January 2016

REPORT ON

Foundation Investigation and Design Replacement of Highway 401 Underpass at Avonmore Road, Site No. 31-177 Highway 401, 8.5 km West of Highway 138 Township of Cornwall W.P. 4382-01-01 G.W.P. 4064-12-00

Submitted to:
MMM Group Ltd.
1145 Hunt Club Road, Suite 300
Ottawa, Ontario
K1V 0Y3

REPORT

Report Number: 12-1121-0099-1730
Geocres Number: 31G-253

Distribution:

3 copies - Ministry of Transportation, Kingston
1 copy - Ministry of Transportation, Downsview
2 copies - MMM Group Ltd.
1 copy - Golder Associates Ltd.

A world of capabilities delivered locally

Golder Associates

Table of Contents

TABLES

DRAWINGS

APPENDICES

PART A

FOUNDATION INVESTIGATION REPORT
REPLACEMENT OF HIGHWAY 401 UNDERPASS
AT AVONMORE ROAD, SITE 31-177
HIGHWAY 401, 8.5 KM WEST OF HIGHWAY 138
TOWNSHIP OF CORNWALL
W.P. 4382-01-01
G.W.P. 4064-12-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by MMM Group Ltd. (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the Design-Build of bridge and culvert replacements at various locations in the Eastern Region of Ontario as part of the 22 Structures MEGA 2 project. Foundation investigation and input for the detailed design of several bridge replacements was added to the overall scope of work following award of the project. This report presents the results of the detailed foundation investigation conducted for the replacement of the Avonmore Road (County Road 15) underpass, Site No. 31-177 located on Highway 401 about 8.5 km west of Highway 138 near Cornwall, Ontario.

The purpose of the foundation investigation was to assess the subsurface conditions for the proposed bridge replacement by drilling three boreholes and carrying out in situ testing and laboratory testing on selected samples. As part of the overall scope of work, previously collected subsurface information pertinent to the site was obtained from the MTO's Foundation Library and included the following:

- Report prepared by Associated Geotechnical Services Limited for the MTO (then Department of Highways of Ontario) titled "*Foundation Investigation Report, Proposed Structure: Hwy 401 & Cty Rr, Lots 30 and 31, Cornwall Township, W.P. 100-59, District No. 9*", dated April 1960 (Geocres No. 31G00-122).

The terms of reference for the scope of work are outlined in the MTO's Request for Proposal (RFP) dated April 2012 and the work was carried out in accordance with Golder's change proposal to MMM, dated July 10, 2014.

2.0 SITE DESCRIPTION

The Avonmore Road underpass is located on Highway 401 at Avonmore Road, approximately 8.5 km west of Highway 138 within the Township of Cornwall in the United Counties of Stormont, Dundas, and Glengarry. Avonmore Road is also known as County Road 15.

The existing bridge consists of a four span continuous concrete T-beam structure overlain with an asphalt wearing surface. The superstructure is founded on three piers and two abutments, all founded on H-piles. Cantilevered wingwalls extend from the abutment stems and are approximately 3.9 metres in length. The existing structure is aligned approximately northwest-southeast, and is about 64.6 m long and 10.4 m wide. It is understood that the structure was built circa 1962 and was previously rehabilitated in 1979 and 1992.

Highway 401 in this area is a four-lane, divided highway. Avonmore Road is a two-lane county road. In the area of the bridge, Avonmore Road has been constructed on embankments that are on the order of about 6.0 m in height above Highway 401 and the natural ground level, with the Avonmore Road pavement surface at about Elevation 88.7 m in the vicinity of the bridge. The Avonmore Road embankment side slopes are oriented between about 2 horizontal to 1 vertical and 3 horizontal to 1 vertical (i.e., 2H:1V and 3H:1V). Based on visual observation at the time of the site investigation, the existing embankment slopes appear to be performing satisfactorily. The pile caps for the abutments are located within the embankments at about Elevation 86 m.

2.1 Regional Geological Conditions

The site is located in the physiographic region known as the Lancaster flats, just east of the Glengarry till plain, as delineated in The Physiography of Southern Ontario.[1]

The Lancaster flats region is characterized by poorly drained clay to fine grained sand deposits over glacial till plains.[1]

[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.

3.0 INVESTIGATION PROCEDURES

The subsurface investigation for the proposed bridge replacement was carried out between October 23 and 31, 2014, during which time three boreholes (numbered 14-731 to 14-733, inclusive) were advanced at the locations shown on Drawing 1.

The boreholes were advanced with 200 mm diameter continuous-flight hollow-stem augers and/or rotary drilling during rock coring with a truck-mounted drill rig, supplied and operated by CCC Geotechnical and Environmental Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to total depths of about 9.4 to 17.2 m below the existing ground surface. Following penetration of the overburden soil, the boreholes were cored between about 3.0 to 3.4 m into the bedrock. Soil samples in the boreholes were obtained at regular intervals using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures.

A standpipe piezometer was installed in Borehole 14-731 to monitor the groundwater level at the site. The standpipe consists of a 51 mm diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within silica sand backfill and sealed by a section of bentonite pellet backfill. The boreholes were backfilled with bentonite pellets, mixed with native soils in the overburden and bentonite pellets in the bedrock. The site conditions were restored following completion of work.

The field work was observed by members of Golder’s technical and engineering staff, who located the boreholes, supervised the drilling, sampling and in situ testing operations, logged the subsurface conditions encountered in the boreholes, and examined and cared for the soil and bedrock samples. The samples were identified in the field, placed in appropriate containers, labelled, and transported to Golder’s laboratory in Ottawa for further examination. Index and classification tests consisting of grain size distribution and water content testing were carried out on selected soil samples, and unconfined compressive strength tests were carried out on selected rock core samples obtained during the investigation. All of the laboratory tests were carried out to MTO LS and/or ASTM standards as appropriate.

The coordinates and ground surface elevations at the borehole locations were determined by Golder personnel using a Trimble R8 GPS survey unit. The geodetic reference system used for the survey is the North American datum of 1983 (NAD83). The borehole coordinates (based on the Modified Transverse Mercator (MTM Zone 8) coordinate system) and elevations are summarized in the following table and are shown on Drawing 1.

Borehole Number	Borehole Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)
14-731	North Abutment	4991810.3	196424.1	88.9
14-732	South Abutment	4991739.5	196461.4	89.1
14-733	Central Pier (Within the median of Highway 401)	4991770.1	196432.6	82.5

Notes: 1) Northing and Easting coordinates shown are relative to the MTM NAD83 (Zone 8) coordinate system.

2) Ground surface elevations shown are relative to Geodetic Datum.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 General

The detailed subsurface soil, bedrock, and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole and Drillhole sheets contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B4 contained in Appendix B.

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

In general, the subsurface conditions at the location of the proposed bridge replacement consist of granular embankment fill overlying gravel and sand, silty clay to clay, and glacial till. The overburden soil is underlain by limestone bedrock.

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

4.2 Pavement Structure and Embankment Fill

The Avonmore Road pavement structure was penetrated in the northbound lane at Boreholes 14-731 and 14-732. At borehole 14-731, the pavement structure consists of about 200 mm of asphaltic concrete overlying about 2.1 m of gravelly sand base. At borehole 14-732, the pavement structure consists of about 100 mm of asphaltic concrete overlying about 400 mm of Portland cement concrete. The concrete at borehole 14-732 is underlain by about 2.2 m of sand and gravel fill.

The granular base is underlain by about 4.9 to 5.3 m of embankment fill. The embankment fill generally consists of gravelly silty sand to silty sand with some gravel that contains cobbles. Trace organic matter was encountered in samples of the fill recovered from both of the boreholes. In addition, asphalt pieces were encountered within the fill at some locations. The embankment fill was fully penetrated to depths of about 7.6 m (Elevations 81.3 and 81.5 m at Boreholes 14-731 and 14-732, respectively).

Standard Penetration Test (SPT) “N” values measured in the fill generally range from 7 to 65 blows per 0.3 m of penetration, indicating the fill is loose to very dense. Refusal to advancement of the split-spoon sampler was encountered in the fill at three locations in Boreholes 14-731 and 14-732, with SPT “N” values of 39 per 0.2 m of penetration, 112 blows per 0.2 m of penetration, and 113 blows per 0.2 m of penetration, respectively; this refusal is inferred to have occurred as result of encountering cobbles and/or boulders in the fill.

The results of grain size distribution testing carried out on three samples of the Avonmore Road approach embankment fill are provided on Figure B1 in Appendix B. The test results do not reflect the cobble/boulder or full gravel content of the material, since the samples were retrieved using a 50 mm outside diameter split-spoon sampler. The measured water contents of selected samples of the fill range from about 4 to 8 percent.

A 200 mm thick layer of topsoil was encountered at ground surface within the median of Highway 401 at Borehole 14-733. This topsoil layer was underlain by fill materials comprised of silt and sand containing some gravel. Organic matter was also encountered within this fill. The fill at Borehole 14-733 was encountered to a depth of 1.5 m. One SPT “N” value of 5 blows per 0.3 m of penetration was measured in the fill, indicating a loose relative density.

4.3 Silty Sand to Sand and Gravel

A predominantly granular deposit varying in composition from silty sand to sand and gravel was encountered beneath the fill materials at Boreholes 14-731 and 14-732. Beneath the north embankment, at Borehole 14-731, and within the highway median, at Borehole 14-733, the deposit consists of silty sand containing varying amounts of gravel. Beneath the south embankment, at Borehole 14-732, the deposit consists of sand and gravel with some silt. White shell fragments were present in the samples obtained at all borehole locations.

The deposit was fully penetrated at depths of about 8.4 m below ground surface at the embankments and about 1.8 m below the existing ground surface at the median (corresponding to elevations of between 80.5 and 80.7 m).

Standard Penetration Test (SPT) “N” values measured in the deposit range from 10 to 23 blows per 0.3 m of penetration, indicating this deposit is typically compact.

The results of a grain size distribution test carried out on a sample of the sand and gravel encountered at the south embankment are provided on Figure B2 in Appendix B. The measured water content of this sample was about 10 percent.

4.4 Silty Clay to Clay

A deposit of silty clay to clay was encountered below the silty sand to sand and gravel deposit at all borehole locations. The silty clay is grey to grey brown in colour and samples obtained from Boreholes 14-731 and 14-733 contain trace sand and gravel. The deposit is about 0.7 to 1.1 metres thick and was fully penetrated in the boreholes to a depth of about 2.9 metres in Borehole 14-733 and 9.1 metres in Boreholes 14-731 and 14-732 (corresponding to elevations of about 79.6 to 80.0 m).

Standard Penetration Test (SPT) “N” values measured in the silty clay ranged from 9 to 14 blows per 0.3 m of penetration, suggesting a stiff consistency.

Atterberg limit testing carried out on two samples of the silty clay obtained from Boreholes 14-731 and 14-732 measured plasticity index values of about 28 to 34 percent and liquid limit values of about 52 to 57 percent, indicating clay of intermediate to high plasticity. The measured water contents of two samples of the grey brown silty clay were about 28 and 26 percent.

4.5 Glacial Till

The silty clay is underlain by a deposit of glacial till. In general, the glacial till is a heterogeneous mixture of gravel, cobbles and boulders in a matrix of silty sand to silt and sand. The surface of the till deposit was encountered between Elevations 79.6 and 80.0 m, and the deposit was fully penetrated in the boreholes to depths between 6.0 and 14.2 m (corresponding to Elevations 74.9 to 76.5 m). At the borehole locations, the glacial till had a thickness between about 3.1 and 5.1 m.

The SPT “N” values measured in the glacial till range from 8 to 84 blows per 0.3 m of penetration, indicating a highly variable, loose to very dense state of packing.

The results of grain size distribution testing carried out on five selected samples of the glacial till are provided on Figures B3 to B5 in Appendix B. These test results do not reflect the cobble/boulder or full gravel content of the material, since the samples were retrieved using a 50 mm outside diameter split-spoon sampler. The measured natural water contents of the five selected samples of the till ranged from about 6 to 10 percent.

4.6 Bedrock

At all borehole locations, bedrock was encountered beneath the glacial till. The bedrock was cored for lengths of between 3.0 and 3.4 m. Photographs of the bedrock core recovered are included in Appendix D. The following table summarizes the bedrock surface depths and elevations as encountered at the three borehole locations.

Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
14-731	88.9	12.5	76.4
14-732	89.1	14.2	74.9
14-733	82.5	6.0	76.5

The bedrock encountered in the boreholes typically consists of grey to black, nodular limestone to shaley limestone and is thinly to medium bedded. The bedrock is fresh and typically strong to very strong.

The Rock Quality Designation (RQD) values measured on the recovered bedrock core samples range from about 75 to 100 percent, indicating a good to excellent quality rock. The discontinuities observed in the rock core are associated with the joints and bedding of the bedrock.

Laboratory unconfined compressive strength testing was carried out on selected specimens of the bedrock core. The results of the testing are summarized on Figure B6 in Appendix B. The results of the unconfined compressive strength testing on three sample of the bedrock indicate values ranging from 98 to 186 MPa.

4.7 Groundwater Conditions

A monitoring well was installed in Borehole 14-731, and the groundwater level measured in the monitoring well is provided in the table below. The measured level about one month following completion of the drilling was at about the natural ground surface at the site.

Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date
14-731	88.9	7.0	81.9	November 26, 2014

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

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Sarah Ghadbane, E.I.T. and Mr. Matt Kennedy, P.Eng. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.

GOLDER ASSOCIATES LTD.

for

Sarah Ghadbane, E.I.T.
Junior Geotechnical Engineer

Matt Kennedy, P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
M. J. KENNEDY
100188946
Jan 13, 2016
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
Jan 13, 2016
PROVINCE OF ONTARIO

Fin Heffernan, P.Eng.
Designated MTO Foundations Contact

SG/MJK/KN/FJH/ob
\\golder.gds\gal\ottawa\active\2012\1121 - geotechnical\12-1121-0099 mrc 22 structures eastern region\foundations\6 - reports\package 7\31-177 avonmore road\12-1121-0099-1730 rpt-001 final avonmore road site 31-177 january 2016.docx

PART B

FOUNDATION DESIGN REPORT
REPLACEMENT OF HIGHWAY 401 UNDERPASS
AT AVONMORE ROAD, SITE 31-177
HIGHWAY 401, 8.5 KM WEST OF HIGHWAY 138
TOWNSHIP OF CORNWALL
W.P. 4382-01-01
G.W.P. 4064-12-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed replacement of the existing Avonmore Road (County Road 15) underpass on Highway 401. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the detailed design of the foundations for the replacement structure.

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

The existing bridge, the location of which is shown on Drawing 1, consists of a two-lane, four-span, concrete T-beam structure that was originally constructed circa 1962. The two middle spans are about 20.1 m long, and the two outer spans are about 12.2 m long. It is understood that the preferred alternative for the proposed replacement consists of a two-span structure on the same alignment as the existing bridge with a nominal increase in width of about 0.6 m. The new underpass will be founded on abutments located within or near the existing abutment foundation footprints. The proposed Avonmore Road pavement grades at the new structure will be up to about 0.9 m higher than the existing pavement grades.

6.2 Existing Foundations

Based on the 1960s design drawings (Drawings TWP #31-177-1-A to TWP #31-177-12-A), the existing abutment foundations are understood to consist of BP10x42 steel H-piles up to about 11 m long, driven to the bedrock surface at elevations between about 75.6 m and 76.8 m. There are two rows of seven vertical piles at each abutment, and an additional pile battered at about 1 horizontal to 4 vertical (1H:4V) beneath each of the abutment wingwalls. Based on the original design drawings, the design load on each existing pile is 30 tons (300 kN). Both abutment pile caps are perched within the existing Avonmore Road approach embankments with the top of each pile cap at an Elevation between about 86.1 m and 86.3 m. The existing pier foundations are also founded on BP10x42 steel H-piles. Drawing TWP #31-177-12-A indicates that the pier piles are about 5.5 m long with a design load of 40 tons (400 kN) per pile.

The piles would have been driven through the lower portion of the embankment fill which consists of a generally loose to compact gravelly silty sand that contains cobbles and boulders. The very dense till contains cobbles and boulders, which may have deflected the piles from their alignment and may have caused the driven piles to "hang up" above their intended design tip elevations.

6.3 Foundation Options

Based on the subsurface conditions, both shallow and deep foundation options have been considered for the replacement of the existing Avonmore Road underpass. 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 through the glacial till to refusal on the limestone bedrock are feasible for support of the replacement bridge structure, and this option would allow the pile caps to be maintained at a higher elevation than for a spread footing option at the abutments, thus minimizing excavation depth, protection system requirements and groundwater control requirements, while achieving relatively higher geotechnical resistances and minimizing settlement. Steel H-pile foundations would also allow for the construction of integral abutments. If the piles are driven, the use of pile points is recommended to minimize damage while penetrating the fill and glacial till deposits (which contains cobbles and boulders) and seating onto the limestone bedrock. Due to the presence of cobbles and boulders in the lower portions of the embankment fill and within the glacial till, some pre-augering may be required to facilitate installation of driven steel piles. As the proposed new abutments and central pier are to be located at approximately the same location as the existing foundation elements, consideration must be given to removal of, or avoiding interference with, the existing abutment and central pier piles.

- **Driven steel pipe (tube) piles:** Closed-ended steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutments, and this foundation option would have similar advantages to steel H-piles in terms of minimizing excavation depth, protection system requirements and groundwater control requirements. However, pipe piles are considered to have a higher risk than H-piles for "hanging up" or being deflected away from their vertical or battered orientation due to the presence of cobbles and/or boulders within the fill and the till deposits. As described for steel H-piles, pre-augering through portions of the embankment fill and the glacial till may be required due to the presence of cobbles and boulders.

- **Drilled concrete caissons:** Caissons deriving their support from bearing within the limestone bedrock are also feasible for this site. Caissons would require the use of temporary or permanent liners to mitigate the potential risks of ground loss from the water-bearing cohesionless till soils during construction. In addition, the caissons must be socketed into the bedrock a sufficient length to provide the required bearing resistance. The presence of cobbles and boulders within the fill and glacial till may require churn drilling and possibly rock coring techniques to penetrate obstructions where encountered. The caisson sockets will also have to be advanced by rock coring and/or chisel drilling into the strong to very strong limestone bedrock. For this deep foundation option, consideration must be given to removal of the existing abutment and central pier piles, as the proposed new foundation elements are to be located at approximately the same location as the existing elements; while new steel H-piles or pipe piles may be able to be located so as to avoid conflict with the existing piles, larger diameter caissons would likely necessitate removal of the existing piles.

- **Spread footings founded on glacial till:** Spread footings could be considered for support of the replacement structure, provided they are founded on or within the compact to very dense native till, below the silty clay encountered at the abutments and central pier location. An integral abutment configuration would not be achievable with this foundation type at the abutments. Some settlement (less than about 25 mm) of the abutment and/or pier footings may occur for footings founded on the glacial till. The groundwater table was encountered about 2 m above the top of the glacial till deposit at the monitoring well installed at the north embankment and therefore groundwater control would be required during excavation and construction. Depending on the preferred alignment of the replacement structure, removal of the existing pile caps and piles must also be taken into account. This option has not been considered further in this report for the abutment foundations due to the significant excavation depths, groundwater control requirements and protection system requirements that would be required for a shallow foundation system in comparison to a deep foundation option at the abutment locations. It is also noted that some differential settlement would occur between the pier and abutments for a pier foundations supported on a spread footing in comparison to all foundation units being supported on pile foundations bearing on bedrock. A spread footing at the central pier is not recommended if the footprint is within that of the existing deep foundation due to the potential for ground disturbance associated with removal of the existing piles.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments for the underpass replacement on steel H-piles driven to found on the bedrock, in an integral abutment configuration, and to also support the central pier on steel H-piles driven to found on the bedrock.

6.4 Shallow Foundations

6.4.1 Founding Elevations

If adopted for the replacement structure, spread footings should be founded on the typically compact to very dense glacial till below the silty clay deposit.

The following table provides the maximum (highest) founding elevations recommended for design of footings founded on the compact to very dense glacial till deposit. Excavation would be carried out to depths of up to about 9.1 m below the existing Avonmore Road grade at the abutments, and up to about 2.9 m below the existing grade at the central pier location.

Foundation Element	Borehole Number	Founding Stratum	Footing Founding Elevation (m)
North Abutment	14-731	Dense glacial till	Below 78.5
Central Pier	14-733	Compact to dense glacial till	Below 79.6*
South Abutment	14-732	Dense to very dense till	Below 80.0

Note: * Compacted Granular “A” fill could be used to raise the foundation level to Elevation 80.8 m, to minimize the concrete requirements while still maintaining the required foundation depth of 1.7 m for frost protection purposes.

The groundwater level was measured in the well installed at the site at Elevation 81.9 m in November 2014 and, therefore, dewatering of the lower portions of the excavations to the founding elevations presented below would be expected to be required depending on groundwater level encountered at the time of construction. A Non-Standard Special Provision has been provided in Appendix C to address this requirement.

At all borehole locations, a deposit of silty clay/clay was encountered beneath the embankment fill and above the underlying glacial till. Spread foundations must be founded below the clay. The thickness of the deposit encountered ranged from 0.7 m at the embankment boreholes to 1.1 m at the central pier borehole, but it may be more variable across the site. Therefore, the footing subgrade should be inspected in accordance with OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*) to check that all existing fill, organic deposits, silty clay, and other unsuitable material have been removed. The founding soils will be susceptible to disturbance and should be protected with a concrete working slab (100 mm thick concrete slab with a compressive strength of 20 MPa) if the concrete for the footing is not placed within four hours of the inspection and approval of the subgrade. A Non-Standard Special Provision has been provided in Appendix C to address this requirement.

6.4.2 Geotechnical Resistance

Spread footings placed on the properly prepared glacial till deposit, at or below the design elevations given in the preceding section, should be designed based on a factored geotechnical resistance of 500 kPa at Ultimate Limit States (ULS) and a geotechnical resistance of 300 kPa at Serviceability Limit States (SLS, for 25 mm of settlement).

These geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its *Commentary*.

6.4.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and subsoils should be calculated in accordance with Section 6.7.5 of the CHBDC. For cast-in-place concrete footings constructed on a concrete working slab that is cast on top of the glacial till, the coefficient of friction, tan δ or tan φ’, may be taken as follows:

- Cast-in-place footing to concrete working slab: tan δ = 0.6
- Cast-in-place concrete working slab to glacial till: tan φ’ = 0.62

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

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

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

6.5.1 Founding Elevations

The pier and abutments for the replacement structure may be supported on steel H-piles driven to found on the limestone bedrock or closed-ended steel pipe (tube) piles founded on the bedrock. As indicated previously, pipe piles are considered to have a higher risk than H-piles for “hanging up” or being deflected away from their vertical or battered orientation due to the presence of cobbles and/or boulders within the fill and the till deposits and, therefore, H-piles are recommended for use at this site. Based on the borehole results from the investigation, and assuming some nominal penetration into the bedrock, the following pile tip elevations are recommended for design of steel H-piles or pipe piles.

Foundation Element	Borehole Number	Bedrock Surface Elevation (m)	Design Pile Tip Elevation (m)
North Abutment	14-731	76.4	76.3
Central Pier	14-733	76.5	76.4
South Abutment	14-732	74.9	74.8

The pile caps should be constructed at a minimum depth of 1.7 m for frost protection purposes, per OPSD 3090.101 *(Foundation Frost Penetration Depths for Southern Ontario).*

If integral abutments are adopted, the upper portion of the piles would need to be cased in a sand-filled, corrugated steel pipe (or similar) to provide suitable flexibility of steel H-piles.

Depending on the preferred location of the abutment foundations, the piles may be driven behind or in front of the existing pile caps and piled foundations. Consideration may also be given to driving the new abutment piles adjacent to (or in between) the existing steel H-piles following removal of the existing pile cap and exposure of the existing piles.

The piles at the central pier may also be driven behind or in front of the existing pile cap and piles, or adjacent to (or in between) the existing piles following removal of the existing pile cap and exposure of the existing piles to reduce the potential conflict. If driving adjacent to the existing battered H-piles is not considered to be feasible, consideration may be given to extraction of the existing piles.

The existing piles (estimated to be about 11 m long at the abutments and about 6 m long at the central pier based on the pile cap elevation shown on the original design drawings and the bedrock surface elevation encountered during the current investigation) were likely designed to derive most of their axial capacity from bearing on the bedrock surface, but significant force may still be required to break the soil adhesion bond and frictional resistance with the silty clay and glacial till. Consideration may be given to using a vibratory hammer during pulling of the piles to reduce adhesion. However, if a vibratory hammer is used, monitoring of the highway and any adjacent utilities within the highway median must be carried out during pile extraction to minimize the impact on existing infrastructure.

Following removal of the existing piles, the voids should be grouted with lean concrete. If caving of the ground occurs following removal of the piles, an undersized pipe could be driven to facilitate grouting. The grouting pipe should be flushed of any soil debris prior to grouting taking place.

The borehole logs indicate that the lower portions of the embankment fill (below about 86.6 m at the north abutment and 86.1 m at the south abutment) is compact to very dense and contains cobbles. The glacial till that underlies the site also contains cobbles and boulders. All boreholes penetrated the embankment fill and glacial till by augering without the need for diamond drill coring. Steel H-piles reinforced at the tip with a pile point should penetrate theses layers and continue to the surface of the bedrock. However, it is recommended that a contingency item be provided to pre-auger through the embankment fill at both abutments and through the glacial till at the south abutment. The auger size should be chosen to loosen the soil within a diameter smaller than the size of the pile. For example, pre-augering for a 310 x 110 H-pile should be carried out using an auger with a cutting diameter no larger than about 300 mm. The loosened soil is to be left in place following augering.

Due to the potential presence of cobbles and boulders within the till deposit, steel H-piles are preferred over closed-ended steel pipe piles as pipe piles are considered to pose a higher risk of “hanging up” or being deflected away from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip with a pile point (e.g. Titus Standard H Point, or equivalent) to improve seating of the piles on the bedrock and to reduce the potential for damage to the piles during driving in accordance with OPSS 903 *(Construction Specification for Deep Foundations).* If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001.100 Type II *(Foundation, Piles, Steel Tube Pile Driving Shoe)*.

6.5.2 Axial Geotechnical Resistance

For design of HP 310x110 piles driven to the bedrock surface at the estimated tip elevations provided in Section 6.5.1, the factored axial geotechnical resistance at ULS may be taken as 2,000 kN. Serviceability Limit States (SLS) resistances do not apply to piles founded on the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. If the piles meet practical refusal in the dense to very dense glacial till, the factored axial geotechnical resistance at ULS may be taken as 1,600 kN. The axial geotechnical resistance at Serviceability Limit States (SLS, for less than 25 mm of settlement) for such piles may be taken as 1,300 kN. Similar axial resistances may be used in the design of closed-end, concrete-filled, 324 mm diameter steel pipe piles having a minimum wall thickness of 9.5 mm.

Pile installation should be in accordance with OPSS 903 *(Construction Specification for Deep Foundations).* The drawings should incorporate the appropriate note stating that the piles should be equipped with bearing points and should be driven to bedrock. For piles driven to refusal on bedrock, and as described in OPSS 903, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and to then gradually increase the energy over a series of blows to seat the pile.

The placement of additional embankment fill at the new abutments and immediate approach embankments will raise the effective stress level in the silty clay that underlies the site, and may produce settlement that could generate downdrag loads on the new abutment piles. These downdrag loads (i.e., negative skin friction) should be considered in design. No significant raise in the grade of Highway 401 is expected and, therefore, downdrag forces are not anticipated on pile foundations at the new central pier.

The downdrag loads could vary depending on the magnitude of the grade raise and on the sequence of construction. However, assuming an underside of the abutment stem of about Elevation 85.2 m that the upper portion of the piles are cased in a sand-filled, corrugated steel pipe (CSP) to about Elevation 82.2 m, the unfactored downdrag load acting on a single HP 310x110 pile over the maximum length of pile is estimated to be about 330 kN. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *Canadian Highway Bridge Design Code (CHBDC).*

6.5.3 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. Alternatively, the resistance to lateral loading can be derived from the soil in front of the piles, and it may be assumed that this resistance will be nearly the same for vertical and inclined piles as indicated in Section C6.8.7.2 of the Commentary to the CHBDC.

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).

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: su is the undrained shear strength of the soil (kPa); and,

B is the pile diameter/width (m).

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

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

Location	Elevation (m)	Soil Type	n_h (MN/m^3)	s_u (kPa)
North Abutment	81.9 – PCL[1]	Compact to Very Dense Gravelly, Silty Sand (Fill)	6 to 15	-
	80.5 – 81.9	Compact Gravelly, Silty Sand	3 to 6	-
	79.8 – 80.5	Stiff Silty Clay	-	75 kPa
	76.4 – 79.8	Compact to Dense Silty Sand (Till)	4 to 11	-
	76.4	Bedrock	-	-
Central Pier	79.6 – PCL[1]	Stiff Silty Clay	-	75 kPa
	78.7 – 79.6	Compact Sand and Gravel (Till)	3 to 6	-
	78.0 –78.7	Loose Sand and Gravel (Till)	1 to 3	-
	76.5 – 78.0	Compact to Dense Sand and Gravel (Till)	4 to 11	-
	76.5	Bedrock	-	-
South Abutment	81.9 – PCL[1]	Compact to Very Dense Gravelly, Silty Sand (Fill)	6 to 15	-
	80.7 – 81.9	Compact Sand and Gravel	3 to 6	-
	80.0 – 80.7	Stiff Silty Clay	-	75 kPa
	76.0 – 80.0	Dense to Very Dense Gravelly, Silty Sand (Till)	8 to 11	-
	74.9 – 76.0	Compact Silty Sand (Till)	3 to 6	
	74.9	Bedrock	-	-

Note: [1] PCL = Pile Cap Level

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor as follows:

Pile Spacing in Direction of Loading (d = Pile Diameter)	Reduction Factor
8d	1.0
6d	0.7
4d	0.4
3d	0.25

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

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

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

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

6.6 Caisson Foundations

As an alternative to pile foundations, support of the abutments or central pier may be provided by caisson (drilled pier) foundations. Due to the relatively high water table, the cohesionless nature of the till that overlies the bedrock, and the difficulty in socketing a liner into the strong to very strong bedrock, it may not be feasible to completely dewater and clean the base of the caisson and, as such, full end-bearing support may not be developed. As such, the axial geotechnical resistance for rock socketed caissons should be based primarily on the side-wall (shaft) resistance of the rock socket rather than end-bearing.

The use of a liner or casing will be required in order to advance the caissons through the overburden with minimal loss of ground. The casing should be extended so that it is at least nominally socketed into the bedrock.

Casing installation through the glacial till containing cobbles and boulders may be difficult. Churn drilling and possibly rock coring techniques will be required to advance the caissons through the glacial till. In addition, the bedrock at this site is strong to very strong, and the caisson sockets will likely have to be advanced by rock coring (possibly supplemented with a down-hole hammer) and/or chisel drilling.

If caisson caps are to be included as part of the design, they should be constructed at a minimum depth of 1.7 m for frost protection purposes, per OPSD 3090.101 *(Foundation Frost Penetration Depths for Southern Ontario).*

6.6.1 Axial Geotechnical Resistance

Due to the relatively high water table and the difficulty in socketing liners into the very strong bedrock, it may not be feasible to dewater and clean the base of the caisson and, as such, full end-bearing support may not be developed. The axial geotechnical resistance for rock socketed caissons is therefore recommended to be based primarily on the side-wall (shaft) resistance of the rock socket rather than end-bearing.

Rock-socketed caissons should be designed based on the side-wall (shaft) resistance of the rock socket and a factored geotechnical resistance at ULS of 1,700 kPa, provided that the caisson socket is within competent bedrock (i.e., RQD greater than 75 percent). This value assumes that the side wall of the socket will be cleaned of any cuttings or smeared material.

To provide full fixity, the caissons should be provided with a minimum socket length equal to 2 times the caisson diameter. The structural engineer should check that the shear strength of the concrete is adequate to support these loads.

For a 0.9 m diameter caisson socketted 2 m in to the competent bedrock, this would equate to a factored axial geotechnical resistance at ULS of about 9,600 kN. SLS resistances do not apply to caissons founded within the limestone bedrock, because the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

As discussed for driven steel piles, the placement of additional embankment fill at the new abutments and immediate approach embankments will raise the effective stress level in the silty clay that underlies the site, and may produce settlement that could generate downdrag loads on caissons supporting the new abutments. No significant raise in the grade of Highway 401 is expected and, therefore, downdrag forces are not anticipated if caisson foundations are adopted at the new central pier.

The downdrag loads could vary depending on the magnitude of the grade raise and on the sequence of construction. However, for initial assessment purposes, assuming an underside of the abutment stem of about Elevation 85.2 m, the unfactored downdrag load acting on a 0.9 m diameter caisson is estimated to be about 1600 kN. 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 *Canadian Highway Bridge Design Code (CHBDC).*

6.6.2 Resistance to Lateral Loads

The resistance to lateral loading developed by the soil in front of the caissons, and the reductions due to group effects, may be determined as outlined in Section 6.5.3.

6.7 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 bearings at 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.

An integral abutment arrangement is considered feasible at this site since the flexible pile-supported abutment foundations discussed in Section 6.5 meet MTO's foundation criteria for integral abutments, provided that the pile caps are designed at an elevation high enough to satisfy the minimum pile length requirements of 5 m.

6.8 Seismic Considerations

The site is located near Cornwall, Ontario and according to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio, A, applicable to this site is 0.2. The corresponding acceleration related seismic zone, Z_a, is 4.

The soils at this site consist of very stiff silty clay and compact to very dense glacial till consisting of silty sand with gravel, cobbles, and boulders below the water table. At this site, these soils are considered to have a low susceptibility to liquefaction.

In accordance with Section 4.4.6 of the 2006 CHBDC, the Site Coefficient, S, for this site may be taken as 1 consistent with Soil Profile Type I.

6.9 Lateral Earth Pressures for Design

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

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

- Select free-draining granular fill meeting the specifications of OPSS.PROV 1010 Granular A or Granular B Type II should be used as backfill behind the walls. This fill should be compacted in accordance with OPSS.PROV 501 (*Construction Specification for Compacting*).

- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost tapers should be in accordance with OPSD 3101.150 (*Walls, Abutment, Backfill, Minimum Granular*), 3190.101 (*Walls, Retaining and Abutment, Wall Drain*), and 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*).

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. 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.

- The granular fill may be placed either in a zone with width equal to at least 1.7 m behind the back of the abutment stem (Case (a) on Figure C6.20 of the *Commentary* to the CHBDC) or within the wedge-shaped zone defined by a line drawn at 1.5H:1V extending up and back from the rear face of the footing or pile cap (Case (b) on Figure C6.20 of the *Commentary* to the CHBDC).

6.9.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not 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, new lateral earth pressures will need to be calculated.

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

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

- For 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	0.27	0.27
At rest, K_o	0.43	0.43
Passive, K_P	3.7	3.7

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

6.9.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must be taken into account in the design in accordance with Section 4.6 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 above, plus the earthquake-induced dynamic earth pressure. The site-specific zonal acceleration ratio (A) for the site is 0.2. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.2.

- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its *Commentary*, for structures which do not allow lateral yielding, the horizontal seismic coefficient (k_h) used in the calculation of the seismic active pressure coefficient is taken as 1.5 times the zonal acceleration ratio (i.e., k_h = 0.3). For structures which allow lateral yielding, (k_h) is taken as 0.5 times the zonal acceleration ratio (i.e., k_h = 0.1).

- 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}

Material	Case (a)	Case (b)	
	Existing Fill	Granular A	Granular B Type II
Yielding wall	0.39	0.30	0.30
Non-yielding wall	0.62	0.50	0.50

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A mm, where A is the design zonal acceleration ratio of 0.2. This corresponds to displacements of up to approximately 50 mm at this site.

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

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

Where:

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

6.10 Approach Embankments

It is understood that the overall grade of Avonmore Road will be raised up to about 0.9 m. In general, the existing width and alignment of Avonmore Road are to be maintained and, therefore, the existing embankments will require nominal widening to accommodate the proposed grade raise.

Based on the results from the boreholes drilled through the existing Avonmore Road embankments, the road structure is generally underlain by embankment fill consisting of sand and gravel, overlying silty sand fill (containing varying amounts of gravel, cobbles, and boulders), gravelly silty sand, silty clay to clay, glacial till and limestone bedrock.

6.10.1 General Embankment Construction

It is recommended that all topsoil/organic material or existing loose surficial fill present within the widening footprint be stripped prior to placement of embankment fill. Due to the nominal thickness and relatively high stiffness of the silty clay to clay deposit, as well as the significant existing embankment height and duration of time since the original construction, removal of silty clay to clay from beneath the footprint of the existing embankments is not required.

The new embankment fill associated with the grade raise and widening for the bridge replacement should be placed and compacted in accordance with OPSS.PROV 206 (Construction Specifications for Grading) and OPSS.PROV 501 (Construction Specification for Compacting). Benching of the existing Avonmore Road 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 (OPSS 802 – Construction Specification for Topsoil) and seeding (OPSS.PROV 804 – Construction Specification for Seed and Cover) or pegged sod (OPSS 803 – Construction Specification for Sodding) 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).

6.10.2 Global Stability

A slope stability assessment of the embankments has been carried out considering the proposed grade raise of up to about 0.9 m using the commercially available slope stability analysis software package SlopeW™ by GeoSlope International Ltd., to verify that a minimum factor of safety of 1.3 is achieved under static conditions and 1.1 under design seismic conditions. These minimum factors of safety are considered appropriate for the proposed bridge approach embankments, considering the design requirements and the available field and laboratory testing data.

The stability analyses were carried out considering that embankment side slopes will be maintained at inclinations no steeper than 2H:1V. The soil stratigraphy used in the analyses was selected to represent soil conditions with the greatest thickness of overburden soil that may be expected at the site and was based on the information available.

Provided that the approach embankment side slopes are maintained at inclinations no steeper than 2H:1V, and the existing embankment side slopes are benched in accordance with OPSD 208.010 (Benching of Earth Slopes), to “key in” any new fill materials placed on the slopes to accommodate the overall grade, the embankments should have an adequate minimum factor of safety of at least 1.3 under static conditions and 1.1 under design seismic conditions. If side-slopes steeper than 2H:1V are to be considered or the Avonmore Road grade is to be increased more than 0.9 m above the existing grades, the embankment side-slope stability will have to be re-assessed.

6.10.3 Settlement

Settlement of the existing embankments has likely occurred over time since the original bridge construction. The additional loading imposed by the proposed approximately 0.9 m grade raise would result in some additional elastic settlement of the subgrade soils present beneath the embankments. For an increase in grade of 0.9 m, the elastic settlement of the silty clay to clay within the footprint of the existing embankments, together with the elastic compression of the underlying glacial till deposit, is estimated to be less than about 25 mm. As described above, any surficial organic matter encountered within areas of the embankments that are to be widened to accommodate the grade raise should be removed prior to placement of any fill.

Additional settlement of the embankments will occur as a result of compression of the new grade fill and the existing embankment fill. The magnitude of compression of the new fill may range from 0.5 to 1 percent of its thickness, assuming approximately 95 percent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. Some nominal compression of the existing fill (less than 0.5 percent of its thickness) is expected to occur under the increased loading. Provided that granular fill is used to raise the grade, settlement of the new fill is expected to occur essentially during embankment construction. Similarly, settlement of the existing silty sand embankment fill will be elastic in nature and should occur essentially immediately following placement of the new fill.

6.11 Construction Considerations

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

6.11.1 Excavation and Temporary Protection Systems

At the central pier, excavation for a spread footing is expected to extend through the grade fill and silty clay to clay and into the typically compact to very dense silty sand till. This excavation would extend below the ground water table up to about 3 m below the existing Highway 401 median grade. If deep foundations are adopted for support of the pier and/or abutments, the excavations for pile caps could be maintained at a higher elevation within the approach embankments.

Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill above the water table would be classified as Type 3 soil, based on the OHSA. According to OHSA excavations that extend to, or into, Type 3 soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V). The silty clay to clay and glacial till below the water table would be classified as Type 3 soil. It is anticipated that excavations in the stiff silty clay to clay and compact to dense glacial till could be maintained at 1H:1V, but may need to be flattened if localized layers of fine sand or silt, or soft silty clay are encountered. However, with appropriate groundwater control to lower the water level below the base of the excavation, it is anticipated that temporary excavation slopes through the glacial till can be maintained at 1H:1V.

If the above open-cut excavation side slopes cannot be accommodated, then a temporary protection system (i.e., temporary excavation shoring) will be required. A shoring system could also limit the Infiltration of groundwater into the excavation depending on the type and quality of installation of the shoring. Where shoring is required, the protection system should be designed and constructed in accordance with OPSS.PROV 539 (*Construction Specifications for Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539.

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

- It is considered that either a soldier pile and lagging system or an interlocking sheetpile system would be feasible at this site. The use of an interlocking sheet pile system has an advantage over soldier pile and lagging in that it would aid in groundwater control; however, the presence of cobbles and/or boulders in the fill or glacial till may impact the depth that sheet piling can be driven and the effectiveness of the system. Therefore, the preferred method of shoring would be soldier piles and lagging, with measures to control seepage and/or mitigate the loss of soil particles through the lagging boards.
- The soldier pile and lagging or sheet piling would have to be socketed to sufficient depth to provide the necessary passive resistance for the retained soil height. Lateral support to the sheetpiles or soldier piles could be provided in the form of walers, tie-backs and/or internal struts/braces.

6.11.2 Groundwater and Surface Water Control

Based on readings taken at the monitoring well installed in the northern embankment and groundwater conditions observed in the boreholes immediately following drilling, the groundwater level is expected to be about 7 m below the existing Avonmore Road grade at the abutment locations and within about 1 m of the existing Highway 401 grade at the central pier location.

The excavations required for construction of shallow foundations at the central pier are expected to extend up to about 3 m below the groundwater level. If deep foundations are used at the central pier, the excavations required for pile/caisson caps are anticipated to extend up to about 2 m below the groundwater level. Dewatering is recommended to lower the groundwater level to approximately 0.3 m below the footing founding level, to minimize disturbance of the subgrade. The water-bearing till at this site is relatively fine-grained (silty), and therefore will have a low to moderate permeability.

The groundwater level is expected to be encountered during excavations for shallow foundations or pile cap construction at the central pier, but may vary at the time of construction. It is considered that less than 50,000 litres per day of water will require handling during excavation for construction of shallow foundations at the central pier, or pile/caisson caps at the central pier. Therefore, a Permit-To-Take-Water (PTTW) should not be required for construction. However, if excavations are to extend to greater depths, the dewatering rate may exceed 50,000 litres per day, and therefore, a Permit to Take Water (PTTW) would be required for this site in this case.

Surface water should be directed away from the excavation areas, to prevent ponding of water that could result in disturbance and weakening of the subgrade.

6.11.3 Subgrade Protection

If the central pier is to be founded on shallow spread footings, all grade fill, topsoil, organics, and soft or loose soils should be removed from below the proposed founding elevations and wasted or reused as landscaping fill, as required. Subgrade preparation should be performed and monitored in accordance with OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*).

The glacial till that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. A Non-Standard Special Provision has been provided in Appendix C to address this requirement.

6.11.4 Vibration Monitoring During Pile Installation

If the existing underpass structure is not completely removed prior to commencement of pile driving, vibration monitoring is recommended during pile installation to assist in maintaining vibration levels within tolerable ranges for the existing portions of the bridge in close proximity to Highway 401. A Non-Standard Special Provision has been provided in Appendix C to address this requirement.

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

6.11.5 Ground/Groundwater Control and Obstructions for Deep Foundation Installation

Where caissons are adopted, or if pre-augering is required for steel pile installation, the use of temporary or permanent liners will be required to minimize loss of ground through the water-bearing cohesionless till deposit.

The presence of cobbles and boulders in the glacial till could affect the installation of deep foundations or protection system elements. If caissons are to be used, appropriate drilling techniques will be required to advance the caissons through the glacial till. If driven H-piles are used, pre-augering of the pile locations may be required through the embankment fill and glacial till.

A Non-Standard Special Provision is provided in Appendix C, for inclusion in the Contract Documents to alert the Contractor to these conditions.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Matt Kennedy, P.Eng., with technical input from Mr. Kevin Nelson, P.Eng. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
M. J. KENNEDY
100188946
Jan 13, 2016
PROVINCE OF ONTARIO

Matt Kennedy, P.Eng.
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
Jan 13, 2016
PROVINCE OF ONTARIO

Fin Heffernan, P.Eng.
Designated MTO Foundations Contact

MJK/KN/FJH/ob

\\golder.gds\gal\ottawa\active\2012\1121 - geotechnical\12-1121-0099 mrc 22 structures eastern region\foundations\6 - reports\package 7\31-177 avonmore road\12-1121-0099-1730 rpt-001 final avonmore road site 31-177 january 2016.docx

Table 1 – Comparison of Foundation Alternatives

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-piles driven to bedrock	▪ Feasible for support of bridge replacement ▪ Preferred option from a foundations perspective	▪ Abutment pile caps could be maintained higher than for spread footings, reducing depth of excavation and temporary excavation support requirements ▪ Higher geotechnical resistances and negligible settlement ▪ Less potential for interference with existing piles (vs. caissons) ▪ Preferred foundation option for integral abutment construction	▪ Potential for encountering obstructions (cobbles and/or boulders) during pile driving that could result in some piles “hanging up” in the glacial till deposit and lower geotechnical resistances and the potential for installation of additional piles. Pre-augering through the embankment fill and till could reduce this risk ▪ Temporary protection systems may be required at the central pier ▪ Some groundwater control would still be required at the central pier	▪ Moderate cost	▪ Risk of driven H-piles “hanging up” in glacial till ▪ Contingency should be included for pre-augering through embankment fill and glacial till
Steel pipe (tube) piles, driven to found on bedrock	▪ Feasible for support of bridge replacement	▪ Abutment pile caps could be maintained higher than footings, reducing depth of excavation and temporary protection system ▪ Higher geotechnical resistances and negligible settlement	▪ Greater risk of piles “hanging up” than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving; this could result in , lower geotechnical resistances, and greater potential for interference with existing piles ▪ Temporary protection systems may be required at central pier ▪ Some groundwater control would still be required at the central pier	▪ Moderate cost	▪ Moderate risk of pipe piles “hanging up” in glacial till ▪ Contingency should be included for pre-augering through embankment fill and glacial till

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Caissons founded on bedrock	▪ Feasible	▪ Could eliminate the need for deep foundation cap at the central pier and allow for structural continuity between caissons and piers ▪ Construction from existing grade would reduce excavation and groundwater control requirements (reduced impact on Highway 401)	▪ Temporary or permanent liners required to control ground and groundwater in water-bearing till deposit ▪ Installation difficulties should be expected advancing casing due to obstructions within the till deposits. ▪ Rock coring, churn drilling or chisel drilling required to form rock sockets in strong to very strong bedrock ▪ Conflict with existing piles at abutments and central pier likely, requiring removal of existing piles	▪ Construction of deep caissons more expensive than alternative foundation options	▪ Some risk of difficulty in removing existing abutment/pier piles to avoid conflict with new caissons
Spread/strip footings on compact to dense silty sand glacial till	▪ Feasible at central pier but not recommended if within same footprint as existing pier foundation ▪ Not practical at abutments due to requirement for significant excavation through existing embankments	▪ Lower cost for construction compared to deep foundations	▪ Significant excavations to depths of greater than 9 m through existing embankments would be required at the abutment locations ▪ Increased settlement in comparison to pile foundations ▪ Excavation at the central pier location, between the travelled lanes of Highway 401, will require temporary protection systems ▪ Removal of the existing piles at the central pier likely required ▪ Groundwater control requirements during construction ▪ Lower geotechnical resistances as compared with deep foundations ▪ Precludes use of integral abutments; potentially greater maintenance required	▪ Less expensive than deep foundations although bridge maintenance costs may be higher due to non-integral abutment configuration ▪ Additional costs required for significant excavation at abutments may reduce any cost differential	▪ Risk of instability of existing embankment slopes without appropriate temporary protection measures during excavation at abutments to significant depth ▪ Risk of differential settlement between pier and pile supported abutments.

MINISTRY OF TRANSPORTATION, ONTARIO

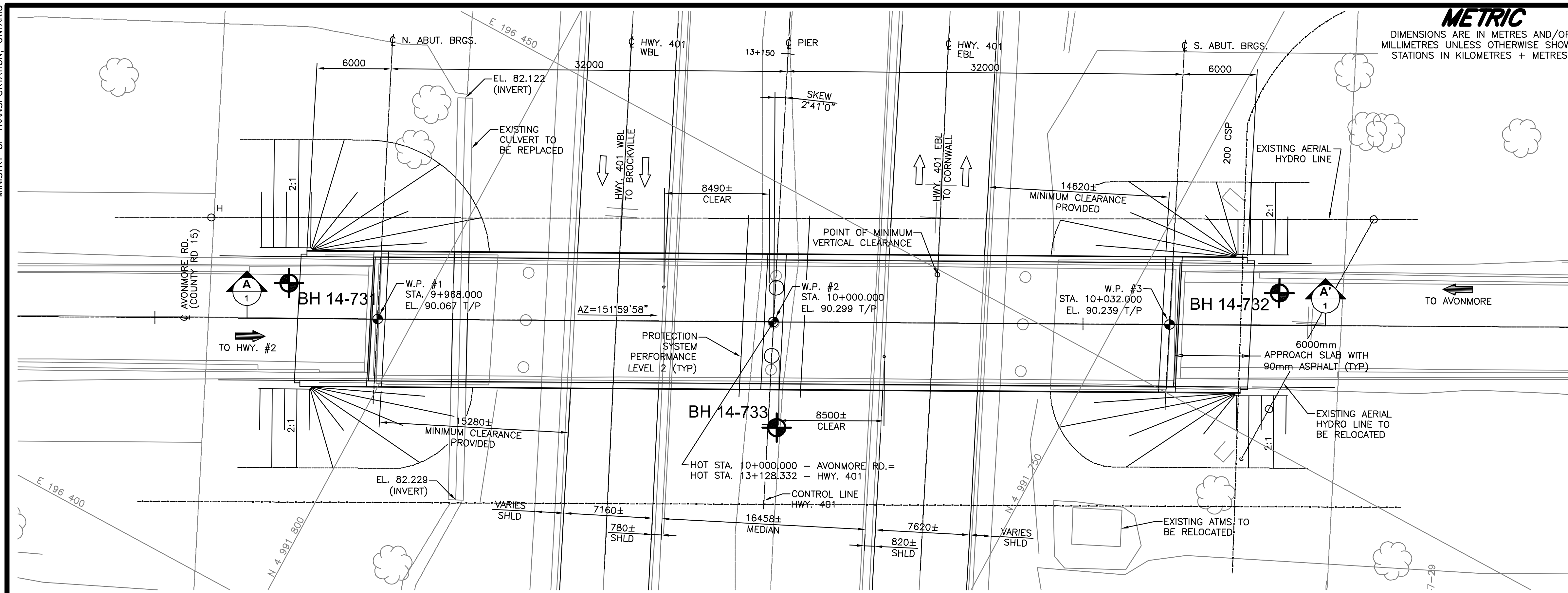
CONT No.
WP No. 4382-01-01
GWP No. 4064-12-00

HIGHWAY 401 UNDERPASS AT AVONMORE ROAD BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

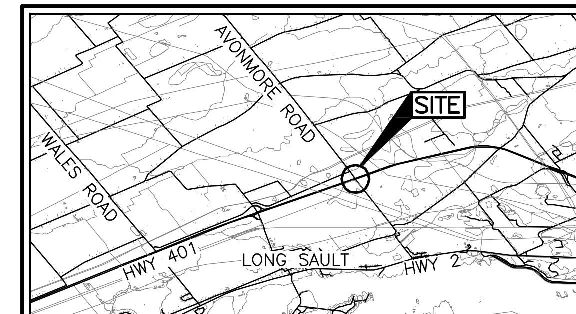
Golder Associates Ltd.
OTTAWA ONTARIO, CANADA

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



PLAN

SCALE 4 0 4 8 m



KEY PLAN

SCALE 1 0 2 2 KM

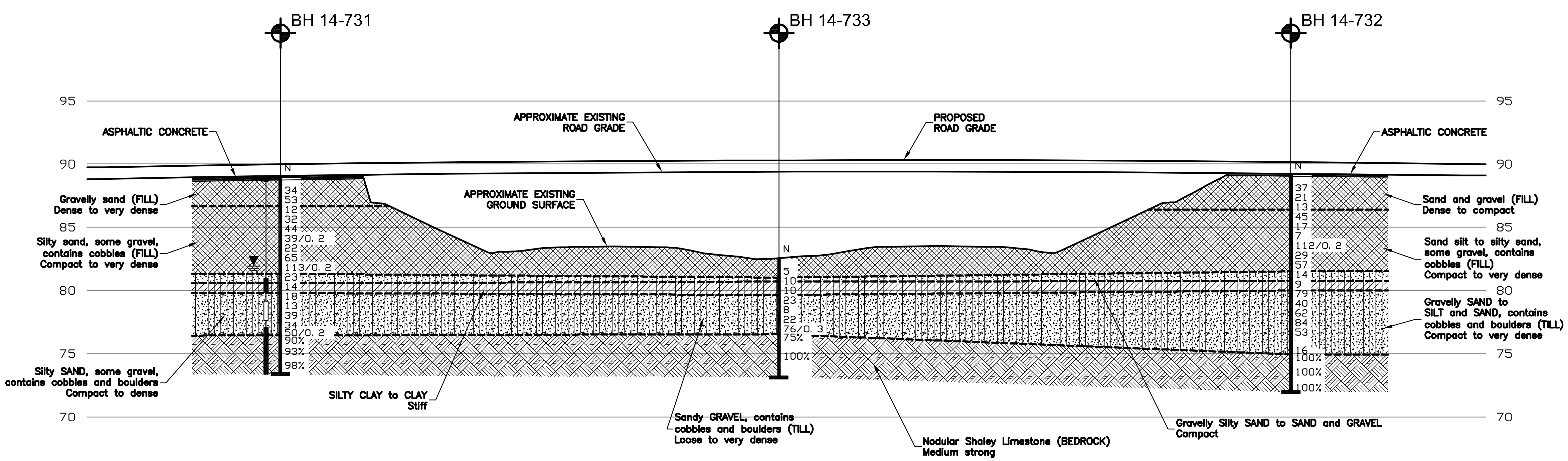
LEGEND

⌖	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Total Core Recovery (REC)
▼	WL in piezometer
▮	Seal
▯	Piezometer

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
14–731	88.9	4991810.3	196424.1
14–732	89.1	4991739.5	196461.4
14–733	82.5	4991770.1	196432.6

LICENSED PROFESSIONAL ENGINEER M. J. KENNEDY 100188946 Jan 13, 2016 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER F. J. HEFFERNAN Jan 13, 2016 PROVINCE OF ONTARIO



PROFILE ALONG AVONMORE ROAD

A-A'

HORIZ. SCALE 4 0 4 8 m
4 0 4 8 m VERT. SCALE

NOTES

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

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

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by MMM Group Limited, drawing file no. 3412039-007-001_GA-AVONMORE.dwg, received Nov. 6, 2015.

NO.	DATE	BY	REVISION

Geocres No. 31G-253			
HWY. 401	PROJECT NO. 12-1121-0099		DIST. Eastern
SUBM'D. MJK	CHKD. MJK	DATE: 01/12/2016	SITE: 31-177
DRAWN: JM	CHKD. MJK	APPD. FJH	DWG. 1

PLOT DATE: January 13, 2016

APPENDIX A

Borehole and Drillhole Records

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO or DP	Seamless open-ended, driven or pushed tube samplers
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split spoon sampler
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample
DT	Dual tube sample
DD	Diamond drilling

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split spoon sampler for a distance of 300 mm (12 in.).

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive an uncased 50 mm (2 in.) diameter, 60^0 cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

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

Cone Penetration Test (CPT):

An electronic cone penetrometer with a 60^0 conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	**N** Blows/300 mm Or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	**C_u or S_u** kPa	Psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	Over 200	Over 4,000

IV. SOIL TESTS

w	Water content
w_p or PL	Plastic limited
w_1 or LL	Liquid limit
C	Consolidaiton (oedometer) test
CHEM	Chemical analysis (refer to text)
CID	Consolidated isotropically drained triaxial test[1]
CIU	Consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	Relative density
DS	Direct shear test
Gs	Specific gravity
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	Organic content test
SO_4	Concentration of water-soluble sulphates
UC	Unconfined compression test
UU	Unconsolidated undrained triaxial test
V	Field vane test (LV-laboratory vane test)
γ	Unit weight

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

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$ x or log x	logarithm of x to base 10
g	acceleration due to gravity
t	time
FOS	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma'$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial vertical effective overburden stress
$\sigma_1 \sigma_2 \sigma_3$	principal stresses (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3) / 3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_1 or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity Index = $(w_1 - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_c	consistency index = $(w_1 - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (overconsolidated range)
C_s	swelling index
C_α	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	overconsolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

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

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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of rock material weathering
Faintly Weathered: weathering limited to the surface of major discontinuities.
Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.
Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable
Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.
Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very Thickly Bedded	> 2 m
Thickly Bedded	0.6 m to 2m
Medium Bedded	0.2 m to 0.6 m
Thinly Bedded	60 mm to 0.2 m
Very Thinly Bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly Laminated	< 6 mm

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 – 60 mm
Medium Grained	60 microns – 2mm
Fine Grained	2 – 60 microns
Very Fine Grained	< 2 microns

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

CORE CONDITION

Total Core Recovery
The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)
The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)
The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index
A count of the number of discontinuities (physical separations) in the rock core, including naturally occurring fractures but not including mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90^0 angle is horizontal.

Description and Notes
An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

BD -	Bedding	PY -	Pyrite
FO -	Foliation/Schistosity	Ca -	Calcite
CL -	Clean	PO -	Polished
SH -	Shear Plane/Zone	K -	Slickensided
VN -	Vein	SM -	Smooth
FLT -	Fault	RO -	Ridged/Rough
CO -	Contact	ST -	Stepped
JN -	Joint	PL -	Planar
FR -	Fracture	IR -	Irregular
MB -	Mechanical Break	UN -	Undulating
BR -	Broken Rock	CU -	Curved
BL -	Blast Induced	TCA -	To Core Axis
II -	Parallel To	STR -	Stress Induced
OR -	Orthogonal		

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 14-731

SHEET 1 OF 3 **METRIC**

PROJECT 12-1121-0099-1730
G.W.P. 4382-01-01
DIST Eastern HWY 401
DATUM Geodetic

LOCATION N 4991810.3 ;E 196424.1
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core
DATE October 24-27, 2014

ORIGINATED BY DWM
COMPILED BY JM
CHECKED BY MJK

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
88.9	GROUND SURFACE							
0.0	ASPHALTIC CONCRETE							
88.7 0.2	Gravelly sand, trace silt (FILL) Dense Brown Dry	1	SS	34				
87.4 1.5	Gravelly sand, some silt (FILL) Very dense Grey-brown Dry	2	SS	53				
86.6 2.3	Silty sand, some gravel, contains cobbles (FILL) Compact to dense Grey-brown Moist	3	SS	12				
		4	SS	32	19	54	23	4
		5	SS	44				
84.3 4.6	Gravelly silty sand, trace clay, contains cobbles and asphalt pieces (FILL) Compact to very dense Brown Moist	6	SS	39/0.2				
		7	SS	22	26	42	23	9
82.8 6.1	Silty sand, some gravel, trace organic matter, contains cobbles and asphalt pieces (FILL) Very dense Grey-brown to black Moist to wet	8	SS	65				
		9	SS	113/0.2				
81.3 7.6	Gravelly Silty SAND, contains shells Compact Grey Wet	10	SS	23				
80.5 8.4	SILTY CLAY to CLAY, trace sand and gravel Stiff Grey-brown Wet	11	SS	14				
79.8 9.1	Silty SAND, some gravel, contains cobbles and boulders (TILL) Compact to dense Grey Wet	12	SS	18				

Elevation scale: 88, 87, 86, 85, 84, 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

Continued Next Page

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

RECORD OF BOREHOLE No 14-731

SHEET 2 OF 3

METRIC

PROJECT 12-1121-0099-1730

G.W.P. 4382-01-01

LOCATION N 4991810.3 ;E 196424.1

ORIGINATED BY DWM

DIST Eastern HWY 401

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

COMPILED BY JM

DATUM Geodetic

DATE October 24-27, 2014

CHECKED BY MJK

--- 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 SAND, some gravel, contains cobbles and boulders (TILL) Compact to dense Grey Wet	13	SS	13				
		14	SS	39	78	o		14 52 26 8
		15	SS	34	77			
76.4		16	SS	50/0.2				
12.5	Limestone (BEDROCK) Bedrock cored from depths of 12.5 m to 15.6 m For bedrock coring details refer to Record of Drillhole 14-731	1	RC	REC 96%	76		UCS = 97.7 MPa	RQD = 90%
		2	RC	REC 100%	75			RQD = 93%
73.3		3	RC	REC 100%	74			RQD = 98%
15.6	END OF BOREHOLE NOTES: 1. Water level in well screen at a depth of 7.0 m below ground surface (Elev. 81.9 m), measured on November 26, 2014.							

Column headings: SOIL PROFILE; STRAT PLOT; 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.

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

RECORD OF DRILLHOLE: 14-731

LOCATION: N 4991810.3 ;E 196424.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 24-27, 2014

DRILL RIG: CME 750

DRILLING CONTRACTOR: CCC 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	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	FEATURES
		Continued from Record of Borehole 14-731	76.43				
13–15	Rotary Drill, NQ Core	Limestone (BEDROCK), with thin black shale interbeds and partings Fresh Thinly to medium bedded Grey Fine to medium grained Non-porous Strong to very strong - Broken core from 12.5 m to 12.6 m	12.51	1	0	JN, BD, BD, BD	UCS = 97.7 MPa
				2	0	BD	
			73.38	3	0	BD, BD	
		END OF DRILLHOLE	15.56				

Depth scale: 13, 14, 15, 16, 17, 18, 19, 20, 21, 22

GTA-RCK 031 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0099 MRC 22 STRUCTURES EASTERN REGION\SPATIAL IM\GINT\1211210099.GPJ GAL-MISS.GDT 01/12/16 JM

RECORD OF BOREHOLE No 14-732

SHEET 1 OF 3 **METRIC**

PROJECT 12-1121-0099-1730

G.W.P. 4382-01-01 LOCATION N 4991739.5 ;E 196461.4 ORIGINATED BY DWM

DIST Eastern HWY 401 BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill HQ Core COMPILED BY JM

DATUM Geodetic DATE October 23, 2014 CHECKED BY MJK

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
89.1	GROUND SURFACE					
0.0	ASPHALTIC CONCRETE					
0.1	PORTLAND CEMENT CONCRETE					
88.6 / 0.5	Sand and gravel, trace silt (FILL) Dense to compact Grey-brown Dry	1	SS	37		
		2	SS	21	○	49 41 9 1
		3	SS	13		
86.4 / 2.7	Sand and silt, some gravel (FILL) Brown Moist					
86.1 / 3.1	Gravelly silty sand, contains cobbles (FILL) Dense Grey-brown Moist	4	SS	45		
85.3 / 3.8	Sand and silt, trace to some gravel, contains cobbles and organic matter (FILL) Very dense to loose Brown Moist	5	SS	17		
		6	SS	7		
		7	SS	112/0.2		
		8	SS	29		
		9	SS	57		
81.5 / 7.6	SAND and GRAVEL, some silt, contains white shells Compact Grey Wet	10	SS	14	○	52 35 11 2
80.7 / 8.4	SILTY CLAY to CLAY Stiff Grey Wet	11	SS	9	w_P–w–w_L	
80.0 / 9.1	Gravelly SAND, some silt to silty, contains cobbles (TILL) Very dense to dense Grey Moist to wet	12	SS	79		

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (89, 88, 87, 86, 85, 84, 83, 82, 81, 80); 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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 14-732

SHEET 2 OF 3

METRIC

PROJECT 12-1121-0099-1730

G.W.P. 4382-01-01

LOCATION N 4991739.5 ;E 196461.4

ORIGINATED BY DWM

DIST Eastern HWY 401

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

COMPILED BY JM

DATUM Geodetic

DATE October 23, 2014

CHECKED BY MJK

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Gravelly SAND, some silt to silty, contains cobbles (TILL) Very dense to dense Grey Moist to wet	13	SS	40		79			
		14	SS	62		78	○		30 40 23 7
77.7 11.4	SILT and SAND, some clay, trace gravel, contains cobbles and boulders (TILL) Very dense Grey Moist	15	SS	84		77			
		16	SS	53			○		8 34 41 17
76.0 13.1	Silty SAND, some gravel, occasional sand seams (TILL) Compact Grey Wet					76			
		17	SS	16		75			
74.9 14.2	Shaley Limestone (BEDROCK) Bedrock cored from depths of 14.2 m to 17.2 m For bedrock coring details refer to Record of Drillhole 14-732	1	RC	REC 100%		74		UCS = 185.8 MPa	RQD = 100%
		2	RC	REC 100%		73			RQD = 100%
		3	RC	REC 100%		72			RQD = 100%
71.9 17.2	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

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

RECORD OF DRILLHOLE: 14-732

PROJECT: 12-1121-0099-1730
LOCATION: N 4991739.5 ;E 196461.4
INCLINATION: -90° AZIMUTH: ---
DRILLING DATE: October 23, 2014
DRILL RIG: CME 750
DRILLING CONTRACTOR: CCC Drilling
DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	FEATURES
		Continued from Record of Borehole 14-732		74.95				
	Rotary Drill HQ Core	Shaley Limestone (BEDROCK) Fresh Thinly to medium bedded Grey to black Fine to medium grained Non-porous Very strong		14.19	1	100	BD BD BD	UCS = 185.8 MPa
15								
16					2	100	BD	
17				71.98 17.16	3	100		
		END OF DRILLHOLE						
18								
19								
20								
21								
22								
23								
24								

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: DWM
CHECKED: MJK

Golder Associates

RECORD OF BOREHOLE No 14-733

SHEET 1 OF 2 **METRIC**

PROJECT 12-1121-0099-1730			
G.W.P. 4382-01-01	LOCATION N 4991770.1 ;E 196432.6		ORIGINATED BY DWM
DIST Eastern HWY 401	BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core		COMPILED BY JM
DATUM Geodetic	DATE October 30-31, 2014		CHECKED BY MJK

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
82.5	GROUND SURFACE								
0.0	TOPSOIL								
0.2	Silt and sand, some gravel, contains organic matter (FILL) Loose Brown to black Wet					82			
		1	SS	5					
81.0 / 1.5	Silty SAND, some gravel, contains shells Compact Brown Wet					81			
80.7 / 1.8	SILTY CLAY to CLAY, trace sand, trace gravel Stiff Grey Wet	2	SS	10					
		3	SS	10		80			
79.6 / 2.9	Gravelly SAND, some silt (TILL) Compact Grey Wet								
		4	SS	23		79	○		33 50 14 3
78.7 / 3.8	Sandy GRAVEL, contains silt seams, cobbles and boulders (TILL) Loose to compact Grey Wet	5	SS	8					
		6	SS	22		78	○		56 35 7 2
		7	SS	76/0.3		77			
		~~8~~	~~SS~~	~~50/0.0~~					
76.5 / 6.0	Shaley Limestone (BEDROCK) Bedrock cored from depths of 6.0 m to 9.4 m For bedrock coring details refer to Record of Drillhole 14-733					76		UCS = 171.5 MPa	
		1	RC	REC 97%		75			RQD = 75%
		2	RC	REC 100%		74			RQD = 100%
73.1 / 9.4	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

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

PROJECT: 12-1121-0099-1730

RECORD OF DRILLHOLE: 14-733

LOCATION: N 4991770.1 ;E 196432.6

DRILLING DATE: October 30-31, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 750

DRILLING CONTRACTOR: CCC 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	
		Continued from Record of Borehole 14-733		76.54														
6	Rotary Drill NQ Core	Shaley Limestone (BEDROCK) Fresh Thinly bedded Grey to black Fine to medium grained Non-porous Very strong		6.00	1	100						FR FR						UCS = 171.5 MPa
7		- Broken core from 6.3 m to 6.5 m																
8					2	100												
9				73.11 9.43														
		END OF DRILLHOLE																

Depth scale marks: 10, 11, 12, 13, 14, 15, 16

GTA-RCK 031 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0099 MRC 22 STRUCTURES EASTERN REGION\SPATIAL IM\GINT\1211210099.GPJ GAL-MISS.GDT 01/12/16 JM

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: DWM

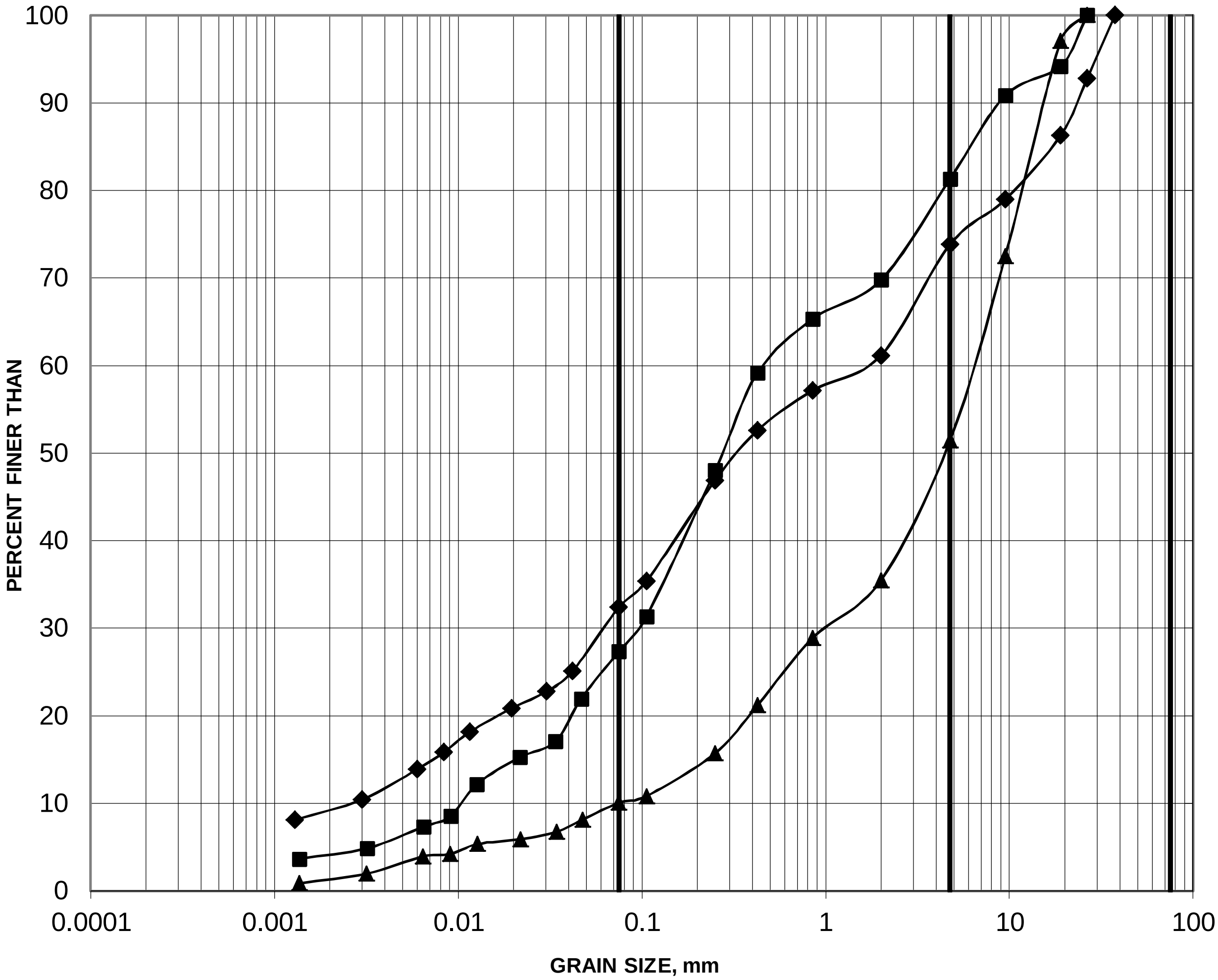
CHECKED: MJK

APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION	FIGURE B1

SILTY, GRAVELLY SAND to SAND and GRAVEL (EMBANKMENT FILL)



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

Borehole	Sample	Depth (m)
14-731 (■)	4	3.05-3.66
14-731 (◆)	7	5.34-5.95
14-732 (▲)	2	1.52-2.13

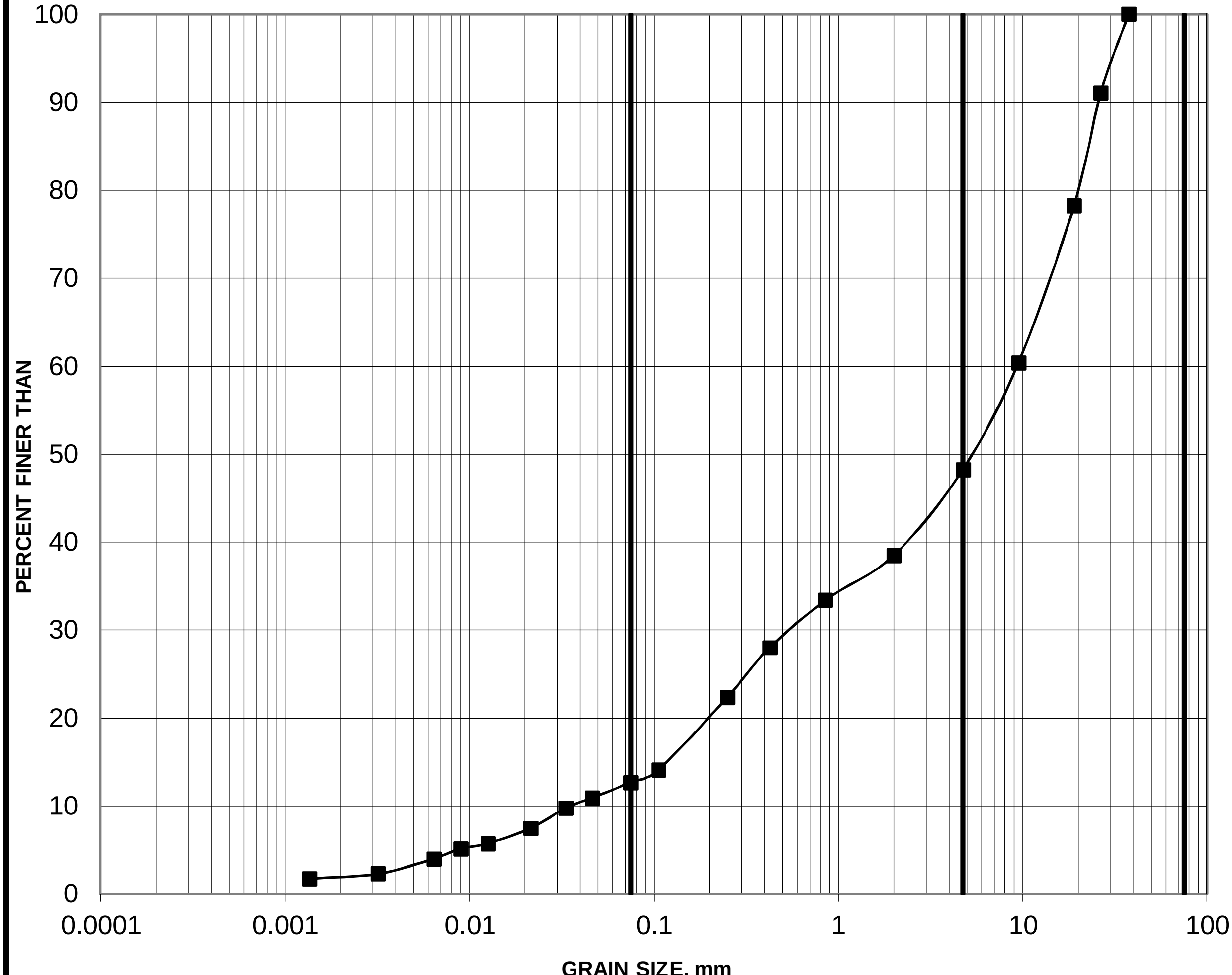
Project: 12-1121-0099/1730

Golder Associates

Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B2

SAND AND GRAVEL



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

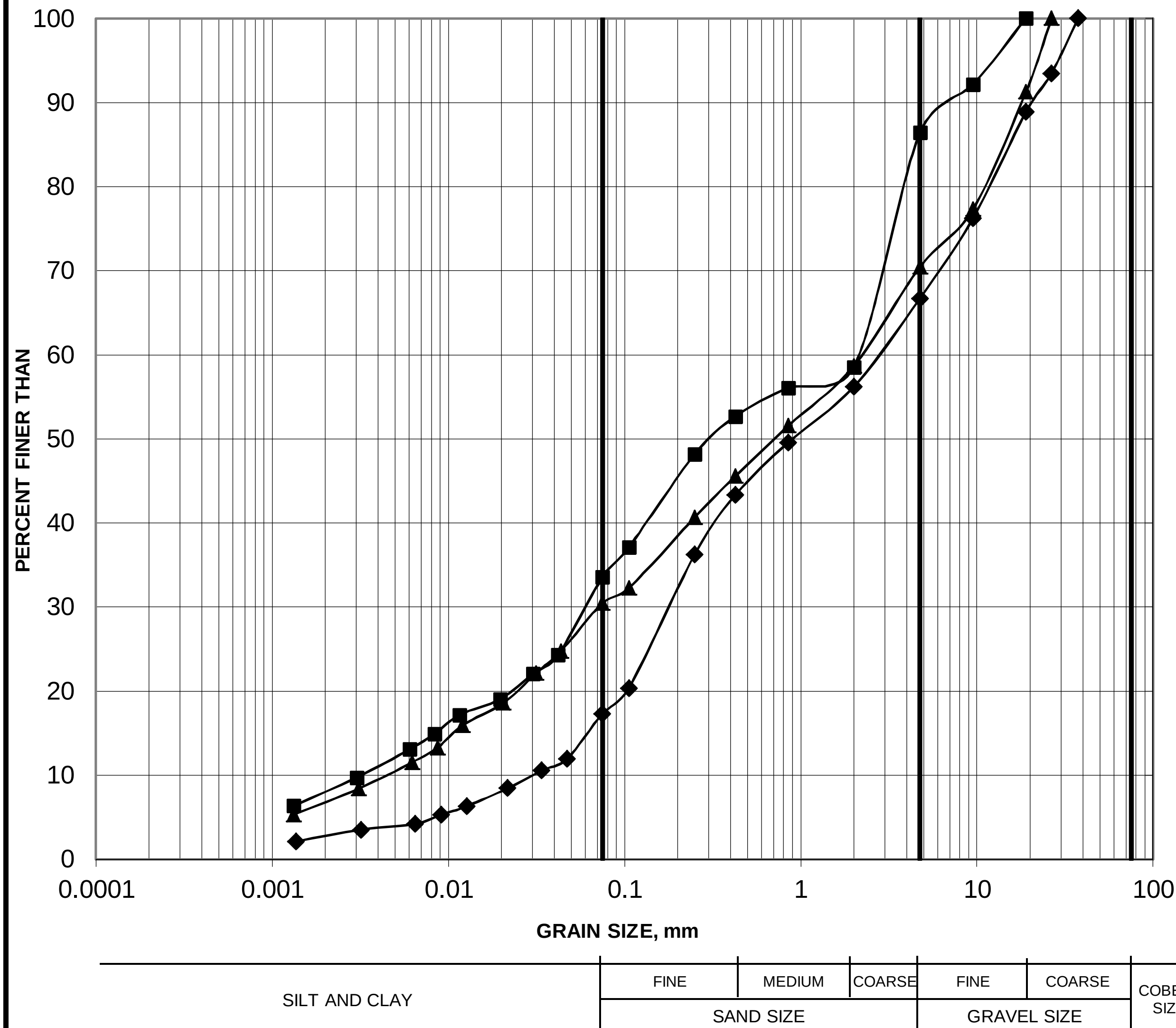
Borehole	Sample	Depth (m)
14-732	10	7.62-8.23

Project: 12-1121-0099/1730 **Golder Associates**

Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B3

SILTY SAND to GRAVELLY SILTY SAND (GLACIAL TILL)



Borehole	Sample	Depth (m)
14-731	14	10.67-11.28
14-732	14	10.67-11.28
14-733	4	3.05-3.66

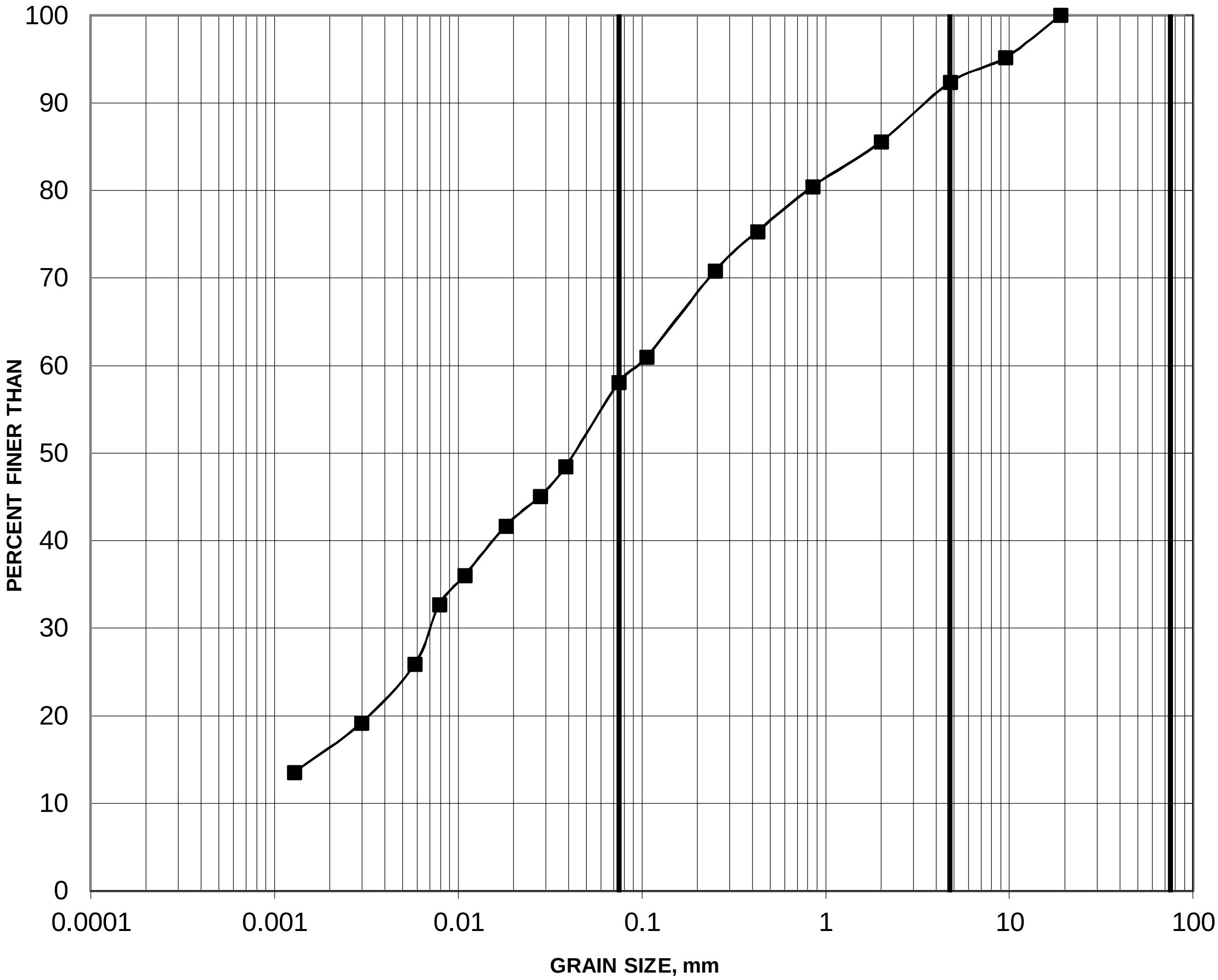
Project: 12-1121-0099/1730 **Golder Associates**

Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION

FIGURE B4

SILT AND SAND (GLACIAL TILL)



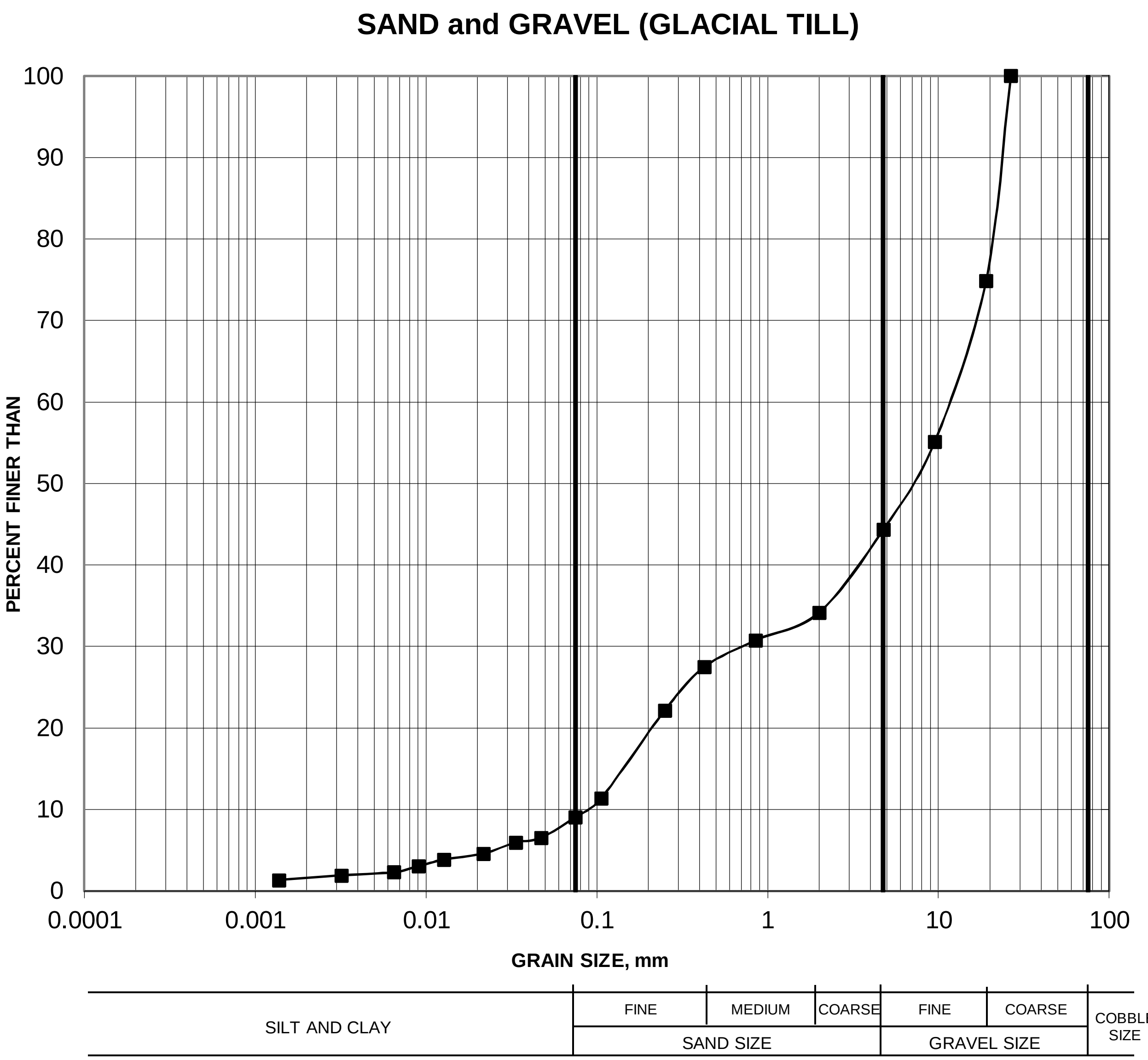
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
14-732	16	12.20-12.80

Project: 12-1121-0099/1730 **Golder Associates**

Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B5



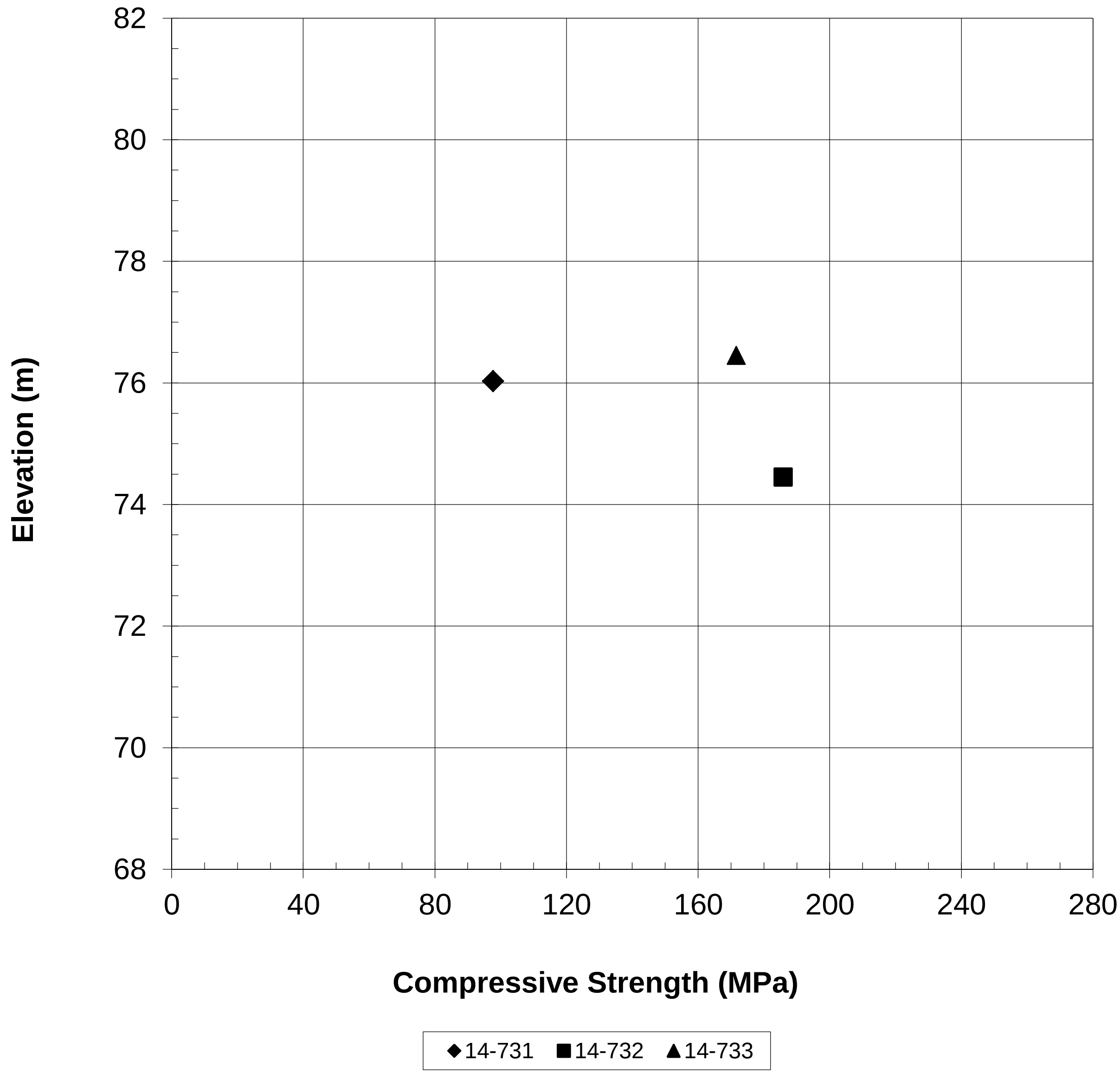
Borehole	Sample	Depth (m)
14-733	6	4.57-5.18

Project: 12-1121-0099/1730 **Golder Associates**

Created by: MI
Checked by: CNM

SUMMARY OF LABORATORY COMPRESSIVE STRENGTH UNCONFINED COMPRESSION TESTS

FIGURE B6



Project: 12-1121-0099-1730

Golder Associates

Created by:	MI
Checked by:	CNM

APPENDIX C

Non-Standard Special Provisions

DEWATERING STRUCTURE EXCAVATIONS – Item No.

Special Provision

Amendment to OPSS 902

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.02 Submission Requirements

Section 902.04.02 is amended by the addition of the following Subsection:

902.4.02.03 Dewatering

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

902.07 CONSTRUCTION

902.07.04 Dewatering Structure Excavation

Section 902.07.04 is amended by the addition of the following:

The Contractor is advised that construction of the new centre pier foundation will require excavation below the groundwater level in the cohesionless till deposit. 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. The Contractor shall reference borehole records as shown elsewhere in the Contract Documents as a guide in determining dewatering requirements.

A continuous dewatering operation shall be provided to facilitate the foundation construction operations at all times. The dewatering system shall be adequate to lower the groundwater level to at least 0.5 m below the founding level for the new centre pier, to allow excavation, subgrade preparation and foundation construction in dry conditions. All components of the dewatering system shall be maintained in an effective, functioning and stable condition during the construction.

The work for dewatering shall be completed in accordance with the environmental and operational constraints specified elsewhere in the Contract Documents.

WORKING SLAB – Item No.

Special Provision

1.0 SCOPE

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

2.0 REFERENCES

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

Ontario Provincial Standard Specifications, Construction

OPSS 902 Excavating and Backfilling – Structures

3.0 DEFINITIONS – Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS – Not Used

5.0 MATERIALS

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

6.0 EQUIPMENT – Not Used

7.0 CONSTRUCTION

7.01 Excavation

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

7.02 Protection of Founding Soil

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

7.03 Dewatering

Dewatering shall be carried out in accordance with OPSS 902.

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT – Not Used

10.0 BASIS OF PAYMENT

10.01 Working Slab – Item

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

END OF SECTION

VIBRATION MONITORING – Item No.

Special Provision

1.0 SCOPE

This special provision describes requirements for vibration monitoring during pile installation for the replacement of the Avonmore Road underpass.

2.0 DEFINITIONS

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years of experience in the field of installation of piling and vibration monitoring or, alternatively, with expertise demonstrated by providing satisfactory quality verification services for a minimum of two (2) projects of similar scope to the contract. The QVE shall be retained by the Contractor to ensure general conformance with the contract documents and issue certificates of conformance.

3.0 SUBMISSION REQUIREMENTS

The Contractor/QVE shall submit details of the vibration monitoring plan to the Contract Administrator for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Equipment and methods used by the Contractor to perform the work that may cause undue vibration.
- Qualifications of vibration monitoring specialist.
- Details regarding proposed instrumentation.
- Proposed location of instruments on the existing Avonmore Road underpass.
- Proposed frequency of readings.
- Action plan to be taken to adjust deep foundation installation methods if readings show vibrations exceeding tolerable levels.

4.0 MONITORING

The vibration monitoring equipment shall be placed on the existing Avonmore Road underpass. The Contractor shall take readings on the existing structure throughout pile driving operations, and shall immediately notify the Contract Administrator if the vibrations exceed the limits specified herein.

The vibrations measured on the existing bridge structures shall not exceed 100 mm/s (peak particle velocity). If the readings are not within these limits, the Contractor must alter the deep foundation installation procedures until the vibrations at the existing structure are within acceptable levels.

5.0 BASIS OF PAYMENT

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

END OF SECTION

DEEP FOUNDATIONS – Item No.

Special Provision

1.0 SCOPE

The predominant soil deposit at this site is a water-bearing cohesionless till, which contains cobbles and boulders. The Contractor is advised that cohesionless soils are susceptible to disturbance under conditions of unbalanced hydrostatic head, and that appropriate equipment and construction procedures will be required for pre-augering into the till for installation of steel piles, or for caisson construction through the till deposit. The Contractor is also advised that appropriate equipment and construction procedures will be required to penetrate or remove obstructions, such as cobbles and boulders, to permit installation of deep foundation elements and shoring elements.

Where caisson foundations are adopted, these will extend into the limestone bedrock, which is very strong. Appropriate construction procedures and equipment will be required to penetrate, and socket a liner into, the bedrock.

2.0 BASIS OF PAYMENT

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

END OF SECTION

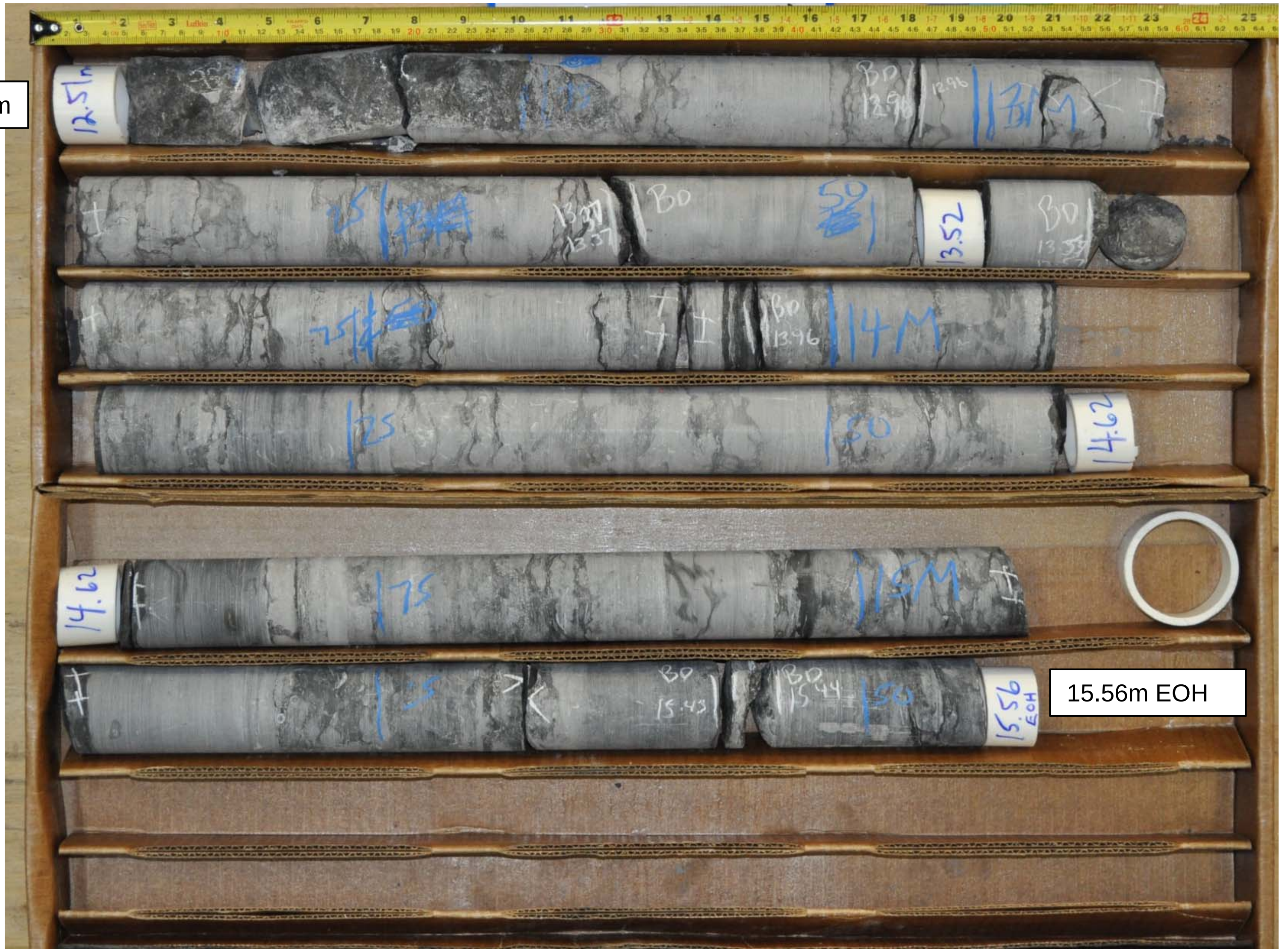
APPENDIX D

Bedrock Core Photographs

Bedrock Core Photographs
Borehole 14-731 (12.5m to 15.6m)

Figure D1

12.51m



15.56m EOH

Date: January 2016
Project No: 12-1121-0099-1730

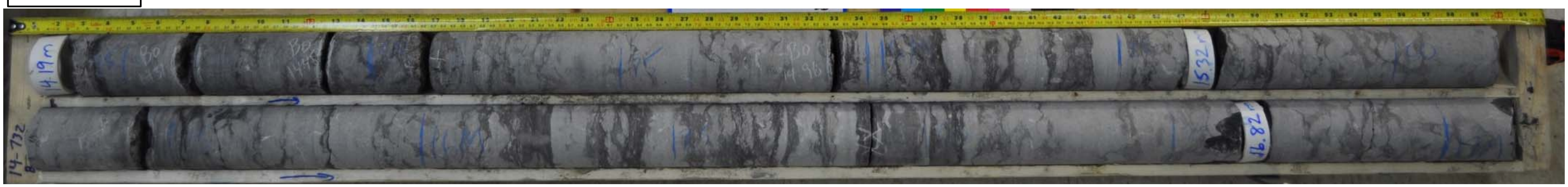
Checked By: MJK

Golder Associates

Bedrock Core Photographs
Borehole 14-732 (14.2m to 17.2m)

Figure D2

14.19m



17.16m EOH

Date: January 2016

Project No: 12-1121-0099-1730

Checked By: MJK

Golder Associates

Bedrock Core Photographs
Borehole 14-733 (6.0m to 9.4m)

Figure D3

Date: January 2016
Project No: 12-1121-0099-1730

Checked By: MJK

Golder Associates

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

Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
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

Golder Associates