be made.

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.

Africa	+ 27 11 254 4800
Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
1931 Robertson Road
Ottawa, Ontario, K2H 5B7
Canada
T: +1 (613) 592 9600